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Work-Zone Traffic Control and Tests of Delineation Material

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Contents

Foreword

This Record contains three papers on work-zone safety, three on pavement markings, and one on retroreflective sign sheeting. The central theme is improving safety on our roadway system. This Record is recommended to administrative, operations, and research engineers concerned with safety of the motoring public; construction and maintenance workers; and pedestrians.

Lewis provides the results of an effort to standardize terminology used in the area of traffic control for highway construction and maintenance work zones. The author draws on his many years of experience in teaching work-zone safety to develop this glossary of terms.

Dudek and Ullman present the results of their research effort on reduced signing requirements for short-term maintenance operations involving lane closures on four-lane divided highways. The researchers evaluated several signing treatments on an open roadway and identified two signing schemes that outperformed the standard traffic control plan.

The identification of the characteristics of accidents in New Mexico construction zones for a 3-year period is the subject of the paper by Hall and Lorenz. Understanding the nature of construction-zone accidents is a key ingredient in the development of strategies to improve worker and motorist safety. The researchers provide detailed descriptions of their findings.

Kalchbrenner presents a historical overview of the development and use of glass beads in pavement markings. The author correlates the relationship between the glass beads and the changes in the marking materials over the last 20 years.

The embedment and retroreflectivity of drop-on glass spheres in thermoplastic markings is the subject of the paper by O'Brien. The author describes the results of a laboratory experiment using uncoated and moistureproofed drop-on spheres on various types of thermoplastic traffic markings. The application rate and size distribution for optimum retroreflectivity are identified.

Attaway presents the results of an effort to evaluate the service life of thermoplastic and long-life pavement marking tape in North Carolina. The author illustrates how portable retroreflectometers can be used in assessing the reflectivity of various pavement markings.

Evaluating the effects of various factors on the retention time of raised pavement markers on asphalt concrete pavement is the subject of the paper by Tielking and Noel. The researchers used laboratory experiments and open roadway sections in Texas to evaluate raised pavement markings. They provide information on the effects of adhesive materials on the retention time and describe maintenance techniques used to replace missing markers.

Ketola describes the problems encountered when attempting to correlate the results of artificially accelerated testing with long-term exterior exposure of retroreflective sheeting. Due to the difficulties in replicating the exterior environment, the author suggests the use of a 45-degree exterior exposure technique to achieve better correlation with long-term retroreflective sheeting performance.

Work-Zone Traffic Control Concepts and Terminology

RUSSELL M. LEWIS

An annotated glossary of the concepts, definitions, and standard terminology currently used and advocated in traffic control for highway construction, maintenance, and related activities is presented. For selected terms and categories, information is provided on the background, use, implications, and relevant principles. The objective is to achieve consistency in the use of these terms in preparing manuals, directives, and contract documents and in training, reporting research findings, and otherwise communicating on this subject. Much of the terminology recommended has been used, modified, and refined through decades of intensive work-zone training courses conducted for engineering, technical, and field personnel. This experience has shown that the precise use of words is not merely an academic exercise. The terminology that evolved promotes understanding and results in higher test scores. Good, clear, and consistent language aids understanding and yields a better product in the field. This in turn enhances safety, which is the ultimate objective.

An annotated glossary of concepts, definitions, and standard terminology in traffic control for highway construction, maintenance, and related activities is presented. These are terms currently used or advocated. For selected terms and categories, information is provided on the background, use, implications, and relevant principles.

The objective is to achieve consistency in the use of these terms in preparing manuals, directives, and contract documents and in training, reporting research findings, and otherwise communicating on this subject. Much of the terminology recommended herein has been used, modified, and refined through decades of intensive work-zone training courses conducted for engineering, technical, and field personnel from public highway agencies, contractors, and utility companies. Many of these courses were part of a certification program that included a comprehensive final examination. This experience has shown that the precise use of words is not merely an academic exercise. The terminology that evolved promotes understanding and results in higher test scores. Good, clear, and consistent language aids understanding and yields a better product in the field. This in turn enhances safety, which is the ultimate objective.

Because many technical terms used in highway engineering are taken from common, widely used words, it is important to recognize the specific meaning when used in a technical context. Moreover, some terms in the general highway engineering literature take on a narrower or altered meaning when applied to work-area traffic control. Although most of the terms presented here have been tested and refined over the years, a few new terms have been developed to fill a void or reduce ambiguity.

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For established terms, most of the definitions given are consistent with those of AASHTO (*1,2*) and the *Manual on Uniform Traffic Control Devices* (MUTCD) (*3*). If this is not the case, the differences are explained or new terms are proposed to meet a recognized need. In some instances, the definitions given are deliberately conceptual rather than numeric. For example, in setting forth the various categories for project duration, specific numeric time limits are not given. Establishing such limits is considered to be an agency function, and agencies may well differ in their selections. Where work-zone tapers are defined, the criterion for taper length has been proposed (*4*). Definitions are grouped functionally rather than alphabetically in the following subject areas:

- General work-zone terms,
- Elements of the road system,
- Traffic control and devices,
- Traffic zones and components,
- Work-zone tapers,
- Project duration,
- Work activity,
- Closure types, and
- Mobile operations.

GENERAL WORK-ZONE TERMS

Road Users

> Road users are all those who may be using any portion of the highway right-of-way and its immediate environs. The term includes vehicle operators and passengers, cyclists, pedestrians, bystanders, and workers.

The term "road user" may be used for convenience and brevity, because it avoids the repetitious listing of all of these parties.

Work Zone, Work Area, and Work Site

> Work zone, work area, and work site denote the general location of a work activity or the subject of work-area traffic control.

The terms "work zone," "work area," and "work site" have been used in the titles of many handbooks, papers, and research reports on the subject of traffic control for highway construction and maintenance operations. In these contexts, the terms do not refer to any specific geographical area that is related to the site. Because they have been so widely used as generic terms and are useful for that purpose, they should continue to be used as such.

The term "work area" has also been used to describe a specific portion of the traffic control zone. A replacement term, "activity area," is defined later. The reason for the proposed change is simply that because "work area" is widely used as a general term, any attempt to maintain a narrow and specific meaning is hopeless and incompatible with the need for consistent and precise terminology.

Traffic Control Plan

> A traffic control plan (TCP) is a plan for handling traffic through a specific highway or street work zone or project.

Traffic control plans may range in scope from a very detailed TCP designed solely for a specific project to a simple reference to such items as standard plans, a section of the MUTCD, or a highway agency manual. The degree of detail addressed in the TCP depends on the project complexity, traffic needs, and the extent of traffic interference with the construction activity.

The TCP definition and discussion are derived from the *Federal-Aid Highway Program Manual* (*5*). The TCP for a large project having significant traffic impact should address the entire traffic corridor. Using terms defined above, the traffic control zone involves only the facility where work is being performed, whereas the TCP should address the entire influence zone.

The design of a traffic control zone for a highway work site involves determining the geometry of the work-zone features and the selection and the location of all temporary traffic control devices. The general belief is that design is an engineering process involving discretionary decisions; therefore the designer needs to be educated or trained to fulfill this function. Field forces, however, are also involved in the selection and placement of devices that constitute traffic control zones.

For example, highway maintenance forces and utility companies are continually performing this work at least once and sometimes several times a day. In these instances, however, the intent is that features and application diagrams (designed by engineers) form the basis for their layouts. Training is needed to familiarize workers with the concepts and procedures, but the task is basically a subprofessional one. Thus, it is considered best to reserve the term "design" for the higher-level, decision-making process and use the term "layout" to denote the process in which typical standard designs are installed and adjusted to meet field conditions.

ELEMENTS OF THE ROAD SYSTEM

Several of the following terms are shown in Figure 1.

Roadway

> The roadway is the portion of a highway intended and available for vehicular use. It includes the traveled way and shoulders. A divided highway has two or more directional roadways.

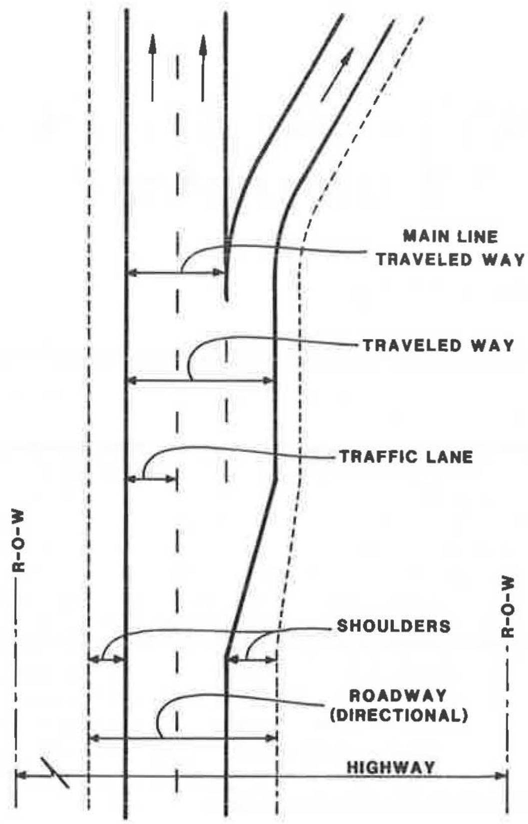

FIGURE 1 Elements of the road system.

The roadway extends from the outside of one shoulder to the outside of the other shoulder. In some instances, where the median is wide, work on the roadway in one direction may be undertaken essentially independently from the other direction.

This definition of the roadway is consistent with that employed by AASHTO (*1,2*) and with the general use of the term by practitioners of work-zone traffic control. However, in Section 1A-9 of the MUTCD, the roadway is defined as "that portion of a highway improved, designed, or ordinarily used for vehicular travel, exclusive of the berm or shoulder." It is not known why this discrepancy in definitions was created. If the more restrictive MUTCD definition prevails, a new term would be desirable to encompass the entire width, as given above.

Shoulder

> A shoulder is the portion of the roadway contiguous with the traveled way for emergency use by stopped vehicles, and for lateral support of the pavement.

A stabilized shoulder serves as a recovery space. The shoulder is normally kept free of all obstacles and roadside appurtenances, such as sign posts. However, if portable signs are used in work areas, they may be placed on shoulders to provide visibility and stability. It is noted that in some parts of the country, particularly in portions of the Midwest, a shoulder is called a "berm." In portions of New England, a shoulder is referred to as a "breakdown lane."

Traveled Way

> The traveled way is the portion of the roadway designated for the ordinary movement of vehicles; it extends from edge line to edge line.

The AASHTO definition, which excludes auxiliary lanes, is contrary to common use for work sites. It is suggested that it would be clearer to refer to this as the "main line" traveled way. Lanes used for travel include not only the main line lanes, but also other designated lanes, including auxiliary lanes (defined by AASHTO to include lanes used for parking, speed change, turning, storage for turning, weaving, truck climbing, etc.). Again, if the more restrictive definition is employed, a term is needed to encompass all travel lanes. To avoid confusion, the term "full traveled way" could be employed.

Traffic Lane

> A traffic lane is that portion of the traveled way for the movement of a single line of vehicles.

TRAFFIC CONTROL AND DEVICES

Traffic Control

> Traffic control is the process of regulating, warning, and guiding road users and advising them to traverse a section of highway or street in the proper manner.

Traffic Control Device

> Traffic control devices are signs, signals, markings, or other devices placed on or adjacent to a street or highway by authority of a public body or official having jurisdiction to regulate, warn, or guide users. At work sites, other traffic control devices are commonly employed, such as channelizing and delineating devices.

Traffic Sign

> A traffic sign is a device mounted on a fixed or portable support to convey an official message by means of words or symbols; it is officially erected for the purpose of regulating, warning, or guiding traffic.

A traffic sign provides information only to road users. If traffic is to be forced to follow certain paths, other devices must also be used.

Supplemental Plate

> A supplemental plate is placed below a sign to provide additional information related to that sign.

Two examples of supplementary plates commonly used in work zones are an advisory speed plate and an advisory distance plate. An advisory speed plate is placed below a warning sign to advise drivers of the appropriate speed of travel. An advisory distance plate is used on symbol signs—such as the WORKER sign and the FLAGGER sign—to warn of the distance between the sign and the situation depicted.

A supplemental plate is not used as a sign by itself—only in conjunction with another sign. When used in this manner, it is placed below the sign it supplements—either directly below the sign, or alternatively on the post adjacent to traffic on a two-post sign. The background color of the supplemental plate should be the same color as that of the principal sign. Therefore, supplemental plates used with construction and maintenance warning signs should have an orange background.

Standard Traffic Sign

> A standard traffic sign, as defined in the MUTCD, is used for a specific purpose and is placed in a prescribed location.

Special Traffic Sign

> A special traffic sign meets a need not covered by a standard sign and has been approved for use by the proper authority.

Nonstandard Traffic Sign

> A nonstandard traffic sign does not conform to the design, application, or placement criteria prescribed by the MUTCD—and does not meet the requirements for a special sign.

The MUTCD permits special warning signs under Section 2C-41, in which it is stated that warning signs other than those specified in the MUTCD may be required under special conditions. The MUTCD further states that such signs should conform with the general specifications for shape, color, and placement of warning signs.

Insofar as possible, legends and symbols used on special signs should be patterned from standard signs to allow rapid recognition and understanding. Many of the best special signs are symbol signs. Many configurations are used in work zones involving numerous lanes shifting in a variety of directions. Effective warning signs showing the required movements have been created using multiple reverse curve symbols or variations in the symbols shown on the DIVIDED ROADWAY AHEAD sign, or both.

Maintenance crews, contractors, and sign shop personnel should be restricted to the fabrication and use of standard signs for normal operations. When a special sign is desired, its design and use should be approved by a senior traffic engineer with the highway agency.

Channelizing Device

> Channelizing devices are used to warn and alert drivers of hazards created by work activities in or near the traveled way and to guide and direct drivers safely past the hazards. Channelizing devices include but are not limited to cones, vertical panels, drums, and barricades.

Channelizing devices are intended to perform their function by being viewed. They should be no more formidable than needed for stability, and when hit should readily yield, collapse, or break away.

Traffic Barrier

> For work-zone traffic control, a barrier is a device designed to prevent vehicular penetration into areas behind the barrier.

Temporary barriers may be placed in or adjacent to the roadway. They may be placed adjacent to traffic lanes to separate the travel area from the work space. They may also be used to separate two-way traffic and vehicular and pedestrian flows. Insofar as practical, traffic barriers should be designed to minimize damage to the vehicles that strike them and the occupants of those vehicles. Barriers perform their primary function physically, whereas channelizing devices perform by being viewed. Barriers may also serve as channelizing devices. When so used, barriers may need supplemental devices or markings to enhance their visibility, especially at night when they are positioned adjacent to a travel lane.

There are two categories of barriers used in work zones—longitudinal barriers and crash cushions. Crash cushions, also called energy attenuators, may be stationary, portable, or vehicle mounted. Mobile longitudinal barriers also have been developed to protect workers, but their use is still essentially in the experimental stage.

The AASHTO barrier guide defines a traffic barrier as "a device used to shield a hazard that is located on the road side or in the median, or a device used to prevent crossover median accidents" (*6*). This definition is not well adapted to temporary barriers used in work zones.

TRAFFIC ZONES AND COMPONENTS

In 1974 when the original work-zone training course was prepared for FHWA, no consistent terminology existed for the description of the various locations in which devices are placed when temporary traffic control is set up for a work activity. To meet this need, three common terms were selected that possessed similar meanings.

The broad term is "zone." The zone is subdivided into several "areas," and a "space" is a portion of an area. The zone and areas begin and end at a line that is essentially perpendicular to the roadway center line. Laterally, they extend across the entire roadway, and in some cases may be thought of as extending to the right-of-way lines. A possible exception is on a divided highway with a wide median where work is taking place only on one side of the directional roadway and the other roadway is completely unaffected by the operation. In this instance the zone and areas may be considered as extending only to the center line of the highway.

An area may contain one or more "spaces." Typically, the space begins and ends at a line transverse to the roadway, but extends over only a portion of the roadway width. Laterally, it may consist of the shoulder or one or more lanes.

These concepts were later incorporated into FHWA's *Traffic Control Devices Handbook* (TCDH) (*7*). The definitions were later compromised when a "buffer space" added to a diagram was improperly drawn and given the attributes of an area.

Traffic Control Zone

> A traffic control zone for temporary traffic control at a work site is the entire section of the roadway over which control related to the work operation is exercised and in which any temporary traffic control devices are placed.

The traffic control zone extends from the first advanced warning sign to the last device, typically a sign indicating the end of the zone (see Figure 2). The traffic control zone includes an advance warning area, a transition area, an activity area, and a termination area. The definitions of these terms are given below. Note that some traffic control zones may not contain all four areas. For example, a zone for shoulder work can be designed with only an advance warning area and an activity area.

The clear demarcation of areas within the zone, as defined below, applies primarily to stationary zones. For mobile operations, discussed later, the areas may overlap. For example, the advance warning and transition areas may in part be provided by high-visibility lights or arrow panels mounted on one or more vehicles located at the rear of the mobile activity area.

Advance Warning Area

> The advance warning area starts at the beginning of the traffic control zone and extends to the transition area—or activity area if no transition area is used. The driver is given information about the hazards ahead and the actions needed to travel safely through the areas beyond. As soon as a channelizing device is encountered or a change in the normal travel path is imposed, the motorist has entered the next area.

All traffic control zones should have an advance warning area. The only temporary traffic control devices placed in this area are the black-on-orange warning signs specified for construction and maintenance operations. The advance warning area may extend for a mile or more on a major highway or a few

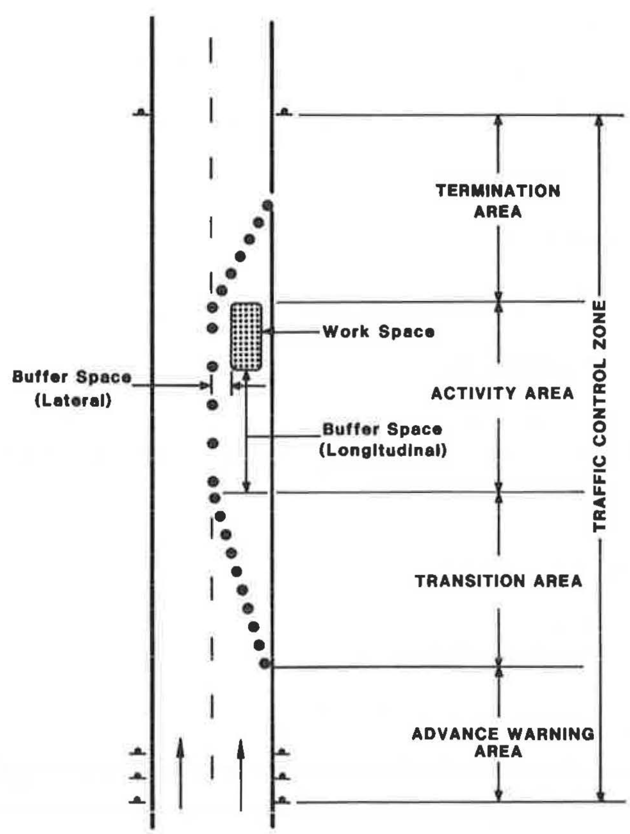

FIGURE 2 Traffic control zone and components.

hundred feet on a local street. In its simplest form, it may contain a single sign.

Where traffic volumes may exceed the capacity available through the work area, the size of the traffic control zone may need adjustment. The advance warning area should be designed so that it encompasses the stretch of upstream roadway in which backups occur or can readily be extended to include that stretch, as needed. This is of primary concern on high-speed roads where slow-moving and start-and-stop operations may surprise drivers. The geometry of the approach roadway may also dictate the extent of the advance warning area.

Transition Area

> A transition area is required where some form of closure occurs. In this area, traffic is channelized from the normal highway lanes to the paths required to move through the activity area. It is the portion of the traffic control zone that commences at the downstream end of the advance warning area and extends to the beginning of the activity area.

The transition area encompasses the tapers that are used to close lanes, shift travel paths, or both.

Activity Area

> The activity area is the portion of the roadway in which any closure is in effect and where the work is taking place. It is the portion of the traffic control zone that commences at the downstream end of the transition area and extends to the beginning of the termination area.

The work area may encompass one or more spaces. These spaces and their use are defined immediately after the following term. The term "activity area" is a proposed replacement for "work area." As discussed previously, a more definitive term is needed to describe this specific portion of the traffic control zone. Alternative terms are "work activity area" and "operations area."

Termination Area

> The termination area is used at work sites to allow traffic to clear the activity area and return to normal traffic operations. It is the final portion of the traffic control zone that begins at the downstream end of the activity area.

The termination area extends from the downstream end of the work area to the END CONSTRUCTION or END ROAD WORK sign. Downstream tapers may be placed in the termination area.

Work Space

> The work space is that portion of the activity area set apart exclusively for workers, equipment, and material storage and is delineated to exclude vehicular and pedestrian traffic.

Buffer Space

> A buffer space is an optional feature in the activity area that provides a recovery space for errant vehicles and separates traffic flow from the work activity or a potential hazard. No work activity should occur and no equipment and materials should be stored within this space.

When used, a buffer space is typically employed within the activity area. To fulfill its recovery function, however, the space must be kept clear of workers, material storage, and operating equipment. Buffer spaces may be positioned either longitudinally or laterally with respect to the direction of traffic flow.

Used longitudinally, a buffer space may be placed in the initial portion of a closed lane that precedes the actual work space, as shown in Figures 2 and 3. Sometimes a shadow vehicle or work vehicle is positioned in a buffer space to provide increased protection for workers occupying the actual work space beyond. When a vehicle is used in this manner, consideration should be given to equipping it with a rear-mounted crash cushion. To be consistent with the preceding definition, the back end of the shadow vehicle marks the end of the buffer space and the beginning of the work space.

Longitudinal buffer spaces may also be used to separate opposing traffic flows that use portions of the same travel lane. An advantageous application is to separate two tapers that are used by opposing directions, as shown in Figure 4. Such a buffer space provides an island for the effective placement of signs directly in the line of sight of affected drivers. This placement is preferable to setting signs in a single line of channelizing devices where they protrude into adjacent traffic lanes. When a formidable device, such as an arrow panel, is placed in an island composed of channelizing devices, only the space in front of the device functions as a buffer space.

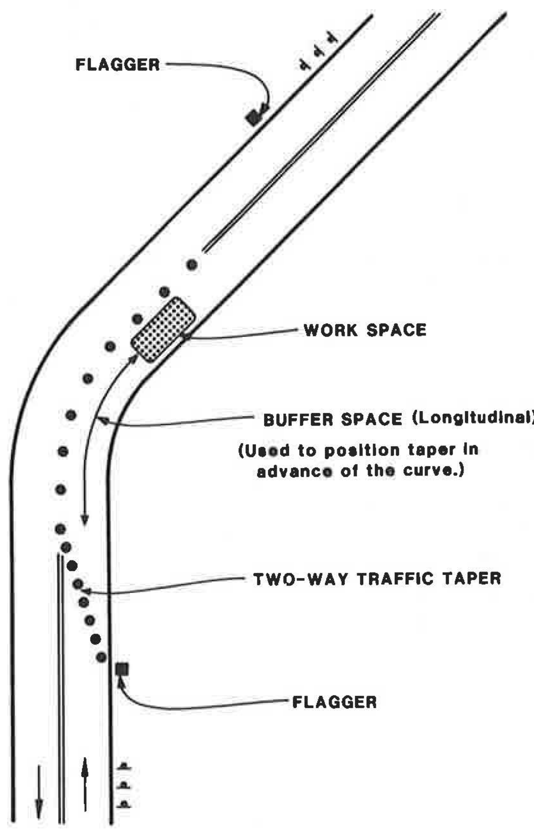

FIGURE 3 Work zone on a two-lane highway (selected features).

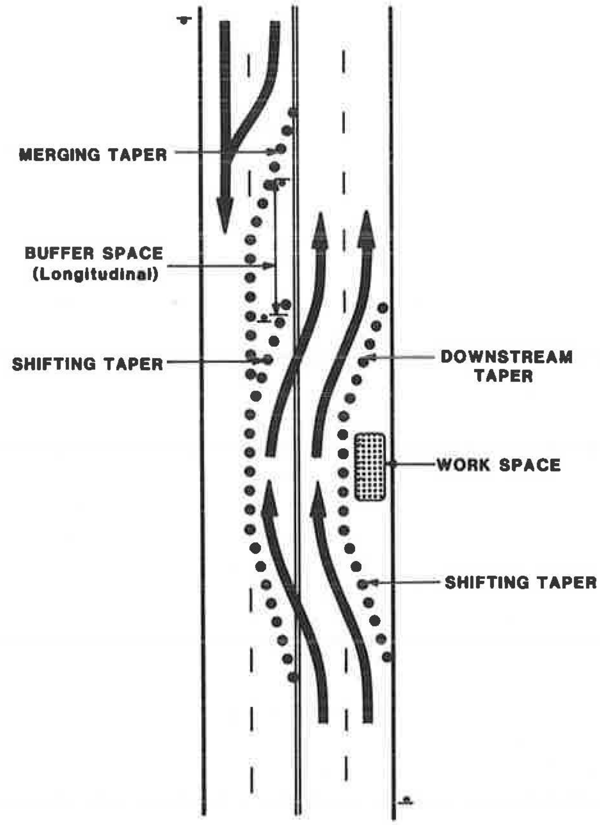

FIGURE 4 Work zone on a multilane street (selected features).

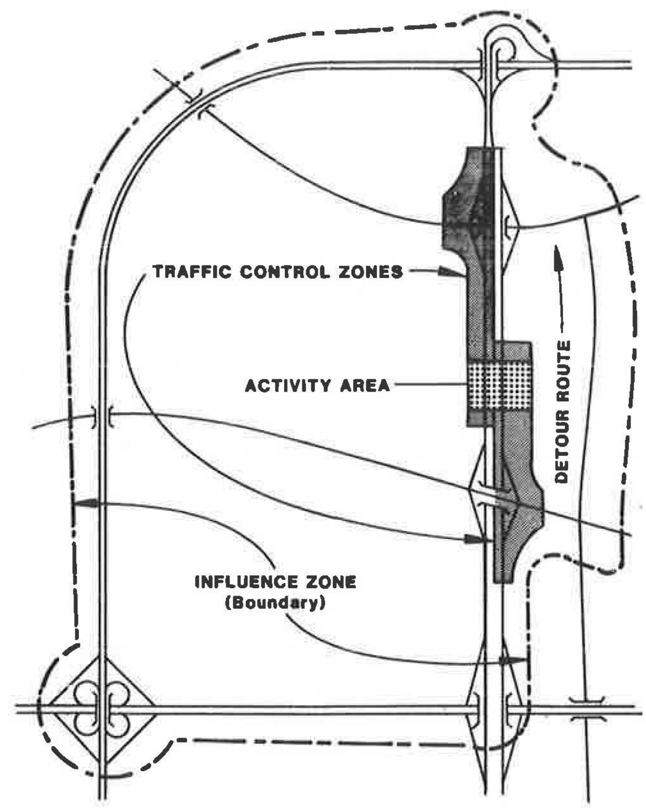

FIGURE 5 Traffic control zone, detour route, and influence zone.

Lateral buffer spaces may be used between two travel lanes, especially where the lanes carry traffic in opposing directions. A lateral buffer space may also be used to separate the travel space from the work space or a potentially hazardous space, such as an excavation or pavement drop-off.

Detour Route

> When a road is closed and a detour is established, the traffic control zone includes the area in which a detour route begins. The detour route extends beyond the zone to divert traffic around the site and return it to the original route.

It is useful to differentiate the detour route from the traffic control zone, because in many instances traffic control and maintenance along the detour route are handled by a different person than the one exercising control at the work site. As shown in Figure 5, the location at which the detour is established is contained within the traffic control zone. The detour route begins at the periphery of the zone and may extend a considerable distance away from the work site. Thus, the only temporary traffic control devices used along the detour route are those that provide navigational assistance.

Influence Zone

> The influence zone for a work operation is the portion of the highway network over which traffic is routed or diverted because of traffic restrictions at the work site and in which traffic control procedures may be used to advise motorists of congestion and alternative routings.

As shown in Figure 5, traffic restrictions in the work site may create problems on connecting roads and parallel routes. In such cases an influence zone may be created in the traffic corridor that requires traffic control to be extended well beyond the traffic control zone for the highway on which the work is being performed. Devices used in the broader influence zone may include warnings of congestion ahead and alternative routing information. Variable-message boards are useful for displaying this information, which is often wordy and may vary with traffic conditions.

"Influence zone" is a new term was created to preclude expanding the definition of a traffic control zone to cover all affected roads. For example, if the issue is the closure of the Chesapeake Bay Bridge Tunnel, the traffic control zone may extend for a few to several miles on either end. The influence zone over which rerouting information will be needed, however, may extend into three to five states.

WORK-ZONE TAPERS

The terms in this section and their definitions have evolved over many years. They have been used in courses given by the American Traffic Safety Services Association (ATSSA) and in their current form have been found to be most useful in communicating the various taper requirements. They were developed to avoid the single word "taper" from being applied to a variety of situations, each with differing requirements. A task force chaired by the author was appointed by the Construction and Maintenance Technical Committee of the National Committee on Uniform Traffic Control Devices

(NCUTCD) to prepare a new section on work-zone tapers for the MUTCD. Following acceptance of the concepts presented below by the NCUTCD, FHWA proposed a change to the MUTCD incorporating this material (*4*).

Upstream Tapers

> Upstream tapers are placed in advance of the space that is being protected. Upstream tapers are directive in that they force traffic in the affected lane to follow a new travel path. The four taper types defined below are all upstream tapers.

Upstream tapers are placed ahead of a portion of the roadway that must be vacated and appear in front of oncoming traffic to direct traffic into a new travel path. Such tapers are placed in the transition area when some form of closure occurs and traffic is channelized from the normal highway lanes to the paths used to move through the work area. Alternative names or attributes for upstream tapers include "directive" and "advance."

There are four upstream tapers—merging taper, shifting taper, shoulder taper, and two-way traffic taper. It is important to differentiate among them because each one has a different length criterion.

Merging Taper

> A merging taper is used to close a lane on a multilane highway and to direct traffic from the lane being closed into the adjacent lane.

In highway work areas, the merging taper (see Figure 4) has the longest required length because drivers must locate a gap in the adjacent traffic stream and move into it. The taper should be long enough so that vehicles approaching side by side have sufficient time to adjust speeds and merge into a single lane before the end of the transition. The minimum desirable length for a merging taper should be computed by the formulas $L = W \times S$ and $L = WS^2/60$ for high and low speeds, respectively (*3*).

Shifting Taper

> A shifting taper is used to move traffic into a different travel path when a merge is not required.

Because a shifting taper does not involve a merge, its length may be shorter than that of a merging taper (see Figure 4). It has a minimum length equal to one-half of that computed using the formulas for the length of a merging taper. Changes in path direction in which no merge is involved also may be accomplished with horizontal curves designed for normal highway speeds. For example, this procedure is often used for the geometric design of median crossovers.

Shoulder Taper

> A shoulder taper is used to close an improved shoulder on a high-speed roadway.

When an improved shoulder having a width of 8 ft or more is closed on a high-speed roadway, it should be treated as a closure of a portion of the roadway, and the work area on the shoulder should be preceded by a shoulder taper. Shoulder tapers should have a minimum length of one-third of that computed using the formulas for the length of a merging taper, provided the shoulder is not used as a travel lane.

Two-Way Traffic Taper

> The two-way traffic taper is used to close one lane of a two-lane, two-way highway. The remaining lane is used alternately by traffic in each direction, typically under the control of flaggers, police officers, or temporary traffic signals.

Traffic may be directed to use the open lane in alternate directions under the control of flaggers, a pilot vehicle, or temporary traffic signals (see Figure 3). In this situation, a short taper having a maximum length of 100 ft should be used in the closed lane to direct traffic into the open lane. A long taper derived by the formulas would be inappropriate, because a long taper encourages drivers to maintain their speed and to change lanes early. Since two-way traffic tapers are used where the open lane is shared by the opposing direction of travel, the potential accident is a head-on collision. A short taper encourages drivers to slow down and then make a deliberate lane change only when instructed to do so.

Downstream Taper

> Downstream tapers are placed beyond (downstream from) a closure to indicate that the traffic may return to the normal traffic lanes. Their use is optional, and the taper is permissive because it does not require a change in travel path.

When used, downstream tapers are generally placed beyond the work space to enable and guide traffic in its return to the full roadway cross section (see Figure 2). As such, the taper is located alongside and behind the traffic that follows the indicated path. Therefore, the length is not critical and a short taper is suitable. It provides a positive message that the work space has been passed as compared with an uncertain assumption based upon the absence of channelizing devices. Alternative names or attributes for downstream tapers include "permissive" and "departure."

Downstream tapers are used to reopen the full approach width to an intersection beyond a mid-block lane closure or to return traffic to a lane serving an off ramp. Downstream tapers formed with channelizing devices may interfere with work operations where vehicles must access the work space at its downstream end. Conflicts of this type can be avoided by downstream tapers that are formed with temporary pavement markings.

PROJECT DURATION

Because the duration of the work undertaken is a major determinant in the design of the traffic control zone, it is useful to designate and define the various durations that are significant.

Long Term

> Long-term activities are those during which the traffic control zone is in place for several days or longer.

From a planning and design viewpoint, there is ample time to install and realize benefits from the full range of traffic control procedures and devices that are available for use. Generally, larger channelizing devices are used for long-term operations, because they have more reflective material and offer better nighttime visibility. Also, the larger devices are less likely to be displaced and tipped over—an important consideration during periods when the work crew is not present to maintain the zone. Also, because long-term operations extend into nighttime, reflective or illuminated devices, or both, are required.

Intermediate Term

Intermediate-term activities require a few to several days to perform; thus nighttime closures are involved.

Because the period is limited, it may not be feasible or practical to use procedures or devices that would be desirable for long-term operations. Examples include altered pavement markings, barriers, and temporary roadways. The increased time to place and remove these devices in some cases could significantly lengthen the project, thus increasing exposure time. In other instances, there might be insufficient payback time to make higher-type traffic control economically attractive.

Short Term (Daytime)

Short-term activities are those that are accomplished during one daylight period.

Most maintenance and utility operations come under this category. The work crew is present to maintain and monitor the traffic control zone. The use of flaggers is a practical and available option. Neither lighting nor reflective devices are required under normal conditions.

Short Duration

Short-duration activities are generally considered to be those in which it takes longer to set up and remove the traffic control zone than it does to perform the work. Typically, the operation can be accomplished in 15 min or less.

Hazards are involved for the crew when installing and removing a traffic control zone. Also, since the work time is short, the time during which motorists are affected is significantly increased as the traffic control zone is expanded. Considering these factors, it is a general belief that simplified control procedures may be warranted for short-duration activities. Such shortcomings may be offset by the use of other predominant devices, such as special lighting units on work vehicles.

There may be some confusion over the difference in meaning between short-term activities and short-duration activities. These terms are suggested because they are often used in this context. However, since they are often misused, perhaps more definitive terms are needed. Thus "daytime" was appended to "short term." There may be a need for a new, more-exclusive term in place of "short duration."

WORK ACTIVITY

Work zones are frequently classified by the type of work performed there. Actually, this means little to drivers, who are only concerned with the impact of the work operation on their use of the facility. The variation in project duration associated with the various types of work is the more significant factor, and it is discussed below.

Construction Operations

Construction projects commonly require a minimum of several weeks and may involve multiple construction seasons or years. A basic condition of construction operations is that traffic control procedures must accommodate both daytime and nighttime conditions. There is ample opportunity to plan the work and develop an effective public information program. A long duration makes it more attractive to invest in high-type traffic controls and facilities, such as barriers and temporary roadways. Typically, highway construction work is performed by construction companies under contractual agreements.

Maintenance Operations

Maintenance operations generally are accomplished more rapidly, rarely exceeding a few days. Most maintenance work is performed during one work day as a daytime operation. Some maintenance activities, however, involve extensive rehabilitation and take on the basic characteristics of a construction project. Maintenance activities generally are performed by the highway agency's own forces, although contract maintenance is becoming more common.

Utility Operations

Utility activities usually are short daytime operations, except under emergency conditions. Often they are performed in lower-volume and lower-speed streets. Operations often involve intersections, since that is where many of the network junctions occur. As the crew size is usually small with only a few vehicles involved, the number and type of traffic control devices placed in the traffic control zone may be minimal. As discussed under short-duration projects, however, in this situation the reduced number of devices should be offset by the use of higher-visibility devices, such as special lighting units on work vehicles.

Other Activities

Emergencies and disasters may pose severe and unpredictable problems. The ability to install proper traffic control may be greatly reduced in an emergency, and any devices on hand may be used for the initial response. If the situation is prolonged, the procedures shown in Part VI of the MUTCD should be followed when closures are involved or work must be done within the roadway. Special events, on the other hand, can be properly planned and coordinated. Part VI of the MUTCD provides guidance as to the proper procedure for closing portions or entire roadways in conjunction with such activities.

