TRANSPORTATION RESEARCH RECORD 923

Highway Geometrics, Interactive Graphics, and Laser Mapping

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1983

Transportation Research Record 923
Price $13.00
Edited for TRB by Scott C. Herman

modes
1 highway transportation
3 rail transportation

subject areas
21 facilities design
54 operations and traffic control

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Highway geometrics, interactive graphics, and laser mapping.

(Transportation research record; 923)
1. Roads–Design–Addresses, essays, lectures. 2. Computer graphics–Addresses, essays, lectures. 3. Cartography–Laser use in–Addresses, essays, lectures. I. National Research Council (U.S.). Transportation Research Board. II. Title: Laser mapping. III. Series.
TE7.H5 no. 923 [TE175] 308.5s [625.7'25] 84-4734
ISBN 0-309-03612-7 ISSN 0361-1981

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Contents

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Prospectus on Airborne Laser Mapping Systems

L.E. LINK, W.B. KRABILL, AND R.N. SWIFT

The state of the art of operating airborne laser mapping systems is summarized; also summarized are the results of field experiments conducted to evaluate system performance capabilities. The projected capabilities of systems currently under development and projected for operational testing in the near future are contrasted to the capabilities established for the operating systems. Current constraints on improving performance are identified and discussed with respect to individual system components (i.e., lasers, data recording and processing systems, positioning systems, and information display systems). A prospectus on the performance of future laser mapping systems is provided for specific technology advances.

Accurate measurements of terrain surface geometry are a common requirement for most engineering and environmental studies. Elevation, slope aspect and magnitude, relief, valley and stream cross sections, and underwater topography are basic inputs for analyses of a multitude of natural phenomena. Historically, these data have been collected on the ground or, when possible, by photogrammetry. The invention of the laser has been the most significant recent advancement in measurement technology. Ground-survey techniques have adopted laser technology, and as early as the mid-1960s lasers were placed in aircraft to examine their potential for airborne terrain (1,2) and bathymetric (3) mapping. Increased attention has been given to the airborne laser system, and the technique has emerged as a powerful new tool for terrain mapping (4-8), bathymetry (9-11), and water quality (12,13) applications.

The objectives of this paper are to summarize the current state of the art in airborne laser mapping systems, identify constraints on improving system performance, and project the impact of emerging technology on the potential performance of future systems. The state of the art and the prospectus on the future of laser and positioning systems are also discussed. A summary of current application capabilities and the potential impact of new and emerging technology on application capabilities is provided.

STATE OF THE ART

Laser Systems

The overland surveying capability of laser ranging systems has been recognized for some time, and a number of systems have demonstrated this capability with varying degrees of success. Notable among the most recently reported works are (a) the results of the Australian WREMAPS I systems (6) and the improved WREMAPS II systems (7); (b) the airborne oceanographic lidar (AOL) operated by the National Aeronautics and Space Administration (NASA) Wallops Flight Center (5); (c) a laser profiling system developed by the Avco Everett Corporation (14); and (d) a laser profiling system--the laser airborne profile recorder (APR)--used in South America (8). The Australian systems and the laser APR have provided topographic information for remote areas.

Considerable variation exists in the transmitter, optical, and recording components of the various terrain mapping systems. The wavelength of the laser transmitter is not critical for most terrain mapping applications, especially for low-altitude mapping. The WREMAPS I system uses a continuous wave (CW) argon ion laser at 488 nm, and the APR system uses a CW helium neon laser at 632.8 nm. The Avco and WREMAPS II systems use pulsed-frequency, doubled Nd:YAG lasers at 532 nm, whereas the AOL is equipped with a 337.1-nm pulsed nitrogen laser. The CW laser systems gauge the vertical distance between the aircraft and the ground by measuring phase delay between the transmitted and received signals. The pulsed laser system measures slant range between the aircraft and the ground by recording the time difference between the transmitted and received laser pulses. The APR system records the analog laser return data on a strip-chart recorder. The remaining systems record computer-compatible digital data.

All of these terrain mapping lidar systems report data acquisition in a profiling mode. However, the AOL acquires detailed data in a scanning mode. Scanning necessarily requires a high laser repetition rate, which limits the candidate lasers that can be used. The AOL produces a conical scan pattern along a 30° swath beneath the aircraft.

A number of airborne laser systems can also gather hydrographic data in relatively shallow coastal waters: (a) the Australian WRELADS II system (15); (b) the Canadian MKII lidar bathymeter (16); and (c) the NASA AOL system (9,10,17). Both the Australian WRELADS II system and the AOL can provide scanning data along a 30° swath beneath the aircraft at sampling rates of 156 and 400 pulses per second (PPS), respectively. The Canadian MKII system produces a profile record at a 10-PPS sampling rate. The Canadian lidar bathymeter is flown as an auxiliary component to a photogrammetric surveying system and the lidar record provides vertical reference points for later photogrammetric analysis. The Australian system provides complete survey data independently. The NASA AOL system is essentially a flying lidar laboratory that has terrain mapping, hydrography, and laser fluorosensing applications. The flexibility in the design of the AOL allows rapid change of lasers as well as modification of transmitter and receiver components. The Naval Oceanographic Research and Development Activity (NORDA) is currently developing the hydrographic airborne laser sounder (HALS) (18), which is to be used as an operational system.

Hydrography presents a number of obstacles not encountered in terrain mapping applications. The absorption characteristics of water restricts the laser transmitter to the blue-green spectral region; thus a relatively few candidate lasers can be used. The AOL hydrographic tests flown with an Avco C-5000 nitrogen laser filled with neon gas yielded a relatively low power 10-ns laser pulse at a 540.1-nm wavelength; however, the power requirements for an operational lidar preclude use of this type of laser. The Canadian, Australian, and HALS systems use frequency doubled Nd:YAG lasers at an emission wavelength of 532.1 nm. The 0.8- to 10-Mw peak power outputs of the YAG lasers from these various systems, coupled with a relatively narrow 5- to 7-ns pulse width, are adequate for performing airborne hydrography. The major problem with YAG lasers is in obtaining sufficient pulse repetition rates necessary for most hydrographic applications. The HALS system will be equipped with a 400-PPS YAG laser developed recently by Avco. The Australians have bypassed this problem by using two separate YAG lasers on an alternating-pulse basis. Additional candidate lasers are addressed later in this paper.

The digitization and recording of information from each laser pulse are essential components of

Table 1. Summary of available positioning systems.

Measurement Technique	Displacement	Velocity	Acceleration
Electromagnetic			
Optical	Laser ranging; optical trackers; tracking photographs	NA	NA
Microwave	Radar; active DME	Doppler navigators (Singer Kearfoot SKK-1000)	NA
Radio	Active DME; range receiver	Satellite doppler systems; TRANSIT; GPS	NA
Mechanical			
Inertial	NA	NA	Accelerometers; gyros
Barometric	Pressure; transducer	Rate of change	NA

Notes: NA = not applicable.
DME = distance measurement equipment.

both terrain mapping and bathymetry lidar systems. All bathymetry lidar systems record temporally resolved backscattered waveforms. The existing terrain mapping systems vary considerably in this regard. Some systems, such as the AOL, record the entire waveform, whereas other systems, which are essentially laser profilers, record only the slant range between the aircraft and the initial laser target. These laser profilers depend on a high laser pulse repetition frequency (PRF) and occasional direct penetration through vegetative cover to determine the ground.

Positioning Systems

Laser systems are capable of making extremely accurate distance measurements. Interpretations of the measurements require accurate information on the position and orientation of the laser at the time of the measurement. The absolute accuracy obtainable by virtually all existing laser mapping systems is constrained by the associated position measurements, not by the laser measurements.

Positioning systems can be classified into two broad categories: electromagnetic (EM) or mechanical. Electromagnetic systems use optical, microwave, or radio frequency energy to measure displacement, velocity, or acceleration. Mechanical systems rely on physical phenomena and can be further subdivided into inertial and barometric types. The data in Table 1 provide a general summary of the different types of positioning systems that have some potential for use in airborne laser mapping applications.

Electromagnetic systems that use displacement for positioning can operate in three geometric configurations. The EM systems and other major systems are briefly discussed in this section.

EM-Optical

EM-optical positioning systems use laser systems for measuring either distance (range) or angular displacement. Three-range systems are possible that provide range measurements from three reference ground stations to the aircraft. Because laser systems provide both accurate range and angle measurements, these systems commonly rely on a single-range measurement coupled with two angle measurements. The EM-optical systems provide about the best resolution (0.01 to 0.02 m) and accuracy (about 0.30 m) of all positioning systems.

EM-Microwave

EM-microwave positioning systems use both displacement and velocity techniques. Those systems that use displacement have either range-range-range or range-angle-angle geometric configurations, whereas those systems that use velocity are primarily doppler navigation systems. The doppler-determined velocity is integrated to determine displacement for the actual position determination.

Basic doppler navigation systems usually have three components: the doppler radar, a compass, and a computer. The doppler radar provides ground speed and drift information; the compass provides a heading reference; and the computer processes this basic information into position and guidance data. Although such systems are extremely valuable for aircraft navigation, they would not provide adequate absolute positioning data for many laser mapping applications.

Virtually all of the microwave displacement systems use a transmitter or receiver on the aircraft and three or more ground reference stations with active transponders. Such systems are commonly used for x-y or horizontal positioning applications. The resolution for microwave displacement systems is approximately 0.1 m (horizontal), and accuracy is roughly 1 to 2 m. The systems are advertised as being capable of determining positions of moving aircraft out to ranges of 185 km, depending on the systems used. Position updates can be made from 2 to 10 times/sec.

EM-Radio

There are two major types of EM-radio positioning systems: those that operate on displacement measurements, and those that measure velocity through doppler techniques. The doppler techniques are primarily used with satellite-based navigation systems.

Radio-displacement positioning systems operate only in the range-range-range geometric configuration. Both the VHF (1.6 and 3.3 MHz) and UHF (450 MHz) frequency bands are used. Virtually all of these systems were developed for horizontal positioning of ships at sea or aircraft over water. It is possible to determine positions out to ranges of 400 km at night and twice as far during the day; range resolutions are approximately 1.0 m, and range accuracy is approximately 2 to 3 m. Typically, position is determined about once every 2 sec. VHF system performance over land can be degraded to about one-half that possible over water.

The TRANSIT satellite currently provides the only operational satellite doppler positioning system. This system, which was initiated in 1967, currently has five satellites in nominal polar orbits. Users determine their position by measuring the doppler shift between their receiver's very stable oscillator frequency and the received frequency from the satellite. Resolution of the TRANSIT system is approximately 200 m.

The NAVSTAR global positioning system (GPS) is a 18-satellite, worldwide radio navigation system that is intended to become operational in the late 1980s. The GPS is intended to provide continuous global coverage for an unlimited number of passive users and also provide users with details of precise position, velocity, and time (19). Accuracies obtainable are expected to be within 8 m horizontal, 10 m vertical, and 0.03 m/sec velocity (19).

Mechanical Inertial

Many aircraft rely on inertial navigation devices for determining their position with respect to destination or some other reference. Inertial positioning systems are also used in the surveying and mapping industry to accurately locate ground points over large areas. In this capacity such systems are used in both ground vehicles and aircraft (primarily helicopters). Inertial positioning systems operate on the principle of Newton's laws of motion and rely on two devices--the gyroscope and the accelerometer--to do their job. Gyroscopes provide an extremely accurate means to measure direction and angular rates of change.

Inertial positioning requires the measurement and double integration of acceleration to determine displacement with time. Usually three orthogonally mounted accelerometers are used to meet this need. Because accelerometers are sensitive to both gravitational and inertial reaction forces, changes in the gravitational component must be compensated for to achieve accurate results, especially in the vertical channel. Three orthogonal accelerometers and two, 2-degree-of-freedom gyroscopes (mounted on a stable platform and isolated from the maneuvering of the carrying vehicle) are normally the heart of an inertial positioning system (20).

Before an inertial mission, a 30- to 50-min stationary period is used to orient the system into the local level coordinate system. During the course of a mission, it is necessary to make periodic zero velocity updates (ZUPT). During ZUPT times, the difference between system velocity as output by the software and the true velocity (zero) is used to estimate future system performance. Kalman filtering is typically used in making error and correction estimates.

The accuracy of the inertial systems depends on the number and spacing of ZUPTs. When inertial systems are operated in a ground vehicle or in a helicopter under specified surveying procedures, the total horizontal distance traveled is generally accurate to within 1 part in 10,000 horizontally and 1 part in 20,000 vertically (21,22).

Mechanical-Barometric

Barometric pressure systems represent one of the oldest methods for vertical positioning of aircraft. A constant barometric pressure (isobaric) surface (assumed to be spheroid concentric with the geoid) is monitored by checking for changes in barometric pressure. When isobars are sloped with respect to the geoid, a meaningful reference for vertical aircraft motion is not represented, except over short distances.

Positioning Systems in Use

The WREMAPS II system initially used a Wild Statoscope type RST 2 as a barometric pressure altitude reference. The device has been reliable in the field and has demonstrated short-term repeatability of approximately ±1 m and long-term repeatability of ±2 m. An advanced Statoscope system has been developed that will improve height measurement repeatability to 0.5 m (7). A 70-mm strip camera is used to record the precise track followed by the laser beam.

The Avco airborne laser mapping system uses a continuous ground-based three-axis microwave positioning system to help determine the vertical and horizontal position of the aircraft. The laser altimeter and positioning system are computer controlled. Aircraft roll and pitch are determined with a two-axis vertical gyro, and a barometric pressure reference monitors additional changes in aircraft altitude. The system output is a digital tape that provides parameters for horizontal and vertical position for both the aircraft and the point of measurement.

The NASA AOL uses a Litton LTN 51 commercial inertial navigation system to acquire positioning information. The velocity output and true heading of the LTN 51 are used to compute the flight path of the aircraft. Roll and pitch parameters are obtained from the same system, and all position data are digitized for subsequent correction of the laser altimeter data. An inexpensive vertical accelerometer is used to monitor short-term vertical motions of the aircraft. When the accelerometer is coupled with the elevations of three points along a flight line, long- and short-term aircraft vertical motion can be removed from the laser data through a quadratic correction. In addition, the aircraft is equipped with a nadir-oriented, 35-mm half-frame camera and a nadir-oriented television camera with a video cassette recorder.

An APR system, as applied by TRANARG C.A., Caracas, Venezuela (8), used a Spectra Physics Geodolite 3-A laser profile system in conjunction with a Statoscope differential barometric pressure altimeter (vertical control) and a Cubic Corporation autotape microwave positioning system (horizontal control). Daily air temperature and pressure measurements were made at several ground stations to help determine changes in the isobaric surfaces. All profile lines flown were relatively short distances that began and ended over known reference points. Aerial photography was also used to help establish the reference points.

Summary of Current Application Capabilities

Terrain Surface Mapping in Open Areas

The ability to map terrain surface geometry is the simplest task for an airborne laser mapping system. The principal constraint is the ability to position the aircraft during flight. A terrain profile acquired in a joint U.S. Army Corps of Engineers Waterways Experiment Station (WES) and NASA experiment (4) that used the NASA AOL system is shown in Figure 1. Aircraft motion was removed successfully by using three known reference points and the output of a vertical accelerometer. The laser profile is compared with a reference profile obtained from low-altitude stereo photography by using standard photogrammetric techniques. The root mean square difference between the two profiles over the 1200-m distance was 27 cm. On similar flight lines that have less topographic relief, the root mean square difference was as low as 12 cm for nonforested areas.

Terrain Surface Mapping in Forested Areas

A portion of the flight lines flown in the joint WES/NASA experiment was over wooded terrain. A laser profile and a reference photogrammetric profile over both open and forested areas for "leaves-off" conditions are shown in Figure 2. The root mean square difference between the laser and reference profiles was 50 cm in the forested area. Close examination of the actual ground profile in that area indicated that much of the difference recorded was due to the inability of the photogrammetric technique to accurately depict the terrain surface in the wooded area. Preliminary analyses of "leaves-on" data for dense forests indicated that only 5 to 15 percent of the laser pulses actually penetrated the canopy, reflected from the ground, and reached the aircraft, as opposed to approximately 40 to 50 percent of the pulses under leaves-off conditions.

Figure 1. Comparison of laser and reference profile for a stream valley.

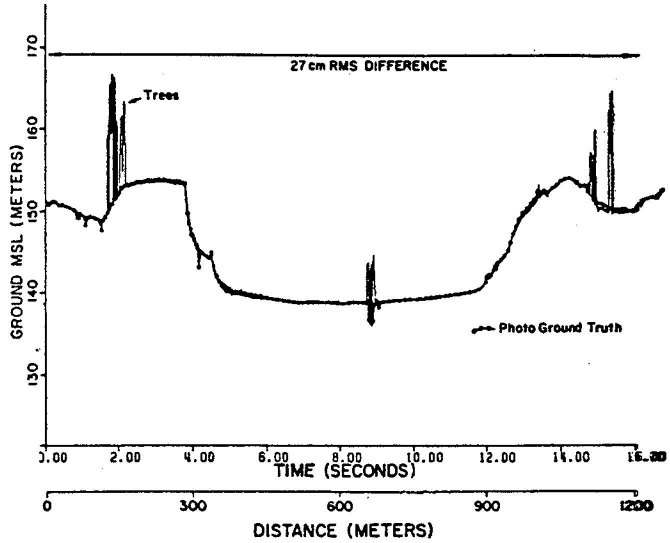
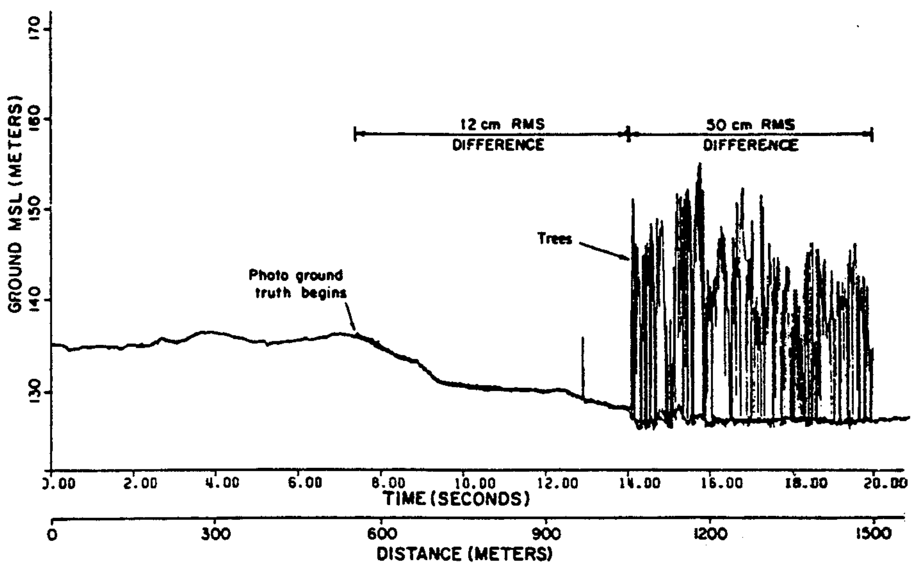


Figure 2. Comparison of laser and reference profile over open and forested areas.



A comparison of three-dimensional perspectives for photogrammetric ground truth and for the composite of two laser-scan passes over a forested (leaves-off) site is shown in Figure 3. The laser data were smoothed for easier comparison to the photogrammetric data. An independent point-by-point comparison indicated that almost 80 percent of the laser data were within 1 m of the photogrammetric data.

Bathymetry

Experiments with the NASA AOL system (9) and the Avco airborne laser mapping system (14) have indicated that the capability exists for reliably mapping bottom geometry in clear ocean water to depths of 10 or 12 m, and in slightly turbid waters to 4.6 m.

The Defense Mapping Agency and NORDA are developing an operational system called HALS. The preliminary work done with the NASA AOL system (a prototype for the HALS) resulted in the following conclusions (23):

1. The technique was able to measure water depths to within the 0.3-m root mean square accuracy standard over the set of conditions experienced;
2. Airborne laser hydrography could be performed for one-sixth the cost of conventional sonar surveys;
3. Airborne laser hydrography required only one-fifth the manpower of conventional sonar surveys; and
4. The technique offered the added potential benefits of a 100-fold increase in the number of soundings per unit area.

In a recent study with the Corps of Engineers Wilmington (North Carolina) District, the NASA AOL system was used for nearshore mapping. An example of both beach and bathymetric laser profile data is shown in Figure 4.

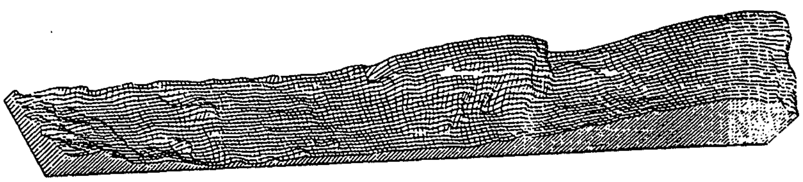
PROSPECTS FOR THE FUTURE

Laser Systems

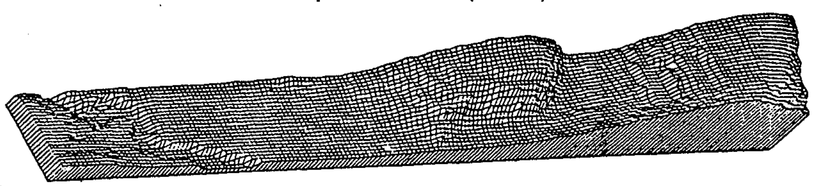
The future laser systems for a host of airborne mapping applications are almost on-line. The capability to produce subnanosecond pulses at various wavelengths will soon be the state of the art, as will be the requisite higher pulse rate frequency (PRF) and output power. Laser subsystem reliability and maintainability will follow shortly thereafter.

Development of scanning mechanisms to directly collect data in a true raster format will take place as higher PRFs and power become available, thus facilitating the subsequent data processing. Moreover, the higher PRF will increase data density to the point where repeat or overlapping passes will not be required, thus saving expensive flight time. Soon airborne laser systems will be operationally limited only by the speed of electronics for command, control, and data capture. Application of these systems is currently limited (and will be in the future) by the capability of navigation and positioning subsystems, not by the lack of measurement quality and quantity from the laser itself.

Figure 3. Comparison of laser scan and reference data for a forested area.

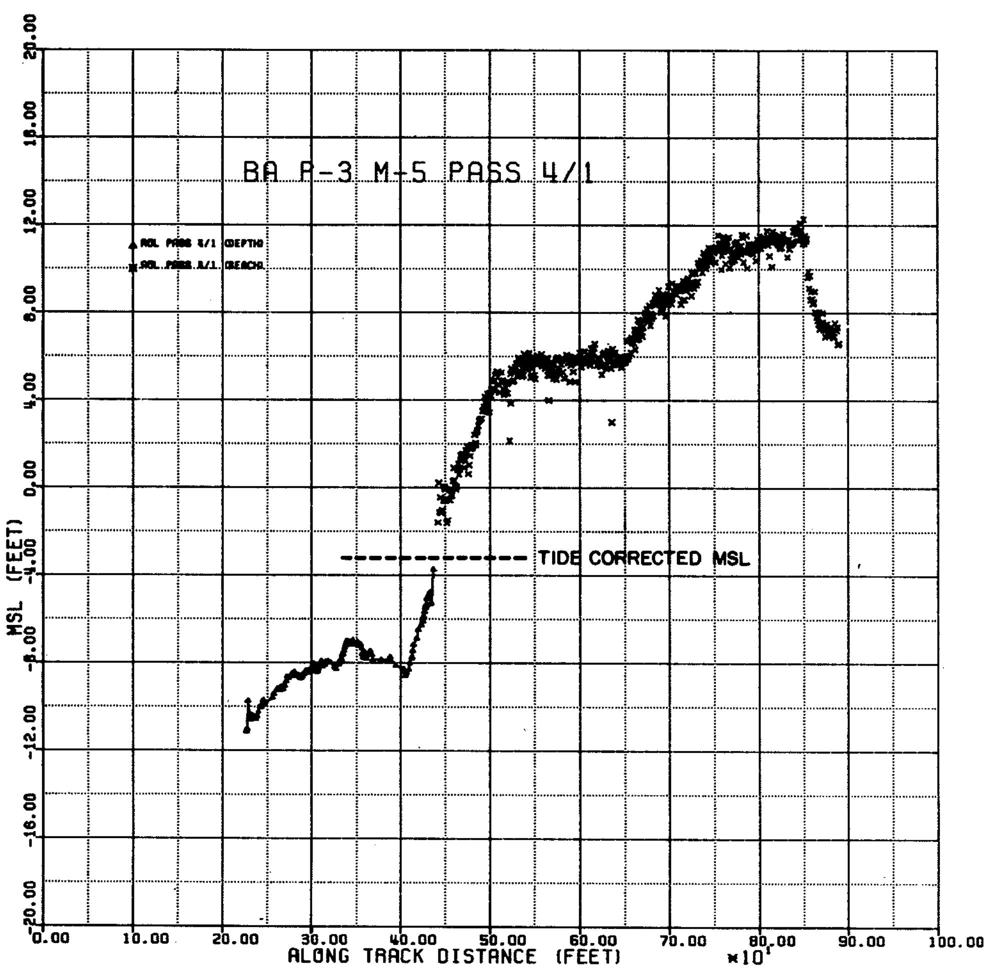


a. Composite of 2 laser scans (smoothed)



b. Photogrammetric ground truth

Figure 4. Example of laser profile data for mapping experiments near shore areas.



Positioning Systems

Many of the systems outlined in Table 1 have adequate horizontal positioning capability for many mapping applications because a vast majority of the systems were designed specifically to give information. The accuracy of vertical positioning capabilities with laser tracking devices is approximately 30 cm, and accuracy with microwave systems is 2 m. These capabilities are adequate for some applications, but marginal for others.

There are valid reasons why the more sophisticated positioning techniques have not been more widely used. Many systems (such as the range-range-range microwave and radio frequency devices) require active ground reference stations that are expensive enough to warrant manning each station to prevent loss or damage to the equipment. The laser tracking devices are extremely accurate and can operate from a single point (given the line of sight) but require a sophisticated and expensive ground platform for tracking. Inertial systems have enormous potential, but the requirement for frequent stops for ZUPT purposes is prohibitive in any aircraft except helicopters. Unfortunately, these systems are not designed specifically to meet the needs of airborne laser mapping systems.

Aside from specific accuracy requirements, the ideal positioning system for airborne laser mapping applications would require no ground reference stations. Of all the concepts currently in use or being developed, only inertial systems or the GPS could provide this capability. If ground stations are necessary, it would be beneficial if they were passive, inexpensive, and required no manpower. Future GPS capabilities are fairly well determined, and for some applications the GPS horizontal resolution will be adequate. Inertial systems that operate in helicopters may provide high accuracy over short survey lengths. Both EM-optical and EM-microwave devices have considerable potential for use with passive reflectors. Finally, combining different types of systems to generate an enhanced total capability has considerable potential for the future.

EM-Optical and Passive Ground Stations

The U.S. Geological Survey (USGS) and NASA are evaluating the use of laser tracking devices in aircraft and passive retroreflectors on the ground. The USGS investigation is in conjunction with development of their aerial profiling of terrain systems (APTS). In the APTS a sophisticated inertial navigation system will define the position of the aircraft in three coordinates, where a two-axis laser tracker will determine long-term drift errors of the inertial platform (the laser tracker data will be a substitute for the ZUPT data). Three or more retroreflectors positioned over known stations interspersed with several other reflectors will provide ground truth every 3 min. The accuracy goals for the system are 50 cm horizontal and 15 cm vertical.

The NASA Goddard Space Flight Center has been engaged in the design of a centimeter-accurate, multibeam, airborne laser ranging system (ALRS). The basic philosophy behind ALRS is to invert the usual satellite tracking laser ranging configuration by placing the ranging and pointing hardware in the aircraft and replacing the expensive ground stations with relatively low-cost retroreflectors. The ALRS will be capable of simultaneously ranging to six retroreflectors, thereby giving aircraft position to less than 1 m.

SUMMARY

The general advantages of airborne laser mapping include the ability to (a) collect data sets in a period of seconds that might otherwise require days or weeks by a ground survey team; (b) acquire data densities that are orders of magnitude greater than those feasible for ground systems; (c) acquire data in areas inaccessible to ground crews; and (d) collect data in a digital form that leads to easy and immediate computer processing. The ability to accomplish terrain surface mapping (even in forested areas) and bathymetric mapping (in reasonably clear water) has been demonstrated. Perhaps the most serious constraint to improving the performance of airborne laser mapping systems is the adaptation of improved positioning technology. Improvements in laser systems can enhance current capabilities, but not as much as improvements in positioning.

The major relevant advances in laser systems will focus on increased repetition rates and peak power levels. Increased power levels alone will not significantly enhance system performance for terrain mapping; however, significant increases in the depths at which bathymetric mapping can be accomplished could be a result of such efforts. Increased pulse rates would allow more efficient scanning by use of raster rather than conical patterns and would also enhance the ability to penetrate denser vegetation canopies.

Positioning systems are available to provide adequate horizontal control for most mapping applications. Nevertheless, most of these devices require relatively expensive active ground reference stations, which can be a manpower and cost constraint. That is not to say that such systems cannot be used cost effectively; in fact, the cost of laser surveys where such systems are used can be as much as one-third or one-half the cost of conventional survey methods. Vertical positioning for detailed surveys can be achieved over short flight paths; however, to do so requires the application of numerous methods in conjunction with considerable ground truth.

Available positioning systems have not been designed for airborne laser mapping. Techniques that use passive rather than active ground reference stations and aircraft-based optical or microwave range and doppler systems are needed to address the horizontal and vertical positioning problems over large areas.

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Publication of this paper sponsored by Committee on Photogrammetry and Aerial Surveys.

Interactive Graphics in Highway Design

H.A. HENRY

The Texas State Department of Highways and Public Transportation has an automated photogrammetric data-acquisition system that uses aerial stereophoto digitizing systems, which are integrated through video graphics, in order to produce digital models of the geographic areas on which highway facilities are proposed to be constructed. Highway design engineers then use a computer-supported system of engineering programs in order to interact with the digital model and proceed through the necessary logical processes at remote terminals to superimpose the design of the proposed highway facility on the existing terrain and develop the required engineering criteria for construction of the facility. The graphics related to these processes can be handled in either the video or hard-copy mode. In addition to generating the usual graphics provided in a set of construction plans, the system allows the designer to view three-dimensional displays of the proposed roadway and bridge structures as they will appear to the driver when completed. The system is applicable to all similar engineering problems, such as airports, waterways, streets, and utilities. The purpose of this paper is to discuss how the automated data-acquisition system functions and how a remote network of video graphics terminals supports the highway design process in a distributed data-processing environment.

Large and complex highway facilities constructed by the Texas State Department of Highways and Public Transportation are designed in a decentralized environment; there are 25 district-level offices and more than 100 subdistrict-level design offices. Detailed digital geographic data bases (Figure 1) are developed by photogrammetric methods in a central facility and are made available to the designers through remote terminals. The geographic data bases support an integrated automated highway design system. The complexity of the design problem will determine the level of terminal support needed; i.e., cathode-ray tube (CRT), card reader and printer, drum plotter, or video graphics station. Simple projects can still be accomplished from data obtained locally by manual field methods, but because of limitations in personnel and escalations in overhead costs it is becoming more feasible to use automation for all design projects.

AUTOMATED DESIGN SYSTEM

The automated design system and subsystems used by the Department have been developed in-house over a period of more than 20 years. Several years ago the users of this system outside the Department organized a users group and, at the Department's request, established a jointly funded systems maintenance arrangement with a software consultant. The Department participates in this user group and contributes to the maintenance contract.

Hard-copy graphics have been an integral part of the design system since digital plotters were first introduced. It was evident that graphics would be the key to the acceptance of electronic computer-oriented technology by senior highway design engineers. The associated computer coding, processing, and printouts were more comprehendible when graphic representations were generated. Currently, young engineers are seeking employment with the Department because of its automation environment.

Video graphics were introduced to Department design engineers at the local level about 3 years ago (Figure 2). The response from both the design staff and their managers has been so positive that the demand for this capability is growing faster than support resources can be provided. The support problem in Texas is also complicated by the Department's decentralized organizational structure and

Figure 1. Graphical representation of geographic data base.

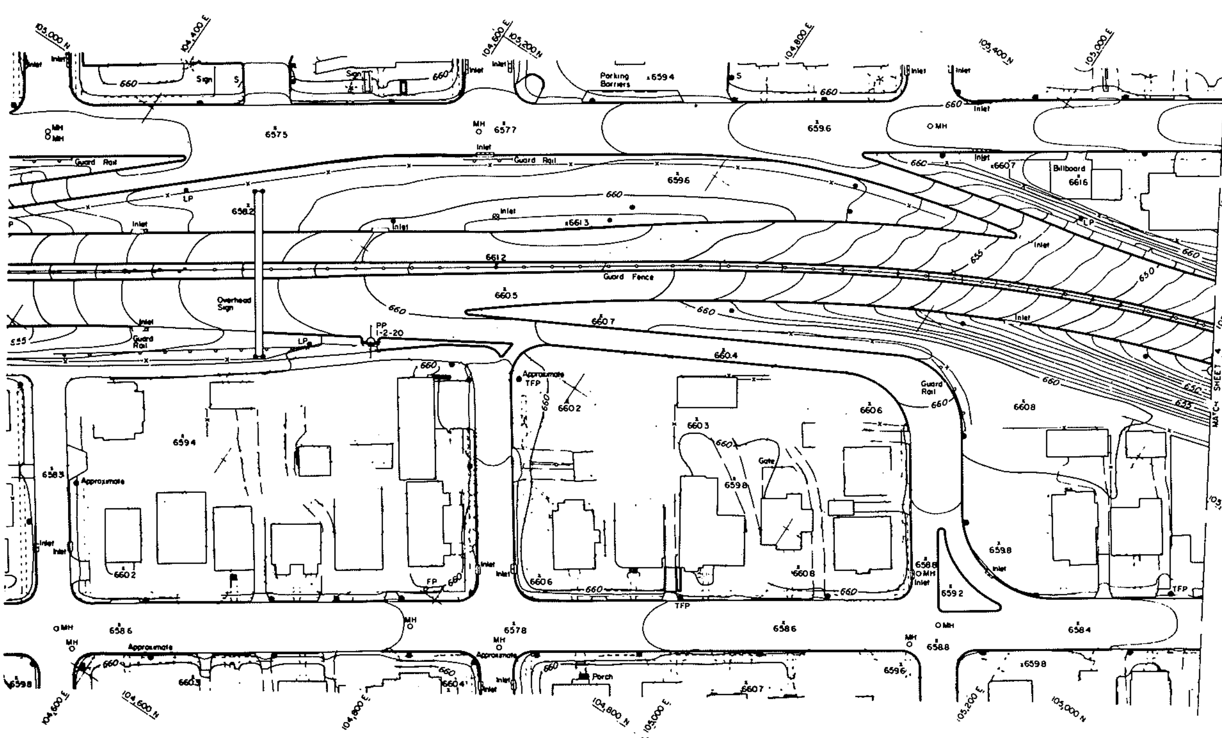
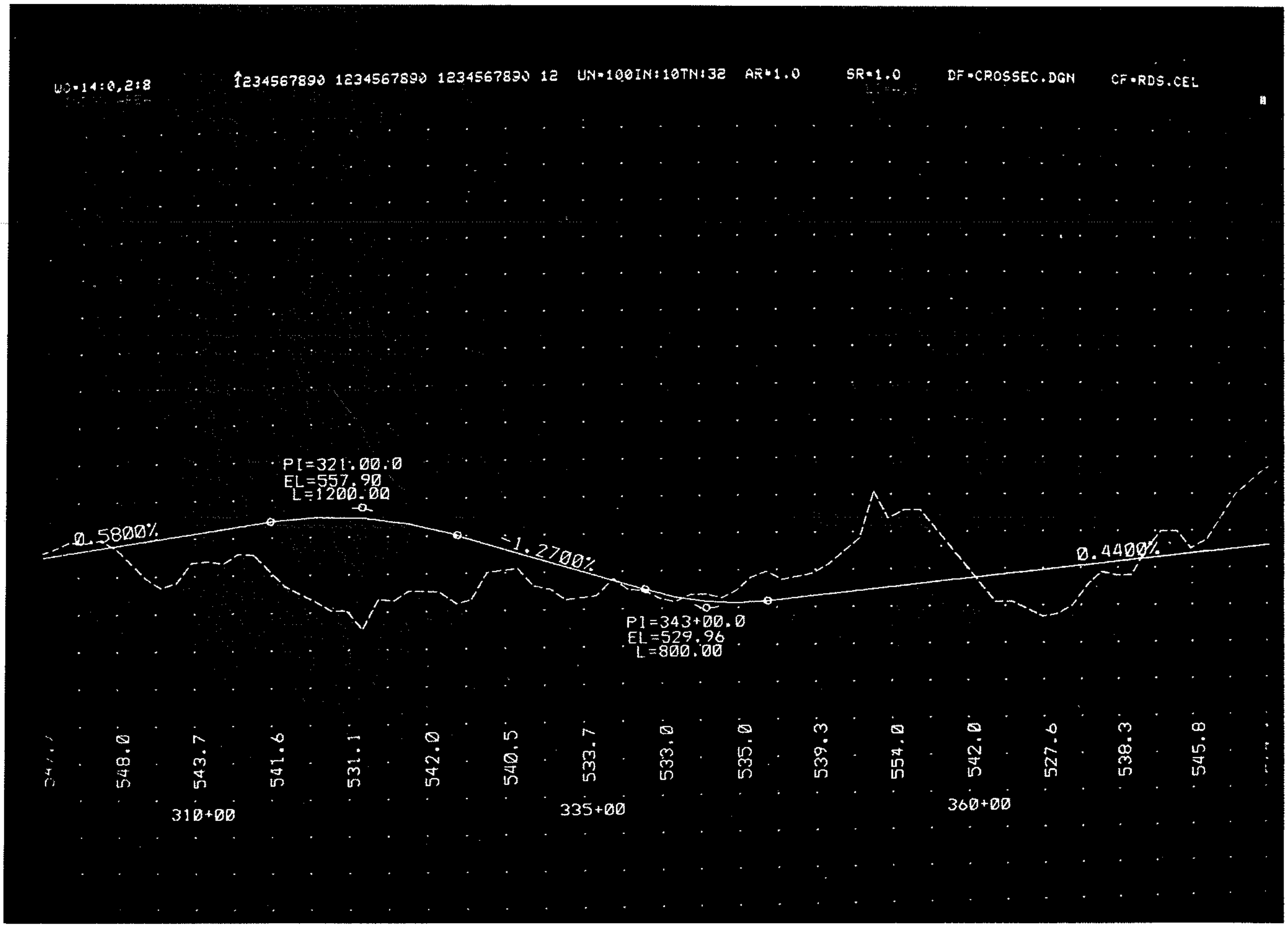


Figure 2. Automated highway design system graphics (roadway profile).



the projected work load. The Department's current commitment is to complete projects worth $1 billion each year for a 20-year period (based on 1976 dollars with a built-in automatic inflation factor).

The Department has further automated the design system so that it interacts with a management information system for project history, finance, accounting, and management controls, Unfortunately the scope of this paper does not permit discussion of the total interactive information system.

The end product of the automated highway design system is a data base from which a set of plans and specifications are produced. These plans are provided to contractors when they bid on the project, and they are also used for the construction of the project (Figure 3). The same data base is used to produce three-dimensional views of the highway facility as it has been designed. These views are superimposed on the existing terrain for final review by the designer or the contractor before bidding.

The initial files of the data base are digital descriptions of the existing culture and terrain that are oriented to the State Plane Coordinate System and U.S. coast and geodetic elevation data. These files can be represented as hard-copy, video planimetric, or topographic map sheets (Figure 4).

The cultural and terrain data files are developed from aerial photography by using a unique automated photogrammetric process--the Texas automated mapping system (TEAMS). The automated process actually begins with ground-control surveys that use total station field surveying equipment, which can record all horizontal and vertical distances, angles, and elevations directly to magnetic tape. The recorded data are input through telecommunication terminals to a digital processor located at the central office, where it is interpreted to determine the usual geodetic parameters of the survey line and the generation of a line drawing. A satellite doppler surveying system has been funded and will be implemented for these surveys as soon as the equipment can be ordered and the techniques developed. An interferometry satellite surveying system is also under consideration for this application.

A minimum number of horizontal and vertical control points are established on the ground because they can be supplemented with additional photographic control points obtained through an analytical stereotriangulation process in order to meet required map accuracies. Horizontal and vertical data are read from aerial photography by using electronically supported high-resolution optical equipment, designed specifically for this purpose, for input to a computer program to generate an accurate coordinate location and elevation determination for each point selected. The capability of providing supplementary control points reduces the cost of a field survey by approximately 80 percent. The result is a plotted grid coordinate system that has all control points identified in their exact positions for orientation of the aerial photographic stereo

Figure 3. Flowchart of the automated highway design system.

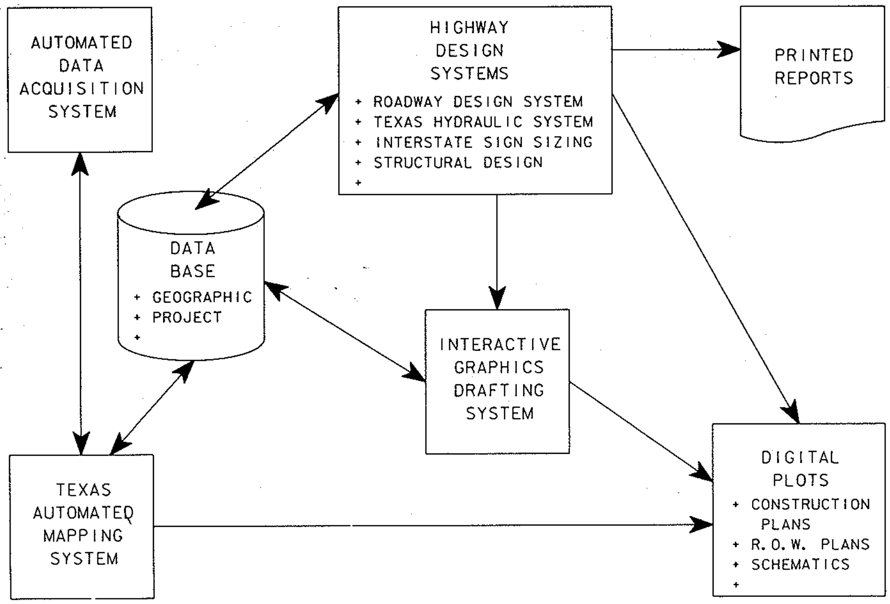


Figure 4. View of planimetric and topographic maps on a graphics station.

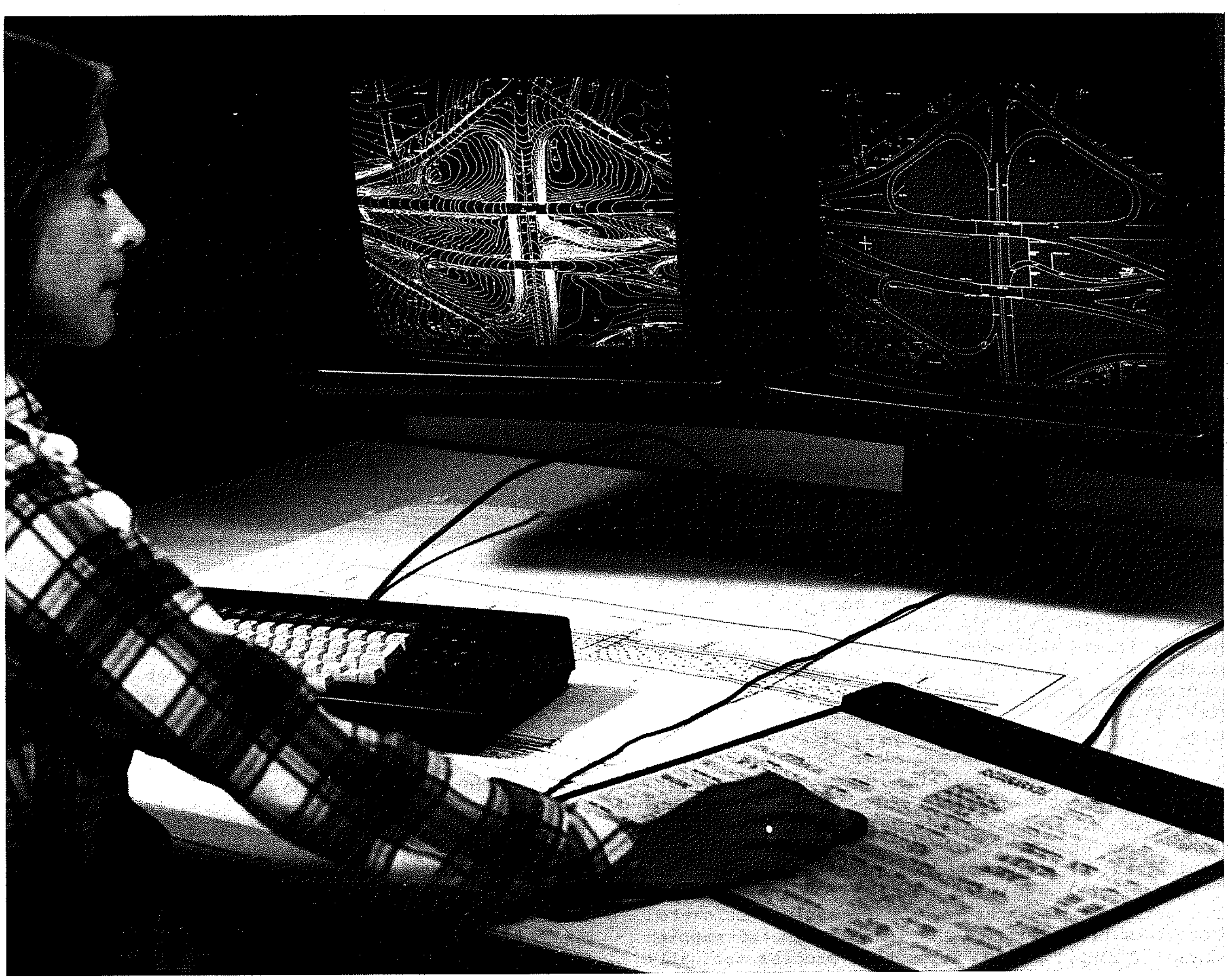
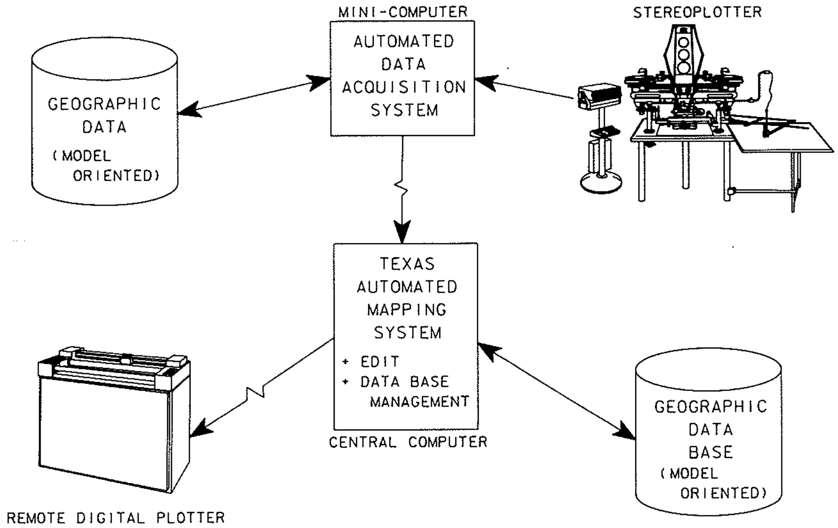


Figure 5. Flowchart of the automated data-acquisition system.



models from which the desired terrain detail will be obtained.

Data from the model are then obtained by using analog optical train stereoplotters modified to accommodate electronic X-Y-Z digitizers and encoders with visual displays that are interfaced with a minicomputer (Figure 5). New automatic adjustment capabilities have been designed into the software to permit the operator more flexibility in setting the model and positioning the coded data relative to the control survey. Fifteen plotter coding stations, operating simultaneously, store the coded data on a magnetic disk supported by a minicomputer.

The communications console of the minicomputer transmits the file from the minicomputer disk to a large processor in the central computer center, with instructions to the system to call up a support program and build instructions for a remote digital drum plotter. The stereoplotter operator handles the routine of building the file, transmitting the file from the minicomputer to the central processor, and then calling for the plot on the digital plotter. The graphic representation of the stereo model is checked for errors, and corrections are made if necessary. The operator signs off on the file when it is complete and correct (Figure 5).

The file in the central computer is then accessed by the interactive graphics minicomputer and written to a disk for further manipulation. An operator trained to use the video presentation of the photogrammetric model makes the final edit by viewing the original aerial photographs in stereo and making any corrections by interacting directly with the file (Figure 6).

Information that is needed to make the graphics more usable must be added to the file from sources other than the aerial photographs. Examples of the information needed are street and stream names, unique objects, and symbol labels; titles, borders, and special numbering may also be desired. These items are entered on a second video graphics terminal. The sheet laid out in the final display may consist of several models from consecutive pairs of stereo photographs. The definition of the file that supports each display is also determined and identified.

The highway design engineer now has all the necessary information to call up a set of programs developed for highway design--known as the roadway design system (RDS)--to establish the location, alignments, grades, geometrics, earthwork quantities, and right-of-way requirements of the roadway (Figure 7). Because all of these functions interact, the designer has the ability to try different design strategies with a minimum of effort and to analyze the results for a design that has optimum serviceability to the public on the bases of cost-effectiveness and minimum environmental impact.

