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Transportation Research Record 1352

Contents

Foreword

Effective management of the roadway maintenance resources; improving worker, pedestrian, and motorist safety in work zones; and maintaining a high level of service during adverse winter conditions are key elements in a highway maintenance program. The information presented in the 10 papers in this Record should be of interest to maintenance managers, traffic safety engineers, and snow removal managers in state and local organizations.

The subjects addressed include the integration of management systems into geographic information systems (GIS); a graphical method for determining maintenance activity intervals with the minimum life-cycle costs; pedestrian safety, traffic sign legibility, vehicle speed profiles, accident characteristics and frequency at entrance ramps, and effectiveness of steady-burn lights for traffic control in maintenance and construction work zones; effects of deicing salt on vegetation; and a benefit-cost assessment of road weather information systems.

Integration of Management Systems into Geographical Information Systems

Carl E. Kurt

The purpose of this paper is to illustrate the use of geographical information systems (GIS) and transportation-related management systems so that a comprehensive management tool for transportation engineers can be developed. The focus of the paper is on microcomputer-based systems. A network-level culvert ranking system was attached to two GIS to illustrate the procedures and approach. It was demonstrated that when data are stored in most standard electronic formats, a significant effort is not usually required to integrate them into GIS. Base maps are readily available for many of the systems because of the TIGER files used during the 1990 census. Examples were presented to illustrate how GIS can be used to enhance the results from one type of management system. Although each system is different, other systems would work in a similar way. The use of microcomputer-based GIS has proved to be a significant time saver and could be used to improve the productivity and capability of transportation officials in the future.

The use of management systems for large transportation systems has been a fertile area of development. The principal objectives of all these management systems have been to effectively manage these large transportation systems with finite resources, optimize their performance, and set or justify current maintenance budgets so that minimum overall costs can be achieved.

Each of these management systems requires data about the transportation system, models of costs and system deterioration for conducting a network-level ranking, and a maintenance budget for each item. Once the critical items are found, many of the management systems will actually select "optimum" project-level strategies for improving the overall transportation system.

All of these management systems (pavement, bridge, and culvert) require significant amounts of data to drive them. If they can be presented clearly, these data can be useful for the management of most transportation systems. However, using standard methods, it is often difficult to determine what information is valuable, what trends are present, and what decisions are correct.

With the advent of relatively inexpensive computer power at the disposal of the transportation engineer, software engineers have been able to make GIS available. The heart of all GIS is the linkage of data and a map of a particular area of interest. This attachment of information to drawing entities is an extremely powerful capability. With many GIS, this information can be interrogated, displayed on a computer-based map, and then interpreted by the transportation official.

The purpose of this paper is to illustrate examples that combine the features of one transportation management system and GIS packages to develop a comprehensive management tool for transportation engineers. The focus will be on GIS that were developed for microcomputer-based systems.

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GEOGRAPHICAL INFORMATION SYSTEMS

Several hundred GIS are available to transportation officials for mainframe, minicomputers or workstations, and microcomputers. A few systems will work on all three computer systems, but this feature is limited among GIS developers. Many systems developed for mainframe or workstation computers have excellent features, but they are relatively expensive and require the management of large systems before they can be justified from cost considerations.

For the management of many systems, the use of a microcomputer-based GIS is an excellent choice. These microcomputer-based systems may provide support of the network environment where information can be shared among different users. In some organizations, this sharing of information can be arranged using different strategies.

When evaluating GIS, it is critical to note that although features of the system are important, the cost of the data driving the systems is usually much higher than those of hardware and software. One approach to minimizing these costs is to take advantage of the Topologically Integrated Geographic Encoding and Referencing (TIGER) (*1*) files made available through the Bureau of the Census. These files were developed by the Bureau in preparation for the 1990 Census. They provide a wealth of information about geographical entities and districts, including street-level detail for every county in the United States. Each street, waterway, railroad, city or county boundary, and so on, is digitized and stored in a standardized electronic format. Some GIS have the capability of reading the TIGER file format directly, whereas other systems must go through a data-conversion process. Privately developed base maps are also available.

The size of these TIGER files will vary depending on the development and size of the area. Because of the large amount of information stored in the TIGER files, the precensus files for 105 counties in Kansas contain more than 483 megabytes of information. The information stored in these files, which are important to transportation officials and many GIS systems, includes the beginning and ending coordinates of each line segment and type of feature (street, water, railroad, etc.). In urban locations, the TIGER files also contain the name of the segment and the address range on the left and right side of the street segments. In rural locations, the files generally do not contain this street-level information.

The focus and experience with PreCENSUS TIGER files have been primarily in the rural areas of Kansas. Many mile-long sections could have as many as three segments defining them. If only one segment per section is desired, two segments must be removed and the third one modified to contain the entire section. It is also necessary to define the names and address range associated with each segment. This could require significant coordination between several departments within the agency. It is necessary if a coordinated effort occurs within the agency.

The updating of the basic TIGER map should be considered as a necessary task before implementing GIS in rural areas. When using the TIGER files, it is important to remember that they were developed to conduct a census and not to serve as base maps for GIS. They should definitely not be used for most engineering applications.

GIS are different from other graphic packages, such as CAD (computer-aided design), because information attached to each drawing entity is efficiently tied to the data base. Because GIS were specifically designed for this operation, they are much faster and more efficient than most CAD-based GIS. GIS must efficiently find the street name, address range, and entity type very quickly. Most CAD systems were designed to draw lines and circles efficiently.

The more complete GIS generally support three types of drawing entities. The simplest type of data element is one that contains information about a point or object. These data are generally stored in point files, typical examples of which are data bases containing information about bridges, culverts, or sign inventories.

The next level of drawing entity supported by most GIS is a line element, which could be a street, water segment, or railroad, as previously discussed. However, it could also be a pavement segment, sewer or water line, or telephone line. Not all GIS will permit additional information to be stored with these drawing entities. Obviously, for some applications such as pavement data, transportation officials must have this capability of storing information with the line element. In this way, street level features such as material type, roughness, deterioration status, and so on, can be stored.

The highest level of drawing entities supported by most GIS are area- or boundary-type elements. In some systems, these are called boundary files. Boundary areas are excellent for determining properties associated with geographical areas; for example, state and counties define areas.

Most GIS systems allow for the evaluation of information associated with a boundary based on data found in point files. For example, to determine the average bridge rating for each district in a county, GIS could look at the bridge ratings for each bridge in the county. Each bridge would be assigned to a district and averaged with all other bridges in the district. Many GIS can provide additional statistical information for areas such as maximum and minimum values of a particular data field.

One desirable feature found in most GIS is the capability to display data-base information using different colors and symbols. If emphasis of the average bridge ratings for each district is required, a process called thematic mapping will select a different color or pattern for each district based on the average bridge rating for the district. In many systems, the ranges for selecting these colors or patterns may be predetermined or user selected.

Thematic mapping can also be used for point files. For example, bridges with good ratings could be displayed in one color and those with poor ratings in another. With paper maps, this would be called a pin map.

This thematic mapping capability is possible with GIS because the maps are efficiently tied to the vast amount of information required to drive the system. Therefore, fast computers with large hard disks, large RAM (random access memory), and good graphic cards are necessary. Fortunately, 386 and 486 computers provide sufficient computer power for most applications.

Besides the obvious graphic capability, these systems must have an efficient access to a data-base system, which could use a standard data-base format, such as dBASE, or it could be a proprietary data-base system. There are advantages to both approaches. However, what is important is that either compatibility or the capability of interfacing with an existing data base must be provided. When GIS is not dBASE compatible, the interface is generally through ASCII and LOTUS worksheet file formats.

GEOCODING

One of the most important procedures in GIS applications is attaching the data base to the computerized map; for example, attaching a bridge to a particular spot on a map can be accomplished in several ways. The specific method selected to locate each data record is dependent on the location information in the data base, capability of the software, and information stored with the map.

Location information stored in the data base could be an address, highway mile post, latitude-longitude or other coordinate system. Most GIS use latitude-longitude as the coordinate system for defining the map. If so, they will use this system for the location of point files. If addresses are available and the software and map support an address method of geocoding, the software will calculate the latitude-longitude coordinates of each point in the data base based on the address and the map data base. These latitude-longitude coordinates are usually stored in the data base in two additional fields.

Some systems also support geocoding by theme files, boundary names, and pointing. A typical example of theme-file geocoding is a point file data base containing ZIP codes. A second data base (theme file) contains records with fields for ZIP codes, and latitude and longitude coordinates. The software will look in the theme file for the ZIP code and coordinates for each record in the original data base. These coordinates will be stored in the original data base.

Boundary names can also be used to geocode a data base that contains a field with the boundary name. If the boundary map includes a field with the same boundary names, the original data base could be geocoded on the basis of the geometry of the boundary. Unfortunately, most systems will place all point file records in the boundary at the same location, usually the boundary centroid.

The last method of geocoding a point file is by pointing to the map location with a mouse or digitizing tablet. This method is simple but is labor intensive and prone to operator errors.

APPLICATION TO A NETWORK-LEVEL CULVERT RANKING SYSTEM

A network-level culvert ranking system was recently developed by Kurt and McNichol (*2*). This system was developed using dBASE III+a data-base management system. Basically, this system took eight culvert parameters and conducted a network-level ranking of each culvert in the system. The culvert management system (CMS) was developed using a level of service concept that is also popular with most bridge management systems.

The original CMS data base was modified to add two fields to the data base so that the latitude and longitude coordinates could be stored. Because the local agency used a grid system for the county, a numerical (N/S) and alphanumeric (E/W) coordinate were already stored in the data base. A location description was also stored in the data base, but it was unrelated to the map data base and therefore was not used in geocoding.

Two different microcomputer-based GIS have been used by the author. The first is GISPlus, developed by the Caliper Corporation (*3*). The other is MAPInfo, developed by MAPINFO Corporation (*4*). Although the systems have similar features, they also contain features that are very different. One feature that both systems have in common is provision of a means for users to write their own applications so that the software can better meet their needs. GISPlus is capable of processing executable files written in most microcomputer-based languages. MAPINFO provides a *C*-based language (MAPCODE) so that users can develop applications that work within MAPINFO.

For this culvert application, a MAPCODE program was written to transform the local agency grid coordinates to a latitude-longitude coordinate system. To account for the curvature of the earth, the following transformation equations were used:

$$XCOORD = 95.056221 - \frac{XL}{69.171 * \mathrm{COS}38.738496} \quad (1)$$

$$YCOORD = 38.738496 + \frac{YL}{69.171} \quad (2)$$

where

$XCOORD$, $YCOORD$ = latitude and longitude of mapping system, degrees;
95.056221, 38.738496 = latitude and longitude of local coordinate system origin, degrees; and
XL, YL = local system coordinates, miles.

The local coordinate system origin was in the extreme southwest corner of the county. The positive directions of the local coordinate system are positive to the north and east. The difference in signs is because the latitude and longitude coordinates are positive to the north and west directions. The cosine term in the XCOORD term accounts for the fact that the number of miles per degree is smaller when traveling from the equator to the poles.

Because MAPCODE reads information directly from a dBASE data base, calculates latitude-longitude coordinates and stores these values in the data base, the transformation process had to be conducted only one time. With the latitude and longitude coordinates calculated, each culvert can be located on the existing computer base map of the local agency. Although a similar program was not written for the GISPlus package, it would not require a significant effort to provide identical capability.

Of course, the proof of the operation is to see a map of the local agency with the culverts displayed. Except for a few cases in which errors were observed in the original data base, all culverts were correctly located.

A brief review of the network level culvert ranking system will now be presented. The development work for this system has been presented in the TIGER/Line Precensus files (*1*). This system is based on a level-of-service concept and defines deficiency points for each culvert. These deficiency points are based on four different conditions: load capacity, hydraulic capacity, culvert width deficiency, and maintenance priority. Each condition had a priority ranking formula developed that is a function of culvert parameters, culvert parameter goals, and weighting factors. The total priority ranking formula has the form:

$$\text{Deficiency points} = \Sigma\, K_i f_i\,(a,b,c,d,\ldots) \quad (3)$$

where

K_i = weighting factors,
$f_i\,(a,b,c,d,\ldots)$ = priority ranking formulas, and
$a,b,c,d,\ldots$ = culvert parameters.

The deficiency point formula developed for each condition is as follows:

Load Capacity

$$CP = WC * (CG - SV)/345 * ADT * DL \quad (4)$$

where

CP = capacity priority;
WC = load capacity weighting factor;
CG = capacity goal, tons;
SV = single vehicle posting, tons;
ADT = average daily traffic; and
DL = detour length, miles.

The capacity priority (CP) may not be less than 0.

Hydraulic Capacity

$$HP = WH * (NF - NG)/365 * [(KF * ADT * DLh) + \$/\text{Flood}] \quad (5)$$

where

HP = hydraulic priority;
WH = hydraulic capacity weighting factor;

NF = number of flood days/year;
NG = number of flood days/year goal;
$KF = .062 * (SV - 3)^{.30}$;
ADT = average daily traffic;
DLh = detour length caused by flooding, miles;
SV = single vehicle posting, tons; and
$\$/FLOOD$ = average damage cost/flood day.

The hydraulic priority (HP) may not be less than 0.

Width Deficiency

$$WP = WW * [(WD^2 - WDG^2)/9380 - (WD - WDG)/338] * ADT \tag{6}$$

where

WP = width priority;
WW = width deficiency weighting factor;
WD = relative culvert width difference, ft;
WDG = relative culvert width difference goal, ft; and
ADT = average daily traffic.

The width priority (WP) may not be less than 0.

Maintenance Priority

$$MP = WM * (MC - MG)/365 \tag{7}$$

where

MP = maintenance priority;
WM = maintenance weighting factor;
MC = maintenance cost, \$/year; and
MG = maintenance goal, \$/year.

The maintenance priority (MP) may not be less than 0.

The total number of deficiency points, DP, assigned to a culvert is the sum of four components and is given from the equation:

$$DP = CP + HP + WP + MP \tag{8}$$

where CP, HP, WP, and MP have been previously defined.

The number of deficiency points assigned to each culvert is based on eight culvert parameters, four weighting factors, and the goals selected for culvert parameters. The four weighting factors were developed so that their value should be equal to one. However, the system is flexible enough so that a local agency could use alternate weighting factors if local conditions warrant.

The number of deficiency points assigned to a culvert is not limited. However, as the number of deficiency points assigned to a culvert increases, the capacity of the culvert to meet the goals assigned to it becomes limited. A culvert with 0 deficiency points fully meets all of the goals for that particular system.

A data base program was developed on the basis of this approach to assist the user in manipulating the culvert parameters, calculating the deficiency points, sorting the results, and displaying and printing the results.

Although this approach provides the user with vast amounts of information, it is difficult to really understand the data and how they affect the performance of the system. For example, where are the reinforced concrete box culverts located in the system? How many of them are structurally inadequate and where are they located? Where are the culverts with the highest number of deficiency points actually located? Although a transportation official can answer these questions by poring over the software output and a local map, there is a better way: the use of a microcomputer-based GIS.

The largest cost in using GIS is usually the collection of the data (application and base map). Because of the TIGER files available, the base maps, with sufficient accuracy, are readily available at nominal cost. Because the culvert data must be collected in order to drive the network level ranking system, the only cost of implementing a GIS application is the geocoding and computer software.

Now, how can GIS be used in a management application? In this particular county there were approximately 1,353 culverts under the responsibility of the local agency. There are 8 different culvert types in the agency. The distribution of culverts by type is as follows: 94 were corrugated metal arch (CMMAC-1); 607 were corrugated metal pipe (CMP-2); 84 were concrete arch (CO ARCH-3); 317 were reinforced concrete box (RCB-4); 93 were reinforced concrete pipe (RCP-5); 92 were simple span (SI SPAN-6); 55 were stone arch (ST ARCH-7); and 5 were Boiler Pipe (BP-8); and several other culverts of an unknown nature. The numbers are given so that the culvert type could be numbered numerically.

After these culverts are geocoded, they are displayed on a map, as illustrated in Figure 1. The road system has not been displayed for clarity, but the roads are evident based on culvert locations. The boundaries (city and townships) are displayed. Because the northeast corner of the county is urban, the local agency does not maintain the culverts in this area.

It should be noted that each culvert type is displayed with a different type of symbol. On a color monitor, each symbol was assigned a different color for clarity. Hopefully, this figure will give a feel for the system. All culvert types can easily be located on the system.

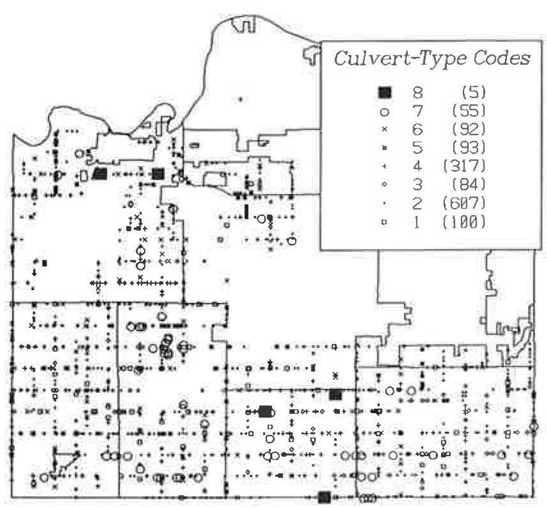

FIGURE 1 Culvert types in local agency system.

Although culvert type may be one way to evaluate the system, the purpose of the culvert network-level ranking system is to select the culverts that have the highest number of deficiency points. These culverts, based on this system, have high user and agency costs. Where are they located? Could this information be used to develop a comprehensive culvert management program? The use of GIS can help in these decisions.

For example, in Figure 2, the ranking of the system based on total number of deficiency points can be seen. Seventeen culverts have more than 100 deficiency points assigned to them. It can be noted that they fall on three or four east-west roads in the southern part of the county. These high-deficiency point ratings could be caused by high ADTs or low load capacity. Three of the worst culverts lie along a 2 mi section of a single road. Perhaps this is a region that warrants further investigation. If this is true, a different type of search of the data base could be conducted to test this theory.

With the search and filter features of most GIS, the original culvert data base can be reduced to locate those culverts that are reinforced concrete box culverts (Culvert Type 4). The thematic map considering reinforced concrete box culverts alone is presented in Figure 3. Again, note that many of the culverts with high deficiency points assigned to them are, in fact, reinforced box culverts.

It can now be learned whether the high number of deficiency points assigned to these reinforced concrete box culverts are caused by inadequate load capacity. Using the reinforced concrete box culverts, a thematic map is prepared based on the load-carrying deficiency point component only. Because load-carrying deficiency points are stored in the data base, a thematic mapping process on the reinforced concrete box culverts can be conducted using the load-carrying deficiency point field as the basis for the thematic map. These results are shown in Figure 4. To reduce clutter, the road system is not shown. Color is added to the screen to assist in evaluating the system.

When comparing the results shown in Figures 3 and 4, it can be seen that the majority of the total deficiency points is caused by insufficient load capacity of the reinforced concrete box culverts. It can also be seen where the most critical culverts are located. A plan for correcting this deficiency can now be developed.

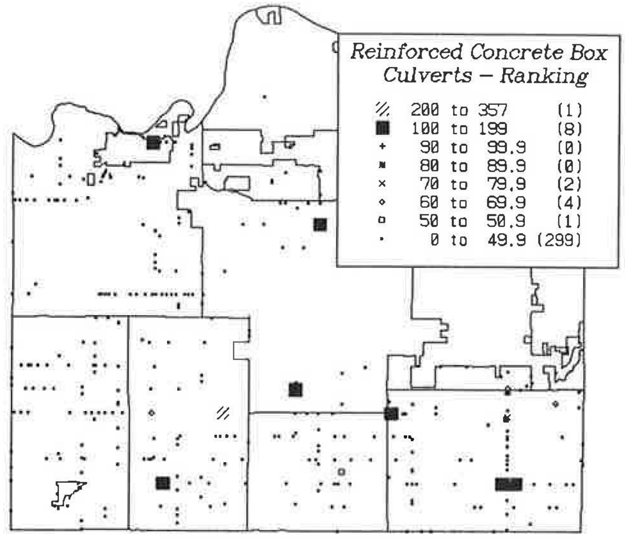

FIGURE 3 Thematic map of reinforced concrete box culverts only, based on total deficiency points.

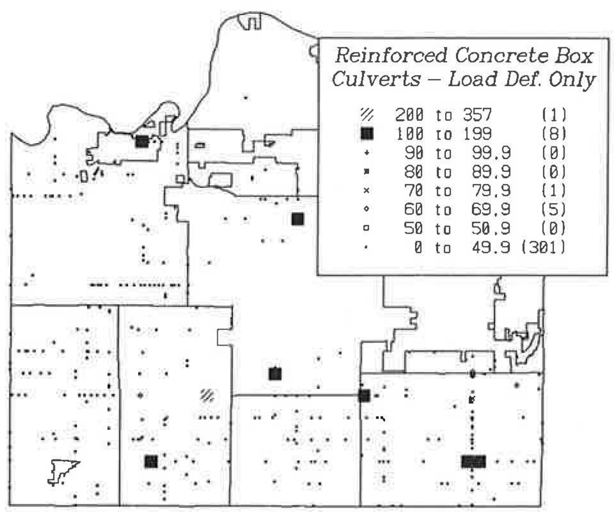

FIGURE 4 Thematic map of reinforced concrete box culverts only, based on load-carrying deficiency points only.

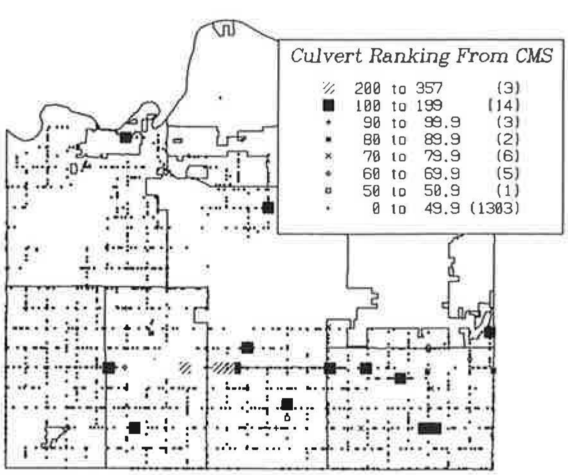

FIGURE 2 All culverts thematically mapped based on total deficiency points.

These examples of thematic mapping are fairly simple. GISPlus has the capability of showing deficiency point value for each of the four components as well as the total number of deficiency points for each culvert. This way, it would be apparent which culverts had a high level of deficiency and why they are deficient.

There are several advantages to this approach of analyzing the vast amount of data over more traditional methods of analysis. The first advantage is that because the data were already stored in electronic format, all of these examples were created in a matter of minutes. How long would it take to create these maps using manual methods? Obviously, much longer.

A second advantage is that because this information is available in electronic format, it can be used with other software packages. Both packages can create either an HPGL plot file or PCX file. This permits the output from either package to be used in many word processors or desktop publishing packages. WordPerfect and other word processors read these figures using either format, so the figures can be brought into a report. Then figures can be printed with the reports to a laser printer or other output device. Of course, if a color output device, plotter, color laser printer, or other color graphics terminal is available, then color-printed output is available for reports or presentations. This approach illustrates the flexibility built into most microcomputer-based GIS.

GIS AND VISUAL DISPLAYS

The last application to be covered is the integration of the data bases, GIS, and the visual display of graphical images. Because of recent advances of digital technology, it is now technically and economically feasible to store vast amount of information in digital format. Some applications of this technology will be presented.

With the culverts displayed on the map, it may be desirable to look at the status of a particular concrete box culvert. Earlier, a crew was sent out to videotape the culvert using standard videotape equipment found in the home. After the tape is brought into the office, the locations that are useful on the tape can be recorded and stored in a data base. Along with these locations, the structure number, or other identifying label, is stored.

A special application or procedure can then be written so that the user is prompted to point to a particular culvert. GIS will look up the identifying label, look into the tape data base, and send a signal to a special VCR to locate the portion of the tape containing this culvert. The videotape showing this culvert will be played on a television monitor. Thus, the transportation official can automatically review a videotape of every culvert in the system at the click of a button.

There are advantages and disadvantages to this type of visual display. The advantages include the capability of seeing the object without having to travel to the site. A documented status summary can be prepared if the videotape is updated periodically. Also, videotaping is relatively inexpensive. The primary disadvantage is that videotape displays are relatively slow. Unless a high use of this tool is anticipated, speed is not critical in many situations.

Another approach is to store the information in digital format on a CD-ROM. A demonstration at Caliper Corporation showed a map of a state highway system. When a segment of the highway system was selected, the author was asked how fast he wanted to travel this segment. A display of the entire segment was provided on a monitor simulating a speed of 80 mph.

Again, what makes this possible is the storage of vast amounts of information in digital format. GIS is the tool that makes all this possible and makes the system easy to use for the operator. In this application, GIS sends a signal to the CD-ROM player to locate the desired segment. The CD-ROM reader sends the signal to the monitor. The advantage of the CD-ROM technology is the vast amount of information that can be stored and accessed quickly. Its primary disadvantage is its cost.

CONCLUSIONS

In this paper, the use of GIS was illustrated in the use of a network-level culvert ranking system. However, GIS have also been successfully used in bridge, pavement, and sign inventory applications. When data are stored in most standard electronic formats, a significant effort is not usually required to integrate them into GIS. Base maps are readily available for many of the systems because of the TIGER files used during the 1990 census. The user should expect to spend some effort modifying these files for their specific applications.

Examples were presented to illustrate how GIS can be used to enhance the results from one type of management system. Although each system is different, other systems would work in a similar way. The use of these microcomputer-based GIS has proved to be a significant time saver and could help improve the productivity and capability of transportation officials in the future.

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Optimum Maintenance Standards for Roads in Developing Countries

H. R. KERALI

Economic appraisal models, such as the World Bank's Highway Design and Maintenance Standards Model (HDM-III) and the Road Transport Investment Model (RTIM2) developed by the Transport and Road Research Laboratory in the United Kingdom, are often used to determine optimum maintenance standards that result in minimum life-cycle costs. A simplified method of using concepts built into these models is described that can be used to determine optimum maintenance standards for roads in developing countries. The theory used in developing a graphical method of determining optimum maintenance standards is presented in this paper. This is defined as the maintenance interval required to achieve minimum life-cycle costs. The method uses charts initially derived from results of analyses conducted using either HDM-III or RTIM2. The maintenance intervals obtained from the charts have been compared with a range of maintenance standards modeled using both HDM-III and RTIM2. The results of the comparisons confirm that the graphical method gives optimum maintenance intervals with the minimum life-cycle costs. It is suggested that the method could be applied in developing countries in situations in which expert knowledge of HDM-III or RTIM2 is not locally available.

Road project appraisal models such as the World Bank's Highway Design and Maintenance Standards model (HDM-III) (*1*) and the Road Transport Investment Model (RTIM2) (*2*) developed by the Transport and Road Research Laboratory (TRRL) in the United Kingdom, have been used extensively to conduct economic appraisals of road projects in developing countries. The models calculate annual cost matrices made up of costs of construction, maintenance, vehicle operation, travel time, and other associated costs or benefits. The complexity and level of detail required to run these models often limit their use to engineers and economists with expert knowledge of the relationships built into the models as well as familiarity with computers. This level of expertise is sometimes not readily available in developing countries, and therefore simplified methods of using concepts built into the model are necessary.

HDM-III and RTIM2 are often used to determine maintenance and rehabilitation standards for roads in developing countries. This typically requires a number of discrete maintenance standards to be compared in order to select the alternative with the minimum total life-cycle cost. The previously mentioned procedure can be simplified to obtain a good estimate of optimum road maintenance standards applicable under a given set of circumstances. The procedure described in this paper is based on the comparison of costs of maintenance and rehabilitation with road user costs incurred on a road before maintenance in order to determine the optimum maintenance standard. This is defined as the maintenance interval or frequency that results in the minimum life-cycle cost. The word maintenance is used throughout this paper to include rehabilitation.

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RELATIONSHIP BETWEEN VOC AND ROAD ROUGHNESS

The relationships used in both HDM-III and RTIM2 were derived from pavement and traffic studies conducted in Brazil, India, Kenya, and the Caribbean. In all these studies, vehicle operating cost (VOC) relationships were derived from measured consumption of fuel, lubricating oil, tires and spare parts, as well as vehicle maintenance labor, crew wages, vehicle depreciation, overheads and interest on capital. A detailed examination of these relationships shows that they depend largely on road roughness. The VOC incurred on a road with fixed geometric and traffic characteristics is primarily a function of the pavement condition measured in terms of roughness. This relationship is illustrated in Figure 1 for five vehicle types derived using HDM-III. An average weighted VOC can be derived from this to represent the average cost per vehicle-kilometer incurred on a road, taking into account the traffic composition, as illustrated in Figure 2. The value of the weighted VOC depends on the geometric characteristics of the road as well as the traffic composition, and increases with road roughness. A good estimate of the total VOC incurred on a road with similar geometric and traffic characteristics can be obtained by multiplying the annual number of vehicles by the weighted VOC obtained from Figure 2 at the average annual roughness.

The horizontal roughness axis in Figure 2 can be replaced by a cumulative traffic axis or a time axis to represent the number of vehicles using the road over a period of time at the corresponding roughness level, as illustrated in Figure 3. The difference between the shapes of the weighted VOC curves in Figures 2 and 3 is caused by a nonlinear roughness progression with time and traffic loading. If the roughness progression rate remained constant, the shapes of the two curves in Figures 2 and 3 would be the same. Also shown in Figure 3 is the weighted VOC line for a 10 percent discount rate derived by applying discount factors to the weighted VOC in the corresponding years.

CUMULATIVE VOC PENALTIES

The total VOC incurred per kilometer at any point in time because of increase in road roughness can be estimated from

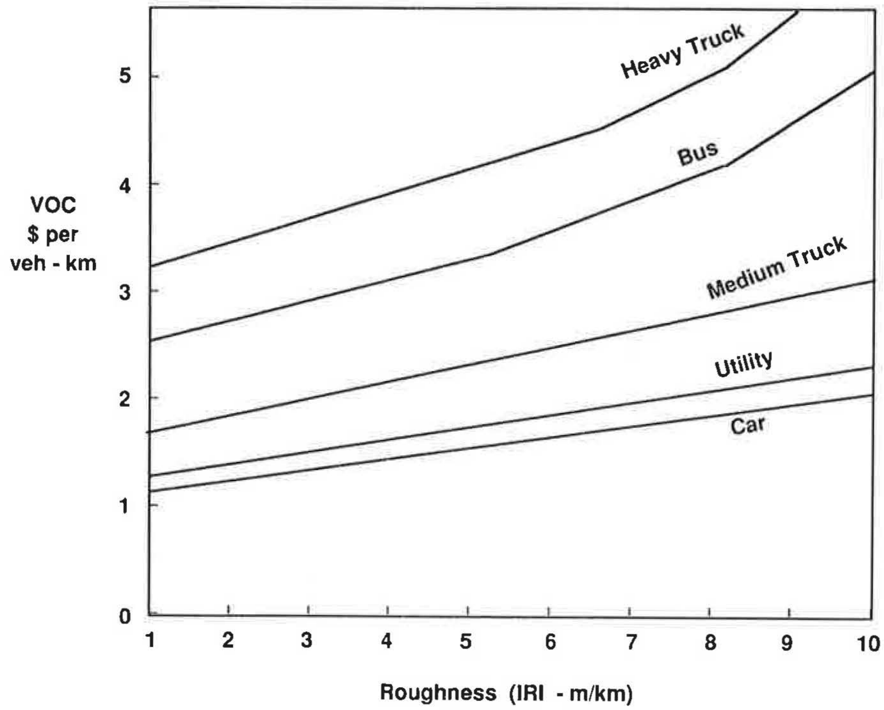

FIGURE 1 Effect of road roughness on vehicle operating costs.

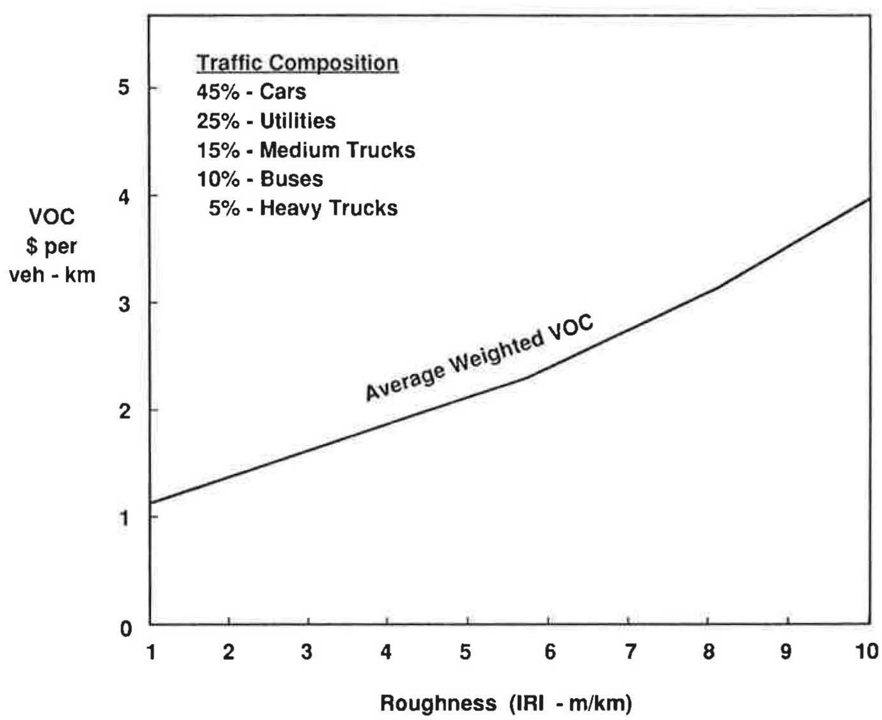

FIGURE 2 Variation in the weighted average VOC with roughness.

the area under the weighted VOC curve in Figure 3. This can be obtained either by mathematical integration, if the equation of the curve is known, or by graphical integration. The shaded area between the weighted VOC curve and a horizontal line drawn from the initial weighted VOC represents penalties incurred by vehicles operating at road condition worse than the initial roughness. This constitutes VOC penalties incurred by road users caused by failure to keep road roughness at the level immediately after construction. The optimum time for maintenance depends on the unit cost of maintenance and the rate at which these VOC penalties increase. The cumulative VOC penalties, when plotted against

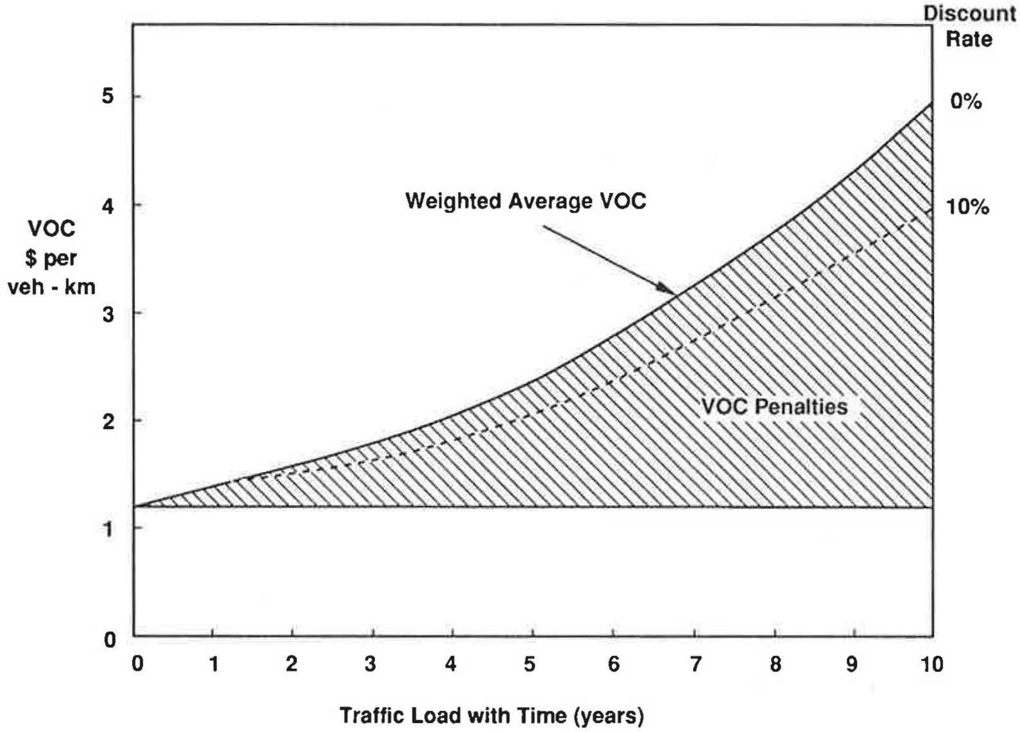

FIGURE 3 Increase in VOC penalties with traffic and time.

traffic loading with time, increase exponentially, indicating large cost penalties to road users when maintenance is delayed (see Figure 4). The optimum maintenance interval therefore depends on the rate of increase in VOC penalties and on the cost of maintenance or rehabilitation. When maintenance is applied, the benefits to road users will be equivalent to the VOC penalties that would otherwise have been incurred.

EFFECT OF MAINTENANCE ON ROUGHNESS

The shaded area in Figure 3 represents VOC penalties resulting from failure to control the increase in road roughness. This assumes that any maintenance applied will reduce roughness to the level immediately after construction. This in practice only applies when a pavement is reconstructed. Main-

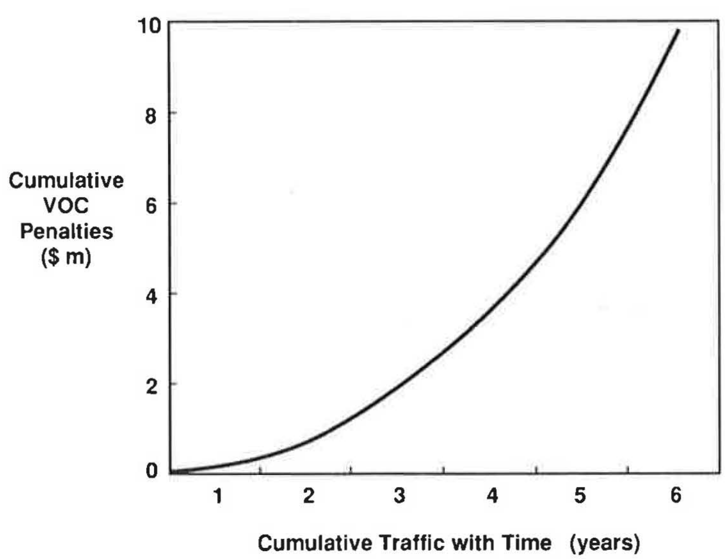

FIGURE 4 Cumulative increase in VOC penalties with traffic and time.

tenance treatments will usually reduce roughness levels by varying amounts depending on the effectiveness of the treatment. This implies that the VOC penalty area is not bounded on the lower side by a horizontal line starting at the initial weighted VOC. An inclined line, representing the VOC at roughness levels immediately after maintenance, marks the lower boundary. The VOC penalty area therefore depends on the efficiency of a maintenance treatment in reducing roughness. For example, in HDM-III, the effectiveness of an overlay in reducing roughness depends on its thickness. Thin overlays have less effect on roughness reduction than thick overlays, hence the benefits derived from a thin overlay will be less. The lower boundaries to the VOC penalty areas for 40-mm and 80-mm overlays and a horizontal line for reconstruction are shown in Figure 5.

VOC PENALTIES ON UNPAVED ROADS

The VOC penalties incurred on unpaved roads also depend on the efficiency of gravel road maintenance activities in reducing roughness. In HDM-III, the roughness immediately after grading (or blading) varies initially but attains a steady state after a few cycles. This steady-state roughness level can be used to determine the lower boundary to the VOC penalty area, as shown in Figure 6. It is therefore assumed in this paper that the lower boundary to the VOC penalty area for unpaved roads is given by the VOC at the average roughness after grading.

DERIVATION OF OPTIMUM MAINTENANCE INTERVALS

The optimum maintenance interval is defined as the cumulative number of vehicle passes after which a maintenance activity should be applied so that the total cost of maintenance plus VOC is a minimum. Shown in Figure 7 is the relationship between VOC penalties and cumulative traffic on a road that is maintained or rehabilitated after T vehicle passes. Because the vertical axis in Figure 7 represents total costs per kilometer, the unit cost of maintenance carried out after every T vehicle passes can be added, as illustrated. The relationship between maintenance frequency and the cumulative increase in VOC penalties depicted in Figure 7 forms the basis of the method presented in this paper for estimating optimum maintenance intervals. The optimum maintenance interval can be determined by varying the traffic interval T so that repeated maintenance applications will result in the minimum total of VOC penalties plus maintenance cost after several cycles.