CLOSURE TYPES

Closure

> A closure is the taking of any portion of the roadway for the exclusive use of a work activity.

Closures may involve a shoulder; one or more lanes; any combination of lanes or shoulders, or both; a direction roadway; or the entire highway. The portion of the roadway remaining, if any, after the closed portion is temporarily removed from service is available for use by traffic passing through the work area.

Lane Closure

> A lane closure involves the closing of a traffic lane in such a manner that traffic is forced to move out of the closed lane and into another lane and the total number of lanes is reduced.

On a multilane roadway, a merging operation is involved. On a two-lane, two-way roadway, alternating directions of traffic must use the remaining lane—typically under the control of flaggers. The implication of this definition is that the closure of an auxiliary lane beginning at its point of inception does not constitute a lane closure, because there is no reduction in the number of lanes available. Examples include the following:

- Closing of a turn bay,
- Closing a deceleration lane approaching an off ramp, and
- Closing a lane where all lanes are maintained by some form of traffic shifting or splitting, or both.

Double-Lane Closure

> A double-lane closure is the closing of two adjacent lanes in the same direction on a multilane roadway.

In accord with the MUTCD an advance warning advising of the double-lane closure is used, such as RIGHT TWO LANES CLOSED AHEAD. The individual lanes are closed one at a time, however, separated by a transition distance. The individual lane closures each have advance signing and a merging taper.

Traffic Shifting

> Traffic shifting is the lateral displacement of one or more travel lanes from their normal travel path in order to accommodate a work space in the roadway. All lanes are carried through and no merging operations are involved.

Traffic shifting may be accomplished by several means, including lane narrowing, use of shoulders, and use of the opposing roadway. Different closure types may be used for each direction of flow. For example, Figure 4 illustrates a lane closure for southbound traffic and traffic shifting for northbound traffic.

Traffic Splitting

> Traffic splitting is the situation encountered on a multilane roadway where open travel lanes are carried around both sides of a work space.

An island work space is formed, with traffic on both sides. It is preferable to avoid this situation, where feasible. This can usually be accomplished by a double-lane closure where an exterior lane is also closed, provided that the traffic volume can be accommodated. When traffic splitting is employed, there are various procedures that may be used to improve the operation. For example, in some instances a lane reduction may be accomplished upstream by closing an exterior lane and creating a dummy work space. Then the remaining open lanes can be carried around the island work space with no further merging required and generally with no lane changing permitted.

Lane Narrowing

> Lane narrowing is a reduction in lane width for those lanes carried through the activity area in order to maintain the maximum number of open lanes while accommodating the needs of the work activity.

Median Crossover

> In the context of work-zone closures, a median crossover occurs where one directional roadway is closed to traffic and that direction of travel is carried diagonally across the median onto the other directional roadway.

Often the single word "crossover" is employed, but the addition of the word "median" makes the term more exact. A crossover may also be used to move traffic between a main line roadway and a parallel frontage road. Unless the entire directional traffic stream is so diverted, the feature providing for this optional maneuver would more properly be called a temporary slip ramp.

TLTWO

> Two-lane, two-way operation (TLTWO) occurs where one directional roadway is closed on a four-lane divided highway and both directions of travel use the remaining roadway with one lane in each direction.

The TLTWO section is implemented with the use of a median crossover in advance of the closed roadway. Note that median crossovers on six or eight-lane highways do not create TLTWO sections in accord with this definition. Considerable attention has been directed towards the TLTWO section, because the open roadway retains the appearance of a divided highway, but is temporarily operated as a two-lane, two-way highway. This is especially the case for the non-crossed-over direction, which continues to operate on the normal side of the median. The problem is that a driver may forget and pull out to pass into opposing traffic. Therefore, special requirements have been established for TLTWO on high-speed highways where the length of the TLTWO section is more than a short runaround (*4*).

Detour

> A detour is initiated when traffic is directed to leave the normal roadway.

Thus, traffic shifting or splitting, or both, accomplished within the confines of the roadway (including shoulders) does not involve detours. This definition is believed to be consistent with interpretation of the term by drivers. A crossover involves a detour, because traffic is directed to leave the directional roadway.

On-Site Detour

> An on-site detour occurs where traffic is diverted onto a temporary roadway generally constructed within or adjacent to the right-of-way or onto a frontage road.

Off-Site Detour

> An off-site detour occurs where traffic is diverted onto another highway in order to bypass the work site.

One-Lane Road

> A one-lane road is the special situation in which one lane is used alternately by both directions of travel.

Special techniques need to be employed to prevent head-on collisions. Flaggers are generally employed to control traffic at both ends of the one-lane section. Temporary traffic signals may also be used.

Single Lane

> Single lane is the applicable term when all traffic flowing in one direction must use one lane.

Thus, the terminology "single lane" may be used on a multilane roadway to inform drivers that only one lane is available through the activity area. For example, the New York State MUTCD uses signs with this wording as part of its standard sign series.

MOBILE OPERATIONS

> Mobile operations are those in which the location of the work activity is continually changing, thus making it difficult or impractical to use stationary traffic control devices.

This category includes both continuously moving operations, such as paint striping, and intermittent-stop operations, such as pothole patching and litter pickup. Note that this concept overlaps short-duration operations, which may also involve intermittent-stop activities.

A work vehicle is involved as an integral part of all of these operations. The work vehicle, when appropriately colored, marked, and fitted with special warning lights, serves as part of the traffic control. Mobile operations are typically performed under favorable conditions with good visibility and outside peak-traffic periods. When this is the case, the argument may be made that the number of devices used may be reduced. Even under adverse conditions, which may be associated with unscheduled work (e.g., repairs to a malfunctioning traffic signal), the case can be made that a large and highly visible vehicle can effectively replace numerous small devices (e.g., signs and cones).

Fast-Moving Operations

> Fast-moving activities are those in which the speed of the operation is in the range of 3 mph to 10 to 15 mph below the posted speed limit—the higher differential being used with higher speeds.

Within this range special warning procedures are required that move with the operation. Typical fast-moving operations are lane striping or roadway sweeping. If volumes are light and sight distances are good, a well-marked vehicle may suffice. If volumes, speeds, or both are higher, a backup vehicle equipped with signs, beacons, or an arrow panel should follow the operation.

A 10- to 15-mph differential speed has long been recognized as the threshold at which slow-moving traffic begins to interfere significantly with normal traffic. For example, the criterion for climbing lanes as contained in the AASHTO policy on geometric design is a 10-mph differential speed (*2*). Likewise, several states utilize signs stating USE 4-WAY FLASHERS BELOW 40 MPH on freeways posted at 55 mph. A work vehicle traveling within 10 to 15 mph of the posted speed is essentially moving "with traffic," and no special traffic control is needed other than an appropriately marked work vehicle. Examples of such activities include photologging and road roughness measurement.

Slow-Moving Operations

> Slow-moving activities are those in which the operations generally proceed in a continuously moving fashion, and the speed of travel is less than 3 mph.

In this speed range there may be some opportunity for stationary devices combined with other special warning procedures that move along with the operation. Stationary devices may need to be relocated periodically to remain within an appropriate distance of the work activity.

Slow-moving and related intermittent-stop operations may involve such work as spraying herbicides, painting pavement markings using walk-behind equipment, and pavement marking removal. When such work is performed on highways, a backup sign-carrying vehicle to warn traffic and protect workmen is used. In slow-moving traffic on minor roads, a single vehicle equipped with signs and beacons may suffice.

Traffic Control for Short-Duration Maintenance Operations on Four-Lane Divided Highways

CONRAD L. DUDEK AND GERALD L. ULLMAN

The authors summarize the results of research conducted to develop and evaluate reduced traffic control sign treatments for short-duration maintenance operations involving lane closures on four-lane divided highways with traffic volumes of 30,000 or fewer vehicles per day. Several candidate sign treatments were developed and compared with the standard traffic control configuration for a lane closure on a four-lane divided roadway. The effect of the sign treatments on the proportion of drivers that moved out of the closed lane at several locations immediately upstream of the lane closure was studied. Study results indicate that the Texas LANE BLOCKED sign or a changeable-message sign placed 1,500 ft before the cone taper influenced drivers to exit the closed lane farther upstream from the work zone than the other candidate sign treatments tested or the standard traffic control treatment.

All streets and highways require periodic maintenance and repair work that necessitates the closure of one or more travel lanes. Proper traffic control is essential during these maintenance activities to ensure the safety of motorists and workers while permitting the maintenance work to be completed in a timely manner. The *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*) provides traffic control guidelines for some of the more typical highway and street maintenance operations. For example, the specified traffic control configuration for a lane closure on a multilane roadway consists of channelizing devices and a series of advance warning signs placed upstream of the work area. Considerable time and effort is usually expended installing and removing traffic control devices at a particular location. For maintenance operations that last several hours, the time and effort involved is easily justified.

However, many roadway maintenance activities require only a very short period of time to complete. Often, only 15 to 20 min is spent at a particular highway location performing the actual maintenance work. A work crew may make several of these short-duration stops along a section of roadway for such activities as pothole patching, crack sealing, or pavement bump planing or profiling. Conversely, work activities such as bridge clearance measurements or guardrail maintenance require a work crew to spend 15 to 20 min at single locations on different roadways.

For these short-duration maintenance operations, the actual placement of the advance warning signs and the channelizing devices that are required by the MUTCD often takes longer than the actual work itself. This traffic control effort limits the productivity of the work crew performing these quick repairs. Perhaps more important, worker exposure to traffic during traffic control installation and removal is greater than the exposure during the actual maintenance work itself. Consequently, there is a need for a reduced traffic control configuration for short-duration maintenance operations that requires less installation and removal time than the standard traffic control configuration specified in the MUTCD, but that is still effective in directing drivers safely and efficiently through the work zone. Such a reduced configuration would lessen worker exposure time during traffic control device placement and removal, lessen motorist exposure time to the lane closure itself, and allow maintenance crews to be more productive. In response to this need, the Texas State Department of Highways and Public Transportation (SDHPT) sponsored a study by the Texas Transportation Institute (TTI) to develop and evaluate candidate traffic control configurations for short-duration maintenance activities. In order to make the research effort reasonable in scope, the study concentrated only on four-lane divided highways with traffic volumes of 30,000 or fewer vehicles per day (vpd). This paper presents the methodology and findings of that research.

The specific objectives of the study were as follows:

- To develop candidate traffic control configurations for short-duration maintenance operations on four-lane divided highways with traffic volumes of 30,000 or fewer vpd, and
- To conduct field studies at actual maintenance operations on four-lane divided highways to evaluate the effectiveness of the candidate configurations in directing drivers out of the closed travel lane.

DEVELOPMENT OF CANDIDATE TRAFFIC CONTROL CONFIGURATIONS

The configuration of traffic control devices normally used during a lane closure on a four-lane divided highway in Texas is shown in Figure 1. The arrangement consists of three static advance warning signs, a cone taper, and an arrow panel placed behind the cone taper in the closed lane. This configuration is specified for a minor maintenance operation (less than one daylight period) on a four-lane divided roadway in the *Texas Manual of Uniform Traffic Control Devices* (*2*) (to be referred to as the standard TMUTCD treatment).

Following standard TMUTCD treatment, as the work zone is approached, the first advance warning sign seen by drivers is a ROAD WORK AHEAD (CW21-4D) sign. The sign is

Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843.

Intermittent-Stop Operations

Intermittent-stop operations are highly mobile activities in which a stop is required to perform the actual work.

Although some stationary traffic controls may be feasible for intermittent-stop operations, this is not the typical case. Usually, they are highly mobile activities, such as pothole patching, litter pickup, and luminaire relamping. The time spent at any one location is usually just a few minutes.

Other Activities

There are various types of activities in which portions of the traffic control zone may change, but the advance warning area and the transition area remain fixed for the entire operation, or at least for significant time periods (e.g., an hour or more). Therefore, these are not classified as mobile operations. Operations falling in this category typically have a varying length for the activity area, which is downstream of the stationary advance warning and transition areas. The third procedure described below is a variation on this theme.

Moving Within a Zone

For some operations, the activity area can best be lengthened or shortened as work progresses during the day. Either of these procedures reduces the impact upon road users as compared with closing the whole section for the entire work period.

Diminishing Zone

A diminishing work zone is one in which the entire traffic control zone is installed initially and is then reduced in size as the work progresses. Work is performed in the direction opposite to the traffic flow, which enables the work space to decrease in length without moving the advance warning signs and taper.

Expanding Zone

For an expanding work zone, the initial installation consists of the advance warning signs, taper, and the delineation of just enough of the activity area for work to begin. The work space is then increased in length as work progresses in the direction of traffic flow. The maximum zone length is determined by work accomplished. If cure time is required, the entire zone must remain in place while the last patch cures.

Leapfrog Method

The third method is similar to the expanding work zone except that the advance warning area and transition area are periodically moved downstream as the work progresses. The second transition may be set up in the protection of the closed lane, and then the upstream taper is removed. This technique further reduces the impact on road users. It requires a duplicate set of warning signs and devices used in the taper; however, the work space may be kept short, thus reducing the number of devices needed in the activity area.

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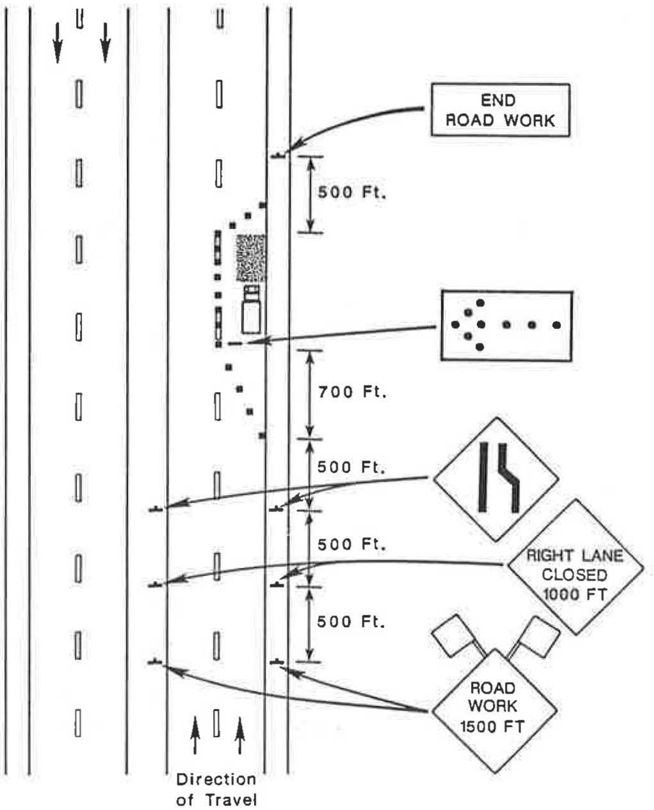

FIGURE 1 Standard TMUTCD traffic control configuration for a lane closure on a four-lane divided highway.

located 1,500 ft from the cone taper and informs drivers that some type of road work exists ahead. The next warning sign—placed 1,000 ft from the cone taper—is the RIGHT/LEFT LANE CLOSED AHEAD (CW20-5D) sign, which indicates to drivers that a travel lane is closed and implies that a lane change maneuver may be required. The final advance warning sign—located 500 ft from the cone taper—is the symbolic LANE CLOSED (CW4-2) sign. The symbol pictured on the sign also indicates to drivers that a lane is closed and that a lane change maneuver may be needed.

An analysis of driver information requirements at work zones was performed—based on "positive guidance" concepts (*3, 4*)—in order to develop the reduced traffic control configuration for short-duration maintenance operations. A complete discussion of this analysis may be found in the study documentation (*5*). Ideally, the reduced configuration would use as few devices as possible while still providing adequate worker safety and driver information (*5*).

On the basis of the analysis of driver information requirements, a reduced traffic control configuration was developed that consisted of an arrow panel positioned behind the cone taper, coupled with an advance warning sign placed 1,500 ft upstream of the work zone. Research has shown the arrow panel to be the primary source of information to drivers approaching moving maintenance operations, and may be the primary source of information when used for stationary work-zone operations (*6–9*). It was hypothesized that the arrow panel would serve as the primary source of information to drivers for the reduced traffic control configuration, whereas an advance warning sign placed 1,500 ft upstream would supplement the arrow panel. The upstream sign would inform drivers that they are approaching a work zone hazard and possibly reinforce the lane change information provided by the arrow panel. Figure 2 shows a schematic layout of the candidate traffic control configuration for short-duration maintenance operations.

Once the basic reduced traffic control configuration was selected, the next step was to determine which advance warning sign should be used 1,500 ft upstream of the work zone. A number of possibilities were considered in the early stages of the research. Four sign treatments were eventually selected for complete field evaluation. Each treatment differed with respect to the amount of information provided to the driver, where the information was provided, and the conspicuity of the devices. A brief description of each candidate configuration is provided below.

Treatment 1—ROAD WORK AHEAD

For this configuration, dual ROAD WORK AHEAD signs (identical to the first sign encountered in the standard TMUTCD traffic control configuration) were placed 1,500 ft upstream from the beginning of the cone taper. This sign does not provide information about the closure, but indicates that some type of road work exists ahead. This candidate configuration relies on the flashing arrow panel in the cone taper to inform drivers that the lane is closed and that they must be in the open lane when they reach the work zone.

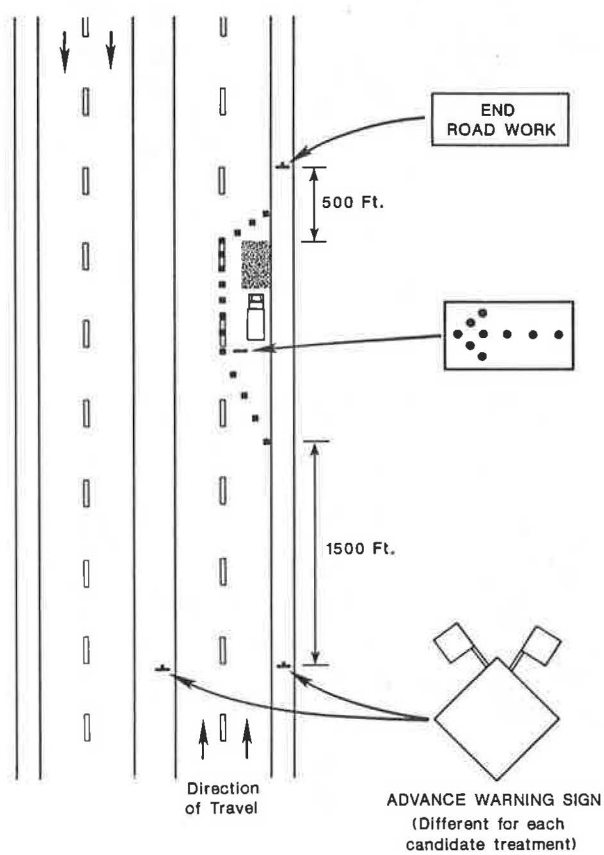

FIGURE 2 Candidate traffic control configuration for short-duration maintenance operations.

Treatment 2—Symbolic LANE CLOSED Sign

For this treatment, symbolic LANE CLOSED signs (identical to the third signs encountered in the standard TMUTCD traffic control configuration) were placed on each side of the travel lanes 1,500 ft in advance of the cone taper. This sign reinforces the message of the arrow panel to exit the closed lane. Referring to Figure 1, the lane closure information supplied by the sign is provided farther upstream than in the standard TMUTCD traffic control configuration.

Treatment 3—Changeable-Message Sign

Treatment 3 utilized a portable changeable-message sign (CMS) placed 1,500 ft upstream from the cone taper. These signs have a high target value and can typically be seen farther upstream than standard static advance warning signs. In addition, a sense of urgency is implied because of the flashing of the message.

In effect, the information provided by the first two advance warning signs in the standard traffic control configuration is consolidated into this one sign. Figure 3 shows the CMS used in this study.

FIGURE 3 Changeable-message signs used in study.

Treatment 4—Texas LANE BLOCKED Sign

The Texas LANE BLOCKED sign was originally designed for moving maintenance operations (Figure 4). It furnishes clear, unambiguous information to the driver about which lane is blocked and which lane is open to traffic. The sign is larger than a normal static MUTCD advance warning sign (7 by 7.5 ft), providing a legibility distance that allows drivers to adequately perceive and process the information presented. The colors used for the sign are black letters on an orange background, consistent with the color coding signifying construction or maintenance work activity. As with all of the other sign treatments evaluated in this study, the sign was placed 1,500 ft upstream from the cone taper.

STUDY APPROACH

A series of field studies was conducted to evaluate the four candidate traffic control treatments and determine how effectively each treatment encouraged drivers to exit the closed lane in advance of the work zone. The standard TMUTCD traffic control configuration served as the basic treatment. If a candidate treatment was to be judged effective, it had to perform at least as well as the standard treatment in persuading drivers to exit the closed travel lane.

FIGURE 4 The Texas LANE BLOCKED sign.

Experimental Design

Several factors were identified that could affect the data that would be collected to evaluate the standard and candidate traffic control treatments. It was hypothesized that the type of lane closure (left lane, right lane), site-specific factors (sight distance, geometrics, type of drivers), and time-of-day variations could possibly affect the study results. An experimental design was developed for the standard TMUTCD and four candidate treatments accounting for (a) type of closure, (b) study site conditions, and (c) time of day. A chi-square design was developed for both left-lane and right-lane closures. Each treatment would be evaluated at each site, but the sequence in which they were evaluated would change from site to site. Table 1 presents the experimental design for this study.

Data Collection Methodology

Data were collected for 1 hr at each site for the TMUTCD and each of four candidate treatments for a total of 5 hr per site. Data were collected only during the daylight, off-peak periods. Two types of data were collected during the field studies. Traffic volumes were recorded at five locations upstream of the work zone. These volumes were recorded in 5-min intervals by lane and vehicle type, and subsequently combined into hourly volumes coinciding with the time at which the evaluation of each treatment began and ended. The first location (Station 1) was positioned approximately 2,500 to 3,000 ft upstream from the cone taper. This station was assumed to be upstream from the effects of the sign configurations, and therefore was used as a control location for each site. The remaining locations (Stations 2, 3, 4, and 5) were positioned 1,500 ft, 1,000 ft, and 500 ft from the beginning of the cone taper as well as at the beginning of the cone taper itself. Figure 5 shows where lane distribution data were collected at one of the study sites.

In addition to traffic volumes, videotape recordings were made of traffic approaching the work zone. Data were collected at, and just upstream of, the cone taper from a bucket truck or other vantage point.

Measures of Effectiveness

The primary measure of effectiveness (MOE) used to evaluate the sign treatments was the percentage of traffic in both travel lanes (measured at the various stations upstream from the

TABLE 1 EXPERIMENTAL DESIGN FOR FIELD STUDIES

	Study Site No.					
Sequence	1	2	3	4	5	6
Right Lane Closures:						
1	TMUTCD	LB	RWA	SYM	CMS	LB
2	RWA	CMS	TMUTCD	CMS	LB	SYM
3	SYM	SYM	CMS	LB	TMUTCD	RWA
4	CMS	RWA	LB	TMUTCD	SYM	TMUTCD
5	LB	TMUTCD	SYM	RWA	RWA	CMS
Left Lane Closures:						
1	TMUTCD	LB	CMS	SYM	RWA	LB
2	RWA	CMS	LB	TMUTCD	SYM	TMUTCD
3	SYM	SYM	TMUTCD	CMS	LB	RWA
4	CMS	RWA	RWA	LB	TMUTCD	SYM
5	LB	TMUTCD	SYM	RWA	CMS	LB

Note:

TMUTCD = Standard TMUTCD Treatment

SYM = Symbolic Lane Closed Sign

LB = Texas "LANE BLOCKED" Sign

RWA = "ROAD WORK AHEAD" Sign

CMS = Changeable Message Sign

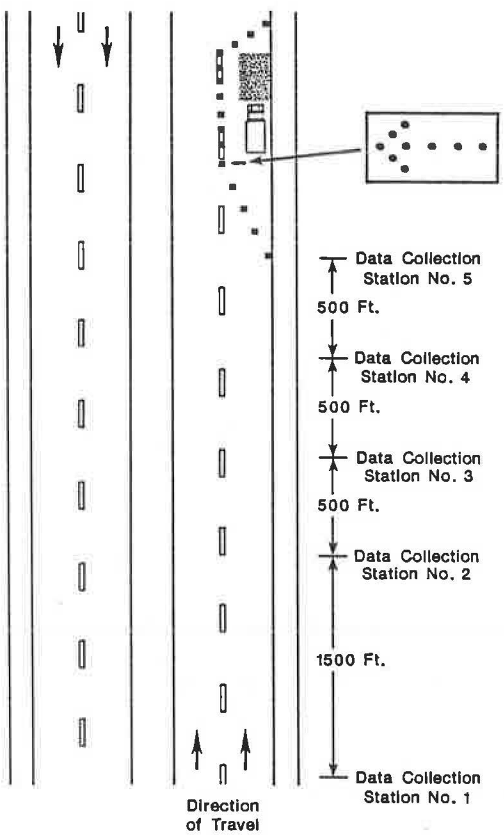

FIGURE 5 Example of data collection locations at the study sites.

work zone) that was in the closed lane at each data collection location. Comparisons were made of these percentages for the candidate sign treatments and the standard TMUTCD treatment. For a candidate treatment to be considered effective, the proportion of traffic in the closed lane at Stations 2, 3, and 4 had to be as low as that observed for the standard TMUTCD treatment.

One other MOE used in this study was erratic maneuvers recorded at or approaching the work zone. Erratic maneuvers were classified according to their severity and type. For this evaluation, only the more severe types of conflicts or maneuvers were considered, such as impacts with the cone taper or other traffic control devices and severe vehicle braking or skidding to avoid hitting a traffic control device or other vehicle.

Study Site Selection and Description

A series of field studies was conducted at rural and suburban freeway work zones near Ft. Worth and Dallas, Texas. The study sites were selected with help from SDHPT personnel. The criteria for selecting study sites were as follows:

- Relatively low traffic volumes (i.e., traffic volumes that were of 30,000 or fewer vpd), so that queues would not form upstream of the work zone when a travel lane was closed;
- Adequate sight distance of at least 1,500 ft to the arrow panel; and
- Actual maintenance work activity being performed and a reason to have a travel lane closed. In addition, the actual work activity had to be located a considerable distance downstream of the lane closure and cone taper.

STUDY RESULTS

Left Lane Closure

Data collected at each site where a left (inside) lane closure was studied were pooled for statistical analysis. Table 2 presents the average percentage of vehicles in the closed left lane at the data collection stations upstream of the work zone. At the data collection location 3,000 ft from the cone taper (Station 1), the percentage of traffic in the closed lane was found to be very similar for all treatments (TMUTCD and candidate) and ranged from 30.7 to 32.8 percent. A chi-square test of the equality of the proportions (a contingency test for independence between the proportion of traffic in the closed lane and that for the treatments) indicated that the percentages were the same for each treatment. This result was expected since it was assumed that traffic had not yet been affected by the traffic control treatment present at the work zone. However, the percentages in the closed lane were found to differ significantly between treatments at each of the data collection locations 1,500, 1,000, and 500 ft from the cone taper (Stations 2, 3, and 4). Generally speaking, the LANE BLOCKED and CMS treatments yielded the lowest proportion of traffic in the closed lane at each of these data collection locations. Furthermore, the LANE BLOCKED and CMS treatments were not found to be statistically different from each other, indicating that their performance was nearly identical in terms of influencing drivers to exit the left travel lane.

TABLE 2 PERCENTAGE OF VEHICLES IN CLOSED LANE: LEFT LANE CLOSURE (ALL SITES)

	Distance From Beginning of Cone Taper (ft)			
Treatment	3000	1500	1000	500
TMUTCD	33.2	28.1	20.1	11.0
ROAD WORK AHEAD	32.8	29.3	22.7	16.7
Symbolic Lane Closed	32.2	24.9	19.6	11.9
CMS	32.4	20.7	13.5	6.7
LANE BLOCKED	30.7	22.3	15.0	7.2

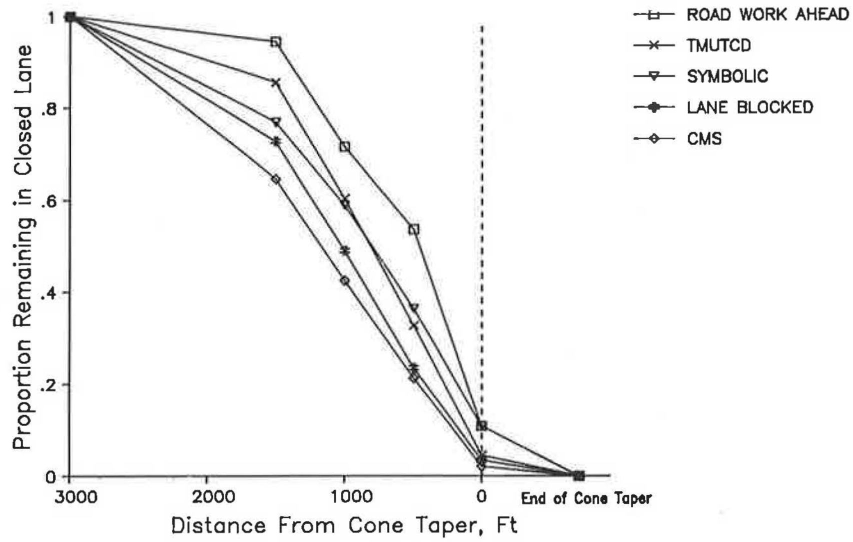

FIGURE 6 Average effect of TMUTCD and candidate signing treatments (left-lane closures; all sites combined).

A slightly different perspective of how the standard TMUTCD and the candidate treatments affected traffic was obtained by normalizing the volumes measured at each of the data collection locations in the left (closed) lane and dividing them by the volume recorded in the left lane at the first data collection location. The resulting proportions illustrate how the treatments affected traffic in the left lane as it approached the work zone. These normalized proportions are shown in Figure 6. As can be seen, the proportion of traffic remaining in the closed left lane was highest for the ROAD WORK AHEAD treatment and lowest for the CMS and LANE BLOCKED treatments. As found in the chi-square test, the proportions were essentially the same for both the CMS and LANE BLOCKED treatments. These proportions show that (a) the CMS and LANE BLOCKED treatments were most effective in influencing motorists to exit the closed lane farther upstream, (b) the symbolic LANE CLOSED treatment performed as well as the TMUTCD treatment (but not as well as the CMS and LANE BLOCKED treatments), and (c) the ROAD WORK AHEAD treatment was the least effective of the signs evaluated.

Right Lane Closure

Table 3 presents the average percentage of the total approach volume that was in the closed lane under each treatment for those sites at which the right lane was closed to traffic. Again, the results of a chi-square test for the data 3,000 ft from the cone taper (Station 1) showed no significant differences among the various treatments. The percentages at this station for each of the treatments were between 65.6 and 67.2. The results at the other data collection locations, however, show significant differences among treatments, with the percentage of traffic in the closed right lane the highest for the ROAD WORK AHEAD treatment and lowest for the CMS and LANE BLOCKED treatments. The results of the symbolic LANE CLOSED and TMUTCD treatments were similar and fell

TABLE 3 PERCENTAGE OF VEHICLES IN CLOSED LANE: RIGHT LANE CLOSURE (ALL SITES)

	Distance From Beginning of Cone Taper (ft)			
Treatment	3000	1500	1000	500
TMUTCD	66.6	52.0	50.4	25.7
ROAD WORK AHEAD	66.4	59.4	49.7	38.0
Symbolic Lane Closed	67.2	45.8	36.8	26.3
CMS	65.6	44.3	27.8	17.5
LANE BLOCKED	66.3	42.3	29.4	18.9

between the ROAD WORK AHEAD results and those of the CMS and LANE BLOCKED treatments.

The data for the right-lane closures were normalized in the same manner as that for the left-lane closed data to show the proportion of traffic remaining in the closed right lane at each of the data collection locations. These data are shown in Figure 7. The trends are similar to those for the left-lane closures; the CMS and LANE BLOCKED treatments were most effective in influencing drivers to exit the closed right lane farther upstream, the symbolic LANE CLOSED and TMUTCD treatments performed similarly but less effectively than the CMS or LANE BLOCKED treatments, and the ROAD WORK AHEAD treatment was the least effective in encouraging drivers to exit the closed lane.

It should be noted that the values in the tables and figures represent averages over the study sites examined. In actuality, considerable variation in performance for each treatment was observed from site to site, which was due to the diverse nature of work zones themselves and other site-specific factors that influence driver behavior at a location. For example, Figure 8 shows the variation in performance for the standard TMUTCD treatment observed over the study sites where the right lane was closed. Similar site variation was also evident for the candidate treatments. However, the relative performance of the TMUTCD and candidate treatments was, for the most part, consistent on an individual site basis. That is, the CMS and LANE BLOCKED treatments were generally the most effective, followed by the symbolic LANE CLOSED and TMUTCD treatments, with the ROAD WORK AHEAD treatment generally the least effective.

Analysis of Erratic Maneuvers

The second type of data collected during the studies of the candidate treatments was erratic maneuvers occurring at or just ahead of the cone taper and work zone. These data were obtained by videotaping traffic movements at the cone taper from a bucket truck or other vantage point. The data were collected to determine whether the candidate treatments resulted in a greater number of erratic or unsafe maneuvers at the lane closure.

Erratic maneuvers and conflicts at the study sites were found to be extremely rare events under any of the treatments stud-

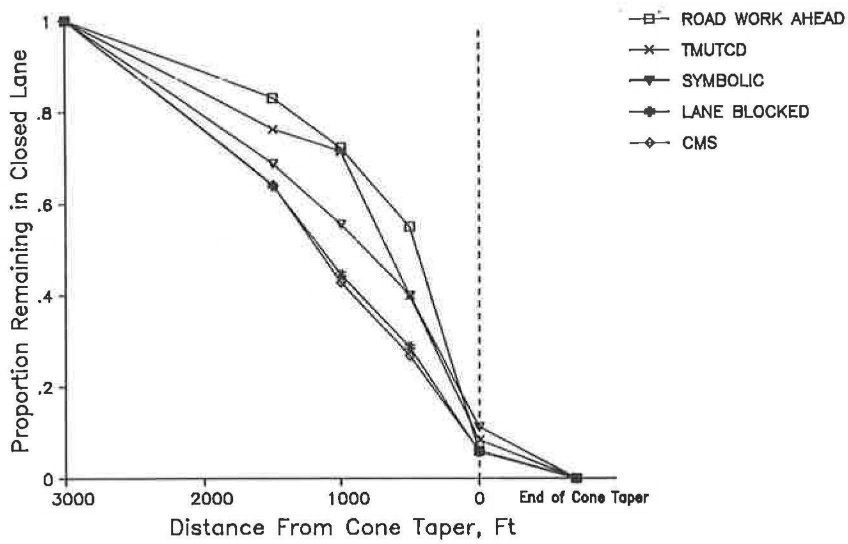

FIGURE 7 Average effect of TMUTCD and candidate sign treatments (right-lane closures; all sites combined).

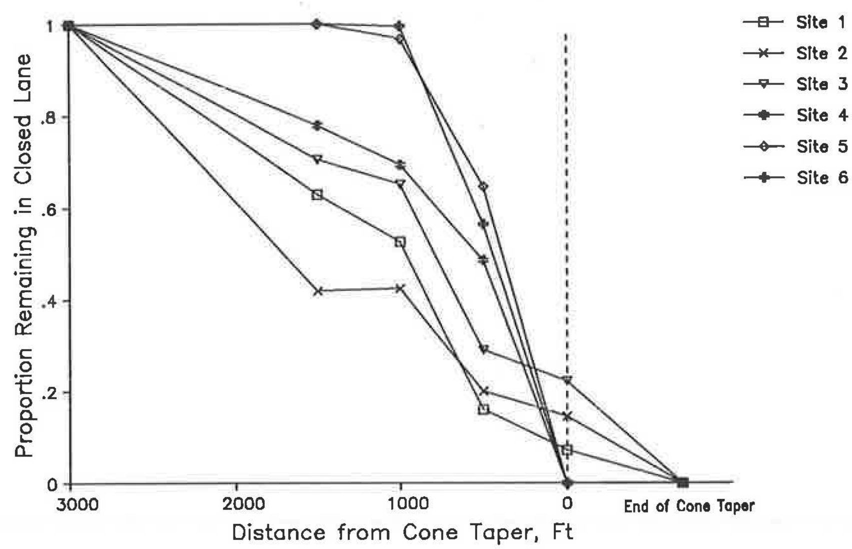

FIGURE 8 Variation in effect of TMUTCD sign treatment (right-lane closure sites).

ied. There were only five incidents overall in which a vehicle ran into the cone taper. However, four of these occurred when the ROAD WORK AHEAD treatment was in place (the fifth incident occurred during the study of the standard TMUTCD treatment). None of these incidents resulted in any type of damage or injury to drivers, workers, vehicles, or traffic control devices. Because of the limited number of these incidents, it was not necessary (or possible) to perform any type of statistical analysis. Nevertheless, it did not appear that any of the treatments (except the ROAD WORK AHEAD) was more hazardous to drivers than any other, including the TMUTCD treatment.