The designer uses the cross sections obtained from the stereoplotter to develop cross sections of the roadway structure being designed to estimate the amount of earthwork involved. Because earthwork is normally a high-cost item on the project, it is an important part of the different strategies considered. When the project is completed, or at certain points during the construction, new aerial photography is obtained and cross sections are developed to determine the exact amount of earthwork that has been done. Generally, the contractor is paid on these determinations. Until recently, the design engineer was interacting with the system by using hard-copy graphics, hand-coded input forms, and keypunch and remote job entry terminal services because the cost of on-line telecommunications systems and remote video graphic terminals for 25 district-level design offices was difficult to justify. A number of factors have changed in the overall evaluation of these functions, and the Department currently has a commitment to a distributed data-processing network that will support interactive graphics and an on-line alphanumeric engineering design system.

VIDEO GRAPHICS

The Department currently has design video graphic stations installed in 16 district offices: Houston, Houston Urban, Dallas, Fort Worth, Lubbock, San Antonio, Corpus Christi, Wichita Falls, Tyler, Abilene, Odessa, Pharr, Amarillo, El Paso, Beaumont, and Austin. Additional districts will be added when the hardware is delivered. All districts will be equipped according to work assignments. These stations are being introduced as drafting stations, where alphanumeric video terminals replace the hand-coded and keypunch input process for engineering problems. Parallel to this implementation in the

Figure 6. Flowchart of the TEAMS process.

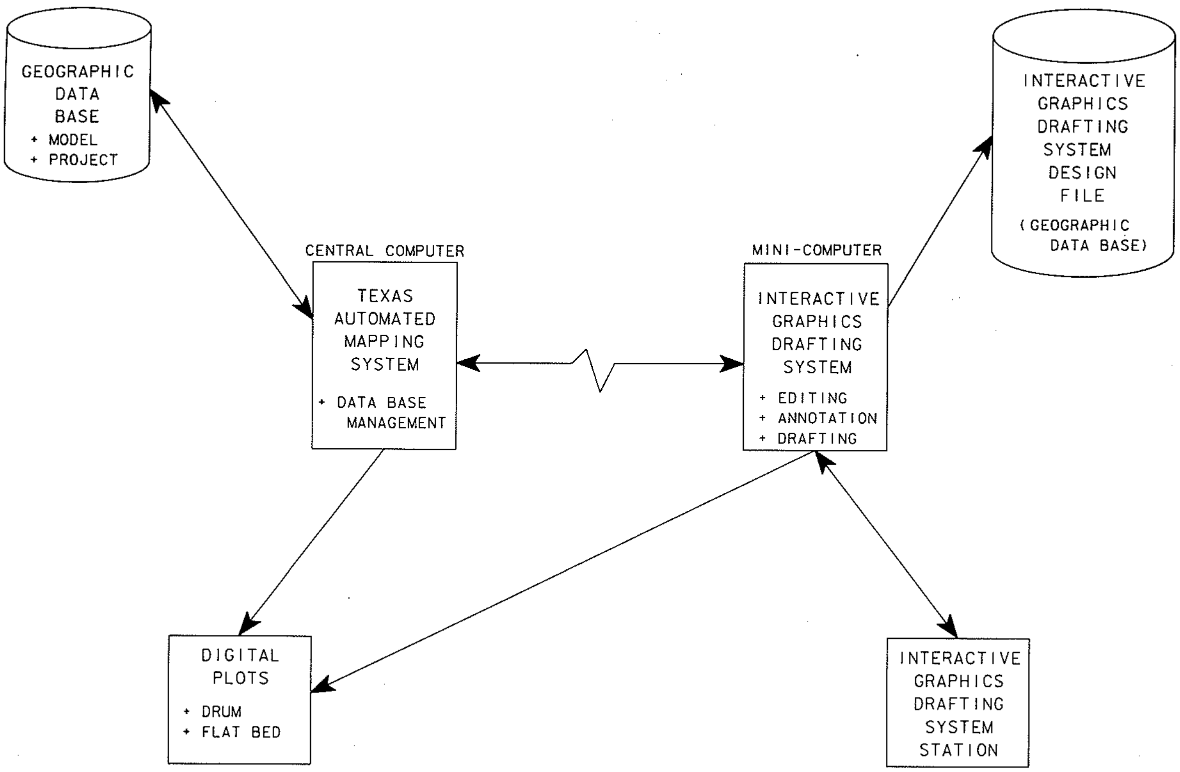


Figure 7. Flowchart of the RDS programs.

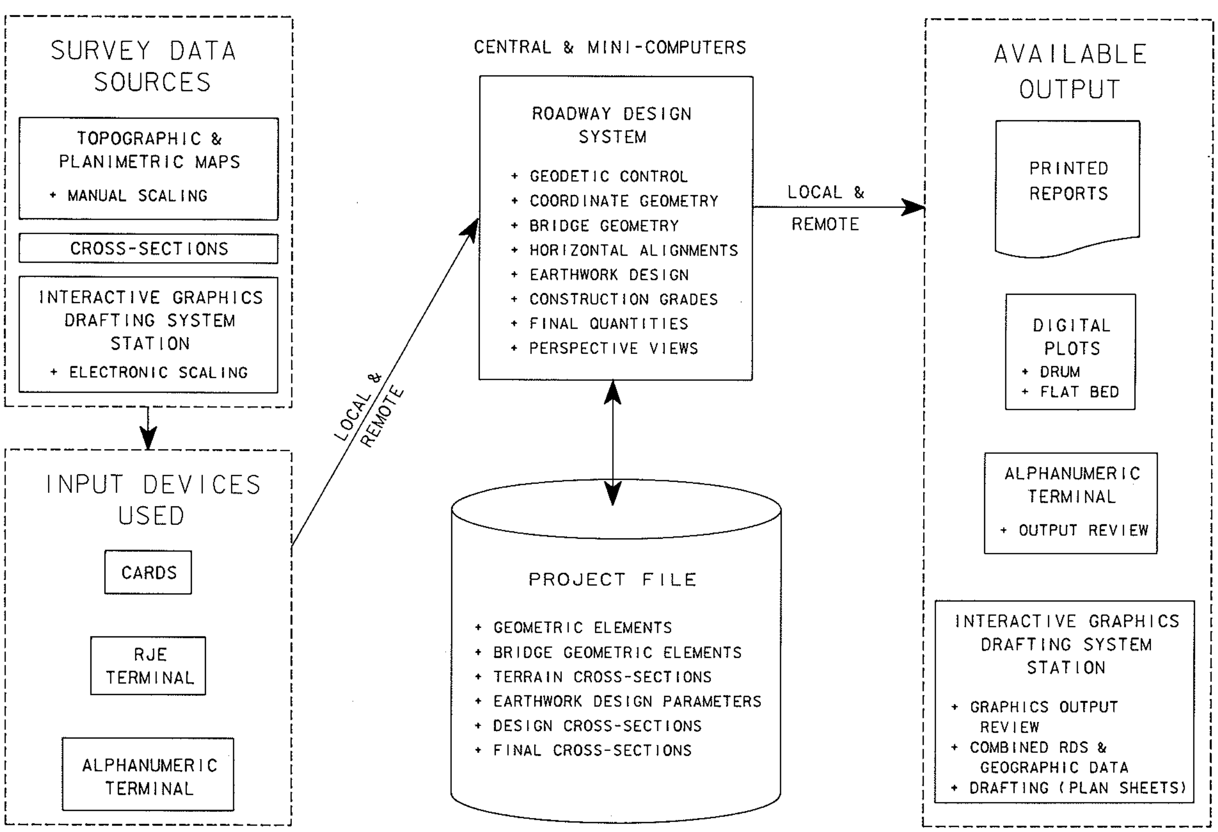


Figure 8. Flowchart of the interactive graphics engineering design system.

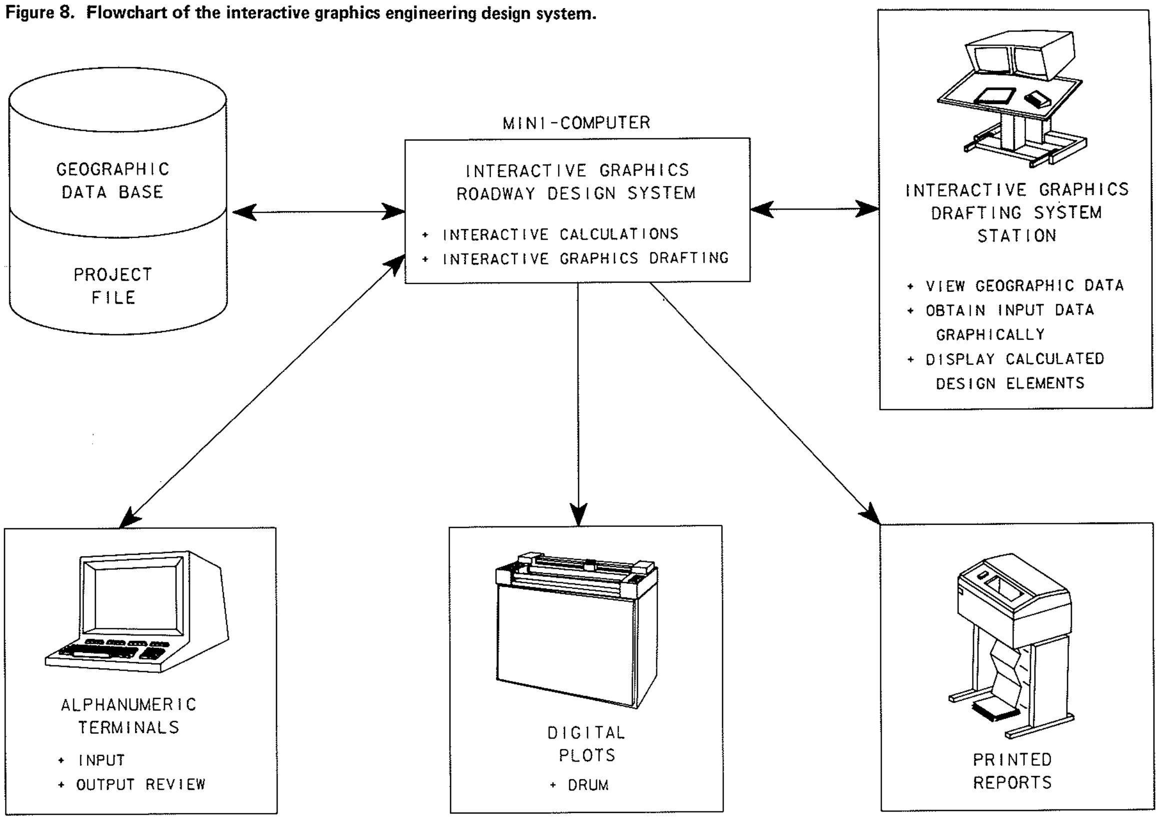
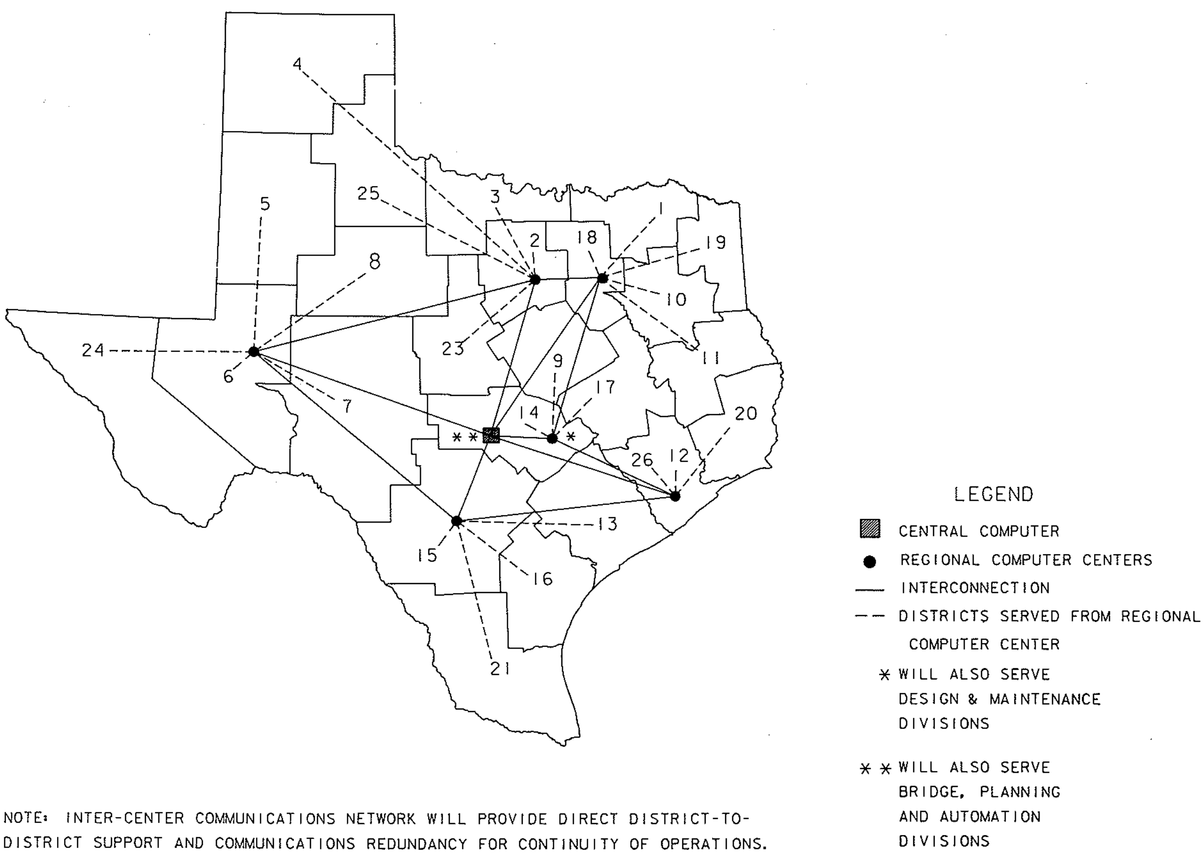
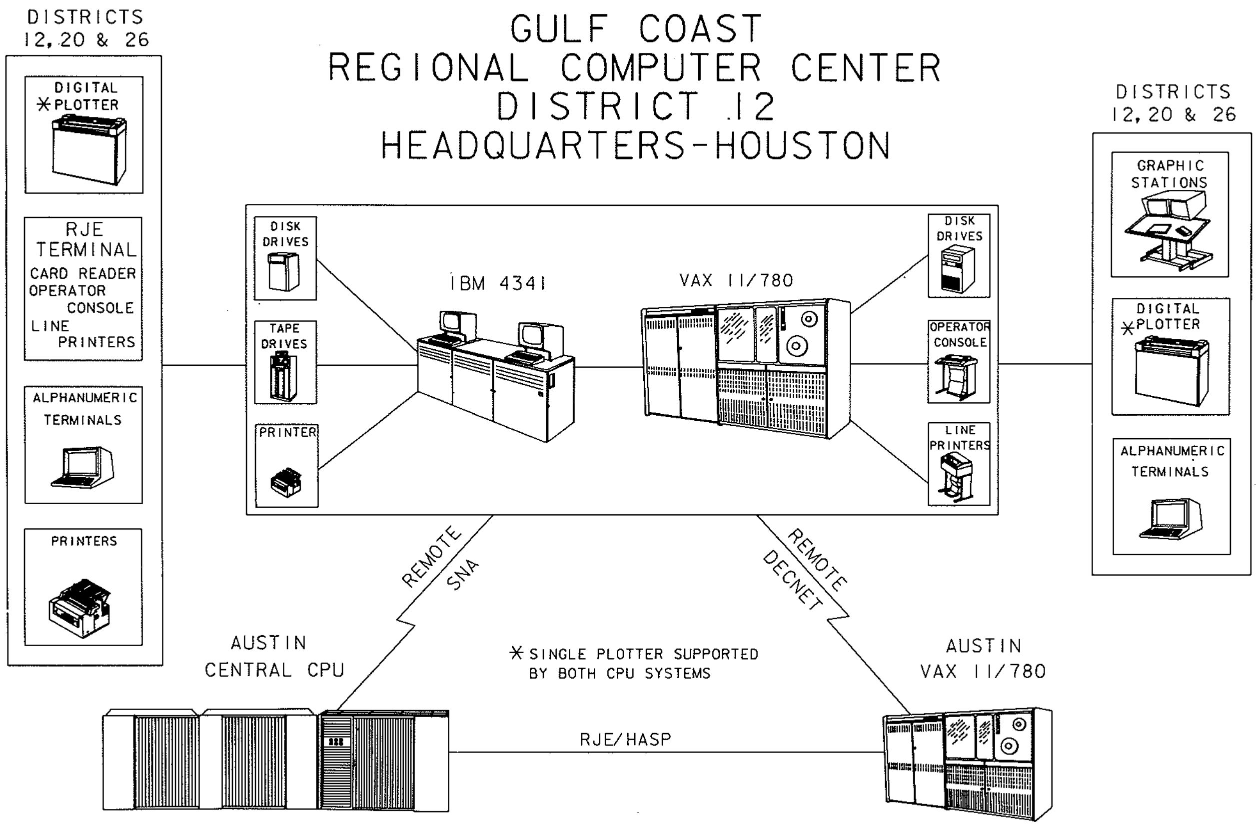


Figure 9. Regional telecommunications network.



NOTE: INTER-CENTER COMMUNICATIONS NETWORK WILL PROVIDE DIRECT DISTRICT-TO-DISTRICT SUPPORT AND COMMUNICATIONS REDUNDANCY FOR CONTINUITY OF OPERATIONS.

Figure 10. Typical regional center setup.



field, a prompted interactive graphics and alphanumeric capability is also being developed.

All of the RDS graphics produced as output on hard-copy digital plotters can now be displayed in the video mode. Interaction with the video display in order to modify the design can be accomplished from alphanumeric terminals because both terminals can enter the same data base, but this is still a batch process. A contract has recently been negotiated with a consultant to complete the interactive graphics software for RDS (Figure 8).

The original RDS was developed to run on IBM equipment. It has since been converted to run on Burroughs equipment and, with the latest version, on the DEC-VAX. The video graphics are programmed for DEC equipment, and the capability to run both systems on the same hardware provides more flexibility to the software.

In addition to improved production by the individual engineer and an improved product through optimization of the design, these systems distribute the work load between design offices when operated in a regional environment supported by a telecommunications network (Figures 9 and 10). This configuration permits design reviews from the controlling design office without the expense of time and travel between the respective operating sites. Thus design offices in high-cost areas, where the time available to professional engineers is limited because of other projects, can release their work to offices in low-cost areas and still retain control through video reviews. Projects let to consulting engineers for design have a contract requirement that the work be done on this system for the same reason. If for any reason the consultant defaults, the project can be completed with a minimum of interruption.

FUTURE DIRECTION

The systems described in this paper have been developed to meet the needs of a department that has an expanding work load, but has no provisions for expanding the work force. In such an environment improved production from available personnel is the only solution. Several enhancements, other than those discussed, are being developed to take advantage of new technology and to address some of the new problems that are constantly evolving.

Publication of this paper sponsored by Committee on Photogrammetry and Aerial Surveys

Prediction of the Sensitivity of Vehicle Dynamics to Highway Curve Geometrics by Using Computer Simulation

JOHN C. GLENNON, TIMOTHY R. NEUMAN, AND BRIAN G. McHENRY

The highway-vehicle-object-simulation model (HVOSM) was used to study vehicle responses and their sensitivity to various highway curve elements. After initial validation runs, a parameter study was conducted by using a simulated path of an inattentive driver. The highway curve and operational elements studied included vehicle speed, highway curve radius, length of curve, superelevation rate, superelevation runoff length, superelevation runoff distribution, presence of spiral transition, and presence of downgrade. The study revealed two significant results: (a) the dynamic response of vehicles traversing a highway curve is sensitive to speed; and (b) the addition of spiral transitions to highway curves dramatically reduces the friction demand of critical vehicle traversals.

One phase of the FHWA research project, "Effectiveness of Design Criteria for Geometric Elements," is presented in this paper. The project is a comprehensive effort aimed at quantifying aspects of highway design that have the greatest impact on highway safety.

The objectives of the research phase presented here are to

1. Demonstrate the applicability of the highway-vehicle-object-simulation model (HVOSM) as a tool for evaluating the dynamic response of vehicles traversing highway curves, and
2. Study the sensitivity of critical vehicle traversals to various highway curve design elements.

RESEARCH METHODOLOGY

HVOSM is a computerized mathematical model of a passenger vehicle that was originally developed by Cornell Aeronautical Laboratories and subsequently refined by Calspan Corporation (1). HVOSM simulates the dynamic response of a vehicle traversing a three-dimensional terrain configuration. The vehicle is composed of four rigid masses: sprung mass, unsprung masses of the left and right independent suspensions of the front wheels, and an unsprung mass that represents a solid rear-axle assembly.

The Roadside Design version of HVOSM currently available from FHWA is used in this study. Because the objective of these HVOSM tests is to study various dynamic response components, irrespective of available skid resistance, a high (0.8) friction factor was used. A 1971 Dodge Coronet was used as the test vehicle because it best represented the current population of passenger cars among the vehicles that have been modeled for HVOSM application. Certain modifications were made to perform the highway curve traversals and to interpret the appropriate dynamic responses:

1. Driver discomfort factor output,
2. Friction demand output,
3. Terrain table generator,
4. Driver model inputs (damping, steer velocity, steer initialization), and
5. Wagon-tongue path-following algorithm.

One significant aspect of the path-following algorithm is the length of the wagon-tongue or probe length. The wagon-tongue is attached to the center of gravity and extends in front of the vehicle parallel to its x-axis. A probe at the end of the wagon-tongue monitors the error from the intended path and activates the driver model inputs.

The probe length simulates the complex interaction that occurs as a driver sees the roadway ahead and responds to what is seen. Selection of a probe length, therefore, determines the type of driver being modeled. Very long probe lengths are indicative of ideal drivers who prepare for the curve well in advance. The resulting simulated behavior closely follows that described by the centripetal force equation, where the simulated vehicle path tracks almost exectly the center of the lane.

Moderate probe lengths create minor path corrections just preceding the curve and allow the vehicle to track in a near-optimum manner. Calculated friction values are somewhat higher than the values generated with the longer probe length.

Very short probe lengths represent either aggressive or inattentive driver behavior. Path corrections in response to the presence of the curve would come only as the vehicle actually enters the curve. The result is a dynamic overshoot at the beginning of the curve, where high lateral friction demand is generated by the vehicle, and the vehicle follows a distinctly noncircular path.

This discussion emphasizes the need to carefully define the driver behavior being modeled. Highly variable results can be obtained by running different probe lengths on the same simulated curve at the same speed.

INITIAL HVOSM STUDIES

Twelve initial HVOSM runs were made to demonstrate and verify that the model yields reasonable dynamic responses for curve traversals. These runs were made on unspiraled highway curves with the AASHO (2) superelevation runoff length distributed at 70 percent on tangent and 30 percent on curve. The idea was to select a long probe length that would allow the vehicle to track the center of the lane with little path deviation. The resulting vehicle dynamics given by HVOSM could then be compared with those predicted by the centripetal force equation.

The calculated and simulated dynamic responses for running the vehicle at design speed for the 12 test curves are given in Table 1. The calculated lateral acceleration ($V^2/127R$) and the simulated lateral acceleration are closely comparable for all tests. Also, the calculated tire responses $[(V^2/127R)-e]$ are comparable with the simulated tire responses.

It is interesting to note that, because of roll-angle, the driver discomfort factor (centrifugal acceleration acting on the driver) is always higher than the lateral acceleration on the tires. Therefore, the design f values in the AASHO process are not the centrifugal acceleration where the driver begins to feel discomfort, but represent the lateral friction on the tires that creates a higher threshold of driver discomfort.

Table 1. Initial HVOSM tests.

Speed (km/h)	Roadway Radius (m)	Super-elevation (m/m)	Calculated Lateral Acceleration ($V^2/127R$)	Calculated Tire Friction [($V^2/127R$)-e]	HVOSM Results: Maximum Lateral Acceleration	Maximum Tire Friction	Maximum Driver Discomfort Factor
33	33	0.08	0.25	0.17	0.25	0.17	0.20
33	39	0.04	0.21	0.17	0.20	0.14	0.18
50	70	0.10	0.26	0.16	0.26	0.17	0.20
50	83	0.06	0.22	0.16	0.22	0.17	0.20
67	143	0.08	0.23	0.15	0.23	0.16	0.18
67	175	0.04	0.19	0.15	0.19	0.16	0.19
83	198	0.10	0.26	0.16	0.27	0.17	0.21
83	259	0.06	0.20	0.14	0.20	0.14	0.18
100	368	0.08	0.20	0.12	0.22	0.10	0.15
100	466	0.04	0.16	0.12	0.18	0.12	0.16
117	499	0.10	0.20	0.10	0.20	0.11	0.12
117	635	0.06	0.16	0.10	0.16	0.11	0.13

NOMINALLY CRITICAL VEHICLE OPERATIONS

With HVOSM verified for use on curve traversals, the model appeared to be a reasonable tool for studying traversals where the vehicle does not precisely follow the center of the lane. The purpose of this exercise was to use HVOSM to study the sensitivity of vehicle dynamics to varying curve and operational parameters.

It was first necessary to define a nominally critical level of driver behavior. Less-critical or near average behavior would result in simulations that tend to mirror dynamics predicted by the centripetal force equation. On the other hand, highly critical levels may not produce realistic results, and thus may not provide a useful basis for comparing variable geometrics.

The selection of an appropriate level of criticality was based on previous vehicle operations research. Studies by Glennon (3) in Texas indicated that most drivers exceed the AASHO design f by a nominal amount, and that some exceed it greatly. The subject report revealed that maximum path curvature was related to highway curvature for various percentiles of the driving population. For purposes of this study, the 95th percentile path was selected to represent nominally critical operations. This relation is

$$R_v = 19{,}825R/(R + 23{,}096) \quad (1)$$

where R_v is the 95th percentile vehicle path radius (m) and R is the highway curve radius (m). By using this path, the critical f factors were calculated by substituting path curvature for highway curvature in the centripetal force equation for any design speed combination of highway curvature and superelevation.

With the establishment of the relation between highway curve parameters and nominally critical f factors, several preliminary HVOSM runs were made to select a probe length that best generated the intended critical operations. This probe length was

$$L = 0.25v \quad (2)$$

where L is the probe length (cm) and v is the forward velocity (cm/sec).

CRITICAL HVOSM STUDIES

With the probe length established, HVOSM could study the sensitivity of vehicle dynamics to various highway curve design and operational parameters under nominally critical path-following conditions. Of particular interest were

1. Vehicle speed,
2. Superelevation runoff length,
3. Superelevation runoff distribution,
4. Presence of spirals,
5. Length of spiral,
6. Presence of downgrade, and
7. Short curve lengths.

Twenty-four HVOSM runs were made by using six AASHO metricated curves. The results of these runs are given in Table 2. The conclusions from these studies are discussed below.

Vehicle Speed

The dynamic responses on any curve are sensitive to vehicle speed. Each of the six test highway curves was run at 20 km/h above design speed. The tire friction for this speed increment was quite sensitive for the lower design speed curves. For the 40 km/h design speed curve, the friction demand was simulated to be 0.52 compared with a design f of 0.16. These results could also be similarly predicted with the centripetal force equation (thus providing one more verification of the HVOSM methodology).

The implications of the test results for speed are significant. These implications suggest that an existing highway curve that is underdesigned for the prevailing operating speed could present a severe roadway hazard. This is particularly true for design speeds below 100 km/h. At these lower design speeds the actual vehicle operating speeds of 10 km/h or more above the curve design speed can be reasonably expected.

Superelevation Runoff Length

Superelevation runoff length was evaluated for 80 and 100 km/h design speeds by comparing the AASHO runoff length with one that was half as much. In this comparison the superelevation runoff length was distributed with 70 percent on the tangent and 30 percent on the curve.

The somewhat surprising result of these tests was that the shorter runoff length yielded smaller friction demands. The only identifiable explanation for this phenomenon is that the maximum friction demands take place in the initial part of the curve, where the shorter runoff length would provide slightly higher superelevation.

Superelevation Runoff Distribution

Superelevation runoff distribution was evaluated for 80 and 120 km/h highway curves that have AASHO

Table 2. Critical HVOSM tests.

Curve Radius (m)	Test Parameters: Maximum Super-elevation (m/m)	Curve Design Speed (km/h)	Length of Superelevation Runoff (m)	Maximum Superelevation on Tangent (%)	Presence and Length of Spiral	Percent Grade	Operating Speed of Test Vehicle (km/h)	Results: AASHO Design f	HVOSM f
750	0.06	120	61	70	None	0	140	0.092	0.190
750	0.06	120	61	70	None	0	120	0.092	0.150
600	0.10	120	92	70	None	0	140	0.092	0.230
600	0.10	120	92	70	None	0	120	0.092	0.160
600	0.10	120	92	20	None	0	120	0.092	0.190
600	0.10	120	50	70	None	0	120	0.092	0.120
410	0.08	100	66	70	None	0	120	0.116	0.260
410	0.08	100	66	70	None	0	100	0.116	0.170
410	0.08	100	33	70	None	0	100	0.116	0.140
410	0.08	100	66	NA	AASHO	0	100	0.116	0.100
210	0.10	80	72	70	None	0	100	0.140	0.390
210	0.10	80	72	70	None	0	80	0.140	0.240
210	0.10	80	72	20	None	0	80	0.140	0.260
210	0.10	80	72	70	None	5	80	0.140	0.240
210	0.10	80	72	NA	AASHO	0	80	0.140	0.120
210[a]	0.10	80	72	70	None	0	80	0.140	0.200
210	0.10	80	72	20	None	5	100	0.140	0.430
130	0.08	60	50	70	None	0	80	0.152	0.400
130	0.08	60	50	70	None	0	60	0.152	0.200
130	0.08	60	50	70	None	5	60	0.152	0.210
130	0.08	60	50	NA	AASHO	0	60	0.152	0.120
50	0.10	40	50	70	None	0	60	0.164	0.520
50	0.10	40	50	70	None	0	40	0.164	0.200
50	0.10	40	50	70	None	5	40	0.164	0.200

Note: NA = not available.
[a]50-m curve length.

superelevation runoff lengths with 70-30 and 20-80 distributions.

As expected, the 70-30 distribution, where most of the superelevation transition is provided on the tangent, produces somewhat smaller friction demands. The differences can be explained almost entirely by the difference in superelevation in the initial part of the curve where the maximum friction demand is generated.

Presence of Spirals

The presence of spirals was evaluated for highway curves with design speeds between 60 and 100 km/h. The comparison was between highway curves with and without AASHO spirals.

This comparison provides the most dramatic results of the study. In all cases the presence of the spiral reduced the friction demand from a value significantly higher than the design f to one that was below the design f.

The reason for this dramatic result is readily evident. For the driver who is inattentive or for some other reason has limited notice of the upcoming curve, the spiral not only reduces the driver's absolute path error over time but requires less severe steering to correct for the desired path because the path of a spiral is less severe than the path of a circular curve.

Length of Spiral

Although the initial plan was to test a spiral that was twice the length of an AASHO spiral, this plan was not carried through after obtaining the dramatic results for the presence of AASHO spirals.

Presence of Downgrade

The presence of downgrades was evaluated for highway curve design speeds between 40 and 80 km/h. In comparing a 5 percent downgrade with level terrain, no difference was found in the friction demand.

Short Curve Lengths

Short curve lengths were evaluated by examining the difference between vehicular response to the approach to a curve (i.e., the dynamics of proceeding from tangent to curve) and the response to a very short curve. Short curves require continuous responses by the driver as he moves in and then out of the curve. A 50-m curve length of an 80 km/h design curve was selected for analysis.

The results of this test indicate that the short curve does not generate any greater dynamic responses than those found on a longer curve that has the same radius and vehicle speed.

SUMMARY

The critical analysis of highway curve design parameters provided two significant results. First, the dynamic response of vehicles traversing a highway curve are extremely sensitive to speed. This implies that existing highway curves that are severely underdesigned for prevailing highway speeds may present serious roadway hazards.

Second, it is significant to note that the addition of spiral transitions to highway curves dramatically reduces the friction demand of critical vehicle traversals.

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Publication of this paper sponsored by Committee on Geometric Design.

Rehabilitation of Existing Freeway-Arterial Highway Interchanges

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The results of a study of interchange rehabilitation projects at freeway-arterial interchanges are presented. The study included the evaluation of 40 recent interchange rehabilitation projects and the development of recommended design procedures and guidelines for interchange rehabilitation projects. The significance of arterial crossroads and crossroad ramp terminals as a source of traffic operational and safety problems at freeway-arterial interchanges is emphasized. An analysis of the safety effectiveness of interchange rehabilitation projects was conducted, and safety measures of effectiveness for major and minor geometric modifications are reported. An overview of the recommended design procedures for interchange rehabilitation projects is also presented. The recommended procedures address six stages of the design process: identify interchanges with operational or safety problems, study problem locations and identify specific deficiencies, identify improvement alternatives, quantify the effects of the improvement alternative, evaluate the alternatives and select the best one, and implement the improvement and evaluate its effectiveness.

Freeways are the safest and most efficient portion of the American highway system because of the complete exclusion of driveways and at-grade intersections. Access from the conventional highway system to the freeway system is provided at freeway-arterial interchanges. Because of the conflicting demands of entering, exiting, and through traffic, most operational and safety problems of the freeway system are concentrated at interchanges.

Redesign of inadequate interchanges can result in increased capacity, reduced delay, and increased safety. However, the cost to remedy all existing traffic operational and safety problems far exceeds the funds available for improvements, and improvement needs are growing. Therefore, the use of cost-effectiveness techniques is vital to assure that the limited funds available are invested optimally.

The results of a multiyear study of procedures and guidelines for the rehabilitation of freeway-arterial highway interchanges are presented in this paper. The study included the evaluation of recent interchange rehabilitation projects constructed by several state highway agencies and the recommendation of design procedures for interchange rehabilitation projects.

The scope of the study was limited to consideration of freeway-arterial interchanges in urban and suburban areas. The term freeway-arterial interchange was interpreted as referring to all interchanges between a freeway and a street or highway with no control or partial control of access. Freeway-freeway interchanges and rural interchanges were specifically excluded from the project. Nevertheless, it is recognized that many of the results from this project are applicable to freeway-freeway interchanges, freeway-arterial interchanges, rural interchanges, and urban and suburban interchanges.

The final results of the study included traffic operational and safety evaluations of 40 recent interchange rehabilitation projects, including general findings concerning their safety effectiveness. The study also developed design procedures for each step in the interchange rehabilitation process, from identification of interchanges that have operational and safety problems through the evaluation of completed interchange rehabilitation projects. Two aspects of the study findings are focused on in this paper: the results of the safety effectiveness evaluations and the recommended procedures for the design of interchange rehabilitation projects. In regard to the latter area, particular emphasis is placed on a series of charts developed to relate particular traffic operational problems to potential solutions.

A more detailed presentation of the results of the study is given in the four-volume final report (1-4) published by FHWA. [All four volumes, including an extensive presentation of interchange rehabilitation case studies in Volume III, are available to interested users through the National Technical Information Service (NTIS).]

EVALUATION OF INTERCHANGE REHABILITATION PROJECTS

One major activity during the study was the collection of data on 40 recent interchange rehabilitation projects. These data were used to evaluate the operational and safety effectiveness of each project. The interchange rehabilitation project evaluations were useful to the study because they (a) reveal the broad range of operational and geometric conditions under which interchange rehabilitation projects are considered; (b) provide measures of operational and safety effectiveness that can be used to estimate the effectiveness of similar projects; and (c) provide practical insight into the problems and data limitations encountered by highway agencies in considering interchange rehabilitation decisions.

The evaluation of each interchange rehabilitation project was developed in the form of a case study of approximately 5 to 10 pages in length. [The case studies are too voluminous to present in this paper; see Volume III of the final report (3).]

Selection of Interchange Rehabilitation Projects

The first step in selecting interchange rehabilitation projects for study was to identify several state highway agencies as potential sources of data and to obtain their cooperation. Cooperating states were chosen on the basis of (a) extensive urban freeway systems that were likely to include an adequate sample of interchange rehabilitation projects; (b) geographical distribution between major regions of the United States; and (c) interest in the goals and objectives of this study. The five states selected were California, Florida, Illinois, Michigan, and New York.

Several interchange rehabilitation projects were selected in each state to represent the types of projects that have been accomplished in that state, including projects for a variety of interchange configurations. The interchange configurations of primary interest to the study were

1. Full diamond,
2. Full diamond with frontage road slip ramps,
3. Partial cloverleaf,
4. Full cloverleaf, and
5. Full cloverleaf with collector-distributor roads.

In accordance with the scope of the study, only rehabilitation projects at freeway-arterial inter-

Table 1. Distribution of 40 interchange rehabilitation projects.

Item	No.	Percentage of Total	Item	No.	Percentage of Total
State			Type of rehabilitation project		
California	8	20.0	Major geometric modification	12	30.0
Florida	8	20.0	Minor geometric modification	28	70.0
Illinois	8	20.0	Portion of interchange modified		
Michigan	9	22.5	Crossroad	31	77.5
New York	7	17.5	Crossroad ramp terminal	34	85.0
Metropolitan area population			Ramp	31	77.5
>3,000,000	16	40.0	Mainline ramp terminal	19	47.5
1,000,000-3,000,000	5	12.5	Mainline freeway	5	12.5
500,000-1,000,000	8	20.0	Project cost[b]		
150,000-500,000	6	15.0	>$2,000,000	9	22.5
<150,000	5	12.5	$1,000,000-$2,000,000	3	7.5
Location within metropolitan area			$500,000-$1,000,000	5	12.5
Central city	9	22.5	$200,000-$500,000	14	35.0
Suburban community	31	77.5	$100,000-$200,000	5	12.5
Mainline freeway ADT[a]			<$100,000	4	10.0
>100,000	3	7.5	Arterial crossroad ADT[a]		
75,000-100,000	6	15.0	>50,000	1	2.5
50,000-75,000	13	32.5	40,000-50,000	4	10.0
25,000-50,000	10	25.0	30,000-40,000	7	17.5
<25,000	8	20.0	20,000-30,000	6	15.0
Interchange configuration			10,000-20,000	16	40.0
Full diamond	20	50.0	<10,000	5	12.5
Partial cloverleaf	8	20.0	Unknown	1	2.5
Full cloverleaf	9	22.5			
Other	3	7.5			

Note: ADT = average daily traffic.

[a]Before period. [b]Cost is not in constant dollars.

changes and projects in urban and suburban areas were considered. (In two cases rehabilitation projects at arterial-arterial interchanges were selected because the operational and safety problems identified and geometric design alternatives considered were equally applicable to freeway-arterial interchanges.) The selected projects ranged from minor geometric improvements (such as increasing the length of speed-change lanes, reducing horizontal curvature, and so on) to major geometric improvements (such as installation of collector-distributor roads or some other change in the basic interchange configuration).

An important criterion in selecting projects was the construction date of the improvement. There was an attempt to select projects that were old enough for adequate after-period data (e.g., at least 1 yr of accident data after project completion) to be available, but not so old that accident data, traffic volume data, or project records from before the construction had been destroyed. These requirements were restrictive, and inevitably compromises in the requirements of data availability had to be made in order to locate enough projects.

The data in Table 1 summarize the distribution of selected characteristics for the 40 interchange rehabilitation projects. These data indicate that the goal of selecting a wide variety of interchange rehabilitation projects was met; they also reveal the types of interchange rehabilitation projects that are actually being constructed by state highway agencies.

The data in Table 2 summarize the interchange configuration and type of geometric or traffic control modification for each rehabilitation project. Also identified in the table is the portion(s) of each interchange modified as part of each project. The data in this table underscore the important relation of the arterial crossroad to operational and safety problems of freeway-arterial interchanges; 34 of the 40 projects involved modification of the crossroad ramp terminals, whereas 31 projects involved adjacent portions of the crossroad and 31 projects involved adjacent portions of the ramps. By contrast, only 19 projects involved the main line freeway ramp terminals and only 5 projects involved the main line freeway itself.

Data Collection

After the selection of the 40 interchange rehabilitation projects, the next activity was to collect physical, operational, safety, and cost data about each project. Two forms of data were collected: office documents concerning each interchange and its associated improvement project, and field data at the interchange site. Most of the data collection depended on office documents because it was not feasible to collect data at each interchange before, during, and after construction.

The office documents obtained for each study interchange included

1. Construction plans showing geometrics before and after the improvement;
2. Project reports and file memoranda justifying the need for the project and the improvement alternatives considered;
3. ADT, hourly traffic counts, turning movement counts, and capacity analyses, where available;
4. Detailed accident data from before and after the improvement;
5. Starting and completion dates of improvement construction; and
6. Construction costs of the improvement.

The field data collected on a site visit to each interchange included site photographs, observation of daytime traffic operations, observation of nighttime traffic operations, and operational studies.

Operational Evaluations

An evaluation of the effect of each interchange rehabilitation project on traffic service was conducted. The operational evaluation focused only on the portion of the interchange that was improved. Whenever possible, explicit use was made of traffic volume data obtained from the cooperating highway agencies to evaluate peak-hour levels of service before and after the improvement. Estimates of missing data were made in some cases where a reasonable basis for judgment was available.

The most frequent type of operational improvement involved crossroad ramp terminals. In most cases detailed signal phasing and timing data were not available to apply the level-of-service evaluation procedures of the Highway Capacity Manual (5). Therefore, operational evaluations of crossroad ramp terminals were based on a critical movement analysis. The specific critical movement analysis technique used was that of McInerney and Peterson (6). More recently, a similar technique was presented in Transportation Research Circular 212 (7) as a potential addition to the Highway Capacity Manual. Operational evaluations of main line freeway segments and weaving areas were conducted by using the procedures in the Highway Capacity Manual.

The results of the operational analyses were presented in case study evaluations of each interchange rehabilitation project. No attempt has been made to summarize the operational effects of the 40 interchange rehabilitation projects in this paper because these results are only meaningful in relation to the specific geometric and traffic conditions and the specific nature of the improvement project at each interchange under study.

Safety Evaluations

A safety evaluation that used accident data was performed for each of the 40 interchange improvement projects. The objective of this analysis was to determine the accident-reduction effectiveness of each project and to estimate the accident-reduction effectiveness of general classes of interchange rehabilitation projects. The following sections identify the procedures followed to collect accident data, the measures of effectiveness used, and the statistical analysis procedures employed. A summary of the results of the safety analysis is also presented.

Accident Data Collection

Accident data were collected for two periods at each

Table 2. Portion of interchange modified in 40 interchange rehabilitation projects.

Modification	Interchange No.	Portion of Interchange: Crossroad	Portion of Interchange: Crossroad Ramp Terminal	Ramp	Modified Main Line Ramp Terminal	Main Line Freeway
Full Diamonds						
Major geometric	1	X	X	X	X	
	2	X	X	X	X	X
	3		X	X	X	
	4	X	X	X	X	
	5	X	X	X	X	X
Minor ramp	6		X	X		
	7	X	X	X		
	8	X	X	X	X	
	9	X	X	X		
	10		X	X		
	11		X	X		
Minor crossroad	12	X	X			
	13	X	X			
	14	X	X			
Minor ramp and crossroad	15	X	X	X		
	16	X	X	X		
	17	X	X	X	X	
	18	X	X	X		
	19	X	X	X		
	20		X	X		
Full Cloverleafs						
Major geometric	21	X	X	X		
	22	X	X	X	X	
Minor ramp and C-D road	23			X	X	X
	24				X	
Minor ramp and crossroad	25	X	X	X		
	26	X	X	X		
	27	X				
	28	X				
	29	X	X			
Partial Cloverleafs						
Major geometric	30	X	X	X	X	
	31	X	X	X	X	X
	32	X	X	X	X	
	33	X	X	X	X	X
	34	X	X	X	X	
Minor ramp and crossroad	35			X	X	
	36	X	X	X		
	37	X	X			
Other Interchange Configurations						
Minor ramp and crossroad	38	X	X	X	X	
	39				X	
	40	X	X	X	X	
	Total	31	34	31	19	5

Note: C-D road = collector-distributor road.

interchange: one period before and one period after construction of the improvement project. A key piece of information needed to establish the before-and-after study periods was the construction starting and completion dates of the interchange projects. In general, the before period was a 3-yr period immediately preceding the beginning of construction and the after period was a 3-yr period immediately following the completion of construction. However, in many cases it was necessary to use study periods shorter than 3 yr. For example, the termination date of each after period was controlled by the most recent date of accident data available from the state, and the result was that some after periods were shorter than 3 yr. Similarly, some before periods were shorter than 3 yr when older accident data were no longer available from the state. In a few cases the before period was longer than 3 yr when additional data were available from the state.

Location boundaries for the interchange improvement projects were established by the milepost system used in each state. Data were obtained at each interchange for the main line freeway, the crossroad, and all ramps in the interchange area. The limits for data requests on the main line freeway and crossroad were at least 1,000 ft beyond the ramp terminal or gore point on each approach to the interchange. In some cases, where the arterial crossroad was not under state jurisdiction, accident data on the crossroad were not available for a full 1,000 ft outside of the ramp terminals.

All available accident data were obtained for each interchange, but only part of the interchange was analyzed in some cases. The objective of this study was to evaluate only that portion of the interchange most directly affected by the improvement. For example, an off-ramp widening project could directly affect the accident experience on the ramp itself, at the crossroad ramp terminal, or on the main line freeway upstream of the gore area. On the other hand, a major reconstruction project could directly affect the accident experience of the entire interchange, including main line freeway, crossroad, and all ramps.

Safety Measures

The primary safety measure for each interchange was the accident rate for the entire interchange or that portion of the interchange directly affected by the improvement. The accident rate was determined by the following equation:

$$AR = N(10^6)/[(365)(ADT)] \tag{1}$$

where

- AR = accident rate per million vehicles,
- N = accident frequency (accidents per year),
- ADT = average daily traffic volume, and
- 365 = number of days per year.

For the analysis of an entire interchange, the variable ADT was the sum of the daily volumes entering the interchange on each main line and crossroad approach (or the sum of the two-way traffic volumes divided by 2). For the analysis of a segment or spot location, such as a single ramp, the variable ADT represented the ADT volume on the segment or passing the spot (e.g., the ramp volume).

The use of the accident rate rather than the accident frequency as the primary measure of effectiveness provides a method to account for changes in traffic volume at the interchange. For example, consider an interchange with 10 accidents/yr before an improvement and 10 accidents/yr after the improvement. An analysis of accident frequency alone would conclude that the improvement project had no effect. However, if the traffic volume entering the interchange increased from 25,000 to 50,000 vehicles/day during the same period, the accident rates before and after the improvement would be 1.10 and 0.55 accidents/million vehicles, respectively, which indicates a 50 percent decrease in the accident rate.

Accident rate and accident frequency data were used in formal statistical analyses, which are described below. Collision diagrams of the accident experience before and after the improvement were also prepared. These collision diagrams permitted an informal evaluation of the types and patterns of accidents at the interchange. These informal evaluations of accident patterns were then used to suggest hypotheses about the project effects that were tested in the formal statistical analyses.

Statistical Analyses

Two statistical techniques were used in the safety evaluations: t-tests and analyses of variance. The t-test is used to compare two sets of data (such as the accident rates before and after the improvement) in order to identify statistically significant differences. Thus the t-test can identify the effect of one factor (e.g., time) with two levels (before and after). Analysis of variance is used to investigate the effects and interactions of two or more factors, each of which may have multiple levels.

Before describing the analysis results, a few words need to be said about the terminology used in statistics, particularly about statistically significant differences. A difference in the mean accident rate before and after an improvement is statistically significant if there is reasonable confidence that it is too large to have occurred due to random variation alone. In such a situation the observed difference in accident rate is presumed to result from the improvement project. A difference that is not statistically significant is small enough so that it could have resulted from random variation alone. When this occurs there are no statistical grounds to presume that the improvement was effective. However, this does not necessarily mean that the project was not effective; rather it means that it cannot be proved that the improvement was effective at some specified level of confidence.

It must be recognized that the statistical analyses of individual interchange improvement projects were performed on relatively limited data sets. Statistical methods are not particularly well suited to small data sets. Two courses of action are available to increase the likelihood of detecting statistically significant effects: (a) use decreased confidence levels, and (b) combine data from similar projects into a single analysis. The latter procedure is generally preferred, and the results of some combined analyses are reported below. All statistical conclusions in this study are reported at a 95 percent level of confidence unless otherwise stated.

Summary of Safety Analysis Results

The results of the safety analyses for 37 of the 40 interchange rehabilitation projects for which adequate data were available are given in Table 3. The interchange configuration and the portion of each interchange modified have been identified previously in Table 2. The data in Table 3 give the observed percentage reduction (or increase) in accident rate and whether or not the reduction (or increase) was found to be statistically significant. The analyses

Table 3. Summary of results of the safety evaluation.