A total cost line drawn from the origin to point A in Figure 7 represents the average rate of increase in the total of VOC penalties plus maintenance cost. The optimum maintenance interval must necessarily have the minimum total of VOC penalties plus maintenance cost over an extended period of analysis. This implies that the total cost line for the optimum maintenance interval must have the minimum gradient. This can be obtained by drawing a tangent to the cumulative VOC penalty curve from an off-set point on the vertical cost axis equivalent to the cost of maintenance or rehabilitation. The optimum maintenance interval is given by the point of inter-

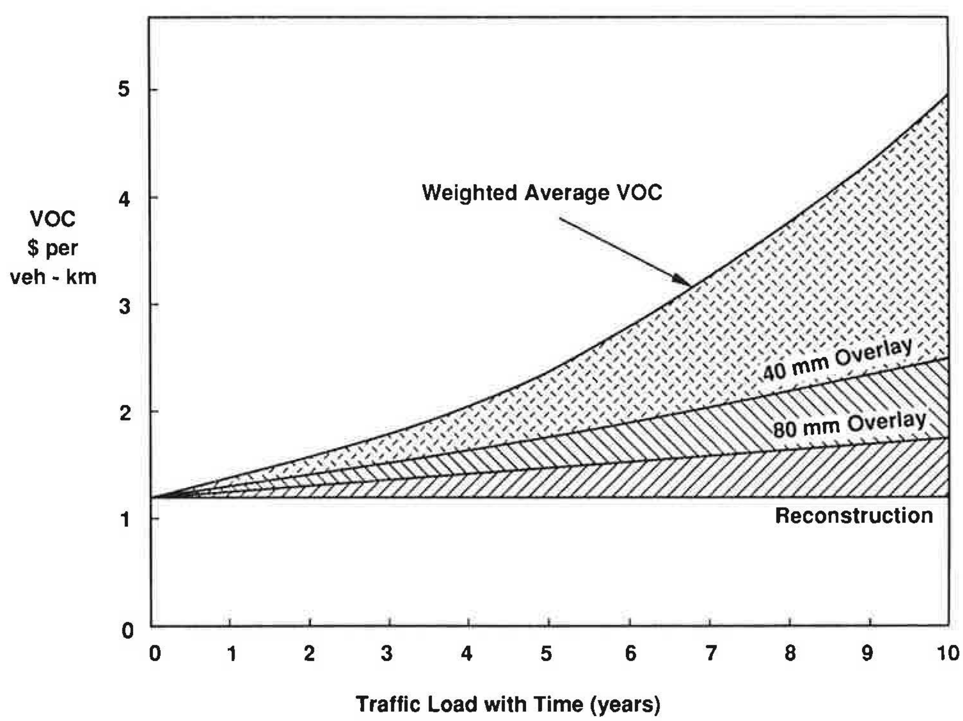

FIGURE 5 Changes to VOC penalty area caused by maintenance efficiency.

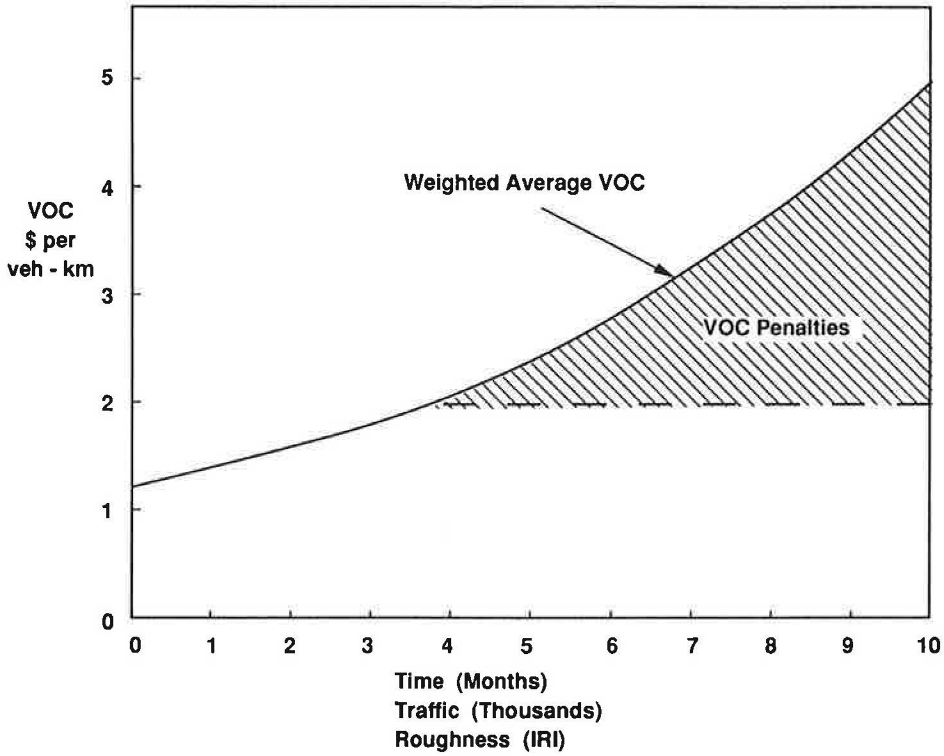

FIGURE 6 VOC penalty area for grading gravel roads.

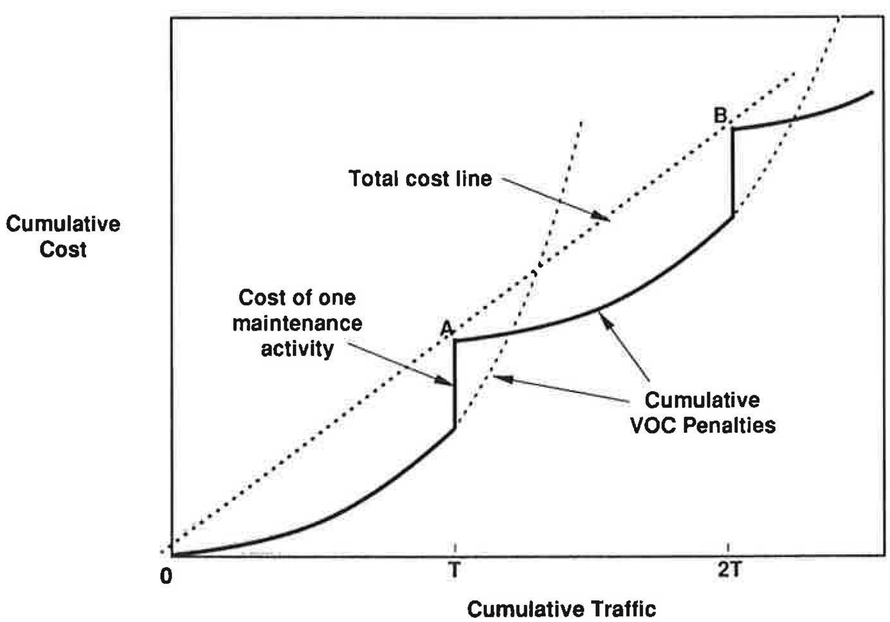

FIGURE 7 Cumulative cost of VOC penalties and maintenance cost.

section between the cumulative VOC penalty curve and a tangent drawn from an off-set point on the negative side of the vertical cost axis equivalent to the cost of maintenance, as illustrated in Figure 8 (*3*).

COMPARISON WITH HDM-III AND RTIM2

The method of deriving optimum maintenance intervals described in this paper has been compared with the results from similar analyses conducted using the two models HDM-III and RTIM2 with data from Costa Rica and Kenya. In each case a range of maintenance intervals, including those obtained using the graphical method, were analyzed using HDM-III or RTIM2. The cumulative increase in VOC penalties derived using HDM-III for a typical gravel road in Costa Rica carrying 100 vehicles/day is shown in Figure 9. For a unit cost of grading of $100 U.S./km, a tangent to the VOC penalty curve gives an optimum grading interval of approximately 2,800 vehicles. The results of comparisons made with a range of other grading intervals obtained from HDM-III for a 10-year analysis period are given in Table 1. It may be seen that the minimum life cycle cost is given by a grading interval of 3,000 vehicles, which is close to the optimum of 2,800 vehicles derived from Figure 9. If the unit cost of grading is doubled to $200 U.S./km, the optimum grading interval would increase to 4,300 vehicles, as shown in Figure 9.

A second analysis was conducted using RTIM2 to determine the optimum overlay interval for the heavily trafficked Nairobi to Mombasa road in Kenya. The VOC penalty curve with the overlay tangent drawn at an off-set cost of Shs 582,000 K./km ($36,500 U.S./km) is shown in Figure 10. This gives an optimum interval of 1.95 million vehicles, or 5.4 million equivalent standard axle loads (ESAL) in both directions for a 50-mm overlay. A series of RTIM2 runs were conducted to test this against a range of selected overlay intervals. A summary of the results is given in Table 2. The results confirm that the overlay interval of 5.4 million ESAL gives the minimum life-cycle cost.

A third analysis was conducted using HDM-III to derive optimum overlay intervals for a typical paved road in Costa Rica. It is shown in Figure 11 that a tangent drawn to the cumulative VOC penalty curve from a unit overlay cost of $40,250 U.S./km gives an optimum overlay interval of 9 million vehicle passes equivalent to 1.26 million ESAL in 6 years. A number of time-scheduled overlay intervals were tested against the optimum derived from Figure 11. A summary of the life-cycle costs calculated over a 25-year analysis period using HDM-III is shown in Table 3. The table contents confirm that overlays applied at an interval of 6 to 7 years give the minimum life-cycle cost.

PRACTICAL APPLICATIONS

The simplified method of determining optimum maintenance intervals has been shown to give good estimates of optimum maintenance standards. It can be used relatively easily in situations in which it is not possible to conduct full-scale economic analyses with HDM-III or RTIM2. All that is required are charts of cumulative VOC penalties plotted against traffic, as illustrated in Figures 9 to 11. These can be derived once for groups of roads with similar geometric, environmental, and traffic characteristics, for example one class of roads in a given part of a country.

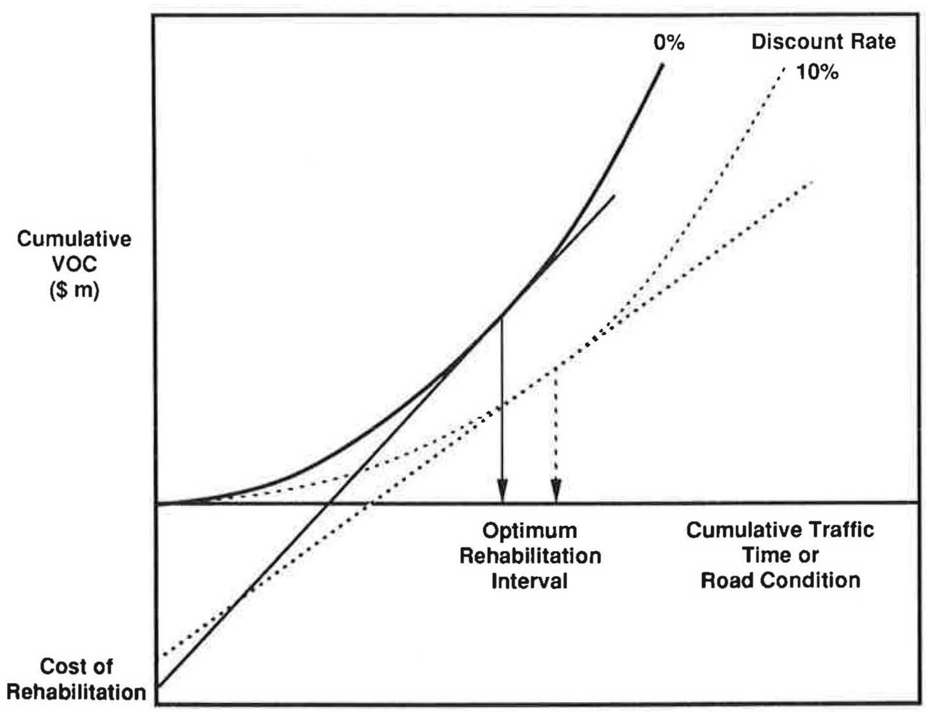

FIGURE 8 Derivation of the optimum maintenance interval.

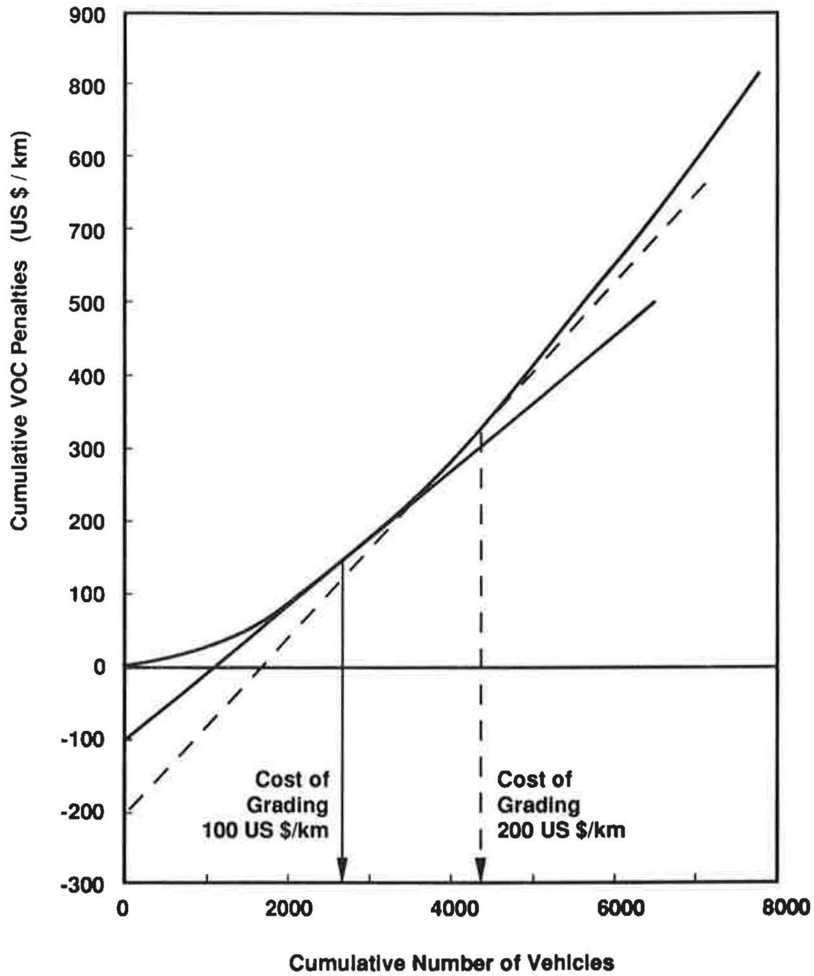

FIGURE 9 Derivation of the optimum grading interval.

SUMMARY OF PROCEDURE

The procedure for deriving optimum maintenance or rehabilitation intervals may be summarized in the following steps:

1. Divide the country into regions with uniform geographic characteristics based on terrain and environment; for example, mountainous, rolling, or flat area with dry, moderate, or wet climate. In addition, divide the road network according to road classes; for example trunk, primary, secondary, tertiary, and so on. This forms a matrix of road types for which typical maintenance intervals are to be derived.

2. For each combination of geographic region and road class, run HDM-III or RTIM2 to derive VOC/roughness relationships similar to Figure 1 using typical vehicle types found

TABLE 1 Comparison of Unpaved Road Maintenance Intervals

	Costs in thousands of US$ per kilometer over 10 years		
Grading Interval (vehicle passes)	Maintenance Cost	Vehicle Operating Cost	Total Life Cycle Cost
1000	54.6	511.2	565.8
2000	36.7	519.7	556.4
3000	26.7	528.9	555.6
4000	22.0	544.4	566.4
6000	16.8	565.6	572.4
8000	14.7	584.2	598.9
10000	13.2	600.4	613.6
12000	12.2	614.5	626.7
16000	11.0	634.0	645.0

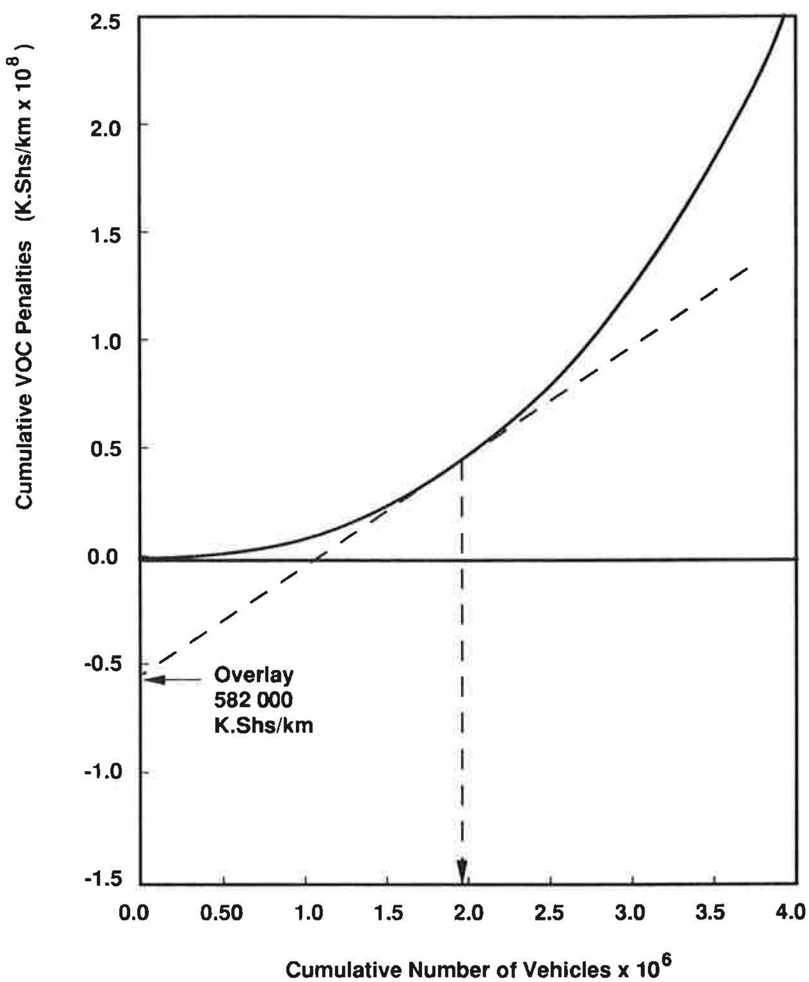

FIGURE 10 Derivation of the optimum overlay interval.

TABLE 2 Comparison of Overlay Intervals Using RTIM2

	Costs in million US$ per kilometer over 15 years		
Overlay Interval (million ESAL)	Maintenance Cost	Vehicle Operating Cost	Total Life Cycle Cost
4.5	0.235	1.759	1.994
5.4	0.219	1.772	1.991
6.0	0.200	1.795	1.995
8.0	0.172	1.838	2.010
10.0	0.145	1.907	2.052

in that part of the road network. The weighted VOC curve may then be derived for the traffic composition observed on individual roads or classes of roads at the discount rate applicable in the country.

3. Superimpose the cumulative traffic using the road at the corresponding roughness level. Determine the lower boundary to the VOC penalty area for the each type of maintenance treatment. The VOC penalty area represents additional costs incurred by road users for operating on roads in suboptimal condition. This is illustrated in Figure 3.

4. Calculate the cumulative increase in VOC penalties with traffic and plot this as shown in Figures 9 to 11 for each type of maintenance treatment. These represent cumulative VOC penalties incurred by road users when the road is not maintained.

5. The unit cost of maintenance or rehabilitation such as an overlay or a reconstruction can then be marked on the vertical cost axis in the negative direction on the corresponding VOC penalty chart. A tangent drawn from this point to the VOC penalty curve gives the optimum maintenance interval in terms of the cumulative number of vehicles, as illustrated in Figures 9 to 11.

6. The maintenance interval can be converted to a roughness intervention level by using observed progression rates,

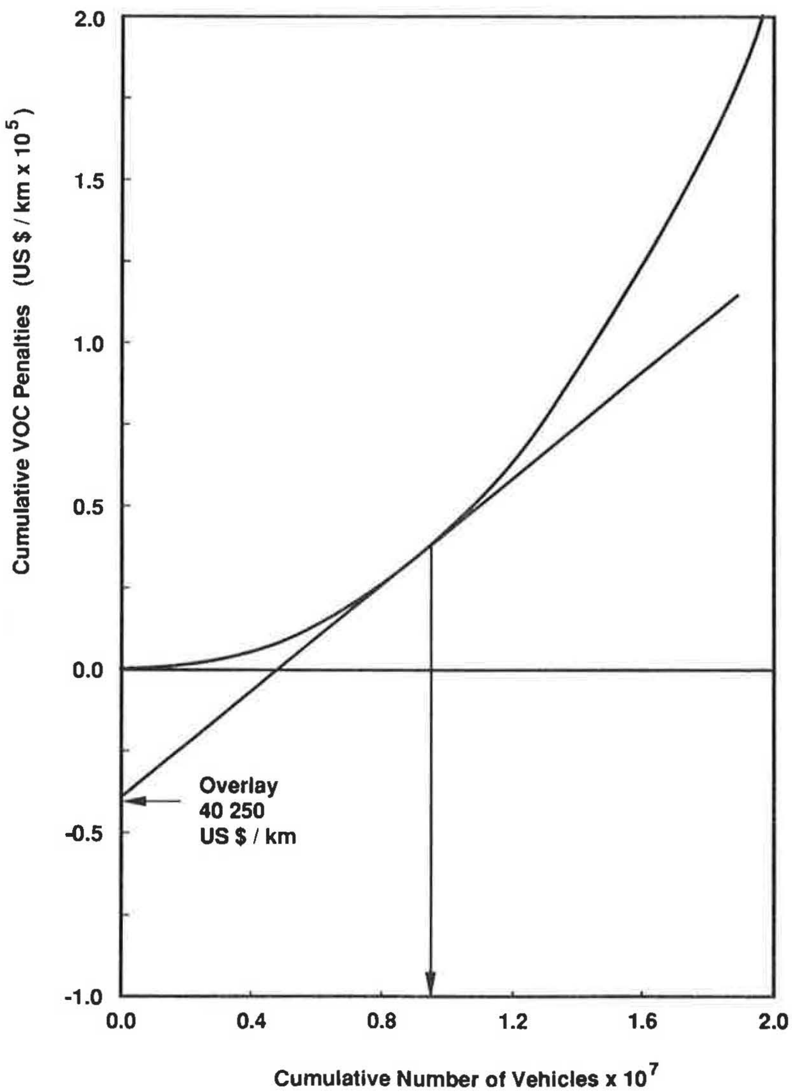

FIGURE 11 Derivation of the optimum overlay interval.

TABLE 3 Comparison of Overlay Intervals Using HDM-III

	Costs in million US$ per kilometer over 15 years		
Overlay Interval (Years)	Maintenance Cost	Vehicle Operating Cost	Total Life Cycle Cost
4	0.220	12.307	12.527
5	0.169	12.349	12.518
6	0.136	12.379	12.515
7	0.112	12.404	12.516
8	0.094	12.429	12.523
9	0.069	12.469	12.538
10	0.052	12.510	12.563

or more simply to a time interval using average daily traffic flows.

CONCLUSIONS

Presented in this paper is a simplified method for determining the optimum interval for maintenance activities on both paved and unpaved roads. It has been shown to give maintenance intervals with the minimum total cost when compared with other maintenance intervals modeled using HDM-III and RTIM2. It should prove particularly useful in developing countries where the lack of adequate computing facilities has often hindered the use of such management tools to plan road maintenance. With the method presented in this paper, the investment programs could be run only a few times to derive weighted VOC/roughness relationships, as shown in Figure 2. Such relationships will generally apply to all roads with similar geometric characteristics within a region or country. Similar figures can also be derived by using tables for calculating VOC published by the TRRL (*4*). A number of VOC penalty charts similar to Figures 9 to 11 may then be derived from the weighted VOC/traffic chart for each type of maintenance treatment and for the combination of road classes and geographic region in a country. Optimum maintenance intervals can then be obtained directly from the charts by applying the tangent method described in this paper with the unit costs of maintenance treatments applicable in each situation.

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Practices and Needs in Work Zone Pedestrian Safety

ERROL C. NOEL

Practices in controlling and protecting pedestrians in work zones are investigated in this paper. The information presented is based on research conducted in the development of the report *Work Zone Traffic Management Synthesis: Work Zone Pedestrian Protection* (*1*). The findings and recommendations are based on interviews, literature review, and field observations of highway, building construction, and maintenance projects in several cities. Good practice in protecting pedestrians in work zones is sporadic. Many cities, including high population centers, have no documented and comprehensive pedestrian safety standards for contractors to follow. Improvement of Part VI of the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) (*2*), to include information on how to accommodate and protect pedestrians in work sites, has the greatest potential for promoting good practice. Ideas for such improvement are presented.

Work zones located in areas with pedestrian traffic can jeopardize the safety of workers and pedestrians. Although much progress has been accomplished to date on work zone safety, the safe accommodation of pedestrians in work areas is frequently neglected by the construction and traffic safety communities. The right of pedestrians to safe passage is an important issue that must be considered in planning, design, and implementation of traffic control for work areas.

BACKGROUND

Pedestrian Right to Safe Pathways

Since 1981, the continuing neglect of pedestrian safety in work zones has been the subject of research papers (*3–6*) based on the efforts of the Federal Highway Administration (*7*) to better understand the issue. These papers (*3–5*) present photographic evidence of the neglectful treatment of pedestrians in urban work zones. Recent field observations (*1*) in the downtown areas of several major cities: Albany, Baltimore, Chicago, New York, Philadelphia, Richmond, San Francisco, Seattle, and Washington, D.C., clearly indicate that pedestrian safety is not a priority item in urban work zones. The abuse of pedestrian pathways during roadway rehabilitation and maintenance, building demolition and construction, and diverse utility operations in downtown areas reflects a continuing unawareness of pedestrian rights. The rights of pedestrians to access properties abutting work areas and to enjoy safe passage through and around construction projects is of no less importance than the right of safe passage accorded to motorists.

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Pedestrian Protection Standards

At present, there is no comprehensive national standard on pedestrian accommodation in work areas. Part VI of the Manual On Uniform Traffic Control Devices (MUTCD) (*2*) is held in high regard for its coverage of traffic control principles and devices for vehicular traffic in work areas, but it is grossly deficient in pedestrian protection material. Some states and localities rely on the fundamental principles presented in the *Traffic Control Devices Handbook* (TCDH) (*8*). However, the TCDH is not regarded as a standard nor as having any legal significance, and thus its principles have not been widely adopted into state manuals. TCDH provides the following guidelines on pedestrian control in highway work zones:

1. Pedestrians and vehicles should be physically separated (i.e., by barriers, barricades, or similar items).
2. Pedestrian walkways should be maintained free of any obstructions and hazards such as holes, debris, mud, construction equipment, stored materials, and so on).
3. Temporary lighting should be considered for all walkways that are used at night, particularly if adjacent walkways are lighted.
4. Walkways should be at least 4 or 5 feet wide, and should be wider in areas of high pedestrian activity.
5. All hazards (ditches, trenches, excavations, etc.) near or adjacent to walkways should be clearly delineated.
6. Walkways under or adjacent to elevated work activities such as bridges or retaining walls may require a protective roof.
7. Where safe pedestrian passage can not be provided, pedestrians should be directed to the other side of the street by appropriate traffic control devices.
8. Signs and traffic control devices should not be a hazard to pedestrians.
9. Signs located near or adjacent to a sidewalk should have a 7-ft clearance.
10. Where construction activities involve sidewalks on both sides of the street, efforts should be made to stage the work so that both sidewalks are not out of service at the same time.
11. In the event that sidewalks on both sides of the street are closed, pedestrians should be guided around the construction site.
12. Reflectorized traffic control devices are of little value to pedestrians. Warning lights should be used to delineate the pedestrians' pathway and to mark hazards as appropriate.

The two methods included in the TCDH for controlling pedestrians during mid-block sidewalk closure are shown in Figure 1. It should be noted that no typical pedestrian information signs are presented in the bypass illustration.

Large cities and counties traditionally rely on the limited provisions of state and local building codes for pedestrian traffic control in downtown work zones. For example, a permit is required for any excavation in the street or sidewalk in the city and county of San Francisco. The street excavation provisions of San Francisco (*9*) stipulate that contractors must provide and maintain safe and adequate passage of pedestrians and vehicles over and adjacent to excavations. However, these provisions seldom include procedural guidelines for the selection and placement of pedestrian protection devices in work zones that are not related to utility and building construction. In spite of its deficiency, the building permit review process is regarded by city officials as the primary opportunity to determine the adequacy of proposed pedestrian management systems for urban construction projects. Current use of pedestrian canopies and fencing are the result of progressive building codes and, to some extent, to the special efforts of contractors and developers as they attempt to minimize their tort liability.

LITERATURE REVIEW

Pedestrian Information Needs

Pedestrians need information to enable them to recognize work areas and potential hazards and to guide them safely through and around work zones. Sometimes the mere use of work area delineation devices is sufficient to alert pedestrians to the potential danger. In complex situations in which pedestrians are required to use bypasses and detours, a special effort must be made to provide positive guidance.

Research publications on methods for accommodating pedestrians in work zones are scarce. Except for a Federal High-

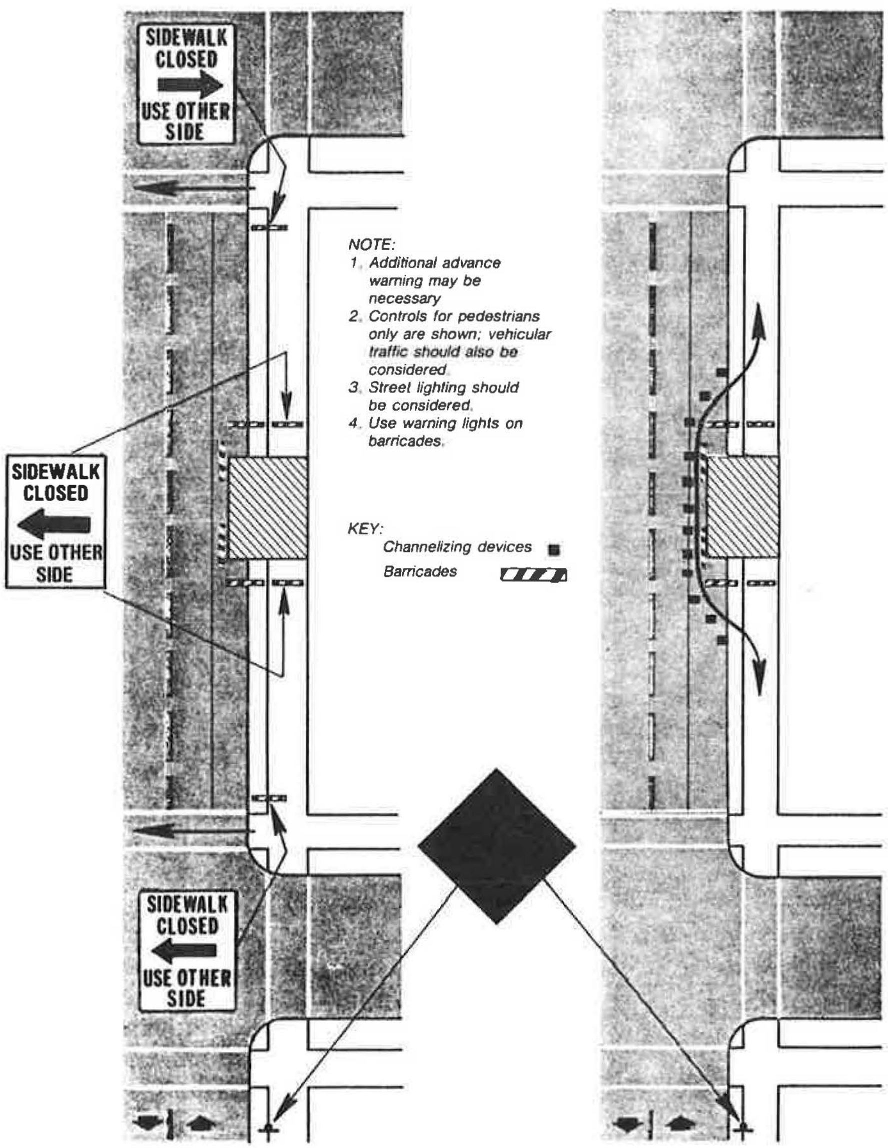

FIGURE 1 Pedestrian control for mid-block sidewalk closure (*2*).

way Administration study (*7*), which formed the basis for several publications by Chadda et al. (*3–6*), the subject of pedestrian safety in work zones has been virtually ignored. Chadda et al. (*3,4*) stated that positive guidance for pedestrians must be made up of four elements: advance information, transition information, work area information, and exit information.

Chadda et al. (*4*) described advance information as information placed at appropriate distances from the work zone that allows pedestrians to make timely decisions about alternative paths. The authors (*3,4*) noted that situations requiring pedestrian pathway blockage or detours are ideal for advance information and presented the following guidelines on the subject:

1. Advance information is needed if the pedestrian pathway is blocked or detoured,
2. Signs are most appropriate for this information,
3. Signs may be tailored to particular circumstances,
4. Signs should be strategically placed at points of decision, and
5. Pedestrian signals that no longer apply must be covered.

The authors (*3,4*) indicated that "Sidewalk Closed Ahead," "Sidewalk Closed—Use Other Side," and "Pedestrian Detour—Follow Arrow" are typical signed messages. The authors' illustration of a typical treatment for a corner sidewalk closure (*3,4*) is presented in Figure 2. The color code for pedestrian signs was not indicated. In another paper, Chadda et al. (*5*) noted that there is no uniformity in the design of pedestrian information. There was wide variation among the states on the colors, size, message, and placement.

Transition information allows pedestrians to find a safe path through and around work zones. This type of information is particularly important when the work activity restricts the width of pathways or requires a pedestrian bypass or detour. Chadda et al. (*3,4*) recommended the following guidelines on transition areas:

1. Transition to redefined or relocated pathways should be clearly delineated by either markings, tapes, tubes, cones, signs, wooden railing, barricades, portable concrete barriers, or other devices to provide positive guidance.
2. Physical barriers may be necessary to restrain pedestrians from using unsafe pathways and wandering into construction areas.
3. If the pathway is used at night, illumination or delineation with steady burn lights should be used.
4. All temporary crosswalks should be clearly delineated by signs and markings.

Work area information aids the pedestrian's passage through the work zone. This information is needed on all pathways except detours. Chadda et al. (*4*) recommend the use of devices that separate and protect pedestrians from the work activity and adjacent vehicular traffic and with clear delineation of pedestrian pathways. Markings, portable fences, barricades, flagging tape, cones, railings, barrels, drums, portable concrete barriers, and other devices were recommended for these purposes. The authors (*4*) noted that the selection of devices should be appropriate to the type of project and the nature of the hazards, and that pedestrians should be informed of pathway geometric and surface conditions that pose special hazards.

According to Chadda et al. (*3,4*), exit information becomes necessary only on new pathways involving bypasses and detours. Exit information can be communicated by signs and channelizations that direct pedestrians back to the original pathway.

Protection of Workers

A 1977 report (*10*) drew attention to the need for protecting workers in fixed and mobile work zones. The report noted that setting up fixed protection and working within the defined area expose employees to traffic hazards, and that consid-

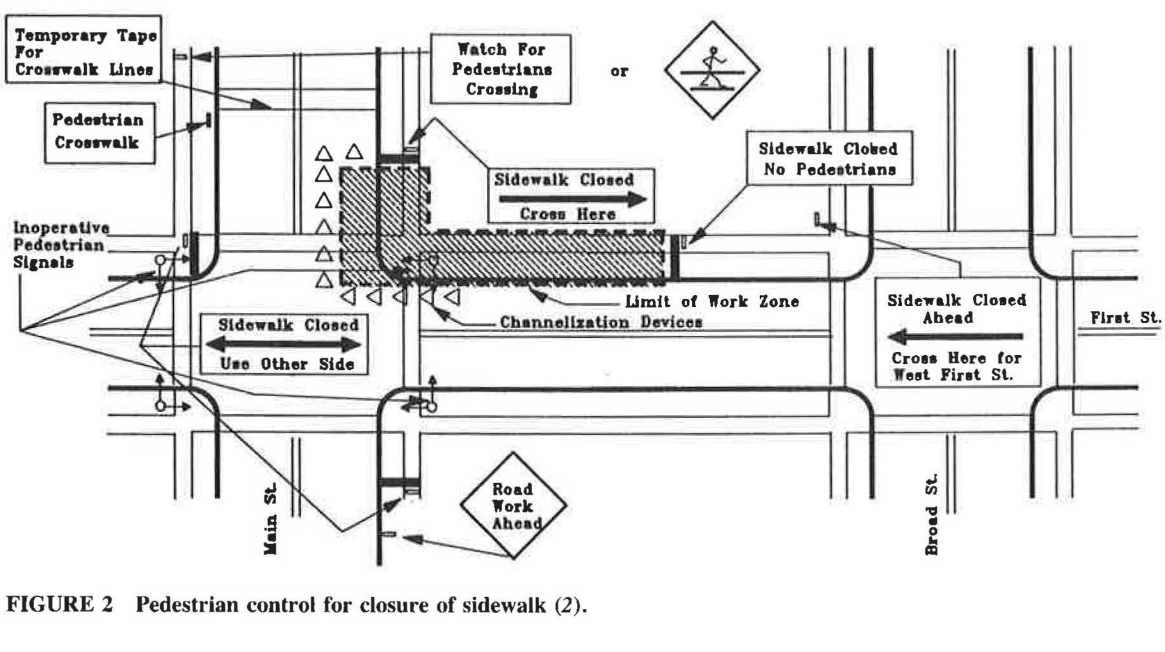

FIGURE 2 Pedestrian control for closure of sidewalk (*2*).

eration should be given to precast concrete safety-shaped barriers for positive guidance and protection of workers and motorists. For mobile operations, the report recommended the use of shadow vehicles equipped with energy absorbing devices as a buffer between workers on foot and the traffic approaching from the rear. Arrow panels were also recognized as being effective in providing advance warning to motorists.

An increasing number of fatalities involving maintenance personnel prompted Brackett (*11*) to investigate the effectiveness of the flagger's uniform in Texas. Two designs of an orange fluorescent mesh vest were developed and distributed in Texas for field use and evaluation: one involved the "11" striped pattern and the other a "W" striped pattern made with fluorescent reflective material. The study concluded that the "W" pattern was more recognizable and that stripes of brilliant yellow-green fluorescent reflective material should be used on traffic control vests. It was also recommended that the main body of the vest be made with red-orange fluorescent mesh material.

Gordon (*12*) also experimented with new designs for improving the effectiveness of the work-zone flagger's vest at nighttime. He advised that there is a need to be selective about the type of material for vest designs. Cotton, nylon mesh, and retroreflective bandoleer were identified as suitable materials for use during hot or cold weather and for providing good visibility during day or night. Vests made with white or silver-colored encapsulated or cube-corner reflective trimming were determined to be more effective at night. Gordon (*12*) discouraged the use of vertical or horizontal stripes for retroreflective patterns on vests. He advised that patterns that outline the flagger's figure are more recognizable than other designs at night. The author also recommended the 2 ft × 2 ft flag used to signal motorists be outlined with a 1-in. margin of retroreflective tape.

PEDESTRIAN PROTECTION PRACTICES

Despite deficiencies in pedestrian control information in work-zone manuals, occasional demonstrations of good pedestrian protection practices were observed in the states and cities visited: Albany and New York City, New York; Baltimore, Maryland; Chicago, Illinois; Lansing and Detroit, Michigan; Philadelphia and Harrisburg, Pennsylvania; Richmond, Virginia; San Francisco, California; and Washington, D.C. Observed efforts to ensure pedestrian safety in work zones are manifested in ways described in the following sections.

Building Codes

Contractors are expected to follow pedestrian safety requirements for work in public rights-of-way as stipulated by state and local building codes. These codes generally do not specify pedestrian safety treatments for diverse work situations. Contractors use their judgment in implementing pedestrian control measures. Local officials may review field sites to assess safety and recommend additional safety measures, if needed.