SUMMARY AND CONCLUSIONS

Twelve field studies were conducted at work-zone locations on rural or suburban four-lane divided highways to evaluate the effectiveness of four candidate traffic control treatments proposed for short-duration maintenance operations in which a lane closure is required. The results of the field studies show that the Texas LANE BLOCKED and CMS treatments were the most effective (of those examined) in influencing a greater proportion of drivers to exit the closed travel lane farther upstream from the work zone. In addition, the symbolic LANE CLOSED treatment was nearly as effective as the standard TMUTCD configuration in this regard but neither of these was as effective as the CMS or LANE BLOCKED treatments. Finally, the ROAD WORK AHEAD treatment was found to be the least effective of the signs studied.

The erratic-maneuver data collected at the study sites showed few serious erratic maneuvers during evaluation of any of the treatments (including the standard TMUTCD configuration). Erratic maneuvers may have been more prevalent during the evaluation of the ROAD WORK AHEAD treatment. However, it was not possible to determine this conclusively.

On the basis of the results of these studies, the Texas LANE BLOCKED or CMS treatments were recommended to the SDHPT for short-duration maintenance operations (that require a lane closure) on four-lane divided highways with traffic volumes of 30,000 or fewer vpd and with at least 1,500 ft of sight distance to the work-zone lane closure. Both of these treatments include the use of an arrow panel and a cone taper immediately ahead of the work zone. The LANE BLOCKED sign costs considerably less than a CMS and can be easily constructed in the maintenance shop, making it an attractive alternative.

Care should be taken not to extend the results of these studies to sites with characteristics not similar to those evaluated. Specifically, it is not known whether these reduced traffic control treatments would be effective at work-zone locations with limited sight distance, on divided highways with more than two lanes per direction, or on four-lane divided highways with traffic volumes exceeding 30,000 vpd. Additional research would be necessary to evaluate the treatments under each of these types of conditions before their effectiveness could be assured.

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Characteristics of Construction-Zone Accidents

J. W. HALL AND V. M. LORENZ

This study was undertaken to improve the safety of highway construction zones in New Mexico. The authors examined construction-zone accidents in New Mexico for a 3-year period. In contrast to the traditional technique, which relies solely on the accident record system, they utilized the locations and durations of construction activity on rural state highways. The record system was then used to identify accidents at these sites during the period of construction as well as during the identical period in the previous year. In comparison with the prior year, accident experience increased by 26 percent during construction. Contingency tables were used to compare the driver, roadway, and environmental characteristics of accidents before and during construction. At the 5 percent level of significance, the only parameter that differed between the two periods was the road surface condition, which was dry more often for accidents during construction. However, there was a moderate overrepresentation of accidents involving multiple vehicles, rear-end collisions, large trucks, and the contributing factors of following too close and improper lane changing. The authors also identified deficiencies in the accident record system that result in a substantial understatement of crash experience in construction zones.

According to FHWA statistics, the total street and highway mileage nationwide increased by only 5,000 mi between 1980–1985. This increase represents less than 0.03 percent per year. During the previous 24 years, highway mileage increased by 0.47 percent per year (*1*). Rather clearly, activity has shifted from the construction of new facilities to the reconstruction of existing facilities. This has created an increased opportunity for conflict between road users and construction workers and equipment.

National statistics also indicate that accidents in construction areas are increasing. Highway fatalities in work zones jumped from 489 in 1982 to 680 in 1985 (*2*)—an alarming increase of 39 percent. According to FHWA, no other highway-related category experienced such a dramatic increase in fatalities during this period. Over half of the construction-zone fatalities occurred at night. Interstate highways, which are generally noted for their safety, accounted for 34 percent of the fatalities. Contractors are justifiably concerned, because an estimated 15 percent of the fatalities involve pedestrians, many of whom are construction workers. Studies of accidents before and during construction on selected projects have documented increases in nonfatal crashes of up to 100 percent on roadways that are under construction (*3, 4*). Most studies agree that site-specific factors, including traffic, geometry, and the environment, together with the accident rate before construction, are of major importance in estimating accident rates during construction. Rear-end impacts are the predominant form of collision in work zones, accounting for 35 to 40 percent of the accidents. FHWA also reports that "alcohol involvement is about the same for work zone fatalities as for all fatalities" (*5*).

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STUDY OBJECTIVES

This study was undertaken to improve the safety of highway construction zones in New Mexico. The specific objectives of this study were

- To establish the true extent of New Mexico's construction-zone accidents,
- To identify the characteristics of these accidents that differ from other accidents on comparable roadways, and
- To develop appropriate forms of remedial action.

The basic plan to achieve these objectives was to compare accidents on specific roadway sections during their construction with the accidents on the same sections in the previous year. The initial step in this process involved the use of New Mexico State Highway Department (NMSHD) records to determine the location and duration of major construction projects. Subsequently, the accident record system was used to identify those accidents occurring during the period of roadway construction and the accidents during the identical period of the previous year when the roadway section was not under construction.

It was recognized from the beginning that there would be some limitations with this study procedure. For example, many crashes, especially those occurring at night, are not reported by drivers or enforcement agencies. In addition, the accident report forms and associated computerized record systems do not provide the type of information necessary to properly analyze those crashes that are reported. And finally, a comparison of accidents occurring before construction with those occurring during construction cannot incorporate changes in traffic for two reasons: (a) New Mexico's traffic volume counting procedures specifically preclude the collection of volume data in construction zones, and (b) traffic counts are updated only once every 6 years.

NEW MEXICO CONSTRUCTION-ZONE ACCIDENTS

In New Mexico, construction-zone accidents are indicated on the Uniform Accident Report form under the Road Design category. The six other choices under this category are One Way, Ramp, Freeway, Undeveloped, Alley, and Other. The Detail file of the computerized record system includes the road design features, as indicated by the investigating officer, using variables identified as RDES3 and RDES4. Because "Construction Zone" is the seventh item on the list of road design features, one of these two variables should be coded with the number 7 for a construction-zone accident. The traditional technique for identifying New Mexico's construction-zone accidents is to search for records in the Detail file coded with a 7 for either RDES3 or RDES4.

Application of this technique to New Mexico's 1977 accident data determined that 0.4 percent of the accidents occurred in construction zones. In the same process applied to 1981–1983 accident data it was found that these areas accounted for 1.0 percent of the state's accidents. It is not clear whether this apparent change is due to increases in construction activity, degradation of safety in these areas, improvements in accident reporting, or other factors. Several interesting characteristics of computerized analysis of the 1,503 construction-zone accidents in 1981–1983 are summarized below:

Characteristic	*Percent*
Occurred in Albuquerque	64
Occurred at night	27
Rear-end collision	36
Principal contributing factor	
Driver inattention	16
Failure to yield	13
None	11

Similar trends were observed for the period 1982–1985. Of the 2,022 construction-zone accidents reported during this 4-year period, 66 percent occurred in Albuquerque; of these, 84 percent were on local streets and the remainder were on the Interstate system through the city. The occurrence of nearly two-thirds of the state's construction-zone accidents in Albuquerque is suspicious. It is inconsistent with the city's share of all accidents in New Mexico (about 34 percent) and with Bernalillo County's share (*6*) of statewide vehicle miles of travel (26 percent). NMSHD records show that during 1982–1985, only 12 percent of the funds for highway construction were spent in Bernalillo County. Without discounting competing explanations, it is possible that officers in Albuquerque are more likely to record the construction parameter on the report form. Conversely, officers in other jurisdictions may be lax in their completion of accident reports.

PILOT STUDY

In response to a suspicion that construction-zone accidents were not being properly documented, the New Mexico Traffic Safety Bureau supported a pilot study of these accidents in 1985. In this study, nine major construction zones in NMSHD District 5 that were active for a portion of 1981–1984 were examined in greater detail. Accidents occurring between the let and completion dates of these projects were identified from the record system. Of the 222 accidents in this data set, 13 (6 percent) indicated the presence of construction activity. More encouraging results were obtained by focusing on the period of May–October, when peak construction activity is expected. Of the 104 accidents on these construction sections during this 6-month period, 11 were reported as being in a construction zone. In other words, for 89 percent of the accidents on sections of road that were under construction the presence of construction activity was not indicated. Although the pilot study did not yield definitive results, it strongly suggests that there is less-than-complete reporting of construction-zone accidents.

The research approach used in this study was similar to that employed in the pilot study. However, the limitations of the pilot study (including the small sample)—uncertainty concerning exact work dates and locations and inaccuracies in the accident record system—had to be overcome.

STUDY SITE SELECTION

The set of work zones of interest to this study consisted of major projects on rural sections of New Mexico's Interstate and Federal-Aid Primary (FAP) and Secondary (FAS) highway systems. The research was limited to those construction projects that began sometime between January 1983 and December 1985. The NMSHD provided a list of 355 projects let during the period October 1982 through December 1985. To provide a more manageable set of projects, this list was screened to eliminate minor projects using the following criteria:

- Projects costing less than $100,000,
- Stockpiling,
- Traffic signal installation or improvement,
- Guardrail projects,
- Projects occurring at "various locations,"
- Projects not on the Interstate or the FAP or FAS systems, and
- Projects not started by December 1985.

The rationale for these criteria are quite reasonable. Projects that primarily involve work off the roadway, such as stockpiling, should not influence highway operation. Projects that were not active during the study period or were not on rural state highways were outside the scope of this study. In addition, projects of relatively short duration as reflected by project type or cost were expected to provide little opportunity for accidents.

Application of these criteria reduced the number of candidate projects to 177. The next step in the analysis used the project list, the NMSHD roadway inventory, and other highway record systems to determine some basic information about the projects. Specifically, lists were prepared showing the following information for each project:

- Identification, route, and county,
- Estimation of beginning and ending milelog,
- Total cost and cost/mile, and
- Estimation of average daily traffic volume (in vehicles per day).

To achieve the objectives of the study, it was essential to have additional information concerning these construction projects. A primary need was a better description of the project location; often, the location given on the NMSHD project list was approximate or referenced to indeterminate points. A second critical parameter was the actual period of construction, including intermediate stop and restart dates. This information was obtained from the NMSHD district offices.

It was not possible to obtain information on all 177 projects because several were involved in litigation and the records were not readily available. Furthermore, the records for some projects indicated that the actual work took place on the roadside or at "various locations." For the reasons discussed previously, these projects were dropped from further study.

The resulting data base consisted of 168 major construction projects on rural state highways. The total contract value for these projects was nearly $400 million. The projects included 172 sections of highway with a total length of nearly 1,045 mi, including 348 mi on the Interstate. The average project length was 6.3 mi. The average project duration (from start to finish, including weekends and other nonwork periods) was 255 days. Based on the project length and duration and the average daily traffic estimates on these sections, the total travel at the construction sites was approximately 1,100 million vehicle-mi (mvm).

PROJECT DURATION

In practice, construction does not proceed continuously between the start and completion dates. Many projects involve the use of aggregate, which requires pit preparation and rock crusher installation. Depending on the other components of the project, there may be little work near the highway during this preliminary period. Similarly, toward the end of the project, the resurfacing and other work on the highway is typically completed several weeks (or sometimes months) before seeding and other tasks on the roadside are undertaken. In most cases, weekends, holidays, and days with adverse weather conditions (rain, snow, and wet or frozen ground) are not counted as work days. The effect of these adjustments varies with the climatic conditions in the area.

The effect of job suspension on traffic operations in the area is a function of the nature of the work and the duration of the stoppage. In general, the available roadway is opened to traffic during weekends and other short suspensions. However, advance construction warning signs and other traffic control devices that are relevant are left in place. Barricades, dropoffs, and hazards in the work zone may continue to exist during these periods. Procedures require that construction equipment and materials be stored a reasonable distance from the traveled way. In contrast, during the 2- to 3-month winter suspensions, the traffic control devices (excluding those at each end of the project) may be removed or covered and the roadway is normally returned to its standard condition. It is possible, of course, that an accident during the period of winter suspension could be related to construction. For the purposes of this analysis, it was assumed that the area remains a work zone only during the time when official work days are routinely counted, including intervening weekends and other short disruptions (such as holidays and occasional rain days).

ACCIDENTS DURING CONSTRUCTION AND COMPARISON PERIODS

The traditional method of using the New Mexico record system to identify construction-zone accidents shows that these areas account for approximately 1 percent of the state's accident total. Since the data base described above includes all the major projects on rural state highways, an analysis was undertaken to check the validity of the traditional approach. The 1983–1985 Detail accident files were searched to identify accidents on rural state highways coded with a 7 for RDES3 or RDES4 (i.e., accidents formally identified by the record system as occurring in construction zones). This search identified 261 accidents; surprisingly, only 18 occurred at the sites and times of the 168 projects in the data set. There are several possible explanations for this situation:

- The accidents were on projects where construction started before 1983,
- The accidents were at the sites of some minor projects that were not included in this study,
- The actual work at the accident site was maintenance rather than construction,
- The locational information regarding the accident was incorrect, or
- The RDES3/RDES4 parameters were coded incorrectly.

It is possible that some form of construction or maintenance was actually taking place on some of these sections. For example, during a 2-month period, six construction-zone accidents were reported on an 8-mi segment of FAP 32. During the remaining 34 months of the study period, no construction accidents were reported on this segment. These facts lead one to believe that some form of construction was taking place on this section, even though the NMSHD list of projects does not show any activity. Even with adjustments for these types of segments and six projects that started before January 1983, there remained 59 construction accidents that could not be explained by NMSHD construction records.

Hard-copy reports on these accidents were obtained from the NMSHD and sketches and narratives were reviewed to determine the actual accident characteristics. Nineteen occurred in minor construction zones or at locations where the narrative indicated the presence of maintenance activity (e.g., flagmen, barricades). With one exception, the remaining 40 accidents did not have the "Construction" box checked on the accident report form, and neither the narrative nor sketch indicated construction activity. Subsequent analysis indicated that all 40 errors occurred between May and August 1984. A reanalysis found that 46 percent of the initial 261 accidents occurred during this 4-month period. For example, all six of the FAP 32 accidents mentioned above occurred in May or June 1984. Although some of the 120 accidents in the summer of 1984 actually occurred in construction zones, there was clearly a massive accident data coding problem during this period that distorts the true extent of these accidents. Although the traditional technique of analyzing accidents coded as "Construction Zone" would therefore be grossly in error, the actual data base and analysis techniques employed in this study avoid this pitfall.

PRELIMINARY ANALYSIS OF ACCIDENTS

The next phase of the study sought to combine the construction project data with accident record information. This is a complicated task because the two data bases rely on different reference systems—accident data are coded by route and milelog, whereas the construction projects are referenced to intersections, political boundaries, or other physical points along the highway. With considerable effort, including a review of the project managers' diaries, it was possible to establish more precise milelogs for the construction project limits. These data were incorporated into a computer program that searched the files and selected accidents meeting the following criteria:

- Accidents occurring between the beginning and ending milelogs, and
- Accidents occurring between the start and completion of construction but not during winter or other suspensions.

For projects with durations of less than 1 year, the program also identified the accidents on these sections during the identical period of the prior year. These latter accidents can be used to develop a comparison of accidents before and during construction. The sites were subdivided into three groups based on project duration and accident experience. The results are summarized in Table 1.

Group A projects took less than 1 year and one or more accidents were experienced during the construction period or during the corresponding period of the previous year, or both. Group B projects also lasted less than 1 year, but no accidents were reported during either the construction period or the prior year. Group C projects had durations of more than one calendar year, but a direct comparison with the prior year was not possible. Accident rates for these groups were calculated using individual project durations, section lengths, and 1984 traffic volumes. For Group A, the accident rates were 0.87 and 1.10 accidents/mvm in before and during-construction periods, respectively. For Group C, the accident rate was 1.08 accidents/mvm.

The sites in Group B had no accidents during the construction period, and thus appear to be quite safe; however, there were no accidents during the identical period of the previous year. As shown in Table 2, the low accident experience for Group B sites may be due to their shorter lengths and lower traffic volumes.

From the perspective of this analysis, the Group C sites pose a problem because it is not possible to compare the accidents during the construction period with those in the prior year. The Group B sites, with no accidents either before or during construction, present a different problem. The data imply that these locations were equally safe in the two periods. In actuality, the short construction periods and lower volumes contributed to the absence of accidents. This absence of accidents contributes little to the understanding of specific treatments that deter accidents. If an objective of this study was to compare the accident rate on roadway sections that are under construction with the rate on the same sections when they are not under construction, then all three groups must be considered. However, the primary purpose of this study was to identify factors that can maintain or improve the safety of a roadway section that is under construction. The 114 sites in Group A provide the best opportunity for this type of analysis.

TABLE 1 ACCIDENTS BEFORE AND DURING CONSTRUCTION

Group	Projects	One Year Before	During Construction
A	114	499	631
B	27	0	0
C	26	–	393

TABLE 2 AVERAGE CHARACTERISTICS OF THE GROUPS

Group	Length (mi)	Duration[a] (days)	Volume (vpd)
A	7.1	177	4,200
B	2.8	177	1,400
C	6.3	450	5,100

[a]Actual durations exceed the values shown because some projects were still under construction on Dec. 31, 1985.

EXAMINATION OF GROUP A ACCIDENTS

The 114 sites in Group A provided the best opportunity to detect a change in accidents due to roadway construction. Compared with the same periods in the prior year, the reported accident experience during construction at these sites increased by 26 percent. The greatest increase occurred on Interstate highways (33 percent), although smaller changes were found on the FAS (25 percent) and FAP (17 percent) systems. Accident experience did not increase at all the sites; in fact, 40 percent exhibited a decrease and 11 percent remained unchanged between the two time periods. However, the 56 sites at which accidents increased accounted for 71 percent of the Group A accidents. At these locations, accidents virtually doubled from 228 before the construction period to 451 during the construction period.

A common technique for determining whether the characteristics of two sets of data (in this case, accidents before and during construction) are independent involves the use of contingency tables. In the statistical sense, events are considered to be independent if the occurrence of one is not affected by the occurrence or nonoccurrence of the other. In the context of this study, for example, independence would mean that the ratio of daytime to nighttime accidents was essentially the same, regardless of whether or not the roadway was under construction. This technique (*7*) compares the observed distribution of a pair of variables with the distribution that would be expected if the variables were independent. When there is a large difference between the observed and expected frequencies, it can be concluded that the variables are not independent. The chi-square statistic (χ^2), based on the possible values of the categorical data and the desired level of significance, is used to determine whether the difference is sufficiently large.

One parameter of interest is the distribution of accidents by roadway alignment. Table 3 shows the observed frequen-

TABLE 3 GROUP A ROADWAY ALIGNMENT VERSUS TIME PERIOD

	Before	During	Total
Curve	51 (49.5)	61 (62.5)	112
Straight	448 (449.5)	570 (568.5)	1,018
Total	499	631	1,130

NOTE: Expected frequency of accidents is shown in parentheses.

cies (O_{ij}) of Group A accidents versus roadway alignment. The expected frequencies (E_{ij}), given by the product of the row and column totals divided by the table total, are shown in parentheses. For example, the expected number of accidents on curves during construction is

$$E_{\text{curve, during}} = (631) * (112)/1130 = 62.5$$

As shown in Table 3, the differences between the observed and expected values are rather small. To determine whether the differences are significant, the chi-square statistic is calculated as follows:

$$\chi^2_{\text{calc}} = \text{Sum } [(O_{ij} - E_{ij})^2/E_{ij}] = 0.096$$

The calculated value of the χ^2-statistic is less than the tabulated value of this statistic (3.84) at the 5 percent level of significance. In practical terms, the difference between observed and expected frequencies is not large, and it is concluded that roadway alignment is independent of the time period (before and during). Alternatively, one can state that the proportion (not the number) of accidents on curves does not change while roadways are under construction.

The technique was applied to other accident parameters. The characteristics evaluated were primarily those of interest to the engineer—that is, factors that might be modified with engineering countermeasures. In addition to roadway alignment, they included severity, contributing factors, weather, light condition, and manner of collision. With the exception of contributing factors, these parameters are objective, and not dependent on the investigating officer's judgment. Contingency tables for these characteristics are shown in Tables 4 through 15.

Only two parameters differ between the before and during-construction periods at the 5 percent level of significance. It was found that construction-zone accidents were overrepresented during clear weather and underrepresented during adverse weather conditions. Similarly, the road condition was dry more often than expected and snowy or icy less often than expected during construction. Although construction is obviously more likely to be carried out during good weather, the crash comparisons were made between identical periods of the year, and therefore the findings are unexpected.

TABLE 4 GROUP A LIGHT CONDITION VERSUS TIME PERIOD

	Before	During	Total
Daytime	316	399	715
Dark	183	232	415
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 0.001$, no significant difference.

TABLE 5 GROUP A ROADWAY GRADE VERSUS TIME PERIOD

	Before	During	Total
Level	339	433	772
On grade	160	198	358
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 0.060$, no significant difference.

TABLE 6 GROUP A DAY OF WEEK VERSUS TIME PERIOD

	Before	During	Total
Weekday	340	442	782
Weekend	159	189	348
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 0.478$, no significant difference.

TABLE 7 GROUP A NUMBER OF VEHICLES VERSUS TIME PERIOD

	Before	During	Total
Single vehicle	314	369	683
Multiple vehicles	185	262	447
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 2.305$, no significant difference.

TABLE 8 GROUP A HEAVY-TRUCK INVOLVEMENT VERSUS TIME PERIOD

	Before	During	Total
No trucks	403	499	902
Trucks	91	131	222
Total	494	630	1,124[a]

NOTE: $\chi^2_{\text{calc}} = 0.983$, no significant difference.
[a]Data are missing for six accidents.

TABLE 9 GROUP A PEDESTRIAN INVOLVEMENT VERSUS TIME PERIOD

	Before	During	Total
No pedestrians	206	485	691
Pedestrians	4	14	18
Total	210	499	709[a]

NOTE: $\chi^2_{\text{calc}} = 0.485$, no significant difference.
[a]Data are missing for 421 accidents.

TABLE 10 GROUP A ACCIDENT SEVERITY VERSUS TIME PERIOD

	Before	During	Total
Fatal	15	20	35
Injury	189	226	415
PDO	295	385	680
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 0.512$, no significant difference. PDO = property damage only.

TABLE 11 GROUP A TIME OF DAY VERSUS TIME PERIOD

	Before	During	Total
12:00 midnight to 6:00 a.m.	106	150	256
6:00 a.m. to 12:00 noon	137	174	311
12:00 noon to 6:00 p.m.	150	186	336
6:00 p.m. to 12:00 midnight	106	121	227
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 1.413$, no significant difference.

TABLE 12 GROUP A WEATHER CONDITION VERSUS TIME PERIOD

	Before	During	Total
Clear	402	544	946
Raining	31	36	67
Snowing	50	27	77
Other	16	24	40
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 14.94$, significant difference.

TABLE 13 GROUP A ROAD SURFACE CONDITION VERSUS TIME PERIOD

	Before	During	Total
Dry	389	531	920
Wet	35	46	81
Snow or ice	72	44	116
Other	3	10	13
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 18.78$, significant difference.

TABLE 14 MANNER OF COLLISION VERSUS TIME PERIOD

	Before	During	Total
Ran off road	137	158	295
Fixed object	83	114	197
Rear-end	47	87	134
Sideswipe	41	43	84
Overturn	20	21	41
Other	171	208	379
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 6.669$, no significant difference.

TABLE 15 GROUP A PRINCIPAL CONTRIBUTING FACTOR VERSUS TIME PERIOD

	Before	During	Total
Inattention	111	134	245
Speed	92	112	204
Alcohol	63	82	145
None	66	67	133
Following closely	20	44	64
Lane change	8	21	29
Other	139	171	310
Total	499	631	1,130

NOTE: $\chi^2_{\text{calc}} = 9.458$, no significant difference.

The absence of significant differences for the other parameters is surprising. For example, the accident distributions by time of day and by light condition are virtually the same in the before and during-construction periods. Despite the frequent use of detours in construction areas, the relative proportion of accidents on curves remained unchanged. Likewise, accident severity did not change significantly; in fact, the proportion of accidents resulting in a fatality or an injury dropped from 0.41 in the prior year to 0.39 during construction. Fatalities occurred in 3.0 percent of the accidents before construction and 3.2 percent of the accidents during construction. Because of the small numbers involved, this difference is not significant. The relative proportion of in-state and out-of-state motorists was the same in both periods. Pedestrian accidents—9 before construction and 14 during—were few in number, and the moderate increase is certainly less than anticipated considering the presence of workers in the construction zones.

Several parameters differ between the before and during-construction periods at the 15 to 25 percent level of significance. Although traditional analyses might properly discount conclusions at these levels of significance, the aforementioned problems with the accident data suggest that these parameters deserve more attention. Two parameters that fall into this category are the manner of collision and the principal contributing factor.

A contingency table analysis using the data in Table 14 shows that the manner of collision is independent of construction activity. A more detailed examination finds that the relative proportion of ran-off-road, sideswipe, overturn, and other accidents decreased by 1 to 2 percent in the during-construction period, whereas the proportion of fixed-object and rear-end collisions increased. In fact, the proportion of rear-end collisions increased from 9.4 percent before construction to 13.8 percent during construction. A test of proportions shows that this particular increase is significant at the 2 percent level. In other words, the overall distribution of crash type is similar in the before and during-construction periods, but the specific category of rear-end collisions shows a significant increase. Likewise, the data in Table 15 show no significant difference in the overall distribution of contributing factors between the two periods. Although the total number of accidents increased by 26 percent, the proportion of accidents for most contributing factors changed by less than 1 percent between the before and during-construction periods. However, there was a decrease in the proportion of accidents for which the investigating officer cited no contributing factor. Of greater importance, the proportion of accidents in which following too close was the principal contributing factor jumped from 4 to 7 percent; a test of proportions finds that this particular increase is significant.

Because of the lack of reliable traffic volume data (for reasons previously discussed), it is not possible to calculate and compare the true accident rates before and during construction. Nevertheless, one might properly inquire whether the apparent increase in traffic accidents during construction was actually due to a growth in travel. In the absence of volume data, it is not possible to answer this question with certainty. It is informative, however, to examine the overall crash experience on the state's rural roads. Between 1982 and 1985, the number of accidents on New Mexico's rural Interstates and FAP and FAS routes decreased at annual rates of 4.5 percent, 6.7 percent, and 2.0 percent, respectively. Therefore these general trend statistics suggest that irrespective of traffic volume changes, overall accidents were declining during these years, but accidents were increasing on sections of roadway that were under construction.

Accident rates on the basis of reported 1984 traffic volumes for the different highway systems represented in the Group A sites are as follows:

	Accidents per Mvm	
System	*Before Construction*	*During Construction*
Interstate	0.67	0.89
Federal-aid primary	1.12	1.32
Federal-aid secondary	2.18	2.67

For all three systems, the accident rates in the before period are similar to the rates on rural sections of these highway systems. If the Group B sites, with no accidents during either of the two periods, were included, the accident rates would drop slightly.

It can be stated with a high degree of certainty that the 631 accidents assigned to Group A during construction actually occurred within the boundaries of the project while construction activity was taking place. However, it is possible that construction was not in progress at the exact time and location of a particular accident. For example, 189 of the accidents occurred on weekends. Although the contractor may not have been working on weekends, many of the hazards inherent in construction zones remain during these periods. In addition, an entire project is not under construction on a particular day. Nevertheless, it is surprising that in the computerized record system, only 46 of these 631 accidents indicated the presence of construction activity. In other words, 93 percent of the accidents that are known to be in a construction zone are not reflected as such in the computer files. This is another indication of the extent to which the accident record system seriously and systematically underestimates the incidence of these zone accidents.

FHWA CONSTRUCTION FILES

The New Mexico division of FHWA maintains records of the inspections that FHWA engineers have conducted at federal-aid construction projects. Reports based on these visits provide a good description of project characteristics, including an identification of deficiencies. FHWA made these records available for this research study. Unfortunately, records in the computerized format were not available before 1985, so many of the projects considered in this research were not contained in the files. The inspections also included a significant number of projects in urban areas, which were excluded from this study. Nevertheless, the files provided some interesting insight into the types of deficiencies observed during spot inspections.

The inspection files and the associated computer records may identify one or more of the 225 "findings," including problems with earthwork, pavements, structures, labor practice, and other items. Despite the diversity of possible "findings," problems related to the proper application of traffic control devices are cited more frequently than any other category. There are several possible explanations for this situation:

- Standards for traffic control are well established; deviation from formal traffic control plans or from good practice may be easily observed by FHWA engineers;
- Standards for traffic control are normally applicable over the entire length and for the complete duration of the project, thus affording more opportunities for deficiencies;
- Periodic inspections may more readily detect the deteriorated quality of signs and markings, which are not obvious to those working on the project daily;
- Possible "findings" are often subjective, although most of those dealing with traffic control devices and related safety features are more objective; and
- FHWA engineers are told to look more closely at traffic through construction zones.

There was no obvious correlation between the inspection results and the observed accident experience at the sites. This conclusion is not unexpected because the inspections are periodic and the time periods covered by FHWA files and the accident data differed. Nevertheless, it appears that the inspections serve a useful purpose by heightening the awareness of field personnel to proper safety techniques. Over a longer period, the inspections may assist in identifying contractors who habitually violate traffic control device standards or types of projects that are more likely to have deficiencies.

CONCLUSIONS

When this study was initiated, the authors envisioned that the accident analysis would highlight those characteristics of construction-zone crashes that could be addressed with the application of engineering countermeasures. As previously discussed, the number and rate of accidents increased during construction, but the increase appears to be uniform for various accident characteristics. It is therefore not possible to recommend safety measures to counteract a specific set of accident types that increase disproportionately during construction.

The authors have found that the frequency of accidents in construction zones is substantially greater than indicated by the accident record system. The existing records do not provide a definitive answer to the role of engineering in the occurrence of construction-zone accidents.

In comparison with the identical period in the prior year, accident experience in construction areas increased 33 percent on the rural Interstate system and 17 percent on the rural FAP and 23 percent on the rural FAS systems. The data clearly indicate that adverse weather conditions are not responsible for the increases. The data also suggest that certain accident characteristics may be overrepresented in construction-zone accidents. These parameters include accidents involving multiple vehicles, rear-end collisions, large trucks, and the contributing factors of following too close and improper lane changing. On the other hand, the relative proportions of accidents resulting in injury or occurring at night remained virtually unchanged.

Although the accident characteristics differ less than expected, they do suggest some opportunities for remedial action. For example, the proportion of rear-end collisions increased from 9 percent in the before-construction period to 14 percent during construction. This collision pattern may be partially corrected by the proper application of traffic control devices and use of flaggers and by techniques to enhance the visibility of work sites. The data collection effort required to assemble the appropriate information prompts a few conclusions regarding the adequacy of the record systems. Although shortcomings of the accident reporting and record systems have been discussed before (*8*), the inaccurate recording and the miscoding of construction accident data are more serious than previously considered.

The FHWA and NMSHD inspections, as well as some engineers' diaries, indicate that improper traffic control is the most prevalent problem in construction areas. A need exists to heighten the awareness of both state employees and contractors' personnel regarding the importance of installing, maintaining, and monitoring the adequacy of these devices. A critique of traffic control deficiencies could be prepared; however, the implied criticism is potentially counterproductive. A more positive educational effort could produce better results. The traditional emphasis in courses on construction-zone traffic control has been on the use of proper devices along with some examples of inferior treatments (*9*). Based on this study, more attention should be devoted to the preparation of traffic control plans, modifications to plans, inspections, and recordkeeping.

ACKNOWLEDGMENTS

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Large Glass Beads for Pavement Markings

JAMES KALCHBRENNER

Many changes have occurred in the pavement marking industry in the past 20 years, especially the commercialization of polymeric nonshrink binders (epoxy, polyester) as durable striping materials. These materials are normally applied at thicknesses between 15 and 20 mils. Thermoplastic materials are used with application techniques and resin systems unknown 20 years ago. These materials are applied at thicknesses between 40 and 125 mils. It became apparent that as striping line durability and net line thickness increased, glass bead characteristics had to change. The extended durability of these films has shown the need for bead surface treatments that improve bead adherence to the binders. The rheology and wet film application of these new materials indicate a need for a large-diameter glass bead. Use of larger beads fits the theoretical requirement of bead embedment and binder thickness for optimum retroreflectivity. The author explains that when pavement markings are viewed as a system (i.e., bead size and surface treatment are compatible with a specific striping material), improved reflective performance can be obtained with the added benefit of wet pavement/nighttime reflectivity.

Glass beads have been used to make pavement markings reflective for approximately 50 years. If properly embedded in a striping material, glass beads have the ability to collect incident light and reflect part of that light back toward its source. It is this ability that makes these small, spherical glass particles unique and ideally suited to make pavement markings visible at night.

BACKGROUND

The principles of retroreflectivity were first studied by Pocock and Rhodes (*1*) in 1952 and subsequently demonstrated by Dale in a 1967 NCHRP Report (*2*). The work was done at the Southwest Research Institute as part of NCHRP Project 5-5: Nighttime Use of Highway Pavement Delineation Materials. Dale's work was further verified and studied by Vedam and Stoudt and published in an appendix to work by Shuler (*3*). The objective of Dale's research was to study ways of improving delineation of roadways under wet and dry conditions either by improving techniques using then-existing materials or by developing new materials and techniques. It was noted that during periods of adverse weather, the small glass beads used as reflective media often became submerged in a film of water. Light from headlights bounced off this water surface and was lost. It was concluded that the retroreflective capabilities of highway beads that functioned well when the roadway was dry were significantly reduced during rain and often during foggy or misty conditions as well.

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As part of the background research for his work, Dale studied the performance of available marking materials in the field, considering both wet and dry conditions, levels of precipitation, and road characteristics. He also studied the performance of glass beads in the laboratory and demonstrated that the optimum embedment for reflectivity of glass beads in a binder was 60 percent of the bead diameter. As a result of this research, an article was published in the January 1969 issue of *Better Roads* (*4*) that alluded to the fact that only a small percentage of drop-on beads were efficient retroreflectors because only a small percentage were optimally embedded to 60 percent of their diameter. The conclusion was that a narrower gradation with a smaller drop rate would be a more efficient solution.

Although this argument was advanced, specifications remained virtually unchanged. The question of optimum gradation—that is, the use of a wide size range (from 20 to 80 mesh) or a narrowed gradation (from 40 to 80 mesh)—was answered by Ritter (*5*). He showed that both for initial and long-term reflectivity, the typical 20 to 80 mesh size is preferred. This was based on the following factors:

- Striping equipment does not apply a uniform film thickness—a nominal application of 15 mils could realistically be 15 ± 5 mils;
- Materials applied at 15 mils wet will dry to 8 mils, assuming 50 percent solids in paints;
- A sphere should be embedded to 60 percent of its diameter for optimum durability and visibility.

The first and third factors above are still valid; however, the assumption that striping materials always dry to 50 percent of their wet film thickness is no longer valid. In the early 1970s, work was initiated on chemically reactive, 100 percent solid, two-component striping materials. The state of Minnesota, with the H.B. Fuller Company, did major developmental work on 100 percent solid epoxy striping material (*6*). This material is typically specified at 15-mil wet film curing to 15 mils. The beads specified are the same as those for paint binders, but they are applied at a rate of 25 lb/gal versus 6 to 8 lb/gal in order to achieve quicker no-track and good reflectivity. This loading of beads has been noted to inflate the total line thickness to 35 to 40 mils (*7*).

Concurrent with the initial work that was being done on epoxy, reactive polyester material was being tested in Minnesota. The material performed well and, combined with good promotion by local manufacturers, became an accepted striping material in Ohio. From Ohio, use spread to neighboring states and beyond. As with epoxy, reactive polyester is also typically specified at 15 mils applied, curing to almost the same film thickness. Bead size is the same as that specified for paint binder systems, but the application rate is increased to 18 to 20 lb/gal. Again, this is to achieve quicker no-track and good reflectivity, because some standard beads are enveloped in the thicker binders.

In addition to the development of field-reacted materials, hot-applied thermoplastics have significantly increased in use. Applied film thicknesses range from 40 to 125 mils. In his 1967 report on the NCHRP project, Dale noted that the practice was to use essentially the same drop-on bead gradation in thermoplastics as was being used in paints. This is still true today, although Dale's recommendations at the time were for the use of larger-size beads to project up through submerging films of water and achieve improved wet reflective performance in thermoplastics. This was an apparent solution to the problem of wet reflectivity and was "begging" for application.

In addition to the advent and increased use of nonshrink polymeric binders, other changes have been made in paint chemistry in recent years. Environmental concerns have encouraged the development of good water-based paints with higher solids content than typical alkyd traffic paint. Alkyd paints have also changed to comply with the requirement for short no-track time, affecting the final film thickness.