Modification	Interchange No.	Observed Percentage Reduction in Accident Rate	Statistically Significant at 95 Percent Confidence Level
Full Diamonds			
Major geometric	1	1.9	No
	2	29.5	No
	3	36.2	Yes
	4	44.9	No
	5	-9.1	No
	Combined	20.7	No
Minor ramp	6	38.2	No
	7	51.9	Yes
	8	28.4	No
	9	81.7	No
	10	-48.1[a]	No
	11	39.8	Yes
	Combined	32.0	Yes
Minor crossroad	12	78.2	Yes
	13	12.7	No
	14	8.3	No
	Combined	33.1	Yes
Minor ramp and crossroad	15	-26.3	No
	16	-23.1	No
	17	–	–
	18	46.3	Yes[b]
	19	51.7	Yes[b]
	20	57.4	Yes
	Combined	21.2	Yes
Full Cloverleafs			
Major geometric	21	5.2	No
	22	-16.7	No
	Combined	-11.5	No
Minor ramp and C-D road	23	7.5	No
	24	-119.0	Yes[b,c]
	Combined	-55.8	No
Full Cloverleafs (continued)			
Minor ramp and crossroad	25	2.6	No
	26	-88.0	Yes[c]
	27	47.4	Yes
	28	6.8[d]	No
	29	–	–
	Combined	-7.8	No
Partial Cloverleafs			
Major geometric	30	39.7	Yes
	31	21.9	No
	32	51.0	Yes
	33	24.0	No
	34	55.5	Yes
	Combined	38.4	Yes
Minor ramp and crossroad	35	73.5	Yes
	36	37.8[d]	Yes
	37	25.2	No
	Combined	45.5	Yes
Other Interchange Configurations			
Minor ramp and crossroad	38	24.9	No
	39	-8.6	No
	40	–	–
	Combined	8.2	No
Summary by Project Type			
Major geometric		23.7	Yes
Minor ramp and crossroad		16.3	Yes
All projects		18.7	Yes

Note: C-D road = collector-distributor road.

[a] Percentage reduction in accident frequency.
[b] Significant at 90 percent confidence level.
[c] Significant increase in accident rate.
[d] Percentage reduction in accident frequency adjusted for changes in traffic volume.

revealed statistically significant reductions in accident rates for 13 projects, significant increases in accident rates for 2 projects, and no significant change in accident rates for 22 projects.

For the 13 projects that had statistically significant reductions in accident rates, the observed reductions in the accident rate ranged between 36.2 and 78.2 percent. For the 22 projects that did not have significant changes in accident rates, the observed changes in accident rates ranged between a 48.1 percent increase to an 81.7 percent decrease.

Several analyses of combinations of projects were conducted and are also reported in Table 3. Projects that had similar interchange configurations and improvement types were combined into 10 categories. A two-way analysis of variance with the factors time (before versus after) and interchange was performed for each category. The results of these analyses, which are also reported in Table 3, revealed a statistically significant reduction in accident rates in five of the categories and no significant change in accident rates in the other five.

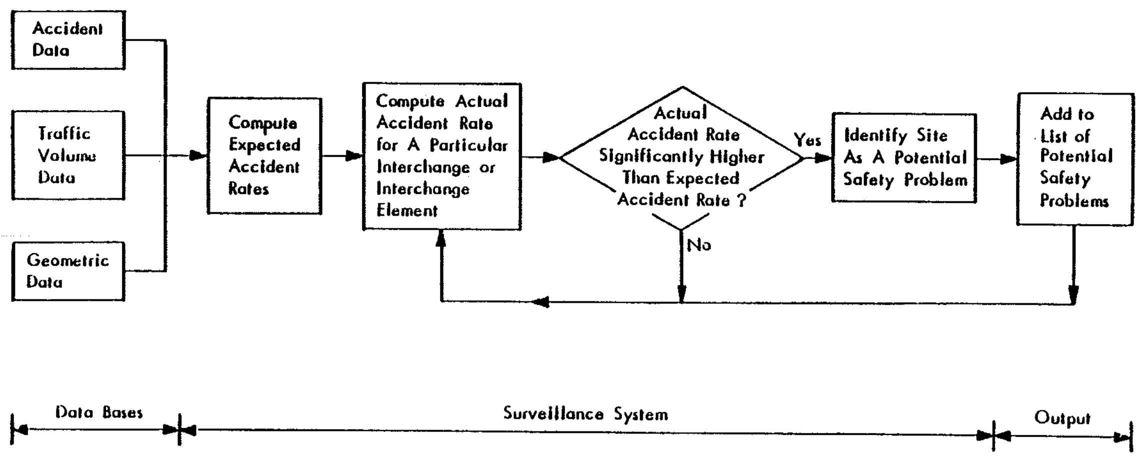
Each improvement project was then classified in one of two categories--major geometric modifications or minor ramp and crossroad modifications--without regard to interchange configuration. The reductions in accident rates for both of these categories and for all 37 interchanges combined were statistically significant. The average major geometric modification project resulted in a 23.7 percent reduction in the accident rate. The average minor ramp and crossroad modification project resulted in a 16.3 percent reduction in the accident rate. The average reduction in the accident rate for all projects combined was 18.7 percent. These results indicate that, on average, interchange rehabilitation projects are successful in improving safety, and that these results may be useful in predicting the accident-reduction effectiveness of other interchange rehabilitation projects.

In summary, it is difficult to demonstrate that interchange rehabilitation projects are effective in reducing accident rates when the projects are studied one at a time. More successful results at demonstrating this effectiveness were indicated by combining projects of a similar nature for the same interchange configuration. The reason for these difficulties is the high variance of accident data. Nevertheless, it has been established that interchange rehabilitation projects, taken as a group, are effective in reducing accident experience. Furthermore, the overall analysis has a statistically significant time-interchange interaction, which implies that some projects are more effective than others.

DESIGN PROCEDURES FOR INTERCHANGE REHABILITATION PROJECTS

Another important objective of the research reported in this paper was to recommend design procedures for interchange rehabilitation in order to provide guidance to engineers responsible for identifying problem interchanges and designing interchange rehabili-

Figure 1. Flowchart of an accident surveillance system.



tation projects. The organization of the recommended design procedures is based on the interchange rehabilitation process, which consists of six steps:

1. Identify interchanges with operational or safety problems,
2. Study problem locations and identify specific deficiencies,
3. Identify improvement alternatives,
4. Quantify effects of improvement alternatives,
5. Evaluate alternatives and select the best one, and
6. Implement improvement and evaluate its effectiveness.

The recommended procedures for each step in the interchange rehabilitation process are briefly summarized in the following sections. A complete presentation of the recommended procedures is given in Volume II of the final report (2).

Identify Interchanges with Operational or Safety Problems

The first step of the interchange rehabilitation process is to identify interchanges that have traffic operational or safety problems that are potentially correctable. The recommended approach is not a formal procedure; instead the approach provides guidance on the use of accident surveillance systems and operational data to prepare a list of candidate interchanges for further study.

Review procedures for the identification of traffic operational problems at interchanges should rely on field reviews of operational conditions, traffic volume counts, capacity analyses, and citizen complaints. Greater emphasis should be placed on a formal surveillance system for identifying safety problems rather than operational problems, because safety problems are often more subtle and difficult to detect.

An appropriate organization for a surveillance system is shown in Figure 1. The effective use of an accident surveillance system for interchanges requires complete accident data for all portions of each interchange (freeway, crossroad, and ramps); a location reference system to identify the portion of the interchange where each accident occurs; estimates of the expected accident rates for an entire interchange or for specific portions of an interchange; and statistical criteria for comparison of the actual and expected accident rates. Excellent examples of surveillance systems applied to accident problems in interchange areas can be found in reports from the California Department of Transportation (8) and the Michigan Department of State Highways and Transportation (9,10).

Study Problem Locations and Identify Specific Deficiencies

Engineering studies are the basic tool for identifying specific deficiencies at problem locations. A set of basic studies, which includes physical inventories, on-site observation, traffic volume counts, accident tabulations and summaries, and collision diagrams, are recommended for the evaluation of each problem location. Supplementary engineering studies, which are suitable for investigating specific types of operational and safety problems, should also be used at appropriate locations. Such supplementary studies could include evaluations of capacity, travel time and delay, speed, traffic conflicts and erratic maneuvers, traffic signals, sight distance, turning radius, and skid resistance.

Identify Improvement Alternatives

A critical step in the interchange rehabilitation process is the identification of alternative solutions. All feasible alternative solutions must be considered by the engineer, if the best alternative is to be selected. Nine charts have been developed to relate identified operational and safety problems to potential solutions. These charts address the following common traffic operational and safety problems at freeway-arterial interchanges:

1. Delays and accidents on off-ramps,
2. Delays and accidents on on-ramps,
3. Delays and accidents on arterial crossroads,
4. Single-vehicle accidents on ramps,
5. Head-on accidents on two-way ramps,
6. Merging accidents at entrance to on-ramps,
7. Wrong-way accidents,
8. Delays and accidents within main line freeway weaving areas, and
9. Delays and accidents between ramp terminals and adjacent intersections.

An example of one of the charts, which gives solutions that are appropriate for interchanges with delays and accidents on off-ramps, is shown in Figure 2. By using the charts a design engineer can quickly identify a set of solutions that are potentially applicable to the particular interchange configuration and problem under consideration. Any additional solutions developed by the designer should also be considered.

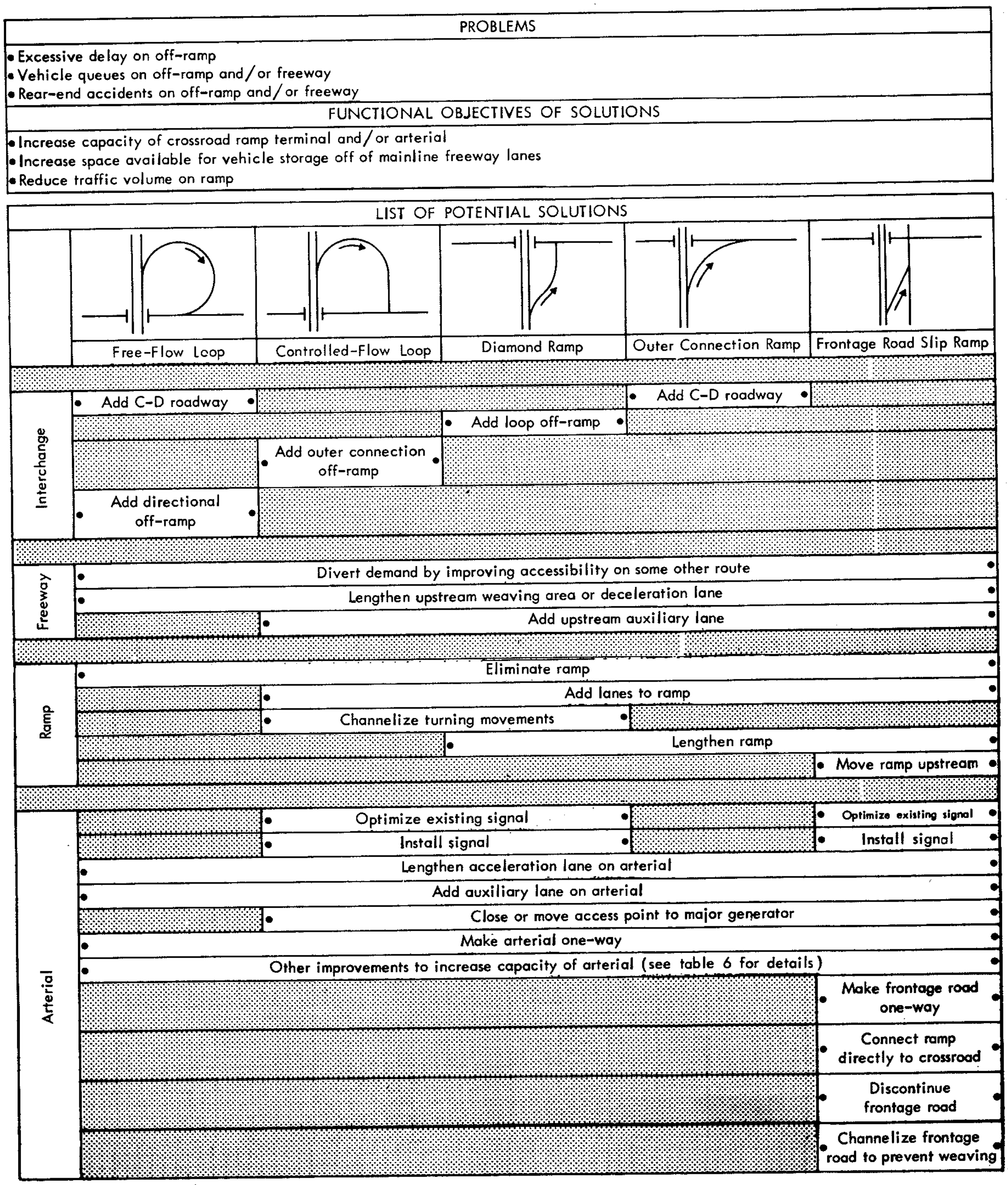
Quantify Effects of Improvement Alternatives

Procedures are provided to quantify the effects of improvement alternatives on travel time, vehicle operating costs, and accidents. Travel time and vehicle operating costs are quantified through the procedures of the AASHTO Manual on User Benefit Analysis for Highway and Bus Transit Improvements--1977 (11), which have been adapted to specifically address the analysis of interchanges. The AASHTO procedures provide measures of both delay and vehicle running costs (including energy consumption) for most, but not all, interchange rehabilitation situations. In particular, the procedures do not address vehicle speeds and delays in weaving areas, either within an interchange or between adjacent interchanges. Also, delay measures for level-of-service F conditions on roadway sections and at intersections are not included in the procedures and must be obtained from field studies or queuing analyses.

Safety effectiveness estimates can be based on any of five alternative evaluation approaches:

1. Estimate safety effectiveness based on the agency's own experience in similar projects,

Figure 2. Potential improvements for interchanges that have delays and accidents on off-ramps.



2. Estimate safety effectiveness based on reported experience in similar projects,
3. Estimate safety effectiveness based on engineering judgment,
4. Assume that the improvement will reduce the accident rate to the statewide average determined by the agency, and
5. Assume that the improvement will reduce the accident rate to an average accident rate determined from the literature.

Each of these approaches has merit and could be the most suitable approach in a particular situation. Therefore, all five approaches have been retained, and the choice of the most suitable approach in any particular situation is left to the designer.

The potential significance of air pollution and noise analyses to some interchange rehabilitation decisions is stressed in the recommended procedures, and it is recognized that other factors, which cannot be quantified, will also influence interchange rehabilitation decisions.

Evaluate Alternatives and Select the Best One

The guidelines for the evaluation of alternatives encourage the use of analytical techniques to compare alternatives, although it is recognized that the choice between alternatives rests heavily on the engineer's judgment. The net return method--a conventional engineering economic analysis technique--is recommended to examine trade-offs between factors that can be quantified in monetary terms. The net return for ranking alternatives relative to a base condition (usually the existing condition or do-nothing alternative) can be computed as

$$\text{Net return} = (TC_B - TC_A) + (RC_B - RC_A) + (AC_B - AC_A) + (OC_B - OC_A) - (CC)(CRF, i\%, n) \quad (2)$$

where

TC = cost of travel time (delay) per year,
RC = vehicle operating cost per year,
AC = accident cost per year,
OC = other costs per year,
CC = improvement construction cost, and
(CRF, i%, n) = capital recovery factor for n years at i percent interest.

The subscript B refers to the base condition and the subscript A refers to the improved condition for the alternative under consideration. All cost elements in the equation for net return are expressed in dollars.

Although an economic evaluation of the type presented here provides an important input to the decision-making process, it cannot be the sole basis for a decision. For example, some important factors cannot be quantified in monetary terms, and many cannot be quantified at all. Nevertheless, the economic analysis can contribute to making design decisions as objective as possible, but the complete consideration of all factors that affect a design decision remains a subjective process that must rely heavily on the judgment of experienced engineers. Because of the important role of engineering judgment in considering design trade-offs, it is vital that the anticipated effects be presented in a formal report or memorandum to document the basis on which such judgments were made. This could take the form of a list of advantages and disadvantages of each alternative and the designer's assessment of the merits of each alternative relative to its construction cost and the budget constraints on the project.

Implement Improvement and Evaluate Its Effectiveness

The final step of the interchange rehabilitation process is to implement the selected improvement project and, subsequently, to evaluate its effectiveness. The objective of an effectiveness evaluation is to compare the actual effects of the project with its predicted effects. Two simple statistical procedures for safety effectiveness evaluations are recommended: a graphical procedure based on the chi-square test and an analytical procedure based on the two-sample t-test. Feedback from the evaluation of completed projects will enable the anticipated effects of planned projects to be more accurately quantified in the future.

CONCLUSIONS AND RECOMMENDATIONS

It is apparent that there is a continuing need to upgrade freeway-arterial highway interchanges in order to alleviate the traffic operational and safety problems that develop. The review of typical interchange rehabilitation projects that was performed in this study indicates the important role of the arterial crossroad and the crossroad ramp terminals in operational and safety problems at freeway-arterial interchanges. It was found that 85 percent of the projects studied involved an improvement to the crossroad ramp terminal, although the frequency of projects that involved freeway ramp terminals or the freeway itself was much lower.

The study results provide some useful safety measures of effectiveness that demonstrate that, on average, interchange rehabilitation projects are successful in reducing accident rates. It was revealed that the average major geometric modification project resulted in a 23.7 percent reduction in the accident rate, whereas the average minor ramp or crossroad modification project resulted in a 16.3 percent reduction in the accident rate. It is worth noting that the major geometric modification projects not only resulted in a larger precentage reduction in the accident rate, but they also produced that reduction throughout the entire portion of the interchange rather than at only one ramp or ramp terminal. It should also be noted that the major geometric modification projects cost much more than the minor geometric modifications.

Finally, the study reported here resulted in a series of recommendations to improve the interchange rehabilitation design process. Specific recommendations were made for each stage of the design process, ranging from identification of interchanges with traffic operational and safety problems to evaluation of completed projects. The specific results that will be of direct assistance to designers include a series of charts that can be used to identify a range of potential solutions for specific traffic operational and safety problems at freeway-arterial interchanges.

ACKNOWLEDGMENT

The study reported in this paper was performed by Midwest Research Institute under contract to the FHWA.

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Publication of this paper sponsored by Committee on Geometric Design.

Notice: The findings and conclusions presented in this paper are the authors' and do not necessarily represent the views of the FHWA.

Cost-Effectiveness of Improvements to Stopping-Sight-Distance Safety Problems

TIMOTHY R. NEUMAN AND JOHN C. GLENNON

Stopping-sight distance (SSD) is one of the most significant design features of highways. The treatment of locations that have deficient sight distance is generally costly and difficult because most such deficiencies involve basic problems with horizontal or vertical geometry. An attempt was made to systematically evaluate the cost-effectiveness of spot improvements of SSD-deficient locations. A framework was established for classifying the accident potential of such locations. Countermeasures to treat sight-distance problems on both rural and urban highways were proposed. Their implementation costs and hypothetical safety benefits were evaluated in order to discover any potential cost-effective improvements. Despite conservative assumptions about safety, the research indicated that only relatively inexpensive countermeasures hold the potential for cost-effectiveness. A greater potential for cost-effectiveness was indicated when improvements are made in conjunction with a planned rehabilitation or reconstruction program.

Safe stopping-sight distance (SSD) is one of the most significant design features of highways. Potentially serious safety problems are created by short vertical curves or by roadside obstructions on the inside of sharp horizontal curves. Although it is easy to identify highway sections with deficient SSD, treatment of the problem is difficult. Most sight-distance deficiencies are created by geometric deficiencies that are costly to correct.

A cost-effectiveness analysis for treating existing sight-distance deficiencies is presented in this paper. A rational approach was used that relied on published accident research and knowledge gained from an FHWA research contract, "Effectiveness of Design Criteria for Geometric Elements." A framework was established to classify SSD problems by their potential impacts on safety.

Countermeasures to treat sight-distance problems were developed, and estimates of their effectiveness were made. By comparing the costs of implementing these countermeasures with the safety benefits derived from hypothesized accident reductions, a measure of the potential cost-effectiveness was obtained.

HIGHWAY SAFETY AND SSD: ISSUES AND RELATIONS

A review of highway safety research forms the basis for evaluating the relative safety of SSD restrictions. The results of previous research, which relate accidents to roadway geometry, are summarized in the next paragraph. The findings are useful in estimating the magnitude and severity of accidents attributable to SSD restrictions.

Foody and Long (1) investigated the incremental hazard associated with geometric elements such as grades, curves, intersections, and sight-distance restrictions. One phase of the project focused on single-vehicle accidents on two-lane rural highways. A regression model was derived that included percentage of SSD restrictions as an independent variable. A sensitivity analysis of the SSD variable predicted an increase of about 1 single-vehicle accident per million vehicle miles in going from 0 to 100 percent SSD restrictions.

Kihlberg and Tharp (2) studied the difference in annual accident rates over 0.5-km highway sections with all combinations of four geometric features: grades (4 percent or more), curvature (4° or more), intersections, and structures. Although SSD restrictions were not directly studied, the geometric variables can be used to evaluate the sensitivity of safety to conditions that create SSD deficiencies (such as vertical curves after steep grades and sharp horizontal curves). The study also provided clues on the variability in accident rates created by variable geometry and conditions. The worst conditions resulted in accident rates about 2.5 times higher than the best conditions.

Table 1. Basic accident rates selected for use in SSD analyses.

Facility Type	Representative Accident Rate[a] (per million vehicle kilometers)	Accidents Resulting in Injury or Fatality (%)
Rural freeway	0.50	38
Rural two-lane highway	1.50	42
Urban freeway	1.10	29
Urban arterial	5.30	32

[a]Data from AASHTO (11), although the statewide accident data from Florida and Illinois were obtained from the FHWA contract, "Effectiveness of Design Criteria for Geometric Elements."

Cirillo and others (3) studied the relations of geometry and traffic variables to accident rates on Interstate highways with a series of regression models. One model, which described accident rates along the main line between interchanges, contained minimum SSD as a geometric variable. Its contribution to predicted accident experience was negligible.

Other studies were inconclusive on the safety effects of SSD. Schoppert (4) judged sight distance as insignificant in explaining variations in accident rates. Raff (5), Gupta and Jain (6), and Sparks (7) did not reach any conclusions on sight distance. Agent and Deen (8) reported that a significant portion of accidents on two-lane rural highways were rear-end collisions, which suggested that sight distance may play a role in accident causation. [Other reports that may be of interest are from Dart and Man (9) and Glennon and Weaver (10).]

This review of safety literature reveals three basis conclusions that relate SSD and accident occurrence:

1. Identification of the specific effects of limited SSD on accident rates has not been achieved by previous research,
2. Indications are that limited SSD contributes to safety problems on a range of highway types, and
3. Combinations of geometric problems (including SSD restrictions) generally result in higher accident rates.

That previous research has not established a strong link between accident experience and SSD is not surprising. First, highway sections that have unusual or severe SSD restrictions are relatively rare; therefore, the inclusion of such sections in a large data base would dilute the effects of the restrictions. Second, a characteristic of SSD restrictions is their relatively short length. Accident studies that rely on long study sections would dilute any effect of the SSD restriction, which usually affects only a small proportion of the section length. Third, the safety history of limited SSD locations would reflect not only the severity of the restriction, but also the effect of other geometric elements present at or near the restriction, such as intersections, sharp curvature, and narrow structures. No study has quantified SSD restrictions to the extent necessary to identify these effects.

DEVELOPMENT OF A SAFETY EFFECTIVENESS RATIONALE

The relation of SSD to safety is hypothesized as a series of basic elements. These elements relate to the functional and operational aspects of SSD, the other geometric characteristics of the highway, and the basic measures of exposure to the SSD hazard. The framework developed to evaluate the sensitivity of SSD to safety has five basic elements: traffic volume, facility type, severity of SSD restrictions, length of SSD restrictions, and presence of other geometric features.

Traffic Volume

Traffic volume is an obvious determinant of the relative hazard at a location that has limited SSD. The risk of an accident resulting from a critical event (e.g., object in road, head-on encounter) within the SSD restriction is directly proportional to the number of vehicles exposed.

Facility Type

Accident experience varies considerably among facility types. Traffic volumes, patterns, and operating speeds; the character of the roadway alignment; and the presence, number, and nature of conflicts all contribute to variances in the types and number of accidents. For example, head-on encounters and crossing conflicts are potential problems on two-lane rural highways, but not on freeways. Conversely, on urban freeways the proximity of entrances and exits combined with high traffic volumes at locations of limited sight distance can create rear-end incidents. The data in Table 1 give the representative accident rates and severities selected for the analysis of SSD safety for the various highway facilities studied.

Severity of SSD Restriction

The relative hazard at a location that has limited SSD depends partly on the severity of the restriction. Severity refers to the amount of SSD available relative to the operating speed of the highway. This concept recognizes the operational and safety differences presented by the range of SSD-deficient locations. For example, consider two crest vertical curves on a highway with an operating speed of 115 km/h. Curve A has a minimum SSD of 200 m, whereas curve B has a minimum SSD of 240 m. Although both vertical curves are deficient (relative to the AASHTO minimum SSD requirement of 260 m), curve A clearly presents a more hazardous situation. With a 185-m braking distance at 115 km/h, curve A only provides 15 m for perception-reaction time, which amounts to 0.47 sec. Curve B has 55 m and 1.72 sec of perception-reaction time.

The severity of SSD restrictions can be characterized by the differential between the design speed and the operating speed of the highway. Therefore, the locations in this analysis were classified by their SSD severity according to the following table, where the severity of a location is measured against the AASHTO minimum SSD (note that operating speed in this table represents the speed at which free-flowing vehicles travel through the area of restricted SSD):

Severity	Increment of Speed (km/h) Under Highway Operating Speed for which SSD Is Sufficient
Moderate	15
Significant	25
Extreme	35

Length of Sight-Distance Restrictions

Another factor that affects the relative safety of a SSD deficiency is the length of highway over which the restricted sight distance exists. This length is a basic measure of exposure to risk; the longer the restriction, the greater the probability that an event (such as an object falling onto the road) will occur within the restricted area. The length of restriction is determined by a sight-distance profile, as shown in Figure 1.

The relations among length of restriction, severity of the restriction, and A (the algebraic difference in grades) are shown in Figure 2. In general, the more severe the restriction, the shorter the length of highway affected. (For analysis purposes, AASHTO minimum SSD is used as the required SSD for the parameters in Figure 2.)

Relations similar to those of Figure 2 for horizontal SSD restrictions are not as readily developed because of the great variability in conditions that create horizontal SSD restrictions.

Other Geometric Features

The fifth element of SSD-deficient locations is the character of the roadway as defined by all other geometric features within the restricted area. A short crest vertical curve that hides a sharp horizontal curve presents a greater accident potential than if no such condition existed. These locations are rightfully viewed as particularly susceptible to sight-distance-related accidents. The data in Table 2 give a classification of confounding geometric features in terms of their estimated relative hazard when combined with deficient SSD.

SAFETY CHARACTERIZATION OF SITES THAT HAVE SSD DEFICIENCIES

A link between deficient SSD and higher accident rates is only indirectly established by previous research. Incremental safety effects of SSD deficiencies defined by various levels of severity of restriction and situational hazards have not been established. Nevertheless, greater accident rates are expected on highway sections that have large SSD deficiencies and other hazards, such as intersections and curves, within the sight restriction. What must be hypothesized is the magnitude of these accident rates relative to average rates.

Accident Frequency Distributions

Accident data collected in three states (for the FHWA contract "Effectiveness of Design Criteria for Geometric Elements") for two-lane rural highway curves indicate how accident rates fluctuate. The data in Table 3 give both the mean accident rates and the numbers of sites that have rates significantly higher than the mean. Despite differences among the states, rates of 3 to 5 times the mean are clearly appropriate for worst-case geometry situations.

This distributional characteristic of accident rates allows the quantification of potential accident rates relative to levels of perceived hazard. For example, it can be presumed that the accident rate attributable to deficient SSD is greater than the average rate (considering that most highway mileage has adequate SSD) and less than 5 times the average rate. Furthermore, it can be presumed that the contribution of deficient SSD, expressed as a multiple of the average rate, would depend on the other important elements previously discussed (i.e., severity of the restriction and other geometric hazards).

Accident Rate Reduction Factors

The basic research objective was to identify potentially cost-effective countermeasures to SSD safety problems. With this objective, optimistic assump-

Figure 1. SSD profiles for vertical curves.

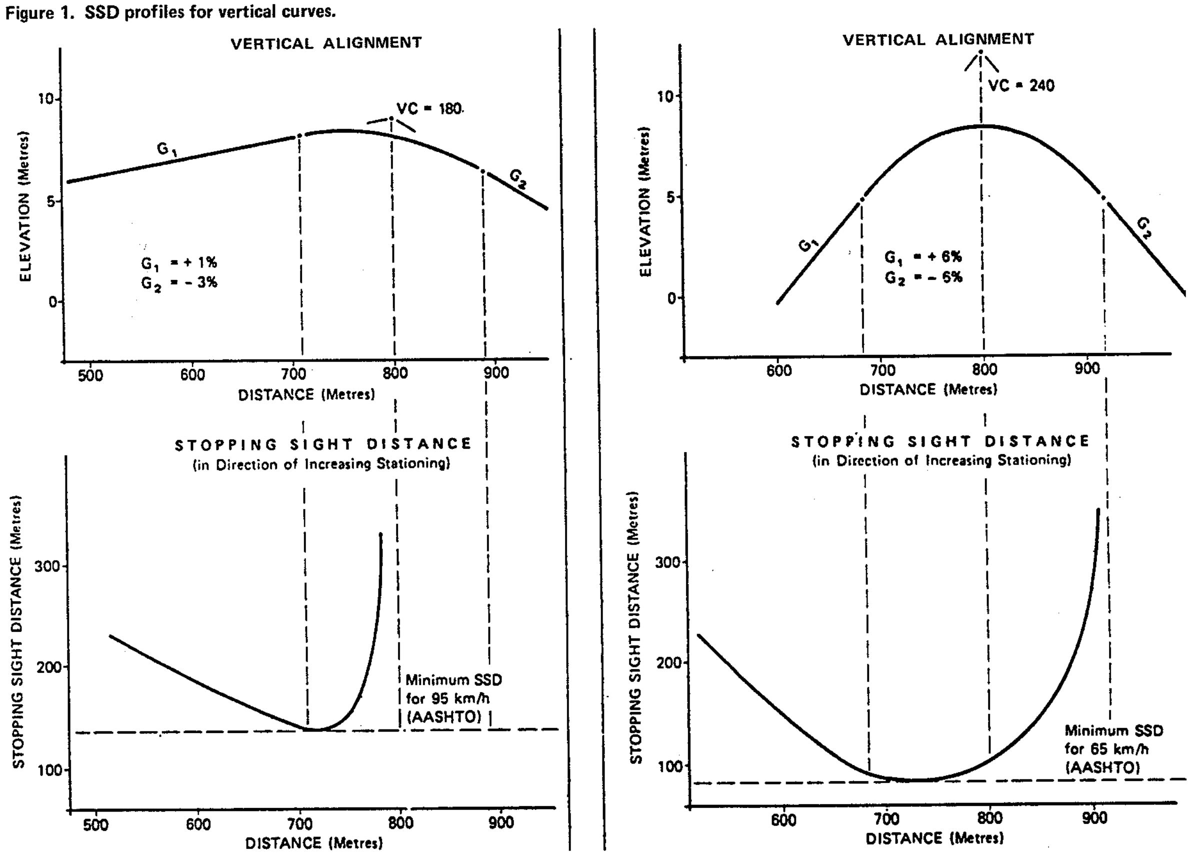

Figure 2. Relations among grades, severity, and length of SSD restrictions on vertical curves.

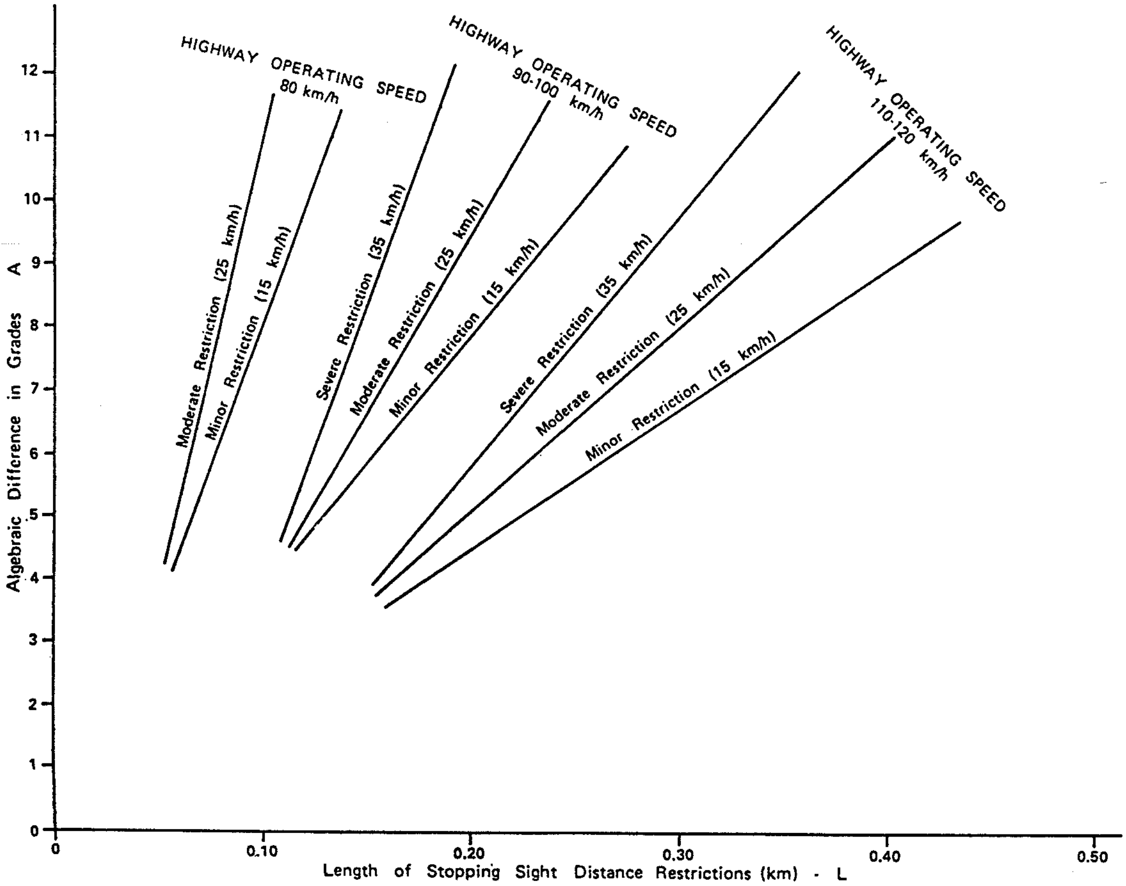

Table 2. Hazard presented by geometric conditions within SSD deficiencies.

Relative Hazard	Geometric Condition
Minor	Tangent horizontal alignment Mild curvature (>600-m radius) Mild downgrade (<3 percent)
Significant	Low-volume intersection Intermediate curvature (300- to 600-m radius) Moderate downgrade (3 to 5 percent) Structure
Major	High-volume intersection Y-diverge on road Sharp curvature (<300-m radius) Steep downgrade (>5 percent) Narrow bridge Narrowed pavement Freeway lane drop Exit or entrance downstream along freeway

tions were built into the framework for evaluating safety benefits, which had the effect of firmly establishing upper limits on countermeasure effectiveness.

With these considerations, a matrix of accident rate reduction factors was developed to describe the hypothesized relations between accident rate and two basic descriptors of SSD conditions: (a) the severity of the restriction, and (b) the presence of other confounding geometric features within the restriction. The factors given in Table 4 for two-lane rural highways represent incremented multiples of the average accident rate for any facility type. These incremental factors were rationally selected to describe the variability in accident rate for all possible combinations of severity of SSD restriction and geometric features.

To illustrate the use of the data in Table 4, note that the greatest factor (4.0) represents a 35 km/h SSD deficiency and includes a major geometric hazard. Such a location is expected to have an accident rate of 5 times the average rate. Full treatment of both aspects (the geometric feature and the severity of SSD restriction) would presumably reduce the rate to the average (1.0). Thus the increment of accident rate attributable to the SSD restriction and the prevailing geometric hazard is 5.0 - 1.0, or 4.0 times the average.

The other tabular entries in Table 4 are rationalizations. An average location, i.e., one that does not have a confounding geometric feature but does have adequate SSD, has an incremental factor of 0, which indicates that SSD does not contribute to the accident rate. Similarly, a highway section that does not have SSD deficiencies but does have a major geometric hazard (such as a high-volume intersection) is expected to have an accident rate twice the average (hence the factor 1.0).

The use of the data in Table 4 completes the analysis framework established to evaluate the accident-reduction effectiveness of countermeasures to SSD problem locations. The framework is defined by the following basic steps:

1. Definition of the highway type;
2. Selection of the appropriate average accident rate (R_h) from the data in Table 1;
3. Definition of the geometry at the location by (a) the severity of the SSD restriction from the

Table 3. Distribution of accidents on two-lane rural highways.

State	ADT	Sample Size	Mean Accident Rate, μ (accidents per million vehicle kilometers)	No. of Sites with Accident Rates 3μ	4μ	5μ
Florida	1,400-2,099	152	0.81	9	4	2
	2,100-3,099	160	0.75	4	2	1
	3,100-4,899	154	0.76	5	3	2
Ohio	1,400-2,099	251	2.20	1	1	0
	2,100-3,099	224	2.02	5	0	0
	3,100-4,899	181	1.97	1	0	0
Texas	1,400-2,099	328	1.14	13	4	2
	2,100-3,099	458	1.16	23	12	5
	3,100-4,899	274	1.11	9	5	2
Total		2,182		75[a]	31[b]	14[c]

Notes: Data are from an FHWA contract, "Effectiveness of Design Criteria for Geometric Elements." ADT = average daily traffic.

[a]3.4 percent. [b]1.4 percent. [c]0.6 percent.

Table 4. Hypothesized accident rate factors for evaluation of SSD restrictions.

Character of Geometric Condition within SSD Restriction	Severity of SSD Restriction by Vehicle Speed 0 km/h	15 km/h	25 km/h	35 km/h
Minor hazard	0	0.5	1.2	2.0
Significant hazard	0.4	1.1	2.0	3.0
Major hazard	1.0	1.8	2.8	4.0

Note: Factor multiplied by average accident rate is accident rate attributable to the combined effects of the roadway geometry and SSD restriction.

Table 5. Average cost per accident by highway type.

Highway Type	Avg Cost of Accident Resulting in Injury or Fatality[a] ($)	Avg Cost per Accident: All Accidents[a,b] ($)
Rural freeway	28,583	11,263
Rural two-lane highway	25,410	11,048
Urban freeway	14,221	4,584
Other urban	11,922	4,256

[a]Data from FHWA (13). [b]Data from AASHTO (11).

previously discussed in-text table, (b) the length of the SSD (L_r) restriction from the data in Figure 2, and (c) the hazard within the SSD restriction from the data in Table 2;

4. Determination of the accident rate factor (F_{ar}) from the data in Table 4;

5. Calculation of the contribution of the defined situation to the annual accident experience:

Annual number of accidents attributable to the SSD restriction = 365 ADT x (R_h x L_r x F_{ar} x 10^{-6}); and

6. Calculation of the accident-reduction effectiveness for the the various countermeasures.

The framework reveals differences in countermeasures based on their effects on the geometry of the location. Safety benefits achieved by increasing SSD (e.g., by lengthening vertical curves) are determined by reading laterally from right to left in Table 4. Similarly, safety benefits achieved by removing or mitigating the confounding geometric condition (e.g., by moving an intersection or flattening a horizontal curve within the SSD restriction) are determined by reading vertically up in Table 4.

CALCULATION OF SAFETY BENEFITS FROM REDUCTIONS IN ACCIDENT RATES

The basis of many benefit/cost and cost-effectiveness analyses is the dollar value assumed to accrue to society when an accident and injury or fatality is forestalled. In this research the dollar accident benefit from SSD improvements was compared to the costs of implementing these improvements.

The proper cost assignable to traffic accidents has been extensively debated. Given the nature of this study, conservative (high) dollar estimates of safety are appropriate. Therefore, NHTSA (12) societal costs of accidents were used. These costs, updated to 1980, are motor vehicle fatality = $358,408; motor vehicle injury = $3,974; and property-damage collision = $648.

Information from other sources (9) allowed the derivation of average costs per accident for all facility types, which are given in Table 5.

SPOT-IMPROVEMENT COUNTERMEASURES TO SSD SAFETY PROBLEMS

Locations that have SSD deficiencies can be treated with a wide range of countermeasures. These countermeasures may be classified in terms of their intended treatment of the problem. Spot-improvement countermeasures, such as lengthening vertical curves or removing trees or obstructions on the inside of horizontal curves, increase SSD and thereby eliminate the hazard. Considering the basic analysis framework for SSD accidents, it is apparent that other countermeasures are also available. For example, a safer condition can be created by treating a geometric feature within the SSD restriction; i.e., by moving an intersection or flattening a horizontal curve hidden by a crest vertical curve, the accident potential should be reduced.

The data in Table 6 give the countermeasures considered in this research, which represent the geometric and operational improvements available for all basic highway types.

Calculation of Costs of Implementing SSD Countermeasures

Cost-effectiveness evaluations of the countermeasures given in Table 6 required estimates of their cost. Construction cost estimates based on 1980 unit costs were computed for each countermeasure. The data in Tables 7 and 8 summarize the costs of construction for the countermeasures studied. Implementation costs for most of the countermeasures vary with the assumed initial geometric condition and the increment of improvement. Note that the most severe SSD cases (e.g., large A values for

Table 6. Countermeasures to sight-distance safety problems.

Highway Type	Countermeasures Designed to Increase SSD	Countermeasures Designed to Reduce Hazard Within SSD Restriction
Rural Interstate	Lengthen vertical curve Increase cut along inside of curve	Move entrance or exit
Rural two-lane highway	Lengthen vertical curve Remove trees; increase cut along inside of horizontal curve Flatten one or more grades	Flatten horizontal curve within SSD restriction Move intersection Move Y-diverge Widen narrow bridge Widen roadway Signing and delineation
Urban Interstate	Flatten horizontal curve Widen median; remove median barrier; or special median design Lengthen vertical curve Increase offset to retaining wall	Move entrance or exit Flatten horizontal curve Variable message signing Marking or signing schemes on median shoulders
Urban arterial	Lengthen vertical curve	Signalize intersection Signing or marking schemes

Table 7. Costs of construction and implementation for SSD countermeasures on two-lane rural highways.

Countermeasure	Cost of Construction and Implementation ($)
Lengthen vertical curve	
Low A	
15 km/h increase in SSD	120,000-170,000
35 km/h increase in SSD	125,000-185,000
Moderate A	
15 km/h increase in SSD	185,000-270,000
35 km/h increase in SSD	200,000-300,000
High A	
15 km/h increase in SSD	270,000-320,000
35 km/h increase in SSD	300,000-350,000
Clear trees or brush from inside horizontal curve	
15 km/h increase in SSD	3,000-6,000
35 km/h increase in SSD	9,500-15,000
Flatten horizontal curve within SSD restriction	150,000-200,000
Move intersection on Y-diverge away from SSD restriction	120,000-220,000
Widen roadway at crest vertical curve	70,000
Widen narrow structure	—[a]

[a]Generally prohibitive.

Table 8. Costs of construction and implementation for SSD countermeasures on urban freeways.

Countermeasure	Cost of Construction and Implementation ($)
Lengthen vertical curve	
Low A	
15 km/h increase in SSD	500,000
35 km/h increase in SSD	820,000
Moderate A	
15 km/h increase in SSD	870,000
35 km/h increase in SSD	1,270,000
High A, 15 km/h increase in SSD	1,300,000
Move entrance or exit ramp	500,000
Remove horizontal obstructions	200,000-300,000
Install real-time variable message signing	20,000
Redesign of median barrier (offset from centerline) or horizontal curve	60,000-100,000
Signing, marking, or delineation	—[a]

[a]Nominal.

vertical curves and 35 km/h deficiency) are the most costly to improve.

Conversion of Costs to Annual Basis

In performing cost-effectiveness analyses based on annual accident benefits, all construction costs must be annualized. This requires an estimate of the useful life for all elements of the highway. For the types of projects considered in this study, earthwork and roadbed construction were assigned a 40-yr life. The useful life of pavements was assumed to be 20 yr. Thus all current annual construction costs represent complete construction, with reconstruction of the pavement after 20 yr.

If a discount rate of 10 percent is used, the following factors apply:

Present worth (PW) of amount 20 yr hence = PW at 10 percent, 20 yr = 0.148644.

Annual share [capital recovery (CR)] of present amount over 40 yr = CR at 10 percent, 40 yr = 0.102259.

RESULTS OF COST-EFFECTIVENESS ANALYSES OF SSD SPOT-IMPROVEMENT COUNTERMEASURES

Application of the cost-effectiveness framework and all cost estimates yields a complete picture on the nature of countermeasures to SSD safety problems. One basic conclusion is apparent, regardless of facility type:

> Only countermeasures that are relatively inexpensive to implement as spot improvements hold the potential for cost-effectiveness for typical traffic volumes on each type of highway.

The reasons for this basic conclusion are apparent when all the aspects of SSD safety are considered. First, deficient SSD is generally the result of a basic geometric deficiency, which is usually costly to correct. Second, the length of highway over which the deficiency affects operations is in most cases relatively short (for vertical SSD restrictions, the worse the deficiency, the shorter the length of highway affected). Such situations would therefore not have a large number of annual accidents, despite having a large incremental rate attributable to the deficiency.

The data in Tables 9 and 10 summarize the cost-effectiveness analyses of spot improvements to SSD problems. The potential cost-effectiveness of SSD safety countermeasures was determined by comparing the hypothesized dollar accident benefits with the annualized construction costs. A benefit/cost ratio of 1.0 or greater indicated potential cost-effectiveness. Because accident benefits are a function of accident rate and traffic volume, the analyses identified traffic volume levels at which cost-effectiveness was achieved.

A number of significant points should be considered when reviewing the data in Tables 9 and 10. It is important to retain a perspective on the concept of cost-effectiveness as it is applied to decision making. In this study countermeasures whose potential annual benefits exceeded annual costs were identified as cost effective. However, this does not necessarily identify them as economically feasible or even desirable. The implementation of projects or programs also depends on other factors, such as available funding and the economic returns provided by competing projects. The net result may be that benefit/cost ratios much greater than 1.0 are required for implementation of certain countermeasures.

Table 9. Cost-effectiveness of spot-improvement countermeasures to SSD safety problems on rural highways.

Countermeasure	ADT Level Required for Potential Cost-Effectiveness	Geometric, Operational, and Traffic Conditions	Notes
Two-lane highways			
Lengthen vertical curve with a 25 km/h deficiency in SSD	13,000-15,000	Significant hazard within SSD restriction	
	17,000-19,000	Minor hazard within SSD restriction	
Lengthen vertical curve with a 35 km/h deficiency in SSD	10,000-14,000	Significant hazard within SSD restriction	
	13,000-17,000	Minor hazard within SSD restriction	
Flatten sharp horizontal curve within a SSD deficiency	11,000-12,000	Design speed of curve at least 25 km/h lower than design speed of highway	
Clear trees or minor obstructions from inside of horizontal curves	200-400	At least 25 km/h SSD deficiency	
Cut earth from inside of horizontal curve	500-1,000	At least 25 km/h SSD deficiency	
Move intersection or Y-diverge away from SSD restriction			Not cost effective under typical geometric, operational, or traffic conditions
Widen roadway at very short crest vertical curve		Very narrow, low-volume minor roads	Not cost effective
Widen narrow bridge within SSD deficiency			Not cost effective under typical geometric, operational, or traffic conditions
Warning signs, delineation, and pavement-marking schemes	Any level	Minor hazard within 25 to 35 km/h SSD restriction; significant or major hazard within 15 to 35 km/h SSD restriction	
Freeways			
Lengthen vertical curve			Not cost effective under typical geometric, operational, or traffic conditions
Cut earth or rock from inside of horizontal curve			Not cost-effective under typical geometric, operational, or traffic conditions
Move entrance or exit ramp away from SSD			Not cost-effective under typical geometric, operational, or traffic conditions

Table 10. Cost-effectiveness of spot-improvement countermeasures to SSD safety problems on urban freeways.

Countermeasure	ADT Level Required for Potential Cost-Effectiveness	Geometric, Operational, and Traffic Conditions	Notes
Arterial streets			
Lengthen vertical curve			Not cost effective under typical geometric, operational, or traffic conditions
Warning signs, delineation, and pavement-marking schemes			Probably cost effective under all typical geometric, operational, or traffic conditions
Signalization			Cost-effectiveness analysis not performed; Manual on Uniform Traffic Control Devices warrants apply
Freeways			
Lengthen vertical curve with a 15 km/h deficiency in SSD	150,000-175,000	Significant hazard within SSD restriction	
Move entrance or exit ramps away from SSD deficiency of 15 to 35 km/h	200,000-225,000		
Rebuild retaining wall to increase horizonal SSD			Not cost effective under typical geometric, operational, or traffic conditions
Flatten sharp horizontal curve within a SSD deficiency			Not cost effective under typical geometric, operational, or traffic conditions
Widen median at controlling curves with narrow medians and barriers			Not cost effective under typical geometric, operational, or traffic conditions
Variable message signing (real time) near SSD deficiencies			Probably cost effective under all typical geometric, operational, or traffic conditions
Marking and signing schemes on median shoulders in horizontal curves			Probably cost effective under all typical geometric, operational, or traffic conditions
Shift median barrier at controlling curves within existing median width	30,000 0	Typical lengths of controlling horizontal curves	

The final determination of potential cost-effectiveness must be tempered by several other considerations, which include costs and benefits not directly included in the analysis but are presumed to apply. In this study only readily identifiable construction costs and accident benefits were applied. Other costs that would be significant on high-volume roadways include the delay to traffic during construction and the direct costs of maintaining traffic. Additional benefits also accrue to those alternatives where significant alignment changes are made. Reductions in vehicle operating costs are expected on flatter grades, longer vertical curves, and milder horizontal curves.