Building Permits

Local governments often require the review of building projects to ensure that adequate traffic control measures are taken during construction. The contractors rely on the limited information in work area traffic control manuals of cities, or the federal or state MUTCD. This is a routine practice in the large cities visited. Traffic engineering officials of Chicago, New York, and Philadelphia speak highly of this process, because it forces the contractors to submit traffic control plans—scaled detail or schematic, depending on the type of building—for approval before a building permit is issued. In large cities visited, all projects that use the public right-of-way (buildings, utility, and road work) are channeled through this review process. Specific devices, their message, size, location, placement, and period for application are approved by local traffic engineers.

Coordinated Management of Traffic

For major projects in San Francisco, a coordinated effort involving state and local traffic engineers, local police, and contractors is used when the disturbance caused by construction in urban centers—subways, freeways, skyscrapers, or rehabilitation of streetcar rails—is estimated to be major and long lasting. The coordination involves the participation of the police and traffic engineers in working with contractors in developing a mutually agreeable traffic control plan for all stages of construction. Subsequently, the police play a major role in enforcement.

Traffic Control Plans

Most states require traffic control plans for highway construction work. Pedestrian safety measures are detailed on traffic control plans where pedestrian traffic is anticipated. The California, Maryland, and Virginia Departments of Transportation use this procedure, although their work-zone manuals do not detail pedestrian protection measures.

General Specifications

Some states, Maryland and Virginia, for example, include a general statement about construction specifications or traffic control plans for highway projects indicating the need for contractors to provide for the safety of pedestrians. However, no details on pedestrian control devices are provided. Contractors are expected to use principles that are acceptable to state officials.

Coordinated Policy on Construction Safety

A coordinated safety policy that brings together several divisions of local government—maintenance, traffic engineering, building permits, police, and street cleaning—is used to ensure that individual public works subdivisions do not work against the interest of pedestrian safety in work zones. This ensures that all construction projects that are likely to disturb public space are subject to traffic safety review before work is initiated. San Francisco routinely follows this strategy.

In spite of the measures described, the actual practice suffers from a general lack of policy to ensure continuing enforcement. Hence, the research team was able to observe many field practices that do not reflect the review and enforcement policies of local governments. A chronic problem that exists at the local levels of government is the lack of training of those individuals who must approve traffic control plans and inspect the field setup for compliance: the lower the cost of the project, the less stringent are the measures to protect pedestrians. This explains why contractors doing curb, gutter, and sidewalk work often display little sensitivity for pedestrian needs in downtown areas. This problem appears to be worse in cities where there are no formal guidelines for protecting pedestrians in work areas, and where approval of traffic control plans is not required for certain types of short-term roadway maintenance projects. A lack of state and local specifications on traffic control devices for pedestrians has allowed room for contractors to be creative about the message, color code, and placement of pedestrian information signs. The following sections provide a sample of field practices.

Pedestrian Information Signs

Pedestrian information signs vary widely in message, size, color code, and placement. An assortment of observed signs and their respective messages and codes are indicated in Table 1. The colors of the worded messages are black, blue, green, red, red and black, and white. In addition, signs with lower formality were spray painted on portable concrete safety-shaped barriers. A flat nonreflective white background is most frequently used. Other observed background colors for pedestrian information signs were orange, yellow, and red. Combinations of black and red were being used to emphasize caution. Mounting height is also subject to wide variation.

Pedestrian Barriers, Canopies, and Fences

Barriers are used to protect pedestrians from work activities and to protect workers and pedestrians from vehicular traffic. The barriers are more prevalent in urban work zones when the construction activity is of long duration. The construction, demolition, and rehabilitation of buildings in downtown areas often require special efforts to ensure that work activities do not endanger pedestrians. To protect pedestrians from this type of danger, fences, canopies, and portable concrete safety-shaped barriers are being used.

Although several designs of fences and overhead protection structures were observed in all the cities visited, portable concrete barriers were less popular for that purpose. The naturally hilly topography of San Francisco has discouraged the use of portable concrete barriers for protecting workers and pedestrians in urban work zones. However, concrete barriers are often used in some cities, for example, Harrisburg, New York, and Philadelphia, as worker and pedestrian protection devices.

As indicated earlier, building codes are often the primary basis for the wide use of fences and canopied structures in

TABLE 1 Pedestrian Information Signs and Color Codes Used in Some Cities

Worded Message	Message Color	Background Color
1. "No Ped Crossing Use Crosswalk," with black arrow	Black	Reflective White
2. "This Stop Temporarily Discontinued Use Stop in Next Block," with black arrow	Black	Construction Reflective Orange
3. "No Bicycle Traffic Beyond This Point"	Black	Flat White
4. "Ped. Walk," with black arrow	Black	Flat White
5. "Sidewalk Closed Use Other Side," with or without black arrow	Black	Reflective Orange
6. "Sidewalk Closed Pedestrians Use Opposite Side of Street"	Red	Flat White
7. "We Apologize for the Inconvenience Please Follow Walkway to 49th Street," with blue arrow	Blue	Flat White
8. "West 49th Street Sidewalk Closed Please Use Other Side of Street," with blue arrow	Blue	Flat White
9. "Sidewalk Closed, Use Opposite Side of Street"	Red	Flat White
10. "Walkway," with red arrow	Red	Flat White
11. "Sidewalk Closed, Permit No. ————"	Red	Flat White
12. "Pedestrian Crossing," on diamond with black arrow	Black	Reflective Orange
13. "*Caution* Sidewalk Repair in Progress *Please* Pass with Care"	Red	Flat White
14. "Sidewalk Closed Please Use Other Side"	Black	Flat White
15. "Notice Sidewalk Closed Please Use Other Side"	Blue	Flat White
16. "Notice Sidewalk Closed Please Use Other Side"	Green	Flat White
17. "Sidewalk Closed Please Use Other Side"	Red	Flat White
18. "Sidewalk Closed" on diamond	Black	Flat White
19. "Pedestrian Walkway to 16th Street," with black arrow	Red	Flat Yellow
20. "Pedestrian Walkway to 16th Street," with black arrow	Black	Reflective Orange
21. "Sidewalk," with black arrow	Black	Reflective White
22. Impromptu "Walkway" signs spray-painted on concrete barriers	Any	Concrete
23. "Sidewalk Closed"	Red	Flat White
24. "Sidewalk Closed"	White	Flat Red
25. "Sidewalk Closed Caution"	Black & Red	Flat White
26. "Sidewalk Closed Pedestrians Please Use Other Side of Street," with blue arrow	Blue	Flat White
27. "Sidewalk Closed Please Use Pedestrian Walkway"	Red & Black	Flat White

urban building projects. The building construction industry generally follows the provisions of local codes. Traffic engineers are becoming increasingly aware that problems in the design of these devices can affect the capacity of walkways and limit sight distances at intersections. The city of Seattle, Washington, for example, requires fence corners at intersections to be made of chain-link material in driveways. Philadelphia, Pennsylvania requires the use of protective canopies in building construction as well as in maintenance activities above public thoroughfares to facilitate good visibility. Richmond, Virginia, has not yet documented its practice, but its officials no longer allow opaque construction fences or walls at intersections and, for example, window washing.

Delineation Devices

Methods for delineating pedestrian pathways include traditional devices such as cones, barricades, concrete barriers, orange construction tapes, and flashing warning lights for nighttime. Extensive use of pedestrian channelizing rails was observed in New York and San Francisco. Officials of both cities expressed satisfaction with the flexibility and performance of pedestrian channelizing rails such as those presented in Figure 3. These rails are used for pedestrian control in work areas, as well as for crowd and vehicular traffic control during emergencies and social events. Their design allows interconnection into a chain of any desired length, and they

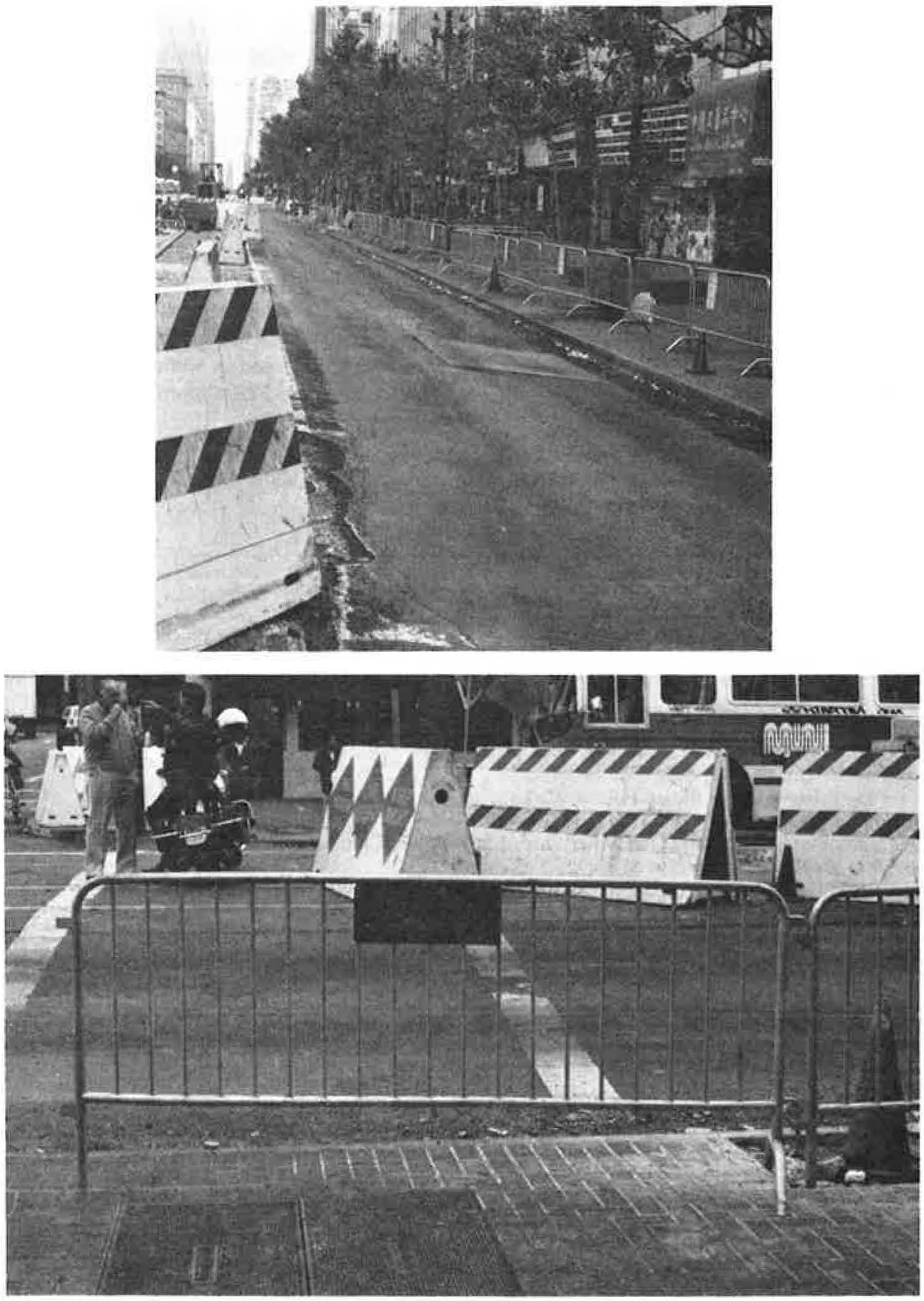

FIGURE 3 Pedestrian channelizing rails used in San Francisco.

are sturdy enough to discourage movement by pedestrians and vandalism by motorists. The design used by San Francisco is presented in Figure 3. New York City uses a similar pedestrian rail system. San Francisco does not paint its rails but chooses to accent them with traffic cones as needed. New York City paints its rails bright yellow.

Recently, construction safety fences made of orange plastic have been appearing on roadway work in urban areas. They are available in 4-ft × 160-ft and 5-ft × 160-ft rolls and require a number of posts for installation. However, in many field installations, drums, barricades, and cones are draped by the orange plastic mesh. It is not known whether the mesh, used as drapery over standard devices, is contributing to further negligence in delineating pedestrian pathways. But with its bright orange color, the mesh is easily visible and has the advantage of closing gaps normally associated with standard barricades.

ASSESSMENT

The assessment summarized below and the conclusions and recommendations discussed later are based on a review of work-zone traffic control manuals of a selection of cities and states and a literature review and field observation of a sample of cities.

1. The safe accommodation of pedestrians and cyclists in work zones is often neglected by state and local governments. This neglect is more severe at the local government levels (counties, cities, and townships) than at the state level. However, the majority of work zones that affect pedestrians are located in urban areas where local safety standards are more prevalent.
2. Although the *Traffic Control Devices Handbook* (TCDH) (*8*) presents some principles for accommodating pedestrians in work zones, many local traffic safety personnel are not aware of its existence. In addition, since the TCDH is not a national standard, there has been no movement to adopt its guidelines on pedestrian safety into local practices.
3. City officials have recognized the need for guidelines for accommodating pedestrians in work areas, but few localities have included written guidelines in their work-zone traffic control manuals. Many cities, including high population centers, have no reference material on their pedestrian accommodation practices, and consequently, no standards for contractors to follow.
4. There is evidence that state highway officials routinely review projects planned for areas with pedestrian traffic to ensure the adequacy of safety measures. However, a lack of concern about the quality and maintenance of pedestrian control devices was observed on many state highway projects. The unavailability of published state standards on the design and application of devices for controlling and protecting pedestrians in work areas may explain the nonuniformity in the design of signs used in some state-administered roadway construction projects in large cities.
5. The state MUTCDs are generally a reflection of the federal MUTCD and have a similar deficiency in their methods for managing pedestrian traffic in work zones. State officials appear to be cautious about adopting formal guidelines on matters that have not been detailed in the federal MUTCD.
6. The actual practices of the state officials do not reflect the lack of information on pedestrian safety in their work-zone manuals. The traffic control plan review process presents ample opportunity to determine whether pedestrian needs will be adequately accommodated.
7. There is very little uniformity in the design and application of pedestrian control devices. The impact of using different colors for the same signed message on different backgrounds is not an apparent concern among state and local officials.
8. Inadequate attention is given to the geometry and surface quality of temporary pathways. The needs of pedestrians with ambulatory handicaps are often neglected.

CONCLUSIONS

1. The traffic engineering community, contractors, and utility companies involved in building construction and road work need safety standards for accommodating pedestrians and protecting workers in work zones.
2. Improvement of Part VI of the MUTCD (*2*), to include information on pedestrian accommodation and worker protection in work zones, has the greatest potential for promoting sound practice at state and local government levels.
3. There is adequate information on effective practices for managing pedestrians in work zones that could be considered for the MUTCD. The TCDH (*8*) is a good start.
4. The abuse of pedestrian rights in work zones can be blamed, in part, on the fact that many types of roadway and building maintenance work escape inspection by city officials or are reviewed and approved by inadequately trained personnel.
5. Work zones involving building construction and maintenance are common in urban areas. They frequently expose pedestrians to hazardous situations. Future improvement in Part VI of the MUTCD should cover pedestrian protection in such work zones.

RECOMMENDATIONS

1. Part VI of the MUTCD should be updated to include material on the principles for accommodating pedestrians in work zones, a standard set of traffic control devices and any caution regarding their use, a set of standard signs and guidelines for customized signs, delineation, illumination, and typical illustrations covering: (a) mid-block sidewalk closure with detour and bypass through pathways along the curb parking lane or through adjacent property, (b) corner closure of sidewalk, (c) crosswalk closure, (d) fencing near intersections, and (e) canopies for protecting pedestrians from the danger associated with overhead work.
2. Current practice leans toward the use of black and white signs for pedestrian information. There is a need to determine whether this practice should be officially encouraged, because these colors have a regulatory significance. There is no evidence that regulations were considered in their selection.
3. Figure 6-24 of the TCDH presents a typical application for controlling pedestrians in work zones. This figure details only a mid-block closure and provides no guidelines on the size and color of signs. This figure should be improved for inclusion in the MUTCD.

4. Sections 6B-5 through 6B-39 of the MUTCD deal with regulatory and warning signs for work zones. Standard designs for a selection of pedestrian signs could be included in these sections. This type of information would aid in standardizing the color codes for pedestrian signs. In practice, the majority of the pedestrian signs used in work zones are for warning. In upgrading Sections 6B-5 through 6B-39, there is need to determine whether there are standard regulatory signs that could be included. The text in these sections should be edited, where necessary, to reflect the added pedestrian information.

5. Section *C* of the MUTCD covers barricades and channelization devices from the perspective of motorists. That text should be modified to include pedestrians. Devices that are also applicable to pedestrian safety should be identified in the appropriate subsections. For example, barricades, drums, cones, and barriers should be identified as being suitable for channelizing pedestrian traffic. This section may also be the place to introduce and discuss other pedestrian channelization devices such as fences and pedestrian rails.

6. There is a need to determine whether Section 6A-5 of the MUTCD, which discusses fundamental principles, should be expanded to include principles that relate to pedestrian safety, or whether a separate section should be created for this purpose. It should be noted that some of the principles articulated in Section 6A-5 also apply to pedestrians and should not be duplicated. However, because there is a need to make the traffic safety community more sensitive to pedestrian needs, a separate section following Section 6A-6 should be considered. Its caption should include the word "pedestrian," and its contents should be oriented toward a number of briefly stated principles that apply only to pedestrians and are excluded from Section 6A-5. The text should cross-reference appropriate illustrations and other relevant material in the entire manual.

7. Workers are as vulnerable as pedestrians to work zone dangers. Practitioners who are far removed from the work site need to be made aware that workers are exposed to two dangers: the work activity and errant vehicles. Although many of the protection devices for pedestrians may apply to workers, a special section following the treatment of pedestrian-protection principles in the MUTCD should address principles that also apply to workers. Reference should be made to sections of the MUTCD that deal with flagger protection and the names and application of special worker protection devices. Typical situations that may warrant special worker protection measures should be noted. Concrete barriers, their connectors and anchorage, should receive special mention. There should be a brief discussion of worker dress, measures to ensure good visibility, and the need for organizations involved in highway work to maintain a continuing effort to promote work zone safety practices.

8. Section *E* of the MUTCD deals with lighting devices. The use of illumination and warning lights for pedestrian safety should be recognized. The illumination needs of detoured and canopied temporary walkways should be discussed.

9. Although retroreflectivity is not often a characteristic of pedestrian signs, the use of fluorescent material should be encouraged to improve visibility under all lighting conditions.

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Maximizing Legibility of Traffic Signs in Construction Work Zones

DAVID A. KUEMMEL

The Wisconsin Department of Transportation contracted with Marquette University to research improvements in construction work zone signs. The RIGHT/LEFT LANE CLOSED and ROAD CONSTRUCTION series were selected because they are the most difficult to improve without increasing sign size above a 48-in. diamond. All signs used high-intensity retroreflective material. Twelve test signs were selected for day and night viewing by 46 observers under age 65, and 38 observers aged 65 and over. Experimental messages included rearranged legends (three to four lines) substitution of WORK for CONSTRUCTION, 18 percent stroke width increases on the inside of letters without increasing letter width, and use of Series *E* letters upper and lowercase [instead of all capitals as required by the *Manual on Uniform Traffic Control Devices* (*1*)]. The conclusion of this study was that few improvements can be made in the LANE CLOSED series without more drastic changes than those tested. In the ROAD CONSTRUCTION series, substantial improvement can be made by substituting WORK for CONSTRUCTION and increasing letter size. The 18 percent stroke-width concept resulted in no improvement and some reduction by day for younger observers. The most promising finding is the improvement possible with Series *E* letters because of their 20 percent increase in stroke width. Less loss of night legibility distance compared with that of day occurred with this alphabet than with any other. Further research substituting tenths of a mile for feet, which would allow larger letter size, is recommended, and field experimentation with Series *E* letter series under Federal Highway Administration requirements is recommended.

The Wisconsin Department of Transportation (WISDOT) has been striving for several years in its research to improve the conspicuity and legibility of its construction work zone (CWZ) signing. Several years ago, to improve conspicuity, WISDOT experimented with all high-intensity orange sheeting on state highway construction and maintenance projects. Since then, no decision has been made but contractors have begun to use high-intensity sheeting regularly, particularly on urban freeways.

Because of the brighter reflective material, WISDOT noted that new signs exhibited a phenomenon characterized as "irradiation" or "overglow," which they and others (*2,3*) had observed tended to reduce legibility of the signs at night, particularly in a rural setting with bright headlights. Although the improved conspicuity of high-intensity retroreflective material has obvious advantages, this research was conducted to explore how legibility could be improved, particularly for the older driver, because there is a growing population for which even current standard highway alphabets are not adequate (*4*). The testing was therefore limited to high-intensity materials.

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Although the research project did not include a literature search, the conclusions reached by several individuals were carefully reviewed for clues to improvements that could be made in the legibility of traffic signs and the methodology for testing them.

A basic paper published by Forbes in 1939 (*5*) pointed out that there are two types of legibility, pure legibility where reading time is unlimited, and glance legibility, where reading times are short because of the demands of driving.

In 1977, Bernstein and Olson (*6*) noted that static far visual acuity was not a good predictor of the ability to read signs at night. They also concluded that legibility distances based on anything other than comprehension measures would be conservative. In a later study, Olson (*2*) also discussed the steps necessary for drivers in interacting with signs and pointed out the time necessary for detection, fixation, recognition, and vocalization. When signs are viewed in the glance legibility mode, this time was deemed significant by this author and led to the decision described later under methodology to use a technique closer to pure legibility on this study.

In Shepard's research in 1987 (*3*), the effect of irradiation with high-intensity sheeting and the apparent reduction in size of letters was noted, particularly for CWZ signing. He pointed out the difficulty of increasing the letter height and spacing within the current *Manual of Uniform Traffic Control Devices* (MUTCD) (*1*) requirements and how few CWZ signs could be improved if the *Standard Alphabets for Highway Signs* (SAHS) (*7*) requirements were followed. He therefore concentrated on increasing the stroke width (SW) of letters by 18 percent, and only on the inside dimension of the letters, based on earlier work in California and Nebraska. He reported increased legibility by night for such legends among the six test subjects. He recognized the limitations of the small subject group and suggested more extensive testing, under controlled environment, with reduced letter spacing and spacing between lines.

RESEARCH APPROACH

Because so many variables can affect the legibility of traffic signs in a highway environment, an overriding consideration throughout all the methodology was to eliminate as many variables as possible so that the results would be as closely related to differences in pure legibility as possible.

Sign Design

The work of Shepard (*3*) somewhat influenced the initial selection of signs to be tested. Because the project budget and time constraints were limited, it was decided that 12 test signs could be tested under day and night conditions. Shepard's research was constrained by the current MUTCD requirements; therefore it was agreed between WISDOT and the researcher that reasonable changes in format and design, including rearranged legend, would be included in this research, even though they may require subsequent change in the MUTCD (*1*) or *Standard Highway Signs* (*8*) if they substantially improved legibility. Actual sign messages tested were proposed by the researcher and approved by WISDOT personnel monitoring the project.

Some of the test signs would include stroke width increases as recommended by Shepard. It was also decided to rearrange legends if that would improve legibility. The two most difficult CWZ sign series were selected to improve legibility. If these two could be improved, the results could also apply to other signs in terms of legibility, distance, and letter size and style. The following are examples of these two series:

RIGHT LANE CLOSED AHEAD
CENTER LANE CLOSED AHEAD
LEFT LANE CLOSED AHEAD
RIGHT LANE CLOSED 1500 FT
ROAD CONSTRUCTION AHEAD
ROAD CONSTRUCTION 1500 FT
ROAD CONSTRUCTION 2 MILES

Sign legends must follow the SAHS (*7*). If a legend size of any of these signs is to be increased, it should be expanded mathematically rather than photometrically, according to Phil Russell, formerly of the Federal Highway Administration's Office of Traffic Operations.

One alternative to improve legibility is to go to larger (60-in. diamond) signs, and this obviously allows letter size to be increased substantially but has two disadvantages: cost and inconvenience. The current 48-in. warning signs are clumsy for one person to handle. In addition, large signs are often overturned in a high wind. This researcher, with WISDOT concurrence, decided not to test signs larger than 48 in.

It was also recognized that symbolic signs can result in significant improvement but some legends cannot be symbolized (CENTER LANE CLOSED) and hence symbols were not used in this project.

A number of traditional ideas were tried first to see if improvements were even spatially possible within 48-in. diamond signs. Some of those tried were rejected because the legend was too crowded. It was decided to reject testing a substitution of tenths of a mile (decimal) for large advance distances in feet. Although they are logical, relate to odometer readings in vehicles, and would allow legend size improvement, it would have involved the introduction of another variable of unknown recognition and was therefore rejected. This concept is addressed in the Conclusion section of this paper.

In the ____LANE CLOSED____ series, standard and rearranged legends and different letter series were selected for testing. In the ROAD CONSTRUCTION____ series, the word CONSTRUCTION can be changed to WORK, with substantial improvements in legibility expected through increased letter size. Different letter series were also selected for testing.

It should be pointed out that WORK may be substituted for CONSTRUCTION in the revised Chapter VI of the MUTCD approved by the National Committee on Uniform Traffic Control Devices (*9*). New York has already tried this on freeway construction. In addition, Series *E* (upper- and lowercase) letters were used in this research even though not allowed in MUTCD (*1*) or *Standard Highway Signs* (*8*) for CWZ signs.

Because many of the signs are similar and were to be viewed in random order, there was concern about recognition and memory recall. It was therefore necessary to consider reversing letters or deliberately varying the sign message to ensure legibility. To do this, words similar to highway sign legends were selected. All standard signs and their test messages are shown as follows:

Sign A "RIGHT LANE CLOSED AHEAD"—6-in. *C* (Standard) TEST SAME

Sign B Rearranged "RIGHT LANE CLOSED AHEAD"—7-in. *C* TEST "RIGHT LANE CLIPPED AHEAD"

Sign C Rearranged "RIGHT LANE CLOSED 1500 FT"—7-in. *B* TEST "READ LINE CLOSED 1500 IN"

Sign D Rearranged "RIGHT LANE CLOSED 1500 FT"—7-in. *B* + 18 percent SW increase
TEST "RIPE LINE CLASS 1600 AT"

Sign E "Right Lane Closed 1500 Ft"—6½-in. *E*
TEST SAME

Sign F "ROAD CONSTRUCTION 1 MILE"—7-in. *C*
TEST SAME

Sign G "ROAD CONSTRUCTION 1 MILE"—7-in. *C* + 18 percent SW increase
TEST "RIDE CONSTRUCTION 1 MILL"

Sign H "ROAD WORK 1 MILE"—7-in. *C*
TEST "RODE WALK 1 MOLE"

Sign I "ROAD WORK 1 MILE"—7-in. *C* with 18 percent SW increase
TEST "ROAD WOKE 1 MILL"

Sign J "ROAD WORK 1 MILE"—8-in. *C*
TEST "READ WALK 1 MOLE"

Sign K "ROAD WORK 1 MILE"—8-in. *E*
TEST SAME

Sign L "Road Work 1 Mile"—8-in. *E* with 18 percent stroke increase
TEST "READ WOKE 1 MALE"

Because some signs are standard and their legends expected, and others have unusual or unexpected legends, care had to be taken when comparing results. When people encounter unexpected messages they can be expected to do poorly compared with their performance when encountering expected messages. For that reason, at least one comparison among the 12 signs (signs *F* and *H*) involved the difference between standard and unexpected signs of the same letter size and series (7-in. Series *C*).

Sign Fabrication

The city of Milwaukee sign shop prepared the cutout letters and applied the black vinyl letters according to the design furnished by the research assistant and based on mathematical interpolations of the table for letter size and spacing of the SAHS (*7*).

Before application, measurements of reflective intensity were made on all reflective sign blanks that had high-intensity sheeting (Type I—Reflectivity II, Federal Specification L-S-300C) (*9*) made by 3M and applied by WISDOT. The blanks were all prepared from rolled goods. The orange material was all in the range of 143 to 158 SIA units (cd/ft.c/ft^2), with less than a 10 percent variation in intensity across the entire blank.

Test Site Conditions

Because the greatest problem with irradiation occurs with the least background luminance (*3*), a location that would simulate a rural highway was needed. The inner and outer tracks at the Wisconsin State Fair Park Auto Race Track were used. The pavement luminance was measured with a United Detector Technology Model 40X light meter with a photometric filter and diffuser and pavement luminance of between 0.01- to 0.02-ft candles. The night site is shown in Figure 1a.

In order to eliminate some of the variables in nighttime viewing, a Ford Tempo 4-door rental automobile with halogen headlamps was used.

Selection of Test Subjects

An estimate of the driver population at some future design year (2020) was made based on *Accident Facts* (*11*) and *Transportation In An Aging Society* (*4*). By the year 2020 there will be more than 13 million drivers age 65 or over. Because this project was intended to improve legibility for all drivers, with particular emphasis on the older driver, older drivers were disproportionately represented in the test sample because literature sources reviewed (*4*) indicate that variation in visual acuity would increase with age. Sufficient sample size was needed to ensure significance because of the large variance.

The driver age groups and the desired and actual sample size are shown in the following table. Male and female differences were ignored because no sexual difference in visual acuity by age was observed. For statistical purposes, test subjects were divided into a younger group (under 65) and an older group (65 and over).

Test Age Group	*Desired Size*	*Actual Size*
18 to 44	20	22
45 to 64	20	24
65 and over	40	38

To recruit test subjects, 1,000 flyers were prepared and distributed in a variety of ways, including door-to-door distribution in some neighborhoods, windshield distribution at shopping centers, and through contacts in retirement communities, all within four miles of the test site. Also copies were distributed to retired and several current Marquette University (MU) employees.

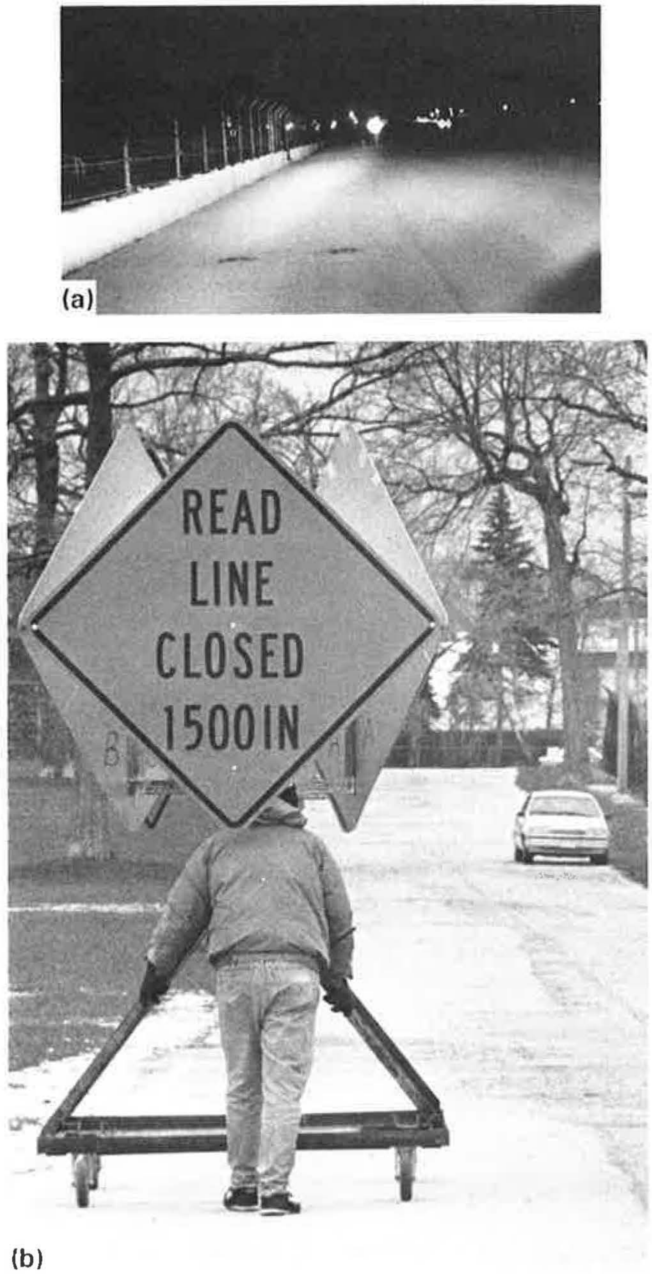

FIGURE 1 Test sites: (*a*) night and (*b*) day, with cart.

Originally, it was expected that the testing and vision screening would take approximately 3 hr. Travel time would add several hours more, so a stipend of $50 was set for test observers, which included any transportation costs, using $10.00/hr as the basis. A test subject had to complete all three tests to receive the stipend. Because paid observers were being sought, no attempt was made to ensure random selection of candidates, other than excluding anyone associated with the College of Engineering at MU.

Vision Screening and Candidate Scheduling

The original charge of the sponsor (WISDOT) was to use the vision screening services of the Wisconsin Drivers License

Examiner's Officer (DLE). Wisconsin statutes require a visual acuity (V/A) of at least 20/40 in one eye for unrestricted licensing. It was agreed that all candidates anonymously pass a standard vision screening administered by the DLE.

Vision screening was administered using a OPTEC 1000 DMV vision tester regularly used in Wisconsin. Visual acuity was recorded for left and right eye only because that is the way vision screening is done in Wisconsin. Data were analyzed later according to best and worst V/A.

A failure rate of 10 percent was anticipated. Actual visual screening resulted in a failure rate of 14.7 percent. People in all age categories, including an 18 year old, failed the vision test. This was partly because DLE insisted they be tested because their license allowed them to drive. Of 93 who passed the vision screening, 84 completed all field testing.

A total of 47 females (56 percent) and 37 males (44 percent) were used as subjects. The females were more heavily represented in the under 65 age groups (72 percent) and the males more heavily represented in the over 65 age group (63 percent). Glasses were worn by 75 percent of all candidates tested.

FIELD TEST PROCEDURES

One of the first decisions made was that this test would be a static one. Because it was only to be a legibility test, the most accurate way to obtain legibility distances would be if the observer were seated in a stationary vehicle and the signs brought forward toward the observer on a sign cart. Signs involved in this study were 4 ft × 4 ft. A sign cart was therefore designed that could hold six of the large signs and would require only rotation of the signs or reversal of the cart to switch the selection of a given sign. Because 12 signs would be tested, two carts were constructed in the machine shop at the MU College of Engineering; one is shown in Figure 1b. By assigning four students to move carts, the observer time was reduced to approximately 20 min/test (day or night) for all 12 signs.

When the subjects entered the driver's seat, the recorder greeted them and oriented them to the task at hand. After orientation, and when the first sign was in place at a distance selected to be beyond the distance at which the observer with a known V/A could possibly see, the recorder inquired if the observers could read the sign, reminding them that they were not to guess.

The cart was moved forward at about 3 ft/sec until the signal was given to stop. When the sign was correctly identified, the radios were turned on and the distance estimated (distances on the course were marked every 10 ft, from 100 to 700 ft) by interpolating within the 10-ft marks. The distance was noted by the recorder on a form prepared in advance, which listed observer data and the random order of signs for that observer. Because this was legibility and not recognition distance owing to sign similarity, the results should be reviewed with that in mind.

The physical offset between sign observer, vehicle, and the center of the sign cart is shown in Figure 2 for the day and night test sites. Test signs are shown in Figures 3 through 5.

Data on sign legibility distances and observer characteristics were analyzed using the statistical SPSS software on the University's Vax computer system. Data on distances were rounded, because rounding the data to the next highest 10-ft increment would be logical, considering how distances were measured and recorded.

STUDY RESULTS

Mean legibility distances are shown in Figures 6 and 7 for all observers, for younger observers (under age 65), and for older observers (65 or over). The figures include the sign alphabet of each test sign to help the reader interpret results.

In Figure 6, mean legibility distances are shown for each observer group by day and by night for the five signs in the RIGHT LANE CLOSED— series (signs *A* through *E*). In Figure 7, the same mean legibility distances are shown for the ROAD CONSTRUCTION— series (signs *F* through *L*). In reviewing comparisons, the differences between legends sometimes make conclusions difficult. Wherever day and night differences were reduced by a test sign compared with a standard, it was concluded that overglow phenomena were reduced, and hence an improvement was made. Qualified conclusions on the comparisons are discussed later.

Further study results are presented by individual sign by day and night in Table 1. They include the ranges (minimum and maximum legibility distances), which show large spreads between "best" and "worst" legibility distances. These spreads are 400 to 500 ft for most signs and show the wide range of drive capabilities encountered.

A cumulative frequency analysis was run on all observer groups to determine the threshold visual acuity at which a given percentile of the observers could see at or better than that level and is reported for the signs at the 85th percentile and is also included in Table 1. Note that the 85th percentile values are much closer to the minimums than the maximums. This could very well be the result of the poor seeing ability of the test subjects. The number of test subjects with V/As at 20/40 or worse in one eye (and still passing) was 45 percent of all observers, 24 percent of younger observers, and 71 percent of older observers.

Multiple regression analysis was performed on each sign result to review linear relationships. Both distance and the log of distance were compared with age, best V/A, and worse V/A. As might be expected, the plots represented a scattered arrangement. Review of each showed that distance and age had coefficients of correlation (CC) of between 0.44 and 0.66. The CCs for distance and best V/A ranged from 0.43 to 0.59, and for distance and worst V/A ranged from 0.31 to 0.51. The equations for the straight-line relationships all had constants of from −2.0 to −4.0 ft/year of age difference, depending on the individual sign design.

Another overall analysis was performed on the differences in range of values (highest to lowest) legibility distance for all signs, for all observers, and for younger and older observers. The reason this was analyzed was because the difference between the best and worst sign was noticed by data recorders, and the difference was generally in the ratio of 1.5:1 to 2:1 (best sign over worst sign). Obviously legend style had an impact on legibility. These differences are as follows:

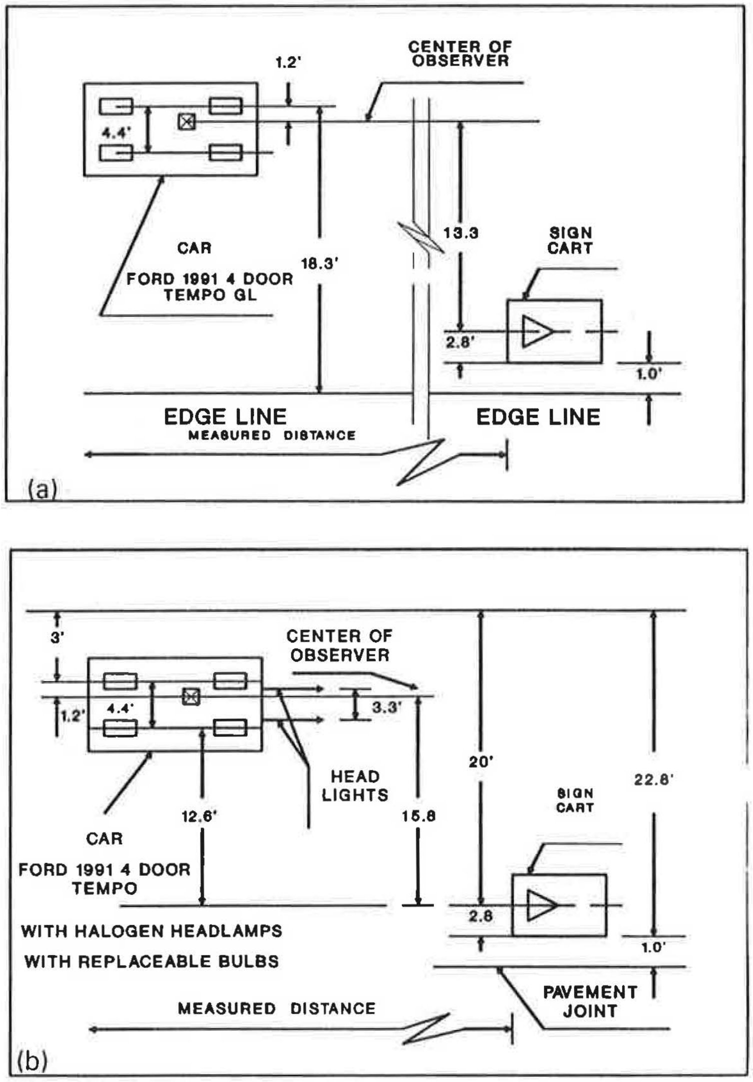

FIGURE 2 Schematic test sites: (*a*) day and (*b*) night.

Observer	*Day*	*Night*
All	202	208
Younger	224	220
Older	174	194

The differences are significant and the difference varies for young and old.

CONCLUSIONS

This study was an attempt to take the two most difficult CWZ signs and test four different concepts to improve legibility.