Thus, over the years there has been a general broadening of binder types and a net increase in thickness of paint or binder film without a commensurate increase in bead size to maintain the optimum 60 percent embedment.

A number of approaches to improve performance have been taken with the typical 20 to 80 mesh beads that have worked so well for so long. For example, wicking around a 20-mesh bead would beneficially change net embedment from 30 percent to 60 percent. However, the same wicking phenomenon would totally submerge an 80-mesh bead. Treating beads with a nonadherent silicone coating would prevent wicking but would also result in poor durability. Thus, silane coupling agents were developed that gave controlled wicking as well as good bead adhesion to the binder system for the 20-80 beads.

Laboratory tests with reactive striping materials (epoxy and polyester) using the bead push-out test, as described by DaForno of Potters Industries at the 1976 TRB Annual Meeting, showed improved glass-binder adhesion with silane coupling agents. In epoxy material, Potters' AC-04 surface treatment improved bead-epoxy adhesion five times over moistureproof-treated beads and was 25 percent better than uncoated beads. In polyester materials, Potters' AC-02 surface treatment improved bead-polyester adhesion 10 times over moistureproof-treated beads and was 50 percent better than uncoated beads. Over time, field experience has verified the value of a proper bead-binder marriage and confirmed the improved performance of adherence coatings.

Silane coupling agents are binder selective. One silane is required for epoxy materials, and another silane is reactive with polyester materials. Because thermoplastics are not a specific chemically reactive material but a generic form of hot-melt adhesives, materials varied from different manufacturers, requiring specific silanes to optimize bead-binder embedment and adhesion.

Still missing, however, was a large-enough bead to give wet pavement/nighttime visibility consistently over the useful life of the line, particularly in the more durable line binder systems that were thicker than conventional paints.

DEVELOPMENT OF THE LARGER GLASS BEAD

In 1984, Potters began experimenting with larger glass beads both in the laboratory and at a field test site in northern New Jersey. It was soon recognized that the combination of a controlled environment and real-world use provided valuable insights into proper bead size and functionality.

LABORATORY PERFORMANCE

Potters' laboratory work with larger-sized beads in pavement markings has included extensive research at the Thomas K. Wood research facility. Better known as Potters' "rain tunnel," the facility provides rain simulation at three different rates (¼, ½, and ¾ in./hr) on a 26-ft wide crowned two-lane road. A simulated, textured road surface was developed as part of the program. Finally, a laser retroreflectometer was developed that facilitated measurement of line performance in the rain. Thus, the wet reflective performance of pavement marking materials could be studied in a controlled environment.

Once performance could be measured, a standard of performance was needed. On a global basis, the International Commission on Illumination suggested a level of 60 mcd/(lux·m^2) as the minimum requirement for retroreflection of pavement marking stripes under wet conditions (*8*). It must be recognized that in measuring performance, a number of devices are being used and developed with varying optics and geometry of both light source and measurement. Inherent in the value of 60 is the acceptance of one device or another as the measuring standard.

Results of laboratory studies measuring wet reflectivity of large beads versus standard beads are shown in Figure 1 for a typical epoxy system and in Figure 2 for a typical thermoplastic system.

As these graphs indicate, the large-bead pavement marking system provides retroreflectivity levels 3 to 4 times higher

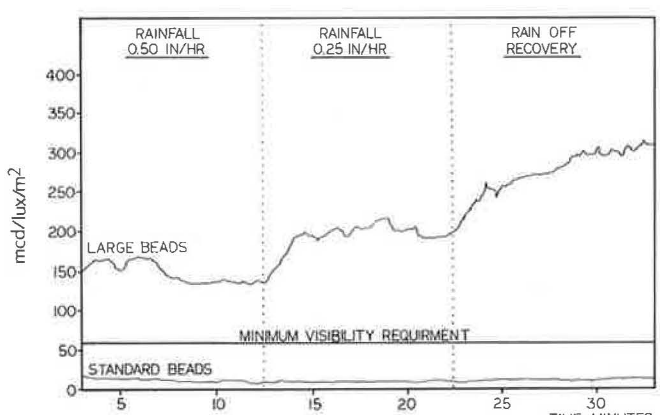

FIGURE 1 Large beads versus standard beads in epoxy.

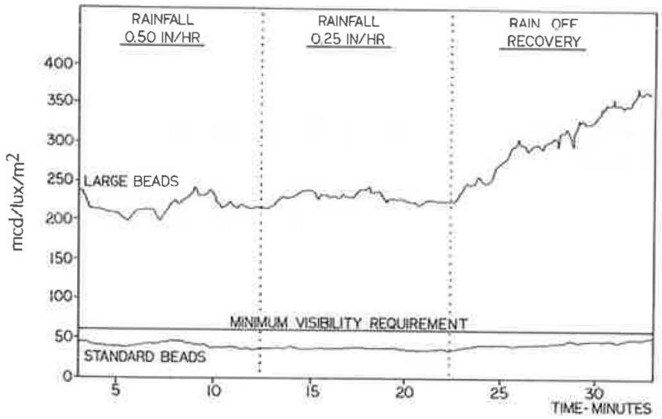

FIGURE 2 Large beads versus standard beads in thermoplastic.

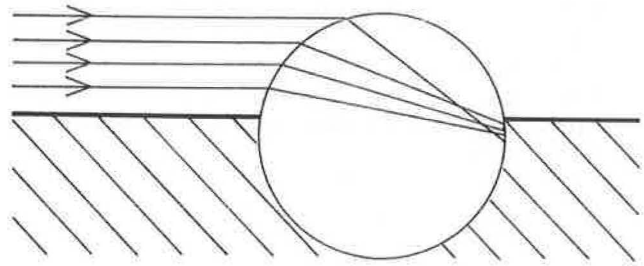

FIGURE 3 Dry bead at optimum 60 percent embedment.

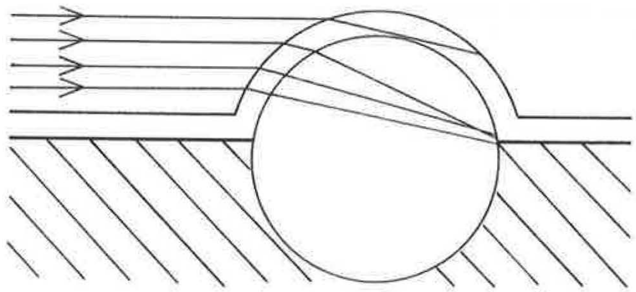

FIGURE 4 Bead with water film preventing retroreflection.

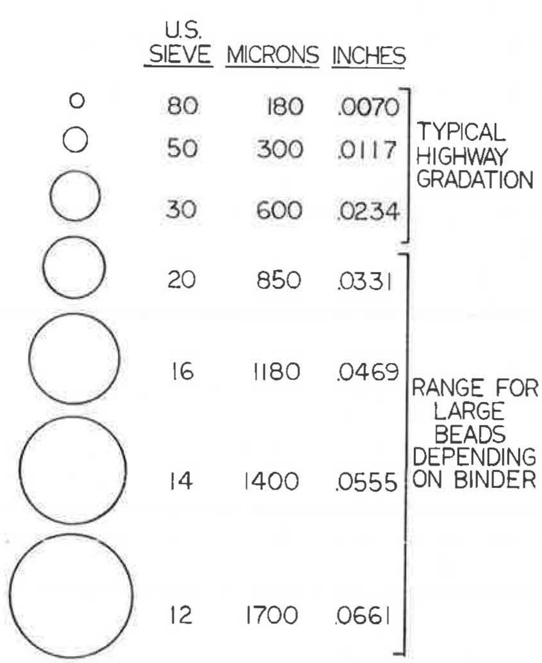

FIGURE 5 Bead size comparison.

than the minimum visibility requirements in rainfall rates up to ½ in./hr—a level twice the ¼ in./hr considered by meteorologists to be heavy precipitation. When the rain stops, the large-bead pavement markings recover quickly to extremely high retroreflectivity values. A pavement marking line of 350 mcd/(lux·m^2) provides very bright guidance to a driver. By comparison, standard highway beads in the same pavement marking binder fall well below the target of 60 mcd in rainfall. Even more important, after the rain stops, pavement markings with standard beads still do not provide effective visual guidance.

Laboratory studies have shown that as rainfall occurs, a thin film of water spreads uniformly over a stripe containing glass beads. This thin film not only prevents the collection and retroreflection of light, but also changes the optics of the bead by increasing the optical embedment without changing the apparent embedment (Figures 3 and 4).

Further, it was found that when the water film builds, surface tension forces are overcome and gravity causes water to flow down the sides of the beads. Kulakowski and DiGiovanni (*9*) studied this effect and found that the equilibrium water film is about 50 microns (2 mils) deep and that this depth is not strongly influenced by rainfall rate or bead size.

After different bead sizes had been tested, it was determined that properly embedded beads within the size range of 10 to 20 mesh, depending on binder, could overcome the water film effect and reflect light back even in rainfall rates of ½ in./hr. This is because, compared with the optical performance of smaller beads, the performance of large beads is not greatly affected by the same water film. Figure 5 shows relative sizes of large beads versus standard beads.

ROAD TEST SITE

Initial field trials with large beads were conducted in West Milford, N.J. In this rural bedroom community, actual field applications were made with thin-film materials of less than 20 mils (epoxy, polyester), as well as with thick-film materials [thermoplastic, polymethylmethacrylate (PMMA)] in order to optimize bead binder systems. Variations in binder-film thickness, bead size, and bead surface treatments were evaluated for reflectivity, durability, and wet pavement/nighttime performance.

Retroreflectivity was measured and documented using portable retroreflectometers as described by the author in 1987 (*10*). Macrophotographs over time at the reflectivity measurement sites were used to correlate bead and binder condition with reflectivity in order to establish durability parameters. Wet pavement/nighttime performance was supported by photographs and videotapes during actual conditions. Rain rates and weather conditions were logged to establish threshold performance parameters. Data generated at this test location, as well as at early state test locations, provided information critical to the development of a wet pavement/nighttime system.

FIELD PERFORMANCE

During the past 3 years, Potters Industries has worked with state and local jurisdictions across the United States to demonstrate the effectiveness of the large-bead system. Demonstrations using existing durable binders have been initiated in 7 geographic areas covering 25 states. Table 1 summarizes field experience with large beads by geography, binder, road type, pavement, and marking application.

Although most tests were applied by contractors, a few were installed by state forces. In all cases technical assistance during installation was supplied by Potters Industries, and timely evaluations were jointly undertaken.

Maryland

One of the earliest test sites was on I-795 northwest of Baltimore. Applied in May 1986, this demonstration was part of a larger contract striping job for epoxy, giving a side-by-side comparison between large beads and standard beads. The installation was evaluated with retroreflectometers at timely intervals with the active cooperation of the Maryland Department of Transportation. Large beads proved to be more retroreflective initially and during subsequent evaluations. In addition, photographic evidence of wet pavement/nighttime reflectivity (see Figure 6), as well as a videotape of wet pavement performance, was obtained 6 months after application in November 1986. Although there was a measurable loss of retroreflectivity due to wear and winter maintenance, macrophotographs show a sufficient amount of large beads still in place after 2 years to provide wet visibility (see Figure 7).

Pennsylvania and Oregon

In November 1986 a contractor-applied epoxy test site was installed on the Schuylkill Expressway, I-76, in Philadelphia. Average daily traffic at this site is in the range of 100,000 vehicles. The installation has performed well through two winters. Wet pavement/nighttime reflectivity was documented on videotape and with photographs (Figure 8). When the test site was last evaluated with state maintenance forces in May 1988 after two winters, a foreman commented that a test area was always evident in the rain. Again, durability was documented with portable retroreflectometers and macrophotographs (Figure 9).

Additional photographic evidence (Figure 10) of wet pavement/nighttime retroreflectivity was obtained from an epoxy test site in Salem, Oregon, 6 months after installation. Water on the pavement from melting snow obliterated all but the large-bead edgeline, which was placed adjacent to a standard edgeline.

Florida and California

Another test site was in Altamonte Springs, Florida, on a two-lane rural road. Installed in March 1987 using spray thermoplastic according to Florida specifications, the site was evaluated by a local observer over a year later and noted as being wet-reflective.

Another thermoplastic demonstration site was in California on a four-lane divided highway. Installed by the California Department of Transportation in August 1987, the site was observed to provide good wet-reflective performance by a state evaluator 6 months after installation.

Ohio

A large-bead polyester system was installed in Ohio in July 1987. This installation was an edgeline at the point where a divided highway becomes a two-lane rural road. Wet-reflective performance was photographed in December 1987 (Figure 11). In the summer of 1988, large-bead/polyester edgelines were installed on the Ohio Turnpike throughout its entire 241-mi length (Figures 12 and 13). Wet pavement/nighttime performance was rated as exceptional.

Figure 14 represents total field experience and evidence of wet pavement/nighttime retroreflectivity.

DISCUSSION OF RESULTS

Whereas visibility in rainfall may only be required for minutes or hours, pavement markings are designed to provide effective guidance for months and years. Service life of the large-bead system is particularly important because final product development to date has been in durable binder materials. In addition to providing effective wet pavement/nighttime visibility, large-bead systems must provide effective dry pavement/nighttime visibility over the life of the line. The International Commission on Illumination has suggested a minimum retroreflectivity level for pavement markings under dry conditions. This level of 150 mcd/(lux·m^2) for white markings is more than twice as high as the 60-mcd level established for wet conditions.

Figure 15 is a compilation of dry-reflective data averaging the relative performance of large beads versus standard beads in epoxy. The data base is from 12 field test sites that include variations in road type, pavement, and line type. The numbers for dry retroreflectivity were obtained using portable Mirolux retroreflectometers. The most recent information shows the curve declining slightly but still above the minimum dry visibility requirement of 150 mcd/(lux·m^2). By comparison, the standard highway beads in epoxy reached the minimum reflectivity level suggested by the International Commission on Illumination within 1 year.

Figures 16 and 17 document the retroreflective performance of large beads in thermoplastic and polyester. Again, the data base is representative of a compilation of test sites and as

TABLE 1 FIELD EXPERIENCE WITH LARGE BEADS

GEOGRAPHY	BINDER	ROAD	PAVEMENT	MARKINGS
Northeast				
Massachusetts	thermo	2 lane	ASP	center
		4 lane		edge & skip
New York	epoxy	Interstate	PCC	edge & skip
Connecticut	epoxy	Interstate	ASP	edge & skip
Middle Atlantic				
New Jersey	epoxy	2 lane rural	ASP	center & edge
	thermo	2 lane rural	ASP	center & edge
	epoxy	Interstate	PCC	edge & skip
Pennsylvania	epoxy	Interstate	ASP	edge & skip
			PCC	edge
Delaware	epoxy	2 lane rural	ASP	center & edge
Maryland	epoxy	Interstate	PCC	edge & skip
	latex paint	Interstate	ASP	edge & skip
Virginia	thermo	4 lane	ASP	edge
Southeast				
North Carolina	epoxy	Interstate	PCC, ASP	edge & skip
Georgia	thermo	4 lane	ASP	edge & skip
Florida	thermo	4 lane rural	ASP	skip
		2 lane	ASP	edge
South Carolina	thermo	4 lane	ASP	edge & skip
Tennessee	polyester	2 lane	ASP	center
Midwest				
Ohio	polyester	Interstate	ASP	edge
		2 lane rural		
Michigan	polyester	2 lane rural	ASP	edge
Wisconsin	polyester	2 lane rural	ASP	center
	epoxy	4 lane	ASP	edge
Illinois	epoxy	2 lane rural	ASP	center
Indiana	thermo/poly	2 lane	ASP	center
Mountain				
Montana	epoxy	Interstate	ASP	edge
Colorado	epoxy	Interstate	PCC	edge
Utah	epoxy	Interstate	PCC, ASP	edge & skip
West				
Washington	epoxy	4 lane urban	ASP	edge & skip
Oregon	epoxy	4 lane	ASP	edge
California	thermo	4 lane	ASP	edge & skip
Southwest				
Texas	thermo	Interstate	ASP	edge & skip

FIGURE 6 Interstate 795, Baltimore, Maryland: large beads in epoxy installed May 1986 on edge and skip; rain of 0.20-in./hr, Nov. 1987.

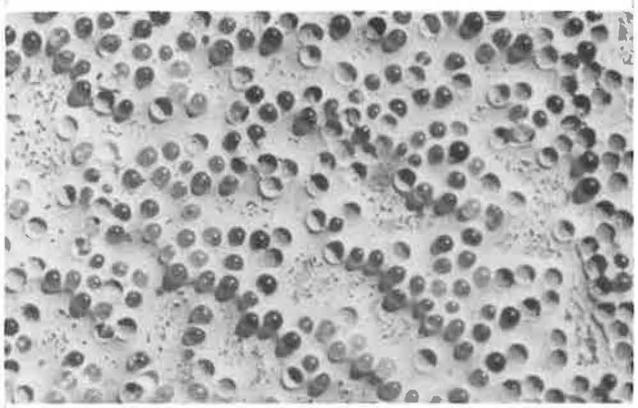

FIGURE 7 I-795 in Baltimore: large beads after 2 years of service.

FIGURE 8 Schuylkill Expressway (I-76), Philadelphia, Pennsylvania: large beads in epoxy installed Dec. 1986; standard beads used for skipline between large-bead skips; Dec. 1986 during recovery after rain.

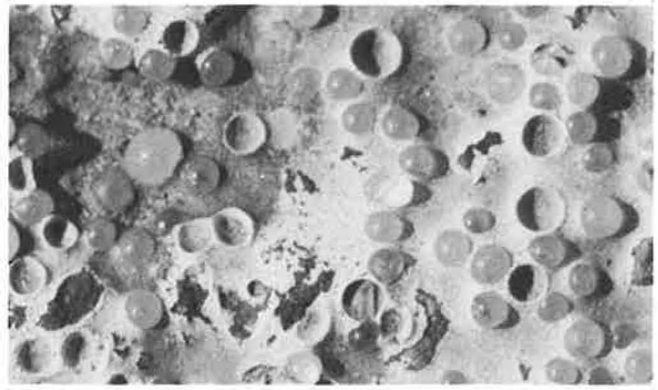

FIGURE 9 Schuylkill Expressway (I-76): large beads after two winters.

FIGURE 10 Salem, Oregon: large beads in epoxy installed June 1987; large bead edgeline adjacent to standard bead line; Dec. 1987, wet pavement/nighttime.

more information is generated, the retroreflectivity curves will be updated. With the thin-film reactive binders, polyester and epoxy, application rates for large beads are the same as those for standard beads. Actual rates are 24 lb/gal for epoxy and 20 lb/gal for polyester. With thicker thermoplastic materials, recommended bead application rates are approximately twice the optimum standard bead application rate because the larger beads do not cover as much area per pound. To date, performance with large beads has been superior to standard bead performance at the same sites.

Potters Industries has extensively demonstrated the effectiveness of large-bead systems since their initial development. In addition to the demonstrations in the United States, large beads are under evaluation in Canada, Europe, Japan, and Australia. Large beads, which Potters has trademarked as Visibeads®, are being actively promoted for use in epoxy, polyester, and thermoplastic. Plans are being implemented to finalize development in traffic paint.

FIGURE 11 Cleveland, Ohio: large beads in polyester installed Aug. 1987; edgeline with standard beads in background; Dec. 1987, wet pavement/nighttime.

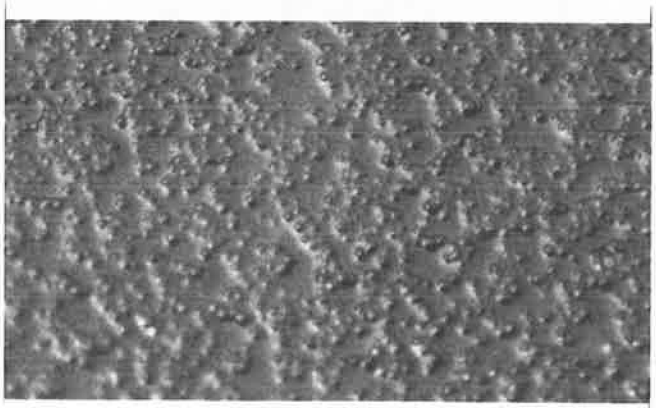

FIGURE 12 Ohio Turnpike: large beads in polyester (top view).

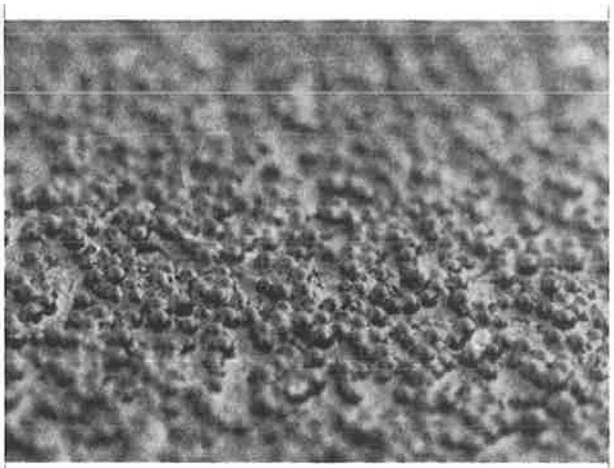

FIGURE 13 Ohio Turnpike: large beads in polyester (profile view).

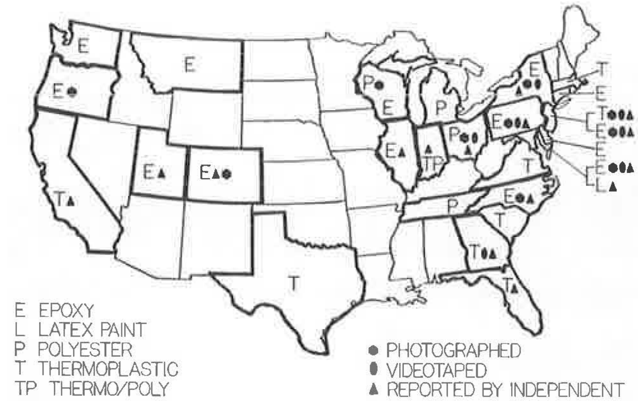

FIGURE 14 Field experience and wet pavement/nighttime evidence.

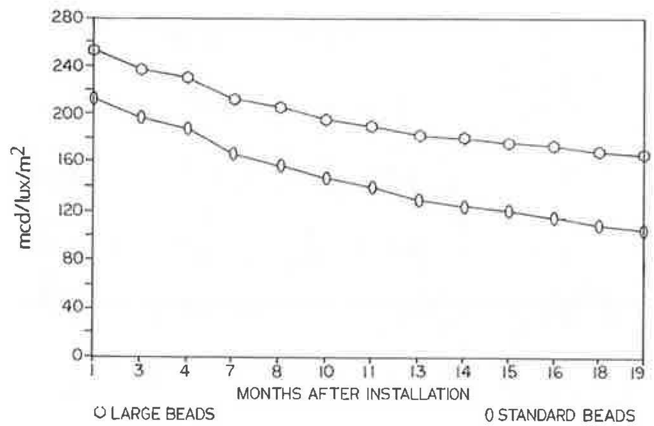

FIGURE 15 Retroreflectivity: large beads versus standard beads in epoxy.

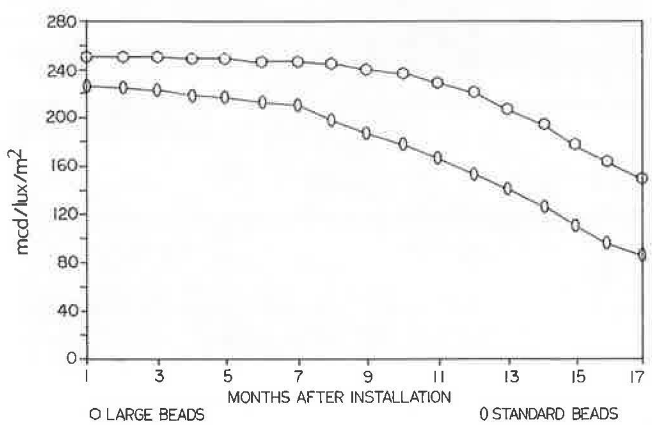

FIGURE 16 Retroreflectivity: large beads versus standard beads in thermoplastic.

RECOMMENDED SPECIFICATIONS

Thin-film, chemically reactive binders (polyester, epoxy, and others) have similar liquid properties, and bead size recommendations are dependent on film thickness. Potters Industries has also experimented with a "dual-drop" application system, in which two separate bead drops are used with large beads applied first, immediately followed by a binder-specific standard bead size. Both single- and dual-drop application gives similar initial wet pavement/nighttime performance and dry retroreflectivity. The field trials with the dual-drop system have thus far shown slightly improved dry performance over time as measured by retroreflectometers. Tables 2–4 give the gradation specifications for 100 percent solid, thin-film materials; standard beads for the dual-drop system; and thick-film binders.

CONCLUSIONS

Current gradations of glass beads are correct if selected materials are properly matched and properly applied; however, much more assistance can be provided to road users by treating pavement markings as a system. The term "system" implies design and synergy. Improved roadway performance and service life have been demonstrated at multiple locations in durable materials by properly sizing and treating beads for the thickness and type of binder used. Not only are dry retroreflectivity and durability significantly improved, but delineation of roadways under wet conditions is attained. Work done on NCHRP Project 5-5, started in 1965, suggested the use of large glass beads, but materials were not available at that time to reach the ultimate end product. Now the tools are available to solve these problems.

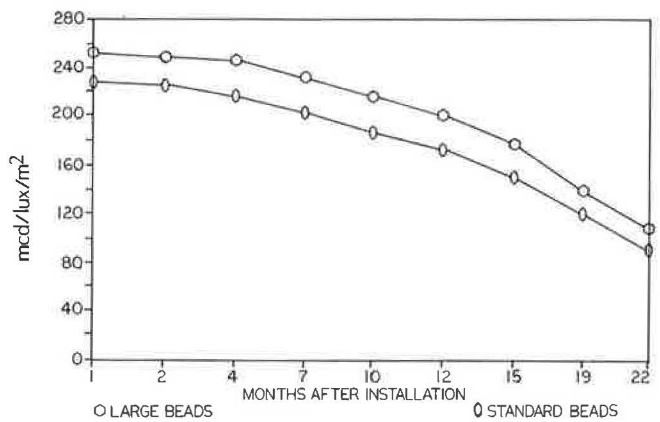

FIGURE 17 Retroreflectivity: large beads versus standard beads in polyester.

TABLE 3 GRADATION OF STANDARD BEADS FOR DUAL-DROP APPLICATION

U.S. Sieve	Percent On
20	0–5
30	5–20
50	30–75
80	9–32
100	0–5
PAN	0–2

TABLE 4 GRADATIONS FOR THICK-FILM BINDERS (THERMOPLASTICS AND PMMA)

U.S. Sieve	Sunbelt	Moderate	Northeast
6	–	–	–
8	0–5	–	–
10	5–20	0–5	–
12	40–80	5–20	0–5
14	10–40	40–80	5–20
16	0–5	10–40	40–80
18	–	0–5	10–40
20	–	–	0–5
PAN	0–2	0–2	0–2

NOTE: Recommended specifications for thermoplastics vary depending on geographic location, with the largest size used in Sunbelt locations. In all cases the dual-drop system is used with thermoplastics.
Application rate: Dual drop—12 lb large + 12 lb std/100 ft^2. Rounds: 75 percent per screen, 80 percent overall. Coating: binder specific.

TABLE 2 GRADATIONS FOR DURABLE 100 PERCENT SOLID THIN-FILM MATERIALS

15 Mils		15-Mil Dual Drop and 20-Mil Single Drop		20-Mil Dual Drop	
U.S. Sieve	Percent On	U.S. Sieve	Percent On	U.S. Sieve	Percent On
8	–	8	–	8	–
10	–	10	–	10	0–5
12	–	12	0–5	12	5–20
14	0–5	14	5–20	14	40–80
16	5–20	16	40–80	16	10–40
18	40–80	18	10–40	18	0–5
20	10–40	20	0–5	20	–
25	0–5	25	–	25	–
PAN	0–2	PAN	0–2	PAN	0–2

NOTE: Application rate: Single drop—epoxy, 24 lb/gal; polyester, 20 lb/gal. Dual drop—epoxy, 12 lb large + 12 lb std/gal; polyester, 10 lb large + 10 lb std/gal. Rounds: 75 percent per screen, 80 percent overall. Coating: binder specific.

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Embedment and Retroreflectivity of Drop-On Glass Spheres in Thermoplastic Markings

JIM O'BRIEN

Embedment characteristics of drop-on moistureproofed and uncoated glass spheres and their subsequent retroreflectivity were evaluated subjectively in various types of hot-applied thermoplastic traffic markings by illuminating test panels in a dark room. In all of the hot-applied thermoplastic traffic marking types tested, uncoated drop-on spheres were generally overembedded because of positive wetting of the spheres by the thermoplastic traffic marking, and their retroreflectivity varied. The use of moistureproofed drop-on spheres in various thermoplastic traffic marking types resulted in optimal bead embedment with subsequent excellent retroreflectivity. The optimal rate of glass sphere application in all of the thermoplastic marking types was found to be 10 lb of moistureproofed glass spheres per 100 ft^2—this rate enhanced retroreflectivity, bead embedment, and coverage. The retroreflectivity of the standard gradation of glass spheres may be enhanced in all of the thermoplastic types by increasing the percentage of spheres retained on U.S. sieves 30, 40, and 50 and by increasing the overall rounds from 70 to 80 percent.

The following is a discussion of the various properties of glass spheres that are needed to define and promote the necessary retroreflectivity for critical nighttime visibility of various thermoplastic traffic marking systems.

Hot-applied thermoplastic traffic markings are 100 percent solid durable markings made initially retroreflective by the simultaneous application of drop-on glass spheres. The control of this glass sphere application is important in providing the maximum amount of initial retroreflectivity and subsequent nighttime visibility.

To ensure the success of drop-on glass sphere applications, the physical characteristics of the thermoplastic system must be considered as well as the physical and chemical characteristics of the glass spheres.

The following tests and observations indicate that the use of moistureproofed glass spheres in various manufacturers' thermoplastic systems helps ensure the optimum bead embedment necessary for maximum retroreflectivity. All cases reveal an average embedment of 60 to 65 percent.

Uncoated glass spheres in various thermoplastic systems become excessively embedded because of positive wetting of the spheres confounded by the resulting envelope of thermoplastic film on the upper periphery of the spheres. Even though the spheres appear to be suspended on the surface, the initial retroreflectivity and subsequent nighttime visibility of the stripe are unacceptable. In some situations, uncoated spheres may be completely embedded in the thermoplastic traffic marking, resulting in no initial retroreflectivity because no spheres are present on the surface at all. The initial retroreflectivity of uncoated spheres in both cases is not effective, and their performance in the various systems is quite unpredictable.

Proper bead rates and uniformity of application are critical factors in promoting and maintaining maximum initial retroreflectivity. Bead application rates should not exceed 10 lb/100 ft^2. Applications above these rates do not improve the effective retroreflectivity. The uniformity of the glass sphere application also affects the retroreflectivity efficiency and provides the uniform luminance necessary for accurate perception of the delineator.

Better retroreflectivity was obtained with uncoated spheres when higher percentages were retained on U.S. sieves 30, 40, and 60 and also when the existence of overall rounds exceeded 80 percent.

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EMBEDMENT OF GLASS SPHERES

Preparation of Test Panels

Illinois uncoated and moistureproofed glass spheres were evaluated in various manufacturers' white and yellow hydrocarbon thermoplastic traffic marking systems by drawing down a 4-in. by 12-in. by .125-in. thermoplastic marking line at 450°F on a black vinyl tile heated to 212°F with a 4- by 4-in. open box doctor blade heated to 450°F. The glass spheres were immediately dropped on the hot stripe through four No. 10 mesh screens to ensure uniform distribution of the spheres over the entire area. The drop-on rate used was 6 lb/100 ft^2 in all cases. This rate is the minimum application rate generally accepted throughout the thermoplastic marking industry. The different thermoplastic systems are differentiated as A, B, C, and D. White and yellow samples were tested from each system.

Evaluation Method

All bead applications were evaluated subjectively in a dark room illuminated by a small flashlight at distances of 10 to 25 ft. Photomicrographs were taken of the various applications perpendicularly and in cross section to show bead distribution and embedment characteristics. Bead embedment

TABLE 1 SUBJECTIVE EVALUATION OF RETROREFLECTIVITY AND GLASS SPHERE EMBEDMENT

WHITE SYSTEMS

(RETROREFLECTIVITY/EMBEDMENT)

System	Sphere 1 Uncoated	Sphere 2 Uncoated	Sphere 2C Moisture Proofed
A	Minimal/95-100	Moderate/80-85	Excellent/60-65
B	Minimal/95-100	Moderate/80-85	Excellent/60-65
C	Moderate/80-85	Moderate-Excellent/70-75	Excellent/60-65
D	Minimal/95-100	Minimal 95/100	Excellent/60-65

YELLOW SYSTEMS

(RETROREFLECTIVITY/EMBEDMENT)

System	Sphere 1 Uncoated	Sphere 2 Uncoated	Sphere 2C Moisture Proofed
A	Minimal/95-100	Moderate/80-85	Excellent/60-65
B	Minimal/95-100	Moderate/80-85	Excellent/60-65
C	Minimal-Moderate/90-95	Moderate/80-85	Excellent/60-65
D	None/100	None/100	Excellent/60-65

TABLE 2 SUBJECTIVE EVALUATION OF COVERAGE, RETROREFLECTIVITY, GLASS SPHERE EMBEDMENT, AND VISUAL ACUITY AS RELATED TO APPLICATION RATES

Application Rates (*) (Lbs./100 ft.^{2})	Coverage	Retro-Reflectivity	Glass Sphere Embedment	Visual Acuity
2	Minimal	Min-Mod	60 - 65	Dull
4	Minimal	Moderate	60 - 65	Dull
6	Moderate	Excellent	60 - 65	Sharp
8	Excellent	Excellent	60 - 65	Sharp
10	Excellent	Excellent	60 - 65	Sharp
12	Excessive	Excellent	50 - 55	Scattered
14	Excessive	Excellent	40 - 45	Scattered

* (Spheres 2C) Moisture Proofed

was subjectively estimated and described visually using cross-sectional photomicrographs. Tables 1 and 2 evaluate coverage, retroreflectivity, and glass sphere embedment.

Mechanisms and Influencing Embedment

It should be noted at this point that surface tension plays an important role in controlling bead embedment in the various thermoplastic systems tested. There is minimum shrinkage of the thermoplastic film compared with most traffic paint systems; thus the effect of various coatings on glass spheres in traffic paint is not wholly applicable to thermoplastic traffic marking systems that are 100 percent solid systems. The film thicknesses involved are six times that of most dried paint films, therefore necessitating greater control by the use of treated spheres.

Uncoated Glass Spheres

Drawdowns made with uncoated spheres in various thermoplastic systems indicate excessive bead embedment by either positive wetting or total embedment, or both. Positive wetting reveals variable embedment of 75 to 95 percent, with another 5 percent interference by the thin thermoplastic envelope characteristically formed around the periphery of the untreated spheres (Systems A, B, and C). Total embedment is encountered in System D. This overembedment results in unacceptable retroreflectivity in all systems (A, B, C, and D). Figures 1–6 illustrate drawdowns, cross sections, and embedment of Spheres 1 and 2. See Tables 3 and 4 also.

Moistureproofed Glass Spheres

Use of coated (moistureproofed) spheres results in optimum embedment of 60 to 65 percent in all thermoplastic systems (A–D). The initial retroreflective properties were excellent (see Figures 7 and 8). These spheres are denoted 2C (see Figure 9). As stated earlier, optimum bead embedment is critical in providing maximum initial retroreflectivity. It is also important in providing the necessary retention of glass spheres to maintain a reasonable level of retroreflectivity until the thermoplastic wears down to expose intermix glass spheres (see Table 5).