OTHER CONSIDERATIONS IN SAFETY DESIGN OF SSD

Two other aspects of SSD safety problems are also of interest. The first concerns the general issue of rehabilitation or reconstruction and its effect on decision making toward treatment of SSD safety problems. The second aspect concerns new designs and construction.

Implementation of SSD Countermeasures During Rehabilitation or Reconstruction

The scheduling of a rehabilitation project for a highway produces an opportunity to identify and treat SSD-deficient locations. In such cases the concept of cost-effectiveness results in quite different answers when compared with the analysis of SSD countermeasures as spot improvements. This is true because the separate decision to invest in reconstruction may result in a reduced incremental cost required to simultaneously treat a SSD problem.

An analysis was performed of the cost-effectiveness of implementing SSD safety countermeasures within a planned rehabilitation project. In such cases certain cost items required for rehabilitation need not be included as costs of treating the SSD problem. (One example is the cost of replacing the pavement, assuming the entire roadway is to be rebuilt.) These implications are important; countermeasures that are not cost effective as spot improvements may be so when incorporated into a planned rehabilitation effort. As an example, consider the basic question of lengthening a vertical curve to increase SSD on a two-lane rural highway. If only earthwork and a portion of drainage and engineering costs are assumed to apply, the percentage of full costs to perform the SSD improvement is estimated as shown in the following table:

Difference in Grades, A	Percentage of Full Cost Applicable under Rehabilitation Project
4	20
6	30
8	40

(Note that for this table that the full-cost basis includes all pavement removal, earthwork, new pavement, and shoulder, drainage, signing, and engineering; and that the cost basis as a part of rehabilitation includes some pavement removal, earthwork, and a portion of drainage and engineering.)

Consequently, the ADT levels required to justify treatments such as lengthening vertical curves are significantly lower within the context of a planned rehabilitation project.

The data in Tables 11 and 12 summarize the estimated cost-effectiveness of countermeasures implemented during a planned rehabilitation project. In some cases the ADT level required to justify a countermeasure is identical to that given in Tables 9 and 10. This reflects the judgment that none of the costs of construction and implementation of the SSD countermeasure would be incurred if the SSD deficiency is not treated. In other cases ADT levels are somewhat lower for implementation within a rehabilitation project. This results from the assignment of significant costs (for example, new pavement) to the necessary rehabilitation, which thereby lowers the marginal cost of treating the SSD deficiency.

SSD Safety in Planning and Design for New Highways

Although many of the countermeasures studied did not offer potential cost-effectiveness as spot improvements or within a rehabilitation context, their application is recommended wherever possible during the planning and design phase of new construction. The operational framework for SSD presented in this paper affords the highway designer the opportunity to rationally plan and design for special situations not explicitly covered by AASHTO design policy (14-16). Provision for additional SSD at critical locations, special design values for highways with high truck volumes, and consideration of all geometry in the design of SSD are all indicated by the research. The important point is that such design sensitivity to the varying requirements and criticality of SSD can result in a better design at no incremental cost when it is applied in the initial design stage.

CONCLUSIONS

An analysis of the potential cost-effectiveness of treating deficient SSD has been presented. A rational framework was established that described the accident potential of SSD-deficient locations. The elements of the framework included (a) type of highway, (b) traffic volumes, (c) severity of the SSD deficiency, (d) presence of other conditions or confounding geometry within the area of deficient SSD, and (e) length of highway with less-than-adequate SSD.

Table 11. Cost-effectiveness of countermeasures to SSD safety problems incorporated within planned rehabilitation projects on rural highways.

Countermeasure	ADT Level Required for Potential Cost-Effectiveness	Geometric, Operational, and Traffic Conditions	Notes
Two-lane highways			
Lengthen vertical curve with a 35 km/h deficiency in SSD	4,000-5,000	Significant hazard within SSD restriction	
Lengthen vertical curve with a 25 km/h deficiency in SSD	5,000-6,000	Significant hazard within SSD restriction	
	6,000-7,000	Minor hazard within SSD restriction	
Flatten sharp horizontal curve within a SSD deficiency	11,000-12,000	Design speed of curve at least 25 km/h lower than design speed of highway	
Clear trees or minor obstructions from inside of horizontal curves	200-400	At least 25 km/h SSD deficiency	
Cut earth from inside of horizontal curve	500-1,000	At least 25 km/h SSD deficiency	
Move intersection or Y-diverge away from SSD restriction			Not cost effective under typical geometric, operational, or traffic conditions
Widen roadway at very short crest vertical curve		Very narrow, low-volume minor roads	Not cost effective
Widen narrow bridge within SSD deficiency			Not cost effective under typical geometric, operational, or traffic conditions
Warning signs, delineation, and pavement-marking schemes			Probably cost effective under all typical geometric, operational, or traffic conditions
Freeways			
Lengthen vertical curve			Not cost effective under typical geometric, operational, or traffic conditions
Cut earth or rock from inside of horizontal curve			Not cost effective under typical geometric, operational, or traffic conditions
Move entrance or exit ramp away from SSD deficiency			Not cost effective under typical geometric, operational, or traffic conditions

Table 12. Cost-effectiveness of countermeasures to SSD safety problems incorporated within planned rehabilitation projects on urban freeways.

Countermeasure	ADT Level Required for Potential Cost-Effectiveness	Geometric, Operational, and Traffic Conditions	Notes
Arterial streets			
Lengthen vertical curve			Not cost effective under typical geometric, operational, or traffic conditions
Warning signs, delineation, and pavement-marking schemes			Probably cost effective under all typical geometric, operational, or traffic conditions
Freeways			
Lengthen vertical curve with a 15 km/h deficiency in SSD	25,000	Significant hazard within SSD restriction	
Move entrance or exit ramps away from SSD deficiency of 15 km/h or more	40,000		
Rebuild retaining wall to increase horizontal SSD			Cost effectiveness is extremely variable and depends on the nature of the rehabilitation project and existing conditions
Flatten horizontal curve within a SSD deficiency			Not cost effective under typical geometric, operational, or traffic conditions
Widen median at controlling curves with narrow medians and barriers			Cost-effectiveness is extremely variable and depends on the nature of the rehabilitation project and existing conditions
Variable message signing (real time) near SSD deficiency			Probably cost effective under all typical geometric, operational, or traffic conditions
Marking and signing schemes on median shoulders in horizontal curves			Probably cost effective under all typical geometric, operational, or traffic conditions
Shift median barrier at controlling curve within existing median width			Probably cost effective under all typical geometric, operational, or traffic conditions

Three important findings resulted from the analysis.

1. Only inexpensive countermeasures such as clearing horizontal sight obstructions, signing, and delineation are potentially cost effective as spot improvements at locations with deficient SSD (based on AASHTO policy).

2. A greater potential for cost-effective treatment of SSD-deficient locations exists when a highway is planned for major rehabilitation or reconstruction. In such cases only the incremental costs of treating the SSD are attributable to a cost-effectiveness analysis. The result is that, for higher volume facilities, treatment of serious SSD deficiencies during rehabilitation may generally be cost effective. Also, inexpensive countermeasures such as clearing horizontal sight obstructions may be cost effective for a wide variety of site conditions.

3. Although many of the countermeasures studied were not potentially cost effective either as spot improvements or as part of a rehabilitation project, their application is recommended during the planning and design of new construction. The operational framework for SSD presented in this paper allows the highway designer to rationally plan and design for special situations not explicitly covered by AASHTO design policy. Provision for additional SSD at critical locations, special design values for highways with high truck volumes, and consideration of all geometric features in the design of SSD are all indicated by the research. The important point is that a sensitivity to the varying requirements and criticality of SSD can result in a better design at no incremental cost when it is applied in the initial design stage.

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Publication of this paper sponsored by Committee on Geometric Design.

Current State of Truck Escape-Ramp Technology

ANDREW J. BALLARD

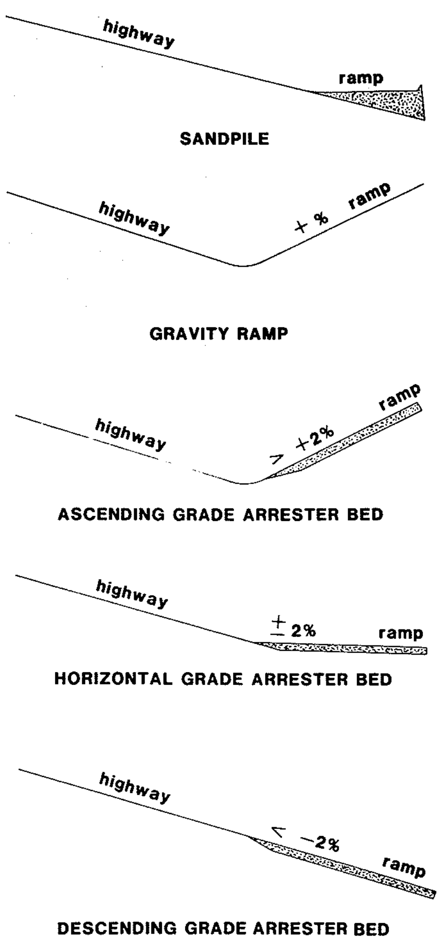
Drivers who lose control of their heavy vehicles on long, steep downgrades have an alternative to riding out the hill when a truck escape ramp is on the grade. There are six basic types of escape ramps in the United States. Only recently has there been an appreciable increase in the advancement of truck escape-ramp technology. Many of these advancements were developed by state transportation agencies and are documented individually in the various states' reports. The purpose of this paper is to provide a pool of information on the characteristics of the many truck escape ramps that are found in the numerous literature sources throughout the United States.

Many states provide escape-ramp facilities for the purpose of reducing the runaway truck hazard on long, steep downgrades. These ramps are used by vehicles that have lost their braking capabilities and are out of control. Escape ramps allow the driver to regain control by slowing or stopping the truck at an acceptable level of deceleration. Such facilities have been present in several states for many years; however, it has been only recently that states have accelerated the advancement of truck escape-ramp technology.

State transportation agencies have largely designed their truck escape ramps based on experience. This has sometimes been coupled with what the designers intuitively believed would improve the operation of the facility. Although they have merit, such empirical methods of design may not be conducive to developing the best design for a given escape-ramp need.

Most of the truck escape-ramp designs and operational successes have not been documented or have been documented individually or in small groups in a statewide status report. The purpose of this paper is to present a single document that describes pertinent aspects of truck escape-ramp technology found throughout the United States.

Figure 1. Five types of truck escape ramps.



CHARACTERISTICS OF CURRENT DESIGNS

The term truck escape ramp encompasses six different types of general designs: sandpile, gravity ramp, ascending-grade arrester bed, horizontal-grade arrester bed, descending-grade arrester bed, and roadside arrester bed. These designs are shown in Figures 1 (1) and 2 (2). All of these ramps function according to at least one of two basic methods of vehicle deceleration: (a) vehicles are decelerated by gravity (the gravity ramp and the ascending-grade arrester bed use this method), and (b) some form of arresting material is used, usually sand or gravel, such that the rolling resistance offered by the material is the predominant means of decelerating the vehicle (most truck escape ramps use this device to different degrees).

Sandpiles are masses of arresting material placed on the roadside such that the top surface is approximately level or at a slightly ascending grade. The surface of the sandpile may or may not be covered with transverse ridges. When a vehicle enters a sandpile truck escape ramp, the arresting material increases rolling resistance against the tires and, if the vehicle sinks in the sand far enough, against the undercarriage.

A gravity ramp consists of a hard-surfaced lane that is on an ascending grade that may or may not have a small aggregate bed near the top. The purpose of the bed is not to contribute significantly to the deceleration of the vehicle but to keep the vehicle in place once it has stopped. If no such aggregate bed is present, there is the possibility that an articulated vehicle may roll backward and jackknife. Vehicles that enter gravity ramps are decelerated primarily by the force that results from gravity acting opposite the direction of movement.

Truck escape ramps that incorporate arrester beds are all similar in design with the exception of the grade of the ramp. An ascending-grade arrester bed consists of a ramp on an ascending grade that has a

Figure 2. Roadside arrester bed.

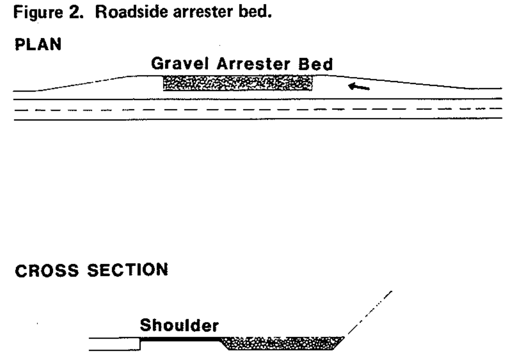
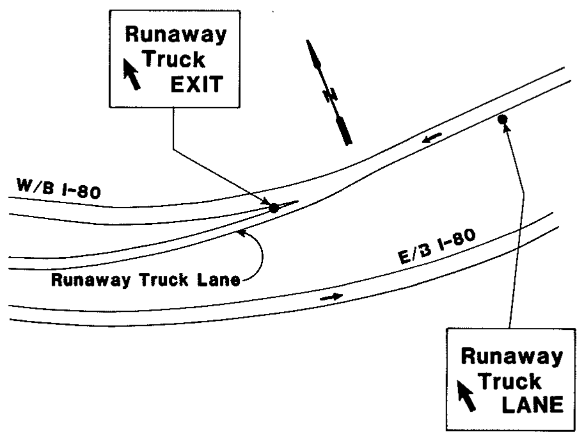


Figure 3. Horizontal-grade arrester bed in Parley's Canyon, Utah.



bed of arresting material (usually sand or gravel). The arresting material and gravity contribute to the deceleration of a vehicle that enters the ramp. Horizontal-grade arrester beds are truck escape ramps that are approximately level. For purposes of classification in this paper, grades of ±2 percent are defined as horizontal. The deceleration of vehicles in these ramps is the result of rolling resistance provided by the aggregate. Descending-grade arrester beds are facilities in which the vehicle is decelerated by the arresting material. The force provided by this material must also counteract the effect of the descending grade.

Another type of escape ramp that is similar to the descending-grade arrester bed is the roadside arrester bed. The roadside arrester bed is parallel and adjacent to the main line and has provisions whereby a vehicle may enter from the side, as well as the upstream end, of the arrester bed.

Every truck escape ramp in the United States today is one of these six types. Because each truck escape-ramp location is unique, the designer must carefully consider several ramp characteristics. The different combinations of the many truck escape-ramp characteristics can lead to either an acceptable or an inadequate design.

TRUCK ESCAPE-RAMP CHARACTERISTICS ASSOCIATED WITH RAMP TYPE

Truck escape-ramp characteristics can be categorized as being associated with a certain ramp type or independent of the ramp type. This section describes the characteristics associated with the ramp type.

Length

The length of a truck escape ramp is a key design feature. The required length of a ramp depends on design entry speed, type of arresting material, and grade. Because these last two factors differ for the different ramp types, the typical lengths for these ramps also differ. Preferably the lengths of truck escape ramps should be determined by analytical techniques. Many facilities in the United States were designed on such a basis. The design parameters for the different truck escape ramps resulted in facilities of various lengths.

The shortest truck escape ramps are the sandpiles, which are usually less than 400 ft long. The shortest cited facility is on US-421 in North Carolina, which is only 210 ft long. However, Crowe (3) reports that such a short length should be expanded to 400 ft in order to avoid having a high-speed vehicle pass completely through the sandpile.

Gravity ramps are typically long because they have only limited means of decelerating runaway vehicles other than gravity. Pennsylvania has gravity ramps of 1,200, 1,525, and 1,550 ft, and Hawaii has one that is 1,300 ft long.

Ascending-grade arrester beds exist with lengths from 330 to 1,560 ft. The longest truck escape ramp is the 2,480-ft horizontal-grade arrester bed in Parley's Canyon on I-80 in Utah (4). It is located in the median, as shown in Figure 3 (4). The length of this truck escape ramp is excessive due to the design assumption of only 10 to 20 percent rolling resistance provided by the aggregate.

Descending-grade arrester beds are generally longer than ascending-grade arrester beds because of the difference that gravity makes in whether it works to the advantage or disadvantage of the deceleration process (5).

Most roadside arrester beds are quite long due to being adjacent to the descending main line, where gravity acts in opposition to the resistive forces. The roadside arrester bed at Mt. Vernon Canyon in Colorado has a 2,075-ft gravel bed, of which the last 325 ft has a sand-barrel positive attenuator, which effectively reduces the standard aggregate bed length to 1,750 ft.

In designing truck escape ramps, regardless of the type, the length should be determined by analytical techniques. Such techniques, in the form of design equations, are discussed later in this paper.

Width

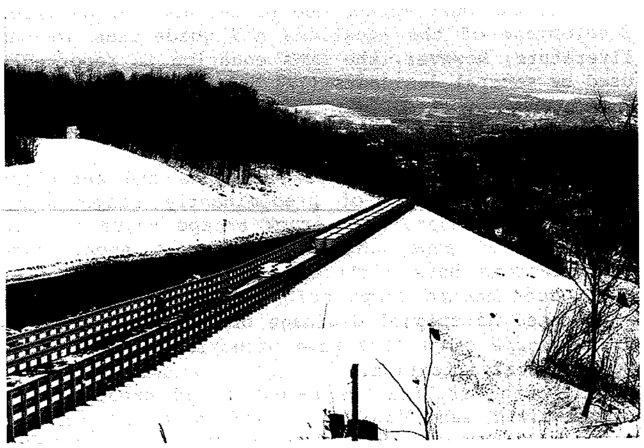
The width of a truck escape ramp is not generally a function of ramp type; it is closely related to the backup measures used, i.e., alternatives for a runaway vehicle in the event that the truck escape ramp is already occupied. Because of this relation, arrester beds and sandpiles typically need to be wider than gravity ramps, which are frequently 12 to 14 ft wide (6). Sandpiles and ascending-, horizontal-, and descending-grade arrester beds need to be wide enough for more than one vehicle to occupy the facility at the same time. Newton (2) suggests that roadside arrester beds, like gravity ramps, do not need to have widths adequate for multiple occupancy. Manifestations of this suggestion are found in the Mt. Vernon Canyon roadside arrester bed in Colorado, which has a width of only 20 ft, and in two roadside arrester beds on US-50 in Nevada, both of which are also 20 ft wide.

The other types of arrester beds generally have widths between 26 and 30 ft, although the Monteagle Mountain horizontal-grade arrester bed in Tennessee is 50 ft wide (7) and the Pali Highway ascending-grade arrester bed in Hawaii is 16 ft wide and tapers down to a 12-ft width at the end. There are other truck escape ramps that have tapered widths, e.g., the descending-grade arrester bed on NY-28 in New York, which tapers from 18 to 12 ft in width (8), as illustrated in Figure 4 (8). The idea behind designing this tapered escape ramp was to channelize the vehicle and minimize excessive yawing and jackknifing. However, the problem with such a design is that fewer vehicles can simultaneously occupy the far end of the ramp than if the width was held constant.

Arresting Materials

One of the first applications of the term sandpile in describing a truck escape ramp was in Virginia (9). This was a good descriptor because the arresting material was, indeed, sand. Pennsylvania has

Figure 4. Descending-grade arrester bed on NY-28 in New York.



truck escape ramps that are identified as sandpiles, yet none use sand; all Pennsylvania sandpiles use a form of gravel. These truck escape ramps are called sandpiles by virtue of their basic design, as shown in Figure 1, without regard to the arresting material.

The material employed in arrester beds is independent of the grade; i.e., ascending-grade, horizontal-grade, descending-grade, and roadside arrester beds all use approximately the same aggregate types. The most common aggregates are pea gravel and loose gravel, where the latter refers to rather angular aggregate as opposed to the rounded pea gravel. The type of aggregate used is a function of availability. For example, truck escape ramps in Hawaii use an angular aggregate because the more desirable pea gravel is unavailable at a reasonable cost. Pea gravel is more desirable because of the high percentage of voids, which provides better drainage than angular aggregate.

Surface Ridges

Early experiences in North Carolina and Virginia indicated that smooth-surfaced sandpiles did not always stop the runaway vehicles. Therefore, the addition of irregular mounds on the surface of the sandpiles was introduced in the expectation that these would increase their deceleration abilities (10). Arrester bed truck escape ramps have smooth surfaces, although some states (11) have considered using transverse surface ridges. Experience has indicated that transverse surface ridges are useful on sandpiles, and research has revealed that they are harmful on arrester beds (8,10,11).

Initial Cost

The initial cost of a truck escape ramp must consider excavation, right-of-way acquisition, and local labor costs. Examples of costs of various ramp types are identified in this section.

Sandpiles are the least expensive type of truck escape ramp. In Virginia two VA-52 sandpiles and the VA-33 sandpile were built in 1972 and 1975 for $10,000 each (9). Sandpiles in North Carolina cost $25,000 each in 1974 and 1975 (12).

The initial costs of arrester beds vary greatly. The most expensive truck escape ramp is the $529,000 ascending-grade arrester bed on I-70 west of the Eisenhower Memorial Tunnel in Colorado (13). The initial costs of arrester beds can be as low as $100,000, which was the cost of the only truck escape ramp built in New York (8).

Because roadside arrester beds are built adjacent to the roadway and do not need to be wide enough for multiple occupancy if they are built in pairs on the downgrade, they are less costly than other arrester beds (2).

Maintenance

There is little documentation on maintenance costs of truck escape ramps. Three sandpiles in North Carolina reportedly averaged $200/use in restoration expense from 1975 to 1978 (3,12). A descending-grade arrester bed in the Siskiyou Mountains in Oregon averaged $25 in 1980 in repair costs for each use of the facility. In 1977 Versteeg and Krohn (5) reported $73/use as the average restoration cost on the two ascending-grade arrester beds on the Willamette Highway in Oregon. In Oregon the driver of the runaway vehicle is billed for this maintenance expense. These monies are used to restore the facility to its design state. Gravity ramps usually have no reported maintenance costs due to ramp use.

Other expenses are usually incurred in the act of removing the vehicle from the arrester bed or sandpile and in routine maintenance that is not a result of ramp use.

Among truck escape ramps, gravity ramps are closet to being maintenance-free, although rollback-induced jackknifing requires some maintenance. The only routine maintenance needed is that associated with the appurtenances for the ramp, e.g., signs and luminaires. All truck escape ramps require this type of maintenance (5).

All other truck escape ramps (i.e., those that have arresting material) require maintenance after each use. When a vehicle enters a facility, its wheels create ruts in the arresting material. These ruts must be eliminated and the shape of the bed must be restored before the next vehicle enters the bed or sandpile.

Predominantly single-sized aggregate is the best aggregate to use because it has good drainage characteristics. The arresting material must be replaced after it has accumulated too many fine particles (14). Replacement-interval requirements due to excessive fines are not currently well defined; thus related maintenance is performed only occasionally. Some facilities are built such that the arresting material is routinely expelled from the ramp during use; maintenance crews occasionally have to replace such material (2).

Sandpiles and arrester beds alike may require a deicing agent if the facility is in an area prone to freezing (6).

Environmental Influence

Truck escape ramps, other than gravity ramps, can be adversely affected by freezing temperatures because the aggregate may freeze to form a hard surface, although some facilities that incorporate arresting material have not had problems with freezing (3).

One ascending-grade arrester bed (15) was reported to have performed satisfactorily even when covered with a layer of snow. The thickness of the snow blanket is unknown, and the report reflects only one such incident.

Driver Comments

Different truck escape-ramp installations sometimes evoke different comments from truck drivers. A primary problem with gravity ramps is their inability to prevent a truck from rolling backwards after it has been brought to a stop. Articulated vehicles are vulnerable to jackknifing. Reportedly, truckers have expressed an unwillingness to use gravity ramps for fear of jackknifing, which leads them to the choice of riding out the grade, which frequently ends in serious consequences.

Several states have grades that have more than one truck escape ramp. Usually, where there are two or more facilities on a grade, they are a few miles apart. Bullinger (16) reports that, on grades that have two escape ramps, experience indicates that drivers prefer to use the lower ramp. This appears to point out the truckers' affinity for riding out a grade as long as possible.

Some truck drivers complain about the misuse of truck escape ramps (17). Passenger car motorists often mistake the safety feature for a roadside park, rest area, or the main line of the roadway. In addition, some four-wheel drive vehicles sometimes get entrapped in the arresting material while purposely playing in the facility (18). Such activity has two deleterious aspects: (a) the surface of the arrester bed or sandpile has been disturbed and consequently requires maintenance, and (b) such vehicles are in danger of being struck by a runaway truck that may enter the truck escape ramp for its intended use.

TRUCK ESCAPE-RAMP CHARACTERISTICS INDEPENDENT OF RAMP TYPE

Aside from the truck escape-ramp characteristics that are related to ramp type, there are other characteristics that appear to bear no relation to ramp type. The characteristics described in this section are of this type.

Design Equations

Analytical methods of determining ramp length are available in the form of design equations. A calculator program has been developed in Idaho (19) that uses an iterative approach toward a solution based on entering and final velocity, grade, friction, frontal area, and truck weight.

A design equation reported by FHWA (1) is simply

$$L = (V_i^2 - V_f^2)/30(R \pm G) \quad (1)$$

where

- L = distance (ft) of grade,
- V_i = velocity (mph) at beginning of distance L,
- V_f = velocity (mph) at end of distance L,
- R = rolling resistance (divided by 100) expressed as an equivalent percent gradient, and
- G = percent grade divided by 100.

The suggested values for rolling resistance are 0.15 and 0.25 for sand and pea gravel, respectively.

Sandpiles in Virginia are designed according to a formula that incorporates speed, friction, air resistance, grade, rolling resistance, a 90-mph entry speed, and a truck weight of 72,000 lb (17).

Because all of these equations and guidelines are based on entry speed, it is necessary to report what some escape-ramp designers use for design entry speed. The Roadway Design Manual in Colorado (20) uses 100 mph for truck escape ramps on the Interstate highway system. For all other highways in Colorado, a speed that is 40 percent greater than the highway design speed is used. Other research (1,16) recommends a design speed of 80 to 90 mph. These choices are in line with estimated speeds reported in the records of ramp use.

These equations and empirical guidelines represent all of the analytical design methods found in the current literature. It is difficult to identify which is the best method due to the lack of detailed development of the equations and guidelines in the literature; however, the FHWA equation is reportedly used by several designers (1,4,5,16).

Drainage Provisions

Sufficient drainage of arrester beds and sandpiles is usually a result of predominantly single-sized aggregate; however, some truck escape ramps require some type of pipe network. Different escape-ramp installations have different drainage requirements. Most truck escape ramps are free-draining. Gravity ramps need no special drainage provisions. However, truck escape ramps that have arresting material need some special attention.

In North Carolina the sandpiles of predominantly single-sized sand drain well and have not experienced problems with freezing (3,12). The sand also has been mixed with calcium chloride (a deicing agent).

Some arrester beds do not incorporate special drainage provisions other than the design of a sloped cross section (4), whereas others use perforated pipes or filter fabrics.

Aggregate Gradation

The best aggregate gradation for a truck escape ramp is one that is predominantly single sized. As an example of such gradation, one sample from the arrester bed at Mt. Vernon Canyon in Colorado has the following sieve analysis results:

Sieve Size	Percent Passing
0.75 in.	100
0.375 in.	91
No. 4	18
No. 10	5
No. 40	1
No. 200	1

The more descriptive characteristic of the aggregate is its maximum size. West Virginia uses a relatively large maximum size aggregate of 1.50 in. There are truck escape ramps throughout the United States that use 1.00-, 0.75-, or 0.50-in. maximum size aggregate (16).

At the other end of the spectrum, neglecting the very small percent passing values, 0.25-in. minimum aggregate is used in arrester beds in New York, Utah, and Idaho (4,8).

Although the the sand in the sandpiles in North Carolina is reportedly predominantly single sized, there is no documented information regarding gradation for sandpiles.

Depth of Arresting Material

It has been reported that arrester beds in Colorado that have 18- to 24-in. depths produced 12-in. ruts. These measurements indicate what may be a necessary minimum depth. Nevertheless, the different arrester beds throughout the country indicate that a variety of bed depths, as well as depth tapers, are currently in use.

Descending-grade arrester beds in Hawaii, Idaho, New York, and Texas have aggregate bed depths of 18 to 24 in. (8). The New York ramp has a tapered entry; i.e., the depth of the arrester bed increases as the vehicle travels into the gravel. This bed depth tapers from 0 to 24 in. and then back to 0 in. Two descending-grade arrester beds in Texas are also tapered at both ends; in the first 300 ft the depth increases from 0 to 18 in., and in the last 300 ft the depth decreases from 18 to 0 in. Care should be exercised in choosing the bed depth so that vehicles are not decelerated too abruptly.

Among roadside arrester beds, which are similar to descending-grade arrester beds, the depth of the aggregate bed is the same as for descending-grade arrester beds, namely 18 to 24 in. (14,18). Newton (2) suggests that roadside arrester beds have tapers at both ends, although all of them do not. The two roadside arrester beds in Nevada have depth tapers from 0 to 18 in. in the first 15 ft at the entry and 18 to 0 in. in the last 15 ft at the low end. The purpose for depth tapers at the low end of roadside arrester beds is to allow a vehicle that has traveled the entire length of the bed to be elevated back to the main line level for reentry onto the main line.

The aggregate beds in ascending-grade arrester beds vary. One such facility in Fulton County, Pennsylvania, has only a 6-in. depth. This truck escape ramp decelerates the vehicles solely by gravity for a distance of 924 ft and then uses the 636-ft-long shallow arrester bed. The ascending-grade arrester bed at Rabbit Ears Pass in Colorado is relatively shallow; it tapers from 4 to 12 in. The third ramp of an ascending-grade arrester bed at Lewiston Hill in Idaho has a 30-in. uniform depth.

The depth of sandpiles always increases from the entry to the far end because the base of the sandpile descends as the main line descends, and the top surface of the sandpile is usually level. The sandpile near Kittanning, Pennsylvania, which is composed of pea gravel, has a maximum height of 11 ft.

As with other truck escape-ramp design elements, there is a wide variety of depths and tapers among the existing truck escape ramps. Research that defines the optimum depth of aggregate for various types of ramps and aggregates is lacking.

Truck Removal

In order to easily remove trucks from escape ramps, many truck escape ramps are equipped with a service lane or shoulder and tow anchors, which allow tow trucks to be anchored while pulling the arrested vehicle from the bed. The types and widths of service lanes and shoulders vary among the types of truck escape ramps. In addition, not all facilities have these truck removal appurtenances.

Secondary Retarders

Because of the possibility of a high-speed vehicle traveling through the entire length of the truck escape ramp, some states have placed a secondary attenuator at the end of the ramp so that, if all else fails, the vehicle will stop and not travel beyond the length of the ramp (8,15,17). States use different types of retarders, including gravel berms, standard crash cushions, and specially designed sand barrels, such as those shown in Figure 5.

The use of secondary retarders should be approached with caution because little or no safety research exists on the use of such devices in truck escape-ramp applications. Therefore, care should be exercised when using such retarders to ensure that the safety of the occupants of heavy vehicles is increased, not jeopardized.

Location on Grade

The selection of the location on the grade for a truck escape ramp is critical. Considerations include how far the escape ramp is from the summit, whether it is above or below the halfway point on the grade, and where it is with respect to a critical grade change. States have different ideas on what site criteria are significant.

In New York a truck escape ramp is located as near the base of the grade as possible. In Hawaii the escape ramp is constructed near a downhill tangent section just before a horizontal curve. In Colorado the location is site determined, i.e., such a decision must be made for each problem grade. It is recommended in Oregon that the location be approximately 4 miles from the summit. Eck (21) reports that truck escape ramps near the summit are seldom used.

Erickson (15) reports that experience in Colorado indicates that 70 to 80 percent of runaway trucks will be intercepted by a truck escape ramp 3 to 4.5 miles from the summit. However, no documented data are provided regarding how this conclusion was developed.

Sandpiles are located 3.3 and 3.4 miles from the summit on US-70 in North Carolina. The upper sandpile is located 1.3 miles downhill from a truck brake check area (12). The sandpile on US-421 in North Carolina is 3.4 miles from the summit and 0.3 mile uphill from a narrow bridge that is immediately followed by a sharp horizontal curve (3). A sandpile north of Roanoke, Virginia, is located just before an 18° curve (9).

A descending-grade arrester bed in Leslie County, Kentucky, is located at an approach to a T-intersection, and one on NY-28 in New York is located just uphill from a village (8).

Lewiston Hill in Idaho has six truck escape ramps; one is located 1 mile below a grade change from 6 to 7 percent, where some runaway truck accidents have occurred.

Some reports identify the location on the grade by the distance of the escape ramp from the summit, and others do so by its distance from another escape ramp on the grade.

Left- or Right-Hand Exit

There is some debate regarding left-hand versus right-hand exits on a divided highway. Arguments supporting the former are based on the idea that speeding runaway trucks operate in the fast lane (i.e., the left lane) and therefore would not have to maneuver around other vehicles to enter a ramp to the left of the main line. Conversely, proponents of right-hand exits maintain that left-hand exits violate driver expectancy (6).

Figure 5. Sand-barrel impact attenuators on pedestals at end of the truck escape ramp on NY-28 in New York.

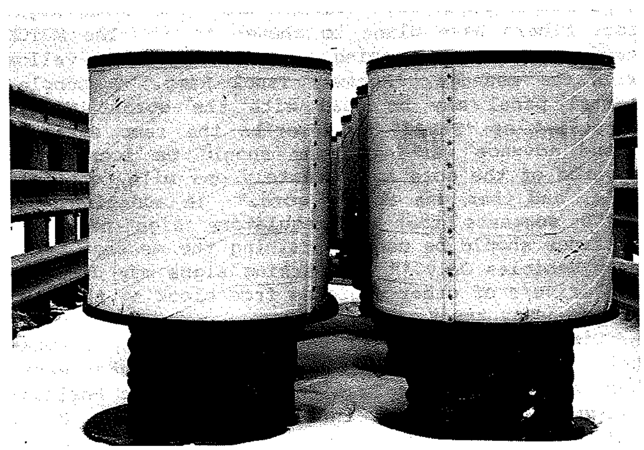
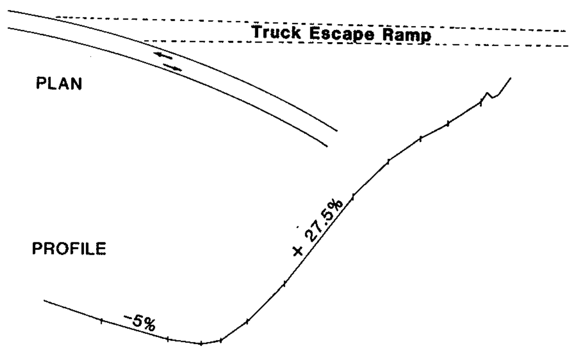


Figure 6. Schematic views of left-hand escape ramp on US-16 in Wyoming.



All truck escape ramps in the United States exit to the right of the main line, with the notable exceptions of those in the median of a divided roadway and some unusual designs in Wyoming. Parley's Canyon in Utah has a horizontal-grade arrester bed in the median. This design was incorporated into the construction plans when the I-80 facility was in the planning stages (4).

Wyoming has three ascending-grade arrester beds that exit to the left side of two-lane undivided highways on US-16 and in Teton Pass. One of these is shown in Figure 6 (21). Such a design obviously means that the runaway truck must enter opposing lanes of traffic. The Wyoming State Highway Department believed that the probability of a truck colliding with a vehicle traveling in the opposing direction as the truck heads for the left-hand ramp is no greater than the probability of the truck striking a vehicle as the driver tries to maneuver the runaway vehicle down the grade by using both lanes. In other words, without a truck escape ramp at all, the runaway truck uses both lanes of the two-lane highway in negotiating the grade, and this could bring the truck into opposing traffic just as using a truck escape ramp with a left-hand exit would. It is important to realize that these highways have low traffic volumes.

Signing

A dual signing continuum is necessary on a steep downgrade. One system of signs informs truckers of the danger of the upcoming downgrade and, where it exists, the location of the brake check area. The second sign system guides the driver of a runaway truck into the truck escape ramp (17).

Before the issuance of the 1978 Manual on Uniform Traffic Control Devices (MUTCD) (22), there was little uniformity in advance signing for truck escape ramps. Today most states follow the MUTCD signing; others have plans to change to it. The MUTCD mandates that the signing "shall be black on yellow with the message, 'Runaway Truck Ramp.' A supplemental panel may be used with the words 'Sand,' 'Gravel,' or 'Paved' to describe the ramp surface. These advance warning signs should be located in advance of the gore approximately one mile, one-half mile, and then one at the gore." In addition, the MUTCD suggests that a "regulatory sign near the entrance should be used containing the message 'Runaway Vehicles Only.'" No Parking signs may discourage drivers of other vehicles from blocking the path of a runaway truck.

The roadside arrester bed in the Siskiyou Mountains in Oregon on I-5, the ascending-grade arrester bed near Rabbit Ears Pass in Colorado, the horizontal-grade arrester bed in Parley's Canyon in Utah, and a descending-grade arrester bed on Mullan Hill in Idaho are among those escape ramps that use MUTCD signing (4,14,15). Some truck escape ramps have signs that are not found in the 1978 MUTCD. Some of these facilities are described in the following paragraphs.

The Parley's Canyon truck escape ramp is located in the median; hence the exit is to the left. Because this violates driver expectancy, all signs, including an advance sign 2 miles uphill from the ramp entrance, have arrows pointing at a diagonal toward the lower left (4).

The ascending-grade arrester bed with the left-hand exit on US-16 between Buffalo and Tensleep in Wyoming has a special signing requirement--there are warning signs that inform drivers climbing the grade that they may encounter a runaway truck in their lane.

Delineation

Delineation at the approach of a truck escape ramp is important in that the driver of the runaway truck must be properly led into the ramp and yet other motorists must not be mistakenly led off the main line into the escape ramp. The MUTCD (22) provides pavement marking, object marker, and post-mounted delineator designs for use throughout the highway system. But because of the dual criteria required for truck escape-ramp delineation, special attention is necessary.

Williams (17) suggests that some new type of delineation mechanism be developed that is different from the standard yellow and white delineators. It is believed that motorists observing standard color delineators can mistakenly be led into the truck escape ramp. To remedy this problem, Williams suggests red delineators. Pennsylvania will soon be experimenting with just such a delineation method.

Backup Measures

In the event a truck escape ramp is occupied, a backup measure is needed. In the current inventory of facilities there are two backup measures: (a) the truck escape ramp is designed to be wide enough for more than one vehicle to occupy it simultaneously, and (b) a second truck escape ramp is constructed downstream from the first.

Some sources (1,23) use trucks with wheel bases of 40 to 50 ft as the design vehicle. Because these trucks are 8.5 ft wide (23), the width of the arrester bed is suggested to be 26 ft or more. This constitutes a backup measure.

The second backup method is the construction of a second facility nearby. A sandpile in North Carolina was constructed solely as a backup measure (12). In the Rocky Mountains some steep, long grades have more than one truck escape ramp, e.g., Lewiston and White Bird Hills in Idaho, Willamette Highway in Oregon, and the hill on US-50 near Carson City, Nevada. Such multiple facilities function as backups.

Gravity ramps usually do not need backup measures because the time of occupancy in the ramp is generally short compared to arrester beds and sandpiles, which usually hold vehicles for a few hours before the vehicle is finally back onto the hard surface (6). However, if a truck jackknifes in a gravity ramp, the time of occupancy can be high.

Regardless of the type of backup measure, the truck escape ramp should be designed such that the driver of a runaway truck can see the entire ramp to know whether it is occupied or not.

Grades

The grades of the various truck escape ramps differ because of the terrain at the sites. Sandpiles, gravity ramps, and arrester beds use a variety of grades.

Most sandpiles in North Carolina have horizontal top surfaces; however, some newer sandpiles in that state have ascending-grade tops. This may be a less-acceptable design because the front ends of the trucks tend to dig into the sand, which results in damage to the truck.

The gravity ramp with the steepest slope is the 1,200-ft ramp in Franklin County, Pennsylvania, which has a +21.5 percent grade (17). The flattest gravity ramp is also in Pennsylvania; it is the lower ramp on Boot Jack Hill. This ramp is composed of two grades: a +6 percent grade followed by a +13 percent grade.

The steepest truck escape ramp in the United States is the ascending-grade arrester bed at Rabbit Ears Pass in Colorado. It has a +42.8 percent grade, which follows a +2.64 percent grade (15).

The steepest descending-grade arrester bed is on NY-28 east of Utica, New York. This facility is on a -10 percent grade. Williams (17) describes a -2.5 percent grade on the 800-ft descending-grade arrester bed west of Buffalo, Wyoming, on US-16. Other descending-grade arrester beds are on grades between these two extremes (14,16,22).

The roadside arrester bed in the Siskiyou Mountains in Oregon is constructed on a -5.5 percent grade (14). The Mt. Vernon Canyon roadside arrester bed in Colorado is on a -5.6 percent downgrade (13).

It is evident that some types of truck escape ramps may be found with a variety of grades. Conversely, other types (i.e., roadside arrester beds) are generally built with similar grades.

CONCLUSIONS

Although truck escape ramps have been present in the United States since 1956 (17), it has been only recently that the state of the art has witnessed accelerated advances. Literature on the details of design and operation of the various escape ramps is generally scattered among transportation agencies.

The characteristics of many of these escape ramps have been discussed in this paper. These characteristics include physical dimensions, arresting material, maintenance, cost, design equations, drainage, truck removal, signing, delineation, and gradient.

The individual truck escape ramps in the United States present alternative responses to each of the characteristics just mentioned. This indicates that optimum designs may not yet be identified, and there is still opportunity for further advancement in truck escape-ramp technology.

ACKNOWLEDGMENT

The preparation of this paper has been financed in part by FHWA. I wish to thank the many state transportation agencies that supplied much of the information cited in this paper. Gratitude is extended to the New York State Department of Transportation for supplying the photographs used in this paper.

In addition, I wish to thank Martin Hargrave of the FHWA, and John J. Labra and C.E. Kimball of the Southwest Research Institute, for their guidance, as well as Loretta J. Mesa, Janet Shuler, and Carol Ballard for their contributions.

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Publication of this paper sponsored by Committee on Geometric Design.

Notice: The opinions, findings, and conclusions expressed in this paper are those of the author and do not necessarily reflect those of the FHWA.

Designing Highways for Buses: The New Jersey Experience

STEVEN R. FITTANTE

In this paper a procedure and set of criteria for accommodating bus operations through transit-sensitive highway design are described. A set of bus operational needs (including bus stopping, passenger waiting, and bus priority requirements) are compared with the current arterial highway design standards and features used by the New Jersey Department of Transportation. Bus needs that are not accommodated by standard highway design are described, including bus turnouts, pedestrian-actuated crossing signals, and bus priority lanes. The transit impact review process includes an evaluation of current and future bus needs, a determination of whether the proposed highway design will serve those needs, and identification of those highway projects that require highway design changes to better accommodate bus needs. The approach taken stresses the joint effort of highway engineers and transit planners in order to (a) evaluate the transit impact of proposed highway improvement projects and (b) suggest workable design modifications.

The provision of bus priority measures through highway design has received considerable attention in transportation research literature during the past decade. Demonstration projects that involve bus priority lanes, bus preferential ramp metering, and traffic signal preemption are among the specialized design elements through which highway design engineers have shaped the transit system environment (1).

Despite the increased interest in strategies for bus operating performance through highway design, most highway designs that are sensitive to bus operating needs involve specialized additions or modifications to the original highway elements. In New Jersey the design of state highways has not specifically considered the needs of existing or future (potential) bus operations. Although the highway project approval process traditionally called for an evaluation of impacts on transit, this review admittedly did not relate the needs of a bus operation to the highway design standards as required by the state.

Criteria for establishing an approach to better accommodate bus operations are outlined in this paper, and a modified highway project approval process for determining situations where existing highway design standards can accommodate bus operations is described. The process further requires that both the lead unit of the New Jersey Department of Transportation (NJDOT) for the highway project and transit planners from the New Jersey Transit Corporation (NJ TRANSIT) share the task of identifying those projects that may require specialized highway design elements in order to properly accommodate bus operating needs. Other states should find this planning approach to transit-sensitive highway design applicable to their respective transportation departments' procedures.

BACKGROUND

The motivation to develop a transit sensitivity process came from an arterial highway project that affected a major commuter bus corridor. Route 9 is a north-south arterial highway that runs the length of eastern New Jersey. The central portion, which traverses Middlesex, Monmouth, and Ocean counties, varies in paved width from a two-lane undivided roadway to a six-lane arterial highway with a median barrier.

The most heavily traveled commuter bus corridor in New Jersey operates on nearly 40 miles of Route 9 in central New Jersey. The various routes operating on Route 9 serve daily passenger trips to and from destinations in Newark, Jersey City, and New York City. Commuter bus passengers represent more than 50 percent of the total passenger volume carried on Route 9 during the peak period.

For these reasons, the impact of highway widening projects on Route 9 became a concern of transit planners at NJ TRANSIT. Because NJ TRANSIT is a new agency, this concern developed after the highway design process for widening Route 9 in Middlesex County had been completed. The particular project replaced an existing grass median with a concrete median barrier and created an additional travel lane by removing the existing shoulder lanes. The completed project had several adverse impacts on bus operations. The removal of the shoulder lanes resulted in the elimination of two major commuter bus stops and the discontinuance of a bus priority lane on the shoulder during the morning peak period. The concrete barrier, which was unbroken for a distance of 1.5 miles, prevented bus passengers from safely crossing the highway, which contributed to the elimination of a southbound bus stop.

Because the project did not require additional right-of-way acquisition, it was classified as a categorical exclusion project. As such, the project was excluded from the more detailed environmental reviews required for major projects because of the relatively minor impacts on the community. However, the waiving of standard highway design features such as shoulder lanes and vehicle turnarounds (which afford breaks in the median barrier) resulted in adverse impacts on existing bus operations.

To cope with the dislocation created by the highway improvements, NJDOT staff worked with NJ TRANSIT staff to negotiate needed accommodations, which included temporary bus loading areas, a permanent bus turnout, and a commuter priority lane for high-occupancy vehicles during the commuter peak period. This cooperative effort led to the development of evaluation criteria and a process for accommodating bus operating needs in the highway design process.

TRANSIT NEEDS CRITERIA AND REVIEW PROCESS

The criteria for evaluating the impact of highway design on transit operations are classified according to the following categories:

1. Bus operating needs: What are the physical roadway characteristics that a bus operation requires to operate safely and effectively?
2. Highway design standards: What are the existing arterial highway design standards that also accommodate bus operating needs?
3. Specially designed features for transit: What specialized design features are required to meet bus operating needs not served by the highway design standards?

The joint review process complements the existing NJDOT highway project review procedures. These existing guidelines, outlined in the NJDOT Action Plan, provide for the review of highway project transit impacts so that modifications to standard highway designs can be made to meet the bus operating requirements at the particular project location (2).

Bus Operation Needs

The physical highway features required by bus operations will vary with the type of bus route and the level of service provided at a particular location. For example, the ability of a bus to easily exit and enter a highway is of greater importance to an express commuter route that serves off-line park-and-ride facilities than to a local arterial route. The following highway design requirements are critical to most bus operations.

Basic to any arterial highway operation, areas are needed, which are separated from the travel lane, where passengers can be picked up and discharged by the buses. As an absolute minimum, a 40-ft commuter bus requires 80 ft to pull off the highway and merge back into the travel lane. Generally, a 96-in.-wide bus requires at least a 9-ft lane to be safely separated from highway traffic and to allow passenger boarding. (Note that these dimensions are intended not as standards but to indicate minimum acceptable values for a bus operation.)

At locations where headways are frequent (generally more than 10 buses/hr), it may be necessary to have a stacking lane at major stops. This is particularly true at bus stops where express buses run and at locations where passenger loading is particularly heavy. At major intersections that have heavy traffic volumes, this lane should extend through the intersection to allow buses to accelerate and merge from an exclusive lane.

Occasionally, buses may need to exit the highway to serve park-and-ride facilities located several hundred feet from the highway. This is easily accommodated by reverse loops or diamond interchanges, which allow all turns to be made from the right lane. On undivided highways where left turns are permitted, channelization or left-turn signals may be needed to accommodate the turning radius of buses, particularly where service headways are frequent.

Bus passengers need to be able to wait safely out of traffic when boarding and disembarking; they must also be able to safely cross a highway to reach cars or residences. This is particularly important where major park-and-ride facilities are located adjacent to a divided highway. Accommodation for bus shelters adjacent to the roadway shoulder lane is needed where passenger waiting occurs.

Finally, some form of signalization, such as green-cycle timing, may be required at intersections near a bus garage so that buses can easily depart from the garage during peak times. In certain cases a signal at the garage itself may be needed in order to properly meter a heavy flow of buses onto a highway.

Highway Design Standards That Serve Transit

The NJDOT Bureau of Design uses a set of standards for highway design elements that can serve some basic bus operating needs. Some of these standards include warrants that specify the frequency or minimum conditions for using the particular design feature.

Generally, where available right-of-way exists, state highways include 12-ft shoulder lanes. These lanes, ostensibly designed for emergency stopping, can also serve as acceptable bus stop areas. The standard width is more than sufficient to meet bus needs.