1. Rearrangement of legend to allow letter size increase (—LANE CLOSED— series).
2. Change from CONSTRUCTION to WORK to allow letter size increase (ROAD CONSTRUCTION— series).
3. The effect of SW increase.
4. Use of Series *E* (upper- and lowercase letters) currently not allowed by the MUTCD).

Other concepts (increased sign size) and use of tenths of miles instead of feet or abbreviations of words were deliberately avoided.

From study results it can be concluded that there is a large variation (range) in nighttime ability to see (read) the current standard CWZ signs (Signs *A* and *F*). The legibility distance for best and worst observers was 650 ft and 140 ft, respectively. The mean of the range of legibility distances between an observer's best and worst sign was approximately 200 ft. Changes tested in this project made a major difference in legibility distance, and not always an improvement.

Those signs that increased both letter size and SW while maintaining or increasing the standard alphabet letter series resulted in the best improvement. Increasing letter size while decreasing the alphabet series (like *C* to *B*) reduces sign

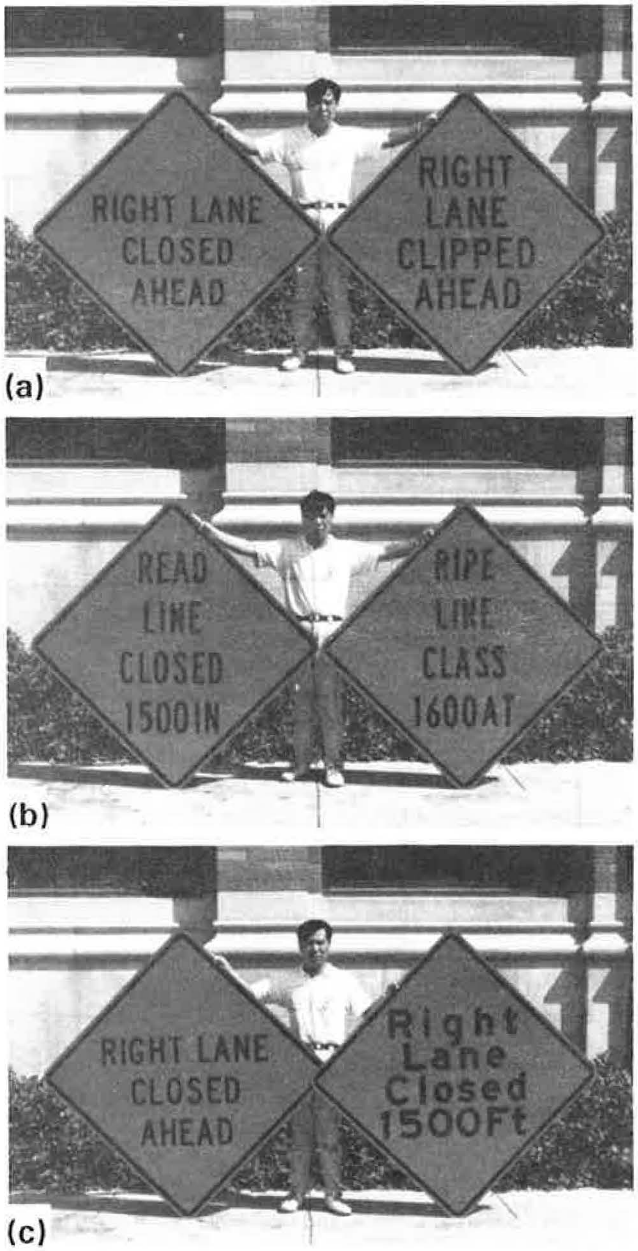

FIGURE 3 Test signs: (*a*) Sign *A* (6-in. *C*) and Sign *B* (7-in. *C*), (*b*) Sign *C* (7-in. *B*) and Sign *D* (7-in. *B* with 18 percent *SW*), and (*c*) Sign *A* (6-in. *C*) and Sign *E* (6½-in. *E*).

legibility, particularly at night. This practice is not recommended.

Effect of Stroke Width Increase of 18 Percent

The increased SW on the inside of letters without changing letter width, as recommended by Shepard (*3*), resulted in either no change or a decrease in legibility by day and night. This was tried on Series *B*, *C*, and *E* and the results were the same.

The alternative method of legend rearrangement or substitution of Series *E* letters with its 21 percent SW increase

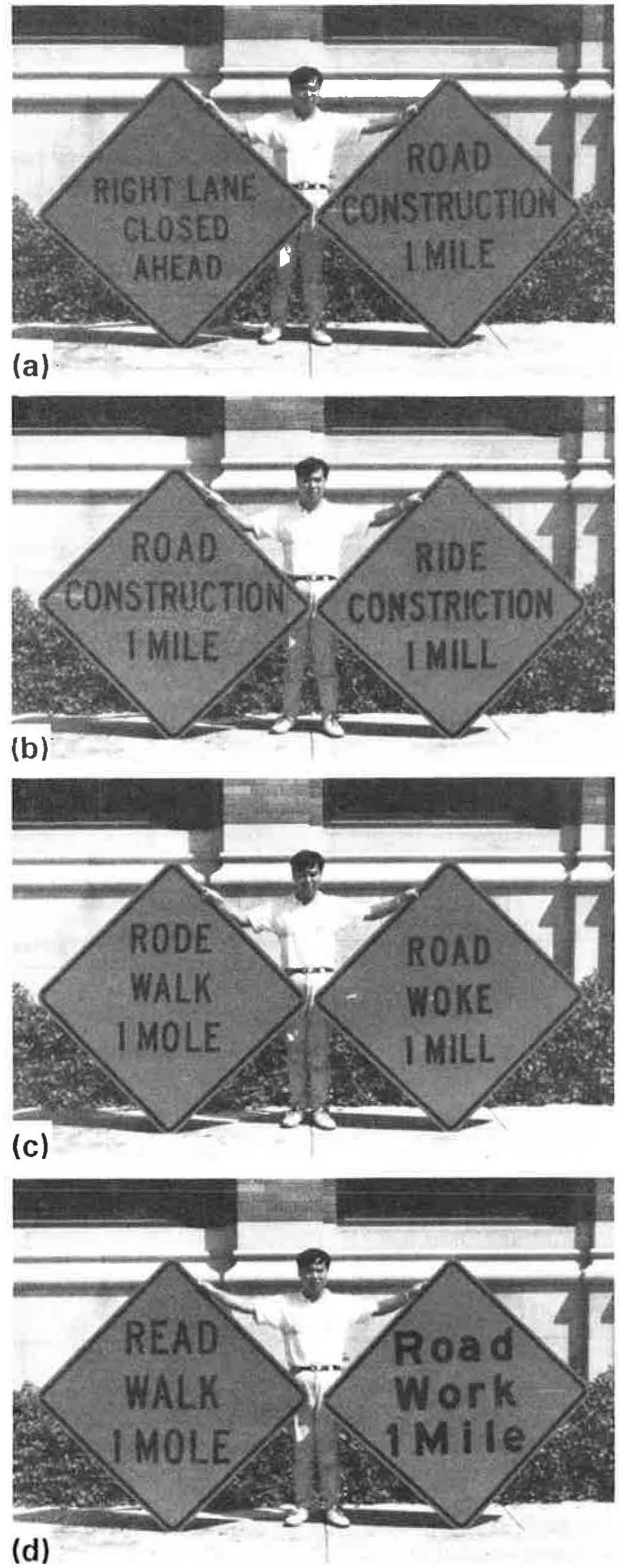

FIGURE 4 Test signs: (*a*) Sign *A* (6-in. *C*) and Sign *F* (7-in. *C*), (*b*) Sign *F* (7-in. *C*) and Sign *G* (7-in. *C* + 18 percent SW), (*c*) Sign *H* (7-in. *C*) and Sign *I* (7-in. *C* + 18 percent SW), and (*d*) Sign *J* (8-in. *C*) and Sign *K* (8-in. *E*).

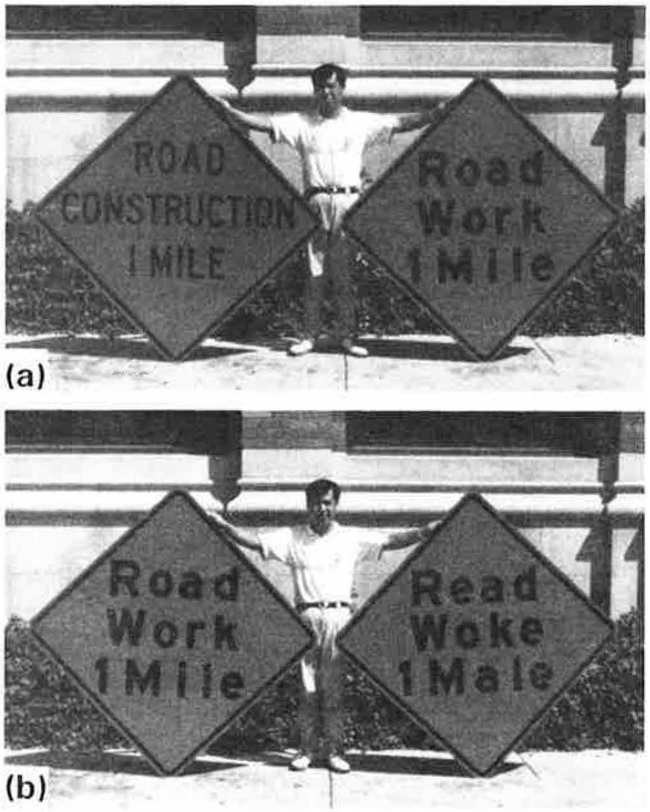

FIGURE 5 Test signs: (*a*) Sign *F* (7-in. *C*) and Sign *K* (8-in. *E*) and (*b*) Sign *K* (8-in. *E*) and Sign *L* (8-in. *E* + 18 percent SW).

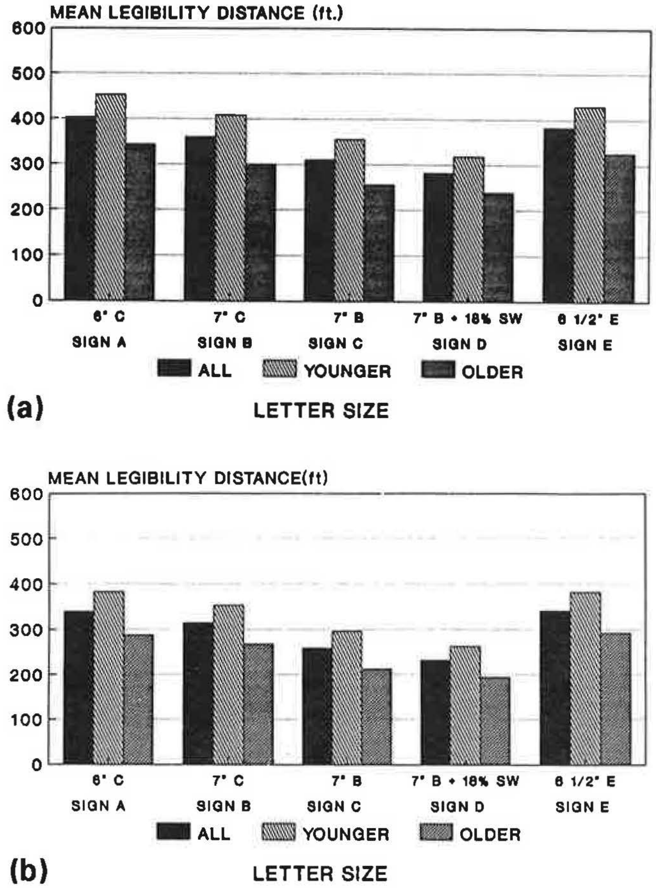

FIGURE 6 Mean legibility distances for Sign *A* through *E*: (*a*) day time and (*b*) night time.

over Series *C*, discussed in the following section, is more promising as a way to overcome the phenomenon of overglow with brighter reflective materials.

Use of Series *E* Letters

The most promising findings of this research are the use of upper- and lowercase letters, even though they are not allowed by the MUTCD. The fact that Series *E* has a 21 percent increase in the ratio of SW to letter height over 8-in. Series *C* letters and a 33 percent increase between a 6-in. Series *C* and 6½-in. Series *E* appears to overcome the overglow phenomenon and yield improved legibility by night without its reduction by day for younger observers. The comparisons of this study did not always make that clear because some signs were standard and some had changed legends. However, the reduction in nighttime legibility distance below that by day for older drivers (85 percentile) was from 0 to 13 ft (Signs *E*, *K* and *L*) only, whereas the reduction for current sign (*A* and *E*) was between 20 and 60 ft (night less than day). This lack of reduced legibility when considering day and night legibility distances indicates that the wider SW overcomes the irradiation or overglow phenomenon, a topic that was one of the purposes of this research.

Sign Legibility Requirements

The results point to the difficulty in bringing legibility distances up to a level at which all drivers would have a minimum of 300 ft of distance (or about 3 sec at 55 mph) to read a sign legend. An examination of the minimum legibility distance for signs *A*, *F* and *K* (all standard) indicates that a relationship of 25 ft of legibility/in. is necessary for legally licensed drivers (those with at least one eye with 20/40 V/A). This means that signs with 12-in. letters would be necessary if a mathematical extension of the results for 6 in. (140 ft minimum), 7 in. (170 ft minimum) and 8 in. (200 ft minimum) letter size are indicative, and if 100 percent of the legal driving public is to be accommodated; slightly less than that called for in Howlett's formula for required visibility distance (*12*).

The frequency of legal drivers of all ages with V/A less than 20/20 and the legibility requirements of older drivers point to the need for new letter size requirements for all signs. The challenge is to do that through careful legend redesign to avoid signs so large that they are unmanageable and costly.

The question that needs to be resolved is how long a CWZ sign needs to be legible (in seconds) while a vehicle is approaching the sign. FHWA's program on Minimum Required Visibility Distance (MRVD) is a help. The MRVD is the distance for a driver to recognize a sign and make the maneuver required. The MRVD takes into account decision sight distance and the distance required by the MUTCD for advance posting of warning signs. The MRVD for both signs is 331 ft at 55 mph and 369 ft at 65 mph (unpublished as of 1991, FHWA data from Jeffrey F. Paniati). A comparison of these

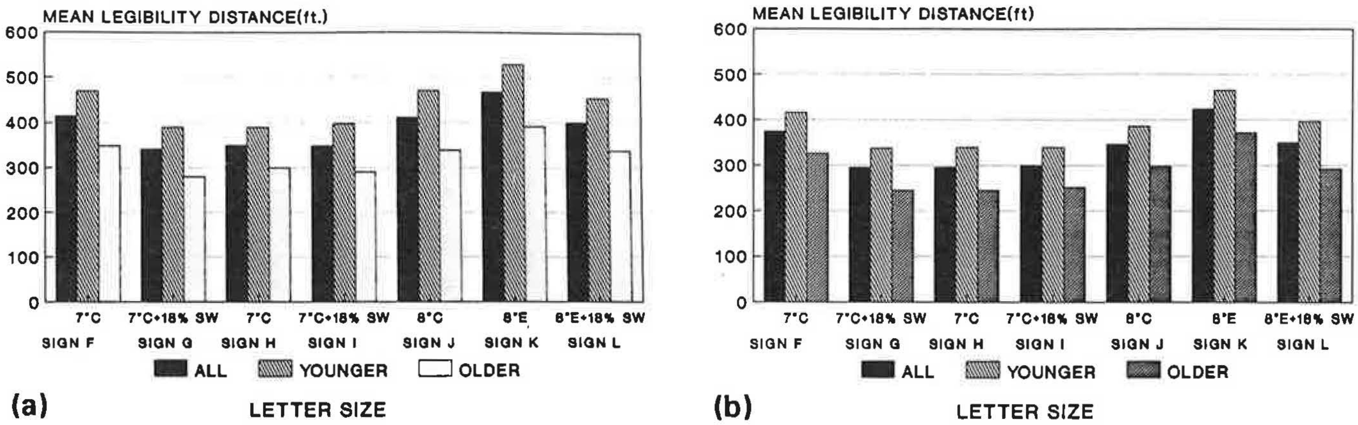

FIGURE 7 Mean legibility distances for signs *F* through *L*: (*a*) day time and (*b*) night time.

TABLE 1 Sign Legibility Data

Sign	Observer category	Legibility Distances (feet)					
		Daytime			Nighttime		
Alphabet		Minimum	85th %	Maximum	Minimum	85th %	Maximum
A	All	180	300	610	140	230	650
6" C	Younger	280	360	610	190	290	650
	Older	180	250	500	140	180	470
B	All	160	260	590	170	220	540
7" B	Younger	250	310	590	210	280	540
	Older	160	210	450	170	200	460
C	All	140	220	500	120	160	450
7" B	Younger	180	280	500	170	230	450
	Older	140	180	400	120	140	350
D	All	140	200	480	100	160	370
7" B + 18%	Younger	180	230	480	140	210	370
SW increase	Older	140	160	420	100	140	320
E	All	180	280	630	150	240	600
6 1/2" E	Younger	270	370	630	230	310	600
	Older	180	220	460	150	210	490
F	All	170	290	630	170	270	630
7" C	Younger	260	340	630	240	310	630
	Older	170	250	500	170	240	500
G	All	140	250	530	120	190	550
7" C + 18%	Younger	210	320	530	190	260	550
SW increase	Older	140	190	440	120	170	480
H	All	160	260	600	100	210	500
7" C	Younger	230	310	600	180	270	500
	Older	160	220	450	100	180	440
I	All	170	240	620	120	210	500
7" C + 18%	Younger	200	320	620	170	270	500
SW increase	Older	170	190	420	120	180	430
J	All	170	280	660	120	240	600
8" C	Younger	280	380	660	200	290	600
	Older	170	240	530	120	200	480
K	All	200	330	680	200	310	620
8" E	Younger	320	440	680	260	390	620
	Older	200	280	630	200	270	580
L	All	180	300	630	170	250	560
8" E + 18%	Younger	270	350	630	230	320	560
SW increase	Older	180	220	500	170	200	550

MRVD distances with the results of this research indicates the almost impossible tasks facing the profession.

Lane Closed Series

No substantial improvement can be made within the current MUTCD requirements and within the current 48-in. size sign. If the word AHEAD could be eliminated (Signs *A* and *B*), improvement would be possible. Use of 6½-in. Series *E* may improve legibility at night but was not proven for this sign series. Further testing is required. If distances in feet can be changed to tenths of a mile, a slight increase may result because letter size can be increased (Signs *C*, *D* and *E*).

Road Construction Series

In the second series, the results show a change from CONSTRUCTION to WORK will allow improvement in legibility without further changes in legend, and this is a significant improvement. This is best shown by comparing mean legibility distances for Sign *H* (7-in. *C*) and Sign *J* (8-in. *C*) for both day and night observations in Figure 7. More drastic changes, however, will be required to make greater improvement, which is required if the needs of all drivers are to be met, and these will require changes in national standards.

RECOMMENDATIONS

The following recommendations were made to WISDOT as a result of this research.

1. Whenever attempting legibility improvements, no decrease in alphabet series should be implemented in order to increase letter height (*C* to *B*). (Example, 7-in. Series *B* instead of 6-in. Series *C* alphabet used in Signs *B* and *C*.)
2. For the RIGHT LANE CLOSED____ series, use of symbol signs (for most applications) will have to supplement word legend signs. For CENTER LANE CLOSED____ series, redundancy of sign placement will have to be used if a 48-in. maximum size is to be maintained.
3. Change CONSTRUCTION to WORK in the ROAD CONSTRUCTION____ series and increase letter size from 7-in. *C* to 8-in. *C*.
4. If irradiation or overglow is to be addressed, and a decision to use high-intensity CWZ retroreflective sheeting is made, WISDOT should pursue experimentation with Series *E* alphabet for both of the tested sign series in this research, as well as other signs in the CWZ series not tested under the procedures of FHWA.
5. Increase letter size and series for all other CWZ signs where possible, setting a sign size limit criteria as with the 48-in. sign size for warning signs.

Future Field Testing and Research

To improve the LANE CLOSED____ series, further research to substitute tenths of a mile for feet and elimination of AHEAD is recommended. Eliminating AHEAD allows a 33 percent increase in letters (6-in. to 8-in. Series *C*), whereas changing to decimals and Series *E* would allow an increase of 16 to 25 percent (6-in. to 7 or 7½-in. Series *E*). Both would help to overcome any irradiation phenomena.

Another more radical change would be rearrangement of the order of the three-line, four-word legend, from RIGHT LANE CLOSED____ to CLOSED RIGHT LANE____. This places the two-word line RIGHT LANE in the center of the diamond, allowing the legend to be increased to 8½-in. Series *C*, an increase in legend size of 2½-in. (or 40 percent). Meaning and legibility would need to be tested because the normal order is reversed.

To further improve the ROAD CONSTRUCTION series, a change to abbreviations for the word MILE in the bottom line is recommended for testing. This, in combination with the substitution of the word WORK, would allow possible letter increase to a 9-in. Series *C* or a 10-in. Series *E*, a substantial improvement over the current 7-in. Series *C* (of at least 28 percent). Legibility improvement would need to be tested.

However, the most important concept that needs further documentation is the switch to Series *E* for CWZ signs using sheetings with high-intensity retroreflective materials and its greater SW to letter-height ratio to overcome the effects of overglow at night, particularly for older drivers.

ACKNOWLEDGMENTS

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Speed-Reduction Patterns of Vehicles in a Highway Construction Zone

RAHIM F. BENEKOHAL, LI WANG, ROBIN ORLOSKI, AND LYNN M. KASTEL

Drivers may change their speeds at different locations within a work zone in response to roadway geometry and traffic control devices. Speeds of vehicles at different locations within a work zone were determined in this study in order to plot their speed-reduction profiles. Vehicles were followed from the time they entered a 1.5-mi-long study section until they exited from it. Automobiles and trucks showed similar speed-reduction patterns. Four categories of drivers were identified on the basis of these patterns. About 63 percent of drivers reduced their speeds considerably after passing the first work zone speed-limit signs (Category 1). Nearly 11 percent of drivers reduced their speeds when they neared the location of construction activities (Category 2). About 11 percent of all drivers did not reduce their high speeds (Category 3). The remaining drivers did not indicate a distinct pattern (Category 4). Three distinct speed-reduction patterns were observed for the drivers in Category 1. The first group decreased their speeds near the first speed-limit signs and had further speed reductions at the work space. The second group drove similarly to the first group, but increased their speed between the two points. The third group reduced their speed near the first speed-limit signs and kept that speed until they passed the work space. The average speed decreased as the vehicles approached the work space, but rapidly increased after passing it. Even at the work space, about ⅔ of automobile drivers and more than half of truck drivers exceeded the speed limit.

Excessive speed of motorists in construction zones has been a safety concern for highway officials. Some states (e.g., Pennsylvania) have doubled their fines for speeding in work zones to discourage drivers from doing so. Most drivers slow down when they perceive a potential danger on the road, such as the presence of a crew or large equipment near the traveled lane (*1*). However, there are some questions that remain to be answered. Where do drivers begin to slow down or speed up in a work zone? Do drivers travel at the reduced speed throughout the work zone? Does the speed limit sign work well as a method for obtaining the speed reduction in a work zone? In order to respond to these questions, speed profiles of vehicles in the work zone are needed.

From speed profile data, velocity of a vehicle at different locations along the work zone can be obtained and the speed-reduction effects of various roadway features and traffic control devices may be determined. The speed profile study provides information that is not available from previous studies (*2–7*), which had measured speeds at only one or two points within work zones. The information from the speed profile study would provide insight into drivers' behavior in work zones that could be used to select more efficient and effective methods of traffic control.

In this study, speeds of vehicles at different locations within a work zone were determined and their speed profiles plotted. The field experiment consisted of obtaining video images of vehicles as they traveled through the construction zone. Discussed in this paper are data collection and reduction, speed profile patterns, driver categories, and speed characteristics of vehicles. Statistical tests on the speed profile data are not included because of space limitations. The terminology suggested by Lewis (*8*) is used, whenever possible, to identify different locations within a work zone.

DATA COLLECTION

Study Site

The study site was in a construction zone on Interstate 57, near Mattoon, Illinois. The highway has two lanes per direction, but one lane on each direction was closed. The work zone was about 4 mi long. The construction work mainly consisted of bridge deck repairs: a bridge over State Route 16 and another over a railroad about 2.5 mi to the south of Route 16. Other construction activities included overlay and shoulder reconstruction on the ramps for Route 16 and I-57. There is a full cloverleaf interchange at Route 16, but the inner loops from Route 16 (on ramps) to the highway were closed. During the data-collection period, several men were working on the Route 16 bridge, and a crane, one or two pick up trucks, and other small pieces of equipment were present on the bridge.

The speed limit inside the construction zone was 45 mph for all vehicles. Two small yellow flashing lights mounted on top of the regulatory 45 mph speed-limit signs were on during data collection. Outside the work zone the speed limit was 65 mph for automobiles and 55 mph for heavy trucks. One of the Illinois Department of Transportation's (DOT's) standard traffic control plans (TCP) was used in this work zone. In general, the TCP follows the procedures given in the *Manual on Uniform Traffic Control Devices* (*9*). The signs used for traffic control in this work zone are shown in Figure 1.

Plan and Profile of Site

The plan, profile, location of the influence points (IPs), and speed stations in the study section are shown in Figure 2.

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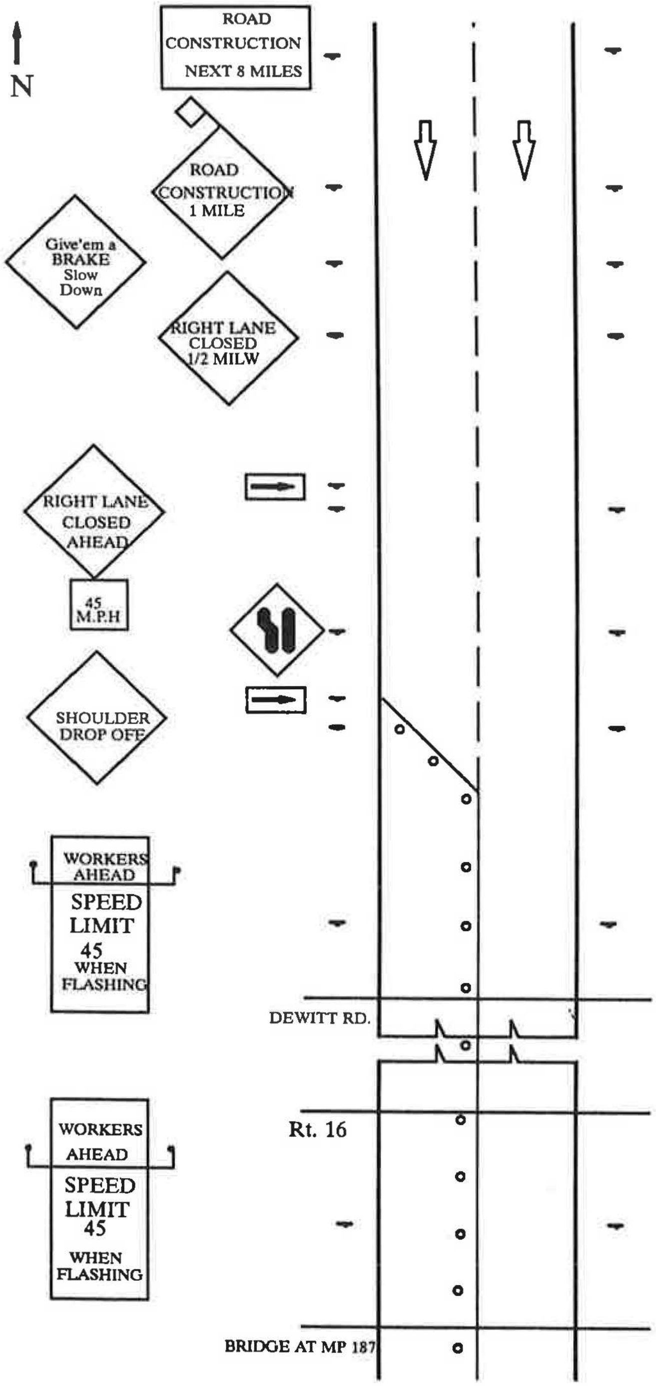

FIGURE 1 Work zone signs on SB I-57 during speed profile study.

There were three sag curves and one crest vertical curve in the study section. The crest curve was approximately 2,800 ft. Proceeding south, the crest curve had a 3 percent uphill slope. The remaining curves had very gentle slopes or short length and would not significantly affect the speed of vehicles. The speed reduction because of the uphill, if any, would be noticeable for truck but not for automobile speeds (*10*). Speeds of vehicles after completion of the construction project were monitored to assess the effect of the upgrade on speed. The average speed reductions for 116 automobiles and 104 trucks were 1.00 mph and 4.96 mph, respectively. From the speed profile data, it was not possible to separate the speed reduction effects of the work zone from that of the upgrade. It was assumed that significant portions of the speed changes were mainly caused by the presence of the work zone and not the upgrade.

Data-Collection Approach

The field experiment consisted of obtaining the speed of the vehicles at various locations as they traveled through the construction zone. The speeds were obtained using video images of the vehicles. Although video images of vehicles have been previously used for data collection (*11*), the data-collection approach of this study is unique. Cameras followed and videotaped the vehicles through the study section.

The study section covered about 1.5 mi of the highway. It was divided into two segments. The first segment was 4,800 ft in length and was videotaped from Camera Location 1. Segment 2 was 5,600 ft in length and was videotaped from Camera Location 2. A 1,600-ft overlapping distance was videotaped from both camera locations.

Each segment was divided into smaller intervals by the road markers. The markers were either of a permanent (e.g., bridge abutment, signs, or light posts) or temporary type. The temporary markers were plastic posts placed on the side of the highway when a permanent marker was not available for some distance. The markers were spaced approximately 400 to 700 ft from each other. A total of 30 markers were established. Normally, 21 markers were used for each vehicle. The remaining 9 markers were considered supplementary, and were used when the time reading for a nearby marker was missing.

Data were collected from May 30 to June 1 of 1990 during weekdays under normal weather conditions. Only vehicles in free-flow conditions were videotaped to eliminate the effects of platooning. The average daily traffic on this section of the freeway was around 12,000, with approximately 22 percent heavy commercial vehicles (*12*). The data contained video records of 208 vehicles that traveled through the study section.

Data-Collection Teams

There were two data-collection teams. The first team was located at Camera Location 1 and the second team at Camera Location 2. The teams communicated with each other using citizen band radio. The first team identified a free-flowing vehicle that was about to enter the study section and started videotaping it. As the vehicle approached Segment 2, information about the vehicle and its location was given to the second team so that the same vehicle was followed. Both teams videotaped the same vehicle on the overlapping intervals. The overlapping intervals were used to check the speed computed from the two video images and confirm that the same vehicle was used by both teams. A description of each vehicle and the time of the day it traveled in the study section were written on the field notes for later use in data reduction.

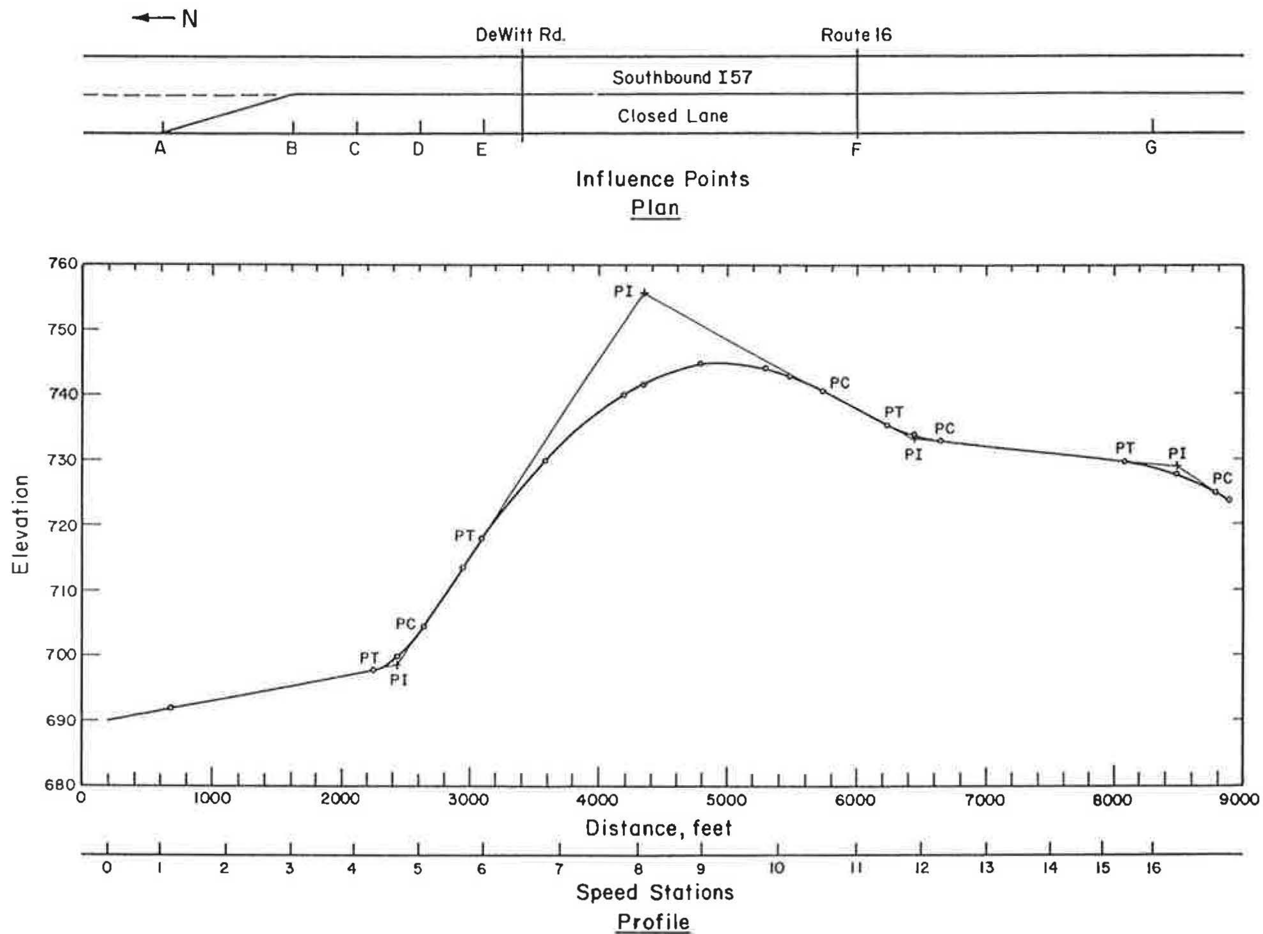

FIGURE 2 Plan, profile, and location of IPs and speed stations.

Camera Locations

Camera Location 1 was on DeWitt Road, which is about 2,600 ft north of Route 16. From Camera Location 1, the vehicles could be seen before entering the lane closure taper until they traveled a distance of 4,800 ft, which is the end of the overlapping intervals and Segment 1. Camera Location 2 was on Route 16. From Camera Location 2 a vehicle could be seen from the time it was on the Dewitt Road overpass, which is the beginning of the overlapping intervals, until the vehicle traveled a distance of 5,600 ft on the highway, which is the end of the study section. Camera locations 1 and 2 were 2,600 and 1,600 ft, respectively, away from I-57.

DATA REDUCTION

The data reduction was very labor intensive. Descriptions of each vehicle were carefully checked, once again, to confirm that the same vehicle was videotaped by the two teams. Also checked was whether this vehicle exited from the ramps or came up on a slow-moving vehicle. This was discerned by watching the videotapes of the vehicle. When it appeared that a taped vehicle was slowed down by another vehicle, the taped vehicle was tagged as suspected for platooning (influenced).

Out of 208 vehicles, 57 vehicles were tagged as influenced vehicles. The remaining vehicles were labeled as uninfluenced. The uninfluenced vehicles were divided into three vehicle types: automobiles (C), semi-trailer trucks (T), and vans and others (V). There were 74 automobiles, 49 trucks, and 28 vans and other vehicles in the uninfluenced group. During data collection for this group, no police were present in the work zone, speed limit was 45 mph, and several workers were working on the Route 16 bridge.

The data reduction consisted of the following steps:

1. Recording the travel times,
2. Calculating the distances,
3. Computing the speeds,
4. Finding velocity at speed stations,
5. Computing speeds at the overlapping segments,
6. Determining speeds at the influence points, and
7. Checking errors.

Each of these steps will be briefly described in the following sections. More information about data reduction is given elsewhere (*13*).

Recording Travel Time

The time a vehicle passed a marker was recorded to the accuracy of 1/30 of a sec. The time a vehicle spent between two markers was computed from the recorded data. For each vehicle, at least 21 readings of the time were recorded from the videotapes. Two successive markers identified a speed station.

Distance Calculation

The distance between two markers as an observer sees it on the television monitor is not equal to the actual longitudinal distance between the markers along the highway. The distance a vehicle traveled during the two readings of the time was the distance subtended between the lines connecting the markers to the camera location. The traveled distance is computed using the lateral distances from the markers to the travel path of the vehicle and the angles between the road and the lines from the markers to the camera location. The actual longitudinal distances between the markers were measured using a measuring tape.

Speed Computation

For each vehicle the average speed between two markers was computed by knowing the time and the distance traveled. The speeds for 9 intervals were computed based on the data that were collected at Camera Location 1. Similarly, the speeds were computed for 10 intervals from the data collected at Camera Location 2. The speeds on the two overlapping intervals were computed using the data from both camera locations.

Speed at Speed Stations

A map of the study area was drawn with a scale of 1 in. equal to 100 ft. A line of sight from a camera location to a marker was extended to cross the southbound lanes of the highway. By this method, it was possible to determine the location of the vehicle on the lanes at the time that it appeared to pass the marker. These lines divided the study section along the highway into smaller intervals. The length of each interval is equal to the distance a vehicle traveled between the two markers as seen from the camera locations. The speeds computed in the previous section are for these intervals. The speeds were assumed to correspond to the speeds at or within 100 ft of the midpoints of the intervals, which are called speed stations. Using the computed speeds and the map, the speed of a vehicle at any point on the highway could be determined.

Speed at Overlapping Intervals

There were two overlapping intervals in the middle of the study section. These were from the DeWitt Road abutment to the double-cross pole, and from the double-cross pole to the overhead sign or light post. The speed of a vehicle on an overlapping interval was computed by both teams. The speeds computed by the two teams were very close. For most of the computations the differences were less than 1 mph. Thus it was decided to use the average of the two speeds as the speed on the corresponding overlapping interval.

Speed at Influence Points

Throughout the construction zone, there are traffic control signs and roadway features that may influence the speed of a vehicle. An influence point is defined as a location within the construction zone that may have such a sign or roadway feature. Seven IPs, labeled *A* through *G*, were used in this paper. The distances from the beginning of the study section to the IPs *A* through *G* are 600, 1,600, 2,100, 2,600, 3,100, 6,000, and 8,300 ft, respectively. The speeds of a vehicle at these IPs were determined using the speed profiles.

Measurement Errors

The speed profile of each vehicle was reviewed to check whether there were any noticeable errors (e.g., a sudden increase or decrease in speed). Common sources of noticeable errors could be mistakes in computing time or distance, input errors, or errors caused by missing data. Possible sources of noticeable errors were identified and corrected (*1*). Moreover, computational errors and sensitivity of the computed speed to the input values were examined. The computational errors are caused by the procedures used to compute the speed. Sources of computational errors are such things as errors in the measurement of the longitudinal distances between the markers, the lateral distances from the markers to the highway, the distance between the camera and the road, and errors caused by the vehicle's location in the lane and the width of vehicle (e.g., automobile versus truck). The magnitude of the computational errors were determined. In general, the computed speed could be influenced by 1 mph or less because of these errors (*1*).

STUDY FINDINGS

This study provided information about drivers' speed change patterns as they traveled through the work zone. Such information was not previously available. The focus of this paper is on the analysis of speed profile patterns and speed characteristics. Statistical tests on the data are not presented because of space limitations. The findings reported are based on a sample size of 123 uninfluenced vehicles, made up of 74 automobiles (only) and 49 trucks (vans are not discussed). Almost all of the vehicles included in the truck category are of the tractor semitrailer type. In the following sections, the speed profile patterns and drivers' categories will be discussed first, followed by the analysis of speed characteristics for automobiles and trucks.

SPEED PROFILE PATTERNS AND DRIVER CATEGORIES

Speed Profile Patterns

The review of speed profiles for 74 automobiles and 49 trucks indicated that there are certain speed-reduction patterns that are repeated by many drivers. The common speed profiles for automobiles and trucks were identified separately. It was observed that some automobiles and trucks have similar speed profiles. Based on their speed profile patterns, the drivers were grouped into four general categories. The general description of a category would apply to automobiles as well as to trucks.