Controlled Wear and Retroreflectivity

Drop-on glass spheres eventually wear off. Controlled wear is important in durable markings to promote continual renewal

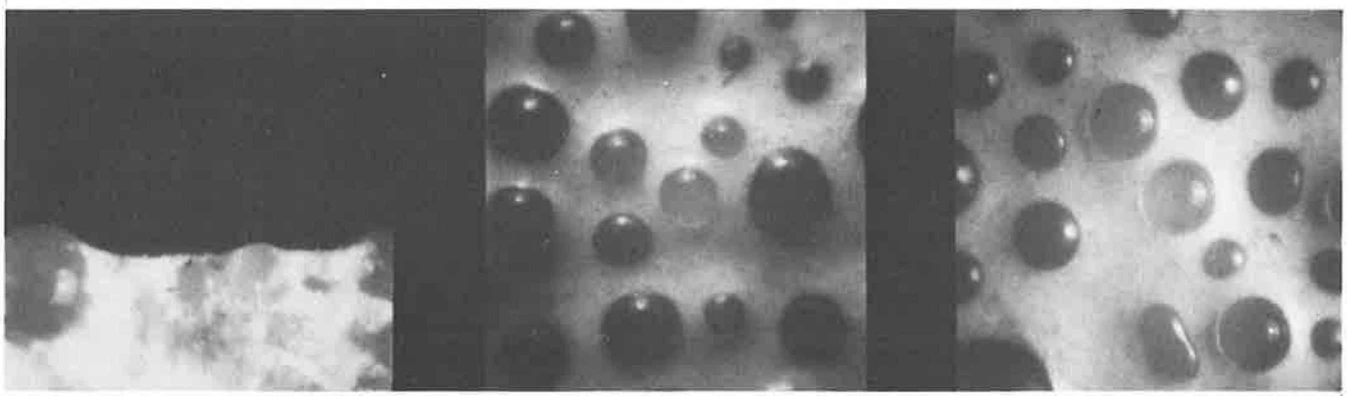

FIGURE 1 Drawdowns and cross section of Systems A, B, and C (bead rate: 6 lb/100 ft^2, white). ***Left*, Sphere 1, cross section (magnified 100×).** ***Middle*, Sphere 1, top view (magnified 100×).** ***Right*, Sphere 2, top view (magnified 100×).**

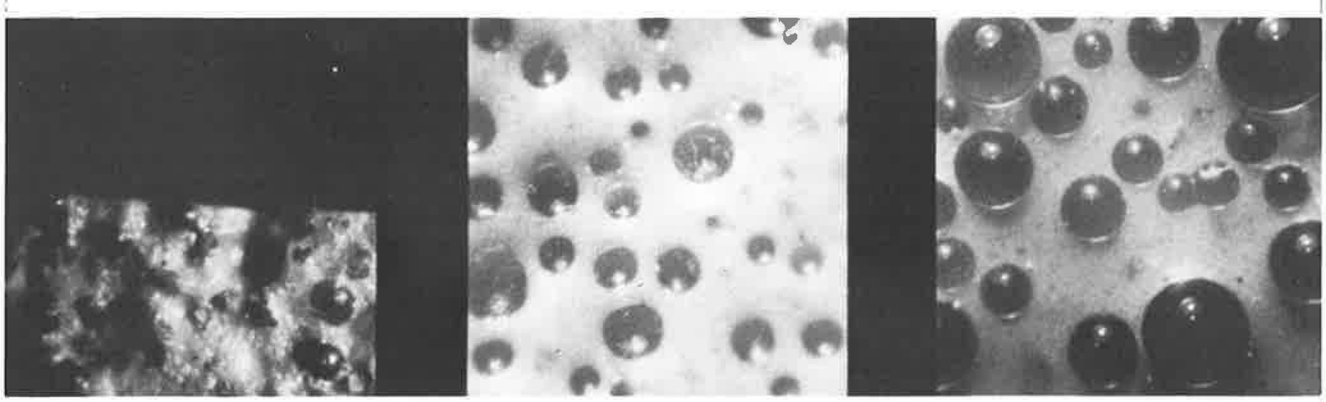

FIGURE 2 Drawdowns and cross section of Systems A, B, and C (bead rate: 6 lb/100 ft^2, yellow). ***Left*, Sphere 1, cross section (magnified 100×).** ***Middle*, Sphere 1, top view (magnified 100×).** ***Right*, Sphere 2, top view (magnified 100×).**

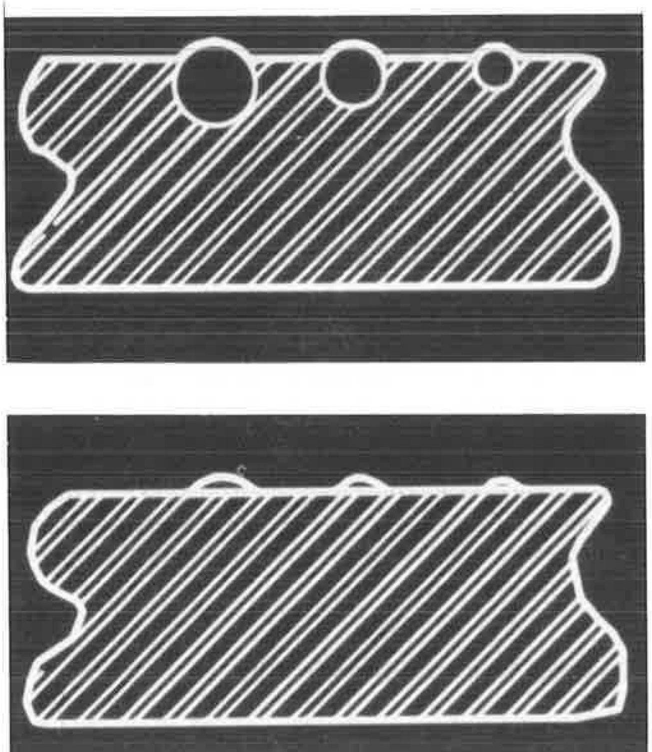

FIGURE 3 Systems A, B, and C, Spheres 1 and 2 (80 to 95 percent embedment).

of the surface. The cleaning and brightening of the surface of the thermoplastic by exposing intermix glass spheres is paramount to continued retroreflectivity and subsequent nighttime visibility.

APPLICATION RATES AND UNIFORMITY

Influence of Substrate and Material Temperatures

Drawdowns were made at substrate and material temperatures of 212°F and 450°F, respectively. The evaluations were made at the higher temperature extremes as worst-case situations. The above substrate and material temperatures exceed maximum temperatures found in any actual field situation.

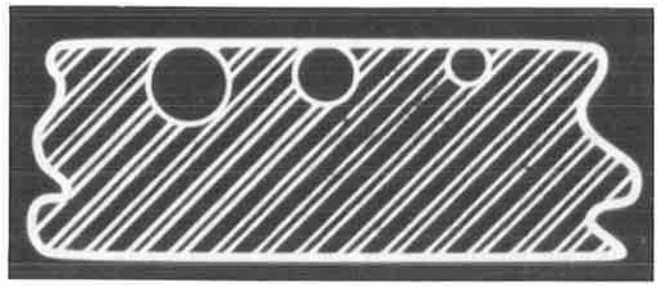

FIGURE 6 System D, Spheres 1 and 2 (total embedment).

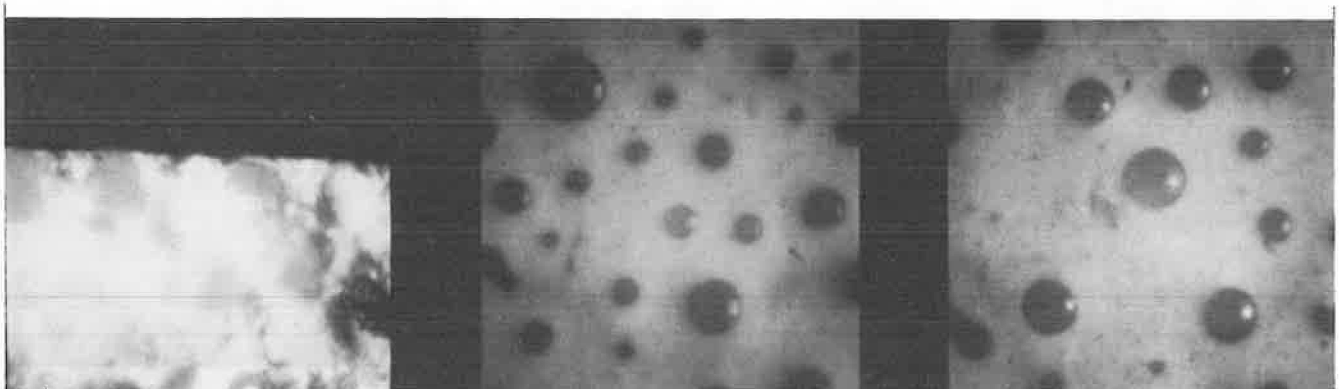

FIGURE 4 Drawdowns and cross section of System D (bead rate: 6 lb/100 ft^2, white). *Left*, Sphere 1, cross section (magnified 100×). *Middle*, Sphere 1, top view (magnified 100×). *Right*, Sphere 2, top view (magnified 100×).

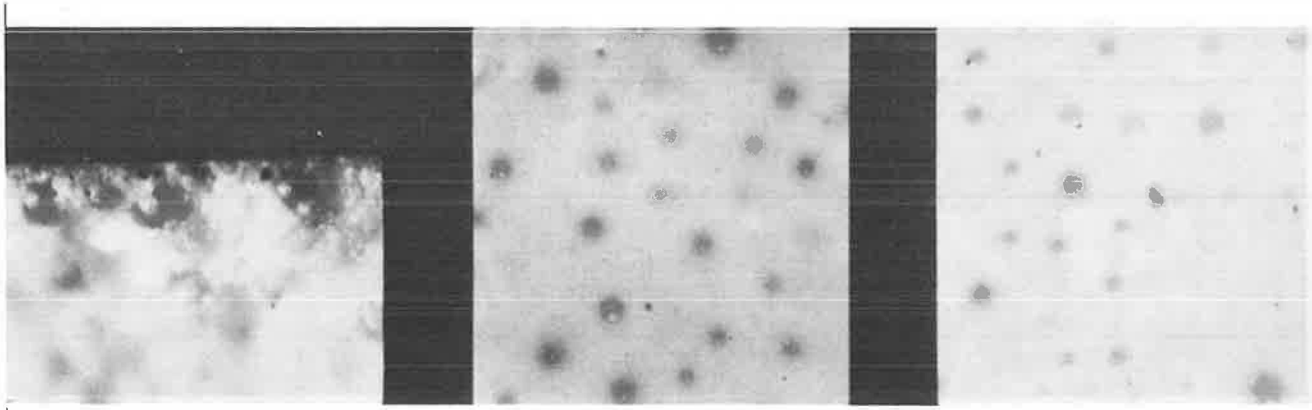

FIGURE 5 Drawdowns and cross section of System D (bead rate: 6 lb/100 ft^2, yellow). *Left*, Sphere 1, cross section (magnified 100×). *Middle*, Sphere 1, top view (magnified 100×). *Right*, Sphere 2, top view (magnified 100×).

TABLE 3 SIEVE ANALYSIS OF UNCOATED ILLINOIS GLASS SPHERES (TYPE 1)

US Sieve	% Retained	% Passing	Spec. % Passing
# 20	0.01	99.99	100
# 30	0.09	99.90	75/100
# 40	2.56	97.34	
# 50	64.36	32.98	15/40
# 60	10.36	22.62	
# 70	15.20	7.42	
# 80	3.43	3.99	
#100	3.75	0.24	0/10
#200	0.24	0.00	0/2
PAN	0.00		

% Rounds - 70 overall

TABLE 4 SIEVE ANALYSIS OF UNCOATED ILLINOIS GLASS SPHERES (TYPE 2)

US Sieve	% Retained	% Passing	Spec. % Passing
# 20	0.00	100.00	100
# 30	1.02	98.98	75/100
# 40	13.77	85.21	
# 50	50.58	34.63	15/40
# 60	18.80	15.83	
# 70	13.31	2.52	
# 80	1.21	1.31	
#100	1.11	0.20	0/10
#200	0.19	0.01	0/2
PAN	0.01		

% Rounds - 83 Overall

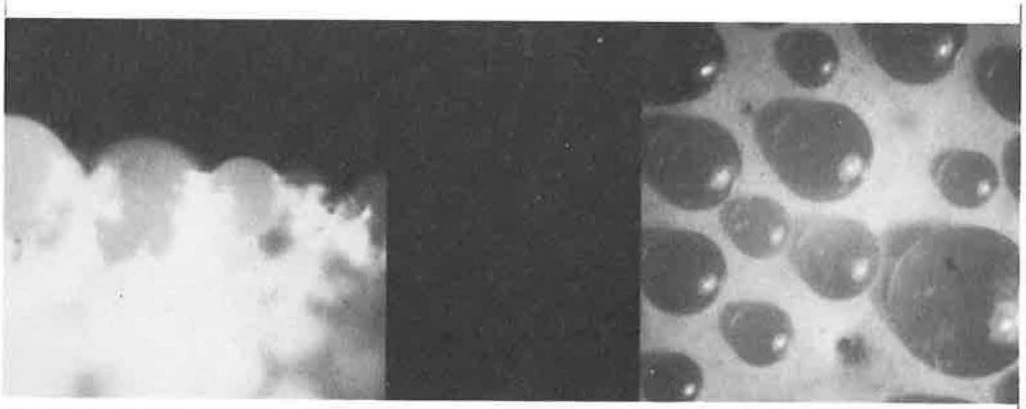

FIGURE 7 Drawdowns and cross section of Systems A, B, C, and D, Sphere 2C (bead rate: 6 lb/100 ft^2, white). ***Left*, cross section (magnified 100×). *Right*, top view (magnified 100×).**

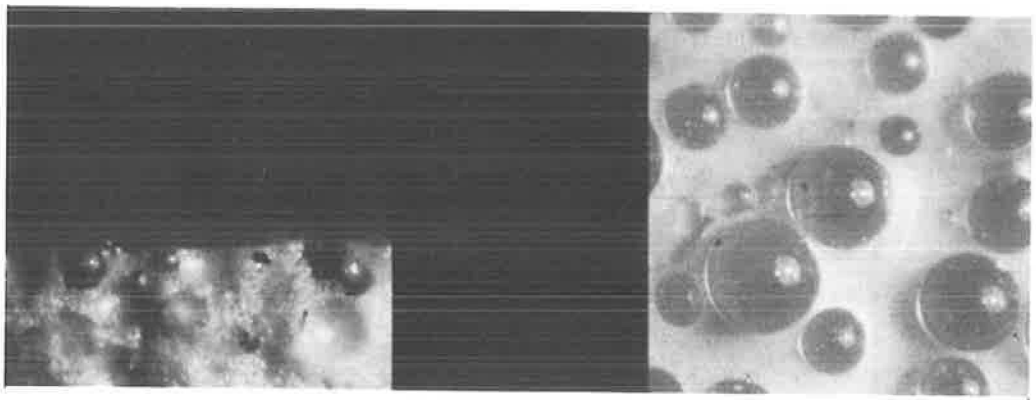

FIGURE 8 Drawdowns and cross section of Systems A, B, C, and D, Sphere 2C (bead rate: 6 lb/100 ft², yellow). ***Left*****, cross section (magnified 100×).** ***Right*****, top view (magnified 100×).**

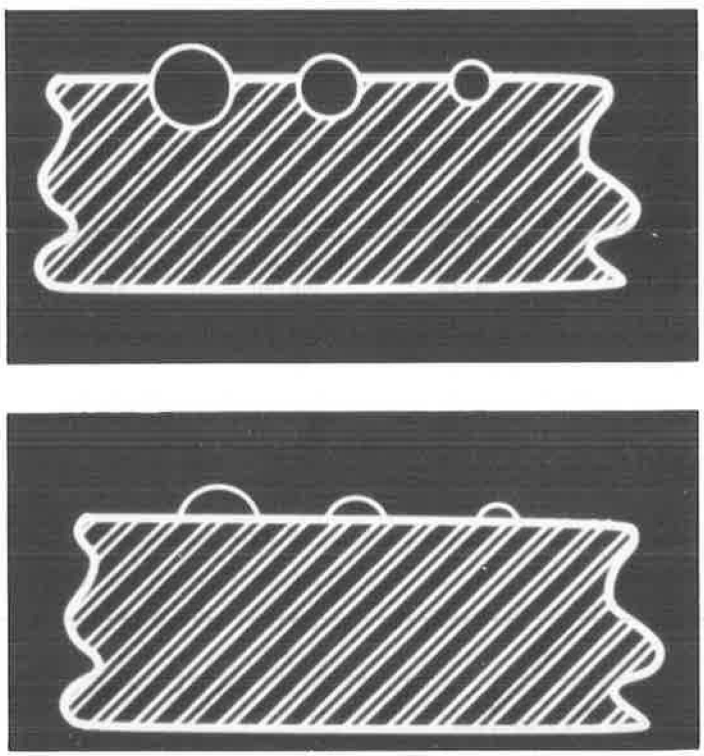

FIGURE 9 Systems A, B, C, and D, Sphere 2C (65 percent embedment).

TABLE 5 SIEVE ANALYSIS OF MOISTUREPROOFED ILLINOIS GLASS SPHERES (TYPE 2C)

US Sieve	%Retained	%Passing	Spec. %Passing
# 20	0.00	99.98	100
# 30	10.16	89.82	75/100
# 50	71.82	18.00	15/40
#100	17.75	0.25	0/10
#200	0.25	0.00	0/2
PAN	0.00		

Influence of Application Rates

The application rate of drop-on spheres also affects the retroreflectivity of the thermoplastic system. By using the same drawdown procedure as described initially, bead drop-on rates of 2, 4, 6, 8, 10, 12, and 14 lb/100 ft^2 were evaluated for visual luminance in System D using moistureproofed spheres 2C. As previously described, use of moistureproofed spheres results in optimum embedment in all thermoplastic systems.

The 2- and 4-lb/100 ft^2 rates were minimally retroreflective and unacceptable due to poor coverage (see Figures 10 and 11). The 6-, 8-, and 10-lb/100 ft^2 applications were all bright and well defined. The 6-lb/100 ft^2 application revealed minimal acceptable brightness as compared with the higher application rates (Figure 12). The 8-lb/100 ft^2 application (Figure 13) was bright and well defined but the coverage was not as uniform as that at 10 lb/100 ft^2 (Figure 14).

When the application rate exceeded 10 lb/100 ft^2, the beads began to overlap (see Figures 15 and 16). Bead embedment diminished, as is evident by the raised appearance of the spheres. It is evident that the spheres were competing for thermoplastic film. Excessive application rates resulted in minimally embedded glass spheres and insufficient retroreflectivity for accurate visual color and delineation perception. The 10-lb/100 ft^2 application was not only bright and well defined but also optimal in coverage. Thus, the optimum application rate is 10 lb/100 ft^2 (see Table 2).

Influence of Uniformity on Glass Sphere Retroreflectivity

Uniformity is important in maintaining uniform luminance necessary for visual perception and acuity of the delineator (stripe). Ten pounds of moistureproofed glass spheres evenly distributed over an area of 100 ft^2 will produce the uniform luminance necessary for accurate perception of the delineator. As previously discussed, glass sphere applications below 10 lb/100 ft^2 do not adequately cover the unit area and result in loss of maximum luminance and visual acuity. In this study, this was accomplished by using a 3½ in. inside-diameter cylinder with four No. 10 mesh screens inserted 1½ in. apart (see Table 2).

FIGURE 10 Glass sphere drop-on rates, System D, Sphere 2C (bead rate: 2 lb/100 ft^2, yellow).

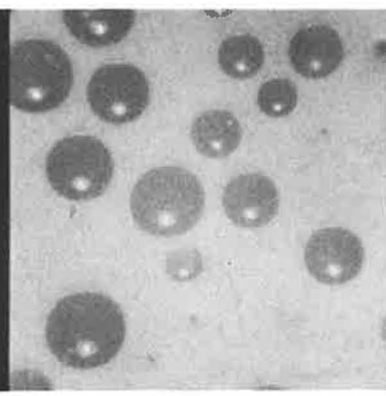

FIGURE 11 Glass sphere drop-on rates, System D, Sphere 2C (bead rate: 4 lb/100 ft^2, yellow).

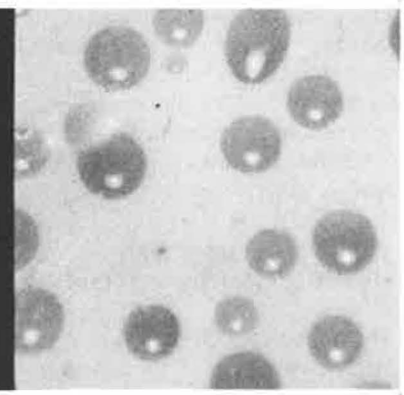

FIGURE 12 Glass sphere drop-on rates, System D, Sphere 2C (bead rate: 6 lb/100 ft^2, yellow).

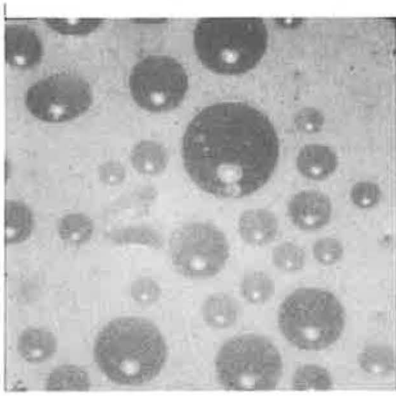

FIGURE 13 Glass sphere drop-on rates, System D, Sphere 2C (bead rate: 8 lb/100 ft^2, yellow).

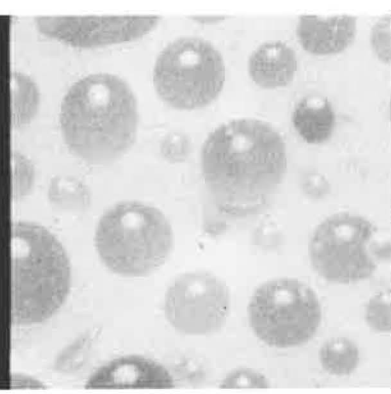

FIGURE 14 Glass sphere drop-on rates, System D, Sphere 2C (bead rate: 10 lb/100 ft^2, yellow).

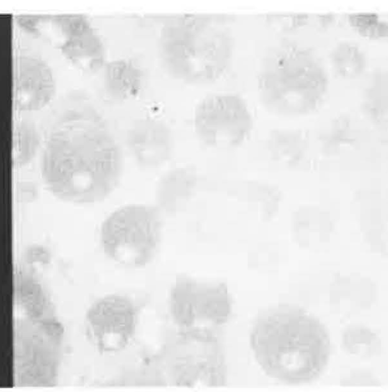

FIGURE 15 Glass sphere drop-on rates, System D, Sphere 2C (bead rate: 12 lb/100 ft^2, yellow).

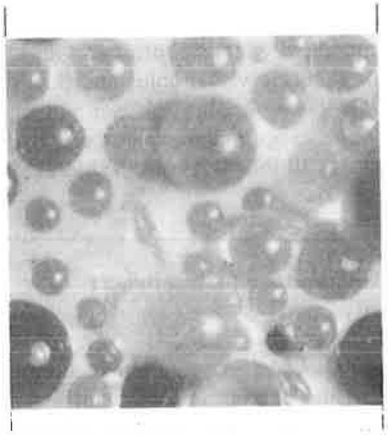

FIGURE 16 Glass sphere drop-on rates, System D, Sphere 2C (bead rate: 14 lb/100 ft^2, yellow).

GRADATIONAL ANALYSIS

Uncoated Glass Spheres (Types 1 and 2)

Finally, an analysis was made of two manufacturers' uncoated spheres that meet Illinois specifications for drop-on glass spheres. Since some differences in the retroreflectivity or brightness resulted in the use of these spheres, it was thought that the brighter panels had some larger spheres. The sieve analysis revealed a higher percentage of +40 and +60 spheres for sphere 2. Also, the overall roundness was 70 percent for sphere 1 and 83 percent for sphere 2 (see Tables 3 and 4) (*1*). While this accounted for some slight differences in brightness, neither sphere produced acceptable retroreflectivity.

Moistureproofed Spheres (Type 2C)

The analysis of the moistureproofed spheres (2C) reveals a larger percentage of 30- and 50-mesh glass spheres than the uncoated spheres. It is hypothesized that larger spheres would aid the initial retroreflectivity of the system by increasing the brightness as it did with uncoated spheres. Further study will be done to confirm this hypothesis (see Table 5).

RECOMMENDATIONS

Moistureproofed Spheres

Based upon the above results, moistureproofed spheres are recommended for all existing thermoplastic traffic marking systems to date. Many states presently specify the use of moistureproofed spheres in their systems.

Specifications

To be properly controlled, the use of moistureproofed spheres should be specified by the state. ASTM and AASHTO standards define tests that can be easily performed in the laboratory to test for moistureproofing (*2*). No significant cost increase is incurred with the use of these spheres, which are produced by all bead manufacturers.

In order for a material to bend or refract light, it must be transparent and spherical. This also applies to glass spheres (*3*). Optimum bead embedment for 1.50 refractive index spheres is 60 percent (*4*). At 75 percent embedment, retroreflectivity qualities and subsequently luminance qualities diminish rapidly. Likewise at 50 percent embedment or less, retroreflectivity is diminished and amplified by the early removal of glass spheres. Further study will evaluate retroreflectivity as a function of glass sphere gradation and roundness.

Quality Control

None of the above specifications are totally helpful if proper control is not used. Application of glass spheres must be monitored to ensure good application techniques. Moistureproofed spheres provide uniform optimum embedment and retroreflectivity. This provides application uniformity.

It is hoped that the above tests and observations will help in the understanding of the mechanisms influencing good retroreflectivity and nighttime visibility. Test procedures and apparatus are detailed for use in evaluating the various systems and are available from the author upon request.

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In-Service Evaluation of Thermoplastic and Tape Pavement Markings Using a Portable Retroreflectometer

Robert W. Attaway

The use of portable retroreflectometers to evaluate the reflectivity of longitudinal markings such as center lines and edge lines, and special markings such as arrows and symbols is described. Two hand-held retroreflectometers (Ecolux and Mirolux 12) were used in these evaluations. Retroreflectometers measure reflectivity of pavement markings through manual placement on a small section of pavement marking. The Mirolux 12 unit was used for most readings because it is easier to operate and speeds data collection. Two evaluation procedures were used. The first procedure evaluated longitudinal pavement markings and involved selecting a section of pavement markings (usually 1 to 3 mi long), breaking that section down into zones, and taking reflectivity readings within those zones. The second procedure evaluated special markings and involved taking 5 to 10 reflectivity readings for each marking to determine the reflectivity level for each marking. The use of retroreflectometers with these evaluation procedures has proven to be a helpful tool for the traffic engineer in evaluating the nighttime performance of pavement markings. Thermoplastic has been used in North Carolina since the mid-1970s and has long been considered a durable pavement marking; however, objective evaluations of its reflective performance have never been made. An in-service evaluation was performed on nearly 350 mi of thermoplastic long lines using portable retroreflectometers. This evaluation indicated that thermoplastic is both a durable and a reflective pavement marking. Another evaluation raised questions about reflectivity during deck testing of preformed tapes. These questions led to an in-service evaluation of approximately 200 special markings. The evaluation indicated poor reflectivity performance of preformed tapes.

Pavement markings are one of the most important traffic control devices available to road users. These devices serve to regulate, warn, and guide the motorist in the use of highways and streets during the day and night. During darkness and adverse weather conditions the driver's performance depends to a great extent on the reflectivity of the marking and its contrast with the pavement surface (*1*).

Reflectivity is the single most important quality of a pavement marking. The fact that "the nighttime fatality rate in the United States is more than three times the daytime rate" (*2*) indicates the need to provide visual guidance to the road user at night. During the day there are many sources of delineation other than pavement markings to aid the driver in the operation of a vehicle. Some of these sources of delineation include the pavement shoulder, roadside foliage, longitudinal joints, the distant view of the road ahead, and roadside development. During the day drivers appear to rely on features in the distance rather than the road surface for delineation and guidance (*1*).

This is not to say that the daytime appearance of pavement markings is not important. Where markings are used as regulatory or warning devices, such as center lines and railroad symbols, they must be intact and visible to be effective. In most instances, however, if the nighttime reflectivity of a pavement marking is acceptable, its daytime appearance is also acceptable.

Pavement markings are reflective because small glass spheres (beads) are embedded in the marking material. These glass beads act as tiny reflectors that collect light from a vehicle's headlights and reflect a portion of it back to the driver's eyes. Figure 1 shows the vision geometry for a typical driver in a vehicle (*3*). Table 1 gives examples of vision geometry for different vehicle models.

This is a simplification of a complex system. The reflectivity of pavement markings is a function of a number of parameters, including (a) the manufacture and application of the marking material and beads and (b) the physical condition of the road, driver, and vehicle (Kalchbrenner, paper in this Record).

USE OF RETROREFLECTOMETERS

In the United States, the night visibility of pavement markings has been evaluated using human observers under nighttime conditions. Although this has been the "ideal" method for determining the true visual performance of pavement markings, human observation has many drawbacks, including the subjective nature of the evaluators and the need to conduct evaluations after normal working hours (Kalchbrenner, paper in this Record).

Retroreflectometers were developed to provide objective measurements of retroreflectivity of pavement markings and to allow daytime evaluation. A retroreflectometer attempts to simulate—on a reduced scale—the nighttime visibility conditions experienced by a driver. The device generally consists of a box that eliminates ambient light, a light source projected on a known area, a light sensor to measure retroreflected light, and provision for calibrating the instrument on a strong retroreflector.

There are two types of retroreflectometers in use today, coarse-geometry and fine-geometry instruments. A coarse-geometry instrument does not closely simulate the conditions experienced by the driver, whereas a fine-geometry instru-

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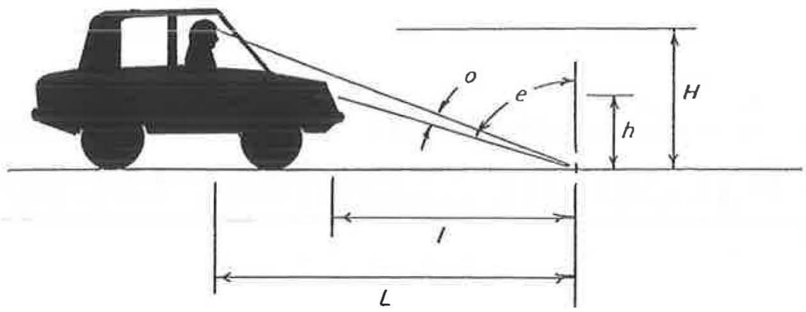

L: distance between driver's eyes and observation point
l: distance between headlights and observation point
H: height of driver's eyes
h: height of headlights
i: incidence angle
e: entrance angle
o: observation angle

FIGURE 1 Driver's vision in a vehicle.

TABLE 1 TYPICAL DRIVER'S VISION GEOMETRY FOR DIFFERENT VEHICLE MODELS (*3*)

Model	H	h	L	l	e	o
Dump Truck International Fleetstar 1910	6'8"	3'8"	64'	58'10"	86.4	2.3
Pickup Truck Dodge Custom 200	5'	3'	56'9"	50'	86.6	1.6
Sedan Chevrolet Impala	4'	2'3"	49'4"	42'	86.9	1.5

ment does. Some coarse-geometry machines would include the Michigan and Virginia Department of Transportation retroreflectometers. Fine-geometry machines would include the Ecolux, Erichsen, Optronik, and Mirolux (*4*).

Three different retroreflectometers were used in gathering reflectivity data in this research: Ecolux, Mirolux–Experimental, and Mirolux 12. These are all fine-geometry instruments, each having an entrance angle of 86.5 degrees and observation angles of 1, 1.5, and 1.5 degrees, respectively.

The Ecolux was used sparingly for in-service evaluations because of its weight and bulkiness. It is approximately 33 in. long and 10 in. wide, and weighs 20 lb. In addition, the Ecolux has a 21-lb battery pack and cable, which requires an additional person to handle it. The Ecolux has an analog scale, which is not difficult to read but does require judgment, and slows down the data-gathering process. The Ecolux reflectivity readings were converted to millicandelas per square meter per lux [(mcd $\cdot$ m^{-2})/lx] using a conversion equation specific to each machine.

In contrast, the Mirolux retroreflectometers are much lighter and easier to use, which resulted in more flexibility and productivity in data gathering. The two units used are hand-held and approximately 18 in. long and 6 in. wide, with an internal rechargeable 12-volt battery pack and a total weight of 14 lb. The read-out is digital and there is a digital battery voltage check. The readings from the two instruments were not the same, and correlation tests were required between the two and between the Ecolux and the two Mirolux retroreflectometers to determine their values. (All the values reported in this paper will be in millicandelas per square meter per lux as measured with an Ecolux retroreflectometer.) The two Mirolux machines were used to gather most of the in-service readings on pavement marking tapes and thermoplastics. In fact, without the Mirolux instruments, the number of in-service readings taken on these projects would not have been possible.

CORRELATION OF RETROREFLECTOMETERS

The two Mirolux retroreflectometers used were different generations of the same instrument. The first unit, the Mirolux–Experimental, was experimental and the second unit was one of the first production models of the Mirolux 12. The readings taken with the Mirolux–Experimental were "machine readings" and could not be converted. However, the Mirolux 12 provided direct readings in millicandelas per square meter per lux. Therefore a correlation test was developed to determine the relationship between these two units so machine readings taken with the Mirolux–Experimental could be converted into equivalent Mirolux 12 readings in the appropriate units.

The correlation test involved taking closely controlled reflectivity readings on different colored sheets of paper, pavement marking tape samples, and painted markings in the field with both instruments. Each instrument was carefully calibrated before readings were taken, using the internal retroreflector, and each instrument was checked using the external test panel of known reflectivity provided with each instrument.

Care was taken to measure the reflectivity in exactly the same location with each instrument. Sixteen different colored sheets of paper were checked, 19 tape samples, and 23 paint locations. These samples gave a good distribution of reflectivity levels from 0 to 989 as measured with the Mirolux 12 unit. Table 2 shows the readings as taken with each instrument for the paper, tape, and paint samples. A linear regression was performed on this data and a straight line was fitted to the data. Figure 2 shows a plot of this data and the equation for the line. A correlation analysis of these data and the straight line indicated that the data closely fit a straight line with a correlation coefficient of $R = 0.996$ and an $R^2 = 0.992$. This analysis enabled the machine readings taken with the Mirolux–Experimental to be converted into Mirolux 12 readings in millicandelas per square meter per lux.

Because the Mirolux 12 retroreflectometer was new, no information was available on how a reading related to the reflective performance of pavement markings, in other words, what the "good" and "bad" reflectivity readings were according to the Mirolux 12. Because some information was available on acceptable levels of reflectivity as measured with an Ecolux reflectometer, reflectivity readings taken with the Mirolux 12 were compared with those from an Ecolux. For this evaluation, an additional correlation test was performed between the Ecolux and Mirolux 12.

This correlation was performed in the same way as the previous correlation test, using paper and tape samples. The correlation test established that a relationship did exist between the readings of these two instruments. Figure 3 shows a plot of Ecolux versus Mirolux 12 readings taken on samples of paper and tape.

ESTABLISHING MINIMUM REFLECTIVITY VALUES

Determining the reflectivity of pavement markings has traditionally been a difficult task for traffic engineers in the United States because of the lack of recognized standards and equipment for making high-speed field evaluations. To the author's knowledge, no uniformly recognized U.S. standards for reflectivity of pavement markings exist.

Reflectivity standards for pavement marking materials are used in other countries, however—most notably in France and Germany. Each country has standards based on reflectivity readings taken with hand-held fine-geometry retroreflectometers manufactured in that country. The French have established an acceptable reflectivity value of 150 $(\mathrm{mcd{\cdot}m^{-2}})\mathrm{lx}$ as measured with an Ecolux retroreflectometer, and the Germans use a range of values from 150 to 70 SL based on traffic

TABLE 2 REFLECTIVITY READING FOR MIROLUX–EXPERIMENTAL VERSUS MIROLUX 12 ON PAPER, TAPE, AND PAINT SAMPLES

Test Sample	Mirolux Expmntl.	Mirolux 12	Test Samples	Mirolux Expmntl.	Mirolux 12
Paper Samples			Tape Samples		
White	28	32	Prismo Remov.-white	320	477
Ivory	28	30	-yellow	151	217
Very Lt. Blue	26	25	Prismo Inglf.-white	282	415
Lt. Blue	21	20	-yellow	188	276
Dark Blue	27	29	Paint - white	105	152
Lt. Pink	24	24	Paint - white	112	162
Pink	31	34	Paint - white	119	181
Lt. Yellow	31	33	Paint - white	112	175
Medium Yellow	27	28	Paint - white	112	170
Dark Yellow	27	27	Paint - white	108	159
Green	23	23	Paint - white	107	166
Orange	29	33	Paint - white	120	188
Red	26	26	Paint - white	170	243
Black I	9	2	Paint - white	113	156
Black II	11	5	Paint - yellow	72	106
Black plate	5	0	Paint - yellow	70	105
Test Plate-Old	207	309	Paint - yellow	82	135
Test Plate-New	305	453	Paint - yellow	74	110
MM Yellow	117	160	Paint - yellow	69	99
Prismo TR-white	531	777	Paint - yellow	73	108
MM white	220	322	Paint - yellow	51	85
Prismo TNR-white	561	813	Paint - yellow	34	64
-yellow	125	180	Paint - yellow	65	98
Swarolite -white	572	989	Paint - yellow	33	49
-yellow	458	650	Paint - yellow	43	61
Catatyle-yellow	237	351	Paint - yellow	43	61
3M Remov.-white	267	397			
-yellow	372	554			

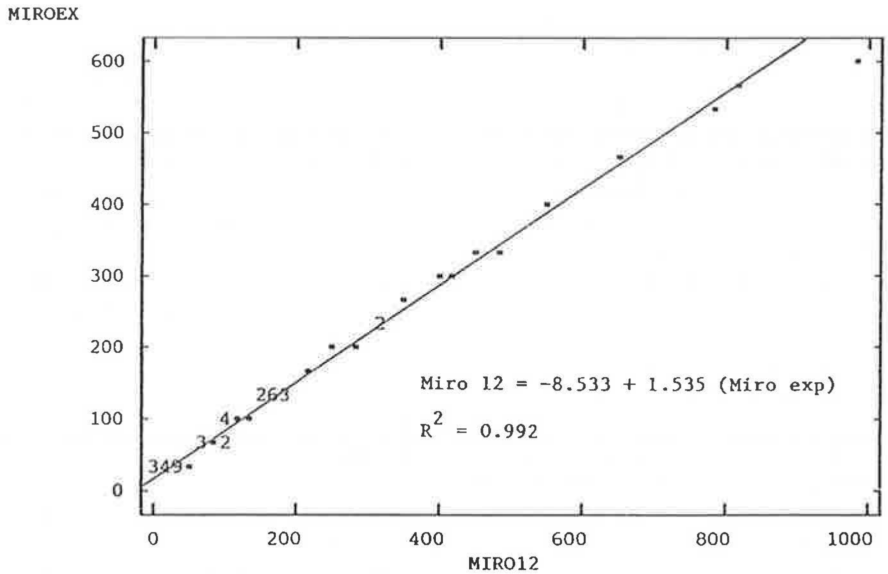

FIGURE 2 Plot of Mirolux–Experimental versus Mirolux 12 readings on laboratory and field samples.