A standard of 0.5 mile between vehicle turnarounds on divided highways is another standard that unintentionally benefits bus operations. This standard was used to ensure convenient access to adjacent land uses and allow for U-turns, particularly for emergency vehicles. The turnaround or jughandle provides a break in the median barrier and may have a traffic signal, which can also serve pedestrian traffic, particularly bus passengers who cross a highway. The transit industry standard for maximum walking distance between a bus stop and either a passenger's home or car is usually 0.25 mile. Therefore, the highway design standard roughly conforms with the standard of transit needs. Adherence to such a standard can reduce the dangerous incidence of bus patrons climbing over a concrete barrier to cross a four- or six-lane highway.

There are several other highway features for which specific standards or warrants are unavailable. Grass medians, which are sometimes traversed by a concrete sidewalk, are a convenient way of preserving the median for future widening and at the same time benefiting bus passengers who must cross a divided highway. At locations with pedestrian-actuated signals, the median can serve as a safe waiting area for pedestrians crossing the highway at the end of the green cycle. Diamond interchanges, which allow a bus to exit off a ramp to reach an intersection bus stop and immediately merge back onto the highway, are excellent for bus operations. They allow the bus the time to decelerate and accelerate when leaving and entering the highway and provide a safe point at which to stop and pick up or discharge passengers.

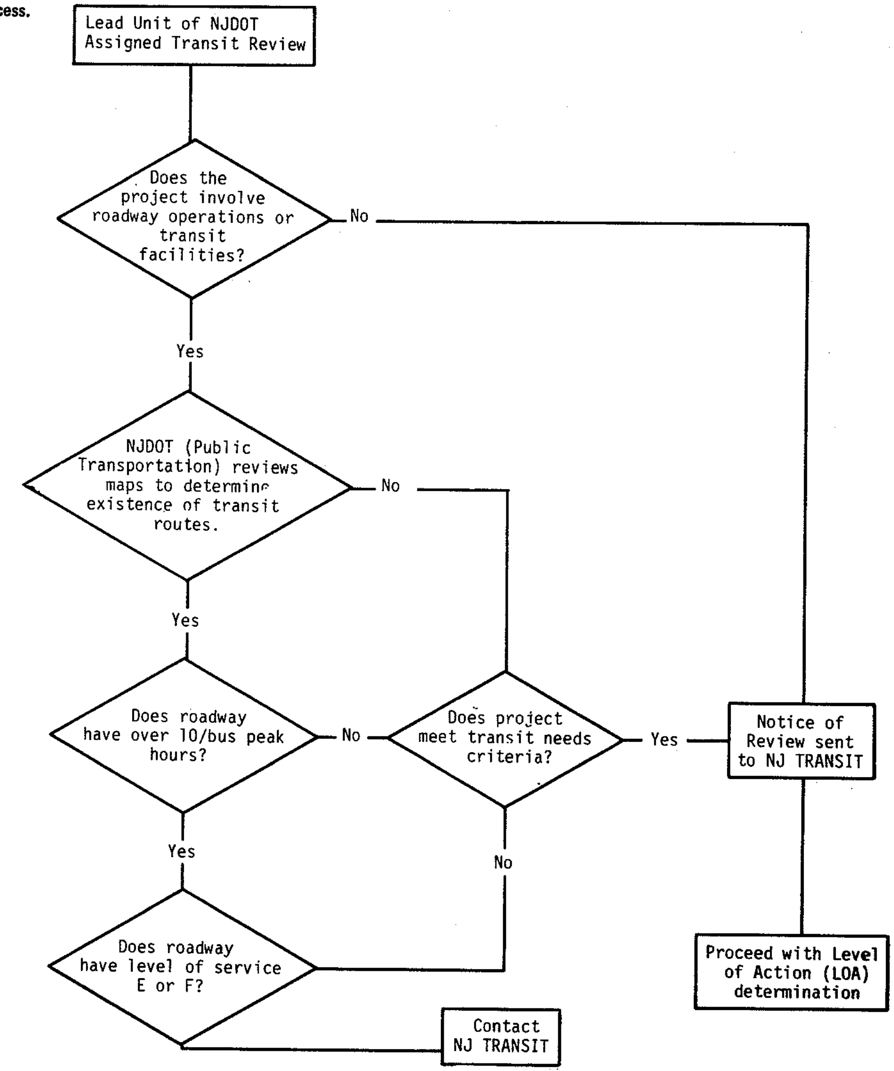
Specially Designed Features for Transit

In most cases highway design standards serve the basic needs of bus operations. There are times, however, when these elements have either been waived or a particular transit need is present that cannot be accommodated by the standard design.

In areas where shoulder lanes are not present or are of a substandard width, a bus turnout may be required to allow buses to stop outside of the travel lanes. The length of bus turnouts varies from 130 to 180 ft, depending on whether the location is midblock, near side, or far side of an intersection. The width of the loading area may be from 10 to 12 ft, and acceleration and deceleration tapers should be at least 3:1 and 5:1, respectively (3).

When a traditional bus stop is located between intersections on a divided highway, some specialized form of pedestrian access may be desirable. Two available options are a pedestrian overpass and a pedestrian-actuated signal. Pedestrian overpasses suffer from high cost and the reluctance of pedes-

Figure 1. Proposed transit impact review process.



trians to climb steps or ramps in order to reach the overpass. Accessibility requirements, when inclined ramps are used instead of stairways, increase this distance and may also eliminate any safety incentive for pedestrians to use the overpass.

Pedestrian-actuated signals provide a relatively low-cost means of affording access across a divided highway. Problems result from the warrants (1,000 passengers/hr), which are rarely attained on arterial highways, and the dangers to pedestrians from automobiles not heeding the infrequent red signal when such warrants are not met. Nevertheless, there are examples of such pedestrian-actuated signals at locations of less than 1,000 crossings/day where such signals have been operated successfully. In these cases most of the crossings are concentrated in the peak period.

In some cases, such as the previously described Route 9 corridor, frequent bus service accounts for a majority of the passenger volume during the peak period. Under these circumstances bus priority may be warranted through either a reversible (contraflow) lane or a dedicated right lane or shoulder. Such measures should be considered where a roadway level-of-service E or F exists and the bus passenger volumes carried in a dedicated lane are greater than the capacity of the mixed-use travel lanes (4).

Transit Impact Review Process

The process outlined in Figure 1 is designed to involve both highway project engineers and transit planners in recognizing the need for specialized highway designs to accommodate transit needs. Unlike the existing transit impact review for highway projects, this process identifies the aforementioned bus operation characteristics that necessitate special design treatment. Project planners also evaluate highway projects with respect to their capacity for meeting the needs of existing as well as future transit operations.

The staff responsible for initiating the highway project initially determines whether a project involves either a roadway operation or a transit facility. Any project that involves one of the following types of actions can affect roadway operations or a transit facility (5):

1. Approval of utility installation along or across a transportation facility;
2. Reconstruction or modification of an existing bridge structure on essentially the same alignment or location;
3. Modernization of an existing highway by resurfacing, restoration, rehabilitation, widening

less than a single lane width, adding auxilliary lanes, or correcting substandard curbs and intersections;

4. Highway safety or traffic operations improvement projects;

5. Corridor fringe parking facilities; or

6. Installation of signs, passenger and bus shelters, and traffic signs.

Once a project is determined to involve roadway operations or transit facilities, the NJDOT lead unit responsible for managing the project reviews the route maps provided by the transit agency and determines whether any bus operations currently operate in the project area. The maps for New Jersey include the routes operated by NJ TRANSIT, subsidized bus carriers, and private (unsubsidized) bus carriers.

If no existing routes are identified, the project design standards are reviewed to see if they meet the transit needs criteria (bus stops, pedestrian access, and so on). This step ensures that the transit agency is aware of projects that would be constructed at less-than-standard design levels and might not accommodate future transit development.

A decision involving bus frequency is designed to identify express bus loading points that would require bus stacking areas and possible bus priority treatments. Similarly, the roadway level-of-service question identifies those projects that might be candidates for bus priority design improvements. The lead unit reviews bus schedules and traffic volume data to make these determinations.

As noted in Figure 1, projects that meet the criteria for specialized design accommodations are sent to the transit planning section for further review. Bus operations personnel and transit planners then determine whether such improvements would be beneficial, and if so, they contact the Bureau of Design at NJDOT to negotiate what design improvements could be made, given cost and other constraints. Projects that will be constructed at less-than-usual design standards are also reviewed by NJ TRANSIT.

It is expected that most projects will not involve adverse impacts and that they will meet the transit needs criteria. Once the review is completed by the lead unit at NJDOT, a notice of review is sent to NJ TRANSIT through the usual Level of Action process.

RECENT DEVELOPMENTS

A task force consisting of representatives from NJDOT lead units and Planning and Bus Operations from NJ TRANSIT has been charged with the task of modifying and implementing the proposed criteria and process. Currently, task force members have agreed to develop additional criteria for local-aid projects that involve secondary roads and to simplify the notification procedure for those projects that are reviewed and have no impact on existing or future bus operations.

The task force intends to conduct a series of workshop sessions to explain the review process to NJDOT project managers and engineering staff who will perform the actual evaluation of project impacts.

CONCLUSIONS

Although implementation of the formal process is not yet complete, the increased cooperation between highway engineers and transit planners is already evident. Transit planners can now anticipate highway design impacts on heavily traveled corridors by reviewing the capital programming documents for highway projects. Highway engineers make greater efforts to notify NJ TRANSIT of construction staging that may temporarily affect bus operations, and they cooperate to minimize these impacts. The negotiation between NJ TRANSIT and NJDOT design engineering staff on ameliorating specific problems such as pedestrian access is currently conducted with both staffs being more aware of the respective constraints on highway design and bus operations.

Regardless of the final form that the implementation process takes, the mutual cooperation fostered to date should result in a highway design process in which recognition of bus operating needs results in a more effective highway operation.

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Publication of this paper sponsored by Committee on Geometric Design.

Guidelines for the Design and Placement of Curb Ramps

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The need for guidelines for the design and placement of curb ramps is evident from the confusing and contradictory standards for these features and the problems with curb ramps that have been constructed. The objective of this research was to develop such guidelines. Information was obtained from surveys of 10 state departments of transportation, 4 large U.S. cities, and 18 departments of public works in Virginia. A sample inventory of curb ramps was made in 15 municipalities in Virginia. Observations were made of curb ramps for wheelchair users and the blind. From the information obtained, guidelines for the design and placement of curb ramps were developed, which specify curb ramp dimensions based on sidewalk width and placement relative to obstructions, crosswalks, and types of intersections.

During an examination of the sections of the Code of Virginia that relate to curb ramps and also the several design standards for these facilities, a subcommittee of the traffic research advisory committee for the Virginia Highway and Transportation Research Council found several variations in the standards and also a number of problems.

Various federal, state, and local agencies responsible for complying with legislation related to curb ramps have established standards for their design, as indicated in Table 1. The largest range of specified values among the standards is the 5.0 to 17.0 percent slope for the ramp. The ramp width varies from 3.0 to 4.0 ft for one-way movements. Three of eight sets of standards require a lip. These and other conflicting design criteria evident in Table 1 promote confusion. (Note in the lower half of the table that the factors consider the placement of the curb ramp in relation to its environment.) Most of the standards, like those of VDH&T, address placement partially or not at all. Although the standards must be applicable for a wide range of situations, they should encourage consistency and uniformity in curb ramps.

Problems encountered in the application of standards include (a) obstructions such as utility poles, mailboxes, and hydrants in the path of the handicapped; (b) indirect paths across streets; (c) curb ramps without sidewalks, which encourage pedestrian activity in hazardous areas; (d) undesirable interference of curb ramps with drainage structures; and (e) lack of maintenance. Special considerations are necessary for the visually handicapped who use curbs as a guide. Because of these problems research was undertaken to develop guidelines for the design and placement of curb ramps.

DATA COLLECTION

Following a review of the pertinent literature, surveys were conducted with agencies involved in the design and construction of curb ramps, and an inventory was made of curb ramps in selected localities.

Surveys

Telephone surveys conducted of 10 state departments

Table 1. Standards for the design of curb ramps.

	Standards							
Characteristic	Code of Virginia (1)	Virginia Department of Highways and Transportation (VDH&T) (2)	AASHO (3)	U.S. Department of Housing and Urban Development (HUD) (4)	General Services Administration (GSA) (5)	American National Standards Institute (ANSI) (6)	FHWA (7)	American Public Works Association (APWA) (8)
Type of ramp	Diagonal			Flared; parallel extended continuous curb	Parallel to pedestrian traffic where possible	Diagonal; parallel offset		Diagonal; parallel offset
Ramp slope (%)	5.0	8.33	8.33	17.0	8.33 preferred; 16.67 maximum	8.33 maximum	8.33 maximum	8.33 preferred; 5.0; 16.67 maximum
Ramp width (ft)	4.0	4.0	4.0	4.0	3.0 minimum; 3.5 preferred	4.0	One-way = 3.0 minimum; two-way = 3.5 minimum	3.0 minimum; 4.0 preferred
Lip (in.)		0.5		≤0.5		None	None	0.5
Surface texture (for the blind)	Nonslip	Nonslip	Nonslip	Nonslip	Color contrast and texture	Nonslip	Brush broom finish and grooves	Wood float or other rough finish
Ramp located within crosswalk		Offset from crosswalk and in front of stop line	Yes		Yes			Yes
Adapt ramp to site			Yes					Yes
Deals with obstructions		Yes	Yes; may require offset from crosswalk					Yes; alternatives
Drainage concerns			Yes					Yes
Pedestrian conflicts			Yes		Yes			
Other			Refers to ANSI	Corner ramps; midblock ramps; ramp alignment; access to ramps (parking)				

Table 2. Summary from survey of state DOTs and traffic engineering units in urban areas.

Characteristic	California DOT (Caltrans)	Florida DOT	Georgia DOT	Kentucky DOT	Michigan DOT	New York State DOT	North Carolina DOT
Type of ramp	Parallel; diagonal; built-up ramp	Diagonal; parallel	Parallel; diagonal on (a) paved walk, (b) unpaved area on walk, (c) end of walk	Parallel; offset parallel; diagonal	Parallel; offset parallel; diagonal	Parallel; offset parallel; diagonal	Parallel; diagonal
Ramp slope	1:12	1:12	1:12	1:12	1:12, 1:18 maximum	1:12, 1:18 maximum	1:12
Ramp width (ft)	4	3	3.5 minimum	4 minimum	4 minimum	4 minimum	3.3
Flare width (ft)	Variable	2	4; 2 minimum	Variable	Variable	Variable	4
Lip (in.)	None	None	None	None	None	0.625	None
Surface texture	Groves 0.25 x 0.25 in. on 0.75-in. centers at top of curb ramps 12 in. wide extending the width of ramps and flares	Tined finish	Tamps, wood floats, and broom finish	Broom finish	Coarse broom finish	Deep grooved, 0.5 in. wide by 0.25 in. deep on 1-in. centers transverse to ramp	Nonskid-type finish
Located within crosswalk	Yes		Yes	Yes	Yes		Crosswalk = 10 ft; no parking within 20 ft of ramp
Adapt ramp to site	Five variations for walk depression or widening	Three alternatives with varying flares, depending on walk design	Three types subject to engineering judgment	Five variations based on types of streets and traffic control	Five variations based on types of streets and traffic control	Subject to engineering judgment	Based on 3- and 4-legged intersections; subject to engineering judgment
Deal with obstructions			Subject to engineering judgment			Subject to engineering judgment	Subject to engineering judgment
Drainage concerns			Catch basins should be at least 10 ft from ramps; drop inlets out of line	Alternate locations identified	Structures out of line with ramp	Drainage pickups upstream from ramps; grates are used in area of ramps	Subject to engineering judgment
Date of standard	1/81	2/79	1/79	7/75	4/74	6/76	1/76
Other		Also a bicycle ramp		Revised version of Michigan DOT standard	Orientation notches on side of ramp for the blind are being eliminated	Walks 11 ft wide are sloped 1:24 to accommodate ramp	

of transportation (DOTs), the traffic engineering units in 4 urban areas, and 18 departments of public works in Virginia yielded the information presented below.

Survey of State DOTs and Traffic Engineering Units

A summary of the information obtained in the survey of state DOTs and traffic engineering units in urban areas is given in Table 2. Six respondents (42.9 percent) used three or more types of curb ramps (including diagonal, parallel, and offset parallel). Five respondents (35.7 percent) used two types of curb ramps, and three respondents (21.4 percent) used one type. The selection of the type of curb ramp to use depended on sidewalk design and type of intersecting streets.

Eleven respondents (78.6 percent) have omitted or will soon omit a lip at the bottom of the curb ramp. The length of the flares ranged from 1 to 6 ft. Fifty percent of the respondents used a broom finish on the curb ramp and 14.0 percent used a grooved surface texture. All except two respondents (14.3 percent) considered placement conditions to some degree. In general, site-specific considerations were subject to engineering judgment. Three standards (21.4 percent) were identified as a bicycle and wheelchair ramp, which promotes dual use of the ramp.

The problems cited were (a) incompatibility between the needs of the blind and those of handicapped persons in wheelchairs, (b) conflicting standards for utility poles and other obstructions, and (c) minor drainage concerns.

Survey of Municipal Departments of Public Works in Virginia

Seventeen municipalities in Virginia that had populations greater than 20,000 and 1 county were surveyed. Ten municipalities (55.6 percent) use the VDH&T standard and 7 municipalities and the 1 county use standards similar to the Department's. The diagonal ramp is the primary type used. Two municipalities base their ramp slope on the Code of Virginia [i.e., 5.0 percent slope (1:20)], whereas all others use an 8.33 percent slope (1:12). Flare lengths range from 2 to 6 ft. Only two standards did not have a lip. Minor departures from the Department's standards include the addition of a midblock design and design variations based on the curb radius or presence of an obstruction.

Only a few problems were cited, the most common of which were (a) conflicting standards for utility poles, mailboxes, hydrants, and so on; (b) enforcement of quality control during construction; and (c) curb ramp use by bicycles and motor vehicles. Note that one point of controversy is whether or not bicyclists should be encouraged to use curb ramps.

Inventory of Curb Ramps

Fifteen areas were selected for the inventory of curb ramps in order to identify effective and ineffective design and placement conditions. In addition, the scope of the problem of curb ramp design and placement was defined. The inventory focused on locations where curb ramps were expected, such as central business districts (CBDs), public buildings,

Figure 1. Curb ramp problem: high lips.

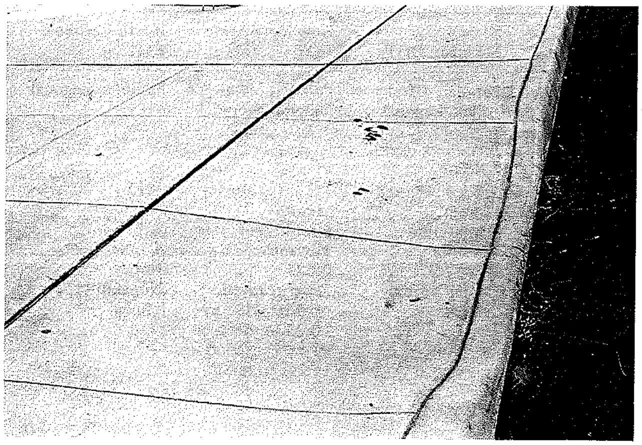


Figure 2. Curb ramp problem: no median break.

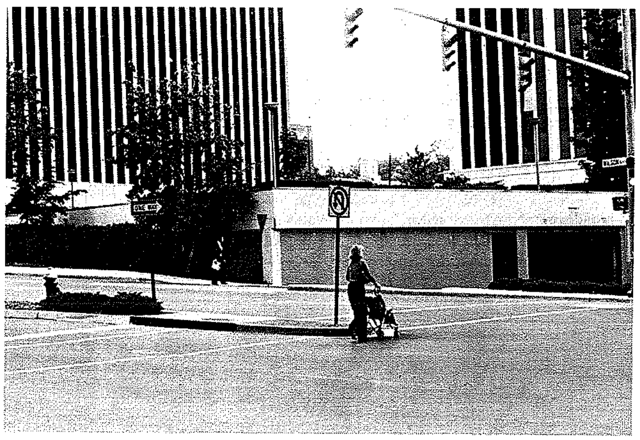


Figure 3. Curb ramp problem: drainage structure affecting placement.

Figure 4. Curb ramp problem: ramp located out of crosswalk.

and residential areas where curb and gutter or sidewalk projects were recently completed. More than 200 sites were reviewed, and 124 were documented in the inventory.

The common problems noted from the inventory are listed below in order of decreasing frequency of occurrence:

1. The absence of matching curb ramps at all corners of an intersection,
2. The presence of high lips (greater than 0.5 in.) and a wide range in lip heights (see Figure 1),
3. Slight problems with obstruction by utility poles and manhole or conduit covers,
4. Ramps offset from the diagonal (or middle of the curb return) for no apparent reason,
5. No median breaks for ramp users on divided highways (see Figure 2),
6. Steep flare and ramp slopes,
7. Presence of drop inlets that affect curb ramp placement (see Figure 3),
8. Curb ramps located outside of marked crosswalks (see Figure 4),
9. Absence of a level area above ramps for turning by wheelchair users, and
10. Parked vehicles blocking the curb ramp.

There appeared to be no distinctions between the curb ramp treatments in rural municipalities and those in urban municipalities, although urban municipalities generally have wider sidewalks. The width of sidewalks varied greatly, from 4 ft in residential areas to 20 ft in large CBDs. Many residential areas had sidewalks on only one side of the street.

Problems 2, 4, and 6 can be eliminated by enforcing quality control in the construction of curb ramps. The remaining problems are related to standards and policy regarding curb ramps; these problems are addressed in the guidelines.

CURB RAMP LIP

Although a lip was desired as an aid for blind persons and to maintain drainage, it was found to be an obstacle to wheelchair users. This section discusses the use of a curb ramp lip.

In a telephone conversation with the assistant executive director of the Braille Institute of America, the need for a 0.5-in. lip as an aid for the blind was discussed. The Institute endorsed the use of a 0.5-in. lip to aid cane users in identifying curb ramps based on the observation of orientation and mobility instructors. The instructors also indicated that some blind persons become disoriented when they step on a curb ramp.

In a laboratory study conducted by Templer (9), the majority of blind persons had little difficulty in detecting a variety of ramps that had different slopes and lips. The observations of mobility training students for the blind at the Virginia Rehabilitation Center for the Blind were consistent with the findings by Templer.

The elimination of the lip presents no problem for the visually impaired. The problem of disorientation caused by the curb ramps can be minimized with consistent placement of the ramps.

When curb ramps were introduced, a 0.5-in. lip was accepted as a compromise between a 1-in. lip to maintain drainage and no lip to avoid an obstacle for wheelchair users. (Note that these data are from a December 23, 1981, memorandum from E.S. Coleman, Jr. of the Locations and Design Division, VDH&T.) The major problem for wheelchair users was that a 0.5-in. lip made it difficult to move up a ramp. In the survey of state DOTs and traffic engineering units in urban areas, no drainage problems were noted by the 11 respondents that did not use a lip. The consensus was that a small lip did not make much of a difference in drainage. Some additional water and debris may accumulate without a lip, but not enough to be considered a problem.

The purpose of curb ramps is to provide accessibility to the handicapped, and wheelchair users benefit from the elimination of the lip. The worst problems with drainage are caused by ice and snow in the winter months when wheelchair users are less likely to use the sidewalk for travel than during other seasons. In areas where there is a low velocity of the runoff water, water and debris accumulate at curb ramps regardless of the presence of a lip.

Based on the above comments, it is concluded that a lip is not necessary to aid the visually impaired or to maintain drainage.

GUIDELINES

The guidelines are divided into four parts: general practices, design, placement, and miscellaneous notes. Before the guidelines are discussed, the goals and objectives of curb ramps are defined.

Goals and Objectives of Curb Ramps

The goal of curb ramps is to provide the physically handicapped, especially persons confined to wheelchairs, access to and from sidewalks so that they are able to traverse streets. There are five objectives related to this goal:

1. Provision of a curb ramp, in respect to design and placement, that is usable by the physically handicapped;
2. Provision of design and placement alternatives for a range of sidewalk and street conditions;
3. Provision of minimal impedance to able-bodied pedestrians;
4. Placement of curb ramps in uniform and consistent locations; and
5. Provision of curb ramps without a lip that are detectable by the blind with no adverse effects.

These objectives have established the framework for the guidelines. There is a trade-off between objectives 2 and 4 in that the design and placement alternatives are limited in order to maintain uniformity and consistency.

General Practices

Five items are included in the guidelines under the heading of general practices.

1. Concrete ramp surfaces shall have a nonskid, broom finish transverse to the slope of the ramp. All concrete shall be class A-3. Ramp surfaces other than concrete do not require a broom finish. Portland cement concrete is the only material referenced in the Road and Bridge Specifications for Curbs and Sidewalks (10). The other most commonly used material--brick--does not lend itself to a broom finish.
2. Matching curb ramps should be provided at all corners of an intersection, or on both sides of a midblock location, to establish a continuous network for ramp users. If curb ramps are not placed at all corners of an intersection, then the ramp user's accessibility is restricted to the paths connecting the ramps. Access to all pedestrian paths should be provided.
3. On new construction projects, utility poles, fire hydrants, and drop inlets should be located so as to provide an unobstructed path to the curb ramp located on the middle of the curb return (also called the diagonal). Because the location of curb ramps may be adversely affected by obstructions, the location of the curb ramp should have priority over the location of potential obstructions.
4. Curb ramps should not be constructed as part of curb projects where no sidewalk exists. As mandated by the Code of Virginia (1) and Section 228 of the Highway Safety Act of 1973 (11), curb ramps are constructed where curbs are constructed or replaced without consideration of the presence of a sidewalk. Some engineers consider this as a form of incremental planning in that a sidewalk and ramp may be added later. However, unpaved surfaces in rough terrain present a potential hazard for handicapped persons. Also, erosion occurring along the curb ramp causes the unpaved surface material to be deposited in the gutter and roadway and creates holes in the unpaved surface.
5. In the event that a situation arises where the guidelines are not applicable, the use of sound engineering judgment is recommended.

Design of Curb Ramps

Three standard curb ramp designs were developed for the guidelines: two to accommodate different sidewalk widths for the middle of the curb return and one to accommodate parallel curb ramps. The designs are based on a curb height of 6 in.

Design note 1: Except at certain locations as defined later, curb ramps shall be located on the middle of the curb return. The location on the middle of the curb return provides the minimal potential for conflicts with obstacles such as utility poles, signal poles, and so on.

Design note 2: The curb ramps shall have no lip with a +0.125-in. tolerance.

The standard curb ramp design for sidewalk widths greater than or equal to 8 ft is shown in Figure 5. The slope of the flares is equal to the slope of the ramp (1:12) to permit ramp users to turn left or right by traversing the flares. The ramp is tapered from 4 ft at the bottom to 3 ft at the top (8).

The standard design for sidewalk widths less than 8 ft is shown in Figure 6. The flares and ramp have a slope of 1:10. Many sidewalks in residential areas are 5 ft wide and cannot accommodate a 6-ft-long ramp. The designs in Figures 5 and 6 are similar except for the slopes of the flares and ramp.

The standard design for parallel curb ramps is shown in Figure 7. This curb ramp is used where the ramp is placed parallel to pedestrian paths in locations such as jogged and T intersections, midblock crossings, and medians. The design dimensions are similar to the dimensions in Figures 5 and 6 in that they are based on sidewalk width. Also shown in Figure 7 is the design to be used when the middle of the curb return is unpaved on sidewalks less than 6 ft wide (12).

Figure 5. Standard design for sidewalk widths greater than 8 ft.

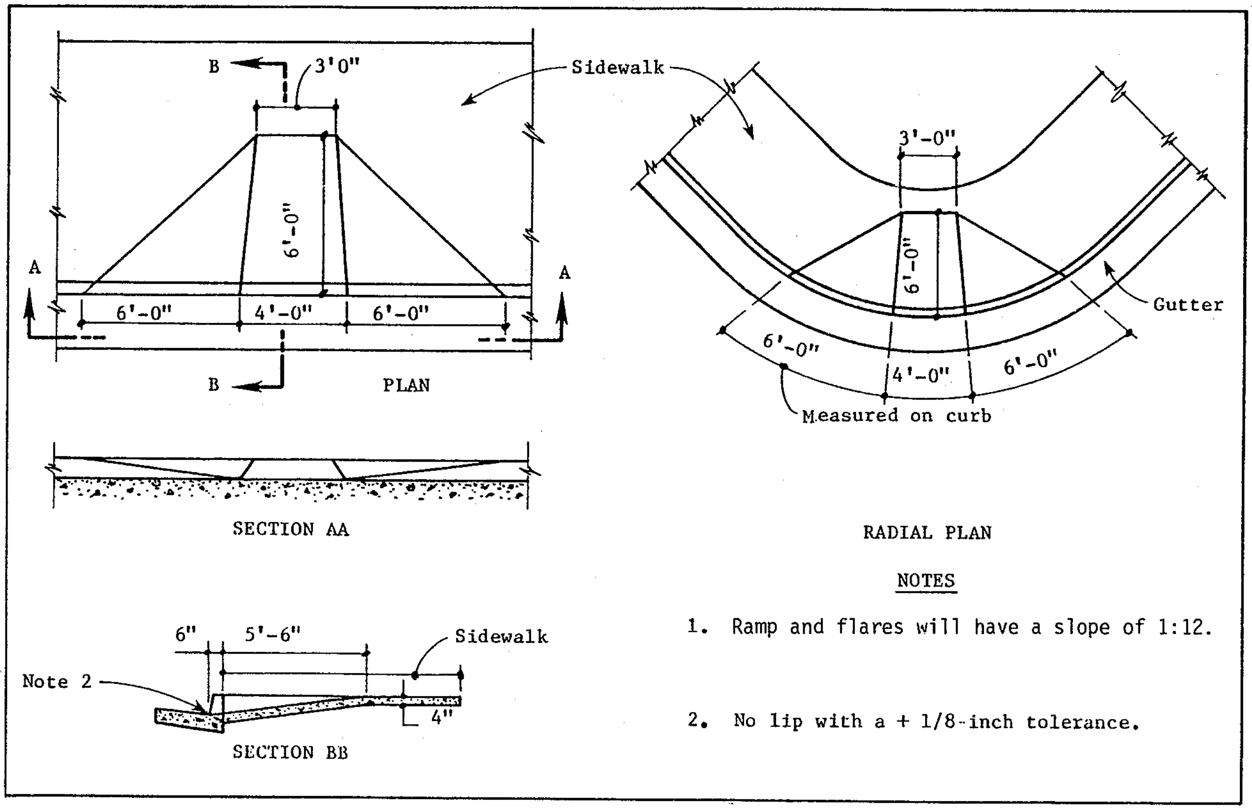


Figure 6. Standard design for sidewalk widths less than or equal to 8 ft.

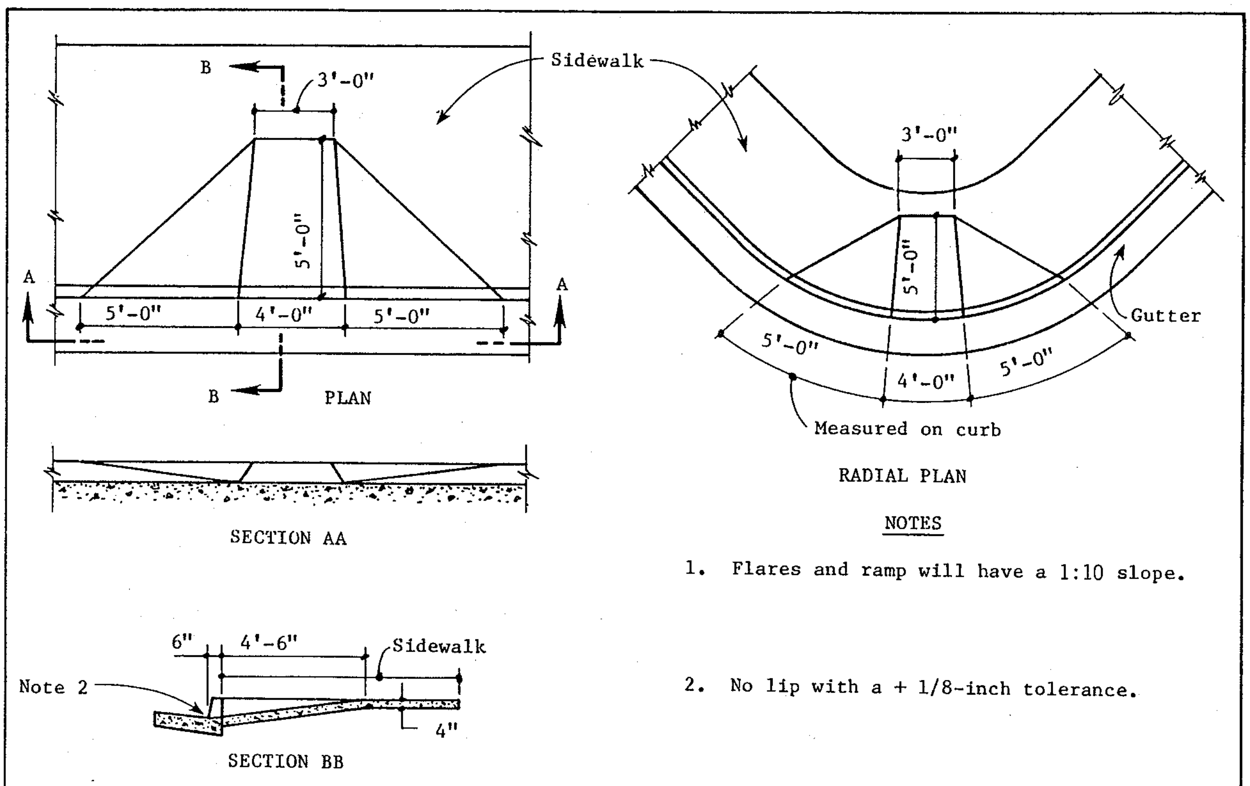
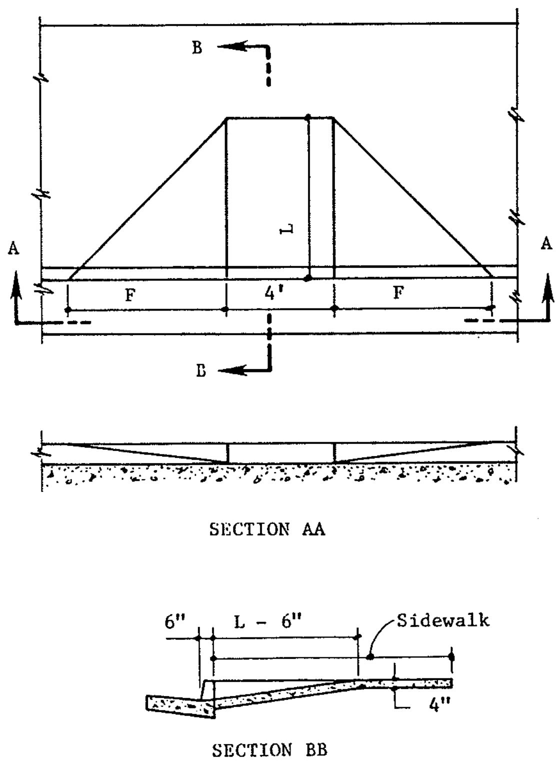


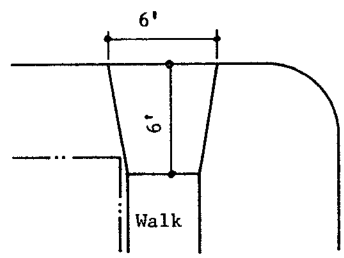
Figure 7. Standard design for parallel curb ramp.



This curb ramp is used for jogged and T intersections, medians, and mid-block ramps.

The design dimensions, based on sidewalk width, are:

Sidewalk Width (ft.)	F (ft.)	L (ft.)
≥ 8	6	6
< 8	5	5



UNPAVED CURB RETURN

If the middle of the curb return is unpaved, use above ramp design.

Placement of Curb Ramps

The placement of curb ramps is as critical to their effectiveness as the design. The three placement issues are placement with respect to obstructions, crosswalks, and intersection types.

Two placement situations, relative to obstructions, are shown in Figure 8. The objective is to maintain consistent and effective placement. When obstructions are located 6 ft or less from the middle of the curb return, the ramp is placed as shown in Figure 8a. It is assumed that the majority of curb ramp users travel in the same directions as the majority of pedestrians. When a drop inlet is located 6 ft or less from the middle of the curb return, the placement of the curb ramp depends on the curb radius (Figure 8b). When there is a radius greater than 20 ft, two parallel ramps are used. The parking restrictions accompanying parallel ramps increase the visibility of curb ramp users to motorists.

Curb ramp placement in conjunction with crosswalks is shown in Figure 9. Where crosswalk markings exist or are planned, curb ramps shall be located within the crosswalks. This may necessitate the widening of a crosswalk. Curb ramps shall be located in front of vehicle stop lines. Crosswalk markings are used to guide pedestrians in the proper paths and are often used where there is substantial conflict between vehicle and pedestrian movements (13). Curb ramp users deserve the same benefits of crosswalks as other pedestrians.

When ramps are located on the middle of the curb return, a minimum of 2 ft of curb shall be located on each side of the ramp for use by the blind and pedestrians who may prefer to use the curb (7). A 4-ft clearance space shall be located within the crosswalk (7). These two items are shown in Figure 9a. The locations of parallel curb ramps relative to crosswalks are shown in Figure 9b. Where crosswalks and walkways through medians are less than 12 ft wide, the curb ramp should be centered in the walk or median (Figure 9c); otherwise the curb ramp should be located to one side with one flare outside of the crosswalk (Figure 9d). Curb ramps in a median should be at least 4 ft apart in order to provide a level section for wheelchair users. When the median is not wide enough to accommodate two curb ramps, a break or gap in the median equal to the width of the crosswalk should be constructed. Parking shall be restricted at least 10 ft (20 ft preferred) from the parallel curb ramps.

Curb ramp placements are presented for oblique angle intersections, multileg intersections, and T and jogged intersections (see Figure 10). Curb ramps on small radii may require that the corner be rounded off to obtain the required 4-ft-wide ramp. The use of oblique angle and multileg intersections is discouraged because they cause problems for the blind, who tend to walk in straight lines perpendicular to the curb.

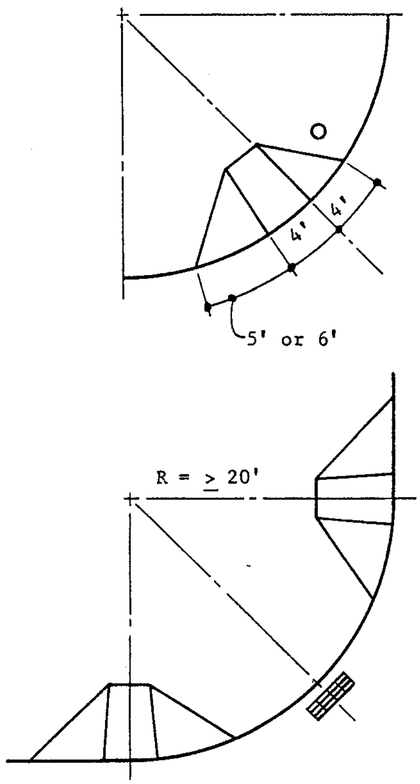
At least one parallel curb ramp should be installed at T and jogged intersections. If one parallel curb ramp is installed, then it should be located in the path of the lightest turning movements from the cross street.

Miscellaneous Notes

Five concerns that deserve mention are curb radius, maintenance of curb ramps, curb height, repavement of streets, and sidewalk slope.

1. Curb radius: The shape of the curb ramp is

Figure 8. Placement relative to obstructions.

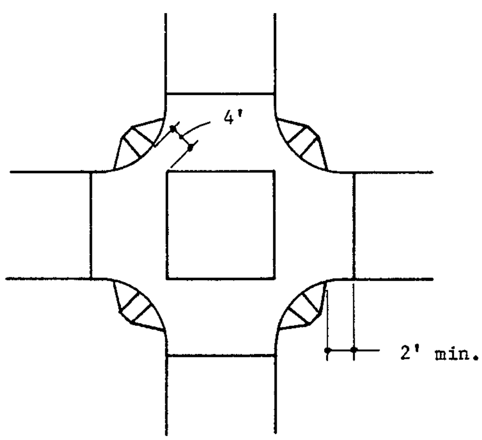


a. If the obstruction is located 0'-6' from the middle of the curb return, offset the ramp in the direction of the major pedestrian movement.

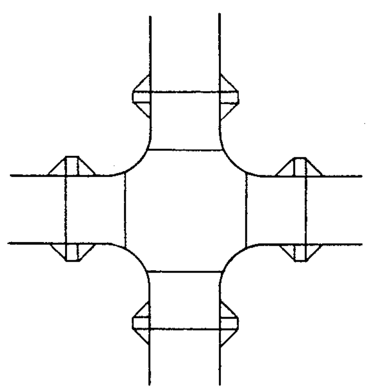
b. If a drop inlet is located 0'-6' from the middle of the curb return with a radius greater than or equal 20', parallel curb ramps should be installed. Parking should be restricted at least 10 ft. (20 ft. preferred) from the curb ramps.

If the curb radius is less than 20', the ramp should be offset in the direction of the major pedestrian movement as in part of this figure.

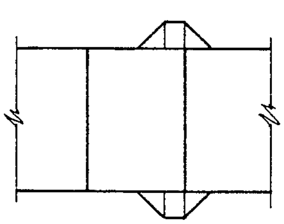
Figure 9. Placement in conjuction with crosswalks.



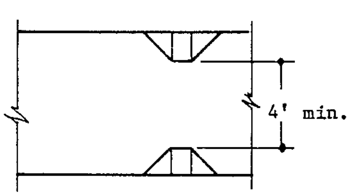
a. Middle of curb return (or diagonal) curb ramps.



b. Parallel curb ramps.



c. Parallel curb ramps located within crosswalks greater than or equal to 12 ft. in width.



Parallel curb ramps in a median. Medians may be made accessible by providing a break in the median or a crosswalk in front of the median.

For crosswalks or medians less than 12 ft. wide, center the ramp in the walk or median.

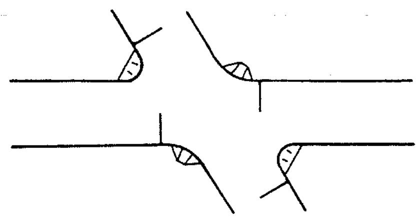
Parking should be restricted within 10 ft. (20 ft. preferred) of the curb ramp.

influenced by the curb radius, as shown in Figure 11. Different curb radii are illustrated to indicate to the engineer that the shape of the curb radius is likely to be different than the shapes shown in Figures 5 and 6.

2. Maintenance of curb ramps: Where there is low or no velocity of the storm runoff water, debris accumulates at the base of the ramp. There is no cost-effective way to overcome this problem from a design and placement perspective. It is recommended that a periodic maintenance schedule be determined by the engineer.

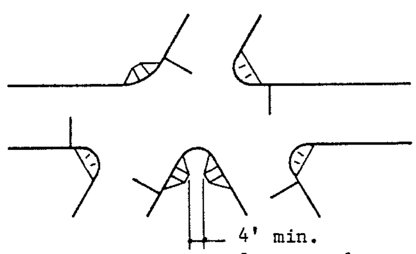
3. Curb height: The design guidelines are based on a curb height of 6 in. In locations where 8-in.

Figure 10. Placement at intersections.



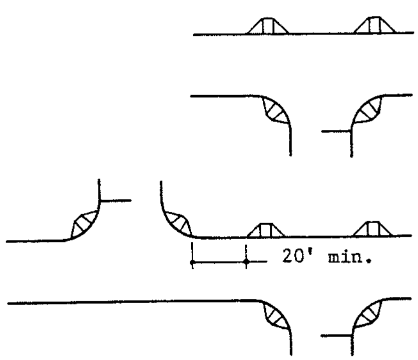
— Curb ramp dimensions may require that the corner be rounded off (4-ft. wide ramp required).

a. Oblique angle intersections.



Note 1. If the spacing between ramps is less than 4', then curb height should be reduced or ramp slope increased to maximum of 10:1. This is similar to a median (Figure 7c).

b. Multi-leg intersections.



T intersection.

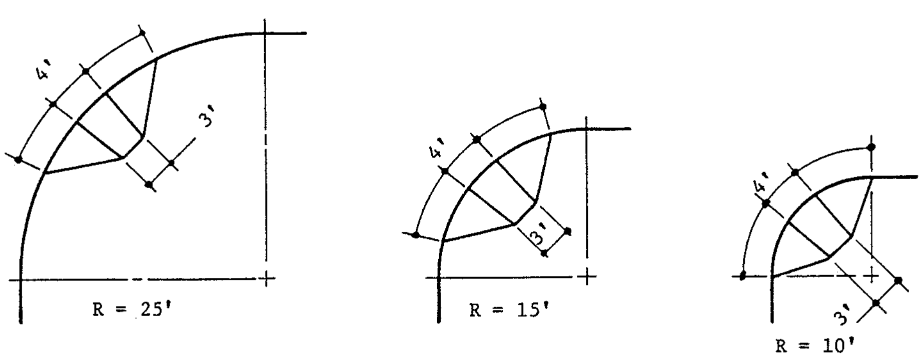
At least one parallel curb ramp should be installed. If one parallel curb ramp is used, then it should be located in the path of the lightest turning movements from the cross street.

Jogged intersection

(The above note is applicable.)

c. T and jogged intersections.

Figure 11. Curb ramps with various curb radii.



curbs are the standard, an asphalt wedge approximately 1 ft long and 2 in. high should be added to the bottom of the ramp if the sidewalk is less than 11 ft wide. Another suggestion is to have sidewalks that slope down (maximum of 1:20) to a 6-in. curb height at the beginning of the ramp.

4. Repavement of streets: Special care should be taken to ensure that the bottom of the curb ramp is not affected when the street is repaved. The city of Charlottesville uses an 8-in. curb (and an asphalt wedge on ramps) so that a 6-in. curb is retained after the street is repaved.

5. Sidewalk slope: Where there is a sidewalk slope to permit drainage from the sidewalk to the curb, the ramp length should be increased to maintain the slope specified in the design. The following equation should be used to calculate the ramp length:

$$L = \{(ECH/RS)/[1 - (SS/RS)]\} + CW \quad (1)$$

where

L = ramp length (ft);
RS = ramp slope;
SS = sidewalk slope;
CW = curb width (ft); and
ECH = effective curb height (ft) = CH − (CW • RS), where CH is the curb height (ft).

CONCLUSION

The guidelines for the design and placement of curb ramps presented in this paper are comprehensive. Curb ramp design dimensions based on sidewalk width and placement relative to obstructions, crosswalks, and types of intersections are addressed.

ACKNOWLEDGMENT

This research was financed from Highway Planning and Research funds administered through FHWA.

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Publication of this paper sponsored by Committee on Geometric Design.

Abridgment

Operational Effects of Two-Way Left-Turn Lanes on Two-Way, Four-Lane Streets

JOHN L. BALLARD AND PATRICK T. McCOY

One method of relieving excessive congestion on a two-way street that has a substantial number of midblock left turns is the construction of a two-way left-turn lane (TWLTL). Although the safety effectiveness of the TWLTL has been the subject of many studies, few studies have been made of its operational effectiveness. The objective of this study was to quantify the effects of a TWLTL on the efficiency of traffic flow on a two-way, four-lane street. By using computer simulation models specifically developed and validated for the purpose of this study, traffic operations were simulated over a range of traffic volumes and driveway densities. The reductions in stops and delays that result from a TWLTL were computed from the outputs of these simulation runs. Isograms of stops and delay reductions were prepared to facilitate the use of the results of this study to evaluate the potential cost-effectiveness of TWLTL installations.

The two-way left-turn lane (TWLTL) is recognized as a possible solution to the safety and operational problems on two-way streets that are caused by the conflict between midblock left turns and through traffic. The extent to which a TWLTL can improve the efficiency of traffic operations depends on the traffic volumes and driveway densities involved. Although the principle of the complex relation between these factors and the operational effectiveness is intuitively apparent, it has yet to be quantitatively expressed for two-way, four-lane streets. McCoy, Ballard, and Wiyaya (1) have reported on the operational effectiveness of a TWLTL on two-way, two-lane streets.

An extensive review of the literature and a nationwide survey of experience with the TWLTL were conducted by Nemeth (2) in developing guidelines for the application of the TWLTL; in most cases it was considered to have noticeably improved the quality of traffic flow. Likewise, in developing guidelines for the control of access on arterial streets, Glennon and others (3) reported that empirical data pertinent to the determination of the operational effectiveness of the TWLTL were lacking.

In response to the need of traffic engineers to be able to more precisely predict the operational effectiveness of a TWLTL and more clearly define those circumstances that justify its installation, a series of studies of the operational effects of a TWLTL were conducted at the University of Nebraska-Lincoln. The objective of these studies was to quantify the effects of a TWLTL on the efficiency of traffic flow on a two-way street. The results of the first study, as reported by McCoy, Ballard, and Wiyaya (1), were limited to the study of a TWLTL on two-way, two-lane streets.

A continuation of the previous study led the Transportation Research Group at the University of Nebraska-Lincoln to study the operational effectiveness of a TWLTL on a two-way, four-lane street. The procedure and findings of this study are reported in this paper.

SIMULATION MODELS

The two computer simulation models developed in this study were written in the general purpose simulation system (GPSS/H) language (4,5). These models are basically the same, except that one is for a two-way, four-lane street with a TWLTL and the other is for a two-way, four-lane street without a TWLTL. A brief description of the input, logic, and output of these models follows.

Input

The input to the models consists of two types of information: traffic characteristics and street geometry. The traffic characteristics are the volume and average speed of traffic in each lane in each direction and the percentage of the traffic volume turning left into each driveway on the street. Because of the nature of the GPSS/H language, the street geometry is defined in terms of sections. Each lane on the street is divided lengthwise into 20-ft sections. Driveway locations and TWLTL entry points on the street are defined by the numbers of the sections in which they are located.