The criteria for the classification were the visual examination of the speed change patterns and a quantitative measure of the speed change. Vehicles that showed similar speed profiles were grouped in one category. If the speed change was not noticeable (less than 5 mph for automobiles and 4 mph for trucks), it was attributed to expected speed fluctuation and was not used as a criterion in the classification. The descriptions of these categories are given in the following paragraphs. The speed profiles for typical automobile and truck drivers in each category are shown in Figures 3a and 3b.

Driver Category 1

Category 1 represents those drivers who reduced their speeds noticeably in the beginning of the one-lane section (near the first speed-limit signs). Some drivers in this category had further speed reduction at the work space (over the Route 16 bridge). The drivers in Category 1 are further divided into three sub-categories (groups).

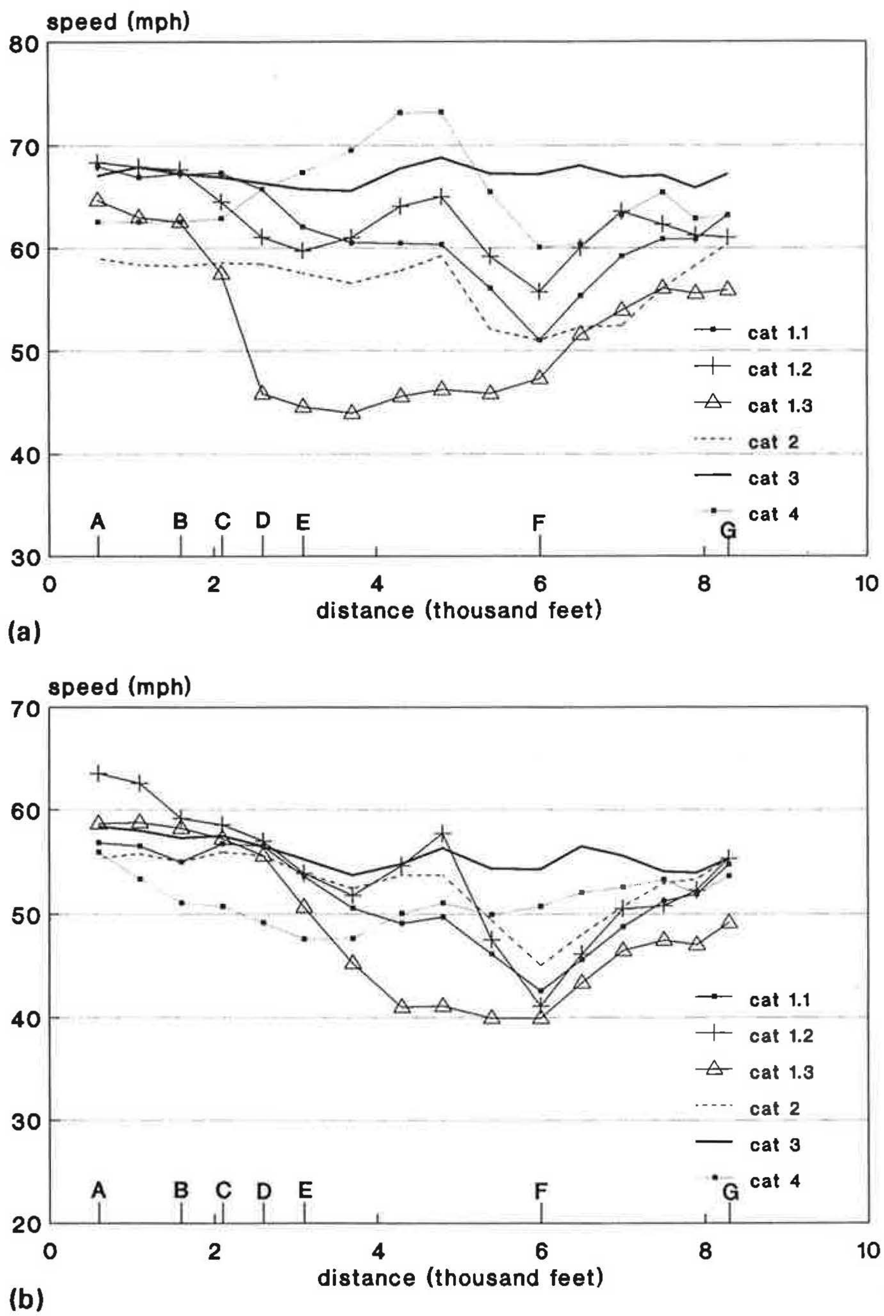

FIGURE 3 (*a*) **Typical speed profile for automobiles and** (*b*) **typical speed profile for trucks.**

Driver Category 1.1

The first group in Category 1 represents the drivers who decreased their speeds near the first work zone speed-limit signs and had further speed reductions at the bridge (work space). Usually, the latter speed reduction was greater than the former. The speed profile for this group was similar to that of automobile No. 197 or truck No. 54. Approximately 22 percent of automobile drivers and 25 percent of truck drivers belong to this group. Generally, the speeds of the automobiles before the bridge, unlike the speeds of the trucks before the bridge, were much higher than 45 mph.

Driver Category 1.2

The second group in Category 1 represents the drivers who slowed down considerably at the first speed-limit signs and at the bridge, but between these two points they increased their speeds. Their speed profiles resembled a "W". The speed profiles for automobile No. 153 and truck No. 126 represent typical speed profiles for this group. About 26 percent of automobile drivers and 20 percent of truck drivers were placed in this group.

Driver Category 1.3

The third group in Category 1 represents the drivers who reduced their speed around the first speed-limit signs and kept traveling at that reduced speed until they passed the bridge. After passing the bridge, some drivers increased their speeds. The speed profile for this group was similar to that of automobile No. 67 or truck No. 60. About 12 percent of automobile drivers and 23 percent of truck drivers belong to this group.

Driver Category 2

The criteria for Category 2 was that the drivers traveled faster than the speed limit and did not have significant speed reduction around the first speed-limit signs (IP *D*). They largely ignored the first speed limit sign, but began to slow down when they arrived at the bridge. The speed profiles for automobile No. 202 or truck No. 201 represent the drivers in Category 2. Nearly 13 percent of automobile drivers and 8 percent of truck drivers were placed in this category.

Driver Category 3

Category 3 includes those drivers who ignored both the first speed-limit signs and the construction activities over the bridge. Examples of the drivers in this category are the drivers of automobile No. 43 and truck No. 192. They drove through the work zone at an almost constant speed that was higher than the 45 mph speed limit. The automobile drivers in this category maintained a speed of about 60 mph or higher, and truck drivers traveled at a speed between 50 and 60 mph. The speed fluctuation for this group was very small (5 mph or less). About 11 percent of automobile drivers and 10 percent of truck drivers were grouped in this category.

Driver Category 4

The fourth category is called "others," which includes those vehicles that could not be classified into categories 1 to 3. Some of the drivers in Category 4 reduced their speed at the first speed-limit signs, but did not slow down at the bridge. Some of them even increased their speeds while passing through the work space. The speed profiles for automobile No. 26 and truck No. 34 represent this category. About 16 percent of automobile drivers and 14 percent of truck drivers belong to Category 4.

Importance of Categorizing Drivers

Knowing the nature and extent of the speeding problems in a work zone would help in the selection of appropriate countermeasures that may result in more effective traffic control plans. Measures taken to slow down the drivers in Category 3 who ignored the speed-limit signs may be different from the measures for the drivers in Category 1 who reduced their speed, but not to the desired level.

The critical points in a construction zone are the locations in which the drivers slow down or speed up. Knowing these points would help in placing the signs at the appropriate locations. For instance, the traffic control signs to encourage the drivers to maintain the reduced speed should be placed before or at the points where the drivers begin to increase their speeds, whereas the signs to reduce their speed should be at the beginning of the work zone.

The distribution of drivers in the four categories indicates that 63 percent of drivers reduced their speed considerably after the first construction zone speed-limit signs. Category 1.3 represents the desirable speed-reduction pattern, and other drivers should be encouraged to follow this pattern. The effects of placing additional speed-limit signs between the first speed-limit signs and the work space to persuade the drivers in Category 1.1 to slow down further, and to discourage the drivers in Category 1.2 from increasing their speed before the work space, need to be studied.

The speed-reduction patterns of drivers may be used to determine the location of work zone signs that would result in more desirable speed-reduction patterns. The drivers in Category 1.2 may have increased their speeds because they perceived that no work was going on or the work space was too far from the first set of speed-limit signs. The location of the signs and the length of section before the work space should be such that most drivers are encouraged to follow the speed limit. Because 63 percent of drivers reduced their speeds around the first speed-limit signs and 74 percent of all drivers reduced their speeds near the work space, more speed reduction may be achieved if the work space is closer to the beginning of the work zone.

Traffic control plans should be carefully prepared to obtain a higher level of compliance from the drivers. Drivers complain about marking a long stretch of highway as the construction zone without any construction activities (*1*). Such a practice reduces the credibility of the work zone signs. The speed profiles for the drivers in Category 2 indicate that this group delayed speed reduction until they saw the construction activities. The length of construction zones should be limited, whenever possible, to the section that is actively under construction.

SPEED CHARACTERISTICS FOR AUTOMOBILES

At each IP, the maximum, minimum, average speed, and its standard deviation were computed. These statistics are summarized in Table 1. The mean speed for automobiles reduced from 63 mph at the beginning of the taper (IP *A*) to 49 mph at the bridge (IP *F*), and then increased to 57 mph at the end of the study section (IP *G*). The speed frequency distributions at four different IPs are shown in Figure 4. The standard deviations were around 7 mph at all locations, except over the bridge where they were 9.23 mph. The large standard deviations indicate that the speed range was high on all IPs and even higher over the bridge. The speeds of automobiles over the bridge were as low as 29 and as high as 67 mph.

Speed Characteristics of Automobiles on Taper

The speed distribution for IP *A* shows one peak around 66 mph and another around 50 mph (see the modal speeds in

TABLE 1 Speed Statistics for Automobiles and Trucks at Influence Points (in mph)

Influence Points	Min Speed		Max Speed		Mean Speed		Standard Dev	
	Cars	Trucks	Cars	Trucks	Cars	Trucks	Cars	Trucks
A	46.1	45.5	77.3	68.8	62.6	57.0	7.24	5.12
B	43.7	38.3	72.2	66.4	59.9	54.2	7.01	5.86
C	44.8	38.9	70.7	66.3	58.8	54.3	7.31	5.85
D	41.7	40.2	69.8	67.0	57.2	53.6	7.37	5.65
E	39.3	39.8	68.6	65.4	55.6	51.4	7.06	5.38
F	29.3	35.3	67.2	59.7	49.3	45.5	9.28	5.13
G	41.8	40.5	72.8	64.8	56.9	52.3	6.93	5.19

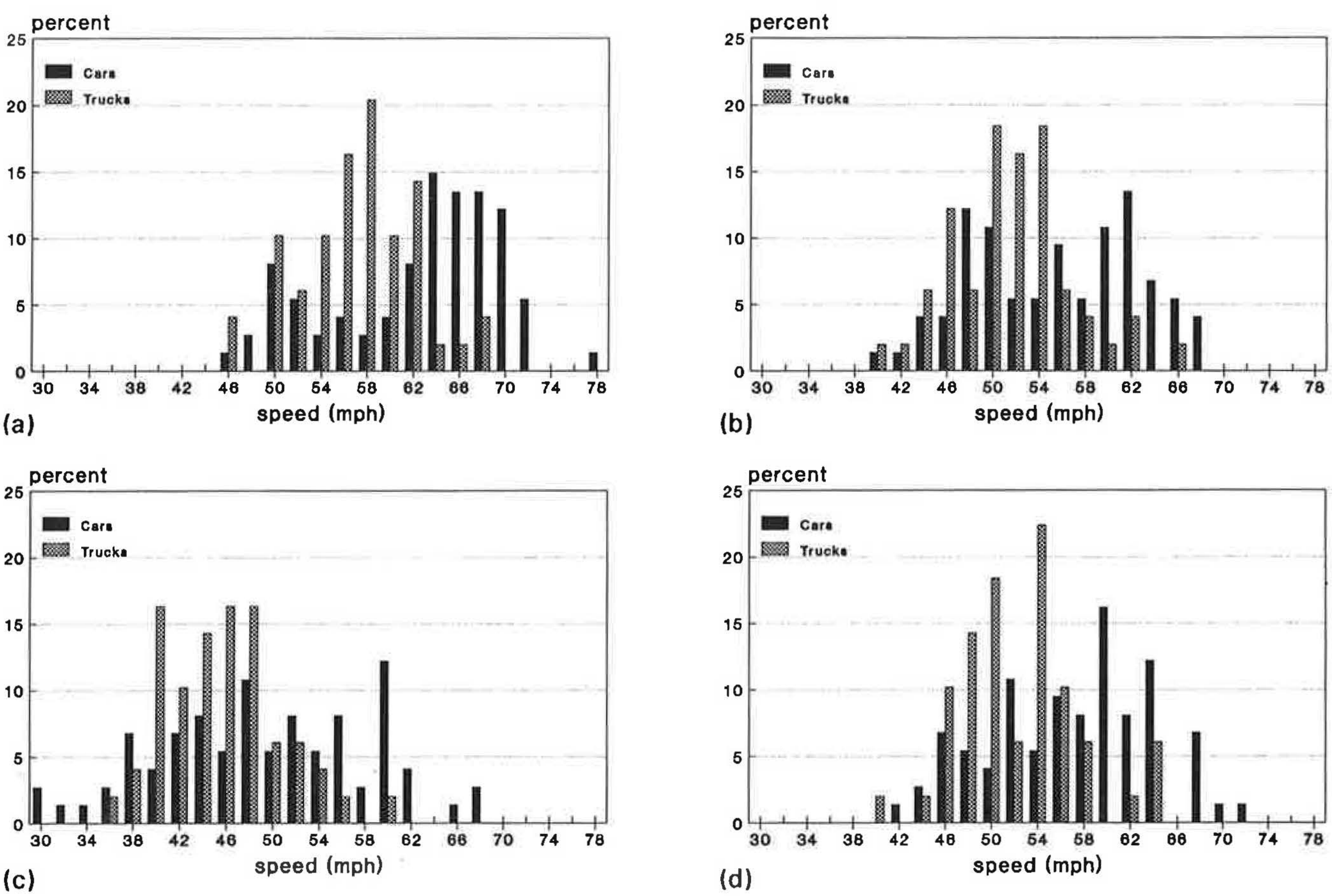

FIGURE 4 *(a)* **Speed distance at IP** *A*, *(b)* **at IP** *E*, *(c)* **at IP** *F*, **and** *(d)* **at IP** *G*.

Figure 4a). These two modal speeds indicate that there were two general groups of drivers. A small group of drivers was traveling at approximately 50 mph, but the rest of the drivers had a speed in the upper 60s or lower 70s. This distinct pattern became less obvious as the vehicles traveled through the work zone. By the time the automobiles reached the end of the taper (IP *B*), the average, maximum, and minimum speeds had decreased, but still almost all of the vehicles exceeded the speed limit.

Speed Characteristics of Automobiles Near First Speed-limit signs

As the vehicles passed IP *C*, located 500 ft before the first construction speed-limit signs, a small decrease (1 mph) in the average speed was observed. At the first construction zone speed-limit signs (IP *D*), the average speed was 57 mph and 96 percent of the automobiles exceeded the speed limit. After passing the first speed-limit signs, the vehicles continued decreasing their speeds. This reduction can be seen by comparing the average speed and speed distributions at IP *E* with that of IP *A*. The speed distribution at this point (Figure 4b) looks very different than that of IP *A*. The drivers did not use a modal speed, but traveled at a wide range of speed around the mean.

Speed Characteristics of Automobiles at Work Space

The average speed of automobiles over the bridge (IP *F*) was the lowest (49 mph) and speed distributions showed a significant shift toward the lower speeds (see Figure 4c). The range of speed was from 29 to 67 mph, with few drivers traveling near the upper bound of this range. The work space over the bridge was physically separated from the travel lane by 250 ft of concrete safety shape (Jersey) barriers. At this point, the open lane was about 15 ft and did not present the feeling of going through a narrow lane. Although a previous study indicates that the concrete safety shape (Jersey) barriers do not affect highway capacity even when they are closer than 6 ft to the traveled lane (*14*, p. 3–11), vehicles decreased their speed when they went through this section (work space). The main reason for the speed reduction seems to be the construction activities in the work space and presence of the concrete shape barriers.

Speed Characteristics of Automobiles After Passing Work Space

After passing the bridge, the vehicles increased their speeds and the speed distributions showed a shift toward higher speeds (see Figure 4d). At IP *G*, which was located at the end of the study section, there was a second pair of construction speed-limit signs. The signs at IP *G* were for drivers entering from the ramp as well as for the drivers on I-57.

It should be noted that the average speed and speed distribution at the second speed-limit signs (IP *G*) were very close to those at the first speed-limit signs (IP *D*). The average speed at IP *G* increased to 57 mph, which is equal to the average speed at IP *D*. About the same number of drivers exceeded the speed limit at these two locations.

Percent of Automobiles Exceeding Speed Level

Almost all of the automobile drivers traveled faster than 45 mph at all IPs except at IP *F*, where nearly 70 percent of them went faster than 45 mph. The percentage of automobiles exceeding a given speed decreased over the bridge and increased to the same level as that reached before the drivers had passed the bridge. The percentage of automobiles exceeding 65 mph was 53 percent at the beginning of the taper, 19 percent at the first speed-limit signs, 4 percent at the bridge, and 12 percent at the second set of speed-limit signs. Similar trends were observed at other speed levels, as shown in Figure 5a.

It is important to note that the percentage of automobiles exceeding a given speed at the second set of construction zone speed-limit signs (IP *G*) reached the level of the first speed-limit signs (IP *D*). This indicates that, on the average, the drivers decrease their speeds to the lowest level near the work space, but after passing it they accelerated to speeds as high as they had at the first speed-limit signs.

Figure 5a clearly shows that the percentage of automobiles exceeding a speed level decreased as they approach IP *F* (the bridge on Route 16), but after they had passed it the percentage increased. Although the percentage of automobiles exceeding the speed limit over the bridge (IP *F*) was the lowest compared with other locations, nearly 70 percent of automobiles traveled faster than 45 mph at this location.

SPEED CHARACTERISTICS FOR TRUCKS

For trucks, the maximum, minimum, and average speed, and its standard deviation, are summarized in Table 1. The average speed for trucks was reduced from 57 mph at the beginning of the taper (IP *A*) to 46 mph at the bridge (IP *F*), and then increased to 52 mph at the end of the study section (IP *G*). The speed frequency distributions at four IPs are given in Figure 4. Trucks had only one modal speed contrary to the automobiles that showed two modal speeds at the first few IPs. The range of speed for trucks was narrower than that for the range for automobiles, as reflected by the smaller standard deviation for trucks (it varied between 5.12 and 5.86). Interestingly, the largest variation in speed did not occur over the bridge, but was at the end of the taper and before the first speed-limit signs. Over the bridge, the speed of trucks was as low as 35 mph and as high as 60 mph.

Speed Characteristics of Trucks on Taper

The speed distribution for trucks showed one modal speed, as shown on Figure 4a. Unlike the automobiles, most of the trucks traveled at about the mean speed. By the time the trucks reached the end of the taper, the average speed decreased by 3 mph and the speed distributions were shifted toward lower speeds. At IP *B*, nearly one-quarter of the trucks traveled at the mean speed at this location.

Speed Characteristics of Trucks Near First Speed-limit signs

The speed variance at IP *C* was as large as that of IP *B*, but the other speed indicators (minimum, maximum, and aver-

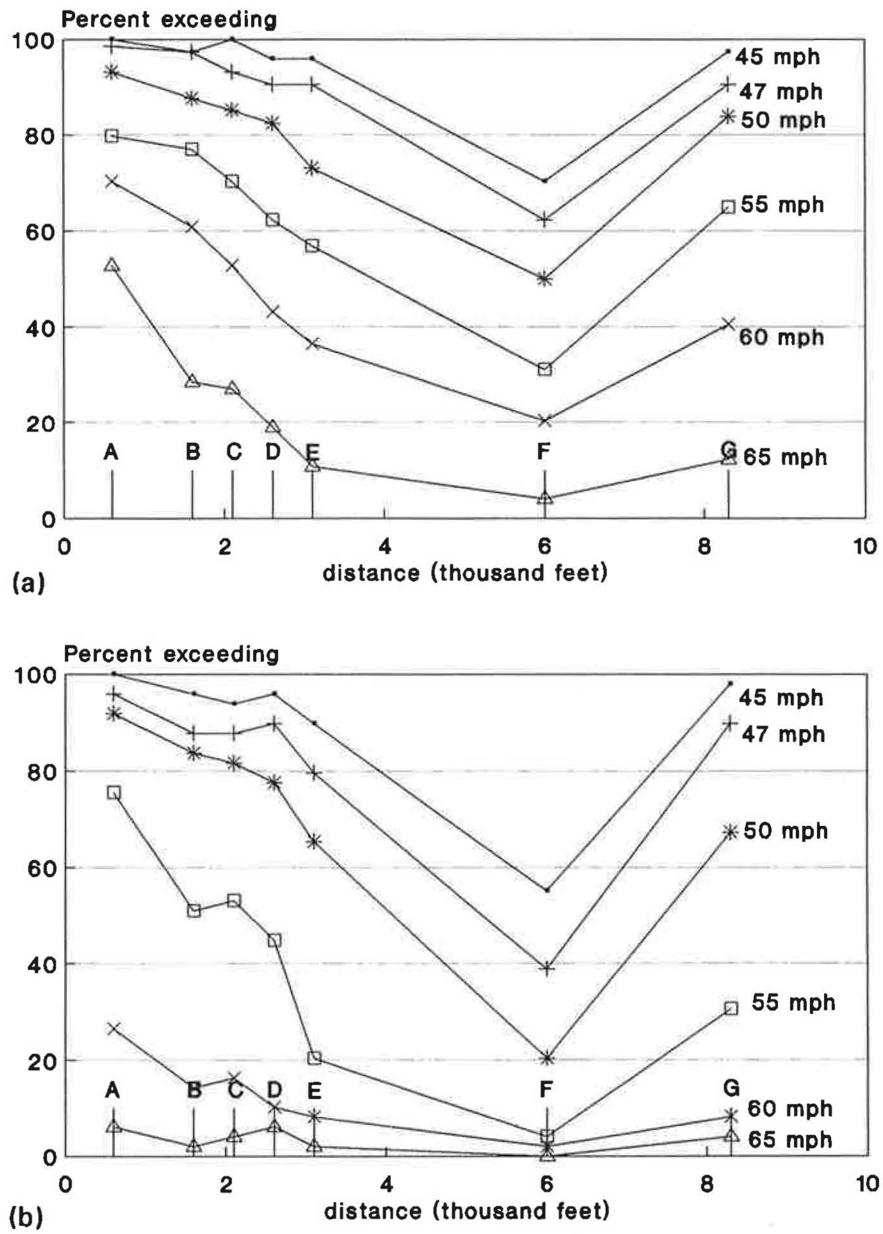

FIGURE 5 (*a*) **Percent of automobiles exceeding a given speed in the work zone, and** (*b*) **percent of trucks exceeding a given speed in the work zone.**

age) remained the same. At the first work zone speed-limit signs (IP *D*), the average speed was 54 mph. After passing the first speed-limit signs (IP *E*), the average speed of the trucks decreased to 51 mph. The speed distributions at this point showed more concentration around the mean speed, as shown in Figure 4b.

Speed Characteristics of Trucks at Work Space

Over the Route 16 bridge (IP *F*), the average speed for trucks dropped to its lowest value, and the speed distributions showed a significant shift toward the lower speeds (see Figure 4c). The average speed at this point was 46 mph and nearly half of the truck drivers traveled faster than the speed limit. About two-thirds of the truck drivers maintained a speed within 5 mph of the speed limit.

Speed Characteristics of Trucks After Passing Work Space

Like automobiles, the trucks increased their speeds after passing the work space. The average speed increased to 52 mph, even though there was a second pair of work zone speed-limit signs at the IP *G*. Comparing the speed distribution for point *F* (Figure 4b) with that of *G* clearly shows the shift toward the higher speeds.

Percent of Trucks Exceeding Speed Level

The percentage of trucks exceeding a given speed at different locations within the study section is shown in Figure 5b. More than 90 percent of the truck drivers traveled faster than 45 mph at all IPs except at IP *F*, where more than half of them went faster than 45 mph. The percentage of trucks exceeding 65 mph was 6 percent at the beginning of the taper, 6 percent at the first speed-limit signs, 2 percent at the bridge, and 4 percent at the second speed-limit signs. The percentage of trucks exceeding a given speed decreased over the bridge and then generally increased after the bridge to the same levels as those observed before the bridge.

It is important to note that truck drivers decreased their speeds, as did automobile drivers, to the lowest level near the construction activities, but after passing them accelerated to their previously higher speeds. The percentage of trucks exceeding a given speed at the first and second construction zone speed-limit signs (IP *D* and IP *G*) is almost equal.

CONCLUSIONS

In response to roadway geometry and traffic control devices, motorists may change their speeds at different locations within a work zone. This study determined the speed-reduction profiles of vehicles in a work zone. Four categories of drivers were identified on the basis of their speed-reduction profiles. The drivers in different categories showed distinct speed-reduction profiles, but automobiles and trucks in a given category indicated similar speed-reduction patterns. About 63 percent of all drivers reduced their speeds considerably after passing the first work zone speed-limit signs (Category 1). Nearly 11 percent of all drivers reduced their speeds when they neared the location of construction activities (Category 2). About 11 percent of all drivers did not reduce their high speeds (Category 3). The profiles for the remaining drivers did not indicate a pattern (Category 4).

The percentage of vehicles exceeding a speed level decreased as the work space was approached (IP *F*), but after passing it the percentage increased. The percentage of vehicles exceeding a given speed at the second construction zone speed-limit signs (IP *G*) reached the level of that observed at the first speed-limit signs (IP *D*). The drivers decreased their speeds to the lowest level near the work space, but after passing it they accelerated to the higher speeds they had reached at the first speed-limit signs. Although the percentage of vehicles exceeding the speed limit at the work space (IP *F*) was the lowest compared with other locations, nearly 70 percent of automobiles and 55 percent of trucks traveled faster than 45 mph at this location.

RECOMMENDATIONS

The locations where drivers slow down or speed up are critical points in a construction zone. Knowing these points would help to place the signs at appropriate locations. It is recommended that the placement and frequency of the work zone speed-limit signs should be examined using the speed-reduction patterns of the drivers. The effects of placing an additional set of speed-limit signs between the first speed-limit signs and the work space to persuade the drivers in Category 1.1 to slow down further, and to discourage the drivers in Category 1.2 from increasing their speed before the work space, need to be determined. The location of the signs and the length of section before the work space should be such that most drivers are encouraged to follow the speed limit.

The analysis indicated that the location of a speed-measuring station has to be carefully selected because it would affect the outcome of the measurements. Furthermore, speed distributions as well as the mean speeds should be analyzed to obtain more accurate speed-characteristic data.

The data-reduction stage should be computerized to reduce the human resources needed. The computational errors of this data-collection method are reasonable. This approach, although time consuming in the data-reduction stage, should be used in future studies.

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Accidents at Entrance Ramps in Long-Term Construction Work Zones

DAVID B. CASTEEL AND GERALD L. ULLMAN

Presented in this paper is an analysis of accidents in entrance ramp areas on two long-term freeway reconstruction projects in Texas. Entrance ramp areas were compared with nonentrance ramp areas to determine the proportionality of accident increases during construction. Accident data were collected for each project for 2 or 3 years before construction and for the duration of the construction phases studied. The two projects studied were on I-35W in Ft. Worth, Texas, and I-45 in Houston, Texas. Accident frequencies increased 35 percent in nonentrance ramp areas and 61 percent in areas of entrance ramps remaining open on I-35W during construction. Both increases were statistically significant at $a = 0.05$. On I-35W, accident frequency increases were disproportionately higher in entrance ramp areas during construction (30 percent higher than increases in nonentrance ramp area accidents, statistically significant at $a = 0.05$). Additionally, on I-35W, property damage only accidents, severe accidents, daytime accidents, and multivehicle accidents (other than rear-end accidents) increased disproportionately in entrance ramp areas during construction. Conversely, accident frequencies did not increase significantly ($a = 0.05$) in either nonentrance ramp areas or entrance ramp areas considered as a group on I-45. No category of accident was found to have disproportionately increased in nonentrance ramp areas or entrance ramp areas on I-45.

Safety in urban freeway construction work zones is of major importance to the designers, builders, and users of these facilities. Many researchers have examined the issue of construction work zone safety. Their findings have often differed in magnitude but most researchers have reported that accident rates in construction work zones are greater than on highways not under construction (*1–5*).

Ullman and Krammes (*1*) reported that total mainlane accidents on five urban freeway reconstruction projects in Texas increased an average of 28.7 percent during construction. They noted that the magnitude of the change in accident rates varied among the project sites studied. The researchers hypothesized that the observed variation in changes in accident occurrence may be better understood through detailed studies of the specific traffic control and geometric design features associated with those long-term freeway reconstruction sites. Ullman and Krammes (*1*) proposed that specific features to be studied should include shoulder widths, ramp geometry, advance signing, lighting, type and location of channelizing devices, and nature of the work activity.

Entrance ramps within construction work zones are areas where numerous decisions must be made by drivers in a limited amount of time. Merging operations at entrance ramps are very complex as entering vehicles and vehicles already on the freeway compete for space (*6*). The merging process is presumably made more difficult in construction work zones when confounded by the presence of construction equipment and construction workers, the presence and proximity of traffic control devices, and geometric constrictions imposed by the work zone.

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SCOPE AND OBJECTIVES OF WORK

The two sites selected for analysis are in the two largest metropolitan areas in Texas: Houston and Dallas-Ft. Worth. The specific objectives of this research were as follows:

1. Determine changes in accident occurrence during construction at urban freeway entrance ramp areas compared with changes in accident occurrence at nonentrance ramp areas within the studied work zones.
2. Through regression analysis, explore the potential relationship between accident rates at entrance ramps in construction work zones and selected geometric factors that are believed to contribute to entrance ramp accidents.

I-35W in Ft. Worth and I-45 in Houston are both radial freeways reconstructed to serve the growing metropolitan areas. Segments selected for study are similar for each site in terms of adjacent land usage (primarily residential and strip commercial). The segment of I-35W studied was 6.4 mi long and extended north from Felix Street to Hattie Street. The segment of I-45 studied was 7.8 mi long and extended north from downtown Houston to North Shepard Drive. Construction phases studied on I-35W and I-45 consisted of similar types of work. Additional lanes were added outside the existing roadway, ramps were upgraded, and frontage roads were improved. These work efforts required that ramp geometrics sometimes be altered during construction to allow contractors room to build the additional lanes. Long-term ramp closures were common on I-35W. Short-term ramp closures were common on I-45. Typical traffic control at the entrance ramps on the two projects is shown in Figures 1 and 2.

Mainline traffic volumes at each site were only slightly affected by construction. Ramp volumes on I-35W appeared to have decreased slightly during construction, presumably because of the closure of several ramps. Ramp volumes on I-45 did not appear to have changed appreciably during construction (*7*).

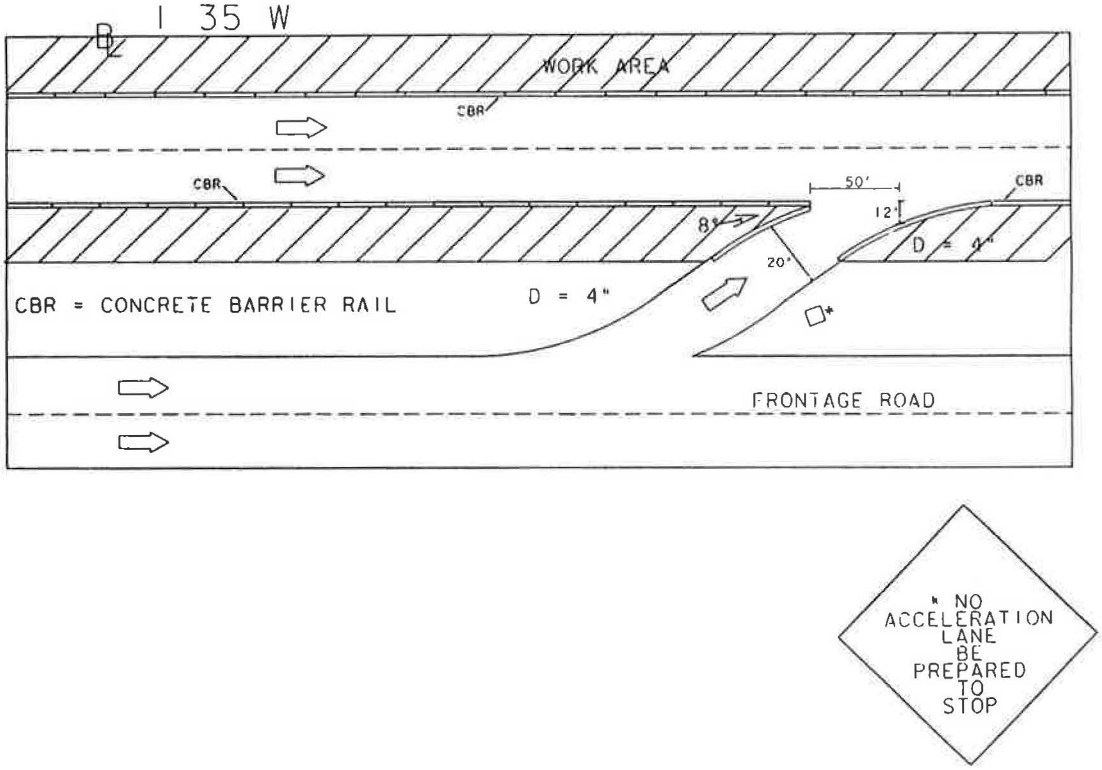

FIGURE 1 Typical traffic control at I-35W.

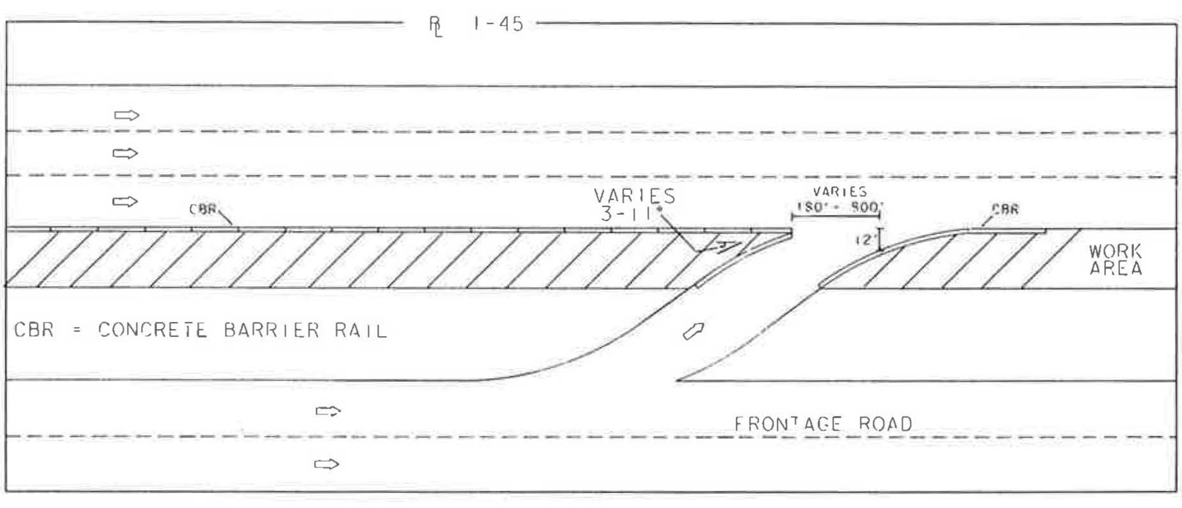

FIGURE 2 Typical traffic control at I-45.

STUDY DESIGN

Data Collection and Analysis

The *Highway Capacity Manual* (*8*) definition of the influence area of an entrance ramp was used to identify the limits of entrance ramp areas for this analysis. This area is defined as extending 500 ft upstream and 2,500 ft downstream of the nose of the entrance ramp. However, the presence of downstream exit ramps can affect the influence area of the entrance ramp, depending on their proximity. The *Highway Capacity Manual* (*8*) specifies the influence area of an exit ramp as extending from 2,500 ft upstream of the ramp to a point 500 ft downstream. Therefore, at entrance ramps with downstream exit ramps in close proximity, the entrance ramp was defined as extending from 500 ft upstream of the nose of the ramp to a point approximately one-half way to the downstream exit ramp, or 2,500 ft, whichever was less.

Considering the ramp area to be much larger than the actual area of influence would act to dilute the effects of the entrance ramp on accidents in the selected section by including an increased number of purely nonentrance ramp area accidents.

Conversely, an area confined to the immediate entrance ramp acceleration lane is not practical because of the nearest 0.1-mi coding procedure used to establish the location of accidents. Also, an entrance ramp conflict may manifest itself in an accident some distance away (if, for example, two motorists already on the freeway are so concerned with entering traffic that they fail to adequately consider the other's position and are involved in an accident with each other).

Conceptually, differences in accident occurrence between entrance ramp areas and nonentrance ramp areas should be the result of the effects of the entrance ramp, provided there were no other localized changes in construction methods or traffic control in either of the areas.

Roadway inventory logs and construction plans furnished by the Texas Department of Transportation (TxDOT) were used to determine the location and geometrics of entrance ramps. Accident data were obtained primarily from Department of Public Safety (DPS) accident file tapes.

Comparative Analysis of Entrance and Nonentrance Ramp Area Accidents

The first objective of this study was to examine the change in the accident frequency of entrance ramp area accidents in relation to the remainder of the construction work zone (nonentrance ramp areas). Accident frequencies were used as the basis of comparison because consistently defined areas were evaluated over time to determine whether accidents increased disproportionately in these areas. The areas were defined as entrance ramp areas and nonentrance ramp areas and the physical dimensions of each of the areas compared remained constant throughout the analysis. In each case, accidents within a control section were compared over time with the construction work zone under study. Comparisons were made of multiple years before and during construction. The analysis methodology followed the procedure outlined by Griffin for the evaluation of the effectiveness of safety improvements at a site (*9*). The methodology has also been used by others for the evaluation of the effects of construction on accidents over a specific length of roadway (*1,10*).

The use of a control section helps to factor out the effects of extraneous variables, and the use of multiple years of data provides some control for the regression-to-the-mean phenomenon. Control sections for the study sites were located immediately upstream or downstream of the construction work zones. The control sections were characterized as having similar before-construction geometrics to the construction work zone areas. Control sections allow for an estimation of how accidents in the construction area would have changed over time if there had been no construction. In other words, if the control and construction sections are comparable before construction, then differences between the changes in accident frequency at the work zone and control section over time are assumed to be caused by construction efforts.

Three phases were used to systematically analyze the effect of construction on accident frequency. The first phase of the analysis was to determine whether accidents increased in the nonentrance ramp areas during construction. The second phase was to determine if there was an increase in accidents in the entrance ramp areas. The third phase was conducted to determine if accidents increased disproportionately in the entrance ramp areas within the construction work zone compared with nonentrance ramp areas within the work zone.

Each phase of the analysis was performed in two steps: (a) check for comparability and (b) determine the impact of construction.

In the analysis of nonentrance ramp areas, the check for comparability was between nonentrance ramp areas in the control section (considered as a group) and nonentrance ramp areas in the work zone (also considered as a group). Likewise, in the second analysis phase, a comparability check was made between entrance ramp areas in the control section and entrance ramp areas in the work zone. Finally in the third phase, a check for comparability was made between nonentrance ramp areas and entrance ramp areas in the work zone.

Analyses of the comparability of control sections and construction work zones were performed for the before-construction period using the maximum-likelihood goodness-of-fit statistics, G_B^2, proposed by Griffin (*9*) and previously employed by Texas Transportation Institute (TTI) researchers (*1,10*). The G_B^2 statistic is used to determine whether the rates of change in accident frequencies between the two sites (control and construction sections) are comparable over periods of time.

The second step in each phase of the analysis, determination of the impact of construction upon accident frequencies, was accomplished by computing the differences in the rates of change at the areas under study, either nonentrance or entrance ramp areas, and the control sections for which comparability was established in the preceding step.