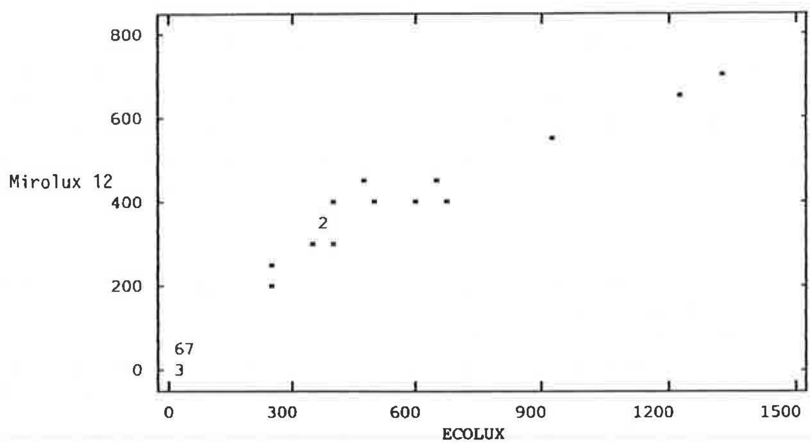

FIGURE 3 Plot of current Mirolux 12 versus Ecolux values on laboratory samples.

conditions as measured with a German-made retroreflectometer (*5*).

Although the United States does not currently have any minimum standards for reflectivity of pavement markings, some research has been performed and discussion is under way. One of the first such studies was an FHWA project by Allen et al. (*6*). Other research includes work done by Ethen and Woltman (*7*). NCHRP Project 4-16—Cost and Service Life of Pavement Markings—is also investigating minimum reflectivity values for markings.

This paper is not intended to establish the minimum acceptable standards for pavement markings in the United States. That task will take much research and discussion. However, if objective reflectivity readings are to be made, some value must be used for comparison of readings.

For this work, the research discussed in the previous paragraph was consulted for information on acceptable levels of reflectivity for pavement markings, and a limited panel evaluation was conducted by the Institute for Transportation Research and Education (ITRE). The panel evaluation was not conclusive. Its scope was limited because funds were limited and only five observers were used to evaluate 13 different markings with different amounts of wear. Even though the panel evaluation was not conclusive, it did aid in the review of previous work in this area and the decision was made to use a value of 100 $(mcd \cdot m^{-2})/lx$ as measured with an Ecolux retroreflectometer as a minimum acceptable value for reflectivity. Therefore all the readings presented in this paper will be Ecolux values.

Again it should be noted that the minimum value used here is not intended to be a standard. Other agencies may choose lower values for markings and some may want a higher level of service and require higher acceptable values. Individual criteria may be chosen until a national standard is adopted.

THERMOPLASTIC LONG-LINE IN-SERVICE EVALUATIONS

Thermoplastic has been used for pavement marking by the North Carolina Department of Transportation (NCDOT) since the mid-1970s. Numerous evaluations have been conducted by the NCDOT on the performance of thermoplastics during this period, but most have been informal or specific to a particular roadway project. The consensus among the NCDOT field traffic engineers has been that thermoplastic is a durable pavement marking material that performs well for several years when installed on asphalt pavements.

The specific number of years of acceptable service varied among engineers but generally ranged from 4 to 6 years. This performance was based primarily on periodic daytime and nighttime visual evaluations by field traffic engineers. Although these evaluations provide useful information on the overall trend in the performance of thermoplastic, problems with this procedure are evident, including lack of standard evaluation procedures, difficulty in measuring reflectivity (nighttime brightness), and the large number of engineers involved in these evaluations. North Carolina has 14 highway division offices with 14 field traffic engineers, all with different perceptions and opinions on the performance needs of pavement markings.

In 1985 the NCDOT Traffic Engineering Branch began a Highway Planning and Research Project on plastic pavement marking materials to determine how well thermoplastic and preformed tape marking materials were performing in North Carolina. The thermoplastic portion of the project focused on the performance of long-line thermoplastic pavement markings. Long-line markings include edge lines, lane lines, center lines, and barrier lines. A variety of projects representing each of North Carolina's geographic regions (mountains, piedmont, and coastal plain) and roadway types was evaluated. The roadway types evaluated included two-lane roadways, multilane undivided roadways, and divided (Interstate-type) roadways. The average daily traffic (ADT) varied from 5,000 to 20,000.

The purpose of the thermoplastic phase of this research project was to determine how well long-line thermoplastic pavement markings perform on different highways across North Carolina and to determine their expected life. To accomplish this purpose, a thorough review was conducted of other research in this area. NCDOT's thermoplastic project information was reviewed, evaluation procedures were developed and tested, and preliminary and final field visits were made to conduct the evaluations.

Research Review

The review covered previous research reports concerning the evaluation and performance of thermoplastic and other pavement marking materials. In total more than two dozen reports and documents were used.

An inventory review was made of all available NCDOT records on thermoplastic pavement marking projects in North Carolina. This review revealed special marking projects dating back to 1976 and long-line projects back to 1978. A complete list of these potential projects was assembled for further investigation and preliminary field visits. The projects were assembled by NCDOT Highway Division, date, county, and project number.

Preliminary Field Visits

A copy of the identified thermoplastic projects in each highway division was sent to each respective NCDOT Highway Division traffic engineer for review and change. Field visits were then arranged with each NCDOT Highway Division traffic engineer to discuss thermoplastic markings, to review the data, and to make field visits to thermoplastic sites.

These visits proved to be a valuable contribution to this project in providing

- Updates and corrections to the list of projects,
- First-hand information about this research project for field personnel,
- A means of learning about the experiences, problems, and opinions of practicing field personnel on the subject of pavement markings, and
- A preliminary review of most of the thermoplastic projects still in service.

These preliminary visits also allowed for a discussion of evaluation procedures and some experimentation with different approaches. A number of nighttime evaluations were conducted on thermoplastic during these visits, first to observe at first hand how well markings were performing at night and second to try different evaluation approaches.

During these visits with the 14 NCDOT Highway Division traffic engineers, one point was repeated—thermoplastic is a durable and effective pavement marking material when used on asphalt. When asked about lifespan of this material, individuals offered responses ranging from 4 years to "until it's resurfaced." There were some negative comments on the performance of the material in heavy snowplow areas and on concrete. In fact, the comments on thermoplastic's performance on concrete were as uniformly negative as the comments on its performance on asphalt were positive. In the final evaluation, only one section of concrete with thermoplastic was found that had not previously been repainted.

Reflectivity Readings and Field Evaluations

Upon conclusion of the preliminary visits, a list of sites for further evaluation was assembled. Of the 1,456 mi of thermoplastic originally identified from traffic engineering records, approximately 800 mi was found to be in place in the field. The remainder of the sections were either resurfaced or painted over. North Carolina had two major resurfacing programs in the 1980s, and many of the sections of Interstate, primary, and urban roadways where thermoplastic was installed in the late 1970s and early 1980s were covered in these programs. Resurfacing is unavoidable in highway work, and despite careful planning pavements can deteriorate more quickly than anticipated. Durable pavement markings are placed on high-volume routes, which are more susceptible to pavement failure and receive a higher level of maintenance.

Resurfacing accounted for the majority of the loss of thermoplastic projects. Over 540 mi of the original projects identified had been resurfaced. In most cases, no information on the condition of these sections at the time of resurfacing was available and in other cases information was not readily available. An evaluation could not be made without information on the condition of the marking at the time of resurfacing. Of the remaining 120 mi of projects, approximately 60 mi had been placed on concrete and repainted within the first 3 years by maintenance forces. Loss of material due to chipping was the major failure of thermoplastic on concrete, according to field contacts. The other 60 mi of thermoplastic placed on asphalt had also been repainted by maintenance forces. Lane lines were the most frequent markings repainted on projects installed in 1981 or before.

Procedures used to evaluate a pavement marking's performance vary substantially among divisions. This was concluded from the field visits with the 14 NCDOT Highway Division traffic engineers and in previous studies with traffic service operations. Some divisions have higher levels of service for pavement markings than others. Without more definitive information on the marking's performance at the time of repainting, its condition at that time remains unknown.

A few general comments can be made concerning these repainted sections of thermoplastic. First, a higher proportion (70 percent) of the sections that had been repainted were in the mountainous and northern piedmont divisions of the state. These divisions experience higher amounts of snow and ice and, according to field observations and contact with field representatives, damage from snowplows. Snow and ice control contributed significantly to a loss of thermoplastic and subsequent repainting.

Second, certain divisions had repainted all thermoplastic markings that were installed before a particular date, possibly indicating a higher level of service than other divisions. This conclusion was based partly on a number of spot reflectivity readings taken on portions of lines not completely retraced, indicating that some of these repainted markings were still providing acceptable levels of reflectivity.

Evaluation Procedures

A number of evaluation procedures were examined for use in this project. Some of these ideas were tested in the preliminary field visits. As in the other evaluations, the three most important factors concerning a pavement marking's performance are appearance, durability, and reflectivity. The appearance and durability evaluations were used in other research projects on thermoplastic and were generally conducted in the daytime from a moving vehicle or from the shoulder of the road. Evaluation of reflectivity was more difficult because it required nighttime visits to each section and then a subjective rating of the condition.

A series of nighttime reviews of thermoplastic markings indicated that useful information could be determined about the reflectivity of pavement marking in this manner. A number of serious problems were encountered with this method, including the need to conduct evaluations after normal working hours and to control evaluation conditions such as oncoming headlights, peripheral lighting, time, and season. Trial measurements were made on pavement markings using an Ecolux retroreflectometer in an attempt to eliminate some of these problems.

The Ecolux eliminated many of the problems encountered in the subjective nighttime evaluations but had problems of its own. The major problem with the Ecolux was that it was not easy to operate under traffic conditions. The Ecolux requires two people to operate—one to hold and read the machine and the other to carry the battery pack and cable. Operating the machine was a laborious task, making extensive use of an Ecolux impossible.

Shortly after these trials with the Ecolux, ITRE was provided with a Mirolux retroreflectometer for testing. The Mirolux solved the task-oriented problems of the Ecolux. The Mirolux was small, light, easy to operate, and proved to be reliable after repeated use. Testing also revealed a direct relationship between its readings and those of the Ecolux. Finally, an objective means of evaluating the reflective quality of pavement markings was available for use in the field. Armed with this new evaluation tool, ITRE began an extensive field evaluation of thermoplastic.

Before field visits were made, the evaluation procedures were finalized and tested. The three areas of evaluation were appearance, durability, and reflectivity.

Appearance

Markings were rated as acceptable or unacceptable using the following definitions. This was an overall rating for the entire section.

Acceptable The intent of the pavement marking to guide, warn, or regulate is clear to the driver from a vehicle operating at normal highway speeds during daylight hours. This is the complete impression conveyed by the marking, including appropriate color (white or yellow).

Unacceptable The intent of the marking to guide, warn, or regulate is not clear to a driver from a vehicle operating at normal highway speeds during daylight hours.

Durability

The durability was evaluated according to the percentage of the material remaining on the pavement. This evaluation was made at the same locations as the reflectivity readings, directly over the marking, using the unaided human eye. Only surface area covered was evaluated. Thickness measurements are not practical without a special instrument, which was unavailable at the time of this evaluation.

Reflectivity

The reflectivity was evaluated using a Mirolux 12 retroreflectometer. Each thermoplastic project was evaluated by taking three sets of reflectivity readings, generally in the first third, the middle third, and the final third of the project. A set of reflectivity readings consisted of at least 18 readings over six skip lines (three readings per skip) and 18 readings distributed evenly over 300 ft of continuous line. This was done for each line (edge line, center line, lane line, or barrier line) in both directions. This procedure was followed for sections approximately 3 mi long. For sections over 3 mi, an additional set of readings was taken at each additional segment up to 3 mi. Readings were taken on randomly selected tangent sections within each of the three zones to allow for comparison between sections and for safety.

The following procedure was used to determine the reflectivity performance for a marking on a project. On a given project, all reflectivity readings for a given marking (i.e., edge line, lane line, or center line) were totaled and then divided by the number of readings to get an average reflectivity value. This reflectivity value could then be used to determine whether the edge line, lane line, or center line met minimum acceptable reflectivity standards.

Site Selection

After the preliminary visits, approximately 800 mi of thermoplastic on 146 pavement marking projects was identified. The decision was made to evaluate thermoplastic sites in all of the 14 NCDOT Highway Divisions. This would give a good distribution of geographic and climate types. Approximately one week was assigned to each division for data collection. The approach was to check as many sites as possible within each division. In 5 of the 14 NCDOT Highway Divisions, all thermoplastic sites were evaluated. In the remaining 9 divisions, approximately half of the thermoplastic sites within each division were evaluated. An attempt was made to evaluate each type of roadway (two-lane, multilane undivided, and multilane divided) within each division.

Results of Thermoplastic In-Service Evaluations

In the final evaluation, approximately 350 road mi of thermoplastic on 60 different projects was evaluated. These projects covered the period from 1979 to 1986. Sections were grouped according to the type of roadway:

- Two-lane,
- Multilane divided, and
- Multilane undivided.

Appearance

With the exception of a few sections of roadway in the coastal region of the state, the appearance evaluation criterion was not the controlling factor in determining the performance of a section of thermoplastic. On these particular sections, the pavement was 7 to 8 years old and contained highly polished aggregate, which gave a pavement color similar to that of concrete. The appearance of these particular markings varied according to the direction of travel and the intensity of sunlight. The end result was that the surface did not provide sufficient contrast with the pavement markings to be effective during all daytime conditions.

Contrast

This study did not have the instrumentation to measure contrast objectively, so it was evaluated subjectively. It is interesting to note that these particular sections had good nighttime reflectivity. Failure due to daytime performance was the exception rather than the rule in these evaluations. In most it was the reflectivity evaluation that controlled the marking performance in this project.

Reflectivity

The importance of reflectivity in the performance of a pavement marking has always been recognized, but an objective evaluation of that performance has been a difficult task. With the use of the Mirolux 12 retroreflectometer, an objective evaluation of reflectivity of thermoplastic projects in North Carolina was performed and shows why thermoplastic pavement markings are so favored by NCDOT field personnel. Results showed that white thermoplastic provided 6 and 8 years of acceptable performance on all types of highway facilities. Yellow markings had not provided comparable results. Less than half of the projects had demonstrated acceptable service after 3 years. The evaluation is based on an acceptable reflectivity value of 100 SL as measured with an Ecolux retroreflectometer. The same value was used on both white and yellow materials. (Yellow is inherently less reflective than white and this may justify a lower level of reflectivity. Experiments in this area were outside the scope of this project, however.) Table 3 gives a summary of the reflectivity performance of the thermoplastic projects evaluated under this project.

Table 3 also shows the number of thermoplastic sections and mileage by year and the percentage (by mileage) that were acceptable (≥ 100) and unacceptable (< 100) as measured with an Ecolux. Table 3 shows that nearly all white thermoplastic markings were found to be acceptable regardless of age. In fact, of the more than 60 projects evaluated, only 2 were found to have white thermoplastic unacceptable because of reflectivity. The performance of yellow thermoplastic was not as good, with more than half of the yellow material failing after less than 6 years of service in both the edge line and center line conditions.

Durability

Information on durability (percent of material remaining) was gathered at the same time as the appearance and reflectivity data. Reflectivity or appearance, or both, were the factors that controlled the failure in all the sections evaluated. However, durability was an integral part of the reason the markings failed. If a marking material loosens from the pavement surface, no reflective material is present to delineate lines.

In the analysis of field data, white thermoplastic markings were rated acceptable according to appearance; reflectivity

TABLE 3 SUMMARY OF THERMOPLASTIC REFLECTIVITY READINGS

TWO-LANE ROADWAYS

YEAR	NUMBER OF SECTIONS	MILES	WHITE EDGE LINE READINGS %>100	WHITE EDGE LINE READINGS %<100	YELLOW CENTER LINE READINGS %>100	YELLOW CENTER LINE READINGS %<100
1979	3	18.0	100	0	44	56
1981	11	38.5	100	0	61	39
1983	1	7.0	100	0	100	0
1984	3	33.0	100	0	88	12

MULTI-LANE DIVIDED ROADWAYS

YEAR	NUMBER OF SECTIONS	MILES	WHITE EDGE LINE %>100	WHITE EDGE LINE %<100	WHITE LANE LINE %>100	WHITE LANE LINE %<100	YELLOW EDGE LINE %>100	YELLOW EDGE LINE %<100
1979	2	14.0	100	0	100	0	43	57
1981	17	84.5	95	4	98	2	39	61
1982	5	33.0	100	0	100	0	36	64
1983	5	32.0	100	0	100	0	37	63
1984	2	9.0	100	0	100	0	100	0
1985	3	21.0	100	0	100	0	100	0
1986	2	17.0	100	0	100	0	100	0

MULTI-LANE UNDIVIDED ROADWAYS

YEAR	NUMBER OF SECTIONS	MILES	WHITE EDGE LINE %>100	WHITE EDGE LINE %<100	WHITE LANE LINE %>100	WHITE LANE LINE %<100	YELLOW BARRIER LINE %>100	YELLOW BARRIER LINE %<100
1981	3	12.0	100	0	100	0	0	100
1985	3	31.0	100	0	90	10	90	10
1986	1	2.0	100	0	100	0	0	100

NOTE: The values shown are Ecolux values [(mcd · m^{-2})/lx].

generally consisted of more than 70 percent of the material remaining. When less than 70 percent of the marking was present, reflectivity was generally at or below the acceptable level and appearance was unacceptable or marginal. For more than 90 percent of the markings evaluated in this project, 75 to 90 percent of the surface area was covered with material. These durability percentages were recorded at the locations of reflectivity readings. (There was not always a relationship between the percentage of material remaining and reflectivity of yellow thermoplastic markings.) As discussed before, yellow materials are inherently less reflective than white materials and, in many cases, more than 70 percent of the marking was in place and the reflectivity level was still below the minimum acceptable level.

Performance on Concrete

Another problem discovered in this evaluation was the poor performance of thermoplastic on concrete. Only one concrete project of approximately 5 mi had thermoplastic markings that had not been repainted. This particular section was multilane divided, with >20,000 ADT, and was approximately 4 years old. It was in excellent condition. All other sections exhibited extensive chipping and loss of material. According to field personnel, each of these sections was restriped with paint within 3 to 4 years after installation. A variety of explanations was presented for these failures, including poor cleaning of curing compound before placement, excessive loss due to snowplow activity, and a lack of sufficient bond to the concrete.

In conclusion, thermoplastic is a durable and reflective pavement marking material when used as a long-line marking on multilane and two-lane roadways with wide lanes. In North Carolina, white thermoplastic is providing acceptable appearance and reflectivity for 6 and 8 years when installed on high-type roadways with traffic volumes up to 20,000 ADT. Yellow thermoplastic is providing a marking life of 3 years under similar roadway conditions. These results are based on an extensive evaluation of reflectivity using objective readings from a portable retroreflectometer.

LONG-LIFE TAPE IN-SERVICE EVALUATIONS

A second part of the 1985 North Carolina Highway Planning and Research Project was an evaluation of pavement marking tapes. This evaluation included test deck testing and in-service testing of different pavement marking tape products. This research covered a 2-year period and provided approximately 20 months of observation time measured from the time of installation. Because long-life tapes are used primarily for special markings such as arrows, school and railroad symbols, stopbars, and crosswalks, one of the in-service tests was to install samples of each long-life tape product in special marking locations. The site selected for this test was in Oxford, N.C. (population 8,000). Oxford's main street was chosen because of on-street parking, signalized intersections, curb and gutter, asphalt pavement, and an ADT of 6,500 to 11,000. The roadway has two lanes with short left-turn lanes at signalized intersections.

Oxford Crosswalk and Stop Bar Tests

Five different long-life pavement marking tape products were installed at the site. At least three samples (approximately 10 ft long) of each of the five products were installed as crosswalks or stop bars, or both. Visual evaluations were made of these samples at 1, 6, 12, and 20 months of service. During these evaluations the markings were analyzed for appearance (acceptable or unacceptable) and durability (percentage of marking remaining). During this 20-month period 5 of the 17 total samples installed failed appearance or durability criteria. At the end of 20 months of service, reflectivity was also evaluated. Five reflectivity readings were taken on each section of crosswalk or stop bar with an Ecolux retroreflectometer. These five readings were then averaged to get a reflectivity value for each sample. Table 4 lists the individual crosswalk and stop bar markings and their average reflectivity readings. Using 100 $(\mathrm{mcd\cdot m^{-2}})/\mathrm{lx}$ as measured with an Ecolux as an acceptable reflectivity level, Table 4 shows that these markings performed poorly in reflectivity. In fact, none of the materials tested met the acceptable level of reflectivity. This test raised questions about the reflectivity performance of pavement marking tapes when used as special markings, which is where most are used.

To determine whether these poor reflectivity results were indicative of the performance of pavement marking tapes in general, an expanded in-service evaluation was conducted on special markings installed by NCDOT maintenance forces in 1985—the same time that test samples were installed. The only long-life pavement marking tape approved for use by NCDOT at that time was 3M Company's Stamark Pliant Polymer® 5730 tape; therefore, it was the only material evaluated.

3M 5730 Long-Life Tape in Special Markings

Approximately 30 projects and locations were identified for evaluation with varying ADTs and traffic conditions. These locations were all in Harnett and Cumberland counties, which are on the coastal plain of North Carolina. A total of 194 individual markings were evaluated. Installation dates were known for all of these markings and all were installed in the summer of 1985. In this evaluation the same criteria for appearance, durability, and reflectivity were used as in the other in-service evaluations.

A variety of special marking types was evaluated, including all types of arrows, railroad crossing symbols, and stop bars at railroad crossings. As in the Oxford study, most of the markings evaluated were acceptable in appearance and durability (approximately 5 percent of the markings evaluated failed due to appearance and durability). Again, reflectivity was the weak point in the performance of these markings. Reflectivity was evaluated using a Mirolux 12 retroreflectometer taking 5 readings per arrow and 10 readings for each railroad symbol. These readings were then averaged to get a reflectivity reading for each marking. Using the 100 SL—as measured with an Ecolux retroreflectometer—as the acceptable value for reflectivity, 74 percent of the markings checked were unacceptable because of reflectivity after only 2 years of service.

Table 5 shows the reflectivity performance by showing the number of special markings evaluated by type and the percentage above and below the 100 SL as measured with an Ecolux retroreflectometer. As the table shows, the only type of special marking that performed reasonably well was the through arrow; however, only 36 percent of those checked were above the 100 SL.

Even if reflectivity values below the established level of 100—as measured with an Ecolux—are used, a significant percentage of the special markings would be unacceptable. Table 6 gives the percentages of each type of special marking that had reflectivity values below 100, 90, 80, and 70 SL.

Because of poor reflectivity readings, it is questionable whether long-life pavement marking tape should be used in special markings except under lighted conditions. Of course, this recommendation is based on an acceptable value of 100

TABLE 4 OXFORD CROSSWALK AND STOP BAR TESTING REFLECTIVITY READINGS AT 20 MONTHS

	Crosswk Locat.1	Crosswk Locat.2	Crosswk Locat.3	StopBar Left Turn 1	StopBar Left Turn 2
A - SEIBULITE "MM"	69	65	67	--	--
B - 3M "STAMARK" 5730	66	59	66	92	--
C - CATA-TILE	58	--	--	80	69
D - PRISMO 60MM	61	56	--	--	75
E - PRISMO 90MM	60	56	--	88	74

NOTE: Reflectivity readings were taken with an Ecolux retroreflectometer. The listed readings are an average of 5 readings per test sample.

A dash (--) indicates that no material was installed at that location.

TABLE 5 REFLECTIVITY READINGS FOR 3M STAMARK® 5730 TAPE USED AS SPECIAL MARKINGS

Special Marking Type	Markings Evaluated	Acceptable Readings≥100 Number	Acceptable Readings≥100 %	Unacceptable Readings <100 Number	Unacceptable Readings <100 %
TURN ARROWS	36	8	22%	28	78%
THROUGH ARROWS	66	24	36%	42	64%
COMBINATION ARROWS	39	1	3%	38	97%
RXR-RAILROAD SYMBOL	33	7	21%	26	79%
--- STOP BAR AT RAILROAD CROSSING	12	1	8%	11	92%
TOTALS	186	41	26%	145	74%

NOTE: The values shown are ecolux values [(mcd · m^{-2})/lx].

Reflectivity readings for all arrows and stop bars at railroad tracks are based on 5 readings for each marking. Railroad symbols are based on 10 readings each.

TABLE 6 PERCENTAGES OF UNACCEPTABLE SPECIAL MARKINGS AT LOWER REFLECTIVITY VALUES

Special Marking Tape	No. of Markings Evaluated	Percentage of Reflectivity Readings (Ecolux) <100	<90	<80	<70
Turn Arrows	36	78%	78%	38%	14%
Through Arrows	66	64	41	20	9
Combination Arrows	39	97	95	62	10
RxR - Railroad Symbols	33	79	70	30	6
---Stop Bar at Railroad Crossing	12	92	75	42	17
TOTALS:	186	74%	64%	37%	14%

(mcd·m^{-2})/lx as measured with an Ecolux retroreflectometer, and lower acceptable values may result in different conclusions. The purpose is to show that a portable retroreflectometer can be used to evaluate the reflectivity performance of pavement markings. Reflectivity has long been recognized as an important part of a pavement marking's performance. Nevertheless, it has been difficult to objectively evaluate the reflective performance of pavement markings because of the subjective nature of nighttime visual evaluations. A portable retroreflectometer makes such an objective analysis possible.

SUMMARY AND CONCLUSIONS

The evaluation of pavement markings for reflectivity under in-service conditions is a difficult task for the traffic engineer. The lack of reflectivity standards and high-speed equipment for making such measurements has resulted in fewer in-service evaluations of pavement marking reflectivity.

Until such standards and high-speed equipment are developed, highway agencies will have to rely on their own capabilities in the evaluation of pavement markings. These might include nighttime visual evaluations and use of the portable handheld retroreflectometers currently available. Highway agencies could establish their own acceptable levels of reflectivity.

A procedure has been presented for using a portable retroreflectometer (Mirolux 12) to evaluate the reflectivity of long lines and special markings. A procedure was presented for taking reflectivity readings under actual field conditions on both types of markings. Information was also provided on the selection of an acceptable level of reflectivity. Two examples were presented in the application of this evaluation procedure, one looking at longline thermoplastic markings and another looking at preformed tape used as special markings.

The thermoplastic evaluation shows that white thermoplastic has been providing acceptable service under a wide range of traffic conditions in North Carolina for 6 and 8 years. Yellow thermoplastic is providing at least 3 years of service and longer in some cases. The special marking evaluation of preformed tape indicated that these materials were not providing an acceptable level of reflectivity after only 2 years of service.

These evaluations were based on an acceptable level of reflectivity of 100 (mcd·m^{-2})/lx as measured with an Ecolux retroreflectometer. This level of reflectivity was based on previous studies in the area of pavement marking reflectivity and a limited panel evaluation.

These examples show that useful information can be derived from this type of evaluation to help the traffic engineer in making decisions concerning pavement marking practices.

ACKNOWLEDGMENT

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Research on Raised Pavement Markers

JOHN T. TIELKING AND JAMES S. NOEL

The results of a study directed toward increasing the retention time of raised pavement markers on asphalt concrete pavement are described. Retention time is believed to be largely limited by fatigue strength of the pavement surface. The kinematics of a tire striking a raised pavement marker were studied by high-speed photography to guide development of a laboratory apparatus that simulates pavement fatigue loading by a tire rolling over a marker. A laboratory investigation of the effect of adhesive type on fatigue strength of asphalt pavement was made. It was found that bituminous adhesive is distinctly superior to epoxy adhesive on new asphalt surfaces. The distinction between bituminous and epoxy adhesive is less pronounced on stiffer (seasoned) pavements. An instrumented pavement marker to record the number of tire hits was also developed during the study. The circuitry is described, and hit count data obtained with instrumented lane line markers are reported. These data, together with the laboratory fatigue data, permit prediction of retention time for a particular application. The paper concludes with an analysis of data from several adhesive test sections on state highways. Data from one test section show that it is possible to replace a missing marker with a new marker installed directly on the pavement failure spot instead of alongside it.

The use of raised pavement markers (RPMs) to supplement highway delineations has been well received by road users. At night the reflective marker enhances lane delineation to give the driver an additional feeling of security. Day and night, by a series of tire-marker impacts, the RPM reminds the driver to check his lane position.

RPMs are far more prevalent in southern states than in the north where snow removal equipment restricts their use. Snowplowable markers are available, but their installed cost is high ($15 to $20 per marker). In snow-free areas, the markers are easily attached by adhesive bonding to the pavement surface. The installed cost is then in the neighborhood of $2 per marker. Several million raised markers are currently in service on Texas highways.

Although the RPMs generally perform well and there are no plans to discontinue their use, two distinct maintenance problems—reflectivity and retention—have arisen. The reflectivity problem has been addressed in earlier research (*1*) at the Texas Transportation Institute (TTI), and is the subject of another ongoing project at TTI. The study described in this paper (*2*) focuses on the retention problem.

It has long been recognized that RPMs are generally lost by a failure in the surface of the pavement itself, instead of by failure of the adhesive or breakup of the marker. Missing markers are usually found by the roadside, intact and with a "divot" of pavement attached to the base. They can become a road hazard. A displaced marker thrown through a windshield by a mower resulted in a lawsuit in a Texas highway district.

A distinct shape effect on retention has been observed (*3*). On all pavements, round ceramic markers (traffic buttons) are retained much better than the square-base plastic markers. The retention problem is more serious on asphalt concrete pavement (ACP) than on portland cement concrete (PCC) pavement. Surveys of square-base markers on ACP have found loss rates of up to 80 percent in 18 months. In Texas the loss rates appear to increase during the spring and fall. If the markers survive 18 months, a service life of 3 to 5 years can be expected.

Until recently, all markers were bonded to the pavement surface with a two-part epoxy. Bituminous adhesive, which must be heated before use, is a primary substitute for the various types of epoxy that have been used. A number of highway districts in Texas have laid test sections with bituminous adhesive, some as early as 1983. Inspection reports indicate generally superior performance; in some cases the retention percentage of generally superior performance; in some cases the retention percentage of markers attached with bitumen was twice as high as that of the markers attached with epoxy. Some disadvantages have appeared also; there have been reports of markers sliding and submerging (apparently because of the reaction of bituminous adhesive with bituminous concrete).

RESEARCH METHODOLOGY

The problem of pavement marker retention was approached by studying the fatigue characteristics of asphalt pavements under the repetitive loads imparted by tires striking a pavement marker. The hypothesis is that the pavement failure when a marker comes loose is a fatigue failure. Contrary to an abrupt fracture of the pavement, the fatigue failure accumulates during a long sequence of repetitive load cycles. A physical indication that marker retention failure is a fatigue failure is the absence of ductile deformation at the asphalt failure site where a marker has been lost. An important feature of the fatigue hypothesis is the possibility of performing accelerated fatigue experiments in the laboratory. Since fatigue is a brittle-type failure, there is very little time dependence involved. The failure depends on the number of load cycles (tire hits) and is relatively insensitive to the frequency of the

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loading. The insensitivity to load frequency makes accelerated testing possible.

In order to design a laboratory fatigue experiment, it was necessary to determine the kinematics of a tire striking a marker. This was relatively easy to do with high-speed photography, and the findings are described here. It would also be desirable to determine the marker impact force, namely, the force impulse vector transmitted by the marker to the pavement. For this study, the impact force was estimated from tire ride data provided by a tire company.

To relate the laboratory fatigue data to marker retention time on a highway, a hit-counting pavement marker was developed. The hit count divided by the daily traffic passing the marker allows the laboratory fatigue cycles-to-failure data to be related to highway marker retention time.

Because the 4- by 4-in. square base marker presents a much greater retention problem than the 4-in. diameter round marker, the fatigue study was restricted to square base RPMs.

TIRE-MARKER IMPACT

A tire traveling at 50 mph traverses the 4-in. span of a pavement marker in 4.5 msec. This duration of traverse is too short to visually determine the effect of a marker on the path of a tire. To study the tire-marker impact event, a high-speed motion picture camera was focused on a 4- by 4-in. RPM on an asphalt test track at the TTI Proving Ground. To avoid enveloping the marker, a small car with P165/80R13 size tires was used. After a few practice runs, it was possible to hit the marker at highway speed with some regularity. The camera was operated at a filming rate of 500 frames/sec. This gave about four pictures (frames) of the tire traversing the marker at 60 mph. A typical sequence of four frames taken at 60 mph is shown in Figure 1. Regardless of speed (up to 60 mph) or inflation pressure (25, 35, or 45 psi), the tire is seen to

- Hit the marker on the upper third of the sloping face,
- Roll over the entire top face, and
- Make contact with the sloping face on the far side of the marker.

The high-speed photography showed that the tire always stayed in contact with the top surface of the marker instead of bouncing off it as was believed likely to occur with a small high-pressure tire. The passenger car tire was studied because laboratory data (*4*) indicate that a truck tire always envelops a marker. Photographic evidence for a truck tire was not obtained in this study. The assumption that any tire striking a marker will stay in contact during the traverse was used in the design of a laboratory experiment to measure the fatigue strength of asphalt experiencing tire-to-marker impact loads.

HIT-COUNTING MARKER

Using laboratory fatigue data to predict pavement marker retention time requires knowledge of hit rates for markers in

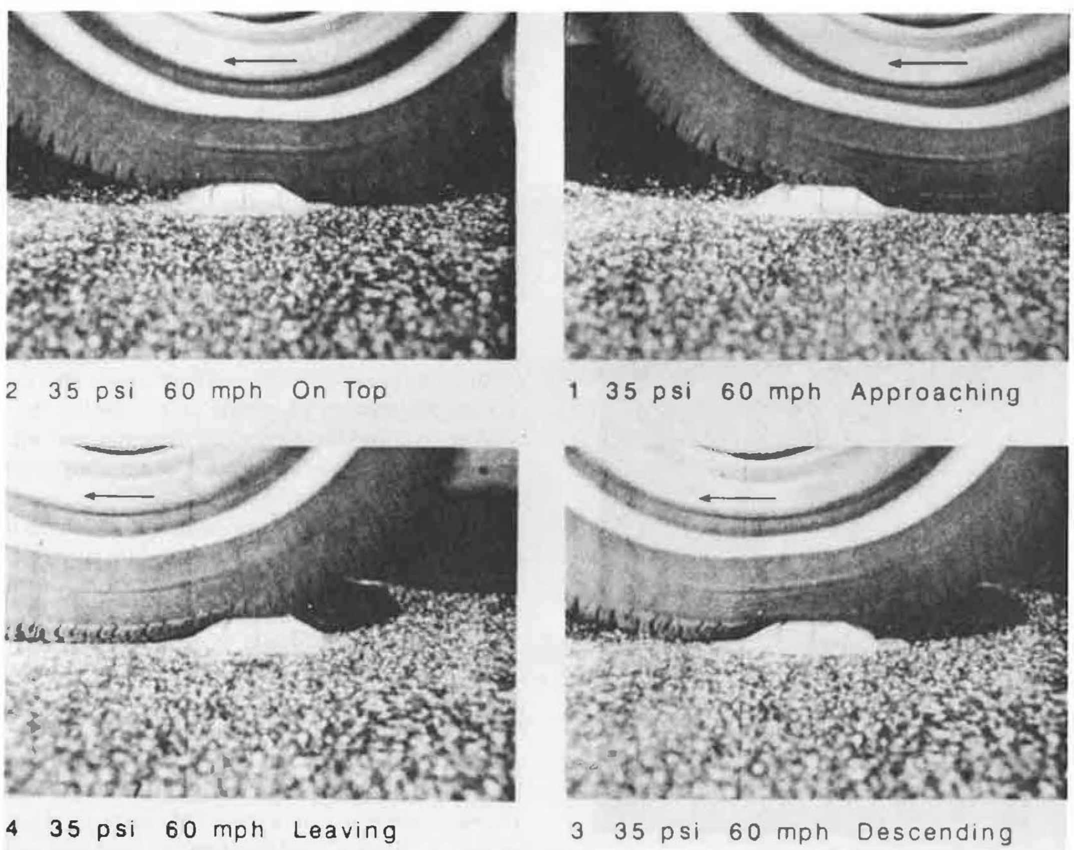

FIGURE 1 Tire-marker impact sequence.

various highway applications. Marker hit rates have been previously estimated by TTI researchers by visual counting during daylight hours. Because it is very difficult to detect when a tire strikes a marker and markers may be hit more often at night than during the day, a means of automatic hit-count data collection is needed.