Logic

In both models traffic enters the street segment at either end in accordance with the traffic volumes and arrival patterns specified in the input. A vehicle entering the segment in the curb lane will traverse the length of the segment without turning. A vehicle entering the median lane will take one of three courses:

1. It will traverse the entire segment and exit in the median lane at the other end;
2. If the median lane is blocked, it may at some point move to the curb lane (change lanes) and exit at its ends; or
3. It may traverse a portion of the segment and exit by turning left at one of the driveways.

The course taken by each vehicle entering the segment is determined probabilistically in accordance with the lift-turn percentages specified in the input. In the model without the TWLTL, turning vehicles remain in the median lane until they reach the driveway into which they turn. However, in the model with a TWLTL, a turning vehicle enters the TWLTL, if possible, at the point designated in the model input for the driveway. At this point the speed is reduced and the vehicle proceeds until it reaches the driveway into which it turns, or until it is stopped by a vehicle already in the TWLTL. In both models a turning vehicle must have an acceptable gap (determined probabilistically) in the opposing traffic stream before it can turn left.

Output

The output from the models includes number of vehicles entering and exiting the segment, number of left turns attempted and completed, number of stops, travel time in the segment, stopped-time delay, and number of lane changes. The travel time, stops, and delay totals are output separately for through vehicles in each lane, turning vehicles, and all vehicles.

PROCEDURE

The operational effects of a TWLTL on a two-way, four-lane street were determined in this study by a pair-wise comparison of the outputs from the two models for identical traffic volumes and driveway densities. The two models were used to simulate traffic operations on a street segment (with and without a TWLTL) under four levels of balanced traffic volume in each direction (350, 700, 1,050, and 1,400 vehicles/hr), three levels of left-turn volume in each direction (35, 70, and 105 vehicles/hr), and three levels of driveway density on both sides of the street (30, 60, and 90 driveways/mile). These values were selected as being comparable to the range of levels of volumes and driveway density that were used by Glennon and others (3) in developing guidelines for control of access on two-way, four-lane arterial streets.

The average running speeds used for each traffic volume level were 35 mph for 350 and 700 vehicles/hr, 30 mph for 1,050 vehicles/hr, and 25 mph for 1,400 vehicles/hr. According to the Highway Capacity Manual (6), these speed-volume relations were reasonable for a two-way, four-lane urban arterial street. Only one configuration of driveway locations--one that had evenly spaced driveways throughout the segment and the same number of driveways on each side of the street--was evaluated for each density level because it was beyond the scope of this study to investigate the differences in traffic operations within driveway density levels.

In conducting the computer simulation experiments with each model, the variability was reduced by using common random numbers so that the same traffic flow and gap acceptance sequence was always used for each driveway configuration. Therefore, for a given combination of traffic and left-turn volume levels, the differences in traffic operations were due only to the effects of the driveway configurations and the TWLTL. Every simulation run was initialized by running the model for a few minutes to achieve system stability. Then the model was run for 1 hr of simulated time.

FINDINGS

The reductions in stops per hour and delay in minutes per hour that result from the installation of a TWLTL on a two-way, four-lane street were computed

Table 1. Reduction in stops by driveway density.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/hr)	Reduction in Stops (no./hr) by Left-Turn Volume[c]		
		35 vehicles/ hr/1,000 ft	70 vehicles/ hr/1,000 ft	105 vehicles/ hr/1,000 ft
30	350	8	6	24
	700	13	59	87
	1,050	100	78	599
60	350	5	9	24
	700	17	45	105
	1,050	98	237	–[d]
90	350	5	7	26
	700	12	38	114
	1,050	88	271	589

[a]Total number of driveways on both sides of street.
[b]Volume in each direction, including left turns (divided equally in each lane).
[c]Volume in each direction.
[d]Jammed flow in no-TWLTL case.

Table 2. Reduction in delay by driveway density.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/hr)	Reduction in Delay (min/hr) by Left-Turn Volume[c]		
		35 vehicles/ hr/1,000 ft	70 vehicles/ hr/1,000 ft	105 vehicles/ hr/1,000 ft
30	350	1.5	1.1	2.7
	700	0.8	6.2	8.7
	1,050	8.4	58.6	112.6
60	350	0.2	1.9	5.9
	700	0.8	4.6	23.9
	1,050	6.8	36.3	–[d]
90	350	0.4	0.9	4.1
	700	0.4	3.0	22.2
	1,050	5.8	44.6	142.3

[a]Total number of driveways on both sides of street.
[b]Volume in each direction, including left turns (divided equally in each lane).
[c]Volume in each direction
[d]Jammed flow in no-TWLTL case.

by a pair-wise comparison of the outputs from the two simulation models. These reductions are given in Tables 1 and 2. The data in these tables reveal that in no case did the TWLTL increase stops and delay, and in every case there were reductions in stops and delay. As expected, the amounts of these reductions increased within each level of driveway density as the traffic volume was increased; in every case but two the amount of these reductions increased within each level of driveway density as the left-turn volumes were increased.

The data in Tables 1 and 2 indicate that the effect of driveway density within each level of traffic volume was not consistent over the range of traffic and left-turn volumes. The lack of a driveway density effect within each left-turning volume is best explained by the fact that the left-turning volume was spread evenly across each driveway; therefore, for the 90 driveways/mile case, approximately one-third less left-turning volume existed at each driveway than in the 30 driveways/mile case. This leads to more queuing at left turns in the 30-driveway case. Also, vehicles waiting to turn left at several driveways will make use of the same gap in the oncoming traffic stream. Hence the effect of driveway density in the amount of reduction in stops and delay is minimized. Similar findings are reported by McCoy, Ballard, and Wiyaya (1) for the two-way, two-lane streets.

The reductions in stops and delay determined in this study provide a basis for evaluating the effectiveness of a TWLTL on a two-way, four-lane street from the standpoint of user costs, energy consumption, and air quality. The reduction values for stops and delay are directly applicable to procedures for evaluating traffic engineering improvements, such as the procedure outlined by Dale (7). Therefore, to facilitate this application of the results of this study, isograms of the reductions in stops and delay were constructed from the data in Tables 1 and 2. The stops-reduction and delay-reduction isograms for 60 driveways/mile are shown in Figures 1 and 2.

Figure 1. Reductions in stops for 60 driveways/mile.

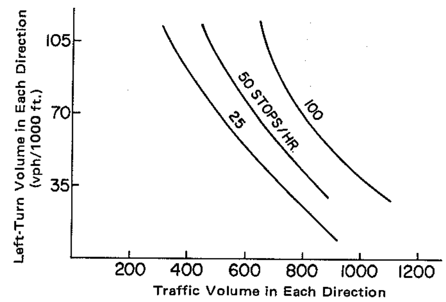
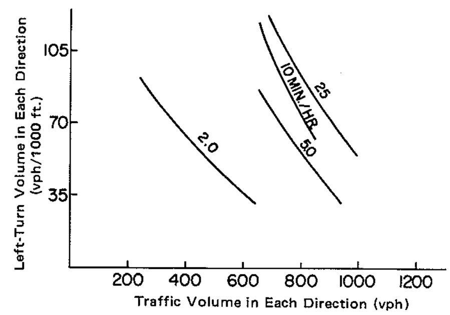


Figure 2. Reductions in delay for 60 driveways/mile.



CONCLUSIONS

Based on the findings of this study it is concluded that the installation of a TWLTL on a two-way, four-lane street improves the efficiency of traffic operations over a wide range of traffic volumes, left-turn volumes, and driveway densities. Under balanced traffic flow conditions a TWLTL is particularly effective at traffic volumes greater than 700 vehicles/hr in each direction with more than 70 midblock left turns per 1,000 ft from each direction.

The findings of this study are similar to those of a previous study (1) of the operational effects of two-way left-turn lanes on two-way streets. The pattern of reductions in stops and delay were similar; however, the magnitude of the reductions was greater for two-lane roads than for four-lane roads.

The stops- and delay-reduction isograms developed in this study facilitate the quantitative evaluation of the operational effectiveness of a TWLTL under balanced traffic flow conditions on a two-way, four-lane street. When used within the context of a cost-effectiveness analysis, these isograms contribute to the identification of the circumstances under which the installation of a TWLTL on a two-way, four-lane street would be justified.

This study is only a start; the need for further research is obvious. Additional studies need to be conducted for more levels of traffic volume, left-turn volume, and driveway density. Further studies should address unbalanced as well as balanced traffic flow conditions, and the effects of driveway configuration need to be evaluated.

ACKNOWLEDGMENT

This research was funded by the Engineering Research Center at the University of Nebraska-Lincoln. This study would not have been completed without the efforts of Chuck Mann, John Goble, Yaya Wiyaya, and Chris Shuptar.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Functional Analysis of Stopping-Sight-Distance Requirements

TIMOTHY R. NEUMAN, JOHN C. GLENNON, AND JACK E. LEISCH

A basic highway design concept is that the driver should be provided a sufficient visible length of highway to enable collision avoidance. Translating this concept to appropriate standards and criteria is an important design consideration. The concept of safe stopping-sight distance (SSD) as developed by AASHTO is reviewed and discussed. A functional SSD model is offered as a means of demonstrating shortcomings and inconsistencies in AASHTO design policy. In addition, the geometry of SSD is evaluated through the use of sight-distance profiles. Significant conclusions are presented that relate to SSD design values on horizontal curves and special problems with trucks on horizontal curves. The functional SSD model is helpful in understanding accidents at locations that have inadequate SSD.

Stopping-sight distance (SSD) is an important highway design feature. The concept of providing a sufficient length of highway visible to the driver for collision avoidance is basic to the safe design of highways. However, translating this concept to design standards and criteria is not as simple as it may appear.

A critical review of current design practice for SSD is presented in this paper. The concepts and conclusions presented are drawn from a study of SSD conducted for FHWA as a part of a research project entitled, "Effectiveness of Design Criteria for Geometric Elements."

A concept of SSD that focuses on highway operational requirements has been developed. Shortcomings and inconsistencies in AASHTO design policy are revealed by applying this operational SSD concept. Also, by using sight-distance profiles, additional insights are gained on the relation between sight distance and highway safety.

OPERATIONAL AND SAFETY CONCEPT OF SSD

Analysis of the operational and safety aspects of SSD requires an understanding of the concept of SSD as it relates to highway operations. The geometric design policy published by AASHTO discusses the need for SSD (1-3):

> If safety is to be built into highways the designer must provide sight distance of sufficient length in which drivers can control the speed of their vehicles so as to avoid striking an unexpected obstacle on the traveled way....
>
> The minimum sight distance available on a highway should be sufficiently long to enable a vehicle traveling at or near the likely top speed to stop before reaching an object in its path. While greater length is desirable, sight distance at every point along the highway should be at least that required for a below average operator or vehicle to stop.

This short discussion alludes to many of the operational elements of SSD: vehicle performance, driver ability, and the roadway alignment. This AASHTO operational model of SSD provides a reasonable starting point for considering SSD and highway operations.

AASHTO SSD Operational Model

AASHTO defines minimum SSD requirements in terms of a passenger car encountering a stationary object in its path. This basic functional model has not changed since 1940. The following review of the evolution of AASHTO SSD policy illustrates the reasoning behind this model. It also demonstrates the need to go beyond this simple abstraction to gain insight on the safety relations of SSD.

In 1940 AASHO formally recognized the need for a sight-distance requirement to help drivers avoid collision circumstances other than passing encounters. Although AASHO recognized that a clear sight line to the pavement was desirable, analyses of how this requirement affected construction cost led to a compromise. A design object height of 4 in. was selected on the basis of optimizing the relation between object height and required vertical curve length. Although the object-height criterion is discussed in the AASHO policy as it relates to objects in the road, the selection of a 4-in. height was not based on the frequency or severity of such objects. This conclusion is further borne out by subsequent changes in AASHO policy to a 6-in. object height; exactly the same discussion was used in relating this height to roadway events.

Selection of other design parameters such as perception-reaction time, eye height, and pavement friction was rational; individual design values were selected based on the existing distributions of these physical values, which were periodically updated, as indicated by the data in Table 1. Yet the underlying methodology was by design an abstraction--a simplified set of elemental factors used to derive a distance--with only an indirect link to the functional needs for sight distance.

Functional Elements of Concern in SSD

It is suggested that attention should be focused on the functional requirements for SSD, which vary depending on a range of factors. SSD requirements are a function of more than a single object height, eye height, or pavement condition. There are many common types of collisions (rear-end, head-on, impacts with large animals, and so on) for which a 4- or 6-in. object bears little or no relation. In addition, accident experience strongly suggests links between geometry (independent of that which produces restricted SSD) and accident causation. Such links are not sufficiently treated in the current AASHTO SSD methodology.

Four factors that contribute to the requirements for SSD are shown in Figure 1. These factors form the basis for a functional SSD model.

Highway Events

A range of common events on the highway creates the

Table 1. Evolution of AASHTO SSD policy.

Year	Design Parameters: Eye Height (ft)	Object Height (in.)	Perception-Reaction Time (sec)	Assumed Tire-Pavement Coefficient of Friction	Assumed Speed for Design	Effective Change from Previous Policy
1940	4.5	4	Variable—3.0 sec at 30 mph to 2.0 sec at 70 mph	Dry—f ranges from 0.50 at 30 mph to 0.40 at 70 mph	Design speed	
1954 (1)	4.5	4	2.5	Wet—f ranges from 0.36 at 30 mph to 0.29 at 70 mph	Lower than design speed (28 mph at 30 mph design speed; 59 mph at 70 mph design speed)	No net change in design distance
1965 (3)	3.75	6	2.5	Wet—f ranges from 0.36 at 30 mph to 0.27 at 80 mph	Lower than design speed (28 mph at 30 mph design speed; 64 mph at 80 mph design speed)	No net change in design distance
1970 (4)	3.75	6	2.5	Wet—f ranges from 0.35 at 30 mph to 0.27 at 80 mph	Minimum values—same as 1965; desirable values—design speed	Increase in SSD of up to 250 ft at 70 mph
1983, proposed (5)	3.50	6	2.5	Wet—f ranges from 0.35 at 30 mph to 0.27 at 80 mph	Minimum values—same as 1965; desirable values—design speed	No net change from 1970

Note: 1 ft = 0.305 m, 1 in. = 25.4 mm, and 1 mph = 1.609 km/h.

Figure 1. Functional relations of SSD.

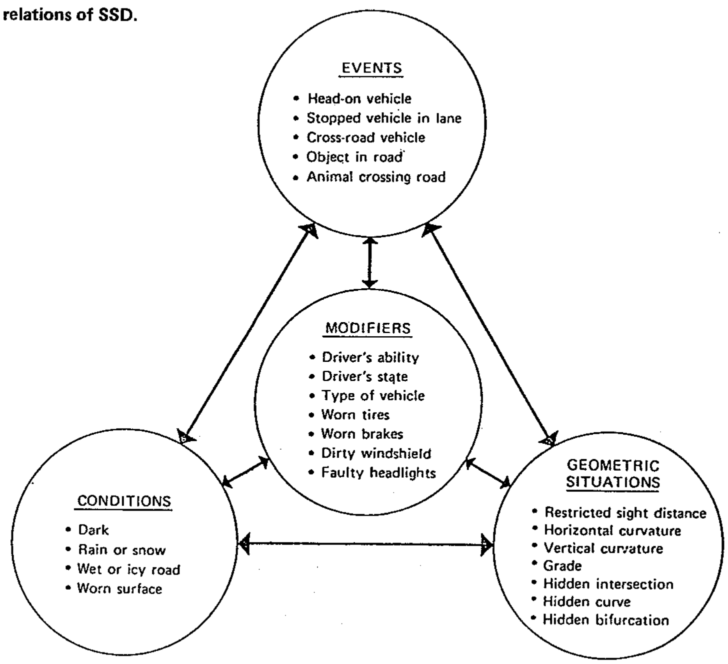


Table 2. Roadway events related to SSD conflicts.

Type of Event	Frequency of Occurrence	Severity of Conflict or Impact	Type of Event	Frequency of Occurrence	Severity of Conflict or Impact
Two-lane rural highway			Urban arterial		
Object in road			Object in road		
Large animal	Variable; generally infrequent	Severe	Large animal	Very infrequent	Severe
Road debris	Infrequent	Minor to moderate	Road debris	Infrequent	Minor
Rocks	Infrequent	Minor	Rocks	Very infrequent	Minor
Small animal	Occasional	Minor to moderate	Small animal	Infrequent	Minor
Icepatch	Infrequent	Minor to moderate	Icepatch	Infrequent to occasional	Moderate
Pothole, washout	Infrequent	Minor	Pothole, washout	Occasional	Minor to moderate
Vehicle in road			Vehicle in road		
Head-on	Very infrequent	Very severe	Head-on	Infrequent	Very severe
Rear-end	Frequent	Severe	Rear-end	Frequent	Moderate to severe
Crossing	Occasional	Severe	Crossing	Frequent	Severe
Pedestrian or bicyclist	Very infrequent	Very severe	Pedestrian or bicyclist	Frequent	Very severe
Rural freeway			Urban freeway		
Object in road			Object in road		
Large animal	Variable; generally infrequent	Severe	Road debris	Frequent	Moderate
Road debris	Infrequent	Moderate	Small animal	Very infrequent	Moderate
Rocks	Infrequent	Moderate	Icepatch	Infrequent	Moderate to severe
Small animal	Infrequent	Moderate	Pothole, washout	Infrequent	Moderate to severe
Icepatch	Infrequent	Minor to moderate	Vehicle in road, rear-end	Frequent	Moderate to severe
Pothole, washout	Infrequent	Minor to moderate	Pedestrian	Very infrequent	Very severe
Vehicle in road, rear-end	Infrequent	Very severe			
Pedestrian or bicyclist	Infrequent	Very severe			

need for sight distance in order to avoid an accident. These events include the AASHTO stationary object in road as well as moving objects (head-on vehicles, crossing vehicles, large animals). The significance of these events with respect to sight distance and safety can be judged by (a) the frequency of occurrence, and (b) the criticality of a potential collision or accident given the event.

The data in Table 2, which summarize common critical events that occur on rural and urban highways, reveal two important concepts. One is that the frequency (and, in some cases, severity) of an event is related to the type of highway. Vehicle-crossing conflicts are clearly a serious problem on two-lane rural highways, but not on freeways. Similarly, the higher speeds prevalent on freeways result in more serious consequences given an encounter with potholes or road debris than is expected on urban arterials. The data in Table 2 also indicate that the proper focus is on frequent or severe events in designing for adequate SSD.

Highway Geometry

The geometry of the highway has a clearly definable effect on SSD requirements. Its primary effect relates to vehicle braking requirements. The following sections discuss the effect of both grades and horizontal curvature on vehicle braking.

Effect of Grades on Stopping Distance

AASHTO policy currently recognizes the effect of grades on vehicle braking distance and, ultimately, the required SSD, as follows:

$$\begin{aligned} SSD &= d_{P/R} + d_B \\ &= 0.278\,Vt_{P/R} + \{V^2/[255(f_B \pm G]\} \end{aligned} \quad (1)$$

where

$d_{P/R}$ = distance traveled during perception reaction time by the driver (m),
d_B = distance traveled while vehicle is braking (m),
V = design speed (km/h),
$t_{P/R}$ = perception-reaction time (sec),
f_B = AASHTO coefficient of braking friction, and
G = percent grade ÷ 100.

The incremental effect that steeper downgrades have on required braking distances is substantial at high speeds. A vehicle traveling on a 6-percent downgrade at 80 km/h requires 21 m of additional braking distance; at 115 km/h, 49 m of additional distance is required.

Effect of Horizontal Curvature on Stopping Distance

AASHTO SSD policy currently does not recognize the complications to vehicle stopping ability caused by horizontal curvature. Such complications result from the AASHTO assumption that full (design) pavement friction is available to a vehicle forced to brake in an emergency situation. (Recall that design values for braking friction were selected by AASHTO from actual pavement friction values measured from skid tests.) Vehicles traveling on horizontal curves, however, do not have full friction available for braking, but instead have a reduced amount because of the friction already used by the vehicle in cornering.

The data in Figure 2 demonstrate that the friction available for braking on curves is that vector resultant of both available friction and cornering demand. Mathematically, this is given as

$$f_B' = \sqrt{f_B^2 - f_C^2} \quad (2)$$

where

f_B' = coefficient of braking friction available on curve,
f_B = coefficient of braking friction on tangent (AASHTO design values), and
f_C = coefficient of side friction demand on curve (AASHTO design values).

Obviously, longer stopping distances on curves are indicated by this equation. These greater stopping distances are particularly significant at higher speeds, as indicated by the data in Table 3.

Figure 2. Friction requirements for stopping on horizontal curves.

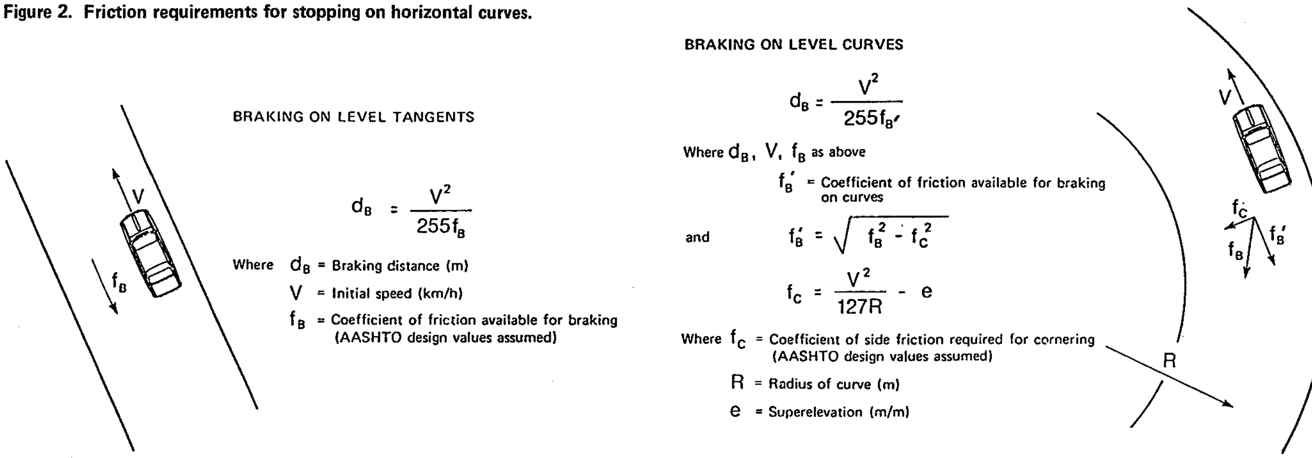


Table 3. SSD requirements for passenger cars on curves (e_{max} = 0.10).

Design Speed (km/h)	Perception-Reaction Distance (m)	Braking on Tangents (wet conditions)			Braking on Curves (wet conditions)			
			Distance (m)				Distance (m)	
		f^a	Braking	Total	f^b	f_c	Braking	Total
50	35	0.347	28	63	0.308	0.159	32	67
60	42	0.328	43	85	0.290	0.153	48	90
70	49	0.313	61	110	0.276	0.147	70	119
80	56	0.301	83	139	0.266	0.140	94	150
90	63	0.294	108	171	0.261	0.134	122	185
100	69	0.288	136	205	0.256	0.128	152	221
110	76	0.282	168	244	0.254	0.122	187	263
120	83	0.275	205	288	0.250	0.115	226	309

[a] $f = f_B$ (AASHTO design values).

[b] $f = \sqrt{f_B^2 - f_c^2}$; f_c = cornering friction required at design speed on controlling curve (AASHTO design values).

Even greater braking distances are required on horizontal curves if the design event is further defined in terms of driver behavior. Studies by Glennon and Weaver (6) and ongoing research under the FHWA contract ("Effectiveness of Design Criteria for Geometric Elements") have indicated that a large proportion of vehicles corner on horizontal curves at path radii significantly shorter than the roadway radius. This sharper cornering requires even greater side friction, thereby further reducing the available friction for braking on the pavement. Therefore, the effect of horizontal curvature on SSD requirements can be considerable.

Combined Effect of Downgrade and Horizontal Curvature on SSD

When drivers encounter combinations of severe grades and controlling horizontal curvature, SSD requirements are much greater than the basic AASHTO values. The combined effect of grades and curvature on SSD are given in Table 4.

Other Geometric Features

The total geometric character of the highway has an effect on safe SSD outside of that quantifiable in terms of braking requirements. Although current AASHTO policy does not explicitly handle this issue, it is clear that certain geometric elements produce especially greater hazards in combination with minimum SSD. Such elements include intersections or driveways, bifurcations, hidden horizontal curves, narrow structures, and railroad crossings. These features partly relate to the highway events previously discussed. They also relate to basic assumptions about driver behavior. Adequate perception-reaction time for collision avoidance is undoubtedly more critical for situations that involve these geometric features.

Table 4. SSD requirements on combined grades and curves for passenger cars (wet conditions).

Design Speed (km/h)	SSD on Tangent (m)	SSD (m) on Controlling Curve with Grade of		
		0 Percent	3 Percent	6 Percent
50	63	67	70	74
60	85	90	96	103
70	110	119	127	138
80	139	150	162	177
90	171	185	200	221
100	205	221	241	268
110	244	263	288	321
120	288	309	340	381

Environmental Conditions

A third aspect of the operational model for SSD is the set of environmental conditions that affect driver and vehicle behavior, the most important of which is pavement condition. AASHTO policy currently accounts for the lower friction provided by wet pavements by assuming wet conditions in the development of design requirements. Other important environmental questions relate to visibility and its effect on the perception-reaction process by the driver. Decreased visibility during rain, snow, and night conditions create sight-distance restrictions because of the limitations of vehicle headlight systems.

Modifying Factors

SSD operational requirements are also influenced by a variety of modifiers, which relate to the performance of both driver and vehicle. The perception-reaction ability of drivers is a direct input to SSD. Current AASHTO policy assumes a worse-than-average driver in establishing a design value for perception-reaction time. However, no variability is indicated in perception-reaction time for the range of events and conditions confronted by drivers.

Vehicle characteristics also play a major role in the design for SSD. Braking distances are a function of vehicle type, tire condition, and brake conditions. Vehicle type is the most important characteristic; trucks require much greater stopping distances than do passenger cars. The eye height of the driver is also a function of the vehicle. This dimension is critical in establishing the sight line from the driver to an object in the road over a crest vertical curve.

AASHTO policy treats the multitude of vehicle characteristics in a cursory manner. Basic SSD design values are a function solely of passenger car braking ability and eye heights of passenger car drivers. Only passing reference to SSD requirements for trucks is made. This is justified by noting that the greater eye heights (and hence longer sight lines) afforded truck drivers tend to balance out the greater truck braking distances.

Nevertheless, a variety of geometric conditions can negate the advantages of greater eye heights for truck drivers. Horizontal sight obstructions (e.g., retaining walls, rock cut, tree lines) restrict the view ahead from trucks and passenger cars equally. Furthermore, a complete functional analysis of such situations reveals a significant inconsistency in AASHTO SSD design policy. As discussed earlier, braking distance requirements on curves are greater than the requirements provided by AASHTO policy. Thus SSD restrictions along horizontal curves present particularly severe problems to trucks. Their greater braking distances, loss of eye-height advantage, and friction demands for cornering contribute to much greater SSD requirements than that indicated by AASHTO design policy.

GEOMETRICS OF SSD

The importance of SSD relative to other highway features can be estimated only after understanding how SSD restrictions are created. A study of the frequency and types of sight-distance restrictions on the highway provides further meaning to the operational model presented previously.

AASHTO recognizes two basic types of SSD restrictions: horizontal and vertical. The following discussion considers the character of these restrictions with horizontal curvature, vertical curvature, grades, and the presence of obstructions adjacent to the traveled way.

Vertical Alignment and SSD

Crest vertical curves restrict available SSD whenever the approach grades are steep, the vertical curve is short, or both. Current AASHTO minimum standards for lengths of vertical curves are based on a combination of design speed and the algebraic difference in the grades (A). The minimum length of vertical curve produces minimum SSD at the assumed design speed.

The salient characteristic of vertical curves to consider in a study of SSD is its distribution throughout the vertical curve. A common misconception is that the minimum SSD provided by a vertical curve is manifest over the entire length of curve. Nevertheless, a plot of SSD along the vertical curve (referred to as a sight-distance profile) reveals SSD decreasing to a minimum value and then rapidly increasing as the vehicle reaches the crest of the curve.

SSD profiles are useful because they reveal the relations among vertical curve length, grades, and SSD. SSD profiles for the range of typical values of A (difference in grade) are shown in Figure 3. Inspection of these profiles reveals three basic characteristics of SSD on crest vertical curves.

1. Vertical curves that create limited SSD do so over relatively short lengths of highway. Similarly, less severe SSD limitations affect longer sections of highway.
2. The length of highway over which SSD is at a minimum is relatively short compared with the length of a vertical curve.
3. Different combinations of grades with the same A have similar lengths of highway at which SSD is at a minimum.

Horizontal Alignment and SSD

SSD restrictions are also created by a combination of horizontal curvature and roadside obstacles or features that obstruct the driver's vision to the pavement ahead. AASHTO policy calls for minimum offsets from such obstacles to the edge of pavement. These requirements are a function of the design speed of the roadway and the curve radius. For example, the AASHTO minimum offset for a 440-m radius curve at 115 km/h is 7.6 m from the edge of a 3.65-m lane.

However, as was discussed earlier, braking requirements on curves are greater than the requirements provided for by AASHTO. These greater braking distances necessitate much greater offsets to roadside obstacles. For example, the same 440-m radius at 115 km/h would require 24 m of offset rather than 7.6 m. Consideration of such great offset requirements is important given that a wide range of conditions and features (buildings, cut slopes, rock cuts, retaining walls, trees, and so on) exist, which create horizontal SSD restrictions. As with vertical SSD restrictions, the character of SSD varies in each case.

SSD profiles are also useful in evaluating the character of SSD on horizontal curves. Consider the SSD profiles for a 440-m-radius curve with different obstructions on the inside (Figure 4). In both cases a sight restriction occurs 7.6 m from the edge of pavement along the curve. The resulting minimum SSD is 183 m. In case A the obstruction is a point (e.g., corner of a building); in case B the obstruction is continuous throughout the curve (e.g., retaining wall, row of trees, or vertical cut slope). The difference in SSD profiles for the two cases is apparent. Minimum SSD in case A is limited to a relatively short length of highway compared with the entire length of curve for case B. (For comparison, also note the required SSD based on the braking on curve operational criterion developed earlier.)

Horizontal SSD restrictions have certain significant characteristics that differ from vertical SSD restrictions.

1. The sight-distance restriction is usually unidirectional; except for extreme restrictions it differs in the direction of travel between the inner lane and the second or outer lane. Generally, only

Figure 3. SSD profiles for vertical curves.

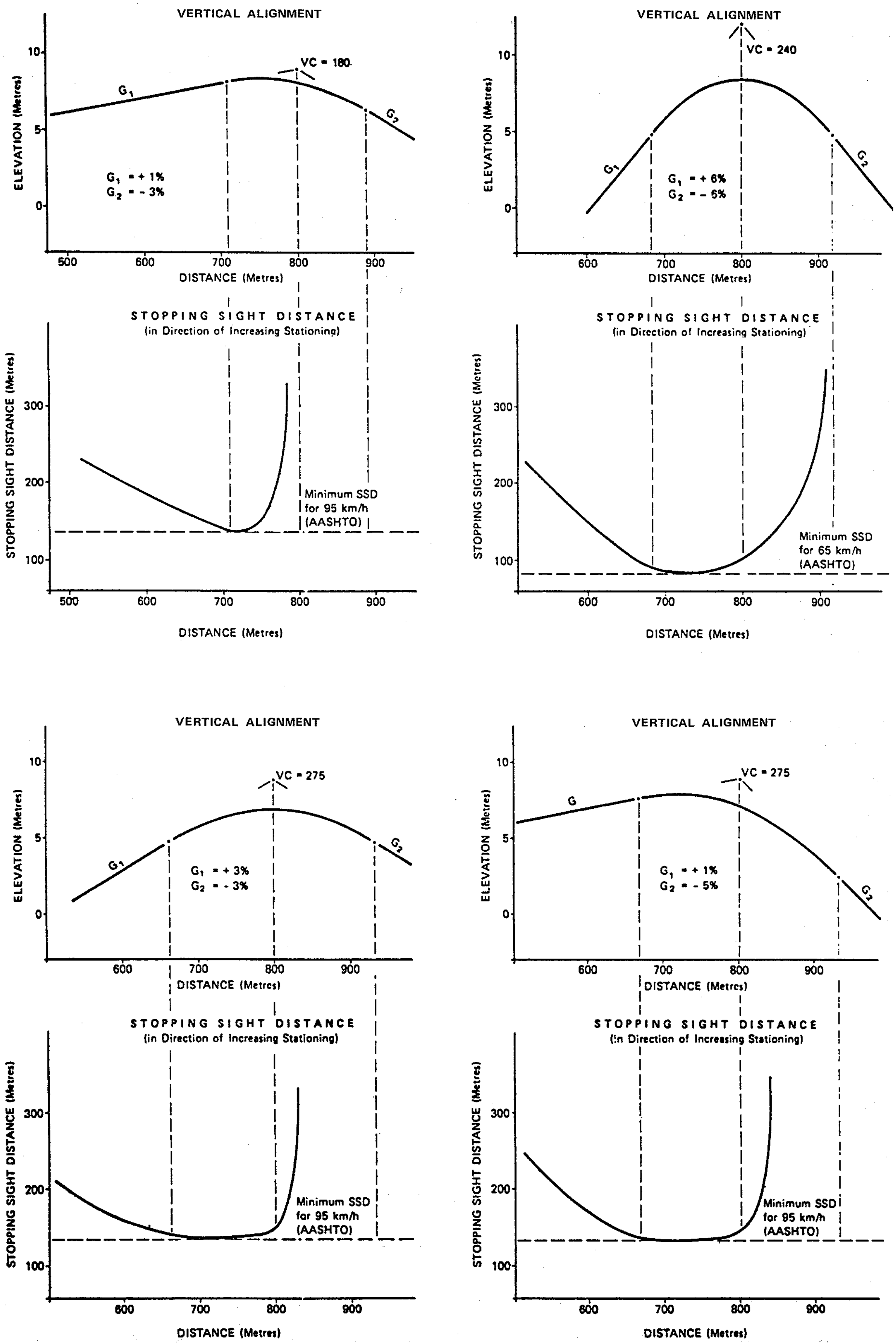
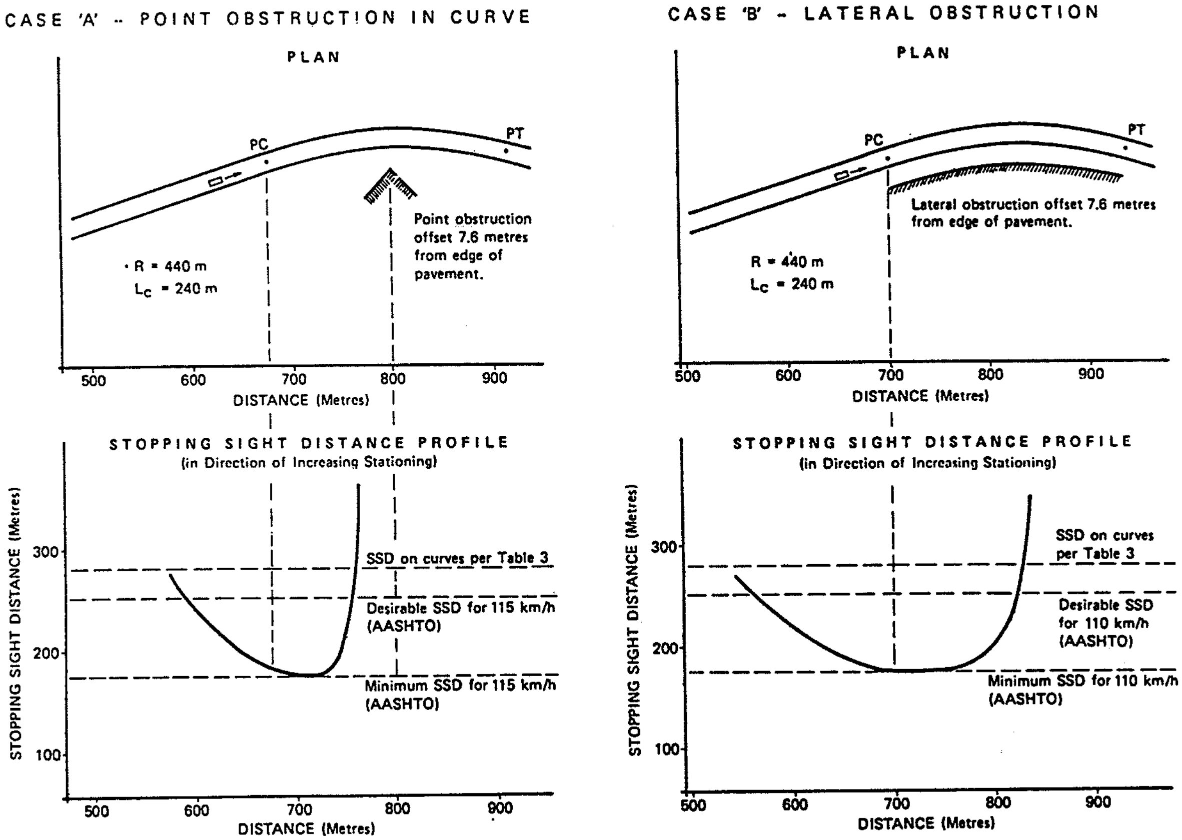


Figure 4. SSD profiles for horizontal curves.



vehicles traveling in the inside lane are subjected to the greatest restriction. Vehicles in the outside lane have an additional lane of lateral offset, which increases available SSD for these vehicles.

2. In some cases (e.g., near vertical obstructions caused by retaining walls, rock cuts, buildings, or rows of trees) driver eye and object heights are not factors in determining SSD.

Point 1 reveals a significant aspect of horizontal SSD restrictions. Because for most conditions the traffic exposure to the sight-distance deficiencies is unidirectional, any accident experience related to the restriction may be a function of only one-half the average daily traffic (ADT) of a two-way roadway. Point 2 provides insight on specific accident problems that involve trucks. The cumulative effect of greater braking distances for trucks, additional requirement for braking on curves (possibly combined with a downgrade that has an additional braking requirement), and loss of benefit from greater eye height indicates a particular vulnerability of trucks to this type of SSD restriction.

SUMMARY OF FUNCTIONAL ASPECTS OF SSD

Analysis of the functional requirements for SSD focuses on the types of accidents and hazardous situations that result from limited SSD. The following points are useful in understanding the link between SSD and safety.

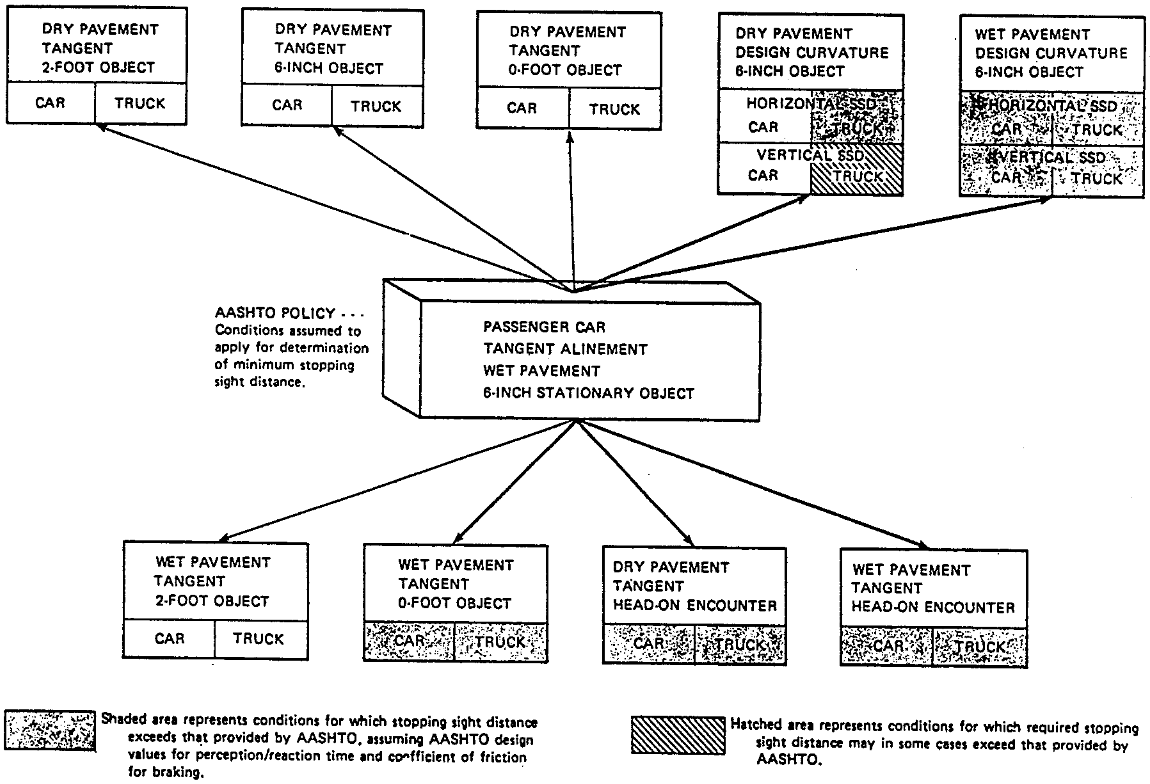
1. SSD accidents are event oriented: The mere presence of a segment of highway with inadequate SSD does not guarantee that accidents will occur. SSD-related accidents occur only after an event(s) creates a critical situation. These events can take the form of arrivals of conflicting vehicles, the presence of objects on the road, inadequate visibility, or unsatisfactory road surface conditions. Some of these events are a function of the highway type (e.g., crossing conflicts at intersections do not occur on freeways), some are related to other geometric or environmental elements (e.g., requirement for severe cornering maneuver on wet pavement), whereas others may be totally random (e.g., presence of an object in the road).

2. The probabilities of critical events occurring within the influence of SSD restrictions define the relative hazard of these restrictions: The relative hazard of various SSD-deficient locations can be estimated by examining the probabilities of critical events. Traffic volume, frequency of conflicts (rear-end, head-on, crossing, object in road), and time exposure of each vehicle to the restricted SSD are all useful in estimating these probabilities.

3. Severity as well as frequency is significant: SSD situations that create severe although infrequent conflicts (e.g., head-on, angle collisions) may be as important as situations with frequent but less-severe conflicts. Cost-effectiveness analysis rightfully values injuries and fatalities forestalled much higher than property-damage-only accidents.

4. Many uncontrollable or unquantifiable factors also contribute to accident causation: Driver performance characteristics such as perception-reaction time, vehicle characteristics such as braking ability, and certain imponderables such as the driver's state of mind contribute to increased accident potential. Although these factors are exclusive of the presence of a SSD-deficient location, their importance is undoubtedly heightened when the deficiency in SSD means that the driver has less

Figure 5. Analysis of functional requirements for SSD on two-lane highways.



time to react to an event. This reduced time may make the difference between collision avoidance and an accident.

The complexity of SSD requirements when viewed as a function of all the elements discussed earlier is shown in Figure 5. Current AASHTO policy, which defines SSD requirements based on only one event and a set of conditions, produces sufficient SSD for certain events or conditions but not for others.

CONCLUSIONS

There is currently great interest in the effects of smaller passenger cars on eye heights and SSD. To date such interest within the traffic engineering and design profession has focused on the traditional parameters associated with SSD: eye height, object height, and perception-reaction time.

It is believed that a broader perspective is necessary when considering SSD requirements. A framework for evaluating such requirements is presented that is based on the functional aspects of SSD. SSD is described in terms of (a) the types and frequencies of conflicts or events that occur on the highway, (b) the geometry of the highway, (c) the environmental conditions, and (d) the variable performance capabilities of drivers and vehicles.

When viewed in terms of these four elements, SSD is revealed as being much more complex than the AASHTO object-in-road model. The inadequacy of the AASHTO model is illustrated by considering the particular problems for trucks with horizontal sight obstructions on curves. Indeed, current AASHTO policy was revealed as being inconsistent for all vehicles encountering limited SSD on horizontal curves. Cornering friction requirements are not included in SSD design policy, even though they are an integral feature in design policy for horizontal curves.

Application of the functional model for SSD revealed a range of situations for which current design standards are inadequate. What implications does this finding have for SSD design policy? It is clearly impossible to design for all situations, and it is not suggested that such a design policy is even desirable. Nevertheless, given the functional model presented here, it appears evident that a fresh look at SSD design policy may be fruitful. It may be appropriate to consider variable facility types in SSD design. A more explicit consideration of other geometric elements such as curvature also appears appropriate. Although comprehensive analyses of all situations were not possible given the research scope, it is believed that sufficient direction is provided for further research.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Abridgment

Accident Analyses for Highway Curves

TIMOTHY R. NEUMAN, JOHN C. GLENNON, AND JAMES B. SAAG

The results of studies of accidents and roadway geometrics on two-lane rural highways are presented. The studies were a portion of federally sponsored research on the safety and operations of highway curves. A data base was assembled from geometry files of four states. Two sets of analyses were performed: (a) a multivariate analysis of the incremental accident effects of five basic geometric and traffic variables, and (b) a detailed study of the geometric and environmental characteristics of site populations with high- and low-accident rates. The study findings demonstrated that degree of curve, extent of roadside hazard, and pavement surface quality (i.e., available friction) have the greatest impact on safety of two-lane rural highway curves. Other notable effects were observed with shoulder width, roadway width, and length of curve.

The results of studies of accidents and roadway geometrics on two-lane rural highways, performed as a part of FHWA-sponsored research, are presented in this paper. Two separate accident analyses were undertaken: analysis of covariance was used to study the incremental accident effects of basic geometric and traffic variables, and discriminant analysis was applied to a detailed study of the geometry of sites that had either very high or very low accident rates.

CHARACTERIZATION OF ACCIDENTS

A data base of 3,557 sites from four states (Illinois, Florida, Ohio, and Texas) was used to perform the analyses. A series of constraints was applied to each state's geometric data base to create pure curve and tangent segments of 1 km in length with uniform geometry throughout. State accident records were used to produce a 3-yr history of accident experience at each site; the records included the following information: location, severity [fatal or injury versus property-damage-only (PDO) accidents], vehicle type, accident type, surface condition, light condition, and weather condition.

A total of 13,545 reported accidents occurred during the analysis period. A number of significant findings were derived from the characterization of the data.

Accident Types

The data in Table 1 give the proportion of accidents by number of vehicles involved and traffic volumes. Slightly more than half (54 percent) of the accidents on the selected analysis segments involved only one vehicle.

Accident Severity

A total of 5,390 accidents (41.5 percent) resulted in an injury or fatality. Single-vehicle run-off-the-road (ROR) accidents on curves were more likely to be severe when compared with multivehicle or other single-vehicle accidents. Regardless of roadway width or degree of curve, nearly half of all single-vehicle ROR accidents involved a personal injury or fatality. By contrast, 41 percent of multivehicle curve accidents and 29 percent of other single-vehicle accidents on curves were severe.

Surface Conditions

Approximately 27.5 percent of all accidents on curve segments occurred when the surface condition was reported as being wet or icy. It was rationalized from average climatology information that roadway pavements in the four states would be wet or icy approximately 10 to 12 percent of the time. Therefore, wet or icy surface conditions appear to almost triple the likelihood of an accident.

ANALYSIS OF ACCIDENTS

Initial analysis efforts focused on the incremental accident effects of five basic variables [average daily traffic (ADT), degree of curve, length of curve, roadway width, and shoulder width] by using the entire data base of 3,304 curve sites.

Analysis of covariance (AOCV) was used to study these incremental effects. This procedure provided a framework that considered both the direct effects of each variable and all of the potential interaction effects between variables. Preliminary analysis by using an AOCV framework, with accident rate as the dependent variable, indicated that all variables, except ADT, had a significant relation with accident rate. Subsequent analyses were conducted by using the following framework variables:

1. Covariates--degree of curve, length of curve (miles), road width (feet), and shoulder width (feet);
2. Factors--state: 1, 2, 3, 4; degree of curve: $\leq$ 1.999, 2.000 to 3.999, 4.000 to 6.999, $\geq$ 7.000; length of curve (miles): $\leq$ 0.1499, $\geq$ 0.1500; road width (feet): $\leq$ 21.999, $\geq$ 22.000; and shoulder width (feet): $\leq$ 5.999, $\geq$ 6.000; and
3. Dependent variables [accidents per million vehicle miles (MVM)]--total accident rate, single-vehicle accident rate, multivehicle accident rate, night accident rate, and fatal plus injury accident rate.

The results of the analysis that used the framework variables were as follows:

1. The multiple R^2 was about 0.19 (the AOCV framework explained 19 percent of the variance) for all matrices where the total accident rate was the dependent variable, and much lower for all other dependent variables;
2. State, degree of curve, and their two-way intersections with other variables accounted for most of the explained variance; and
3. The raw regression coefficients for each of

Table 1. Percentage of reported accidents by number of vehicles involved and volume class.

Traffic Volume Class (ADT)	Percentage of Reported Accidents		
	Single Vehicle		
	ROR	Other	Multivehicle
≤2,099	42.5	23.9	33.6
2,100-3,099	41.4	22.4	36.2
3,100-4,899	35.2	19.7	45.1
4,900-9,999	28.6	14.7	56.7
≥10,000	14.9	8.2	76.9
All volumes	35.1	19.1	45.8

Note: ADT = average daily traffic, and ROR = run-off-the-road accident.