Accident frequencies in entrance and nonentrance ramp areas were compiled for several years before and during construction. Several categories of accidents were evaluated. These categories were total accidents, accident severity, time of day, and type of accident. Severe accidents were classified as fatalities, injury, and possible injury accidents. Noninjury accidents were classified as property damage only (PDO) collisions. Dawn, dusk, and night accidents were classified as nighttime accidents; others were classified as daytime collisions. Three types of accidents were classified: single vehicle, rear-end, and other multiple-vehicle accidents.

If the control sections and work zones were found to be comparable, the magnitude of the impact of construction on accident frequency was computed through the use of a cross-product ratio on a collapsed 2×2 (before-during, control-construction) contingency table. The percentage change in accident frequencies caused by construction was then computed using this cross-product ratio. The significance of the percentage change was determined using a two-tailed z-statistic.

The statistical analysis procedure is analogous to a before-after analysis with a control section and check for comparability with only one after period. The duration of the construction period is analogous to the one after period. The procedure is more fully explained by Griffin (*9*) and Ullman and Krammes (*1*).

Ramp Geometrics and Accident Rate Analysis

Research indicates that acceleration lane length, angle of convergence, and ramp grade are important geometric factors that affect the safety of entrance ramps (*11–13*).

The second objective of this study dealt with the possible relationship between geometric factors and accidents occurring in entrance ramp areas within freeway work zones. Multiple variable linear regression analysis was used to investigate these relationships. In this analysis, accident rates, instead of accident frequencies, were used as the basis of comparison because individual entrance ramp areas were being compared with other individual entrance ramp areas. The exposure factor used to determine the accident rate was the sum of the volume on the freeway mainlanes just before the entrance ramp and the volume entering on the ramp.

STUDY RESULTS

Summarized in Table 1 are the periods of analysis and accident frequencies analyzed for the two projects. The construction studied on the I-35W project lasted from August 1984 through June 1988. The nonentrance ramp areas were the freeway mainlanes between the 15 entrance ramp areas before construction. The areas at the eight entrance ramps that were closed for extended periods during construction were not included in the analysis. The construction activities studied on I-45 in Houston extended from March 1985 through May 1987. There were 17 entrance ramps in the I-45 construction section.

I-35W, Ft. Worth: Analysis of Nonentrance Ramp Areas

Summarized in Table 2 are the results of the I-35W Phase One analysis. It was determined that nonentrance ramp areas in the construction work zone and the control section were comparable for almost all the categories of accidents studied. Total accidents, PDO accidents, daytime accidents, rear-end accidents, and other multivehicle accidents were found to have followed similar trends in the control and construction section nonentrance ramp areas during the before-construction period. Statistically significant ($a = 0.05$) increases in accident frequencies were found for several categories of accidents. Total accidents increased significantly (35 percent) during construction in nonentrance ramp areas. Likewise, the 31 percent increase in PDO accidents was found to be statistically significant, as was the 44 percent increase in severe accidents. Additionally, daytime accidents increased 29 percent and rear-end accidents increased 73 percent, both statistically significant ($a = 0.05$).

I-35W, Ft. Worth: Analysis of Entrance Ramp Area Accidents

Summarized in Table 3 are the results of the second phase of the I-35W analysis. Comparability between the construction section entrance ramp areas and the control section entrance ramp areas was accepted for all but the PDO accident frequencies ($a = 0.05$).

Total accidents increased significantly (61 percent) during construction in the entrance ramp areas. Statistically significant increases in severe accidents were computed to be 95 percent; likewise, daytime accidents were found to have increased 64 percent, whereas nighttime accidents increased 56 percent. Increases in rear-end accidents were computed to be 64 percent and increases in other multivehicle accidents were found to be 97 percent (both statistically significant).

TABLE 1 Summary of Analysis Periods and Accident Frequencies

I 35W, FT. WORTH		
Section	Accident Frequency Before-Construction January, 1982 - July, 1984	Accident Frequency During-Construction August, 1984 - June, 1988
Construction Section:		
Non-Entrance Ramp Areas	591	1155
Entrance Ramp Areas	294	749
Control Sections:		
Non-Entrance Ramp Areas	284	412
Entrance Ramp Areas	521	823
I 45, HOUSTON		
Section	Accident Frequency Before-Construction January, 1982 - December,1983	Accident Frequency During-Construction March, 1985 - May, 1987
Construction Section:		
Non-Entrance Ramp Areas	1054	1282
Entrance Ramp Areas	902	1137
Control Sections:		
Non-Entrance Ramp Areas	110	105
Entrance Ramp Areas	62	70

TABLE 2 Accident Analysis Table for I-35W Nonentrance Ramp Areas

Accident Category	Check for Comparability Before- Construction G_B^2	Percent Change in Accident Frequency	Significance of Percent Change in Accident Frequency (Z-Statistic)
Total Accidents	0.46 [a]	+ 34.7 [b]	3.23
Accident Severity:			
PDO	0.48 [a]	+ 30.7 [b]	2.46
Severe	0.69 [a]	+ 44.0 [b]	2.08
Time-of-Day:			
Daytime	2.87 [a]	+ 29.0 [b]	2.20
Nighttime	8.20	+ 46.1	2.49 [c]
Type of Collision:			
Single Vehicle	7.47	+ 38.3	1.89 [c]
Rear-End	4.74 [a]	+ 73.0 [b]	3.05
Other Multi-vehicle	1.40 [a]	+ 16.1	1.07

[a] Control and construction sections are statistically comparable prior to construction ($\alpha = 0.05$); $G_B^2 < 5.99$.

[b] Percent Change due to construction is statistically significant ($\alpha = 0.05$); $|z| > 1.96$.

[c] Conclusions concerning statistical significance cannot be made due to lack of comparability before construction.

TABLE 3 Accident Analysis Table for I-35W Entrance Ramp Areas

Accident Category	Check for Comparability Before-Construction G_B^2	Percent Change in Accident Frequency	Significance of Percent Change in Accident Frequency (Z-Statistic)
Total Accidents	3.83 [a]	+ 61.3 [b]	5.39
Accident Severity:			
PDO	6.45	+ 50.4	3.94 [c]
Severe	1.51 [a]	+ 95.1 [b]	3.82
Time-of-Day:			
Daytime	1.49 [a]	+ 63.7 [b]	4.59
Nighttime	4.75 [a]	+ 55.9 [b]	2.81
Type of Collision:			
Single Vehicle	3.40 [a]	+ 5.8	0.30
Rear-End	0.27 [a]	+ 64.2 [b]	2.90
Other Multi-Vehicle	1.84 [a]	+ 97.4 [b]	5.33

[a] Control and construction sections are statistically comparable prior to construction ($\alpha = 0.05$); $G_B^2 < 5.99$.

[b] Percent Change due to construction is statistically significant ($\alpha = 0.05$); $|z| > 1.96$.

[c] Conclusions concerning statistical significance cannot be made due to lack of comparability before construction.

I-35W, Ft. Worth: Analysis of Nonentrance Ramp Areas Versus Entrance Ramp Areas

Summarized in Table 4 are the results of the third phase of the analysis of the I-35W accident frequency data. Comparability was computed between the nonentrance and entrance ramp areas within the construction work zone for the before-construction period. All categories of accidents were found to be comparable before construction. This was expected because the two areas being compared were merely parts of the same section of roadway and thus should have been exposed to the same extraneous events during the before-construction period.

The item of interest in this analysis was the relative difference in the change in accident frequencies between nonentrance and entrance ramp areas. This difference was computed using the cross-product ratio outlined previously, and indicates the magnitude to which increases in accident frequencies between entrance and nonentrance ramp areas were disproportionate. A ratio greater than 1 indicated that accident frequencies increased more in entrance ramp areas than in nonentrance ramp areas. Conversely, a ratio less than 1 indicated that accident frequencies increased more in nonentrance ramp areas than in entrance ramp areas.

Five categories of accidents were found to have increased significantly (a = 0.05) more in entrance than in nonentrance ramp areas. No category of accidents was found to have increased disproportionately more in the nonentrance ramp areas. It was determined that total accident frequencies increased significantly (30 percent) more in entrance ramp areas than in nonentrance ramp areas during construction on the I-35W project in Ft. Worth. Likewise, PDO accident frequency increases were 26 percent greater and severe accident increases were 46 percent greater in entrance ramp areas than in nonentrance ramp areas. Daytime accident increases in entrance ramp areas were 35 percent greater than in nonentrance ramp areas. Finally, multivehicle accidents (other than rear-end accidents) were found to have increased 49 percent more in entrance ramp areas during construction.

I-45, Houston: Analysis of Nonentrance Ramp Areas

Summarized in Table 5 is the first phase of the analysis of the I-45 accident frequency data. All categories of accidents were found to be statistically comparable between the control section and work zone before construction. Construction was found to have a significant (a = 0.05) impact on only two categories of accidents. Severe accidents were found to have increased 64 percent and nighttime accidents increased 123 percent during construction in nonentrance ramp areas.

I-45, Houston: Analysis of Entrance Ramp Areas

A summary of the second phase of the I-45 analysis is shown in Table 6. Work zone entrance ramp areas and control section entrance ramp areas were found to be comparable before construction for all of the categories of accidents studied.

No percent change in accident frequencies in entrance ramp areas was found to be statistically significant (a = −0.05). The failure to find the percentage changes in accident frequencies to be significant may have been because of Type II errors in many cases. Also it should be noted that some of

TABLE 4 Accident Analysis for I-35W Nonentrance Ramp Areas Versus Entrance Ramp areas in Construction Section

Accident Category	Check for Comparability Before-Construction G_B^2	Percent Difference in Change in Accident Frequency	Significance of Percent Change in Accident Frequency (Z-Statistic)
Total Accidents	0.57 [a]	+ 30.4 [b]	3.10
Accident Severity:			
PDO	0.29 [a]	+ 26.1 [b]	2.33
Severe	4.96 [a]	+ 45.6 [b]	2.23
Time-of-Day:			
Daytime	0.62 [a]	+ 34.7 [b]	2.86
Nighttime	1.59 [a]	+ 22.5	1.36
Type of Collision:			
Single Vehicle	3.00 [a]	+ 4.4	0.24
Rear-End	0.50 [a]	+ 15.4	0.90
Other Multi-Vehicle	0.50 [a]	+ 49.2 [b]	3.23

[a] Control and construction sections are statistically comparable prior to construction (α = 0.05); $G_B^2 < 5.99$.

[b] Percent difference in the change in accident frequencies due to construction is statistically significant (α = 0.05); $|z| > 1.96$.

TABLE 5 Accident Analysis Table for I-45 Nonentrance Ramp Areas

Accident Category	Check for Comparability Before-Construction G_B^2	Percent Change in Accident Frequency	Significance of Percent Change in Accident Frequency (Z-Statistic)
Total Accidents	2.40 [a]	+ 27.4	1.70
Accident Severity:			
PDO	2.42 [a]	+ 5.2	0.26
Severe	0.15 [a]	+ 64.3 [b]	2.34
Time-of-Day:			
Daytime	2.81 [a]	- 14.4	- 0.81
Nighttime	1.02 [a]	+ 122.8 [b]	3.58
Type of Collision:			
Single Vehicle	1.10 [a]	+ 40.8	1.20
Rear-End	1.00 [a]	+ 22.9	0.80
Other Multi-Vehicle	0.38 [a]	+ 27.4	1.12

[a] Control and construction sections are statistically comparable prior to construction ($\alpha = 0.05$); $G_B^2 < 3.84$.

[b] Percent Change due to construction is statistically significant ($\alpha = 0.05$); $|z| > 1.96$.

TABLE 6 Accident Analysis Table for I-45 Entrance Ramp Areas

Accident Category	Check for Comparability Before- Construction G_B^2	Percent Change in Accident Frequency	Significance of Percent Change in in Accident Frequency (Z-Statistic)
Total Accidents	0.10 [a]	+ 11.7	0.61
Accident Severity:			
PDO	1.06 [a]	+ 7.8	0.29
Severe	0.17 [a]	+ 32.2	1.09
Time-of-Day:			
Daytime	0.25 [a]	- 8.9	-0.41
Nighttime	0.06 [a]	+ 57.4	1.52
Type of Collision:			
Single Vehicle	0.005 [a]	+ 25.5	0.63
Rear-End	0.18 [a]	+ 52.9	1.19
Other Multi-Vehicle	0.02 [a]	- 10.1	-0.41

[a] Control and construction sections are statistically comparable prior to construction ($\alpha = 0.05$); $G_B^2 < 5.99$.

[b] Percent Change due to construction is statistically significant ($\alpha = 0.05$); $|z| > 1.96$.

the ramps within the work zone maintained fairly good operations during construction. Considering all entrance ramps together within the work zone may have diluted the effects at ramps where conditions were not so favorable. Nevertheless, it was felt that it was not appropriate to attempt to isolate only those ramps on which problems existed, and so all entrance ramps were considered as a group.

I-45, Houston: Analysis of Nonentrance Ramp Versus Entrance Ramp Areas

The results of the third phase of analysis of accidents on I-45 are summarized in Table 7. Trends in all categories of accidents were found to be comparable between nonentrance and entrance ramp areas in the construction section before construction.

There were no statistically significant (a = 0.05) differences in the effect of construction on entrance and nonentrance ramp areas on I-45. Differences in the change in total accident frequencies between entrance and nonentrance ramp areas were less than 4 percent. Thus, the relative effect of construction on these areas appeared to be about equal.

Results of Entrance Ramp Geometric Feature Analysis

The dependent variable and independent variables used in the multiple linear regression analysis for I-35W, Fort Worth, are summarized in Table 8, and those for I-45, Houston in Table 9. Ramp geometrics on I-35W in Ft. Worth were greatly altered during construction. Conversely, ramp geometrics on I-45 in Houston were not as greatly affected during most of the construction efforts. Most of the entrance ramps in Ft. Worth had approximately the same angles of convergence and acceleration lane length. The geometrics of the ramps in Houston varied considerably.

From Tables 8 and 9, it can be seen that accident rates during construction were generally higher in Ft. Worth than they were in Houston. Because of the apparent differences in the two projects and the lack of range in the independent variables on the I-35W project, separate regression models were developed for the two projects. A multiple regression analysis, which combined the two projects, may have been misleading because interactions among some of the independent variables may not have been similar among projects.

A summary of the regression analysis of the I-45 data is shown in Table 10. The overall model was found to be statistically significant (F-value = 4.15), and the coefficient of determination (R^2) was computed to be 0.58 (that is, 58 percent in the variation in the data could be explained by the model). However, only the entrance ramp accident rate before construction was found to be a significant variable in the prediction of accident rates at the ramp during construction. The final model related accident rates before construction to the natural logarithm of accident rates during construction. Hence, the model indicates that accident rates during construction are exponentially related to accident rates before construction. That is, those ramps already experiencing higher accident rates before construction will be more adversely affected by construction than those with lower accident rates initially.

A similar analysis performed on the I-35W data did not yield a statistically significant model. A possible reason for

TABLE 7 Accident Analysis Table for I-45 Nonentrance Ramp Areas Versus Entrance Ramp Areas in Construction Section

Accident Category	Check for Comparability Before- Construction G_B^2	Percent Difference in Change in Accident Frequency	Significance of Percent Change in Accident Frequency (Z-Statistic)
Total Accidents	0.07 [a]	+ 3.6	0.59
Accident Severity:			
PDO	0.18 [a]	- 2.0	-0.28
Severe	0.01 [a]	+ 19.0	1.55
Time-of-Day:			
Daytime	0.02 [a]	+ 10.0	1.28
Nighttime	0.44 [a]	- 7.3	-0.71
Type of Collision:			
Single Vehicle	2.54 [a]	+ 9.8	0.59
Rear-End	0.22 [a]	+ 1.4	0.10
Other Multi-Vehicle	1.50 [a]	- 1.7	-0.21

[a] Control and construction sections are statistically comparable prior to construction (α = 0.05); $G_B^2 < 5.99$.

[b] Percent difference in the change in accident frequencies due to construction is statistically significant (α = 0.05); $|z| > 1.96$.

TABLE 8 Summary of Geometric Factors During Construction at Entrance Ramps, I-35W, Ft. Worth

RAMP ID *	ANGLE OF CONVERGENCE (DEG.)	LENGTH OF ACCEL. LANE (FT)	RELATIVE GRADE (%)	AVG. ACC. RATE BEFORE (ACC/MVM)	AVE. ACC. RATE DURING (ACC/MVM)
S358	8	50	-1.7	1.40	11.76
N265	8	50	-1.8	2.45	9.10
N414	8	50	-2.1	2.93	4.25
S278	8	50	-2.3	3.95	9.51
S433	8	50	-3.7	3.84	7.63
N340	8	50	-4.1	3.84	4.95

* Ramps identified by direction of travel and approximate station number.

TABLE 9 Summary of Geometric Factors During Construction at Entrance Ramps, I-45, Houston

RAMP ID *	ANGLE OF CONVERGENCE (DEG.)	LENGTH OF ACCEL. LANE (FT)	RELATIVE GRADE (%)	AVG. ACC. RATE BEFORE (ACC/MVM)	AVE. ACC. RATE DURING (ACC/MVM)
N169	3	280	+0.5	1.83	2.05
S291	3	500	-3.0	4.85	4.68
S219	4	500	+2.5	1.45	2.45
S158	5	180	-0.5	1.02	2.74
S130	5	750	-3.0	1.49	2.44
N246	6	220	+2.0	5.90	6.76
N203	6	300	-4.0	2.39	1.37
S186	6	380	+4.0	1.59	2.56
S373	6	430	0	2.01	4.28
S250	7	700	0	4.80	5.40
S478	8	375	+4.0	1.71	1.94
N111	8	800	-1.1	1.31	1.86
S434	10	240	-0.5	1.59	2.14
N344	10	400	-0.5	1.63	3.25
N460	10	450	-1.0	2.14	3.08
S321	11	250	0	2.79	6.74
N409	11	425	0	1.63	2.99

* Ramps identified by direction of travel and approximate station number.

this was that accident rates before construction did not vary as dramatically as at the I-45 site. Also, since entrance ramp geometrics during construction were somewhat similar, the regression analysis may not have been able to statistically account for their effect on accident rates.

SUMMARY OF FINDINGS

Both nonentrance and entrance ramp areas on I-35W experienced statistically significant ($a = 0.05$) increases in total accident frequencies during construction. Total accident frequencies increased 35 percent in nonentrance ramp areas and 61 percent in entrance ramp areas during construction. Increases in accidents in the entrance ramp areas during construction were found to be significantly greater than increases in accidents in the nonentrance ramp areas. On I-35W, total accident frequencies increased 30 percent more in entrance ramp areas relative to nonentrance ramp areas (statistically significant at $a = 0.05$).

On I-35W, PDO and severe accident increases were disproportionately concentrated in entrance ramp areas. Also, daytime and multivehicle accidents (other than rear-end accidents) were found to have disproportionately increased in entrance ramp areas on I-35W during construction.

TABLE 10 Summary of Regression Analysis for Exploratory Model, I-45, Houston

LOG (ACC RATE) = β_0 + β_1 (ANGLE OF CONVERGENCE) + β_2 (ACCEL. LANE LENGTH) + β_3 (RELATIVE GRADE) + β_4 (ACC RATE BEFORE)		
INDEPENDENT VARIABLE	PARAMETER ESTIMATE	t_{calc}
Intercept	0.23	0.61
Angle of Convergence	0.04	1.32
Acceleration Lane Length	0.00	-0.07
Relative Grade	0.03	0.70
Accident Rate Before Construction	0.25	3.98*
Model F-statistic =4.15 ** Model R^2 = 0.58		

* statistically significant at $\alpha/2$ = 0.025
** statistically significant at α = 0.05

On I-45, neither nonentrance ramp nor entrance ramp areas experienced statistically significant (a = 0.05) increases in total accident frequencies during construction, when examined separately. The increases for nonentrance and entrance ramp areas (though not statistically significant) on I-45 were found to have been 27 percent and 12 percent, respectively. Accidents in entrance ramp areas increased 4 percent more relative to accidents in nonentrance ramp areas on the I-45 project. This difference in proportional increase was not statistically significant at a = 0.05 No accident category increased disproportionately in either nonentrance ramp or entrance ramp areas on I-45 in Houston.

Because of the limited data, relationships were not found between geometric data and accidents in the entrance ramp areas at either project. However, the I-45 data may suggest that entrance ramps having higher accident rates before construction were more adversely affected during construction than were ramps with lower accident rates before construction. It may be wise to give extra attention to the work zone traffic control at entrance ramps with higher accident rates.In some cases, it may be prudent to actually close these ramps during construction rather than further compromise ramp geometrics (or sight distance) during construction.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views and policies of the Texas Department of Transportation. The paper does not reflect a standard, specification, or regulation.

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Effectiveness of Steady-Burn Lights for Traffic Control in Tangent Sections of Highway Work Zones

PRAHLAD D. PANT AND YONGJIN PARK

Examined in this study is the effectiveness of Type *C* steady-burn lights on drums marked with high intensity (Type *G*) reflective sheeting in tangent sections of highway construction work zones. The study was performed on several rural, unlighted four-lane divided highways, including a freeway. Curved and tapered sections of highways were excluded from the study. The measures of effectiveness included speed, lateral placement, acceleration noise, weaving, traffic conflict, and driver preference. Right and left lane closures were separately examined. Study results indicated that steady-burn lights on drums marked with high-intensity reflective sheeting have little effect, if any, on driver behavior in tangent sections of rural divided highways. The study recommended that the use of steady-burn lights on drums marked with high-intensity reflective sheeting in tangent sections of construction work zones in rural divided highways including Interstate freeways be discontinued.

A highway construction work zone creates a conflicting situation for vehicular traffic and work activity. In Ohio, the *Traffic Control Application Standards Manual* (*1*) and the Ohio *Construction and Materials Specifications* (*2*) require drums in highway construction work zones to have Type *G* (also known as high-intensity) reflective sheeting, and Type *C* steady-burn lights during lane closures at night. However, the effects of steady-burn lights used in conjunction with high-intensity reflective sheeting have not been understood.

The objective of the study was to examine the effects of steady-burn lights on drums with high-intensity reflective sheeting in tangent sections of highway construction work zones. The study was limited to rural unlighted four-lane divided highways including a freeway. Curved and tapered sections were excluded from the study. The lights were evaluated under dry, rainy, and foggy weather conditions.

LITERATURE REVIEW

National Cooperative Highway Research Program Report 236 (*3*) examined the effectiveness of selected traffic control devices, including steady-burn lights. The results of the NCHRP study were not particularly applicable in Ohio because Ohio uses drums with high-intensity reflective sheeting in work zones, not the Type I barricades with engineering grade sheeting as tested in the study.

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A study by the New Jersey Department of Transportation (DOT) (*4*) recommended switching from steady-burn lights to 5- × 10-in. yellow reflectors for delineating temporary concrete barriers in construction work zones. A study by the Kansas DOT (*5*) found that at least 29 states used steady-burn lights in construction zones, but 9 did not. A Virginia study (*6*) found no difference in vehicle placement and in speed using the steady-burn lights or the reflectorized panels on top of concrete barriers. A study by the Wisconsin DOT (*7*) found that Type *H* sheeting on barrels outperformed yellow warning lights and that therefore the lights could be omitted. Additionally, studies by the Wisconsin and New York DOTs found that when vehicles strike drums, battery-powered lights fly off, crashing into windshields and flying apart at high speeds. However, none of the previous studies examined the effectiveness of steady-burn lights on drums marked with high-intensity reflective sheeting.

MEASURES OF EFFECTIVENESS

This study is based on an examination of the driving behavior of a sample of driver subjects who were asked to drive an instrumented automobile in highway construction work zones. The measures of effectiveness are as follows:

1. *Speed*. The speed of the vehicle for each subject was measured continuously at 1.0-sec intervals over a length of ½ to ¾ mi of tangent sections of construction work zones.
2. *Lateral Placement*. The distance maintained by the subject between the vehicle and the longitudinal pavement markings was recorded continuously at ½-sec intervals.
3. *Acceleration Noise*. The acceleration noise used to determine the smoothness of the trip was calculated using the equation derived by Drew et al. (*8*).
4. *Weaving*. Weaving is defined as the rate of change in lateral displacement per unit time. The average weaving during the trip period consisting of n intervals is expressed as:

$$W_{avg} = \sum[ABS(X_i - X_{i-1})]/n$$

W = weaving,
ABS = absolute value,
X_i = lateral placements of the vehicle at times t_i (in this study, t was measured in units of ½ sec),
X_{i-1}, = lateral placements of the vehicle at times t_{i-1}, and
n = number of intervals

5. *Traffic Conflict.* A traffic conflict is defined as an unusual or drastic action such as braking to which a motorist may have to resort while driving a vehicle in a highway construction work zone.

6. *Driver Preference.* Immediately after the completion of the tests, each driver was asked to fill out a questionnaire describing age, sex, vision, education, driving experience, and mileage typically driven in a year. If subjects noticed the absence of steady-burn lights, they were asked to fill out a second questionnaire providing a subjective evaluation of the steady-burn lights.

DATA COLLECTION

An instrumented automobile was used for collecting speed and lateral placement data in several highway construction work zones. A distance measuring instrument was installed in the automobile. The distance traveled by the automobile at 1.0-sec intervals was downloaded into a portable personal computer. The automobile was equipped with a video camera installed on the roof.

The field data were collected in three rural highway construction work zones, including an Interstate freeway. Each highway was a limited access, unlighted four-lane divided highway with a wide grass median. The posted speed limits were either 65 or 55 mph. The test sections had no significant grade, and were generally ½ to ¾ mi long. Drums marked with high-intensity reflective sheeting were erected at 100- to 120-ft spacings. Some of the drums and pavement markings were dirty or worn and some were in relatively good condition.

The age group of driver subjects was divided into six categories from 16 to 75 years old. The sample size for the study was 132, with the number of driver subjects for each type of lane closure being 66.

None of the driver subjects were told why they were asked to drive the automobile. The first set of data for each subject was collected in the construction work zone in late afternoon. Then, during hours of complete darkness, the first set of nighttime data was collected with steady-burn lights placed on the drums. During the second set of nighttime data collection, the steady-burn lights were covered, without the subjects' knowledge, with black-surfaced wallpaper bags.

DATA ANALYSIS AND RESULTS

The data for right- and left-lane closures were separately analyzed (Table 1). Several hypotheses for speed, lateral placement, acceleration noise, and weaving were tested by performing t-tests for means and F-tests for variances at 5 percent level of significance. Additionally, paired t-tests were performed for these measures of effectiveness. The results of the analysis are described in the following paragraphs.

Speed

First, the mean speeds and speed variances at each site were separately tested. Then the data for all sites were combined to perform the remaining tests. Generally, the tests showed that there were no significant differences between the mean speeds or speed variances during the three periods. In a small number of cases in which the null hypotheses were rejected, the differences were too small to have any practical significance. The data were then categorized by weather conditions, ages of driver subjects, sex, and subjects who noticed the absence of steady-burn lights. The hypotheses were tested for each category. In general, all null hypotheses were accepted, indicating that the mean speeds or speed variances were not significantly different.

Additionally, paired t-tests were performed to examine whether the mean speeds between any two periods were significantly different. Similar results as those previously described were found from the tests (Table 2).

Lateral Placement, Acceleration Noise and Weaving

The statistical tests for lateral placement, acceleration noise, and weaving followed the same procedures as for mean speeds described in the previous section. In general, the null hypotheses were accepted, indicating that the means and variances are not significantly different. In some cases, the null hypotheses were accepted, indicating that the differences were significantly different. However, the differences were too small to have any practical importance.

Traffic Conflict

The tests showed that the absence of steady-burn lights on the drums did not cause any unusual or drastic action on the part of the subjects.

Driver Preferences

The results showed that only 13 subjects, or 10 percent, noticed the absence of steady-burn lights on drums during the second nighttime experiments. These subjects were asked to respond to specific questions for right- and left-lane closures on a scale of 1 to 10 as follows:

0	1	2	3	4	5	6	7	8	9	10
Strongly Disagree		Disagree			Undecided		Agree			Strongly Agree

In general, the subjects were undecided if their drives were affected by the absence of lights on the drums, if the drums did not provide a good path for them to drive through the work area, and if the absence of lights on the drums made them feel unsafe. Their responses lay between 5.0 and 5.9.

CONCLUSIONS AND RECOMMENDATIONS

The study has shown that steady-burn lights have little effect, if any, on driver behavior in tangent sections of rural, unlighted divided highways including Interstate freeways. It indicated that the high-intensity reflective sheeting outperformed the steady-burn lights and hence the presence or absence

TABLE 1 **Average Speed, Lateral Placement, Acceleration Noise, and Weaving**

		ALL	I-71	U.S.127	U.S.27
SPEED (MPH)	N	66	32	29	5
	SP1	53.3 (4.5)	55.9 (2.9)	50.4 (4.4)	53.7 (4.5)
	SP3	51.3 (5.2)	53.9 (3.6)	48.9 (5.4)	48.2 (6.5)
	SP5	52.4 (5.1)	54.2 (4.2)	50.7 (5.1)	50.1 (7.6)
	N	66	20	19	27
	SP2	53.2 (4.1)	55.6 (3.4)	50.2 (4.0)	53.4 (3.2)
	SP4	51.6 (4.6)	55.4 (3.5)	48.4 (3.5)	51.0 (4.1)
	SP6	51.9 (4.7)	55.1 (3.7)	48.5 (3.7)	51.9 (4.4)
LATERAL PLACEMENT (ft)	N	66	32	29	5
	LP1	0.9 (0.4)	0.9 (0.4)	0.9 (0.4)	1.0 (0.3)
	LP3	0.9 (0.4)	0.9 (0.5)	0.9 (0.4)	0.9 (0.2)
	LP5	1.0 (0.4)	0.9 (0.5)	1.0 (0.4)	1.0 (0.3)
	N	66	20	19	27
	LP2	1.5 (0.4)	1.5 (0.6)	1.3 (0.30	1.6 (0.3)
	LP4	1.5 (0.4)	1.4 (0.6)	1.5 (0.3)	1.6 (0.4)
	LP6	1.6 (0.4)	1.6 (0.6)	1.5 (0.3)	1.5 (0.3)
ACCELER-ATION NOISE (ft/sec^2)	N	66	32	29	5
	AC1	1.1 (0.2)	1.2 (0.2)	1.0 (0.1)	1.0 (0.0)
	AC3	1.0 (0.1)	1.1 (0.2)	1.0 (0.1)	1.0 (0.1)
	AC5	1.0 (0.2)	1.1 (0.2)	1.0 (0.1)	1.0 (0.1)
	N	66	20	19	27
	AC2	1.1 (0.2)	1.3 (0.2)	1.0 (0.2)	1.0 (0.1)
	AC4	1.0 (0.2)	1.2 (0.2)	0.9 (0.1)	0.9 (0.1)
	AC6	1.0 (0.2)	1.2 (0.3)	0.9 (0.1)	1.0 (0.1)
WEAVING (ft)	N	66	32	29	5
	WI1	1.1 (0.3)	1.3 (0.3)	0.9 (0.1)	0.8 (0.0)
	WI3	1.0 (0.3)	1.2 (0.3)	0.9 (0.1)	0.8 (0.0)
	WI5	1.1 (0.3)	1.3 (0.3)	0.9 (0.1)	0.8 (0.0)
	N	66	20	19	27
	WI2	0.9 (0.1)	1.0 (0.2)	0.8 (0.1)	0.8 (0.1)
	WI4	1.0 (0.2)	1.1 (0.2)	0.8 (0.1)	0.9 (0.1)
	WI6	0.9 (0.2)	1.1 (0.2)	0.8 (0.1)	0.9 (0.1)

For key to the variables, see TABLE 2.

TABLE 2 *T*-Test and Paired *T*-Test for Average Speed (5 Percent Level of Significance)

T-TEST			PAIRED T-TEST	
HYPOTHESES	MEAN	VARIANCE	HYPOTHESES	MEAN
SP1=SP3	S	NS	SP1-SP3 = 1 MPH	NS
SP1=SP5	NS	NS	SP1-SP5 = 1 MPH	NS
SP3=SP5	NS	NS	SP3-SP5 = -1 MPH	NS
SP2=SP4	NS	NS	SP2-SP4 = 1 MPH	NS
SP2=SP6	NS	NS	SP2-SP6 = 1 MPH	NS
SP4=SP6	NS	NS	SP4-SP6 = -1 MPH	NS

Key to variables

NS = No significant difference
S = Significant difference
SP = Speed
LP = Lateral placement
AC = Acceleration noise
WV = Weaving
( )= Standard deviation
N = Sample size

1,3 and 5 refer to right lane closure during daytime, nighttime with steady burn lights and nighttime without steady burn lights respectively.

2,4 and 6 refer to left lane closure during daytime, nighttime with steady burn lights and nighttime without steady burn lights respectively.

of the lights seemed to have little impact on the subjects' speed, lateral placement, acceleration noise, or weaving. There was little difference in the mean speeds among right- and left-lane closures. The tests showed that the subjects relied heavily on pavement markings for delineation, and to a lesser extent on drum placements.

The mean lateral placement for right-lane closures was about 60 percent higher than for left-lane closures. The results showed that highways with higher traffic volume and speed have a higher acceleration noise and weaving than those with lower volume and speed. In general, vehicles seemed to weave at a slightly higher rate in right-lane than in left-lane closures.

Finally, it is concluded that steady-burn lights are not required when drums with high-intensity reflective sheeting are used as channelizing devices in tangent sections of rural divided highways, including freeways.

It is recommended that the use of steady-burn lights erected on drums that are marked with high-intensity reflective sheeting be discontinued in tangent sections of construction work zones in rural divided highways, including Interstate freeways.

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Steady-Burn Lights in Highway Work Zones: Further Results of Study in Ohio

PRAHLAD D. PANT, XIAO H. HUANG, AND SUNDARESAN A. KRISHNAMURTHY

Examined in this paper is the effect of steady-burn lights on driver behavior in divided and undivided highways with horizontal and vertical curves; and with and without external lighting, entrance and exit ramps, tapered sections of lane closures, and median crossovers. The study was based on an examination of the driving behavior of a sample of driver subjects who were asked to drive an instrumented automobile in several highway work zones during the day, during the night when steady-burn lights were placed on drums, and during the night when steady-burn lights were removed. The effect of steady-burn lights on the lane-changing behavior of motorists in advance of the tapered sections was determined by recording traffic volume at three to four locations in each travel lane before the taper. The results showed that steady-burn lights had little effect, if any, on driver behavior in highway work zones. It appears that the high-intensity sheeting on drums and the flashing arrow panel had a powerful effect on drivers, thus leaving the steady-burn lights without any practical value in the work zones. It is concluded that steady-burn lights are not required for traffic control when drums with high intensity sheeting and flashing arrow panel are used as channelizing devices in these highway facilities.

A highway construction work zone generally creates conflicts between vehicular traffic and work activity, which leads to a reduction in the number of traffic lanes and to traffic conditions that violate motorist expectations. The work activity is also frequently compromised and worker movements restricted.

In Ohio, plastic drums are generally used for traffic control in highway construction work zones. These drums are required to have Type *G* (also known as high-intensity) reflective sheeting and Type *C* steady-burn lights for lane closures at night. A previous study (*1*,*2*) examined the effects of steady-burn lights in tangent sections of several rural, unlighted (e.g., without any external illumination), four-lane divided highways, including an Interstate freeway under dry, rainy, and foggy weather conditions. The study found that steady-burn lights have little effect, if any, on speed, acceleration noise, lateral placement and weaving when drums with high-intensity sheeting are used as channelizing devices in tangent sections of rural divided highways. No erratic driver behavior was observed when steady-burn lights were absent on drums. The study recommended that the use of steady-burn lights in tangent sections of construction work zones in rural divided highways, including freeways, be discontinued in the future.

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Further, the effects of steady-burn lights in curved, lighted, unlighted, and tapered sections, ramps, and median crossovers in work zones were examined during field tests in Ohio. The objective of the study was to determine whether steady-burn lights are needed in divided and undivided highways with horizontal and vertical curves, with and without external lighting, entrance and exit ramps, tapered sections of lane closures, and median crossovers. The following sections describe the research method, data collection, and results of the study.

RESEARCH APPROACH

An instrumented automobile was used to collect the required data in several highway work zones. Driver subjects were asked to drive the automobile during the day, at night when steady-burn lights were placed on drums, and at night when steady-burn lights were removed. A total of 321 runs were made, representing a sample of 107 driver subjects. The driver subjects did not drive the automobile in the same sequence. Some of the subjects started their first drive in daytime, some at nighttime with steady-burn lights on drums, and some at nighttime without steady-burn lights. The percentage of subjects with first drive in each period is shown in Figure 1. The driver subjects were not told of the presence or absence of steady-burn lights on the drums. None of the subjects were told why they were being asked to drive the automobile. Each subject was asked to drive the instrumented automobile as he or she would drive any other automobile. The tests without steady-burn lights were performed by covering the lights with black plastic bags. The bagging of steady-burn lights had the effect of having no steady-burn lights on the drums.

Right and left lane closures were examined separately. In addition, both right and left curves were examined by collecting data in both directions, as described in the later sections of this paper. The measures of effectiveness employed in the study consisted of

1. Speed: The speed parameters were the mean and variance. The speed of the vehicle for each test was measured continuously at 1.0-sec intervals. A large increase in mean speed or variance caused by the absence of steady-burn lights may indicate the existence of an abnormal and unsafe condition in the work zone. Similarly, a large decrease in mean speed may lead to delay and congestion which, under some circumstances, could create a hazardous condition, especially if the variance increased.

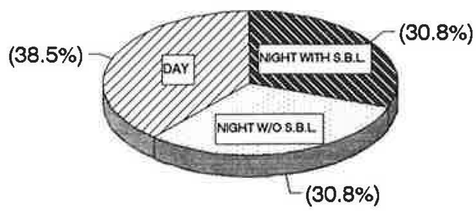

FIGURE 1 Sequence of starting first drive.

2. Lateral placement: When drivers see and approach an object near the path of the vehicle, they tend to move the vehicle away from the object. In this study, the lateral distance between the vehicle and the longitudinal pavement marking was measured at ½-sec intervals. Large variations in lateral placement may indicate driver confusion or an unsafe condition.

3. Acceleration noise: The standard deviation of acceleration is called acceleration noise and is used to determine the smoothness of the trip. A high acceleration noise normally represents a "rough" trip and a low acceleration noise a "smooth" trip. Acceleration noise was calculated for each trip based on the vehicular speeds recorded continuously at 1.0-sec intervals.

4. Weaving: Weaving is defined as the rate of change in lateral displacement per unit time. If X_i and X_{i-1} are lateral placements of the vehicle at times t_i and t_{i-1} respectively, then the average weaving over the trip period T consisting of n intervals is expressed as

$$W_{\text{avg}} = \sum [ABS\,(X_i - X_{i-1})]/n$$

where

W = weaving,
ABS = absolute value,
X_i = lateral placement of the vehicle at times t_i(in this study, t was measured in units of ½ sec),
X_{i-1} = lateral placement of the vehicle at times t_{i-1}, and
n = number of intervals.

A low value of weaving would indicate a "smooth" trip and a high value would indicate a "rough" trip.

5. Traffic conflict: A traffic conflict is defined as an unusual or drastic action by a driver while driving through a highway construction work zone. Evidence of an unusual or drastic action as a direct result of the absence of steady-burn lights would indicate a potentially dangerous situation.

6. Lane change before closure: Drivers in the lane to be closed are expected to change the lane long before they reach the taper or at least before the beginning of the taper. The effect of steady-burn lights on the lane-changing behavior of drivers was determined by recording traffic volume at three to four locations in each travel lane upstream of the taper during the three study periods (day, night with steady-burn lights, and night without steady-burn lights).

7. Driver preference: Immediately after the completion of the three tests, each driver was asked to fill out a questionnaire describing age, sex, vision, education, driving experience, mileage typically driven in a year, and involvement in any accident in a highway work zone in the past. Each driver subject was asked if there was any change or difference in the work zone between the two night tests. If the subject noticed the absence of steady-burn lights during the night tests, a subjective evaluation of the steady-burn lights was conducted through another questionnaire. The subjects who did not notice the absence of steady-burn lights were not given the second questionnaire.