Instrumentation

A 4- by 4-in. pavement marker was instrumented to record the number of tire hits received. A cavity was milled in the base of the marker to hold a piezoelectric crystal sensor (XDCR), electronic components, and two small 3-volt lithium batteries. A rectangular side opening was cut to hold a 20-pin female connector, flush mounted with epoxy. All of the components were packed in the marker with epoxy adhesive, which has proved to be a good insulating compound. This is the same epoxy that is used to attach the marker to the pavement. Figure 2 shows an instrumented (hit-counting) lane line marker installed on a pavement. It is completely self-contained and cannot be distinguished from other markers by a vehicle driver.

A hit count is read by plugging a display unit into the connector on the side of the marker. This takes about 3 to 4 sec at the marker location on the highway. The count is held in the digital display until recorded and the display is cleared. A block schematic diagram of the circuitry in both the marker and the display unit is shown in Figure 3.

The lithium batteries power the instrumented marker for about 3 months. The hit-count data reported below were taken near College Station, Texas, during May 1987. There were some very hot days and some rainy days, but weather effects on the instrumented marker could not be detected. The counter was checked by driving a car over it at highway speed.

FIGURE 2 Hit-counting pavement marker.

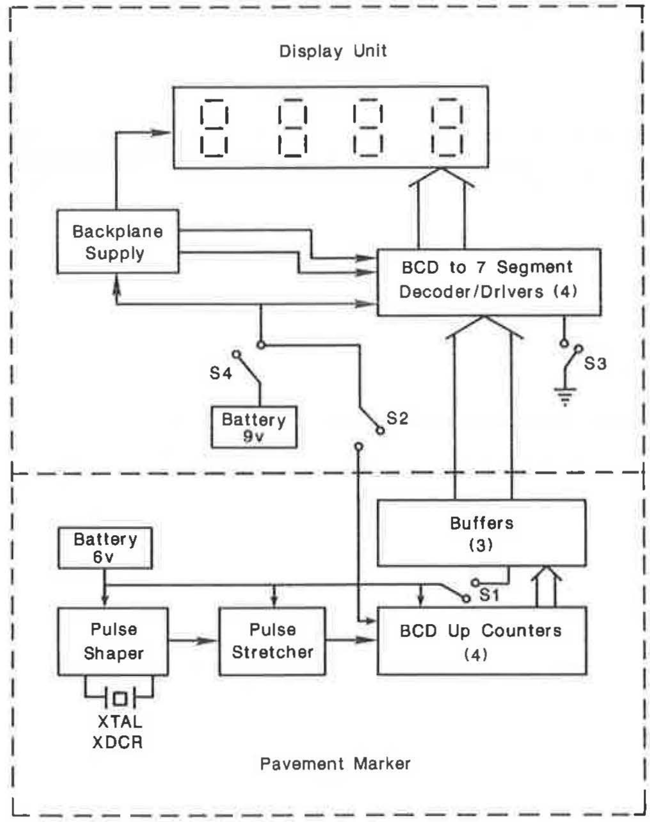

FIGURE 3 Block schematic of pavement marker and display unit electronics.

Highway Hit-Count Data

Two instrumented lane line hit counters were placed on a straight section of eastbound FM 60 near College Station. FM 60 is a four-lane divided highway with a posted speed limit of 50 mph. One counter was installed at location A (see Figure 4) and the other was installed at location B (not shown), 560 ft east of location A. There were no driveways or other means of access to the highway within the test section.

During the week of May 18, 1987, a traffic counter tube was placed across the two eastbound lanes about 100 ft east of hit-counting marker A (see Figure 4). The traffic counter records the number of vehicle axles crossing the tube, which is taken to equal the number of tires that may hit a pavement marker. Table 1 gives the traffic count data and the daily hit counts recorded by the instrumented markers at locations A and B. An indication of the reliability of these data is given in Figure 5, which shows a straight line fit to the cumulative marker hit and axle-count data. The slope of this line is the hit incidence factor (hit rate), here found to be 0.0058 hit/axle for the 1-week period. Assuming two axles per vehicle, these data imply that 1.16 percent of the traffic will strike a particular lane line marker in this test section. It was estimated in earlier resarch (*3*) that lane line markers are hit three times as often as center line markers on four-lane divided highways. Highway geometrics are clearly a factor in marker hit rates. The data reported here were taken on a no-access straight section of highway, and thus may be considered a lower bound on lane line hit rates for this highway.

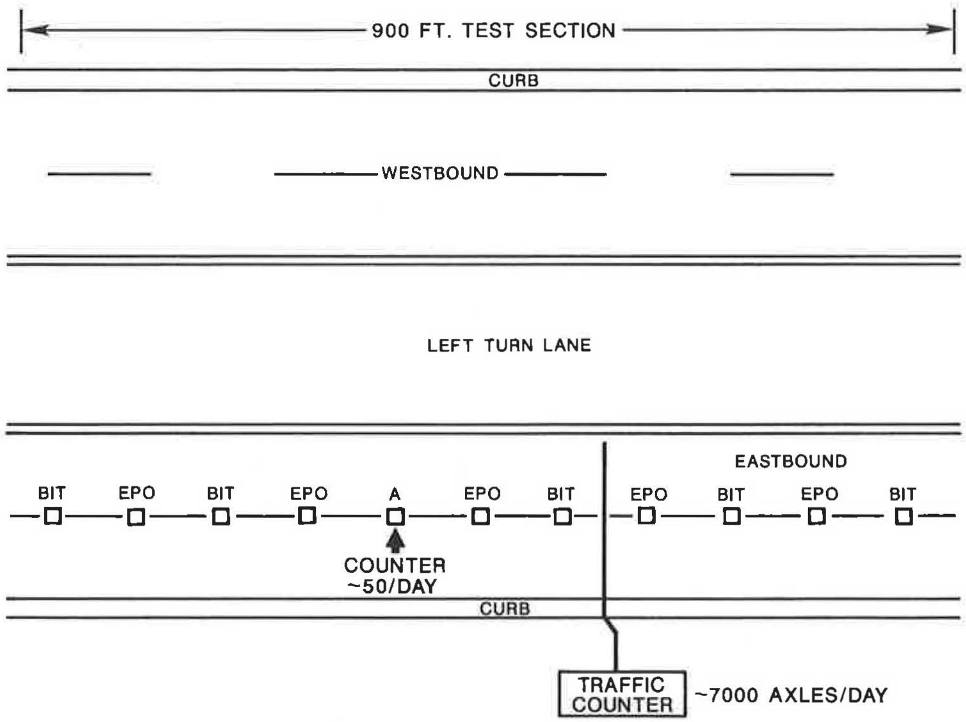

FIGURE 4 Seal coat test section with location of hit-counting marker (*A*) and traffic counter.

TABLE 1 MARKER HIT AND TRAFFIC DATA ON FM 60

	Hit Count		Axle Count	
Date	A	B	Daily	Total
May 18 (Mon.)	70	–	–	–
May 19	34	84	6,791	6,791
May 20	40	64	6,902	13,693
May 21	44	30	7,064	20,757
May 22 (Fri.)	26	94	7,190	27,947

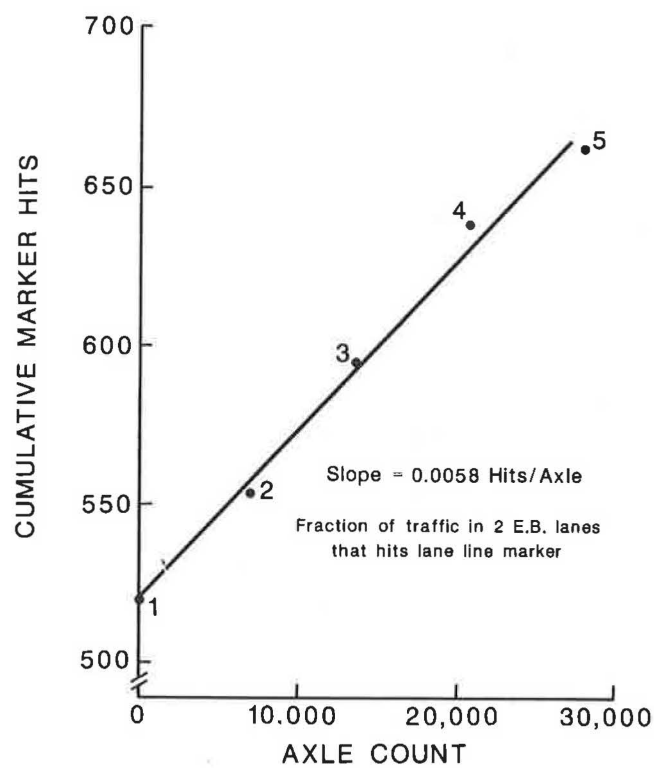

FIGURE 5 One-week accumulation of marker hit data.

PAVEMENT FATIGUE STUDY

With the likelihood that marker retention will be improved when the fatigue life of the asphalt concrete supporting the marker is increased, a fatigue test was designed to simulate the repetitive loads that a marker imparts to the pavement when hit by car or truck tires. The high-speed photography, described above, showed that a tire striking a marker remains in contact during the traverse. Because the center line of a tire seldom passes over the center of the marker, a tire impact imparts a rocking motion to the marker in addition to the vertical load. The repetitive rocking motion from tire impacts on either side of the marker applies a fatigue loading to the pavement surface.

Fatigue Test Apparatus

To simulate pavement marker fatigue loading, the laboratory apparatus shown in Figure 6 was constructed. Here pavement loading is applied by three pneumatic rams acting on a steel beam attached to a 4- by 4-in. steel plate. The steel plate is bonded to the asphalt concrete sample with the same adhesive that would be used to attach a pavement marker. The purpose of these experiments was to study pavement surface failure and any effects an adhesive may have on the fatigue life of a pavement surface. The 5-in. ram at the center of the beam

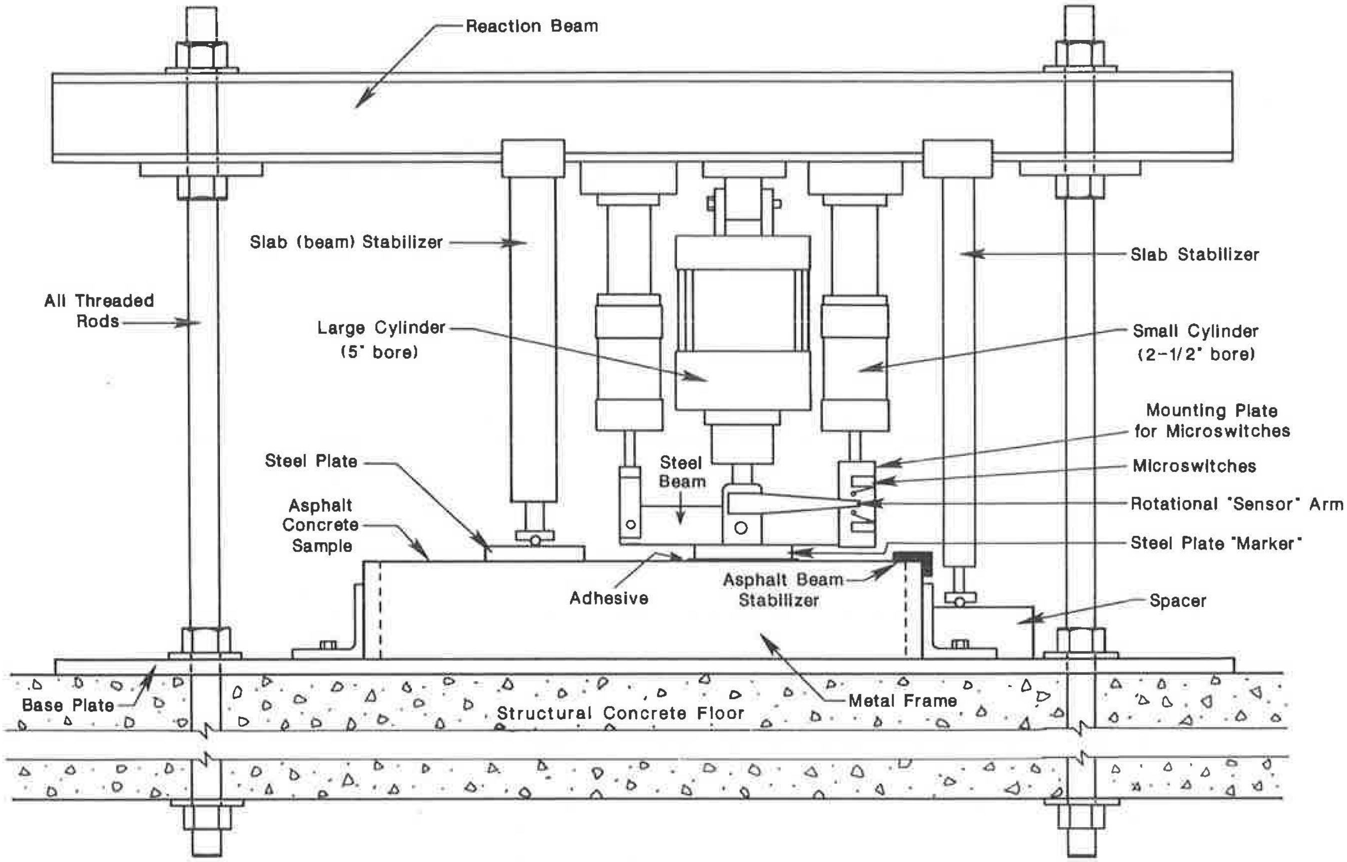

FIGURE 6 Fatigue test apparatus.

applies a pulsating vertical load on the center of the marker, whereas the two 2.5-in. rams at the ends of the beam apply an alternating moment to the marker plate. The control system timing the rams is completely pneumatic, and the entire apparatus runs on a very small volume of the laboratory air supply at 100 psi. The fatigue test is performed under load control, with the load frequency fixed at 1 cycle/sec.

As the load cycle count builds up, the small angular motion of the marker plate bonded to the asphalt begins to increase in amplitude. This is an indication that fatigue failure of the asphalt is imminent. The point at which fatigue failure has actually occurred is arbitrary. It was defined in these experiments as occurring at a rotation angle of 1.55 degrees. This value ensures that the asphalt under the edges of the marker plate has separated but is small enough for the test to be stopped before total failure occurs. The test is halted by tripping either one of the microswitches with the rotational sensor arm (see Figure 6).

When a fatigue test is completed, the cycle counter is read and the rocker beam is disconnected from the markerplate. The plate itself can then (usually) be tugged away from the asphalt and the failure examined. The asphalt failure surface produced by the laboratory fatigue test is remarkably similar to the failure surface seen when a marker has been lost from a highway pavement. Adhesive failure does not occur in this test. Further details of the fatigue test apparatus and data obtained from it are given in a marker-pavement compatibility study by Fernandez (*5*).

Test Results

Although little is known about the forces involved when a tire strikes a raised pavement marker, the impact is believed to generate asphalt stress levels in the range of 100 to 1,000 psi at the marker edges. The pneumatic rams were designed to apply alternating tensile stresses in this range for the fatigue study.

To study the possible interaction of adhesive and asphalt, laboratory test pavements were prepared with crushed limestone and two different grades of asphalt cement (AC-5 and AC-20). Two basically different marker adhesives (bituminous and epoxy) were used to attach the marker plates to these pavements for fatigue testing.

The fatigue test results shown in Figures 7–10 give the number of cycles to failure (N) for a test run at a certain tensile stress amplitude of the cyclic load. Each data point represents one surface failure. At the laboratory load frequency of 1 cycle/sec, a data point at $N = 10^4$ cycles was acquired in 2.8 hr. From the highway hit rate found by a hit-counting marker, this data point corresponds to 200 days of lane line life, indicated on the horizontal axis of Figure 7.

Although there is considerable data scatter, as is common in fatigue testing, it is possible to distinguish trends in the data when log-log plots are made as shown in Figures 7–10. The straight lines in these figures represent the power equation $\sigma = AN^B$ with A and B determined by the least-squares curve-fitting procedure.

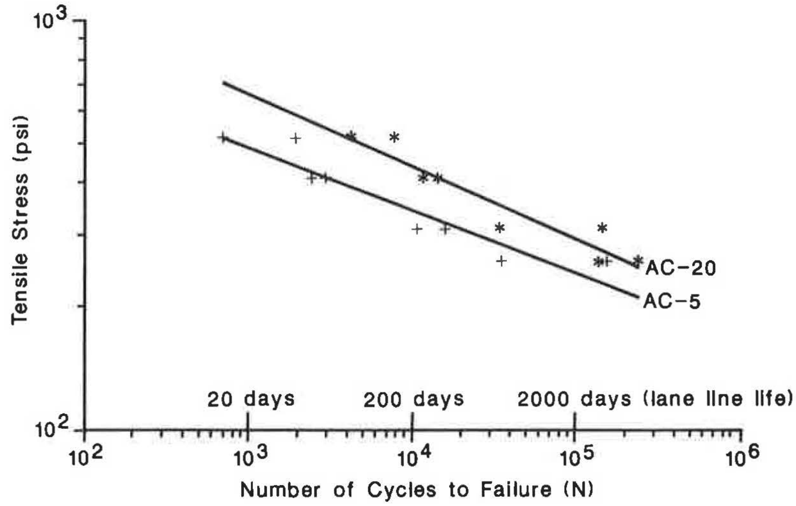

FIGURE 7 Fatigue failure of ACP by pavement marker load cycles: test pavements prepared with AC-5 and AC-20 binders, epoxy adhesive.

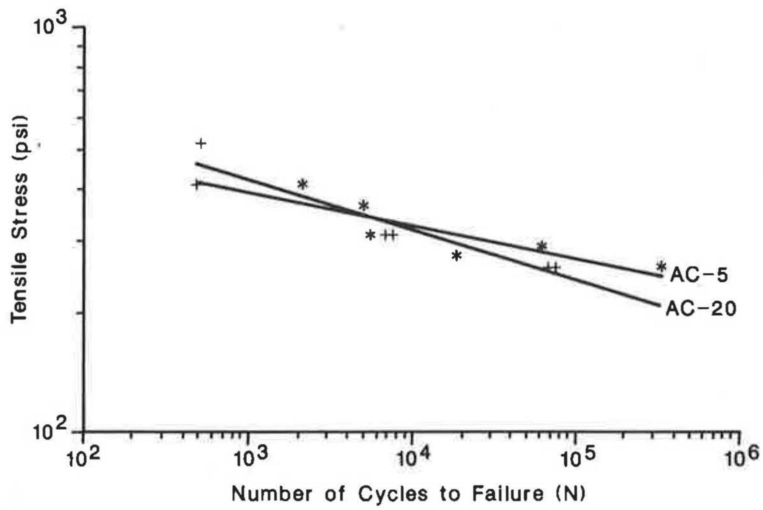

FIGURE 8 Fatigue failure of ACP by pavement marker load cycles: test pavements prepared with AC-5 and AC-20 binders, bituminous adhesive.

Discussion of Results

From the data in Figure 7, it is seen that fatigue failure occurs sooner for the AC-5 concrete than for the AC-20 concrete when epoxy adhesive is used. These pavements differ mainly in stiffness, with the AC-20 binder giving the stiffer surface. When the markers are attached with bituminous adhesive, the distinction between binder grades is not as clear (see Figure 8) and is seen to depend on stress level. In order to use the laboratory results in a comparative analysis of adhesives, Figures 9 and 10 were made to show the effect of the two adhesives used with each asphalt cement grade. These plots clearly show that marker retention time is affected by the adhesive used. This result is notable because very little adhesive appears to penetrate the pavement surface. For the lower-grade binder (AC-5) pavement the retention time is less at higher stress levels when bituminous adhesive is used (see Figure 9). When a higher-grade binder (AC-20) is used, giving a stiffer pavement, there is a consistent difference between the retention times obtained with the different adhesives. The retention on the stiffer laboratory pavement is better at all stress levels when epoxy adhesive is used (see Figure 10).

These results are supported by the fact that, in the field, better retention has been obtained with bituminous adhesive. During the service life of an asphalt pavement, its stiffness increases with aging and traffic. In the early life of an ACP, when its stiffness is lower, the retention time of markers using bituminous adhesive may be expected to exceed that of markers placed with epoxy adhesive. With time, the advantage of the bituminous adhesive over the epoxy adhesive is predicted (by the laboratory findings) to decrease until, on an aged

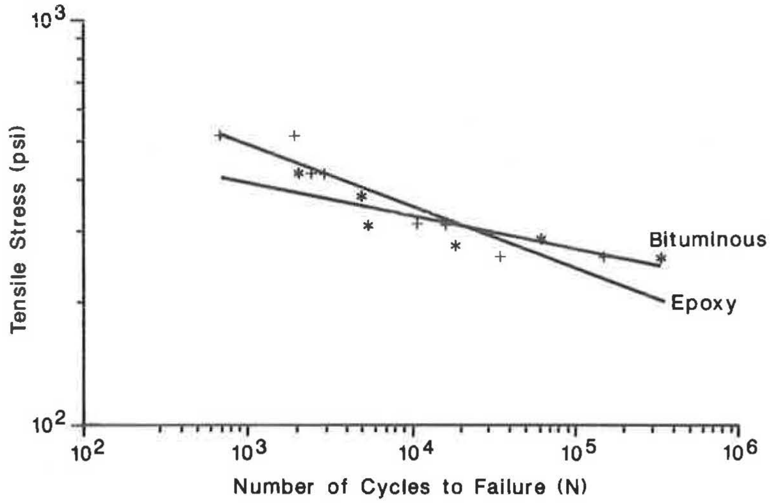

FIGURE 9 Fatigue failure of ACP by pavement marker load cycles: marker plates attached with bituminous and epoxy adhesives, test pavements prepared with AC-5 binder.

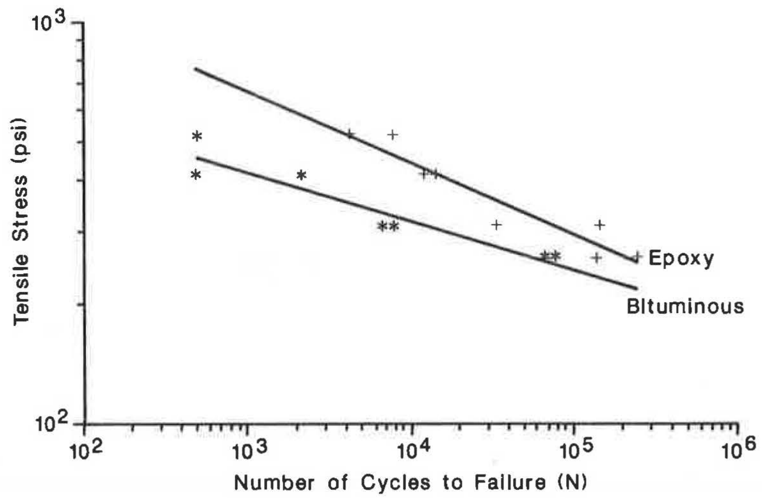

FIGURE 10 Fatigue failure of ACP by pavement marker load cycles: marker plates attached with bituminous and epoxy adhesives, test pavements prepared with AC-20 binder.

pavement, the retention of markers attached with epoxy may become comparable or exceed the retention of markers attached with bituminous adhesive.

HIGHWAY TEST SECTIONS

The survivability of a variety of test marker systems has been monitored since the inception of this study. It was determined that the Weibull distribution function reflects the marker loss rates reasonably well. This two- (sometimes three-) parameter statistical distribution function was first proposed in the early 1950s (*6*). The function has been found to be particularly well suited to characterization of the fatigue failure rates of large numbers of identical parts subjected to similar or identical load histories. Algebraically, the distribution is written as

$$P_s(n) = \exp[-(n/b)^c]$$

where $P_s(n)$ is the so-called survival function, the fraction of the original population that survives after n loadings. For raised pavement markers, $P_s(n)$ represents the fraction of the markers remaining after they have each been subjected to n tire hits. The constants, b and c, are parameters selected to best fit the observed data. Sometimes c is referred to as the shape parameter and b as the characteristic life. Just why b is called the characteristic life becomes clear when one realizes that, irrespective of the value of the constant c, P_s is 0.368 when the variable n is equal to b.

A systematic procedure was used to pick the constants b and c to represent observed marker loss rates. The constants were selected to minimize the sum of the squares of the differences between the observed loss rates and the loss rates predicted by the survival function. This made it possible to compare the retention performance of different marker systems either by comparing cumulative distribution curves or by comparing the values of the shape and characteristic life parameters.

Markers Placed with Epoxy Adhesive

As an example of the applicability of the Weibull distribution, consider the results of a 2-year study of the retention of raised RPMs placed with epoxy adhesive. This study was conducted by the Texas State Department of Highways and Public Transportation (SDHPT) in the late 1970s. The observations were made on three major highways: one in Dallas and two in San Antonio. The location in Dallas was on a six-lane divided highway (SH 183 from Mockingbird Lane to near International Place) where the markers were placed on both the inside and outside lane lines. The two locations in San Antonio (IH 10 from Fredericksburg Road southeast to IH 35, and IH 35 from the stockyards south to IH 10) were both four-lane divided highways and the markers were placed only on the single lane lines. Several types of markers are represented and were systematically placed so that similar numbers of each type faced traffic in each direction at each location. At four time intervals (3, 6, 12, and 24 months after the test began), counts of the markers remaining in place were made by SDHPT personnel. The complete results have been reported elsewhere (*3*). The condensed data shown in Table 2 give the results of the count for 4- by 4-in. RPMs.

To get a broad overview of the test results, the retained fraction of the 4- by 4-in. pavement markers was plotted as a function of the number of tires estimated to have hit each marker. This estimate was made using the daily traffic reported in the two adjacent lanes (to the markers) and the hit rate for lane line markers determined by the instrumented marker described earlier. The Weibull distributions were then fit to the observations with the results shown in Figures 11–13, where the solid curve is the prediction given by $P_s(n)$. In this particular study, the asphalt concrete pavement of IH 10 (see

TABLE 2 FRACTION OF MARKERS REMAINING IN SAN ANTONIO AND DALLAS RETENTION STUDIES

Location	Total No. of Markers Installed	At 3 Months		At 6 Months		At 12 Months		At 24 Months	
		Fraction Remaining	Est. Hits	Fraction Remaining	Est. Hits	Fraction Remaining	Est. Hits	Fraction Remaining	Est. Hits
San Antonio IH 10 (asphalt pavement)	234	0.996	24,300	0.953	48,600	0.877	97,200	0.826	194,400
IH 35 (asphalt pavement)	123	1.00	21,600	1.00	43,200	0.992	86,400	0.871	172,800
Dallas: SH 183 (PCC pavement)	360	0.997	12,600	0.989	25,200	0.931	50,400	0.737	100,800

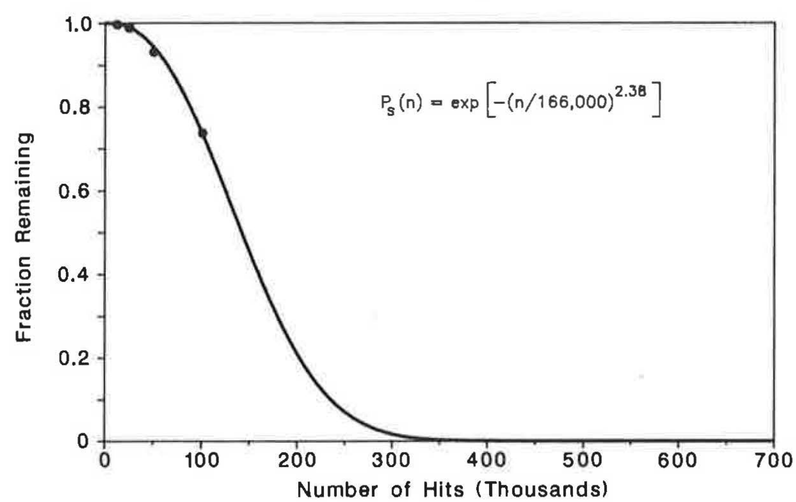

FIGURE 11 Loss rate curves for 4- by 4-in. markers using epoxy adhesive: SH 183 in Dallas (portland cement concrete).

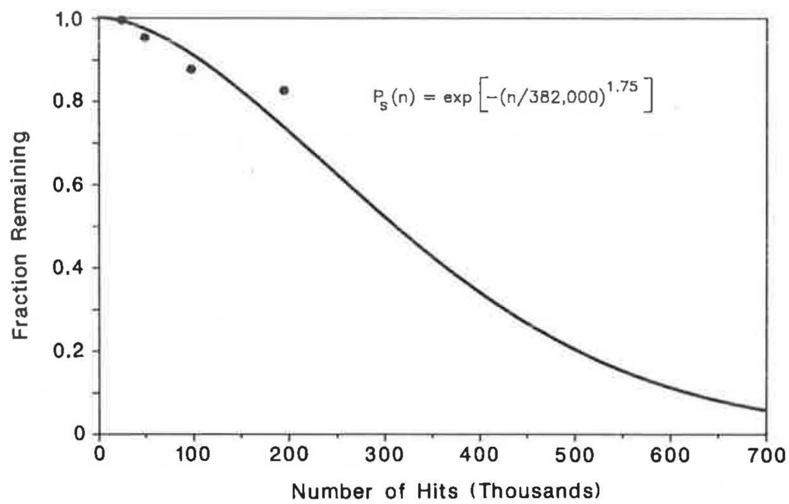

FIGURE 12 Loss rate curves for 4- by 4-in. markers using epoxy adhesive: IH 10 in San Antonio (asphalt concrete).

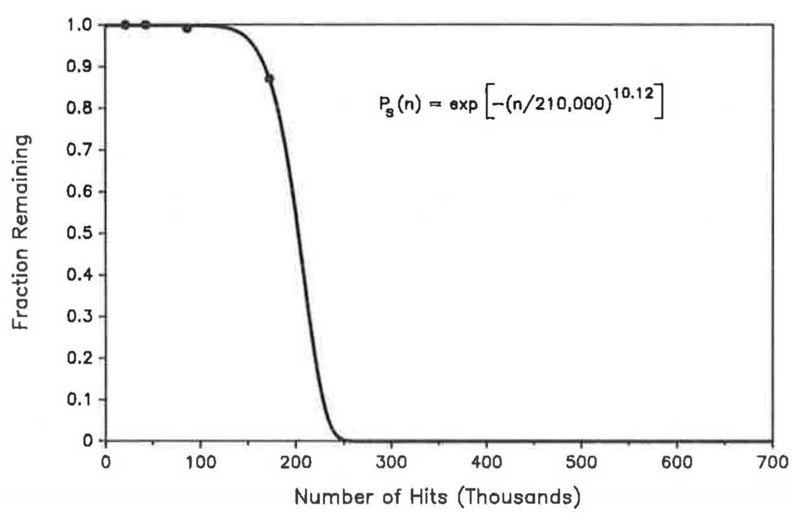

FIGURE 13 Loss rate curves for 4- by 4-in. markers using epoxy adhesive: IH 35 in San Antonio (asphalt concrete).

Figure 12) retained the markers better than the PCC pavement of SH 183 (Figure 11). However, the latter retained the markers longer than the asphalt concrete of IH 35 (see Figure 13).

Bituminous Adhesive Versus Epoxy Adhesive

An alternative adhesive (to epoxy) first suggested by the Stimsonite Company and recommended by Roger McNees of TTI is a black, solid, bituminous adhesive marketed specifically for raised markers. This single-component material must be heated to nearly 400°F (200°C) for use. This temperature is slightly above the softening point of asphalt, which may account for its success on asphalt pavements.

Several hundred of the low-profile (2- by 4-in.) reflective markers were installed using bituminous adhesive and a like number using conventional epoxy. These tests were all in District 16 of the Texas SDHPT near Corpus Christi. As shown in Figure 14, the superiority of the bituminous adhesive over epoxy was found to be pronounced.

Several engineers with experience using the bituminous adhesive on Texas roads report similar results, suggesting that the service life of markers bonded to asphalt with this adhesive is significantly increased. Specifically, two side-by-side comparisons of the bituminous and epoxy adhesives are known to have been made on Texas highways. The first was made by Joe Graff, a maintenance engineer in the Texas SDHPT, in 1985–1986 on IH 20 in Smith County. The traffic count at this site was very high and included an especially high percentage of trucks (estimated to be nearly 50 percent). After about 1 year in place, Graff reported that approximately 8 percent of all markers placed with bitumen and 47 percent of all markers placed with epoxy had been lost.

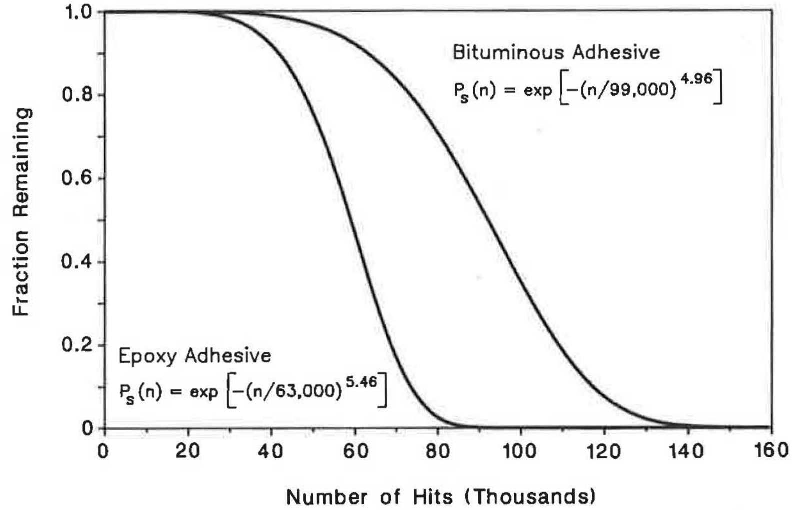

FIGURE 14 Loss rate curves comparing the retention properties of bituminous and epoxy adhesives.

TABLE 3 FIELD TEST RESULTS FOR BITUMEN AND EPOXY ADHESIVES COMPARED ON THE SAME HIGHWAY AFTER 30 MONTHS

Highway	Location	Initial Number	Percent Retained by	
			Bitumen	Epoxy
FM 369	Wichita Falls	283	60	78
US 279	Wichita Falls	118	94	60
US 90	San Antonio	242	98	47
US 281	San Antonio	183	96	54

In the second comparison, Robert K. Price, a materials and test engineer in the Texas SDHPT, made several counts of the relative performance of bituminous and epoxy adhesives on test sections with low-profile markers (*7*). A representative selection of Price's findings is given in Table 3. Of a total of 10 highway test sections in five districts, the only low-profile markers retained longer by epoxy were found on FM 369 (Table 3).

Seal Coat Test Section

A test section to compare bituminous and epoxy adhesives on seal coat was placed on a high-speed (50-mph) straight section of FM 60 in College Station, Texas. This is a four-lane divided highway that had been seal coated about 3 months before the markers were installed. Lane line markers (4- by 4-in.) were placed in skip stripe gaps alternatively with epoxy and bituminous adhesive, as shown in Figure 4. Twelve markers were placed with epoxy and 12 with bituminous adhesive in both the eastbound and the westbound lane lines, giving a 900-ft test section in each direction of traffic. A count taken 14 months after installation found all 48 markers intact. However, 6 months later (20 months after installation) four of the markers attached with bituminous adhesive were missing. None of the markers attached with epoxy were lost.