Table 2. AOCV results.

Covariate	Practical Range of Covariate	Regression Coefficient	Difference in Accident Rate (accidents/ MVM)
Degree of curve	1° to 20°	0.056	1.12
Length of curve	0.05 to 0.40 mile	-0.141	0.05
Width of traveled way	18 to 24 ft	-0.023	0.14
Shoulder width	0 to 10 ft	-0.057	0.57

the covariates were as follows: degree of curve = 0.056; length of curve (miles) = -0.141; road width (feet) = -0.023; and shoulder width (feet) = -0.057.

The regression coefficients are the best overall estimates of the incremental effects of each covariate. They indicate logical relations, with the possible exception of length of curve. But in reality longer curves are usually associated with lower degrees of curve.

No logical trends could be derived from the individual regression of the cells in the AOCV matrix. The overall regression coefficients derived in the second step of the analysis are therefore the best-available predictors of the incremental accident effects of each covariate. It is informative to determine the predicted incremental differences over the practical range of each covariate, as given by the data in Table 2.

Degree of curve appears to have a sizable effect on accident rate over the practical range of usage. The effects of the other covariates, however, appear to be relatively small.

Analysis of High- and Low-Accident Sites

The limited success of the AOCV prompted a second approach: an analysis procedure that maximized the potential for discovering any geometric or accident relations. Two distinct populations were selected from the curve data base. The populations were defined as accident outliers; i.e., the sites were selected on the basis of either a very high accident rate or a very low rate. Differences in the geometric characteristics of these high- and low-accident populations were then investigated.

This approach ensured the discovery of any safety or geometry relations that may exist because the study sites were selected on the basis of dissimilarities in their accident experience rather than differences in geometric or other features that would only be hypothesized as being related to accidents.

The sites were partitioned into three ADT classes (1,400 to 2,099, 2,100 to 3,099, and 3,100 to 4,899) to control for any effects of traffic volume. Sites that had accident rates at least twice the mean rate for that state's ADT class were designated as high-accident sites. For all but the highest ADT class, low-accident sites experienced no accidents over a 3-yr period. A total of 330 sites that had extreme accident histories was thus selected.

Field studies were performed at all 330 sites to further define their geometric and environmental character. The following information was collected: degree of curve; road width (on tangent and in curve); shoulder width (on tangent and in curve); superelevation in curve; superelevation transition length; superelevation distribution; characteristic of horizontal alignment upstream from the curve; sight distance to the curve; relative hazard of roadside (slopes, objects, and so on); pavement condition; pavement skid resistance; signing; pavement markings; and presence of driveways, structures, minor roads, and so on.

The formal analysis of the high- and low-accident sites used a statistical technique known as discriminant analysis, which is used to statistically distinguish between two or more populations. Data that describe the characteristics on which the populations are expected to differ are collected and analyzed. In this case the two populations were the high- and low-accident sites. The characteristics (or discriminating variables) were the geometric and environmental variables studied in the field.

Discriminant analysis distinguishes between the populations being studied by forming a linear combination of the discriminating variables. The discriminant function is of the following form:

$$D = d_1Z_1 + d_2Z_2 + \dots + d_pZ_p$$

where D is the score on the discriminating function, d's are weighting coefficients, and Z's are the standardized values of the discriminating variables. The best derived discriminant function was

$$D = 0.071257(DC) + 2.9609(LC) + 0.10737(RR) = 0.035161(PR) - 0.14504(SW) - 1.54544 \quad (1)$$

where

D = discriminant function (nondimensional),
DC = degree of curve,
LC = length of curve (miles),
RR = roadside rating (a measure of roadside hazard),
PR = pavement rating (a measure of pavement skid resistance), and
SW = shoulder width (ft).

Because a higher discriminant score indicates a higher likelihood that a site is a high-accident location, the variables appear to contribute to the expected results.

The relative discriminating power of the variables in Equation 1 is as follows:

Variable	Relative Discriminating Power
RR	2.11
SW	1.39
LC	1.39
DC	1.14
PR	1.00

Equation 1 correctly classifies 75.9 percent of the high-accident sites, 60.2 percent of the low-accident sites, and 69.1 percent of all sites.

Interpretation of Results

The discriminant analysis procedure predicts or classifies a site as being a high- or low-accident site based on the actual distributions of D values for the two groups of sites in the data base. The analysis procedure decides which classification is appropriate by calculating probabilities that each D score belongs to the high or low distribution.

The value of the discriminant analysis results is primarily in the ability to predict high-accident locations. Because the D score distributions of the high- and low-accident sites overlap considerably, it is probably more efficient to concentrate on sites that have relatively high probabilities of being high-accident sites.

The procedure enables analysis of any probability criterion level. Figure 1 shows the relation between D and P(H); i.e., the probability that a site is a high-accident site. Selection of any P(H) criterion level can be translated into a minimum D score (based on Equation 1) for analysis purposes.

A P(H) criterion of 80 percent was chosen for further study. As shown in Figure 2, with this criterion it appears that almost all sites that have high roadside hazards would qualify as high-accident sites. Likewise, almost all sites that have low roadside hazards would not qualify. The results are more mixed with moderate roadside hazards. Generally, moderate roadside hazards must be combined with either very sharp curvature or a combination of two variables that are moderate or worse.

In summary, hazardous roadside design appears to

Figure 1. Relations between D and P(H).

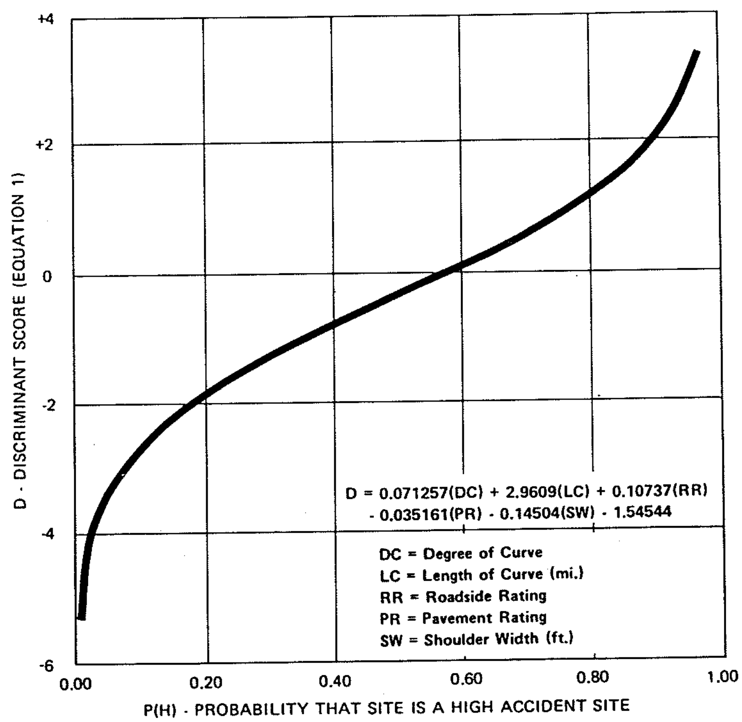

Figure 2. Probability that a highway curve site is a high-accident location.

Low Roadside Hazard (20)
High Pavement Skid Resistance (50)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Long (.30)	0	50	53	58	-	-
	4	37	39	45	-	-
	8	22	24	27	-	-
Moderate (.17)	0	42	45	50	-	-
	4	30	32	37	-	-
	8	18	20	23	-	-
Short (.05)	0	34	37	42	52	64
	4	23	25	30	38	52
	8	14	16	19	26	38

Moderate Roadside Hazard (35)
High Pavement Skid Resistance (50)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Long (.30)	0	86	87	88	-	-
	4	76	77	80	-	-
	8	66	67	70	-	-
Moderate (.17)	0	82	83	85	-	-
	4	69	71	74	-	-
	8	58	60	62	-	-
Short (.05)	0	74	76	78	85	91
	4	62	64	66	77	86
	8	50	52	54	65	79

High Roadside Hazard (50)
Moderate Pavement Skid Resistance (35)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Long (.30)	0	91	92	93	-	-
	4	85	89	90	-	-
	8	73	79	82	-	-
Moderate (.17)	0	87	89	90	-	-
	4	78	84	86	-	-
	8	66	72	75	-	-
Short (.05)	0	82	84	86	90	94
	4	71	76	79	84	89
	8	59	65	68	74	82

High Roadside Hazard (50)
Low Pavement Skid Resistance (20)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Long (.30)	0	97	97	98	-	-
	4	95	95	96	-	-
	8	92	92	93	-	-
Moderate (.17)	0	96	97	97	-	-
	4	94	95	95	-	-
	8	91	92	92	-	-
Short (.05)	0	96	97	97	98	99
	4	94	95	95	96	97
	8	91	92	92	93	94

Tabulated values are percent probabilities as given by Equation 1 and Figure 1.

be the largest contributor to high-accident experience at highway curves. Other less-prominent contributors are sharp curvature, narrow shoulders, low pavement skid resistance, and long curves.

APPLICATION OF RESULTS

For application at existing curves, Equation 1 indicates that improving roadside design, pavement skid resistance, and shoulder width may be valid countermeasures. The reduction of curvature may not be practical or productive because of high costs and the apparent trade-off between degree and length of curve for a given central angle. This study does not suggest that other design deficiencies, such as extremely unsatisfactory approach sight distances, extremely narrow lanes, extremely unsatisfactory transitions, and extreme shoulder slope breaks, might not be considered in an improvement program. Regardless, the discriminant analysis does provide guidance concerning the effects of roadsides, pavement surfaces, and shoulders.

Discussion

Thomas E. Mulinazzi*

Neuman, Glennon, and Saag have tackled a difficult problem in their paper. The TRB Committee on Operational Effects of Geometrics has been addressing this problem since they held their Workshop on Forgiving Roadways in July 1976.

It is not surprising that roadside hazards appear to be the largest contributor to high accident experience on highway curves. Based on the material presented in the paper, it is assumed that the analyses were based strictly on reported accidents. There is the belief that a majority of the ROR accidents go unreported, but those people who are unlucky enough to hit a roadside obstacle on leaving the roadway end up a statistic. On the other hand, this conclusion substantiates other research results that indicate that the removal of roadside obstacles on the outside of horizontal curves may be one of the most cost-effective safety projects that could be implemented.

Therefore, a more detailed description of how the roadside rating factor and the pavement rating factor were determined should have been included in this paper so that the research could be duplicated in the future.

In the multivariate analysis, the regression coefficients indicate that the accident rate increases as the degree of curvature increases, and decreases as the length of curve, road width, and shoulder width increase. The authors agreed that these appeared to be "logical relations, with the possible exception of length of curve." They go on to state "in reality, longer curves are usually associated with lower degrees of curves." They imply that lower degrees of curves are associated with safer roadways and, therefore, longer curves are safer. However, in the discriminate analysis it was determined that, by increasing the degrees of curve, lengths of curve, and roadside hazard ratings, the discriminate score increased, which meant that there was a higher likelihood of a high-accident location. The inclusion of length of curve as a variable to indicate a high-accident location appears to be contradictory to the rationale stated in the discussion of the multivariate analysis.

For future reference, a bibliography of research associated with this study would have been appreciated.

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Authors' Closure

We thank Mulinazzi for his comments on our paper. Further analysis of the discriminant analysis findings has verified that treatment of roadsides holds the greatest potential for cost-effective improvements to existing rural highways.

In responding to the comments and questions regarding the paper, it should be emphasized that the paper documents preliminary research results. The work was performed as part of a larger study of rural highway curve safety and operations. In attempting to complete the accident analysis and present it to TRB in an abridged form, certain points may not have been clear or complete. We appreciate the opportunity to clarify these points.

The study was indeed based on reported accidents. As in almost all studies of this nature, which rely on existing data bases, reported accidents are the safety-related variable of interest. What is important in trying to identify hazardous situations is that severe accidents (i.e., those that involve injuries and fatalities) be identified. For obvious reasons, such accidents tend to be reported.

In the interest of brevity, roadside and pavement rating factors were not covered in the paper. We agree that a description of these variables is required for duplication or separate analysis of the research.

ROADSIDE RATING FACTORS

Roadside rating factors were obtained from pictures and sketches of each of the sites observed in the field. The basis for the factors used is reported in NCHRP Report 148 (1). The model describes the likelihood of a severe accident as a function of roadside encroachment frequency, probability distributions for lateral displacement given a roadside encroachment, a measure of the hazard displacement given a roadside encroachment, and a measure of the hazard associated with the roadside. The roadside hazard is described by (a) a roadside slope break at a given distance, (b) a clear-zone width, (c) an obstacle coverage factor (i.e., a measure of the frequency of obstacles), and (d) severity indices for roadside slopes and obstacles.

The hazard ratings for various roadside configurations, assuming the average side slope break point is 10 ft from the edge of road, are given in Table 3.

PAVEMENT RATING FACTOR

Pavement rating factors were obtained from field crew observations of pavement surface roughness and depth of asperities. Pictures of the pavement were taken to verify the field crew's judgment. The rating scheme was developed to approximate the skid number at 60 mph.

LENGTH OF CURVE VERSUS ACCIDENT RATE

The relation between length of curve and accident

Table 3. Roadside hazard ratings.

Side Slope	Coverage Factor	Roadside Hazard Ratings by Lateral Clear Width (ft)						
		30	25	20	15	10	5	0
6:1 or flatter	90	24	28	32	34	42	46	47
	60	24	27	29	30	35	38	39
	40	24	27	27	27	32	34	34
	10	24	24	24	24	25	26	26
4:1	90	35	37	39	41	44	48	49
	60	35	36	38	39	40	43	44
	40	35	36	37	37	39	41	41
	10	35	35	35	35	36	37	37
3:1	90	41	42	42	43	44	48	49
	60	41	42	42	42	43	45	46
	40	41	42	42	41	41	44	45
	10	41	42	42	41	41	42	42
2:1 or steeper	90	53	53	53	53	45	49	50
	60	53	53	53	53	46	49	50
	40	53	53	53	53	48	50	50
	10	53	53	53	53	50	50	50

rate requires some clarification. The raw regression coefficients reported in the paper express the relation between the covariate and dependent variable for the entire data base. Because these coefficients do not reflect interactions between variables, they must be viewed with caution. In the case of curve length, the negative coefficient appears illogical unless one considers the strong correlation between curve length and degree of curve. In any event, as was discussed in the paper, the actual effect of this coefficient is negligible given the practical range of curve length.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Abridgment

Some Partial Consequences of Reduced Traffic Lane Widths on Urban Arterials

CLINTON L. HEIMBACH, PAUL D. CRIBBINS, AND MYUNG-SOON CHANG

When four-lane, urban, undivided arterials are newly built or reconstructed in place, traffic lane width becomes one of the major determinants of total right-of-way width. A particularly difficult problem arises when the urban corridor permits the construction of 10- or 11-ft-wide lanes, but not 12-ft-wide lanes. There are no guidelines for the roadway designer to indicate the trade-offs in traffic safety and operations as the width of a traffic lane is reduced below the 12-ft standard. The relationship of operating speed and accidents to a series of independent variables that characterize roadway design features and traffic volumes is investigated. It is concluded that both peak and off-peak operating speeds, as well as traffic accidents, are significantly related to traffic lane width on urban arterials. Specifically, operating speeds decrease and accidents increase as traffic lane width decreases. Operating speeds are also influenced by the posted speed limit, traffic volume, and total traffic lane width. Also, the number of accidents per year is related to the number of intersections per mile, the number of access trips to and from commercial driveways, average daily traffic, total traffic lane width, and changes in horizontal and vertical alignment. An example of the cost trade-offs between changes in traffic lane width and accidents and operating speeds is presented.

Traffic lane width is a key design element for the roadway; it influences right-of-way width, land costs, construction costs, levels of service, and traffic operational characteristics. In the past highway designers and traffic engineers have generally disregarded any lane width other than 12 ft, but because of current reductions in funding for improvements, there is increased interest in the feasibility and desirability of narrower traffic lanes.

The issue of lane width arises with restricted right-of-way width in urban corridors for new construction or reconstruction. It also arises with transportation system management (TSM) types of improvements in urban locations, where additional traffic lanes can be obtained by decreasing individual lane widths. Reducing lane widths, however, presents a dilemma. Although reduction in traffic lane width may result in reduced capital improvement cost, which may be the only way to accomplish the roadway improvement, such reduction does not meet most design standards and may result in permanent lowering of the roadway level of service.

It is clear that there are significant trade-offs involved when traffic lane widths are reduced to less than 12 ft. It is important that the decision maker know what these trade-offs are in order to evaluate the impact of a proposed reduction in lane width. Neither the technical research literature nor the design manuals provide guidelines to the decision maker for determining these trade-offs. This question of trade-offs is addressed in this paper. The scope of the investigation was limited to four-lane, undivided, urban arterial highways. Finally, it should be emphasized that no departure from current roadway standards for traffic lane widths is suggested in this paper.

LITERATURE REVIEW

The research literature on traffic lane width indicates that investigators have concentrated their efforts on rural, two-lane highways (1-5). When the effect of traffic lane width relative to highway speeds has been studied, the results have been inconsistent (6,7). When total traffic lane width was investigated relative to accident rates, the results were also inconsistent (1,8,9). Because traffic lane width should be investigated as a part of the total roadway system, it is appropriate to note the research results for certain other roadway design elements relative to measures of accident exposure. Investigators have reported that, as the number of urban intersections, access points, commercial

Table 1. Traffic lane width relative to pavement width for four-lane, undivided, urban arterials.

Total Pavement Width[a] (ft)	Center Traffic Lane Width (ft)	Curb Traffic Lane Width (ft)
40	9	11
44	10	12
44	11	11
46	11	12
48	11	13
50	12	13
52	11	15
52	12	14
54	13	14
56	14	14

[a]The pavement width is symmetrical with respect to the roadway centerline.

driveways, and commercial units increases, accidents also increase (8-10). A similar increase in accidents has been reported when the grade on vertical tangents increases (11,12).

In summary, it appears that neither the research literature nor the standard design manuals (13-15) offer the designer or decision maker any definitive guidelines on the consequences of reducing traffic lane widths to less than 12 ft.

METHODOLOGY

Study Observation Unit

The highway sections in urban locations that were studied were classified into the intersection proper, its approach, and the roadway between. The intersection approach was assumed to be 500 ft, subject to confirmation in the field. Because the research focused on the effect of traffic lane width, the impact of traffic signals on accidents and traffic flow had to be eliminated from the data collected in the field. Thus the study observation unit consisted of that portion of an urban, four-lane, undivided arterial between two approach legs of a signalized intersection, and excluded the intersection proper and the 500-ft-long approaches on both ends. All minor street intersections within the study observation unit were controlled with stop signs.

Data Base

Roadway traffic and accident data were collected for 108 four-lane, undivided, urban highway sections that had signalized intersections more than 2,000 ft apart in 8 urbanized areas in North Carolina. Site selection was limited to urban highways that had no access control, an asphalt surface, a curb and gutter, no parking on either side, a posted speed limit no more than 45 mph, and a minimum of 1 yr of accident experience without any change of roadway characteristics. Fifty-seven sites, which had a balance between total pavement width, traffic volume, and access density, were selected for investigation.

Roadway data included information on roadway section length, lane width, posted speed limit, horizontal and vertical alignment, intersections, private driveways, commercial driveways, and abutting land use. Traffic data included peak- and off-peak-hour traffic volumes and speeds, traffic composition, average daily traffic (ADT), and access trips to and from private driveways, commercial driveways, and intersecting streets. Accident data included a 100 percent sample of 1,936 accidents on all 57 sections over a 6-yr period.

Pavement and Traffic Lane Width Data

An analysis of the symmetrically balanced pavement width sections indicated the pattern of traffic lane widths given in Table 1. Because of symmetry with respect to traffic lane widths about the roadway centerline, the analysis used total pavement width to characterize the four individual lane widths, thereby reducing the number of variables.

Models Used in Analysis

Two general models were formulated for study: (a) measures of highway speed were related to traffic volume and roadway characteristics, and (b) measures of accident exposure were related to combinations of traffic volume and roadway characteristics. (Guidelines for model formulation were found in the work of other investigators reported in the reference list.) A total of 31 dependent variables and 36 independent variables were analyzed. Logarithmic and square root transforms of dependent accident variables were utilized by using stepwise multiple linear regression.

ANALYSIS

Off-Peak-Hour Traffic-Flow Model

For off-peak-hour conditions, the best traffic-flow model developed was one that related off-peak-hour speed (OPOS) on the urban arterials to posted speed (PS), off-peak-hour volume (OPHV), and total traffic lane width (LW), as follows:

$$\mathrm{OPOS} = 9.057 + (0.644)\mathrm{PS} - (0.003)\mathrm{OPHV} + (0.143)\mathrm{LW} \qquad (1)$$

The coefficient of multiple determination (R^2) is 57 percent. The regression coefficients for the independent variables are all significantly different from zero at the 10 percent level of significance.

Peak-Hour Traffic-Flow Model

For peak-hour conditions, the best traffic-flow model found was one that related peak-hour operating speed (POS) to posted speed (PS) and total traffic lane width (LW), as follows:

$$\mathrm{POS} = 6.315 + (0.533)\mathrm{PS} + (0.258)\mathrm{LW} \qquad (2)$$

The coefficient of multiple determination (R^2) is 53 percent. The regression coefficients for the independent variables are significantly different from zero at the 1 percent level.

Accident Model

The models recommended for predicting the total number of vehicle accidents on four-lane, undivided urban arterials are

$$\begin{aligned}\mathrm{RVAPY} = {} & 1.32 + (0.091)\mathrm{NNINT} + (0.045)\mathrm{ATCDW} + (0.077)\mathrm{ADT} \\ & + (0.070)\mathrm{HR} - (0.051)\mathrm{LW} + (0.081)\mathrm{VR}\end{aligned} \qquad (3)$$

and

$$\mathrm{VAPY} = -0.064 + 1.084(\mathrm{RVAPY})^2 \qquad (4)$$

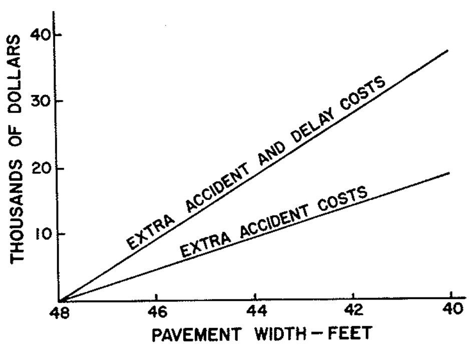
where

VAPY = total vehicle accidents per year,
RVAPY = square root transform of the total vehicle accidents per year,

Table 2. Illustrative problem.

Roadway Characteristics	Value
Highway section length (miles)	1.2
Pavement width (ft)	48
Individual traffic lane widths, curb to curb (ft)	13, 11, 11, 13
ADT (vehicles per day)	15,000
No. of minor street intersections	10
Posted speed limit (mph)	40
Peak hourly traffic volume (vehicles/hr)	1,350
Off-peak hourly traffic volume (vehicles/hr)	80
No. of access trips to and from commercial driveways per day	2,050
Cumulative sum of absolute value of changes in vertical elevation along arterial centerline (degrees)	100
Cost of opportunities foregone due to increases in vehicle travel time ($/hr)	5.00
Cost of equivalent property-damage-only accident ($)	760.00

Figure 1. Extra accident and delay costs for illustrative problem when total traffic lane width is reduced.



NNINT = number of side street intersections per mile of arterial roadway,
ATCDW = number of access trips to and from commercial driveways along the roadway section,
ADT = average daily traffic,
HR = square root of the cumulative sum of the absolute values of changes in horizontal direction (in degrees) along the arterial,
LW = total traffic lane width, and
VR = square root of the cumulative sum of the absolute values of changes in vertical elevation (in feet) along the arterial.

The coefficients for the independent variables in the regression model are significantly different from zero at the 10 percent level.

ILLUSTRATIVE APPLICATION OF RESEARCH RESULTS

The data assumed for a four-lane, undivided, urban arterial highway are given in Table 2. The research results are applied to these data in order to compute an estimate of the likely impact of reducing pavement and traffic lane widths from 48 to 40 ft. The arterial section is assumed to be 1.20 miles long between signalized intersections. Only the extra accident and delay costs are estimated in these calculations.

The numerical results are shown in Figure 1. The data in the figure indicate that the extra delay and accident costs associated with a reduction in pavement width from 48 to 40 ft will be approximately $37,229/mile/yr. If it is assumed that this annual cost prevails for 20 yr and that money is worth 8 percent, this annual cost is equivalent to a present worth of $365,300/mile at time year zero. The economic trade-off is obviously significant. For a complete analysis, estimates of the savings in construction and land costs as well as the extra costs due to accidents and time delays would have to be made.

CONCLUSIONS

The analysis and findings suggest that the traffic lane width on four-lane, undivided, urban arterials between signalized intersections (but not including the intersection proper or the approach leg) is significantly related to accidents and operating speeds.

ACKNOWLEDGMENT

We wish to acknowledge the financial and technical support for this project from the North Carolina Department of Transportation. Robert J. Hader, Department of Statistics, North Carolina State University, provided invaluable assistance on the experimental design and in reviewing the analysis and results.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Effectiveness of Clear Recovery Zones

JERRY L. GRAHAM AND DOUGLAS W. HARWOOD

Clear recovery zones outside the highway shoulder provide an opportunity for vehicles that leave the roadway to come to a safe stop or to return to the roadway. The results of a research effort to determine the effectiveness of clear recovery zones in reducing the number and severity of run-off-the-road (ROR) accidents and to provide an approach for the cost-effective application of clear recovery zones are presented. Actual accident data were obtained and analyzed to compare three different roadside design policies: the 6:1 clear zone policy, the 4:1 clear zone policy, and the nonclear zone policy. Three highway types were also considered: two-lane highways, four-lane freeways, and four-lane divided nonfreeways. Road sections that had these highway types and roadside design policies were identified in Illinois, Minnesota, and Missouri. Analysis of the accident data for the study sections revealed that roadside design policy had a statistically significant relation to single-vehicle ROR accident rate for all of the highway types studied. The differences in the accident rate between roadside design policies were quantified for each highway type. An analysis of the severity of single-vehicle ROR accidents revealed that the severity distribution did not vary between the roadside design policies on any of the three highway types studied. Four design examples were developed in order to illustrate the cost-effectiveness implications of the safety effectiveness measures developed in the study. The examples compared the accident reduction benefits and typical construction costs for improving highways from one roadside design policy to another.

A clear recovery zone is a relatively flat roadside area that is free of unprotected fixed objects and other nontraversable hazards; it is intended to provide an opportunity for vehicles that leave the roadway to come to a safe stop or to return to the roadway. Since the late 1960s a highway cross section with 6:1 embankment slopes within a 30-ft clear recovery zone has been generally adopted for new construction and major reconstruction projects. However, many existing highways have clear recovery zones with 4:1 (steeper) embankment slopes or do not have a clear recovery zone at all. It has been suggested in recent design guidelines that there is a need for clear recovery areas wider than 30 ft when embankment slopes steeper than 6:1 are used.

Some of the major findings of NCHRP Project 17-5, which was conducted to help highway agencies with limited funds develop a rational basis for making cost-effective applications of clear recovery zones, are presented. The full study has been published in NCHRP Report 247 (1).

OBJECTIVES AND SCOPE

The objectives of the study were to determine the safety effectiveness of clear recovery zones of differing slopes and widths in reducing the number and severity of run-off-the-road (ROR) accidents and to describe a framework based on clear zone effectiveness that could be used in design practice to assure the cost-effective application of clear recovery zones. The study was not intended to consider criteria for the installation of guardrail at specific sites or blanket fixed-object removal programs. Rather, the study was intended to evaluate the safety effectiveness of providing clear recovery zones by flattening slopes or removing or treating fixed objects.

The project scope included the consideration of several types of highways in rural areas, including two-lane highways, four-lane freeways, and four-lane divided nonfreeways. Specifically excluded from consideration were intersections, interchange ramps, low-volume highways [average daily traffic (ADT) less than 750 vehicles/day], and highways with urban development.

RESEARCH APPROACH

The general research approach for NCHRP Project 17-5 was to determine the effectiveness of clear recovery zones from actual accident data for existing highway sections by comparing the accident experience of highway sections constructed under different roadside design policies. Three distinct roadside design policies have been used by U.S. highway agencies; they are called 6:1 clear zone, 4:1 clear zone, and nonclear zone in this study. The policies vary in both the embankment slopes used and the presence of unprotected fixed objects outside of the highway shoulder.

The average safety effects of the three roadside design policies, as applied by highway agencies to the actual terrain in the field, can be determined if the highway sections constructed under different policies have no major differences other than roadside design or if such differences that do exist can be identified and accounted for. Thus the objective of the analysis was not to determine the safety effects of an individual geometric feature (for example, shoulder width), but to assure that an effect of shoulder width was not mistaken for an effect of roadside design policy.

The primary measures of effectiveness (or dependent variables) for the study were the single-vehicle ROR accident rate for all levels of accident severity and the fatal and injury single-vehicle ROR accident rate. These measures of effectiveness were restricted to ROR accidents because it would be questionable to presume a relation between roadside design policy and accidents where no vehicle left the roadway. The measure of effectiveness was restricted to single-vehicle ROR accidents; multiple-vehicle ROR accidents were excluded because single-vehicle ROR accidents are more frequent on rural highways than multiple-vehicle ROR accidents and because the severity of single-vehicle ROR accidents can be attributed in large degree to roadside design. Single-vehicle ROR accidents that involve both the outside (right side) and the median (left side) of the roadway on divided highways were considered.

The independent variables for the analysis, be-sides roadside design policy, were state, ADT, and shoulder width. The basic technique used for the statistical evaluation was analysis of covariance.

ROADSIDE DESIGN POLICIES

The key to understanding the safety effectiveness measures developed in this study is to understand the roadside design policies that were evaluated. The policies have been given the designations 6:1 clear zone, 4:1 clear zone, and nonclear zone; they are described briefly in this section. The policies vary in both the embankment slopes used and the presence of fixed objects outside the highway shoulder. Typical cross sections for the three roadside design policies are shown in Figure 1. The data in the figure indicate that the roadside slopes used for 6:1 and 4:1 design policies can, in some circumstances, be steeper than the nominal slope suggested by the name given to the policy. The roadside slopes used for nonclear zone highway sections are too highly variable to be illustrated by one typical cross section.

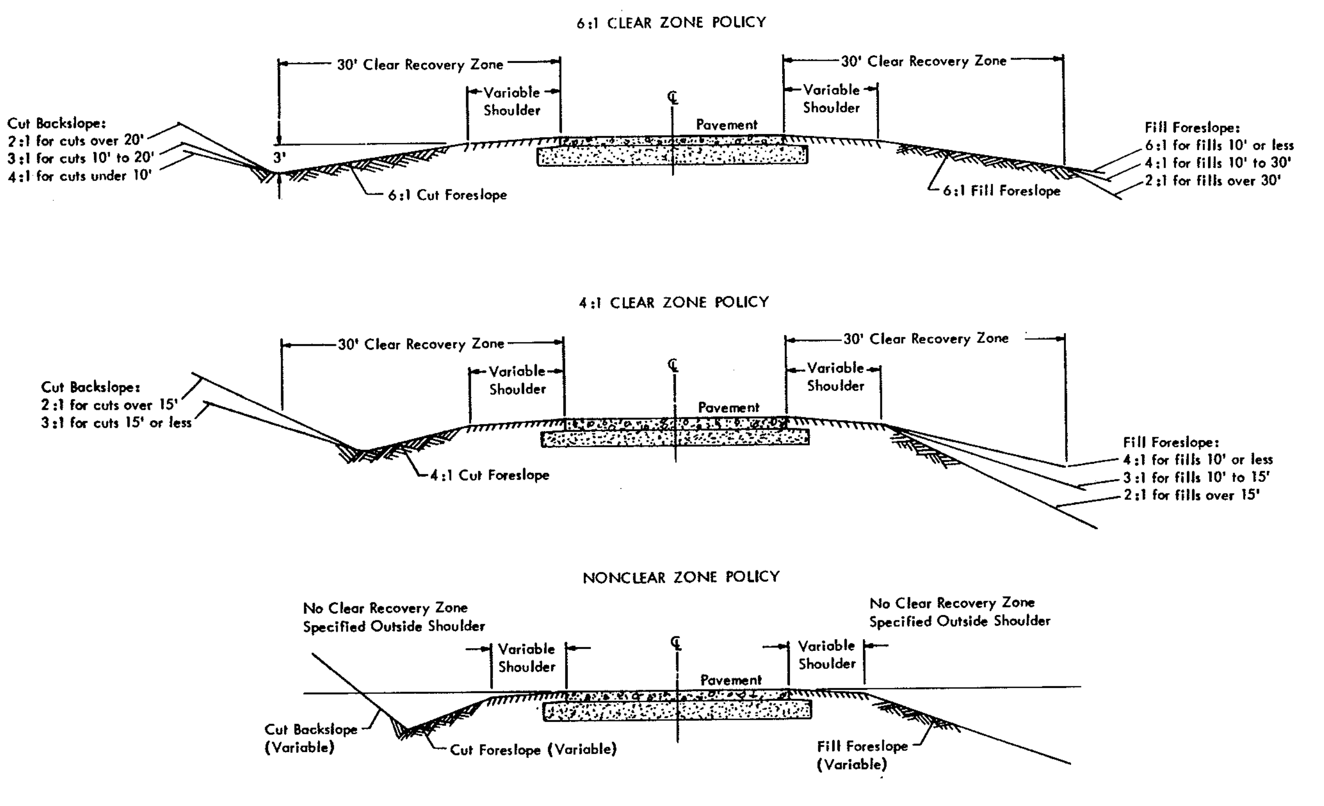
The 6:1 clear zone roadside design policy has been generally used in freeway construction and in some reconstruction projects on other types of highways since the late 1960s. Highways constructed under this policy generally have foreslopes of 6:1 or flatter within 30 ft of the traveled way. Embankment slopes of 4:1 or steeper are found on limited portions of these sections (up to 4 percent of the length on freeways and 12 percent of the length on two-lane highways in this study, for example).

The field survey indicated that the average embankment foreslope of 6:1 clear zone sections was 5.8:1 for two-lane highways, 6.9:1 for freeways, and 5.7:1 for four-lane divided nonfreeways. The slope on higher fill embankments often becomes 4:1 or steeper beyond 30 ft from the traveled way. These sections are generally, but not completely, clear of roadside fixed objects (other than those of breakaway design or protected by guardrail) within the 30-ft clear zone. The mean fixed-object coverage factor at 30 ft from the traveled way was 10 percent for 6:1 clear zone sections on two-lane highways, 7 percent on freeways, and 4 percent on four-lane divided nonfreeways. (The fixed-object coverage factor is a single measure that reflects the combined frequencies of both point and continuous objects on the roadside. The mean fixed-object coverage, which is expressed as a percentage, roughly corresponds to the probability of striking a fixed object given that a vehicle runs a specified distance off the roadway.) These mean fixed-object coverage factors represented the combined total for unprotected fixed objects, guardrail, and bridge rail.

The 4:1 clear zone roadside design policy was in general use by many highway agencies before the 6:1 clear zone policy was recommended by AASHTO, and it is still in use for some projects today. The majority of the length of these sections have foreslopes of 4:1 or flatter within 30 ft of the traveled way, but the 4:1 design policies often permitted 3:1 or 2:1 foreslopes on fills higher than 10 to 15 ft. The average embankment foreslope of 4:1 clear zone sections was 3.7:1 for two-lane highways, 4.3:1 for freeways, and 4.2:1 for four-lane divided nonfreeways.

Freeways constructed under the 4:1 clear zone policy are generally clear of unprotected fixed objects within 30 ft of the traveled way (in many cases this is the result of roadside improvement programs since the original freeway construction). On two-lane highways, there are substantially more unprotected fixed objects within 30 ft of the traveled way on 4:1 clear zone sections than on 6:1 clear zone sections. The mean fixed-object coverage

Figure 1. Typical cross sections for roadside design policies on two-lane highways.



factor at 30 ft from the traveled way for 4:1 clear zone sections was 17 percent on two-lane highways, 11 percent on freeways, and 23 percent on four-lane divided nonfreeways, including unprotected fixed objects, guardrail, and bridge rail.

It is apparent that neither the embankment slope nor the fixed-object clearance aspects of the 6:1 clear zone and 4:1 clear zone roadside design policies are as uniform as the names given them in this study might suggest.

The nonclear zone roadside design policy is dominated by sections with 3:1 and 2:1 embankment slopes (with some flatter slopes) and little or no control of unprotected fixed objects adjacent to the traveled way. The lack of control on fixed objects means that numerous trees, utility poles, and other objects are found within 30 ft of the traveled way on these sections.

Study sections with a nonclear zone roadside design policy were found on two-lane highways and four-lane divided nonfreeways, but not on freeways. The average embankment foreslope on nonclear zone sections was 3.1:1 for two-lane highways and 3.5:1 for four-lane, divided nonfreeways. The mean fixed-object coverage factor at 30 ft from the traveled way for nonclear zone sections, including unprotected fixed objects, guardrail, and bridge rail, was 24 percent for two-lane highways and 42 percent for four-lane divided nonfreeways.

PREVIOUS CLEAR ZONE EFFECTIVENESS STUDIES

Since the adoption of the 30-ft clear zone concept, several studies of the effectiveness of the updated clear zone criteria in reducing accidents have been conducted. The major emphasis of these studies was on the effect of embankment slope (4:1 versus 6:1) rather than on the width of the clear area free of fixed objects. These studies, performed by the Minnesota Department of Transportation (MnDOT), the Missouri Highway and Transportation Department, and the University of Illinois, are described and compared in this section.

MnDOT conducted a study of ROR accidents that occurred on rural, two-lane highways with 55-mph speed limits. (Note that these data are from a June 1980 unpublished report by MnDOT, "Comparison of Accident Rates Related to 4:1 and 6:1 Inslopes on 2-Lane Rural Trunk Highways.") The accident experience of roadways that had 6:1 foreslopes was compared with roadways that had 4:1 foreslopes. Both the 6:1 and 4:1 study sections had 30-ft clear zones and were chosen because they were comparable in lane width, shoulder width, and other geometric features. The study included 24 sections (215 miles of highway) that had 6:1 foreslopes and 23 sections (234 miles of highway) that had 4:1 foreslopes.

The Minnesota study indicated that the fatal, injury, property-damage-only (PDO), and total accident rates for the sections that had 4:1 foreslopes were all larger than the corresponding rates for the roadway sections that had 6:1 foreslopes. The total accident rates and the accident rates for corresponding severity levels were compared statistically between the 4:1 and 6:1 sections by using the t-test. (The Beherns-Fisher procedure was used because the variances of accident rate were unequal for the 4:1 and 6:1 sections.) The results indicated that the injury, fatal plus injury, and total accident rates were significantly different for the 4:1 and 6:1 sections at the 95 percent confidence level. There were no significant differences between the fatal or PDO accident rates for the two types of roadway sections.

Another study of roadside design policies was performed in Missouri in the mid-1970s. The results of this study are presented in an unpublished memorandum of the Missouri Highway and Transportation Department entitled, "Summary of Accident Experience on Sections of Road Constructed with 20-Foot 'Safety Zones'". This study compared the accident experience of roadway sections constructed with 20-ft safety zones and roadway sections constructed with similar design standards but without 20-ft safety zones. (The 20-ft safety zone is an obstacle-free area extending 20 ft beyond the shoulder of the road; therefore, if a road has a 10-ft shoulder, there would be a 30-ft clear recovery area from the edge of the traveled way.) The sections with safety zones generally had 6:1 embankment slopes, whereas the sections without safety zones had embankment slopes of 4:1 or steeper.

Comparisons between accident rates for roadways with and without safety zones were made for four highway types (Interstate, primary dual-lane, primary two-lane, and supplementary); four accident types (multiple-vehicle collisions, roadside obstacle collisions, overturning accidents, and total accidents); and three severity levels (fatal, injury, and PDO). The authors of the Missouri study did not find a statistically significant difference in the overall accident rate or severity between sections with and without safety zones for any of the highway types considered.

A 1974 University of Illinois study evaluated various improvements in roadside safety design policies that had been implemented by the Illinois Department of Transportation during the preceding years. (Note that the data for the Illinois study are from V.E. Dotson's 1974 unpublished Master of Science thesis, "An Evaluation of the Thirty-Foot Clear Zone," prepared for the University of Illinois, Urbana.) Study sections were selected on four Interstate routes in one Illinois highway district. The study sections included 211.19 miles of rural freeway. Of this mileage, 90.57 miles were constructed to the latest safety standards (i.e., 6:1 embankment slopes and a 30-ft clear recovery zone), whereas 89.15 miles were constructed with 4:1 embankment slopes and had not been upgraded since the adoption of the safety standards.

A comparison of accident experience between sections that had 6:1 and 4:1 embankment slopes was conducted by using the data for three of the four Interstate routes. A two-way analysis of variance that used slope and route factors indicated that neither factor was statistically significant. A similar analysis for multiple-vehicle accident rate revealed the same result.

The results of the three studies that compared 6:1 and 4:1 slopes are summarized in Table 1. The studies used varying statistical techniques. The Missouri study covered all types of highways, and no significant difference in accident rates was revealed for any type of highway. The University of Illinois study did not reveal a significant difference in accident rate for freeways. The Minnesota study revealed a statistically significant difference between 4:1 and 6:1 slopes on two-lane highways.

Despite such varying conclusions, it is significant to note that the relative differences in the single-vehicle accident rate are consistent between the three studies. In both the Minnesota and Missouri studies the roadside accident rates for two-lane highways are about 60 to 70 percent higher for sections with 4:1 slopes than for sections with 6:1 slopes. In both the University of Illinois and Missouri studies the roadside accident rates for freeways are about 25 to 30 percent higher for sections with 4:1 slopes than for sections with 6:1 slopes. This consistency of findings suggests that, with a larger data base and more sophisticated anal-

Table 1. Summary of studies that compared 6:1 and 4:1 embankment slopes.

Study	Highway Type	Accident Measured Used	Accident Rate (accidents per million vehicle miles) 6:1	4:1	Statistical Test Used	Conclusions
Minnesota[a]	Two lane	ROR accident rate	0.154	0.266	t-test[b]	Both total and fatal and injury accident rate were significantly lower for 6:1 sections
Missouri[c]	Primary two lane	Total accident rate	0.948	0.959	Chi-square	No statistically significant change in overall accident rate or severity; collisions with roadside obstacles decreased, but collisions between vehicles increased
		Multiple-vehicle accident rate	0.700	0.547		
		Single-vehicle accident rate	0.248	0.412		
	Freeway	Total accident rate	0.379	0.430		
		Multiple-vehicle accident rate	0.174	0.163		
		Single-vehicle accident rate	0.205	0.163		
University of Illinois[d]	Freeway	Single-vehicle ROR accident rate	0.333	0.421	Analysis of variance	No statistically significant change in single-vehicle ROR or multiple-accident rate

[a]Data from June 1980 unpublished report ("Comparison of Accident Rates Related to 4:1 and 6:1 Inslopes on 2-Lane Rural Trunk Highways") from MnDOT.
[b]This t-test used the Beherns-Fisher procedure for unequal variances.
[c]Data from unpublished memorandum ("Summary of Accident Experience on Sections of Road Constructed with 20-Foot 'Safety Zones'") from the Missouri State Highway Commission.
[d]Data from V.E. Dotsun's 1974 unpublished Master of Science thesis ("An Evaluation of the Thirty-Foot Clear Zone") prepared for the University of Illinois, Urbana.

ysis techniques, the observed effects might be statistically significant.

DEVELOPMENT OF PROJECT DATA BASE

The project data base includes study sections for three highway types (two-lane highways, four-lane freeways, and four-lane divided nonfreeways) and three roadside design policies (6:1 clear zone, 4:1 clear zone, and nonclear zone). Originally, it was planned to include only two-lane highways and freeways in the study, but data on four-lane divided nonfreeways (i.e., four-lane divided highway without full control of access) were readily available in Missouri and, therefore, these data were also used. Excluded from the study were highways with urban development and highways with ADT volumes less than 750 vehicles/day. All of the sections had 55-mph speed limits and shoulders at least 4 ft wide. Lane widths were either 11 or 12 ft.

A data base that contained the accident history, traffic volumes, and geometrics of these highway sections was assembled. The basic data-collection approach pursued was to make maximum use of the data already available in Illinois, Minnesota, and Missouri, while obtaining more recent accident data and expanding the number of highway sections studied, the range of highway types included, and the length of the study period. The data obtained for highway sections in Missouri included many of the same highway sections from the previous Missouri study, but also incorporated nonclear zone sections. The data base in Illinois was expanded to include highway sections from throughout the state rather than from a single district. In Minnesota the highway sections from the state study of 6:1 and 4:1 sections on two-lane highways were supplemented with two-lane nonclear zone sections and sections on freeways.

The accident data obtained for the sections in Illinois and Missouri covered the 5-yr period from 1975 through 1979, inclusive. The accident data in Minnesota included the 4-yr period from 1977 through 1980. The project analyses used each year of data from each study section as a separate observation. If any study section was either not open to traffic during a given calendar year or was undergoing major construction activity, that year was excluded from the study.

The entire data base contained 836 study sections that covered 4,601 miles of highway with a total exposure of more than 41 billion vehicle miles of travel during the study period. These highway sections experienced 11,649 single-vehicle ROR accidents during a study period that averaged about 4.4 yr in duration. Detailed breakdowns of the mileage and accident experience for the study sections can be found in the project report (1).

ANALYSIS OF ACCIDENT DATA

The three objectives of the statistical analysis of accident data conducted in this study were to

1. Determine whether roadside design policy has a statistically significant effect on traffic accident experience,
2. Determine the magnitude of the effect of roadside design policy on accident experience when the effect is statistically significant, and
3. Express the roadside design policy effect in a form most useful for application by highway agencies in design decisions.

Analysis Approach

The statistical analysis approach was developed in a series of three steps: (a) select measure(s) of effectiveness (dependent variables) for the analysis; (b) select independent variables for the analysis; and (c) select a statistical technique to determine whether the effect of roadside design policy is statistically significant.

Measures of Effectiveness

The primary measure of effectiveness (or dependent variable) selected to evaluate roadside design policies is the single-vehicle ROR accident rate. The single-vehicle ROR accident rate for a 1-yr period, expressed as accidents per million vehicle-miles, was defined in the conventional manner:

$$AR = [(N)(10^6)]/[(ADT)(D)(L)] \quad (1)$$

where

AR = single-vehicle ROR accident rate (accidents per million vehicle-miles);
N = number of single-vehicle ROR accidents;
ADT = average daily traffic volume (vehicles/day);
D = duration of study period (days) [in this study, 365 days or 1 yr]; and
L = length of study section (miles).

The study also included an evaluation of accident severity measures, including both the accident severity distribution (proportion of fatal, injury, and PDO accidents) and the single-vehicle ROR accident rate for fatal and injury accidents only.

Independent Variables

The independent variables for the accident analysis are those variables that may have an influence on the measure of effectiveness (or dependent variable)--single-vehicle ROR accident rate. In accordance with the analysis objectives, the primary independent variable for the analysis is roadside design policy. Three different levels of roadside design policy were evaluated, and any differences in accident experience between the policies were identified.

Several other independent variables were considered because they may also influence the single-vehicle ROR accident rate; these included state, ADT, volume, and shoulder width.

State was considered an independent variable because statistically significant state-to-state differences in single-vehicle ROR accident rate were found for four of the five combinations of highway type and roadside design policy tested (all except 4:1 clear zone sections for two-lane highways). It might be tempting to explain these differences by variations in the reliability of the accident reporting systems of the three states. However, three of the four significant differences persisted even when the analysis was restricted to fatal and injury accidents only. This analysis indicated that (a) although the observed state-to-state differences could be partly due to unreliable accident reporting, they also represent, in part, true differences in accident experience between the states; and (b) the influence of these state-to-state differences must be accounted for or corrected before assessing the statistical significance of the effect of roadside design policy.

Even within one highway type the highway sections for different roadside design policies also differ in ADT. For example, the average ADT for two-lane highways is 2,031 for 6:1 clear zone sections, 2,778 for 4:1 clear zone sections, and 2,745 for nonclear zone sections. Because the ROR accident rate is known to vary with ADT in some situations, it is important that this effect be accounted for before assessing the effect of roadside design policy.

One concern raised during the study was the influence of roadway geometrics on accident rate. For example, some of the observed differences between 6:1 clear zone sections and sections with other roadside design policies could be due to improved roadway geometrics on the 6:1 clear zone sections that make it less likely for vehicles to run off the road. A field survey of the study sections examined the differences between roadside design policies in roadway geometrics, including lane width, shoulder width, median width, and proportion of tangents and horizontal curves. Of these variables, only shoulder width differed significantly between the roadside design policies. Therefore, shoulder width was used as an independent variable whose effect was accounted for before assessing the effect of roadside design policy.

Statistical Analysis Approach

The simplest and most direct method of evaluating the statistical significance of a factor that has more than two levels, such as roadside design policy, is the one-way analysis of variance. If the factor is statistically significant, Duncan's multiple range test can then be used to determine which differences between the individual roadside design policies are statistically significant.

Although this approach was used in the initial analyses, it did not respond to one important goal of the analysis--to assess the significance of roadside design policy only after accounting for the effects of other independent variables such as state, ADT, and shoulder width. Another analysis approach is used to accomplish this goal--analysis of covariance--which is a statistical technique used to assess the effects of both independent variables with several discrete levels (known as factors) and independent variables with values on a continuous scale (known as covariates). The independent variables of roadside design policy and state are discrete variables most naturally treated as factors (in this case with three levels each), and ADT and shoulder width are continuous variables most naturally treated as covariates. Two dependent variables were used in separate analyses of covariance: single-vehicle ROR accident rate and fatal and injury single-vehicle ROR accident rate.