DATA COLLECTION

The test automobile was equipped with a distance measuring instrument that continuously recorded the distance traveled by the automobile at 1.0-sec intervals and downloaded it into a portable personal computer. A Hi-8mm video camera was installed on the automobile roof and a 9-in. color monitor in the rear interior of the automobile. The video camera was enclosed in an environmental chamber consisting of a wiper, defogger, heater, and blower that allowed video photography under adverse weather conditions. The camera provided an approximately 6-ft-wide view of the roadway, including a partial view of the front exterior of the automobile. The automobile was equipped with extra lights underneath the front bumper so that a limited amount of illumination could be directed to the pavement markings for better photography at night. These lights, however, provided no extra illumination onto the driver's path. In addition, a flashlight bulb was connected to the automobile's rear brake system and installed in the rear interior to record braking, if any, during the test drive. None of the driver subjects were aware of the existence of the electric bulb. All highway construction work zones conformed to Ohio Department of Transportation (ODOT) specifications. Drums with high-intensity sheeting were placed at 100- to 120-ft spacing, which was reduced to 50 ft at tapered sections. The drums were generally placed 1 to 2 ft inside the lane closed for construction. The lane width in all sites was standard 12 ft. As required by ODOT specifications, a flashing arrow panel operated at the beginning of the taper. The length of the taper conformed to ODOT specifications. The contractor generally maintained the steady-burn lights in good condition. The conditions of the drums and the pavement markings varied from poor and dirty at some locations to good at others. A brief description of the characteristics of the work zones is provided in the following paragraphs.

Interstate 275 (Hamilton County)

This is a rural, six-lane divided highway (three lanes in each direction) with no external lighting and with right-lane closure in each direction. The posted speed limit was 65 mph. The highway had a degree of curvature of 2 deg. 45 min. 00 sec. and a grade of 3.0 percent. The average widths of the left and right shoulders were 9 to 10 ft and of the grass median 41 ft. Data were also collected at an entrance ramp and an exit ramp with right-lane closures. The posted speed limit in the ramps was 35 mph.

Interstate 74 (Hamilton County)

This is a suburban, four-lane divided highway (two lanes in each direction) with a 74-ft-wide (average) grass median. The highway work zones in each direction consisted of both lighted and unlighted roadways. The posted speed limit was 55 mph and the highway sections had right-lane closures in each direction. The maximum degree of curvature on the highway was 3 deg. 30 min. 00 sec. and the maximum grade was 3 percent. The widths of the left and right shoulders were 11 ft in the northbound direction and 7 to 11 ft in the southbound direction. Data were also collected at an entrance ramp and an exit ramp with right-lane closures. The posted speed limit in the ramps was 35 mph.

US-52 (Scioto County)

The section of US-52, where data were collected in both directions, is a rural, four-lane divided highway (two lanes in each direction). The posted speed limit was 55 mph. The data were collected both under lighted and unlighted conditions by switching the street lamps on or off as necessary. The work zone consisted of a right-lane closure in one direction and a left-lane closure in the other. The highway had a maximum degree of curvature of 3 deg 38 min. 33 sec. and a maximum grade of 1.0 percent.

US-27 (Hamilton County)

The data were collected in both directions of US-27 (just north of I-275), which is a suburban, four-lane undivided highway (two lanes in each direction) with no external lighting. The posted speed limit was 55 mph. The highway had right-lane closure in each direction. The degree of curvature was 4 deg. 00 min. 00 sec. and the grade was 3 percent. The average widths of the left and right shoulders were 4 to 8 ft.

SR-16 (Licking County)

This is a rural, unlighted four-lane (two lanes in each direction) divided highway with the posted speed limit of 55 mph. The data at this work zone were collected in a median crossover with a reverse curve (a left-lane closure followed by a right-lane closure). The degree of curvature for each curve was 3 deg. 00 min. 00 sec. The width of the left and right shoulders was 8 to 10 ft and the median width was approximately 40 ft.

The study grouped the driver subjects according to their ages as follows:

- Under 30 years,
- 31 to 45 years,
- 46 to 60 years, and
- 61 to 75 years.

The proportion of driver subjects in each age group is shown in Figure 2. A list of the test sites and the number of runs is shown in Table 1.

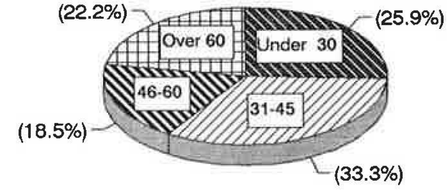

FIGURE 2 Age group of drivers.

DATA ANALYSIS AND RESULTS

In general, the speed and lateral placement data for each subject at each site were collected from the end of the tapered section to the end of the curved section of the work zone, with the total length of up to ¾ mi. The mean vehicular speeds for each period at each site are presented graphically in Figure 3. Because the geometric and traffic conditions at each site were different, the speeds at different sites are not comparable. For example, the daytime mean speed at I-275 (eastbound) was 66.4 mph, and at US-27 (northbound) was 47.7 mph. The analysis showed that the nighttime speeds were generally lower than the daytime speeds. However, the nighttime speed varied between different sites, with no clear indication of any pattern of high or low speeds when steady-burn lights were absent on the drums. A past study (*3*) has indicated that a speed difference of 4 mph or less is not of practical significance. In this study, no difference of greater than 4 mph was found at any site between the two night periods, with and without steady-burn lights. However, a few differences greater than 4 mph were found between the day and night speeds.

The reference line for measurement of lateral placement was generally the pavement marking closest to the camera. The average lateral placements for each period at each work zone are presented in Figure 4. These data indicate that lateral placements were slightly higher or lower at the sites when steady-burn lights were absent on drums, but provided no clear pattern on the effects of the steady-burn lights.

The acceleration noise and weaving for each period at each work zone are presented in Figures 5 and 6, respectively. In general, acceleration noises during daytime or on ramps were higher than those during other periods or at other facilities. However, no specific pattern of increase in acceleration noise existed on any particular type of lane closure or lighting condition.

In order to determine if the differences in the means between any two study periods were significant, several hypotheses were tested for each of the first four measures of effectiveness (MOEs) (speed, lateral placement, acceleration noise, and weaving) by performing paired *t*-tests at 5 percent level of significance. The SAS and STATGRAPHICS software were used to perform the tests. The paired *t*-tests examined the null hypothesis that the speed, lateral placement, acceleration noise, or weaving during any two of the three test periods (day, night with steady-burn lights, or night without steady-burn lights) remains unchanged. For example, the speed of each individual driver was compared for each of the three periods by taking the difference in speeds for two periods at a time. In this way, any difference in speed between day and night and also between the two nighttime periods for the same driver can be determined. The tests were first performed separately for each site. The data for the two directions of the same highway were then combined and further testing of the hypotheses was performed. For example, the

TABLE 1 List of Sites

No.	SITE	NUMBER OF RUNS	SPEED LIMIT (MPH)
1.	I-275 WB[a]	18	65
2.	I-275 EB[b]	18	65
3.	I-74 EB_1[c]	18	55
4.	I-74 EB_2[d]	18	55
5.	I-74 WB_2	18	55
6.	I-74 WB_1	18	55
7.	US 52 EB_1	15	55
8.	US 52 EB_2	30	55
9.	US 52 WB_2	30	55
10.	US 52 WB_1	15	55
11.	US 27 NB[e]	18	55
12.	US 27 SB[f]	18	55
13.	I-275 EX[g]	18	35
14.	I-275 EN[h]	18	35
15.	I-74 EX	18	35
16.	I-74 EN	18	35
17.	SR 16 CO[i]	15	55

[a]*West Bound* [b]*East Bound*
[c]*Lighted roadway* [d]*Unlighted roadway*
[e]*North Bound* [f]*South Bound*
[g]*Exit ramp* [h]*Entrance ramp*
[i]*Crossover*

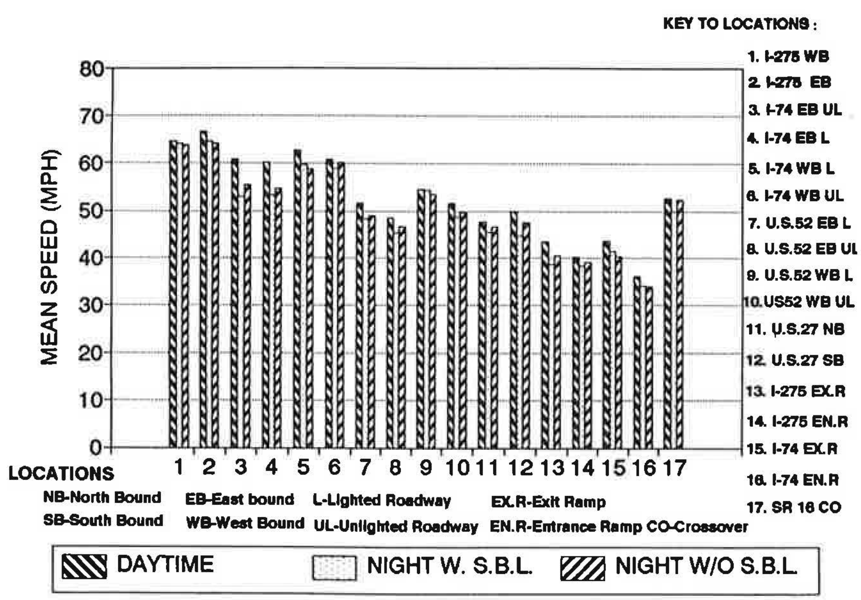

FIGURE 3 Mean speed.

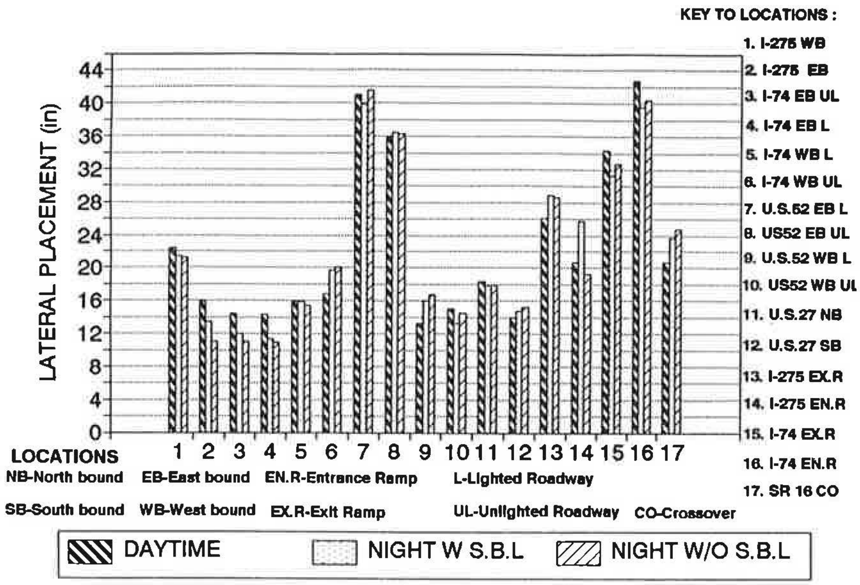

FIGURE 4 Mean lateral placement.

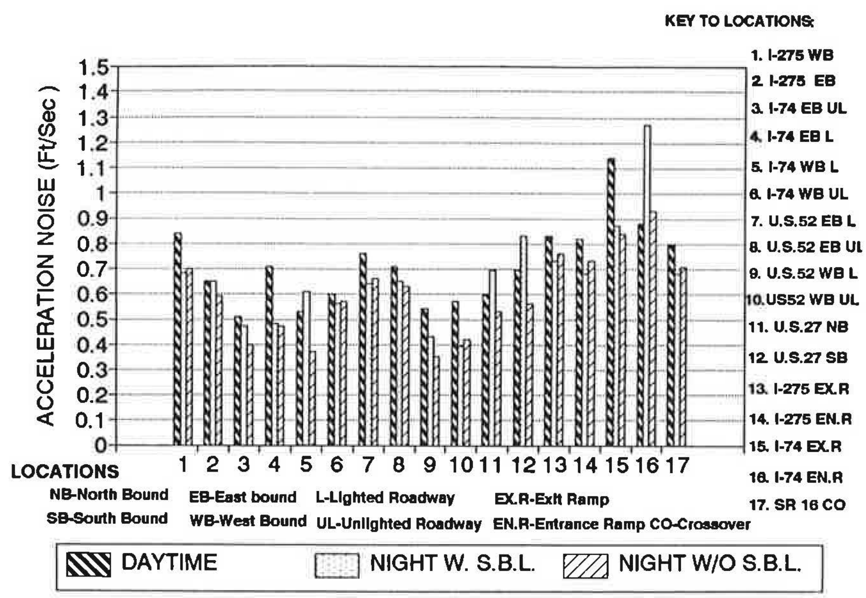

FIGURE 5 Mean acceleration noise.

speed data for west- and eastbound I-275 were combined because both directions had right-lane closures and the speed limits were the same. Finally, paired *t*-tests were performed by combining the data for all sites.

Further statistical analysis of the data was done by conducting one-way analysis of variance. For example, the speeds for the two nighttime periods were examined by taking the daytime speed as a control value. In other words, the differences between daytime and nighttime with steady-burn lights, and between daytime and nighttime without steady-burn lights were calculated and testing was conducted to determine whether these two speed differences were significant.

In general, the null hypotheses were accepted, indicating that the MOEs (speed, lateral placement, acceleration noise, or weaving) during the three periods were not significantly different. Analysis showed that steady-burn lights had no impact on the MOEs during lane closure at night. In a few cases, the null hypotheses between day and night tests (either with or without steady-burn lights) were rejected. However, there was no significant difference in the MOEs between the two night periods (with and without steady-burn lights on drums), indicating that the presence or absence of steady-burn lights had no impact on the MOEs. When tests were performed for weaving, a few null hypotheses between the two nighttime periods were rejected; however, the differences were too small to be of any practical importance.

In summary, the results of the study showed that there was no impact on the means and variances of speed, lateral place-

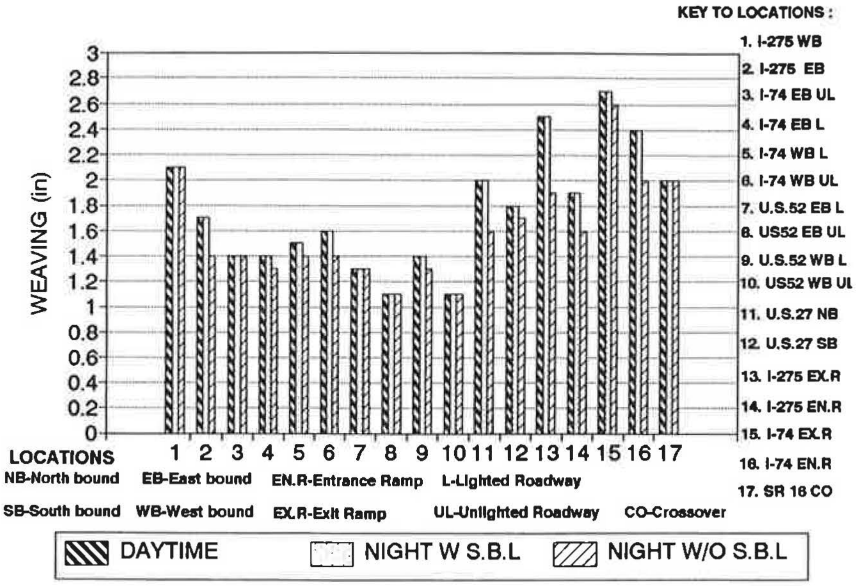

FIGURE 6 **Mean weaving.**

ment, acceleration noise, and weaving during the absence of steady-burn lights on drums with high-intensity reflective sheeting.

Traffic Conflict

The tests showed that a small number of subjects applied their brakes when they encountered slow or merging traffic. The absence of steady-burn lights on drums did not cause any unusual or drastic action on the part of the subjects.

Lane Change Before Closure

Traffic volume counts recorded in advance of the tapered sections were analyzed to determine the effects of steady-burn lights on the lane-changing behavior of motorists before the beginning of the tapered section. The highway section in advance of the taper was subdivided into three to four subsections (an example is shown in Figure 7). The analysis consisted of comparing the proportion of traffic volume changing lanes (from the lane to be closed to the open lane) and by testing the null hypothesis that the proportion of traffic volume in each subsection of the highway remained unchanged during the three periods. A rejection of the null hypothesis would indicate that the lane-changing behavior had been affected during the period under study. Z-tests were performed separately for each site to test the hypotheses. The results showed that there were some significant differences in the proportion of traffic volume during daytime and nighttime, either with or without steady-burn lights. However, the null hypotheses during the two nighttime periods were accepted, indicating that the proportions of traffic within each subsection were not significantly different during the two night periods. It showed that steady-burn lights had no effect on the lane-changing behavior of motorists at night.

Driver Preference

Among the 27 subjects who responded to the question, 9 (33.3 percent) noticed the absence of steady-burn lights during their drives. Seven of the 9 subjects were in the "under 30 years"

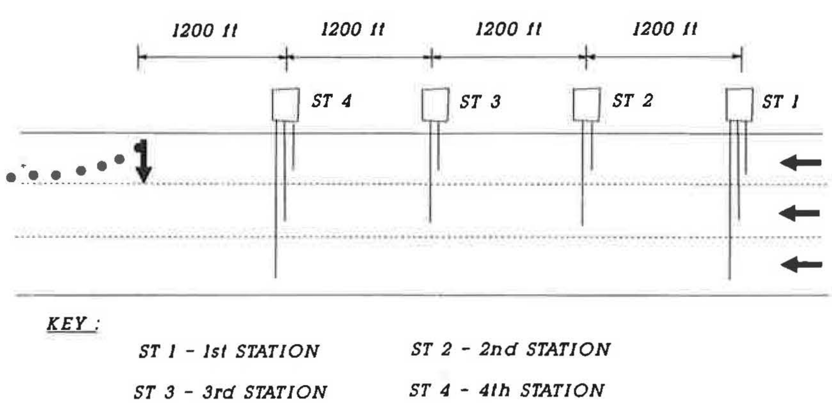

FIGURE 7 **Traffic counting on westbound I-275.**

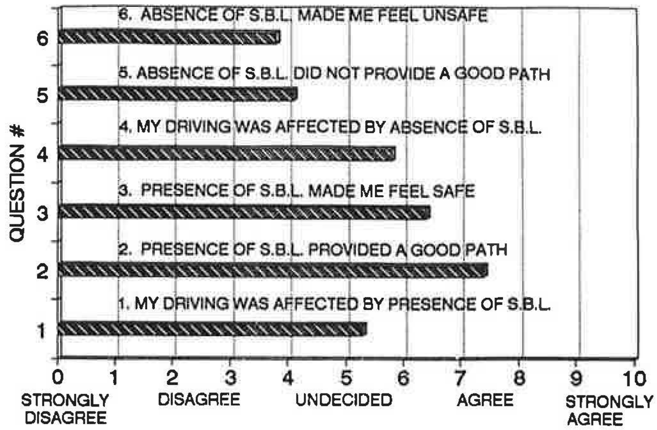

FIGURE 8 Results of driver responses.

age category, perhaps indicating that younger drivers were either sensitive to the change or curious about the experiments. The subjects who noticed the absence of steady-burn lights were asked six more questions, among which the first three were related to the presence of steady-burn lights, and the remaining three to the absence of steady-burn lights. Their responses on the following scale of 1 to 10 are shown in Figure 8.

0	1	2	3	4	5	6	7	8	9	10
Strongly Disagree		Disagree			Undecided		Agree			Strongly Agree

The results showed that the subjects did not have any strong opinions on the absence of steady-burn lights and did not feel unsafe because of the absence of steady-burn lights on drums.

CONCLUSIONS

Examined in this study is the effectiveness of steady-burn lights on drums with high-intensity sheeting in divided and undivided highways, including curved, lighted, unlighted, and tapered sections, and ramps and crossovers. The results showed that steady-burn lights had little, if any, effect on driver behavior. The statistical analysis repeatedly showed that the means and variances of speed, lateral placement, acceleration noise, and weaving were not significantly different when steady-burn lights were absent on the drums. Similarly, the proportions of traffic volume in different subsections of the highways in advance of the taper were not different during the absence of steady-burn lights, indicating that steady-burn lights had no effect on the lane-changing behavior of motorists at night. It appears that the high-intensity sheeting on drums and the flashing arrow panel in the beginning of taper had a powerful effect on the motorists, thus leaving the steady-burn lights without any practical value in the work zones. None of the drivers showed any erratic behavior when steady-burn lights were absent on drums. Some driver subjects noticed the absence of steady-burn lights during the tests and even their responses indicated that steady-burn lights had little effect on driving behavior or safety in the work zone.

In conclusion, the study has shown that steady-burn lights are not required in curved, lighted, unlighted, and tapered sections of highway work zones, including ramps and crossovers, when drums with high-intensity sheeting and flashing arrow panel are used as channelizing devices. It is recommended that the use of steady-burn lights in these highway facilities be discontinued in the future.

ACKNOWLEDGMENT

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Effects of Deicing Salt on Overstory Vegetation in the Lake Tahoe Basin

ROBIN A. EPPARD, JACK W. NORBERG, RICHARD J. NELSON, AND JOHN ALLISON

The impact of deicing chemicals used in winter maintenance is now being investigated throughout the country. The most common deicer, sodium chloride, has been identified as the cause of vegetation damage along roadway corridors. This study investigated the impact of sodium chloride on overstory vegetation in the Lake Tahoe Basin, an environmentally sensitive area. The percent of salt-affected vegetation within the study plot is described as a function of slope and soil group. In addition, the type and degree of vegetation damage caused by all types of factors is described. Forty-five percent of the vegetation analyzed from 9 control and 226 experimental plots showed one or more damage symptoms from biotic and abiotic sources. Fifteen percent of the vegetation analyzed showed damage from salt, and was second only to damage from insects (17 percent). Mitigation recommendations specific for the Lake Tahoe Basin are discussed, such as erosion control, revegetation with salt-tolerant species, and innovative deicing practices.

The use of deicing salt (sodium chloride) to melt snow and ice on highways in the Lake Tahoe Basin has been common practice since the 1940s [personal communication, Nevada Department of Transportation (NDOT)]. Sodium chloride provides the most economical means of controlling snow and ice buildup in practice today. Historically, little attention was paid to the timing of salt application with respect to storm events, or to the rate at which it was applied to the roadway. The consequences of these deicing practices led to the wastage of deicing and abrasive material in addition to possible salt accumulation in roadside soils, vegetation, and watersheds.

Recent investigations have addressed the environmental impacts associated with the use of deicing salt. Deicing salt deposited outside the roadway corridor through mechanisms such as surface runoff (*1,2*), mechanical snow removal, or spray from vehicle passage has been shown to cause damage to vegetation or changes in soil structure (*1,3–7*). The distance from the edge of pavement (EOP) to the furthest salt-affected tree is defined as the zone of influence (ZI) of salt damage within the roadway environment. The degree of salt damage decreases significantly as the distance from EOP increases (*1–4,6,8–10*) and the ZI extends further downslope than upslope (*1,11*). Previous studies have defined the ZI as extending from the EOP to 20–50 ft (*1*), 40 ft (*12*), 50 ft (*11*), 148 ft (*3*) and 300–400 ft on a very steep downslope (*1*).

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Once deicing salt has entered the roadway environment, its effect on vegetation is dependent to a large degree on the type of soil encountered. The rate and depth of salt infiltration are functions of soil compactness, physicochemical processes, hydraulic gradients, moisture, and overlying topsoil (*7*). Soils in the Lake Tahoe Basin consist of porous alluvial and glacial deposits that are not predisposed to accumulations of sodium or chloride ions. Spring and summer runoff is typically sufficient to leach salt below topsoil layers (*1,6,7,12,13*), resulting in reduced amounts of deicing salt for uptake by plants during the summer months.

SCOPE OF STUDY

Described in this study is the impact of deicing salt (sodium chloride) on overstory vegetation in roadway corridors of the Lake Tahoe Basin. Specific parameters that are described include (a) Zone of influence of deicing salt, and (b) Type and degree of tree damage caused by all types of injury. In addition, sodium and chloride levels in selected soil and foliar samples were determined. Revegetation recommendations associated with erosion concerns caused by roadway deicing in addition to alternative deicing practices are presented as possible mitigation measures for current deicing concerns.

METHODS AND MATERIALS

Project Area Description

The Lake Tahoe Basin lies on the east side of the Sierra Nevada physiographic province, between elevations of 6,200 and 10,000 ft. Lake Tahoe constitutes 38 percent of the 500 square miles occupied by the basin (*14*) (Figure 1). Regional uplift, faulting, and erosion of the predominantly granitic rock have produced a rugged topography characterized by steep slopes and narrow canyons. Lakeshore areas not dominated by bedrock consist primarily of glacial moraine and outwash terrain (*14*). Annual precipitation in the Lake Tahoe Basin ranges from more than 80 in. at higher elevations to 25 in. at lower elevations, and more than 80 percent of the annual precipitation occurs between October and April. Because of the freezing temperatures, much of the precipitation occurs as snow.

The soil profile from nonglacial slopes is immature, deriving from porous alluvial and glacial deposits (*12,15*) described as well to excessively drained with low water holding capacities

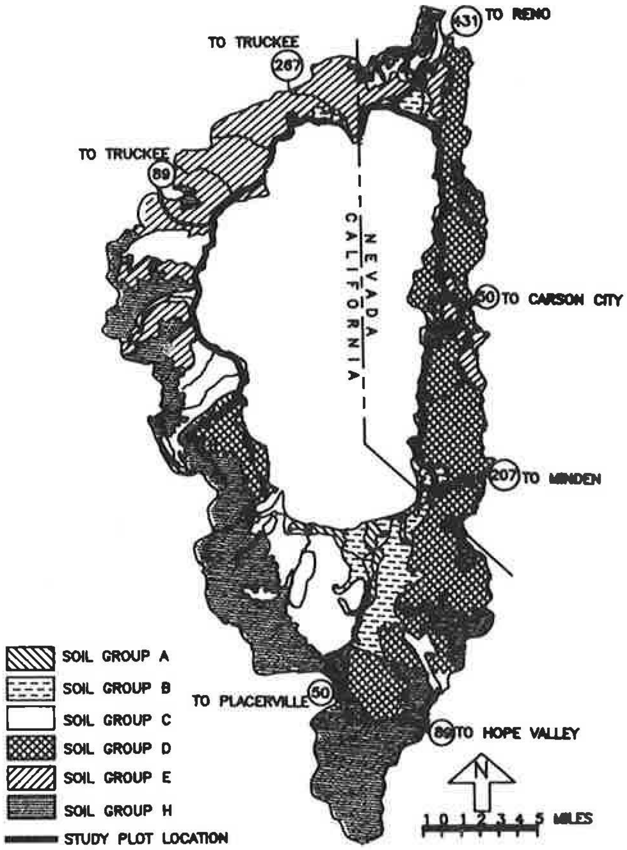

FIGURE 1 Lake Tahoe Basin, soil group distribution and study area.

(*16*). More than 60 different soil types found within the project area were categorized into six soil groups (*16*) (Table 1, Figure 1). Vegetation associated with these soils includes desert, montane, and alpine species such as pine and fir forests with understories of sagebrush and manzanita (*12*).

Plot Description

Nine control and 226 experimental plots were established along highways 431, 28, 50, 207 and 89 at 0.5 mile increments (Figure 1). Excluded from the study area were roadways lying within the city of South Lake Tahoe (not forested because of development) and the Camp Richardson Area (no-salt policy enforced 1975–1986). The standard forestry plot area of ⅕ acre was modified from circular to square (93.34 ft × 93.34 ft) to uniformly represent vegetation and soils perpendicular to the roadway corridor. Plots were offset from the edge of pavement to exclude cut-and-fill slope morphology (Figure 2). Two-thirds of the plots were located downslope and ⅓ upslope with respect to the roadway. Tree inventory for each plot included total number, species composition, distance to EOP, and slope.

Vegetation Assessment

Field Analysis

All trees from each study plot that were larger than 4-in. diam at breast height were assessed for damage caused by the following factors: undamaged, salt damaged, drought, mechanical, ozone, plants or fungi, insects, blight, or dead or unknown symptoms as described in Table 2. These biotic and abiotic infestations may directly cause tree mortality (*17*) or reduce plant vigor (*18*).

Criteria used for damage assessment were based on standard forestry methods and conducted under the direct supervision of a registered professional forester. Salt damage assessment was based on those developed by Scharpf and Srago (*11*). The visual symptoms for salt, insect infestation, fungus attack, or drought are similar and are often collectively interpreted as salt damage. The identification of damage types, therefore, was aided with the use of binoculars and hand lenses.

TABLE 1 Soil Groups in the Lake Tahoe Basin Study Area

Group A	Formed in recent alluvium deposited along stream channels and meadow bottoms; variable textured loams to gravelly sands; all subject to flooding and have a high water table.
Group B	Dominated by soils over 40 inches deep on alluvial fans and outwash; occur on nearly level to strongly sloping alluvial fans and terraces; deep, well drained loamy sands and sandy loams, some stony and cobbly.
Group C	Dominated by gravelly and stony soils over 40 inches deep to pans formed in glacial moraines and outwash; occur on nearly level to steep moraines or outwash formed in stony parent materials; gravelly and stony loamy sands or sandy loams over cemented substratum found below 40 inches deep.
Group D	Dominated by rocky soils less than 40 inches deep to weathered granitic bedrock; occur on gently rolling to very steep granitic uplands; shallow to moderately deep over weathered granite; loamy coarse sands and sands; excessive drainage.
Group E	Dominated by stony and cobbly soils over 10 inches deep, formed in volcanic bedrock; occur on gently sloping to steep volcanic uplands; well drained stony and cobbly loams; moderately deep to deep over bedrock.
Group H	Dominated by rock and stony land; occur on undulating to steep uplands; barren rock outcrop, talus slopes, volcanic rubble and stony colluvium.

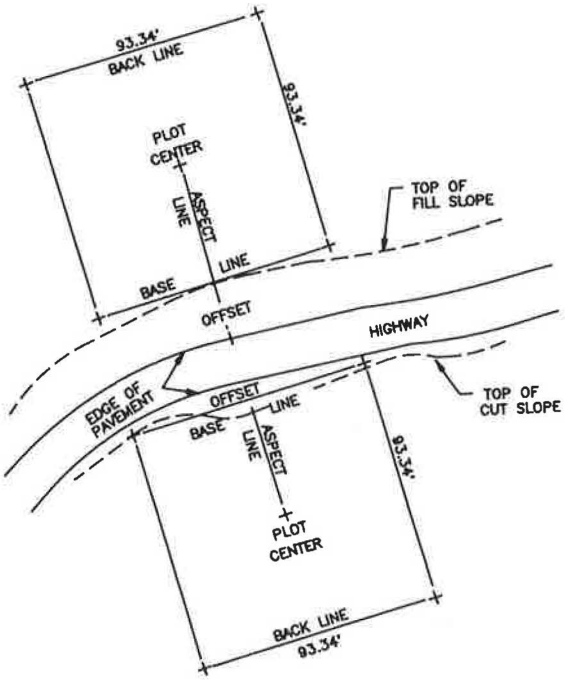

FIGURE 2 Plot layout with respect to cut and fill slopes.

Laboratory Analysis

Soils. Soil samples at four locations from 8 control and 20 experimental plots were collected at depths of 6–12 in., 18–24 in. and 30–36 in. Samples were sieved through a 2-mm screen and air dried before laboratory analysis. The pH of soil samples was determined colorimetrically using Bromcresol Purple or Bromthymol Blue indicator. Electrical conductivity was determined from saturation extract on a GLA Agricultural Electronics M33 Salinity Drop Tester. One soil sample from each plot was analyzed for the soluble salts sulfate, chloride, boron, calcium, magnesium, and sodium (Fruit Growers Laboratory, Santa Paula, California). Duplicate soil samples from each plot were analyzed for chloride using silver nitrate titration (Fruit Growers Laboratory, Santa Paula, California), and ion chromatography (Chemax Laboratories, Reno, Nevada).

Foliar Tissue. Foliar samples were collected from vegetation within the ZI that showed obvious signs of salt damage and from vegetation well outside the influence of roadway salt and analyzed for leaf sodium and chloride (percent oven-dry weight). Needles and leaves were separated from other plant structures and wiped with distilled water to remove dust. Pine needles were cut into from 1- to 1½-in. lengths. Leaves and needles were placed in 50 mL open glass bottles and dried at 45°C for 24 hr. The tissue was then ground in a micro-wiley mill to pass a 20 mesh screen. After mixing, a subsample was dry-ashed at 450°C for 4 hr, then diluted to 50 mL in distilled-deionized water and allowed to stand for 15 min. The samples were filtered through a 0.45 micron nylon filter and the filtrate was transferred to plastic bottles for sodium and chloride analysis using ion chromatography (EPA Test Method 300.0) and atomic absorption (Chemax Laboratories, Reno, Nevada).

TABLE 2 Vegetation Damage Symptom Descriptions

Salt	Sharp line delineates tan or reddish tip of needle from proximal green section, occasional dark reddish bands within the tan portion.
Drought	Needles with green merging to yellow then tan as needle tip dies.
Needle Miners	Most needles show dieback from tips, which are tan/yellow; relatively sharp green delineation; exit hole for insect in distal tan portion.
Saw flies	Distal one-half of needle is eaten away.
Bark Beetles	Pitch tubes, small masses of pitch, and boring dust present on needles. Pitch tubes from these insects cause narrow streams of pitch to flow down tree bark from tip of tree.
Needle Blight	Needles tan to red colored; difficult to distinguish from salt damage from a distance.
Dwarf Mistletoe	This parasite can be observed growing throughout the tree canopy and in conjunction with drought stress causes tree mortality.
Frost	Distal needle dieback on branches near ground.
Ozone	Lemon yellow irregular spots with fading edges on underside of Jeffrey and ponderosa pine needles.
Red Ring Fungus	Red rings encircle needles
Needle Scale	With heavy infestation the needles become covered with the white insect. The needles turn yellow then brown as juices are sucked from the needles.
Others	Root rots, rust, aphids, wood borers, cankers, galls, frost crack and lightning strikes.

Statistics

One-way analysis of variance procedures was used to test for significant differences between soil group or percent slope class and the ZI or percent salt-affected vegetation within the study plots. Students' *t*-test was used to test for significance differences between ZI distances between uphill and downhill slopes, and to test for differences between control and salt-affected soil and vegetation samples.

RESULTS

Vegetation Damage

The frequency of occurrence of each damage category for overstory vegetation within the study area demonstrated that 54.66 percent of the trees were unaffected by any of the identified damaged sources. Insect damage from bark beetles, borers, aphids, needle weevils, scale, sawflies, and needle miners affected 17.46 percent of the trees. Salt damage was identified for 15.05 percent of the trees, plants and fungi affected 6.42 percent, blight 3.89 percent, drought 3.45 percent, mechanical injury 1.68 percent and ozone 1.68 percent of the trees. Dead trees made up 7.63 percent, and 2.83 percent of the trees had damage from unknown sources (Figure 3).

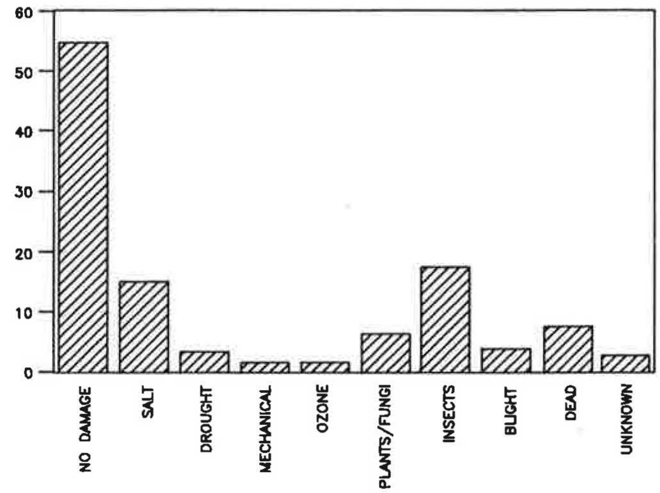

FIGURE 3 Relative frequency of damage type to vegetation. Each category treated independently.

Zone of Influence

The mean ZI for all uphill slopes was significantly less than that for all downhill slopes combined ($p = 0.95$). The mean ZI described for plots associated with soil group *C* was significantly less than for those associated with soil group *E* ($p = 0.90$) (Table 3). Within the ZI, trees associated with soil groups *B* and *C* were significantly less affected by salt than were trees associated with soil group *H* ($p = 0.95$) (Table 3). Soil group, therefore, affected both the ZI and the percent of salt-affected trees within the ZI.

Slope Effects

Percent slope was grouped into 6 ranges for comparison: uphill +(26–53), +(1–25); level (0); downhill −(1–25), −(26–50) and −(51–82). The mean ZI varied significantly only between flat and downhill very steep slopes ($p = 0.95$) (Table 4). Trees associated with downhill, very steep slopes were significantly more affected by salt than for any other slope class ($p = 0.90$) (Table 4).

Species Specificity

The percent occurrence of each type of vegetation damage was calculated for the different overstory species from control and experimental plots (Table 5). Control plots were located

TABLE 3 Zone of Influence and Vegetation Damage as a Function of Soil Group

Soil Group	n[a]	Zone of Influence (feet) Mean ± SE[b]	Zone of Influence (feet) Grouping Mean[c]	Percent Salt-affected Vegetation Mean ± SE	Percent Salt-affected Vegetation Grouping Mean
A. Hydric	5	34.0 ± 19.8	ab	30.0 ± 17.0	ab
B. Deep well drained alluvium	54	25.1 ± 3.5	ab	12.7 ± 2.6	b
C. Deep gravelly stony glacials	37	21.5 ± 4.1	a	11.6 ± 2.6	b
D. Rocky granitic <40 inches deep	49	32.4 ± 3.3	ab	23.5 ± 3.3	ab
E. Stony cobbly volcanic >10 inches deep	31	39.4 ± 4.4	b	21.2 ± 3.6	ab
H. Rock outcrop, talus, stony colluvium and volcanic rubble	30	36.6 ± 5.4	ab	33.4 ± 6.0	a

[a]n = Sample size
[b]SE = Standard error
[c]Means followed by same letter not significantly different.

TABLE 4 Zone of Influence and Vegetation Damage as a Function of Slope Class

Soil Group		n[a]	Zone of Influence (feet) Mean ± SE[b]	Grouping Mean[c]	Percent Salt-affected Vegetation Mean ± SE	Grouping Mean
Uphill, steep	+(26-53)	20	27.9 ± 5.9	ab	24.5 ± 5.8	a
Uphill, gentle	+(1-25)	55	21.0 ± 3.1	ab	8.8 ± 1.7	a
Flat	(0)	8	16.9 ± 8.8	a	6.7 ± 3.2	a
Downhill, gentle	-(1-25)	75	34.2 ± 3.2	ab	22.3 ± 2.6	a
Downhill, steep	-(26-50)	32	30.3 ± 3.8	ab	21.9 ± 3.8	a
Downhill, very steep	-(51-82)	16	52.6 ± 6.9	b	50.5 ± 8.6	b

[a]n = Sample size
[b]SE = Standard error
[c]Means followed by same letter not significantly different.

TABLE 5 Frequency of Damage Factors for Overstory Species

Species	(n)[a]	Not Salt-Affected (%)	Salt-Affected (%)	Drought (%)	Mechanical (%)	Ozone (%)	Plants/ Fungi (%)	Insects (%)	Blight (%)	Dead/ Unknown (%)
Lodgepole pine	276	71.7	19.9	0	.73	.73	9.78	19.9	2.9	4.7
Pinus contorta	(30)	(100.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)	(16.7)	(0.0)	(0.0)
Sugar pine	52	63.5	19.2	1.9	0	0	0	0	13.5	5.8
Pinus lambertiana	(2.0)	(100.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)
Jeffrey pine	2,603	69.0	16.1	5.0	0.04	0.7	7.7	9.4	4.8	3.0
Pinus jeffreyi	(61)	(100.0)	(0.0)	(0.0)	(0.0)	(0.0)	(1.6)	(8.2)	(31.1)	(1.6)
Ponderosa pine	199	80.4	11.6	5.0	0	0	1.0	9.1	3.0	.5
Pinus ponderosa	(0.0)	(n.d.)	(n.d.)	(n.d.)	(n.d.)	(n.d.)	(n.d.)	(n.d.)	(n.d.)	(n.d.)
Red fir	81	87.7	4.9	0	2.5	0	0	50.6	1.2	22.2
Abies magnifica	(15)	(100.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)	(46.7)	(0.0)	(0.0)
White fir	2,046	75.6	5.9	1.4	1.8	0	2.1	24.5	1.0	4.4
Abies concolor	(148)	(100.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.7)	(77.0)	(10.8)	(5.4)
Incense cedar	190	64.2	10.0	2.6	0	0	3.2	2.1	3.2	5.8
Calocedrus decurrens	(16)	(100.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)	(0.0)	(3.3)	(3.3)

[a]Total number of trees counted - numbers in parenthesis are from control plots.
n.d. no data

in areas outside the ZI. No salt damage was identified for trees within control plots; however, damage caused by plants or fungi, insects, and blight was common. More than 77 percent of the White fir and 47 percent of the Red fir from control plots showed insect damage. Damage from drought, mechanical means, or ozone was not noted for any species in control plots. Within the experimental plots, Lodgepole pine showed the highest frequency of salt damage (19.9 percent), damage from ozone (0.7 percent) and plants or fungi (9.8 percent). Sugar pine also had a relatively high frequency of salt damage (19.2 percent), in addition to the highest frequency of blight (13.5 percent). Jeffrey and Ponderosa pine showed the highest frequency of damage caused by drought (5.0 percent each), and Jeffrey pine exhibited the highest frequency of damage caused by ozone (0.7 percent). Red fir and White fir demonstrated a low frequency of salt damage, but exhibited the two highest frequencies of insect damage (50.6 percent and 24.5 percent, respectively). Incense cedar showed a low frequency for most damage types, with 10 percent exhibiting salt damage symptoms.