Replacement Marker Test Section

Replacement markers are ordinarily placed adjacent to the surface failure left by the missing marker. The exposed failure is then subject to further deterioration by traffic and weather. Installing a replacement marker on the failure spot would appear to be advantageous in (a) using the slightly larger surface depression area for adhesive bonding and (b) sealing the surface failure left by the missing marker. However, the perceived advantages may be outweighed by other effects such as the susceptibility of the surface failure to additional failure.

A replacement marker test section of 16-lane line markers (4- by 4-in.) was placed on FM 60 in a high traffic area that had lost all of its markers. The markers were placed with epoxy, alternately in front of and on top of the failure spots in the skip stripe gaps. A shot of compressed air was used to blow debris out of the failure depression before filling it with epoxy and placing a marker on top. When the test section was driven at night, the 640-ft illuminated marker string gave no indication of any difference in marker placement. This test section was installed on July 17, 1986. Twenty-two months later, all 16 markers were still in place. Although the test section was resurfaced a month later (in 1988), terminating the test, it appears that this maintenance technique can be used to simultaneously repair a pavement flaw and replace a missing marker.

CONCLUSIONS

Perhaps the most significant finding of this research is that the adhesive material used to bond the markers to the pavement surface can influence the fatigue strength of asphaltic concrete. This is true even though there is very little penetration of the adhesive into the pavement. The fatigue studies show that a more compliant adhesive (e.g., bituminous) gives a new asphalt pavement, the more compliant pavement, a longer fatigue life than a stiffer adhesive such as epoxy. Ostensibly, a longer fatigue life means that the marker stays in place for a greater number of tire impacts.

The laboratory studies indicated that for stiffer asphaltic concrete surfaces the advantage of the bituminous adhesive decreased. The advantage of bituminous adhesive also decreased as the force level increased. Thus, it is concluded that for older pavement surfaces and for pavements with heavier (truck) traffic, the advantage the bitumen exhibits over epoxy is largely lost. These findings imply that it may some day be possible to tailor the properties of the adhesive to match the pavement surface properties and thereby optimize the retention lives of the markers.

ACKNOWLEDGMENTS

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Durability Testing for Retroreflective Sheetings

W. D. KETOLA

Although there is considerable published research pertaining to the usefulness of or problems associated with durability testing of polymers or paints and coatings, there is little published work specifically addressing durability testing for retroreflective sheetings. The simulation of exterior exposure stresses in artificial accelerated tests is discussed and measurements of the spectral power distribution of light sources commonly used in accelerated tests are compared with that for sunlight. Two exposure experiments are described that show how poor simulation of exterior stresses can lead to reversals between predictions from artificial tests and exterior exposures. Variability in exterior exposures is illustrated by results from a single set of sheetings exposed at different times at the same site. Experiments using a single lot of retroreflective sheeting exposed in identical devices all operating the same test cycle are used to quantify the variability associated with artificial accelerated testing. Artificial accelerated and exterior exposures of a series of eight lots of retroreflective sheeting are compared and show that there are large differences in sheeting rank performance between the artificial accelerated tests and 5-year exterior exposures. An experiment comparing several exposure orientations for a model retroreflective sheeting indicated a 2:1 increase in degradation rate for a 45 degree angle or solar tracking exposures relative to the vertical.

Retroreflective sheetings are complex multilayer composite products in which deterioration of performance can be caused by any of a number of mechanisms. Typical failure modes are

- Destruction of the metallic reflector coat;
- Disruption or distortion of the optical elements within the sheeting, making retroreflection of incoming light less efficient;
- Degradation or destruction of the outermost polymer layer;
- Fading of dyes or pigments used to produce appropriate color in the sheeting or screen-printed graphics; and
- Failure of bonds between layers, causing separation or delamination of the composite.

The type of failure or degradation can depend on the type of sheeting being tested (enclosed lens, encapsulated lens, or cube corner) and also on the composition of the individual layers within the sheeting construction. Failures can be initiated or accelerated by a particular combination of environmental stresses that may only occur in certain geographical locations or climates. For example, in some environments the combined effects of sunlight and moisture initiate reactions that can cause corrosion of the metallic reflector coat. Other climates may produce disruptions of the optical path in the sheeting by repeated expansions and contractions of polymer layers during cycling between wet and dry or hot and cold conditions.

Scientists involved in development of materials for exterior applications and those involved with setting specifications have long desired to assess durability by using results from artificial accelerated testing ("machine weathering") rather that waiting for results from long-term exterior exposures. Although there are numerous studies in the literature on the establishment of "acceleration factors" equating X hours in an artificial accelerated test to Y months' exterior exposure, there are several very important reasons why such relationships are meaningless, namely, variability in exterior climates, poor replication of exterior stresses in artificial tests, and variability in the accelerated testing devices. In order to evaluate how these problems affect durability tests for retroreflective sheeting, a series of exposure experiments was conducted. In addition, the spectral power distributions (SPD) of light sources used for artificial accelerated tests were measured and compared with the SPD of sunlight. The experiments and how the results can affect durability testing protocols for retroreflective sheeting are described in the following sections.

EXPOSURE AND EXPERIMENTS

All retroreflective sheeting lots used in the exposure experiments were prepared in the 3M Traffic Control Materials Division laboratory or as part of production experiments conducted in 3M manufacturing plants. Pressure-sensitive adhesives were used in all lots. (The durability data presented here should not be taken as representative of the performance of any commercially available 3M retroreflective sheeting product.) Artificial accelerated exposures were conducted in the Weathering Services Laboratory of 3M's Analytical and Properties Research Lab. The artificial exposure tests used are described in Table 1.

Outdoor exposures were conducted per ASTM Standard Practice G7 at sites in Miami and Phoenix. Florida exposures were on open racks, whereas Arizona exposures were conducted with samples mounted on plywood-backed racks.

Sheetings were applied to 5052H33 (0.025 in. thick) aluminum panels that were chromate treated per ASTM B449, Class 2. Replicate samples of each lot tested were used for all exposure tests. Table 2 summarizes the exposure tests and number of replicates used for each sheeting lot or series.

Before exposure and after each exposure increment, samples were tested for retroreflectivity (coefficient of retroreflectance at −4-degree entrance angle, 0.2-degree observation angle) per ASTM E810. Where indicated, 60-degree

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TABLE 1 ARTIFICIAL ACCELERATED EXPOSURE TEST CONDITIONS

ASTM G23-84, Type E, Method 1		
	Filtered (#7058 Corex) open flame carbon arc	
	Cycle:	102 minutes light only
		18 minutes light plus water spray
		63°C black panel temperature
ASTM G26-84, Type B		
	Water cooled 6500 watt xenon burner	
	Borosilicate inner and outer filters	
	Cycle:	102 minutes light only
		18 minutes light plus water spray
		63°C black panel temperature
ASTM G53-84		
	FS-40 UVB fluorescent lamps	
	Cycle:	4 hours UV at 60°C black panel temperature
		4 hours condensation at 50°C black panel temperature

gloss and yellowness indexes were tested per ASTM D523 and ASTM D1925, respectively. Results are reported as the mean of tests made on replicate panels.

Irradiance measurements were made using a model DG-52 spectroradiometer system from EG&G Gamma Scientific equipped with an NM9H double grating monochromator with 2-nm resolution and a model 50B cosine receptor. The spectroradiometer was calibrated with deuterium- and tungsten-calibrated reference lamps traceable to the National Bureau of Standards. Measurements were made at 2-nm increments using the average of 200 individual readings. The cosine receptor was positioned at the sample plane in a fluorescent ultraviolet (UV) device while measurements in the carbon and xenon arc devices were made with the cosine receptor positioned near the sample plane using a specially modified door. In all cases, measurements were made at the center of the allowed exposure area. Irradiance at the sample plane for the carbon and xenon arc devices was obtained by multiplying the measured irradiance values by correction factors calculated using the inverse-square law, which accounts for the relative position of the cosine receptor and the sample plane. The solar spectral power distribution was obtained by Ohio Spectrographic Service from measurements made in Phoenix (clear sky, solar noon, summer solstice, cosine receptor mounted on an equitorial follow-the-sun motor drive, measurements made at 1-nm increments with 1-nm bandpass).

Measurements of black panel temperatures were made at the South Florida Test Service facility in Phoenix (Wittman), Arizona. Temperatures were continuously recorded on a strip chart for a 3-week period during April–May 1986. The black panels were 18-gauge steel coated with Rust-Oleum Bar-BQ Enamel®, oven dried for 8 hr and air dried for 1 week before use. The thermocouples were attached to the backs of the panels and were calibrated at 0° and 60°C. Measurements were made with the black panels mounted on 34- and 90-degree open backed racks.

RESULTS AND DISCUSSION

Exposure Stress Simulation

In an attempt to produce rapid results, artificial accelerated tests often use light sources that have significant emissions below the solar UV cutoff (290 nm). Figures 1 and 2 show results for UV spectral power distribution measurements made at the sample position in several artificial accelerated devices. One can easily see the differences between these light sources and sunlight. Figure 2 clearly shows the much higher irradiance levels for the artificial light sources at shorter wavelengths. It is this short-wavelength radiation that can produce rapid photodegradation that may not be representative and could overwhelm important degradation reactions that occur in exterior exposures.

In addition, water spotting is a persistent problem when water spray is used as a moisture source, even in systems where great care is taken to control dissolved solids. Cutrone (*1*) reported the formation of silica deposits on paint samples exposed in an accelerated test using water spray (per British Standard BS3900, part F3) and speculated that colloidal silica in the spray water was the cause. Even though water used in accelerated test devices at 3M is treated by being passed through a multistage deionizing system to remove particulates, anions, cations, and organic materials, silica deposits have been detected on retroreflective sheetings and other materials exposed in those devices. These deposits detract from appearance and

TABLE 2 SUMMARY OF EXPOSURE TESTS CONDUCTED

Sheeting Lot or Series	Number of Samples Tested	Description of Exposure Tests
Lots A and B	2	Carbon Arc (G23) and Fluorescent UV/ condensation (G53)
	2	12 month Florida 45°
	4	24 month Florida 45°
	6	36 month Florida 45°
	8	48 month Florida 45°
Lot C	3	Carbon Arc (G23), six units all using the same cycle, 3 replicate samples per unit
Lot D	3	Fluorescent UV/condensation (G53), four units all using the same cycle, 3 replicate samples per unit
Series I	18	Nine lots from a designed experiment evaluating formulation variations for a single type of sheeting. Two replicate samples for each lot. Florida 45° exposures.
Series II	2	Eight lots of retroreflective sheeting (enclosed lens, encapsulated lens and cube corner), 2 replicate samples of each lot used for all artificial accelerated tests (G23, G53, and G26)
	6	Six replicates for each of the eight lots were used for the five year Florida and Arizona exposures.
Series III	72	One lot of sheeting was used, 12 samples were sent out for each of the six exposure orientations. One sample was recalled each month for evaluation.

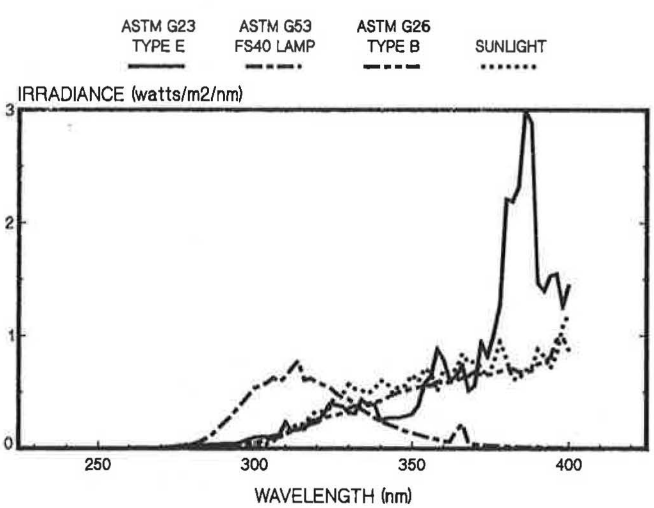

FIGURE 1 Representative spectral power distribution for light sources used in artificial accelerated exposure testing.

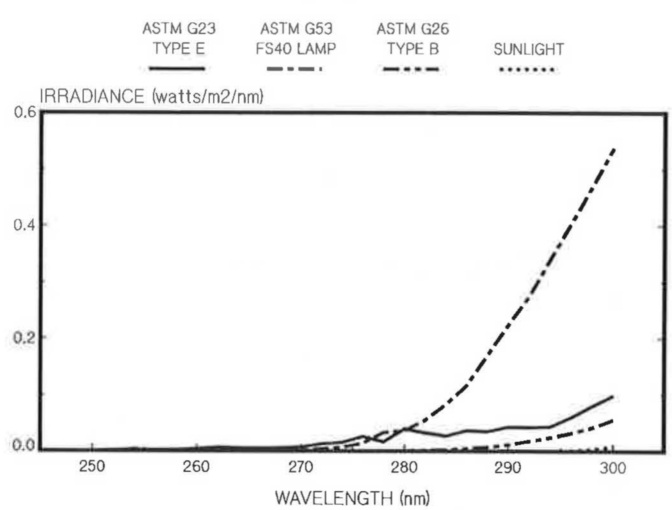

FIGURE 2 Representative short-wavelength UV power distributions for light sources used in artificial accelerated exposure testing.

can lead to unrealistic surface deterioration that does not occur in exterior exposures.

Furthermore, artificial accelerated testing may not subject samples to certain stresses that are very important in exterior exposures. Work by Yamaski (*2*) showed that materials exposed outdoors are wet between 21 and 35 percent of the time in humid continental climates such as Ottawa, Canada (45 degrees N latitude). Rain events account for only a small amount of the total wet time, so samples are wet with condensation or dew. Condensed moisture is fully saturated with oxygen, an essential element in polymer oxidation. Accelerated tests using water sprays do not control oxygen level in water and more probably simulate rain and not condensation.

Standard artificial accelerated tests do not expose retroreflective sheeting samples to the effects of acid dew or acid rain, industrial pollutants such as sulfur dioxide or ozone, or deicing salts used on roadways. Research has shown that the rate of aluminum corrosion is directly related to the deposition rate of sulfur dioxide and chloride ions (*3,4*). Increased degradation of polymers in the presence of relatively low sulfur dioxide concentrations has also been reported (*5,6*). Comparison of natural dew and rainwater showed dew to have over 2 times the level of sulfate ion and 16 times the chloride ion concentration as rainwater (*7*).

Results presented in Figures 3 and 4 show how sheeting performance can be misjudged when artificial accelerated tests fail to reproduce exterior exposure stresses. In Figure 3, retroreflective sheeting lot A shows relatively poor performance in the ASTM G23 or ASTM G53 artificial accelerated test cycles described previously, but shows excellent results after 48 months of Florida 45-degree exposures. Conversely, the same artificial accelerated exposures of reflective sheeting lot B (Figure 4) show very little brightness loss after 2,500 hr, whereas Florida 45-degree exposures produced rapid failure.

Variability in Exposure Tests

Exterior Tests

Differences in climate between locations as well as between seasons are well understood and need no further explanation.

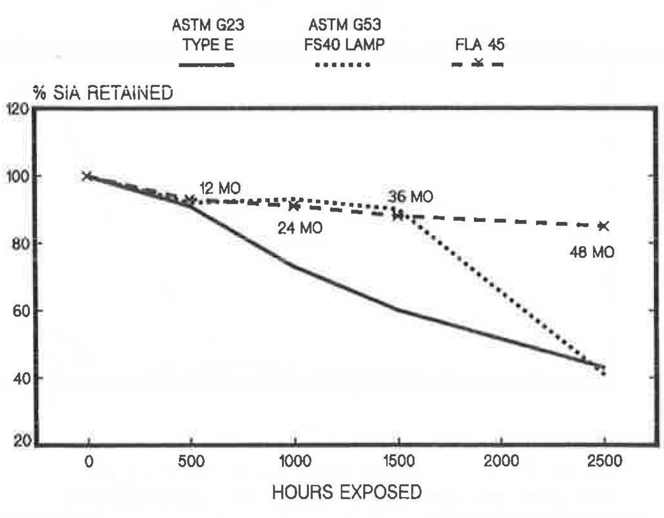

FIGURE 3 Comparison of results for artificial accelerated and Florida 45-degree exposures of retroreflective sheeting lot A.

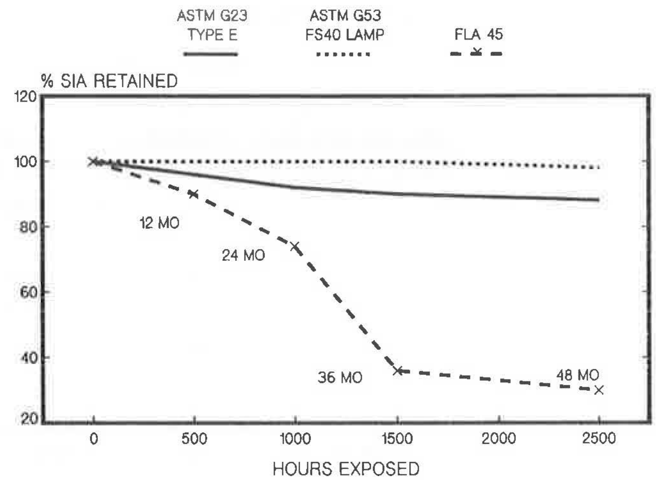

FIGURE 4 Comparison of results for artificial accelerated and Florida 45-degree exposures of retroreflective sheeting lot B.

What is important is the fact that virtually all exposure periods in any given location are unique. There can be significant differences in rate of failure for materials exposed at different times in the same location, as shown in the following example. A series of nine experimental retroreflective sheeting constructions, each a point from a designed experiment evaluating minor formulation changes for a single product, was exposed in Florida at a 45-degree angle for 12 months during 1982, with all nine constructions showing significant brightness loss. This set of samples was retired and a second exposure of an identical set from this same series was started in 1984 and continued for an additional 12 months in 1985–1986. The mode of failure (reflector coat oxidation) was the same for both sets of samples. Results on these identical samples and exposure times for these different dates are compared in Table 3 and show dramatic differences in failure rate. This is a clear example of how year-to-year differences in climate at the same location can affect results. Rate of failure may also depend on whether an exposure starts in the spring or fall of the year. Comparisons between materials should only be made by using a reference or control with all exposure evaluations. Use of multiyear exposures can improve reliability of results by averaging the effects of seasonal or year-to-year variability.

Artificial Accelerated Exposures

Artificial accelerated tests have traditionally been assumed to stress samples with temperature, light, and water in a much more consistent manner than exterior exposures. In essence, they have been almost considered to be analytical tests with a high degree of repeatability. However, recently published work indicates that results from artificial accelerated tests are highly variable. Blakey (*8*) reports that exposures of identical panels of a titanium-dioxide-pigmented air-dry alkyd paint in carbon arc units operating per British Standard BS3900, part F, produce very large machine-to-machine and within-machine differences. After 2,000 hr, 60-degree gloss values ranged from 17 to 60 for identical samples exposed in supposedly equivalent machines. Even larger variations were reported for a thermosetting acrylic formulation. The Association of Automobile Industries' Working Group on Test Methods for

Paints (*9*) reported results from a round-robin exposure study of a series of paints. Exposures were conducted in xenon arc devices all using the same test cycle and showed that after 2,000 hr, gloss values for identical paints varied by 50 percent of the mean for all machines.

In order to evaluate the degree of variability associated with artificial accelerated testing of retroreflective sheetings, two studies were conducted in which identical samples of retroreflective sheetings were exposed in equivalent machines using the same test conditions. The results for identical samples of retroreflective sheeting lot C exposed in six different XW Sunshine Carbon Arc Weatherometers operated per ASTM Standard Practice G23, Type E, Method 1 are shown in Figure 5. Each data point is the mean of three replicate samples exposed in each unit. The large differences in degradation rate between the individual units is readily apparent. This type of difference between supposedly identical units operating under supposedly identical test conditions can affect decisions regarding material acceptability. For example, if a specification stated that this type of sheeting had to have a minimum of 50 percent retained brightness after 1,500 hr, results from three of the units could be used to reject the material whereas results from the other three units would support material acceptance.

Similar results are shown in Figure 6 for exposures of identical samples of retroreflective sheeting lot D in four fluorescent UV/condensation devices operated per ASTM Standard Practice G53-84. Again, each data point represents mean retroreflectivity retention for three replicates exposed in each machine. All units used FS-40 UVB fluorescent lamps and were operated using a cycle of 4 hr UV at 60°C and 4 hr condensing moisture at 50°C. If a specification required 80 percent brightness retention after 2,200 hr exposure, results from Unit 3 would be sufficient for rejection, those from Units 2 and 4 would be considered marginally acceptable, and those from Unit 1 would easily pass the requirement.

The data in Figures 5 and 6 indicate the high degree of variability in artificial accelerated exposure testing. Clearly, 1,000 hr in one unit is not the same as 1,000 hr in another even when both are operating to the same conditions described in a specified test method. The inherent variability associated with artificial accelerated exposure testing means that use of specifications mandating a specified performance level after a specific exposure period can lead to decisions on product acceptability based on test variability rather than product performance.

Prediction of Sheeting Performance

Instead of using absolute measures of performance, several authors have advocated ranking the performance of a series

TABLE 3 RETROREFLECTIVE SHEETING SERIES 1—FLORIDA 45-DEGREE EXPOSURES: EFFECT OF EXPOSURE PERIOD ON RESULTS

Exposure Period	Total Time Exposed	% Retained Retroreflectivity for the Nine Lots in the Series: Mean	Standard Deviation
12/81 to 12/82	12 months[1]	27.9%	7.1%
4/84 to 4/85	12 months	96.0%	2.1%
6/85 to 6/86	24 months	64.6%	11.1%

[1] Samples retired

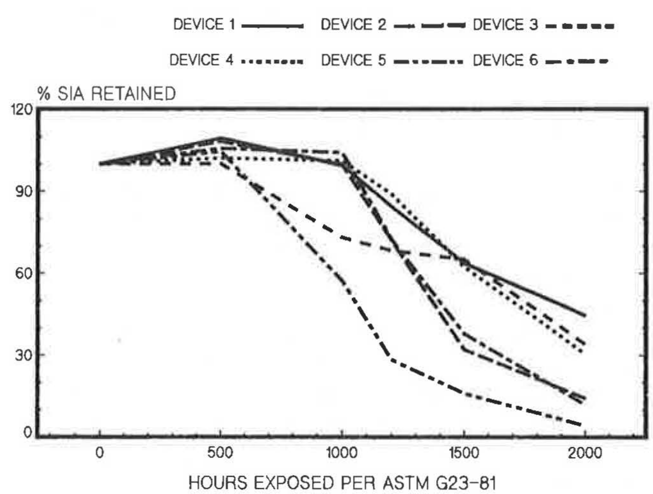

FIGURE 5 Results for identical samples of retroreflective sheeting lot C exposed in six Atlas Model XW Sunshine Carbon Arc Weatherometers all operated per ASTM G23-84, Type E, Method 1.

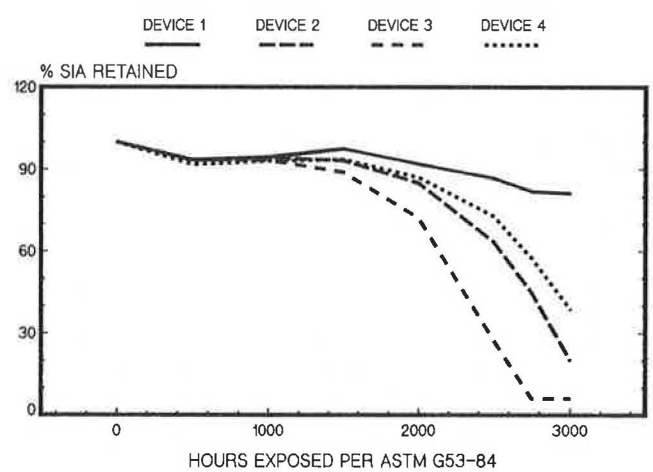

FIGURE 6 Results for identical samples of retroreflective sheeting lot D exposed in four fluorescent UV/condensation devices operating using an identical cycle per ASTM G53-84.

TABLE 4 PERCENT RETAINED RETROREFLECTIVITY OF SHEETINGS EXPOSED IN ARTIFICIAL ACCELERATED AND EXTERIOR TESTS

Reflective Sheeting Number	2500 Hour ASTM G23 type E	3000 Hour ASTM G26 type B	2500 Hours ASTM G53-84 UVB Lamp	Five Years F45^{o}	Five Years AZ45^{o} [1]
1	83	79	80	66	79
2	70	75	33	60	84
3	86	78	24	23	41
4	58	82	14	14	76
5	88	84	98	30	94
6	43	61	41	68	67
7	38	62	30	50	59
8	82	72	75	89	77

[1] Samples exposed in Arizona were mounted on racks backed with black painted plywood and those exposed in Florida were on open backed racks.

TABLE 5 CORRELATION COEFFICIENTS (r) FOR RETROREFLECTIVITY RETENTION OF SHEETINGS IN ACCELERATED AND EXTERIOR EXPOSURES

Correlation Pair	r
2500 hr G23 and 5 yr F45	-.05
2500 hr G23 and 5 yr AZ45	.25
3000 hr G26 and 5 yr F45	-.55
3000 hr G26 and 5 yr AZ45	.37
2500 hr G53 and 5 yr F45	.39
2500 hr G53 and 5 yr AZ45	.62

of materials in artificial accelerated tests (*10*). These ranks are then compared with those obtained in exterior exposures. The results for artificial accelerated and exterior exposures of sheeting series 2 are shown in Tables 4–7. This series is made up of eight different retroreflective sheetings of varying type (enclosed lens, encapsulated lens, and cube corner). The artificial accelerated exposures were conducted using the test cycles described in the section on Exposure Experiments. As indicated previously, the Arizona and Florida 45-degree angle exposures were conducted per ASTM G7. Table 4 summarizes the percent retained retroreflectance for each sheeting obtained in the three artificial accelerated tests and in the Arizona and Florida 45-degree angle exposures.

Table 5 shows the results obtained when the retro-reflectivity retentions from the accelerated tests are correlated with those from the Florida or Arizona exposures. The correlation coefficient, r, was calculated according to Equation 1 and is a measure of the degree of association between two sets of data. Values of r range from -1 to 1, with values near 0 indicating no association and absolute values of 0.90 or greater indicating strong association. One can easily see that the correlation of retroreflectivity retention between artificial accelerated exposures and exterior exposures is very poor.

$$r = \frac{\Sigma (x - \bar{x})(y - \bar{y})}{[\Sigma (x - \bar{x})^2 \, \Sigma (y - \bar{y})^2]^{0.5}} \quad (1)$$

The percent retained retroreflectance was then used to rank the performance of the eight sheetings in the artificial accelerated tests and the exterior exposures. These performance rankings are shown in Table 6 and were used to determine rank correlation coefficients, again using Equation 1, which are summarized in Table 7. The correlation coefficients are higher than those obtained for the brightness retentions but are still too low to allow the use of these accelerated tests as any predictor of exterior performance. The validity of rank correlations depends on sample size, but it is generally accepted that rank correlation coefficients of 0.90 or greater are necessary for highly predictive results (*11*). Note the poor rank correlation between the Florida and Arizona exposures. This is a clear indication that performance in one location cannot

TABLE 6 RETROREFLECTIVE SHEETING PERFORMANCE RANKING IN ARTIFICIAL ACCELERATED AND EXTERIOR EXPOSURES

Reflective Sheeting Number	2500 Hour ASTM G23 type E	3000 Hour ASTM G26 type B	2500 Hours ASTM G53-84 UVB Lamp	Five Years 45° F	Five Years 45° AZ
1	6	6	7	6	6
2	4	4	4	5	7
3	7	5	2	2	1
4	3	7	1	1	4
5	8	8	8	3	8
6	2	1	5	7	3
7	1	2	3	4	2
8	5	3	6	8	5

NOTE: In this ranking, 1 is poorest, 8 is best.

TABLE 7 RANK CORRELATION COEFFICIENTS (*r*) BETWEEN ARTIFICIAL ACCELERATED AND EXTERIOR EXPOSURES OF RETROREFLECTIVE SHEETINGS

Correlation Pair	r
2500 hr G23 and 5 yr F45	-.17
2500 hr G23 and 5 yr AZ45	.43
3000 hr G26 and 5 yr F45	-.62
3000 hr G26 and 5 yr AZ45	.50
2500 hr G53 and 5 yr F45	.57
2500 hr G53 and 5 yr AZ45	.69
5 yr F45 and 5 yr AZ45	.19

necessarily be used to predict how a product will perform in another environment.

Use of Artificial Accelerated Testing

Artificial accelerated testing can still serve as one of the many tools used to test retroreflective sheetings. However, the significant variability inherent in these tests must be taken into account. Replicate samples (for example, Federal Specification LS300-C requires exposure of three replicates) of each lot being tested must be used. Use of multiple replicates allows evaluation of results by using statistical techniques of such as analysis of variance. However, a control lot or reference material must be exposed with each series being tested to compensate for the variability inherent in the test. The control should be a material of known performance in artificial accelerated and exterior exposures. This is especially important if one is attempting to compare exposures between different units of the same type or between exterior exposure periods. If these precautions are taken, artificial accelerated testing can be used to help assess the durability of materials having similar composition and construction.

The large variability in artificial accelerated tests coupled with the year-to-year differences in climate at a single exterior exposure site make development of "acceleration factors" used to extrapolate exterior performance from artificial accelerated results a meaningless exercise. Comparing performance within a series using rank correlation techniques (*11*) is a promising approach to evaluating exposure data and may assist in the development of artificial accelerated test cycles that will some day provide more realistic assessment of durability.

Obtaining Accelerated Exposure Results for Retroreflective Sheetings

The results described in previous sections show that standard or commonly used artificial accelerated exposure tests are not satisfactory for predicting long-term exterior durability for retroreflective sheetings. However, one is still left with the problem of obtaining reliable indications of long-term durability in a shortened time frame. The use of outdoor exposures at a 45-degree angle facing the equator is a commonly used practice for obtaining "accelerated" outdoor exposures and probably stems from early work in temperate latitudes where 45-degree exposures are the optimum angle to maximize total UV stresses. Zerlaut (*12*) has shown that samples exposed horizontally in humid southern climates receive 10 percent more solar radiation than those at 45 degrees. However, experiments evaluating alternate exposure angles have shown no significant increases in failure rates relative to those seen in 45-degree exposures (*13*).

Materials exposed at 45-degree angles receive significantly higher levels of each of the primary stresses that produce polymer degradation (UV, moisture, and temperature) than those exposed vertically. Results from solar UV measurements (*14*) between 300 and 400 nm for a south-facing 45-degree angle and vertical exposures are shown in Table 8. One can see that samples exposed at a 45-degree angle receive 50 percent more solar UV annually and 74 percent more during the warmer summer months than those exposed vertically.

Table 9 summarizes results (*1*) from time of wetness measurements made on samples exposed vertically and at a 45-degree angle. Over a 12-month exposure, samples at 45 degrees are wet 47 percent longer than those exposed vertically. During the summer months (April–September), a 45 degree orientation increases wet time by 32 percent. In addition, there are many more wet and dry cycles that could produce mechanical stresses due to expansion and contraction as polymers swell with absorbed moisture and then shrink as moisture evaporates.

There is little published research comparing temperatures between angled and vertical exposures. For 3 weeks during April–May 1986, black panel temperatures for vertical and 34 degrees (latitude angle) were continuously monitored in Phoenix, Arizona. Table 10 summarizes the results for several times and shows that during midday, the 34-degree angle black panel temperature is typically 12°C higher than that for the vertical orientation. It is recognized that this temperature difference will double the rate of many chemical reactions, including those involved in polymer degradation. Long-term studies comparing temperatures of retroreflective sheetings at 45-degree angles and vertical exposures are now under way in several locations.

The results presented in Tables 8–10 illustrate how 45-degree angle exposures increase the stresses producing polymer degradation but do not provide an indication of the increase in failure rates for materials exposed at 45 degrees. In order to estimate the effect of exposure orientation on failure rates for retroreflective sheetings, multiple samples of a model sheeting based on an aminoplast cross-linked polyester polymer were exposed during 1986–1987 in Phoenix and Miami. Twelve samples were prepared for each exposure condition and one sample was recalled each month for testing. From plots of property versus exposure time, failure times were determined for retroreflectance loss, gloss loss, and yellowing. Failure time was defined as the time to 50 percent loss of retroreflectivity or 60-degree gloss, or time to maximum yellowing as measured by ASTM D1925 Yellowness Index. Table 11 summarizes failure times for each property in each exposure orientation.

The acceleration achieved by 45-degree exposures is generally near 2:1 but depends somewhat on the property being monitored. This agrees with the work of Yamasaki and Blaga (*15*), who reported that 45-degree exposures produced a 2:1 acceleration in loss of tensile impact strength for polyvinyl chloride relative to vertical exposures. Note that solar tracking exposures did not produce faster retroreflectivity loss or yellowing relative to static 45-degree exposures for this model sheeting. The only increase in failure rate for the solar tracking exposures was for gloss loss in Arizona.

TABLE 8 SOLAR UV FOR 45-DEGREE AND 90-DEGREE SOLAR EXPOSURES (*14*)

	MJ/m^2 Solar UV (300-400nm)	
Time Interval	90°	45°
12 months	131	197
summer (Apr-Sept)	76.5	133

TABLE 9 TIME OR WETNESS FOR 45- AND 90-DEGREE SOLAR EXPOSURES (*2*)

	90°			45°		
Time Interval	Wet Time (hours)	% of Total	# of wet/dry cycles	Wet Time (hours)	% of Total	# of wet/dry cycles
12 months	1875	21.3%	345	2753	31.3%	407
summer (Apr-Sept)	913	26.2%	188	1204	37.8%	240

TABLE 10 BLACK PANEL TEMPERATURES AT 34 AND 90 DEGREES

Time	Orientation	Mean Temp (^{o}C)	Standard Deviation	F-Ratio for ANOVA[1]
8AM	air temp	20.6	4.7	21.96
	90^{o}	24.8	4.8	
	34^{o}	30.3	6.3	
10AM	air temp	24.6	4.8	81.98
	90^{o}	33.1	4.6	
	34^{o}	44.5	7.0	
Noon	air temp	27.5	4.5	152.80
	90^{o}	39.5	4.6	
	34^{o}	51.9	5.9	
2PM	air temp	29.3	4.5	124.85
	90^{o}	39.7	4.3	
	34^{o}	51.9	6.1	
4PM	air temp	29.1	5.1	30.41
	90^{o}	34.6	6.0	
	34^{o}	43.1	8.0	

[1] For this analysis, to be 99% confident that the means are significantly different, the critical F ratio is 8.0166.

CONCLUSIONS

Artificial accelerated exposure tests are inadequate for assessing durability of retroreflective sheetings because they are poor replications of exterior exposure conditions and produce highly variable results for identical samples exposed in equivalent devices. Performance rankings of retroreflective sheetings exposed in standard artificial accelerated tests correlate poorly with those obtained in exterior tests. Accelerated indications of retroreflective sheeting durability can be obtained using 45-degree exterior exposures. These exposures produce higher levels of solar UV, moisture, and temperature than those in a vertical orientation and typically accelerate failures by a factor of 2:1. Surprisingly, solar tracking exposures produced little increase in failure rate relative to static 45-degree exposures. Performance of sheetings in one location is not necessarily a good predictor of performance in another environment. Therefore, exterior exposure testing at sites representative of end use applications are necessary. Multiyear exposures are recommended to minimize seasonal and year-to-year effects that contribute to variability of results.

In order to obtain reliable results, exposure testing should always use several replicates of each material being tested and must include a control of known performance as a reference.

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TABLE 11 FAILURE RATES FOR MODEL RETROREFLECTIVITY SHEETING AS A FUNCTION OF EXPOSURE ORIENTATION

Exposure	Property	Time to Failure[1]	Acceleration Relative to 90° Exposure
Arizona 90°	COR[2]	6 months	
Arizona 45°		3 months	2:1
AZ solar track[3]		3 months	2:1
Arizona 90°	60° Gloss	9 months	
Arizona 45°		3 months	3:1
AZ solar track		2 months	4.5:1
Arizona 90°	Yellowing	8 months	
Arizona 45°		5 months	1.6:1
AZ solar track		5 months	1.6:1
Florida 90°	COR	9 months	
Florida 45°		4 months	2.2:1
FL solar track		4 months	2.2:1
Florida 90°	60° Gloss	10.5 months	
Florida 45°		6 months	1.8:1
FL solar track		6 months	1.8:1
Florida 90°	Yellowing	10 months	
Flordia 45°		6 months	1.7:1
FL solar track		6 months	1.7:1

[1] Failure is defined as 50% drop in coefficient of retroreflection, 50% gloss loss, or time to maximum yellowness index

[2] COR = coefficient of retroreflection

[3] East-west solar tracking, latitude angle (26° in Florida, 34° in Arizona) exposures

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