Foliar Ion Levels

Sodium levels were significantly higher ($p = 0.95$) in salt-affected foliage when compared with controls for White fir and Jeffrey pine needles. Chloride levels in foliage of salt-affected trees were significantly higher than controls ($p = 0.95$) for White fir, Jeffrey pine, and Incense cedar (Table 6).

Electrical Conductivity (EC) and pH

Qualitative soil analysis for EC and pH indicated no significant variation from values typical for each soil group.

TABLE 6 Foliar Sodium and Chloride Levels

SPECIES	CONTROL			SALT-AFFECTED		
	Mean Sodium Level (% by weight)	Standard Error	Number of Samples	Mean Sodium Level (% by weight)	Standard Error	Number of Samples
White Fir[a]	.0058	.0006	9	.0971	.0256	8
Jeffrey Pine[a]	.0076	.0016	7	.1628	.0645	10
Lodgepole Pine	.0067	.0003	3	.0550	.0255	3
Sugar Pine	.0050	.0000	2	.2830	.2580	2
Incense Cedar	.0055	.0005	2	.0765	.0545	2
	Mean Chloride Level (% by weight)	Standard Error	Number of Samples	Mean Chloride Level (% by weight)	Standard Error	Number of Samples
White Fir[a]	.0601	.0078	9	.3981	.0810	8
Jeffrey Pine[a]	.0224	.0056	7	.4334	.1188	10
Lodgepole Pine	.0183	.0094	3	.0690	.0372	3
Sugar Pine	.0050	.0000	2	.5305	.3795	2
Incense Cedar[a]	.0260	.0156	2	.2755	.0134	2

[a]Statistically significant difference (95% confidence level) between the salt-affected sample and the control.

DISCUSSION

Damage Ratings

The types and extent of damage to overstory vegetation in the Lake Tahoe Basin roadway corridor have been described in this study. Approximately 45 percent of the trees were affected by one or more factors such as insect infestation, fungus, ozone, or salt. The two main types of vegetation damage were attributed to insects (17.46 percent) and salt (15.05 percent).

Vegetation Assessment

Zone of Influence

The mean ZI described for the project area averaged 30.2 ft from EOP, and varied as a function of soil group and slope class. The difference in the width of the ZI between plots on soil group *C* and soil group *E* may be caused by the difference in soil texture and organic content. Characteristics of soil group *C* include a finer texture and greater organic content, which may allow for increased soil water holding capacity as compared to soil group *E*. The ZI varied significantly only between flat and downhill, very steep slopes. This indicates that slope may play a relatively minor role in affecting the susceptibility of vegetation to salt damage caused by the high percolation rates characteristic of soils from the project area.

Percent Salt-Affected Vegetation Within the Study Plots

Vegetation within the study plots was found to be more susceptible to salt damage when located on particular soil groups or slope classes. Trees associated with deep gravelly stony glacial (*B*) or stony cobbly volcanic soils greater than 10 in. deep (*C*) showed a lesser degree of salt damage than trees associated with rock outcrop, talus, stony, colluvium, or volcanic rubble (*H*). Soil group *H* characteristics are the least conductive to plant growth and may render vegetation more susceptible to exogenous impacts.

Trees on steep downhill slopes exhibited the greatest degree of salt damage, which may be caused in part by an increase in aerial deposition of salt spray onto foliage.

Species Specificity

All of the overstory species within the project area showed some degree of salt intolerance. Lodgepole pine and Sugar pine showed the highest frequencies of salt damage; Red fir and White fir showed the lowest frequencies of salt damage. The degree of salt damage differed between species: Lodgepole pine damage was rated as light; damage to Sugar pine and Jeffrey pine was rated as severe (*11*). On the basis of this rating, Jeffrey pine and Sugar pine may be considered somewhat salt intolerant in the project area. Ponderosa pine showed a moderate frequency of damage caused by salt, and together with Jeffrey pine showed the greatest degree of damage caused by drought.

Red and White fir appeared to be the most salt tolerant, with only 4.9 percent and 5.9 percent, respectively, of the trees exhibiting damage symptoms. Both of these species, however, showed heavy infestations from insects, which may have interfered with an accurate diagnosis of all of the contributing damage factors. Red fir is not a typical roadway corridor species, and is often found in locations that are not affected by salt.

Incense cedar showed moderate levels of damage caused by salt, drought, blight, and dead or unknown causes, and low frequencies of damage from plants or fungi and insects. Incense cedar may be a rather tolerant species for most of the damage types found in the project area.

Individual overstory species clearly exhibit differing degrees of tolerance to the types of damage found within the project area. This tolerance is based not only on species specificity, but also on other, exogenous factors, such as historic precipitation, microclimate, and soils.

Soils

The Lake Tahoe Basin is characterized by well-drained sand, loamy sand, and sandy loam soils with very low exchange capacities, rapid to very rapid permeability rates and very low available water capacities. Because of these characteristics, the soils of the study area do not retain ions well. The electrical conductivity and pH data, collected during the months of June and July, indicate that much of the salt had leached beyond the root zone during the spring runoff, and agrees with reports by Kliejunas (*1*) OECD (*6*), Jones (*7*), and Hanes (*19*). Residual salt levels, although significantly higher than background, were still well below sodium or chloride levels that have been reported to be toxic to vegetation. Soil pH ranges, although significantly different between the different soil groups, are well within the typical pH range for each soil group. Cumulative salt effects, therefore, are likely to occur at a rate far slower than that for locations with clay-type soils or higher content or organic matter.

Mitigation

Erosion Control

Loss of vegetation along the roadway corridor has exacerbated preexisting erosion concerns in the Lake Tahoe Basin. Uncontrolled erosion leads to sediment production and deposition in drainage systems, steam environment zones, and ultimately Lake Tahoe. The rate and location of soil erosion and sediment production adjacent to roadways is influenced by precipitation intensity, form of precipitation, storm duration, runoff drainage patterns, slope steepness, slope length, ground cover canopy and type, soil material, slope aspect, and slope disturbance. A reduction of tree canopy and understory cover on slopes and subsequent loss of duff and litter will increase the potential of erosion during periods of surface runoff. Conversely, an increase in vegetative cover decreases the erosion potential. Clearly, loss of roadside vegetation increases the erosion potential in the Lake Tahoe Basin.

Site-specific revegetation programs to stabilize slopes and decrease the potential for erosion have been implemented by the NDOT and California Department of Transportation in the Lake Tahoe Basin. Salt-tolerant species are recommended to reduce vegetation mortality. Removal of dead trees is recommended to reduce fire hazard and curtail insect infestation and spread of disease. Damaged trees should not be removed as they provide useful cover, and may potentially recover.

Grasses and forbs along with specific shrub species adapted to site limitations are recommended for roadside revegetation in the project area. Annual replacement of foliage in grasses and forbs reduces the impact of salt accumulation in leaves, and its low profile avoids mechanical damage from snow removal operations. Native shrubs did not exhibit the same potential as grasses and forbs because they retain foliage from year to year.

All evergreen tree species in the Lake Tahoe Basin have varying degrees of salt intolerance, and if used in revegetation plans would be susceptible to salt damage. In addition, the establishment and maintenance of a clear zone for driving safety would be best accomplished if grasses and forbs were used in place of trees adjacent to the roadway. Therefore, revegetation of roadside areas should not include native tree species.

Recommendations

The use of deicing salt (sodium chloride) has resulted in vegetation damage in the roadway corridor of the project area. This damage is the result of past practices of salt application and recent drought conditions. Recommendations include upgrading equipment and use of the automated weather and pavement condition site information system, in addition to investigating alternative chemicals for use as roadway deicers or implementing more frequent and longer-duration chain control programs (personal communication, NDOT). Currently, the NDOT and California Department of Transportation are implementing all of the previously listed recommendations. Equipment upgrading also includes extensive training of personnel on the use and care of deicing equipment. In addition, both departments of transportation are investigating possible alternative chemicals for use in deicing roadways. Environmental concerns on water quality, impact to vegetation and wildlife, in addition to deicing effectiveness, operational criteria, roadway impacts, and cost are being considered for alternative roadway deicers.

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Benefit-Cost Assessment of the Utility of Road Weather Information Systems for Snow and Ice Control

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In 1988, the Strategic Highway Research Program initiated a project to look at the potential effectiveness of road weather information system technologies for improving and reducing costs of highway snow and ice control. These technologies include pavement and meteorological sensors, pavement and weather condition forecasts, roadway thermography, and the communications, both human and electronic, required for effective dissemination of the information. The investigation required the performance of a benefit-cost analysis of road technologies. The benefit-cost assessment is complex because it has to take into account variations in the distribution of weather events and road conditions, as well as snow and ice control practices, in different regions of the country. Described in this paper is the statistical model used to perform the benefit-cost assessment, to include the one-, two-, and three-dimensional matrices that form the basis for computing costs. Benefits are the reductions in snow and ice control costs resulting from the use of the weather information technologies. Finally, presented in the paper are the results obtained from running the model. Model results show that the use of weather technologies can be cost-effective when decisions become proactive with the use of weather information. The model shows that detailed forecasts of road conditions provide the greatest benefit-cost ratio; however, the combination of forecasts, sensors, and road thermography synergistically provides improved level of service for snow and ice control, and a significant reduction in decision errors, as well as a benefit-cost ratio greater than one.

Research under the Strategic Highway Research Program (SHRP) Contract H-207, Storm Monitoring/Communications, has shown that weather information can improve highway maintenance managers' ability to assign their labor, equipment, and materials for snow and ice control. By becoming proactive with information, rather than reactive to conditions, more timely and efficient decisions can be made. Described in this paper are a methodology and computer model developed to quantify the benefits of using road weather information system (RWIS) data compared with the costs of reacting to present conditions.

Because of weather patterns, many European countries experience a high frequency of icing conditions on their roadways. This, combined with relatively high population and traffic densities, drives a need to provide improved snow and ice control response. Today, in some European countries, RWIS are implemented countrywide. Forecasts of road temperatures and conditions, as well as weather conditions, are commonplace. Meteorological and pavement sensors monitor conditions in the road environment. Road thermography, a technology which was developed in the United Kingdom, employs instrumented vehicles and downward-pointing infrared radiometers to create temperature profiles of road surfaces. Climate and weather consultants work closely with highway maintenance agencies to form a common understanding of abilities and needs related to road weather information.

In order to foster an exchange of information, a European forum evolved, devoted to the subject of road weather: the Standing European Road Weather Commission (SERWEC), which meets every 2 years, and every 4 years in conjunction with the Permanent International Association of Road Congresses (PIARC). In 1990, SERWEC became SIRWEC, the I standing for International, with the addition of members from the United States and Asia. In addition, the European Community sponsors research on the road weather information systems through the Cooperation in Science and Technology project. However, much of this European development is a result of governmental support. In Europe, a completely packaged RWIS can frequently be found that is backed by government-subsidized consultants and hardware manufacturers, all of whom work closely with their national meterological service.

In the United States, however, the implementation of RWIS technology has not taken hold as rapidly. Each state highway agency has its own budgets and policies, but not all have research programs; and each state highway agency tends to rely on its own resources to identify or implement innovations. Autonomy also exists between state and federal agencies that need weather information. In fact, it is a credit to the industriousness and marketing talents of one United States vendor of RWIS technology that RWIS hardware is in place in some of this nation's roadways. These systems, originally designed for airports, where snow and ice control are also significant winter problems, are now being put to roadway use.

In 1988, SHRP initiated a project to investigate the use of TWIS technologies to provide guidance to the states on how to implement the technologies should they be proven effective. The investigation involved defining the state of the art in RWIS sensors; identifying the communications required for disseminating the information, assessing the ability of the meterological community to provide support for decision makers, and determining whether the technologies are cost-effective. SHRP contracted with The Matrix Management Group, who, with its subcontractors, the Washington State Transportation Center (TRAC) at the University of Wash-

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ington, and the University of Birmingham in the United Kingdom, have conducted the investigation.

Questionnaire surveys were sent to all states and provinces of Canada; in-person interviews were conducted in 10 states and one province of Canada. The information was required in order to determine whether the RWIS technology is feasible and cost-effective. Early indications from some state highway agency tests indicated that savings of up to 10 percent of snow and ice control costs might be possible through the use of RWIS technologies. The survey results and Federal Highway Administration statistics show that the cost of snow and ice control in the United States and Canada exceeds $2 billion per year (*1*). Even a 1 percent reduction in this figure would generate at least a $20 million saving.

Field testing for the SHRP investigation involved seven states: Colorado, Massachusetts, Michigan, Minnesota, Missouri, New Jersey, and Washington. These states were selected because they are located in different climate areas, they have different snow and ice control practices, and each had elected to test some forms of RWIS technology. Additional assistance and data were obtained from the following three states.

- The Minnesota Department of Transportation (DOT) had installed one brand of sensors in the Minneapolis area; installed a second brand at its road research facility near Monticello, which could be used in analyzing variations in pavement temperatures across lanes of traffic; installed a third brand in the Duluth area; had also contracted for road thermographic and climatologic analysis in Duluth; had contracted for weather forecasting services to support snow and ice control managers; and had hired a meteorologist as a staff weather advisor.
- The Colorado Department of Highways had installed a large number of sensors in the Denver area that could be used for analysis of the spatial variability of temperatures and requirements for numbers of sensor sites; and had contracted for weather forecasting services.
- The Washington Department of Transportation had contracted for road thermographic analysis and installed sensors in the Seattle area; had contracted for weather forecasting services for a number of areas in the state; and participated in a unique, multi-agency RWIS sensor system installation in the Spokane area.

Gathering information from these three states provided the ability to investigate the benefit-cost relationships and feasibility of nearly all of the forms of RWIS technologies.

BENEFIT-COST MODEL

There are many possible considerations for inclusion in a benefit-cost analysis for snow and ice control. There are indirect and direct benefits and, for the most part, only direct costs. Indirect benefits can be categorized as societal; snow and ice control practices can improve traffic flow, reduce fuel consumption, reduce accident rates, decrease insurance premiums, and so forth. Direct benefits include reduced expenditures for labor, equipment, and materials. The indirect costs are difficult to estimate and are controversial. Direct costs can be gleaned from records of expenditures for snow and ice control. To ensure a feasible level of effort and to maximize objectivity of the results, the decision was made to focus on direct costs. Benefits, then, would be reductions in direct costs.

Although one possible result of improved snow and ice control decision making is reduced costs, the other possibility is an improvement (or reduction) in the level of service provided to the traveling public. For the purpose of this study, level of service has been defined as what the highway agency does for snow and ice control. The research team decided that, at a minimum, any potential savings should not be at the expense of the traveling public, (i.e., a reduction in the level of service). Therefore, the methodology should also track the level of service provided in order to determine if it were degraded or not.

A computer model was developed that computes the costs associated with the allocation of snow and ice control resources and monitors the level of service provided by each allocation decision. Because the methodology deals with different snow and ice control strategies and different weather regimes, a statistical model was developed that uses the frequencies of occurrence of weather events and road conditions as a starting point. Cyrus G. Ulberg at TRAC, with considerable computer and benefit-cost experience and expertise, developed the model. The model is written in FORTRAN and runs on an IBM-compatible, 80286 or 80386 personal computer with a graphics card, 640 bytes of memory, and a mathematics coprocessor. Because the model was developed expressly for the analysis required in this project, it is not documented for general release.

Model Inputs

The model accounts for different snow and ice control practices. For example, in much of the country, especially the Northeast and Midwest, highway agencies use a great deal of chemicals, [e.g., sodium chloride (salt)], to remove snow and ice. Progressing farther west, proportionally less chemicals are used, although more abrasives are applied. The amount of salt used by state, based on survey results from this project, is shown in Figure 1 and the amount of abrasives used are shown in Figure 2. States where "No Data" is shown did not respond to the survey. Each of the practices has its own effectiveness, cost, and associated weather-related thresholds for decision makers. The model needed to be flexible enough to include those variations.

Additionally, the climate varies greatly from east to west and from north to south. The benefit-cost analysis includes evaluations of practices in different climates. Standard climatological data available from local National Weather Service forecast offices were selected as the best descriptors of climate for each area analyzed. These climatological summaries provide the frequency of occurrence of weather phenomena on a monthly basis. The weather phenomena used in this analysis include the frequency of occurrence of snow or ice, rain, and fog (for frost formation). It was assumed that the winter season runs from October through March; the frequencies were summed over that period to get a seasonal frequency of occurrence.

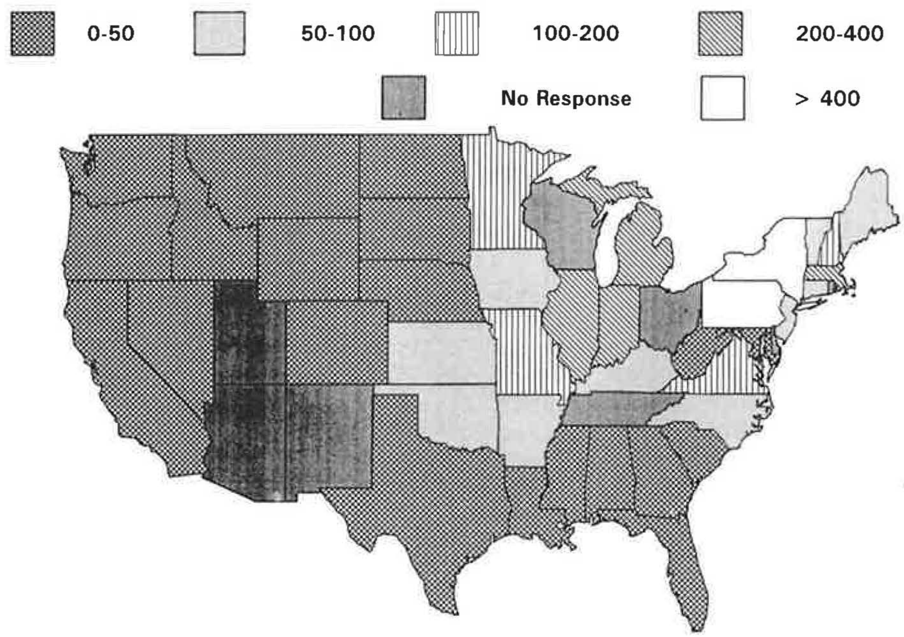

FIGURE 1 Annual salt usage by state for snow and ice control (in thousands of tons).

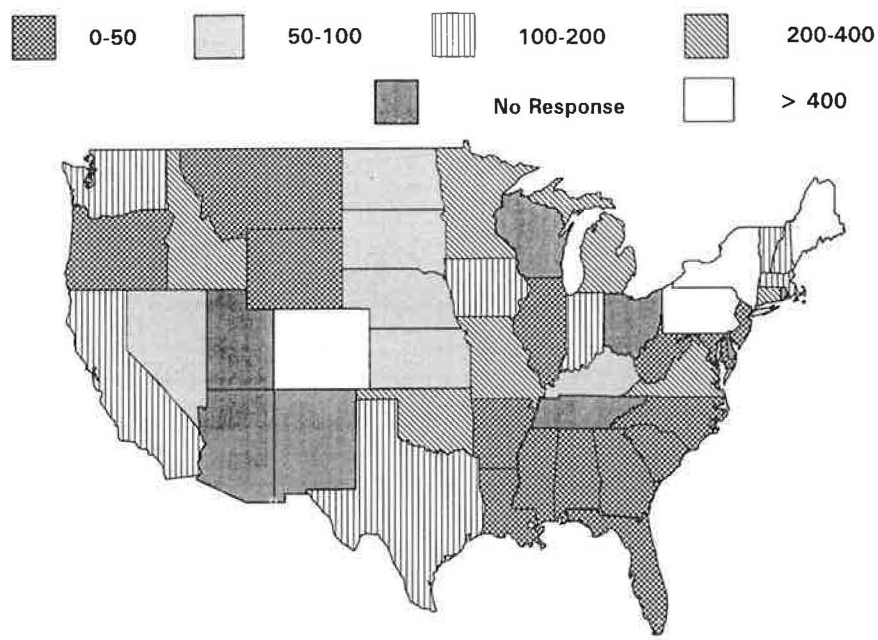

FIGURE 2 Annual sand usage by state for snow and ice control (in thousands of tons).

However, highway maintenance decision makers react to the condition of the road as well as to the weather. Also, the climatologies described above refer to specific points, namely the airports from which the meteorological measurements are taken. The road conditions can vary considerably in a small region because of many influences, such as the presence of water, topography, orientation of a road toward the sun, cuts, and other exposure considerations. For example, if it is snowing at the airport, there may be a distribution of road conditions varying from dry, to snow-covered, to wet. As a result, the model was designed to provide a distribution of road conditions over a road network, based on the climatological input.

The purpose of the model is to generate costs of snow and ice control actions taken based on the weather and pavement condition information available to the maintenance manager. The assumption used in constructing the model is that the maintenance manager will make the correct decision based on the information at hand. That information will vary as a function of the type of information used. The flow of information that generates the output is shown in Figure 3.

The first piece of information generated by the model is a weather event. The model uses a random number generator applied to the climatology distribution. Over a large number of iterations, the model will also generate the climatology of events because the random number generator will generate an even distribution of numbers from 0.0 to 1.0. How a frequency distribution of weather events is generated by using a computer's random number generator is shown in Figure 4.

The weather event is used to produce the road conditions to which the maintenance managers react. In the example of "No Significant Weather" already described, the road segments will have been given "ice," "snow," "clear," or "wet" conditions, based on the climatology or road conditions. Once a road condition is specified, another routine produces the weather information the maintenance manager uses to decide

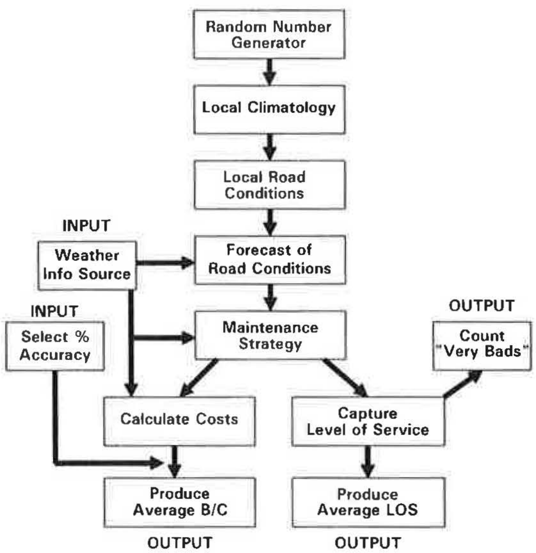

FIGURE 3 Flow of information in the benefit-cost model.

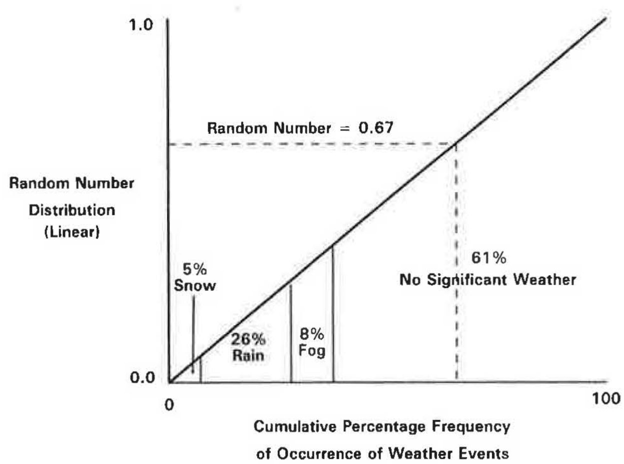

FIGURE 4 Example of generating a weather event.

to allocate snow and ice control resources. Based on the selection of the source of RWIS information, an action is selected. Each action has an associated cost and a level of service.

Once the decision is made on what resources to mobilize, the model captures the cost of the resource allocation and the level of service attached to the action. The level of service is rated on a subjective scale of 1 to 5, 1 being best and 5 being worst. If roads are dry, then any maintenance action selected is a "1," but has a cost associated with it. On the other hand, if the road is icy and the forecast was for "wet" and no action was taken, then the level of service would be "5."

At this point, the model has generated a cost and level of service for a specific maintenance action. This cost and level of service is then compared with the actions selected based on different weather information, in order to determine the benefit-cost ratio for the analysis. As more and better information is made available to the maintenance manager, the better is the decision that can be made. A typical scenario would be for the maintenance manager to get weather information from the media. The decisions made with that information are then compared with decisions that were made based on detailed forecasts and interaction with a forecaster for the weather and road conditions at specific locations. The costs of each action are compared, and a benefit-to-cost ratio is computed based on the costs of the maintenance actions and the costs of the information used to make the decisions.

In order to calculate the benefit-cost ratio (B/C), the model must be run for a number of iterations in order for a B/C to converge. At every hundredth iteration the model computes how many iterations are required to have either 2 or 5 percent accuracy in B/C at the 95 percent confidence levels. In some cases, the number of iterations required for a 2 percent accuracy was extraordinarily high; therefore the B/C is computed at least to the 5 percent accuracy level with 95 percent confidence. This appears very adequate because the computed B/C is usually much greater than 1. If the B/C were close to 1, increased accuracy might be desired.

Model Matrices

All of the information used in the model is contained in matrices of information. Each element of a matrix is a number (value) that is assigned to a piece of information (name). This section explains the matrices used in the model.

One-Dimensional Matrices

The one-dimensional matrices are lists of names that have values assigned to them. They are one-dimensional because there is one list of names, each with a value assigned. These matrices include the climatology of weather events, the costs of snow and ice control resources, and the costs of weather information technologies.

The climatology of weather events is nothing more than the list of possible winter weather events that can be extracted from standard climatological summaries for a location, and their frequencies of occurrence. A climatology input matrix is shown in Table 1.

TABLE 1 Sample Climatology for Input to Benefit/Cost Model

Weather Event (Name)	Frequency of Occurrence (Value)
No significant Weather	0.61
Rain	0.26
Snow	0.05
Fog	0.08

The second one-dimensional matrix contains the snow and ice control resources and their associated costs. Resources include people such as equipment operators; vehicles such as pickup trucks, trucks with spreaders, and trucks with plows; and materials such as abrasives and deicing chemicals.

The third one-dimensional matrix is the list of weather information technologies to be used in the decision process and their costs. These could include information obtained from media sources, in-place meterological and pavement surface sensors, detailed forecasts from private meteorological services, road thermography, and combinations of these technologies. Each technology has a cost assigned based on the prices being paid by the highway maintenance agencies. The cost is reduced to a daily rate. Technologies having a one-time cost use a daily rate calculated by amortizing costs over 5 years and 180 days of winter per year.

Two-Dimensional Matrix

The benefit-cost model employs one two-dimensional matrix. This matrix assigns the snow and ice control resources to a maintenance strategy. It is this matrix that generates the costs. The costs of resources assigned are based on the assumption that the snow and ice control activity (strategy) will require 8 hr, and that the resources must deal with 100 mi of four-lane highways during that 8-hr period. Examples of resources for one strategy could be three people driving three trucks with spreaders applying chemicals at the rate of 300 lb/lane-mile for an ice event, one person driving a pickup truck for winter patrolling, or combinations of these.

Three-Dimensional Matrices

Four three-dimensional matrices are used in the model. The first is used to distribute road conditions over the road network. Because each model calculation is initialized by climatology, the first number produced is a weather condition. However, actions are taken based on road conditions. This matrix uses local knowledge to estimate a frequency of occurrence of road conditions because there is no "climatology of road conditions." For example, in an area, if snow is occurring at the airport reporting station, there is likely to be snow on some roads, whereas others may be wet or even dry. The values in the matrix are the frequency of occurrence of a road condition given that a weather condition is specified (two dimensions). The third dimension of the matrix is the road segment. Being able to specify the road segment allows for making some roads more troublesome for snow and ice control (i.e., more snow, more ice, or more frost).

The second three-dimensional matrix generates the weather information that is used in the resource-allocation decision process. This matrix assigns the probability to the forecast (two dimensions). For instance, if the road condition generated from the matrix above is "snow," then the likelihood that snow was forecast to accumulate on the road might be 0.75 from a meteorological service. The third dimension of the matrix is the weather information source. The maintenance manager could also have based a resource-allocation decision on information heard over media, National Weather Service forecasts, pavement temperature sensors, and so on. Because the benefit-cost methodology assumes that the resource allocations for snow and ice control are based on information received, this matrix is used with the next one to generate the strategies for which costs are calculated.

The third three-dimensional matrix generates the snow and ice control action. If a road condition is forecast, a resource-allocation response occurs (two dimensions). For example, if the expected road condition is "snow," then snowplows are called for; if only wet roads are expected, no action would be the appropriate response. The values in the matrix are either 0 or 1. A 1 selects the strategy for the specified road condition forecast; all other elements of that row are 0. Only one strategy can be selected for a given road condition. The third dimension of this matrix is again the source of the weather information. It is assumed that changes in assigning resources will result based on the information source: normal procedure in one maintenance area might have been to use nightly winter road patrols; with detailed forecasts of road conditions available, the routine patrolling can be discontinued. Simple three-dimensional (2 × 3 × 2) matrices for this scenario are shown in Figures 5 and 6.

The last three-dimensional matrix assigns a "level-of-service" value to the response strategy for a given road condition. The values range from 1 (very good) to 5 (very bad), and are assigned based on a subjective hierarchy that was established

FORECAST CONDITIONS	MAINTENANCE STRATEGY:	No Response	Patrol	Truck + Plow
	No significant Weather	0	1	0
	Snow	0	0	1

FIGURE 5 First-level weather information source: media.

MAINTENANCE STRATEGY

FORECAST CONDITIONS		No Response	Patrol	Truck + Plow
	No significant Weather	1	0	0
	Snow	0	0	1

FIGURE 6 Second-level weather information source: detailed forecasts from a private service.

for this model. For example, "very good" to "good" would be assigned to any strategy selected when the roads are dry; "very bad" would be assigned to doing nothing when snow or ice is expected to accumulate on the roads. In between might be a 2 or 3 for implementing chemical applications for ice in a timely fashion, to a 4 for doing the same thing as a reactive measure. This matrix allows the model to keep track of the average level of service in order to make the comparisons between weather information technologies. The model also counts the number of 5 occurrences. The level-of-service matrix is constructed using 5 to reflect Type I errors, the errors that occur when someone should have taken action, but did not. The purpose of monitoring the Type I errors is to determine the effectiveness of the RWIS information source in reducing Type I errors.

During early model runs, it was discovered that the costs of snow and ice control responses were not being valued properly. If taking no maintenance action was selected because of bad information and the roads were icy or snow-covered, the level of service would reflect a "very bad," but there would be no cost associated with doing nothing. In order to correct this discrepancy, new response strategies were devised that simulate what really happens. If resources are not mobilized in a timely way for snow situations, it usually takes longer to remove the snow. Similarly, if chemicals are not applied quickly, ice or snow situations can take longer to mitigate. Each of these situations provides a reduced level of service and costs the highway agency more. Strategies were devised and resources assigned to capture the additional costs of incorrect decisions. The model in its present form also has the capability to include additional societal costs. No attempt is made to include those costs because they are too difficult to quantify.

Model Results

Simple scenarios were created to gain an understanding of the model. These included developing a one-segment road network 100 mi long; creating simple two-element matrices of climatologies, weather information strategies, resources, and actions; and building 2 × 2 matrices for all of the other inputs.

An example scenario contained a climatology of snow-no snow, weather information from a forecasting service versus media-only information; and response strategies that included a winter patrol in use with media information versus no patrol and snowplow response with forecast service, as already described. The values assigned to the names in the matrices were varied in order to conduct an analysis of the sensitivity of the model to various inputs. This also included an initial look at using perfect forecasts for setting a limit to the benefit-cost ratio.

These initial results showed that the B/C varied inversely with the frequency of occurrence of "bad" weather. A possible explanation for this is that there is little chance to make a wrong decision if it snows every day. However, if snow is an infrequent occurrence, then erroneous decisions can be very costly.

Additionally, the early results pointed out that any increase in forecasting ability over chance (50-50 guess) produces a high B/C (> 10.0). This was because of the baseline scenario in which road patrols are used without forecasting support and are not used with the forecasting support. Reducing the overhead of road patrols provides a large savings. Also, the cost of a weather forecasting service is very small when compared with the costs associated with snow and ice control activities.

Following the initial familiarity runs of the model, scenarios were developed for each of the three states previously described. Matrices were built that reflected each state's snow and ice control practices, potential weather information sources, climatologies, and characteristic road condition distributions. The following sections describe the general results of the model as applied to scenarios that more closely represent reality.

Forecasting Support

In all cases, model runs show a high B/C (> 20.0) when decisions are made proactively using forecast support when compared with no forecast support. Even a slight improvement in decision making based on typical costs of private meteorological services as compared with media information shows a significant cost savings and high B/C. A cost of $25/day for forecasting services for a small area was used. This was an average of known contracting costs with an added daily communications cost. These costs are somewhat deflated currently because of competition and frequent low-bid contracting by highway maintenance agencies. The consequence of the lower cost, however, may be reduced meteorological support in terms of the quantity and quality of the information provided to the decision maker. The model shows, however, that if the costs of private meteorological services are increased as much as tenfold, the B/C is still greater than 1.0. In addition to the high B/C, the level of service (reduced bad decisions) improved markedly.

RWIS Hardware

The data available to maintenance managers from RWIS sensors typically include pavement temperature, air temperature, relative humidity, wind speed and direction, pavement condition (wet, dry, icy) and an indication of the amount of deicing chemicals on the surface. These data can be used to monitor the potential for ice or snow bonding and frost formation. Sensor data tend to provide information to which a maintenance manager can only react. However, with the wealth of experience most maintenance managers have—which can be supplemented by sensor information—the manager can frequently make timely resource allocation decisions. These can include not implementing snow and ice control because sensors indicate that the pavement is too warm for snow or ice bonding, or that sufficient chemicals remain on the road surface to warrant no response, or that plowing only, without chemicals, may be appropriate.

RWIS sensor systems can cost as much as \$40,000 per location. For instance, five sensors systems located over the 100-mi network of roads used in the model, plus a central computer and workstations needed to process data, can easily put the cost of a sensor system at more than \$250,000. Amortized over 5 years and 180-day winters, the daily cost can exceed \$300. In each of the three states used in this investigation, the number of sensors in an area varies. Actual daily costs varied by location: Washington, \$222; Colorado, \$500; and Minnesota, \$350, based on a recent procurement action for additional systems. In the scenarios developed, and with the subjective input of the marginal improvement possible in road condition forecasting with sensors compared to media information, the B/C calculated were small, and ranged from −1.5, where the increase in cost of the RWIS technologies exceeds the decrease in direct maintenance costs, to very close to 1.0. This shows that sensors alone are not the answer to saving costs of snow and ice control practices.

Road Thermography

Very little road thermography has been conducted outside Europe. Only Wisconsin, Washington, and Minnesota had thermal profiles made of some of their highways. There is, in general, little experience in the snow and ice control community using road thermography to assist snow and ice control decision makers. Such data also have little value by themselves for resource allocation decisions. In addition, the temperature profiles are cumbersome to use. In theory, if a pavement temperature is available for a given point, then with road temperature profiles, an estimate of road temperatures elsewhere can be made. Washington has been evaluating this capability and has demonstrated some success. Little if any change can be made to snow and ice control decision processes using temperature profiles by themselves, however.

Analysis of road thermography data conducted in this project does indicate such data have applicability in conjunction with other pavement condition information, such as pavement temperature forecasts for a specific location, particularly where there is a large range of pavement temperatures. This also implies that some measure of improved proactive decision making can be obtained in this manner. For instance, only certain spots may need attention during a snow or ice weather event.

During interviews conducted as a part of this project, it was found that road thermography data had allowed snow and ice control managers to revise plowing routes to capture the coldest segments first. The data also showed that some structures remained warmer than surrounding road surfaces, and where the maintenance forces had been treating the bridges first, they reversed their priority. There are potential cost savings in these types of decisions. Other savings may be possible by using such data to assist in the placement of sensor systems. Potentially, the daily cost of sensor systems can be reduced by decreasing the number of sensor systems in place. That can be a significant improvement in the B/C for this technology.

Although the price tag for road thermography appears high, it is because it is a one-time, up-front cost. If the cost of the thermographic analysis is amortized over the same 5-year, 180-day winter period, the cost is not much different from the private meteorological support on a daily basis. In the Washington state scenario, using the model with an assumed 5 percent increase in forecasting skill, the B/C also increased about 5 percent, even though the cost of the weather information increased with the added cost of road thermography.

Sensors, Road Thermography, and Forecast Support

There is no single weather information technology that will approach the ability of detailed, local forecasts of road conditions in terms of B/C. Sensor systems by themselves are costly, but provide some increase in decision-making capability. They can provide information for analysis, such as temperature trends, roadway deicing chemical concentrations for taking or not taking action, and icing conditions in particular locations. Road thermography provides little information on its own for decision making. However, there is a synergistic relationship when these technologies are combined. Each allows a meteorological services provider to produce more accurate forecasts. This in turn allows for more efficient and timely snow and ice control decisions. For the combined technologies, the model produces a B/C of approximately 5.0. However, the model also shows a significant increase in the level of service with the combined technologies. Average computed level of service improvements were on the order of 20 percent. An even larger reduction in the number of Type I errors is made possible, with reductions by as much as 90 percent.

CONCLUSIONS

The benefit-cost model previously described shows that the use of road weather information technologies can significantly reduce the costs of snow and ice control. Using weather and pavement condition forecasts proactively to mobilize snow and ice control resources ahead of snow and ice problems, and to not mobilize or patrol when it is not necessary, saves money. Similarly, deploying resources in a timely and efficient way and at the right location also provides a better level of service to the road users.

The greatest benefit-cost ratios are produced by using weather and pavement condition forecasts. B/C > 20.0 result from the low cost of forecast services when compared with the cost of snow and ice control activities. In addition, the model assumes that the maintenance manager always makes the right decision when presented with the weather and pavement condition information. This is an idealistic scenario, given the general conservative nature of maintenance managers who would rather err on the side of safety. It also assumes that perfect communication takes place between the forecast provider and the forecast user. However, the model indicates that tenfold increases in the cost of such services still provide a positive return on investment. It also shows that using winter safety patrols is a costly practice; reliable forecasts can help reduce the costs of snow and ice control significantly if patrols can be eliminated or reduced.

Calculations of benefit-cost ratios using information from sensor systems and road thermography by themselves are significantly lower than those that also use a detailed weather and pavement condition forecast support system. In fact, use of these technologies singly may cost more than they save without forecast support. On the other hand, they can improve the level of service. The model can be used, however, to assess what the maximum expenditures for weather information could or should be in an area by varying the costs of the RWIS technologies.

Sensitivity analysis shows that low B/C results are likely in severe climates (i.e., having many snow and ice problems). The reason is that there are fewer decisions to make when snow or ice happen most of the time. This same reasoning would argue that RWIS support is less likely to provide a great deal of savings when maintenance managers deploy snow and ice control resources throughout the winter in multiple shifts, unless they decide to mobilize with each snow or ice situation. There could be significant savings for these managers, however, in the spring and fall transition seasons, when resources are deployed less regularly.

The benefit-cost model developed for this project shows that through the use of road weather information technologies, the costs of snow and ice control can be reduced. The greatest single savings is possible through the use of detailed weather and road condition forecasting support. The inclusion of road weather information system sensor technology, and the use of road thermography with the forecasts, which result in a reduced benefit-cost ratio, will significantly improve the snow and ice control level of service and will reduce the Type I errors of omission.

REFERENCE

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