The specific form of analysis of covariance used was a hierarchical analysis of covariance, in which the effects of the independent variables are accounted for in sequence so that a factor or covariate is statistically significant only if it explains a significant portion of the variance remaining after the variables considered previously have been accounted for. The relation between hierarchical analysis of covariance and the classic approach to analysis of covariance is analogous to the relation between multiple regression and stepwise regression. The independent variables were considered in a fixed order in the analysis (state, ADT, shoulder width, roadside design policy) so that the effect of the first three variables would be considered before the effect of roadside design policy. This approach prevents an effect of state, ADT, or shoulder width from being mistaken for an effect of roadside design policy.

In an analysis of variance or covariance that has a balanced design, the best measure of effectiveness for each roadside design policy is the average (or arithmetic mean) accident rate for that policy. The experimental designs used in this study were not balanced because the sample sizes in the cells defined by the experimental factors (state and roadside design policies) were not equal and the covariates (ADT and shoulder width) did not have the same mean in every cell. The best measure of effectiveness for each roadside design policy in such an unbalanced design is the least square mean for that policy. The least square mean compensates for the differences between the cells in sample sizes and covariate means. The least square mean is, in effect, the mean accident rate that would result if every cell had the same sample size and the same mean for each covariate. The differences in accident rate between roadside design policies can be represented by the differences in the least square means.

Comparisons of the distribution of the accident severity were made by using the Kolmogorov-Smirnov test (a nonparametric test for distribution shifts) and the Z-test for difference of proportions (based on the standard normal approximation to the binomial distribution) (2).

Analysis Results

The results of the accident data analysis are presented in the following sections. First, the results of several analyses of variance and covariance are presented in order to demonstrate that roadside design policy has a statistically significant effect on accident rate. Second, the mean and adjusted (least square mean) accident rates are presented in order to indicate the magnitude of the differences in accident rate between policies. Third, the effect of roadside design policy on accident severity is discussed. Finally, the relation of ADT on accident rate in the study data is illustrated.

Results of Analysis of Covariance

The results of a series of analyses of variance and covariance that were performed on the ROR accident rate and the fatal and injury ROR accident rate are presented in this section. These analyses were performed by computer by using the general linear model procedure of the statistical analysis system (SAS) (3). All conclusions presented here regarding statistical significance are at the 95 percent confidence level unless otherwise stated.

The data in Table 2 give three analyses of covariance of the single-vehicle ROR accident rate where the effects of roadside design policy and the other independent variables were considered. These analyses indicate that the effect of roadside design policy on single-vehicle ROR accident rates is statistically significant, even after consideration of the effects of state, ADT, and shoulder width. All four independent variables in the analysis of covariance for two-lane highways were statistically significant. The shoulder width covariate was eliminated from the analysis of covariance for freeways because it was not statistically significant. The state factor was not included in the analysis of four-lane divided nonfreeways because all of these data were from one state; the ADT and shoulder width covariates were eliminated because they were not statistically significant. Thus only the factor of roadside design policy remains in the analysis of covariance for four-lane divided nonfreeways. Both two-lane and freeway analyses contain a term for the interaction between state and roadside design policy; the state-policy interaction was statistically significant for two-lane highways, but it was not statistically significant for freeways.

This same analysis was repeated by using the ROR accident rate for fatal and injury accidents as the dependent variable. The fatal and injury accident rate would normally be expected to be more reliable than the total accident rate because of the exclusion of PDO accidents, which are subject to variations in reporting levels. The data in Table 3 for fatal and injury single-vehicle ROR accident rates are entirely analogous to the data in Table 2 for total single-vehicle ROR accident rates. As in the earlier analysis of covariance, the effect of roadside design policy remains statistically significant after consideration of the variables state, ADT, and shoulder width. With only two exceptions, the same independent variables and interactions that were statistically significant for the total ROR accident rate are also statistically significant for the fatal and injury ROR accident rate. One exception is that the ADT factor was omitted from the analysis of freeways in Table 3 because its effect on fatal and injury accident rate was not statistically significant. The other exception is the state-policy interaction for two-lane highways, which also was not statistically significant.

The state-policy interaction in Tables 2 and 3 indicates whether the effect of roadside design policy on accident rate varies from state to state. If the interaction effect is significant, it implies that the accident rate for a given roadside design policy depends on the state. If the interaction effect is not significant, the roadside design poli-

Table 2. Analysis of covariance of roadside design policy and other variables for single-vehicle ROR accident rates.

Source of Variation	Sum of Squares	Degrees of Freedom	Mean Square	F	Significance (at 95 percent confidence level)
Two-lane highways (n = 1,958)					
State[a]	32.603	2	16.302	36.59	SIG
ADT[b]	2.865	1	2.865	6.43	SIG
Outside shoulder width[b]	18.426	1	18.426	41.36	SIG
Policy[a]	18.150	2	9.075	20.37	SIG
State-policy[c]	9.557	4	2.389	5.36	SIG
Explained	81.601	10	8.160	18.31	SIG[d]
Error	867.470	1,947	0.446		
Total	949.071	1,957			
Four-lane freeways (n = 1,045)					
State[a]	3.250	2	1.625	42.40	SIG
ATD[b]	0.380	1	0.380	9.90	SIG
Policy[a]	2.434	1	2.434	63.50	SIG
State-policy[c]	0.032	2	0.016	0.42	NS
Explained	6.097	6	1.016	26.51	SIG[e]
Error	39.789	1,038	0.038		
Total	45.886	1,044			
Four-lane divided nonfreeways (n = 580)					
Policy[a]	10.488	2	5.244	46.08	SIG
Explained	10.488	2	5.244	46.08	SIG[b]
Error	65.665	577	0.114		
Total	76.153	579			

Note: SIG = statistically significant and NS = not statistically significant.
[a]Factor. [b]Covariate. [c]Interaction. [d]$R^2 = 0.086$. [e]$R^2 = 0.133$. [f]$R^2 = 0.138$.

Table 3. Analysis of covariance of roadside design policy and other variables for fatal and injury single-vehicle ROR accident rate.

Source of Variation	Sum of Squares	Degrees of Freedom	Mean Square	F	Significance (at 95 percent confidence level)
Two-lane highways (n = 1,958)					
State[a]	3.815	2	1.908	8.11	SIG
ADT[b]	1.442	1	1.442	6.13	SIG
Outside shoulder width[b]	3.175	1	3.175	13.50	SIG
Policy[a]	5.475	2	2.738	11.64	SIG
State-policy[c]	2.212	4	0.553	2.35	NS[d]
Explained	16.119	10	1.612	6.85	SIG[e]
Error	457.979	1,947	0.235		
Total	474.098	1,957			
Four-lane freeways (n = 1,045)					
State[a]	0.604	2	0.302	25.98	SIG
Policy[a]	0.261	1	0.261	22.41	SIG
State-policy[a]	0.026	2	0.013	1.13	NS[f]
Explained	0.891	5	0.178	15.33	SIG
Error	12.076	1,039	0.012		
Total	12.967	1,044			
Four-lane divided non-freeway (n = 580)					
Policy[a]	2.868	2	1.434	44.95	SIG
Explained	2.868	2	1.434	44.95	SIG[g]
Error	18.404	577	0.032		
Total	21.272	579			

Note: SIG = statistically significant and NS = not statistically significant.
[a]Factor.
[b]Covariate.
[c]Interaction.
[d]Statistically significant at 90 percent confidence level.
[e]R^2 = 0.034.
[f]R^2 = 0.067.
[g]R^2 = 0.134.

cies can be assumed to have the same accident rate in every state. The analysis results indicate that the state-policy interaction is not statistically significant for freeways.

The situation for two-lane highways is more complicated. The interaction effect is statistically significant for the total ROR accident rate but is (barely) not statistically significant for the fatal and injury accident rate. Because the analysis of fatal and injury accident rates is assumed to be more reliable (and because of reasons of simplicity), the state-policy interaction for two-lane highways has been treated as being not statistically significant and it is assumed that the mean ROR accident rate for each highway type and roadside design policy is representative of all three states. Nevertheless, the use of a separate estimate in each state for the mean accident rates of each roadside design policy on two-lane highways could be justified. The magnitudes of the mean accident rates, both state by state and combined, are discussed in the next section.

The conclusion drawn from the analyses of covariance is that roadside design policy has a statistically significant effect both on the total ROR accident rate and on the fatal and injury ROR accident rate. Statistical significance implies that the effect of roadside design policy on accident rate is large enough that it is unlikely to have occurred because of random variation alone. Nevertheless, statistical significance does not necessarily imply that the effect of roadside design policy on accident rate is large enough to be significant in a practical sense. Practical conclusions must be based on the magnitude of the observed differences in accident rate between roadside design policies and on the cost-effectiveness implications of those differences. The statistical analysis can only provide confidence that the observed differences, however large or small, are real.

Mean and Adjusted Mean Accident Rates

In this section the mean and adjusted (or least square mean) accident rates for the three highway types and the three roadside design policies are compared. The most elementary measure of effectiveness for roadside design policy is a simple comparison of average or arithmetic mean accident rates. For example, on freeways the mean ROR accident rate is 0.235 accident/million vehicle-miles for 6:1 clear zone sections and 0.329 accident/million vehicle-miles for 4:1 clear zone sections. The difference in these mean rates--0.094 accident/million vehicle-miles--is a measure of effectiveness for improving a 4:1 clear zone design to a 6:1 clear zone design. After adjustment for the effects of state and ADT on the accident rate, the least square mean accident rates are 0.182 accident/million vehicle-miles for 6:1 clear zone sections and 0.289 accident/million vehicle-miles for 4:1 clear zone sections. The corresponding difference in mean accident rates is 0.107 accident/million vehicle-miles. In most cases the difference between roadside design policies in the least square mean accident rate was slightly larger than the difference in the arithmetic mean accident rate, although the increase was never large.

The data in Table 4 summarize the arithmetic and least square mean accident rates for each highway type and roadside design policy for the individual and combined states. The arithmetic means in Table 4 are the averages of all of the available data without regard to state, ADT, or shoulder width. The least square means are preferable to the arithmetic means as a measure of effectiveness, and the least square means for the combined states are the single best measures of effectiveness for roadside design policy. Statistical comparisons performed by using the least square means procedure of the SAS computer package confirm that, for each highway

Table 4. Comparison of accident rates between roadside design policies.

Highway Type	Data Set	Statistic	Accident Rates by Roadside Design Policy[a]: 6:1 Clear Zone	4:1 Clear Zone	Nonclear Zone	Differences in Accident Rates Between Roadside Design Policies[a]: 6:1 versus 4:1	4:1 versus Nonclear Zone
Two lane	Illinois	Arithmetic mean	0.385	0.543	1.180	0.158	0.637
	Minnesota	Arithmetic mean	0.141	0.237	0.471	0.096	0.234
	Missouri	Arithmetic mean	0.238	0.389	0.505	0.151	0.116
	All states	Arithmetic mean	0.243	0.379	0.639	0.136	0.260
	Illinois	Least square mean	0.415	0.580	1.189	0.165	0.609
	Minnesota	Least square mean	0.123	0.234	0.325	0.111	0.091
	Missouri	Least square mean	0.223	0.393	0.526	0.170	0.133
	All states	Least square mean	0.254	0.403	0.680	0.149	0.277
Freeway	Illinois	Arithmetic mean	0.270	0.377	–	0.107	–
	Minnesota	Arithmetic mean	0.133	0.225	–	0.092	–
	Missouri	Arithmetic mean	0.136	0.267	–	0.131	–
	All states	Arithmetic mean	0.235	0.329	–	0.094	–
	Illinois	Least square mean	0.272	0.375	–	0.103	–
	Minnesota	Least square mean	0.135	0.224	–	0.089	–
	Missouri	Least square mean	0.138	0.267	–	0.129	–
	All states	Least square mean	0.182	0.289	–	0.107	–
Four-lane divided nonfreeway	Missouri	Arithmetic mean	0.155	0.319	0.607	0.164	0.288
	Missouri	Least square mean	0.155	0.319	0.607	0.164	0.288

[a]Accident rates are measured for single-vehicle ROR accidents per million vehicle miles.

Table 5. Adjusted mean accident rates by highway type and roadside design policy.

Highway Type	Accident Rate by Roadside Design Policy[a]: 6:1 Clear Zone	4:1 Clear Zone	Nonclear Zone	Differences in Accident Rates Between Roadside Design Policies[a]: 6:1 versus 4:1 Δ	Significance[b]	4:1 versus Nonclear Zone Δ	Significance[b]
ROR accidents							
Two lane	0.254	0.403	0.680	0.149	SIG	0.277	SIG
Freeway	0.182	0.289	–	0.107	SIG	–	–
Four-lane divided nonfreeway	0.155	0.319	0.607	0.164	SIG	0.288	SIG
Fatal and injury ROR accidents							
Two lane	0.098	0.183	0.320	0.085	SIG	0.137	SIG
Freeway	0.068	0.100	–	0.032	SIG	–	–
Four-lane divided nonfreeway	0.057	0.129	0.298	0.072	SIG	0.169	SIG

Note: SIG = statistically significant.

[a]Accident rates are measured in accidents per million vehicle miles.

[b]Statistically significant at the 95 percent confidence level.

type, the least square mean accident rates for all of the roadside design policies are significantly different. Results analogous to those given in Table 4 were also obtained for the fatal and injury ROR accident rate.

Also given in Table 4 are the measures of effectiveness obtained from the data for the individual states. The state data indicate that, although the magnitudes of the accident rates themselves vary markedly from state to state, the differences in mean accident rate between roadside design policies are consistent from state to state, with only one exception. The one exception is the comparison between 4:1 clear zone and nonclear zone sections for two-lane highways, which is larger in Illinois than in the other two states.

The key measures of effectiveness from the accident analysis are summarized in Table 5. The data in this table give the least square means for the individual highways types and roadside design policies and for the differences between policies. The data in Table 5 indicate that the differences in accident rate between the roadside design policies are statistically significant and that the roadside design policies vary markedly in accident rate when compared. For example, the fatal and injury accident rate for a 6:1 clear zone section on a two-lane highway is about half the rate for a 4:1 clear zone section, which is in turn about half the rate for a nonclear zone section. Nevertheless, the differences in accident rate between roadside design policies are small in absolute magnitude. For example, the largest difference between roadside design policies as given in Table 5 is 0.288 accident/million vehicle-miles for four-lane divided nonfreeways, which corresponds to 0.46 accident/mile/yr.

Tests for Effects of Accident Severity

The analysis described in the preceding section established that the ROR accident rate decreases as the roadside design policy improves. Further statistical tests were conducted to determine whether the roadside design policies also differ in the severity distribution for reported accidents. Accident severity is generally classified in three categories: fatal, injury, and PDO accidents. If there is a shift in the distribution between these levels of accident severity from one roadside design policy to another, this shift should be considered in any cost-effectiveness analysis of design policies.

The severity distribution for ROR accidents are given in Table 6 by highway type and roadside design

Table 6. Accident severity distribution for single-vehicle ROR accidents.

		Accidents							
		Fatal		Injury		PDO		Total	
Highway Type	Roadside Design Policy	No.	Percent	No.	Percent	No.	Percent	No.	Percent
Two lane	6:1 clear zone	12	2.5	201	41.1	276	56.4	489	100.0
	4:1 clear zone	48	3.0	720	44.5	850	52.5	1,618	100.0
	Nonclear zone	22	1.4	684	43.0	886	55.6	1,592	100.0
Four-lane freeway	6:1 clear zone	32	1.6	705	35.7	1,237	62.7	1,974	100.0
	4:1 clear zone	71	1.5	1,638	34.8	2,999	63.7	4,708	100.0
Four-lane divided nonfreeway	6:1 clear zone	4	1.5	109	41.5	150	57.0	263	100.0
	4:1 clear zone	15	1.8	348	42.9	448	55.3	811	100.0
	Nonclear zone	2	1.0	95	49.0	97	50.0	194	100.0

Table 7. Statistical tests of distributions in accident severity for ROR accidents.

Highway Type	Road Design Policy Comparison	3 Levels[a] (fatal versus injury versus PDO)	2 Levels[b] (fatal versus injury)	2 Levels[b] (fatal and injury versus PDO)
Two lane	6:1 clear zone versus 4:1 clear zone	NS	NS	NS
	4:1 clear zone versus nonclear zone	NS	SIG[c]	NS
	6:1 clear zone versus nonclear zone	NS	NS	NS
Four-lane freeway	6:1 clear zone versus 4:1 clear zone	NS	NS	NS
Four-lane divided nonfreeway	6:1 clear zone versus 4:1 clear zone	NS	NS	NS
	4:1 clear zone versus nonclear zone	NS	NS	NS
	6:1 clear zone versus nonclear zone	NS	NS	NS

Note: NS = not statistically significant and SIG = statistically significant.

[a]Used Kolmogorov-Smirnov test for distribution shift.
[b]Used Z-test for difference of proportions.
[c]Statistically significant at 95 percent confidence level.

policy. The table entries represent the combined data for Illinois, Minnesota, and Missouri.

The differences in the distribution in accident severity between roadside design policies, as indicated by the data in Table 6, are small, and there is no consistent pattern of lower accident severity on improved roadside design policies. Statistical tests were conducted to compare the distributions in accident severity for pairs of roadside design policies within individual highway types. Three different forms of the distribution in accident severity were tested: one form that used three severity levels (fatal versus injury versus PDO), and two other forms that used two levels (fatal versus injury; and fatal and injury versus PDO). The three-level comparisons were performed by using the Kolmogorov-Smirnov test for distribution shifts (2). The two-level comparisons were performed by using the Z-test for differences in proportions (4).

The results of the statistical tests involving the distribution in accident severity are given in Table 7. Of the 21 tests performed, only one was statistically significant. Furthermore, the comparison that was statistically significant was in the opposite sense to that expected; the proportion of fatal and injury accidents involving fatalities was larger for 4:1 clear zone sections than for nonclear zone sections on two-lane highways. This result to the contrary nonwithstanding, it was concluded that there is no difference between roadside design policies in the severity distribution of reported accidents.

None of these analyses reveals any consistent trend toward improvements in roadside design policy decreasing the fatal and injury accident rate to a greater extent than PDO accidents. The reader should not misinterpret this finding to mean that improvements in roadside design policy do not decrease the frequency of severe accidents; the finding means only that such improvements are equally effective in reducing both fatal and injury accidents and PDO accidents.

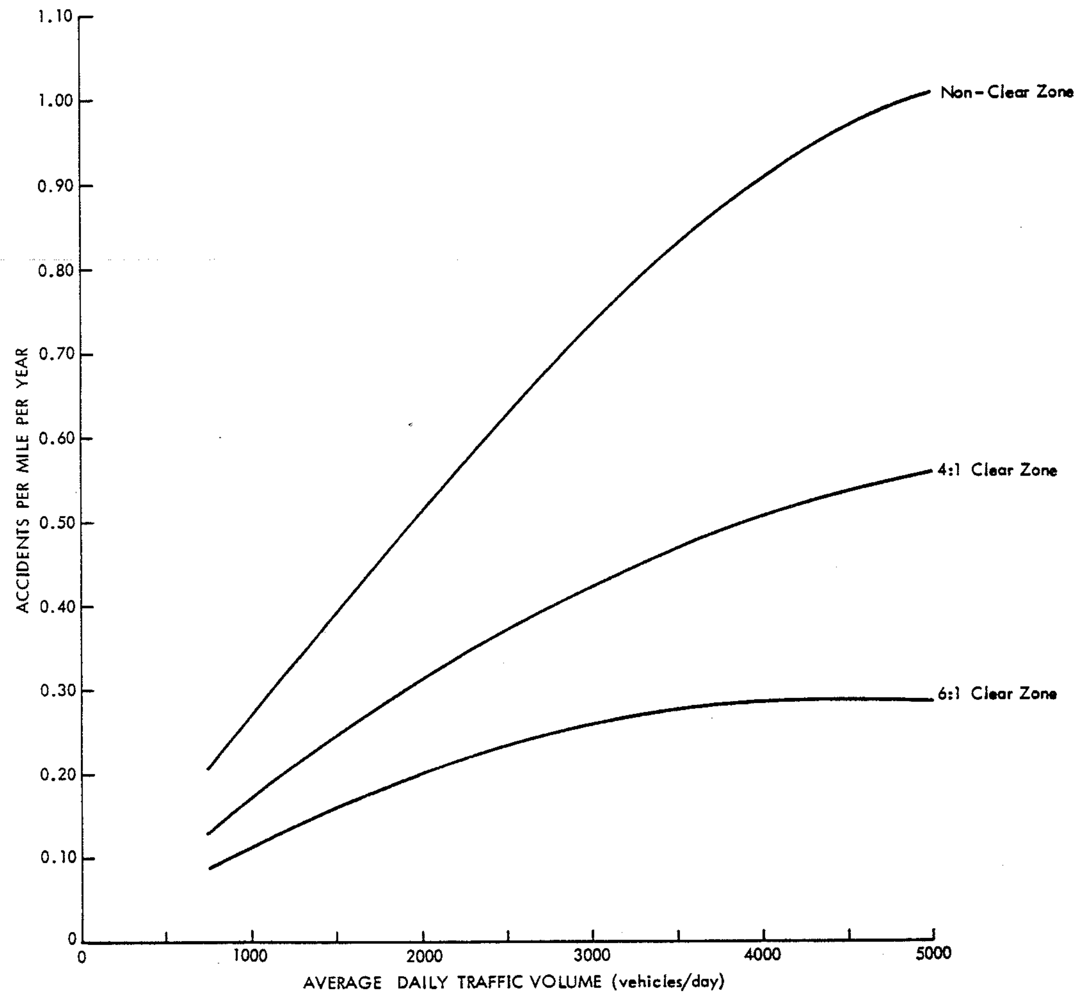
Relation of ADT and Single-Vehicle ROR Accident Rate

A further analysis was conducted to quantify the relation between ROR accident rate and ADT. The analysis of covariance results reported earlier have revealed that ADT can have a statistically significant influence on accident rate; therefore, the magnitude and direction of this influence was determined.

An analysis of covariance model generally represents the relation between a covariate and the dependent variable (in this case, the accident rate and ADT relation) as a straight line. As an extension of the analysis of covariance procedure, a statistical test can be used to determine whether the slope of the linear accident rate and ADT relation differs between the roadside design policies or whether a single common slope can be used for all roadside design policies. This determination requires a different model of analysis of covariance than the one used earlier because the ADT covariate must be entered into the model after, rather than before, the factor of roadside design policy. The comparison of slopes is performed by an F-test described by Ostle (4, p. 204) and is performed if, and only if, there is a significant linear relation between accident rate and ADT. The specific procedure used to perform the slope comparisons when using the SAS computer package was that described by G.A. Milliken and D.E. Johnson in Chapter XI of their unpublished paper "Analysis of Messy Data" prepared for an Institute of Professional Education seminar, Washington, D.C., June 1981 (298 pp.).

For two-lane highways, the slopes of the ROR accident rate and ADT regression lines for the three roadside policies are -0.038, -0.050, and -0.027 accident/million vehicle-miles per 1,000 vehicles/day for 6:1 clear zone, 4:1 clear zone, and nonclear zone sections, respectively. These three individual linear relations between accident rate and ADT were each statistically significant (i.e., the slope of

Figure 2. Relation between single-vehicle ROR accidents per mile per year and ADT for two-lane highways.



each regression is significantly different from zero).

An F-test to compare these slopes indicates that they do not differ significantly [$F(2,2018) = 0.596$], which indicates that the common slope of -0.041 accident/million vehicle-miles per 1,000 vehicles/day can be used for all three roadside design policies.

The negative slope of this relation indicates that the single-vehicle ROR accident rate decreases with increasing ADT. This same trend has been observed in other studies, where the total accident rate for two-lane highways has been found to decrease with increasing traffic volume, particularly at low traffic volume levels where single-vehicle accidents predominate (5). This relation can be expressed as

$$AR = -0.041\ ADT + b_o \tag{2}$$

where

- AR = single-vehicle ROR accident rate (accident/million vehicle-miles),
- ADT = average daily traffic volume (1,000 vehicles/day) and
- b_o = a constant that depends on the roadside design policy (b_o = 0.361 for 6:1 clear zone sections, 0.510 for 4:1 clear zone sections, and 0.787 for nonclear zone sections).

Figure 2 shows the variation of accidents per mile per year with ADT, based on the relation presented in Equation 2. The accident experience shown in Figure 2 has been expressed as an accident frequency per mile per year, rather than as an accident rate, in order to illustrate the magnitude of the average differences in roadside design policies; for this reason the relations shown in Figure 2 are nonlinear.

A similar analysis for fatal and injury ROR accident rate revealed that a common slope of -0.026 accident/million vehicle-miles per 1,000 vehicles/day should be used [$F(2,2018) = 1.06$]. This relation can be expressed as

$$AR_{FI} = -0.026\ ADT + b_o \tag{3}$$

where

- AR_{FI} = fatal and injury single-vehicle ROR accident rate (accident/million vehicle-miles),
- ADT = average daily traffic volume (1,000 vehicles/day), and
- b_o = a constant that depends on the roadside design policy (b_o = 0.166 for 6:1 clear zone sections, 0.251 for 4:1 clear zone sections, and 0.388 for nonclear zone sections).

For freeways, the accident rate and ADT regression lines for the 6:1 and 4:1 clear zone policies were not statistically significant. This finding means that the best estimate of the slope of the accident rate and ADT relation for freeways is zero. The same result was obtained for the relation between fatal and injury accident rate and ADT for freeways. The slope of the accident rate and ADT relations for four-lane divided nonfreeways were also not significantly different from zero. These results are in contrast to the results given for freeways in Tables 2 and 3, where ADT and the variables for roadside design policy were considered in a different order.

Estimation of Accident Rates

Accident rates were estimated by using the model in NCHRP Report 148 (6) for several situations where accident data were not available. For example, there were no freeway sections in the study that had a nonclear zone design policy because most freeways either had a 30-ft clear recovery zone originally or have since been upgraded. The model in NCHRP Report 148 was used to estimate the single-vehicle ROR accident rate for a freeway that had a roadside design comparable to a two-lane nonclear zone section. This estimation process required an adjustment of the model (described in detail in the project report) because the accident rates predicted by the model for the types of highway sections evaluated in this study were much higher than those actually found in the project data and reported in Table 5. After adjustment of the model, the estimates obtained for nonclear zone sections on freeways were 0.149 accident/million vehicle-miles for the fatal and injury single-vehicle ROR accident rate and 0.407 accident/million vehicle-miles for the total single-vehicle ROR accident rate.

It was also determined that, by using the model in NCHRP Report 148 for both two-lane highways and freeways, highway sections with 20-ft clear zones would experience single-vehicle ROR accident rates approximately 10 percent higher than similar highway sections with 30-ft clear zones. It was determined that, if the 4:1 clear zone policy was more uniformly applied (where roadsides were completely clear of unprotected fixed objects and no slopes were steeper than 4:1), reductions in the accident rate of about 5 percent could be obtained on freeways and about 25 percent on two-lane highways. Similar results were obtained for a more uniform application of the 6:1 clear zone design policy on freeways and two-lane highways.

COST-EFFECTIVENESS IMPLICATIONS

Four design examples have been developed to help readers interpret the cost-effectiveness implications of the findings obtained from the study. The purpose of these examples is not to suggest that the choice of roadside design policies can or should be based on a single design situation. It is recognized that both ROR accident rates and construction costs may vary from site to site. These examples are intended only to compare the average reductions in the accident rate and the typical construction costs for improving highways with one roadside design policy to another. Highway agencies are encouraged to develop similar cost-effectiveness approaches based on accident rate and cost data appropriate for their locale.

The cost-effectiveness analysis technique used for the design examples was a benefit-cost comparison of the present worth of both accident cost savings and construction costs. The criterion used to compare benefits and costs is the benefit-cost (B/C) ratio. Roadside improvements are considered cost effective or economically justified whenever the B/C ratio equals or exceeds 1.0. The computation of the B/C ratio was based on an analysis period of 20 yr. Because the major capital items in each improvement (earthwork and right-of-way) generally have service lives longer than 20 yr, those items were assigned a residual value at the end of the analysis period. The accident reduction estimates used for the analysis were determined from the accident rates given in Table 5 and an assumed value for the average ADT over the 20-yr analysis period. The discount rate used to obtain the present worth of future costs and benefits was 4 percent/yr. The 4 percent discount rate represents the real long-term cost of capital over the inflation rate. This rate allows the analysis to be conducted on a constant-dollar basis, with the effect of inflation excluded.

The differences between the roadside design policies in accident frequency per mile per year, and therefore in accident costs savings and the B/C ratio, increased with increasing traffic volume. Therefore, one method of illustrating the results of the benefit-cost analysis is to determine a break-even ADT; i.e., the traffic volume at which the present worth of the benefits of the accident reduction is exactly equal to the present worth of the construction cost of the improvement. The break-even ADT represents the minimum traffic volume at which a roadside design improvement on an average highway section would be cost effective. The break-even ADT has been computed, for illustrative purposes, in generalizing the design examples presented here. However, the computation of the break-even ADT is not essential when evaluating roadside design policies for an individual highway section with a known ADT. To determine the economic justification for roadside design improvements in such a case, it is necessary only to compute the B/C ratio for each incremental roadside design improvement (nonclear zone to 4:1 clear zone, or 4:1 clear zone to 6:1 clear zone) and compare it to 1.0.

The four design examples include a comparison of (a) nonclear zone and 4:1 clear zone roadside design policies for freeways, (b) 4:1 clear zone and 6:1 clear zone roadside design policies for freeways, (c) nonclear zone and 4:1 clear zone roadside design policies for two-lane highways, and (d) 4:1 clear zone and 6:1 clear zone roadside design policies for two-lane highways. The results obtained from the benefit-cost evaluation for each design example are given in Table 8, including the expected reduction in the accident rate, the cost savings per accident reduced, the construction cost of the improvement, and the break-even ADT. For each example, the break-even ADT was computed separately based on accident cost estimates developed by the National Safety Council (NSC) and the NHTSA.

The design examples for freeways indicate that the improvement from a nonclear zone to a 4:1 clear zone roadside design policy becomes cost effective in the ADT range of 3,820 to 5,410 vehicles/day. Thus, based on assumed conditions of construction cost and terrain, the use of at least a 4:1 clear zone roadside design policy is economically justified, on average, for all but a small portion of rural freeway mileage. The use of a 6:1 clear zone roadside design policy becomes cost effective for rural freeways in the ADT range of 6,100 to 8,650 vehicles/day. Although the results of these examples are not intended to provide a specific traffic volume level on which roadside design policy should be based, they do indicate that there is no single roadside design policy that is the most appropriate in all situations on rural freeways. Thus a flexible policy is needed where the most cost-effective

Table 8. Summary of benefit-cost evaluation for four design examples.

Roadside Design Policy Improvement	Freeways: Nonclear Zone to 4:1 Clear Zone	Freeways: 4:1 Clear Zone to 6:1 Clear Zone	Two-Lane Highways: Nonclear Zone to 4:1 Clear Zone	Two-Lane Highways: 4:1 Clear Zone to 6:1 Clear Zone
Expected accident rate reduction (accidents per million vehicle miles)	0.118	0.107	0.277	0.149
Accident cost savings ($/accident reduced)				
Based on NSC accident costs	7,748	7,748	9,266	9,266
Based on NHTSA accident costs	10,977	10,977	14,502	14,502
Improvement construction cost ($/mile)	31,265	47,148	19,029-66,804	22,984
Residual value of improvement after 20 yr ($/mile)	14,753	25,407	8,873	13,622
Breakeven ADT (vehicles/day) for B/C = 1.0[a]				
Based on NSC accident costs	5,410	8,650	1,180-4,930	2,450
Based on NHTSA accident costs	3,820	6,100	750-3,150	1,560

[a]For computation, see NCHRP Report 247 (1).

roadside design is selected for each section of highway.

The results of the design examples for two-lane highways are not as clear as the results for freeways because the roadside designs found on two-lane highways and the costs of improvements in roadside design are more variable than for freeways. The construction cost for improving a two-lane highway with a nonclear zone roadside design policy to a 4:1 clear zone roadside design policy was highly dependent on the number and type of roadside objects to be removed. Depending on the construction cost used and the selection of the NSC or NHTSA accident costs, the improvement from the nonclear zone policy to a 4:1 clear zone policy could become cost effective anywhere in a broad ADT range of 750 to 4,930 vehicles/day. If a 4:1 clear zone design policy is justified for a highway section, or if an existing highway already has a 4:1 clear zone design policy, a further improvement to a 6:1 clear zone design policy would become cost effective in the range of 1,560 to 2,450 vehicles/day. There are situations on two-lane highways where either the 6:1 clear zone, the 4:1 clear zone, or the nonclear zone roadside policy may be the most cost-effective approach. Although the results obtained from the design examples for two-lane highways are more difficult to generalize than the results for freeways, the need to consider cost-effectiveness in determining the roadside design policy remains the same.

CONCLUSIONS

The major conclusion of this study is that there is a statistically significant relation between single-vehicle ROR accident rate and the roadside design policy used outside of the highway shoulder. The study findings provide estimates of the single-vehicle ROR accident rates for highway sections with and without clear recovery zones and for clear recovery zones of varying slope and width. These measures of effectiveness are summarized in Table 5. The variation of these measures of effectiveness with ADT can be examined by using Equations 2 and 3.

Four design examples demonstrate a cost-effectiveness comparison of the average accident reduction benefits and typical construction costs for improvements in roadside design policy. It is not suggested that decisions about roadside design policy can be based on these four examples. Nevertheless, the examples do indicate that there are situations where it is most cost effective to provide clear recovery zones with 6:1 slopes, other situations where it is most cost effective to provide clear recovery zones with 4:1 slopes, and still other situations where it may not be cost effective to improve roadside design outside the shoulder area at all.

The major recommendation resulting from the research is that roadside design policies should be flexible in order to provide a cost-effective roadside design for each highway section (e.g., each highway project). The benefit-cost evaluation procedure used for the design examples in this study is suitable for the evaluation of roadside design policies. The maximum return will be obtained from roadside design improvements if a cost-effectiveness analysis is conducted for individual highway sections.

It is recommended that the average accident rates developed in this study be used to determine the benefits of improvements in roadside design policy, unless more site-specific data can be obtained. Particular attention should be paid to adjusting the measures of effectiveness for sites that have extremely high or extremely low roadside accident rates. Site-specific estimates of the construction costs for improvements in roadside design should also be used. Nevertheless, it is recognized that agencies, for legal and administrative reasons, may want to adopt policies that use consistent designs for highways of similar functional class and traffic volumes. Such policies can be developed by each agency for classes of similar highways in a manner analogous to the design examples presented in this paper, based on estimates of construction costs, accident costs, interest rates, and service life appropriate for that agency. It is recommended that sufficient flexibility should be retained in such policies to allow modified designs for locations with extremely high or extremely low values of construction cost or effectiveness.

ACKNOWLEDGMENT

The research reported in this paper was performed as part of Project 17-5 of NCHRP. We gratefully acknowledge the assistance provided by the NCHRP staff, the project panel, and the cooperating state highway agencies in the performance of the research.

Discussion

Tom Mulinazzi*

I am impressed by the way Graham and Harwood document the findings of their research. I especially approve of one of their major recommendations--that roadside design policies should be flexible in order to provide a cost-effective roadside design for each highway section or highway project.

It is my belief that a large percentage of the single-vehicle encroachments off the roadway go unreported. This is shown in Figure 2. I do not believe that the 4:1 clear zone, the 6:1 clear zone, or the nonclear zone helps keep the vehicle on the traveled way. I do believe, however, that the use of the 6:1 clear zone permits many more single-vehicle encroachments to go unreported than on the 4:1 clear zone, and also more unreported encroachments on the 4:1 clear zone than on the nonclear zone. The 4:1 clear zone is steeper than the 6:1 clear zone, which would have a tendency to force vehicles to travel further from the traveled way on the 4:1 clear zone. The nonclear zone, by definition, has more fixed objects for an errant vehicle to hit. To repeat myself for emphasis, I do not believe that a 6:1 clear zone keeps vehicles on the traveled way; however, I do believe that the 6:1 clear zone allows more vehicles to reenter the roadway without having a reported accident.

In conclusion, I commend the authors for writing a well-documented paper and presenting some facts that should be useful for practicing highway engineers.

J.W. Hall**

Graham and Harwood have made a significant contribution by quantifying the level of effectiveness of clear recovery zones. Because their research provides the highway engineer with a technique for assessing roadside safety improvements, the interested reader is strongly encouraged to review the authors' work in NCHRP Report 247 (1). Because of its greater length, the full report more thoroughly documents the procedures and analyses used in this study.

The authors' research was funded by NCHRP, which imposes certain technical, financial, and time constraints on researchers. These constraints are clearly evidenced in their paper, which primarily focuses on the roadside while devoting only minimal attention to the roadway. The authors also incorporate previous reports and some secondary data sources to fill gaps that could not be examined in suitable detail because of the project's budgetary and temporal limitations. The reader who is cognizant of previous research involving ROR crashes will have little difficulty in recognizing how this research supplements the current state of the art; the more casual reader, however, could misinterpret some of the study results. The intent of this discussion, therefore, is to comment on several items that could cause confusion.

In Appendix D of NCHRP Report 247, the authors distinguish roadside encroachments from ROR crashes. The paper fails to make this distinction; consequently, the reader might erroneously conclude that improved roadside design will reduce the frequency with which vehicles leave the road. What has been learned from the project is that only those encroachments that meet the definition of an accident and that are reported decrease with improved roadside design. The researchers' difficulty in quantifying this decrease could be due in part to an underreporting of encroachments on safer roadsides where accident threshold damages are exceeded, but where the impacting vehicle is still drivable.

The project also conducted a rather casual qualitative evaluation of roadway curvature and no measurement of gradient at 130 randomly selected sites. A number of previous studies (7,8) have found a relation between these geometric features and the occurrence of ROR crashes (and presumably encroachments). The researchers were unable to distinguish roadside type on the basis of the proportion of sites with tangents for any of the three roadway types studied. Although this finding is mildly surprising, the reader should not interpret this to mean that alignment does not influence the occurrence of ROR accidents.

Because of project limitations the researchers were forced to rely on a restricted sample for this study. The researchers offer appropriate justification for their use of data from Illinois, Minnesota, and Missouri; indeed, the more than 11,000 accidents considered in this project appear adequate. Nevertheless, the authors fail to discuss the extent to which these states typify the situation in the remainder of the country. Without detracting from the findings in the study, it is noteworthy that, as a group, these three states exhibit accident characteristics that are somewhat different from those of other states.

Data from the Fatal Accident Record System (FARS) for 1980 reveal that these three states account for 8.26 percent of the fatal accidents studied in this research (rural, single-vehicle, ROR accidents on two- and four-lane highways), a figure slightly in excess of their 7.82 percent share of all fatal accidents. It is also noteworthy that a significantly lower percentage (46 percent) of these fatal accidents in the study states occur on curves than that reported for the remainder of the country (52 percent). It may also be relevant that less than 13 percent of the field sites studied in this research exhibited curvature; therefore, these may not be representative of the actual sites where these crashes are occurring. In addition, roadway alignment differences among these three states may partly account for the state-to-state differences in single-vehicle ROR crash rates observed by the researchers. Although it would be improper to draw far-reaching conclusions from the generalized alignment data provided by FARS, jurisdictions where adverse geometrics are more frequently associated with these types of crashes should recognize this difference when interpreting the results of this study.

The authors have clearly indicated that what they refer to as a 4:1 clear zone policy is actually a variable criteria that does not preclude slopes steeper than this value. Unfortunately, the policy is so variable that the average foreslope values cited are of questionable value. Actually, the inherent variations in roadside slopes with both longitudinal and lateral displacement make it exceedingly difficult to categorize this parameter, even for those sections of roadway supposedly built according to a specified policy. It is also not clear if the measurement techniques used by the researchers ignored slopes less than 5 ft.

The researchers used the model developed by Glen-

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non (6) to estimate accident rates for various roadside design policies. Predicted accident rates for freeways were 8 times the observed values. Although these differences are clearly troublesome, they are consistent with Glennon and Wilton's admonition (9) that the encroachment frequencies used in Glennon's earlier model are simply "order of magnitude" estimates. It has been suggested in recent research that the original encroachment data from Kennedy and Hutchinson have been applied far beyond their limits of applicability, thus producing unreliable results. In their paper the researchers have attempted to estimate accident rates for freeway sections with nonclear zone designs by adjusting predicted accident rates with a correction factor. Although the analysis appears to be conservative, the expected reduction in the accident rate for this type of roadway must be viewed with suspicion.

At the risk of engaging in a semantic debate, the researchers' use of the term "unprotected fixed object" must be criticized. Fixed objects do not need protection; people do. Since the mid-1960s highway engineers have recognized that one of their major responsibilities is to protect vehicle occupants from injury. A number of terms, including the possibly overworked "roadside obstacle," would be more suitable.

Although the focus of their paper is on clear recovery zones, the researchers incorporate the techniques of engineering economy to demonstrate the application of the findings of their study. They properly note some limitations of their examples and suggest that individual analyses be conducted at specific sites. They have chosen to use the 4 percent discount rate mentioned in the AASHTO Red Book (10) without noting that a number of researchers, including Kimboko and Henion (11), have raised objections to this approach. The use of a break-even analysis with a B/C ratio of 1.0 was appropriate for demonstration purposes in their paper, but the reader should be cautioned that such an analysis could yield incorrect results if it were applied in an attempt to determine the optimal expenditure of highway safety funds.

The researchers have provided an adequate assessment of the effectiveness of clear recovery zones. Although the study implies that the effectiveness derives primarily from roadside rather than roadway characteristics, evidence suggests that ROR crashes are not distributed uniformly or randomly along the roadway, and this concept is reinforced by the alignment differences between the FARS and random site data for the study states. Those people who use the findings from this study should recognize that the actual effectiveness of an improved roadside design policy will be greater at those locations where roadside encroachments are more likely to occur.

Authors' Closure

We thank Mulinazzi and Hall for taking the time to review and discuss our paper; we appreciate their comments.

Hall is quite preceptive to recognize that funded research must address the objectives defined by the sponsor and within constraints set by the sponsor. The objective of this study was to evaluate the safety effects of roadside design policies rather than to evaluate the safety effects of particular geometric features. The study was intended to address the following questions: Are clear recovery zones generally effective? and, What roadside design policies are most appropriate for application over relatively long sections of highway that traverse a variety of terrain features? The study was not intended to investigate the optimal roadside design at any specific location.

The evaluation was accomplished by using study sections that represent the variety of terrain, roadway geometrics, and roadside geometrics actually found in the field on highways constructed under each policy. The safety measures developed must be interpreted as averages over the mix of cut-and-fill sections and the distribution of embankment slopes, embankment heights, and fixed objects actually found for each policy in the field. The section on Research Approach explains that any further consideration of the incremental effects of specific roadside features or roadway geometrics would have required a detailed inventory of highway sections that would have been far beyond the resources available to the study. Nevertheless, we believe that the results reported in the paper address the objectives defined by the sponsor of the research.

Hall notes, as we did, that the highway sections constructed under the 4:1 clear zone policy contained some embankment slopes steeper than 4:1 and some fixed objects within the 30-ft clear recovery zone. We addressed the issue by using the model in NCHRP Report 148 (6) to estimate that, if the 4:1 clear zone policy had been uniformly applied, the resulting accident rates would have been 5 percent less than the reported results on freeways and 25 percent less than the reported results on two-lane highways.

Hall correctly observes that highways in the three states considered--Illinois, Minnesota, and Missouri--are slightly less likely to contain horizontal curves than highways in the nation as a whole. Based on our field survey sample, horizontal curves constitute approximately 13 percent of the total length of the study sections. (But we wish to point out that this is not the same as saying that only 13 percent of the study sections contained horizontal curves.) By contrast, the geometric inventory of highway sections assembled for the FHWA study, "Effectiveness of Alternative Skid Reduction Measures" (12), which included more than 2,000 miles of highway in 15 states from all regions of the country, found 15 percent of the total length of these sections to be on horizontal curves.

Hall raises the question of whether slopes shorter than 5 ft were measured. We wish to reassure him that the slope and length of all embankments, however short, were measured.

Hall makes a valid semantic point about possible misinterpretation of the term "unprotected fixed object," by which we meant a fixed object that was not behind a guardrail. Although the meaning of the term unprotected used in this sense is obvious, we agree that another term such as "exposed fixed object" might be preferable.

Finally, we are in complete agreement with Hall's statements that ROR accidents are not distributed uniformly or randomly along the roadway and that the effectiveness of improved roadside designs will be greater at locations where roadside encroachments are more likely to occur. It may be far more cost effective to concentrate the available dollars on improving the roadside at specific locations with the greatest need (such as on the outside of horizontal curves) than to require a uniform roadside design over extended sections of roadway. We strongly believe that further research is needed to quantify the roadside accident rates on horizontal curves and other high-encroachment-rate locations to provide a basis for more flexible and cost-effective design policies.

We are gratified that the comments by Mulinazzi echo this need for flexibility in roadside design. The results of this study indicate that improvements in roadside design can be cost effective, but unfortunately we are a long way from achieving the goal of maximum cost-effectiveness through flexible design.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Notice: The findings and opinions presented in the paper are those of the authors and do not necessarily reflect the opinions of NCHRP, its sponsors, or the highway agencies that assisted them.

Abridgment

Comparing Operational Effects of Continuous Two-Way Left-Turn Lanes

DAVID P. McCORMICK AND EUGENE M. WILSON

In this paper the operational effects of continuous two-way left-turn lanes (TWLTLs) were compared with four-lane sections and five-lane Z-turn-pattern sections. Both three-lane and five-lane TWLTL sections were examined. These comparisons were made in order to determine under which circumstances a particular alternative will produce the best results from the standpoint of movement efficiency and safety. The following variables were monitored and evaluated in this study: traffic counts, speed surveys, lateral placement, conflicts, accident histories, site accesses, turning movements, day or night operations, and dry or wet pavement conditions. The TWLTL treatments were effective under a variety of turning and main line volumes. Statistically, lower mean conflict rates were observed for the five-lane TWLTL when compared with either the four-lane or Z-pattern treatments. Three-lane TWLTLs were superior to four-lane segments under more selective circumstances.

One of the consequences of locating retail establishments outside the confines of a central business district (CBD) has been the advent of strip commercial areas. These areas rely on the linear high densities provided by traffic on arterial systems as a substitute to the CBD, while catering to the public's desire for drive-up convenience. Although such areas have been successful for some retail establishments, the transportation implications of strip commercial zones create one of the toughest design problems for the transportation engineer.

In developed strip commercial areas, continuous two-way left-turn lanes (TWLTLs) have been used as a possible arterial improvement. In this paper the operational effects of TWLTLs were compared with four-lane sections and five-lane Z-turn-pattern sections. This comparison was made in order to determine under what circumstances a particular alternative will produce the best results from the standpoint of movement efficiency and safety.

LITERATURE REVIEW

A literature search revealed several aspects about TWLTL operations. Glennon and others (1) suggested that the use of a TWLTL is warranted when the average daily traffic (ADT) volumes are between 10,000 and 20,000. This range is typical of the operating conditions found in the literature. In many cases the volume data were not broken down further than ADT. The appeal of using peak-hour data was alluded to by Cribbins and others (2): "Findings of this investigation indicate that median openings, per se, are not necessarily accident prone under conditions of low volumes, wide medians, and light roadside development; however, as volumes increase and development increases commensurately, the frequency of median openings does have a significant effect on accident potential." Following this logic, the proper condition for monitoring problems in median control is under high-volume, peak-hour use.

Left turns from median openings account for the largest proportion of driveway accidents. Of the left-turn accidents, rear-end occurrences are the most numerous. Studies indicate that about 42 percent of all median opening accidents were accountable to left turns, 19 percent to right turns from the street, and 39 percent while exiting the driveway (2). Utkotter (3) indicated that left-turn movements were involved in 71 percent of personal injury driveway accidents. Left-turn accidents in a strip commercial setting are significantly more severe than other types of driveway accidents.

The majority of all studies on TWLTLs have focused on accident rates. Many reports indicated that TWLTLs reduce accidents where no previous turn lane existed (1,4-6). Accident reductions up to 62 percent were reported.

Relatively few researchers chose to apply conflict analysis to their study sites. Of those that did, the following conclusions were reached. Nemeth (4) participated in three before-and-after studies--two studies on two lanes that were restriped in order to accommodate three lanes, and one study on a four-lane section that was expanded to five lanes. His studies indicated that, at one site, the conflicts were reduced significantly by the use of the TWLTL. Walton and others (5) observed conflicts and indicated no significant variation in conflicts by type of design.

It has been suggested that TWLTLs be used only where conventional raised or flush medians are not practical. The warrant developed by Glennon and others (1) stated that "the level of development should exceed 60 driveways per mile, with less than 10 high volume driveways." Staggered accesses have been indicated as being particularly well suited for TWLTLs.

When a four-lane roadway section has been changed to a five-lane TWLTL, main line delay on high-volume, high-access-density facilities has decreased. Nemeth (4) indicated that when a four-lane section was retrofitted with a three-lane roadway section with a TWLTL, an increase in delay occurred because of the reduction in the number of lanes. He concluded that, "It appears that the access function of this roadway was improved at the price of measurable deterioration of the movement function." Volume appeared to be the key variable when retrofitting a four-lane roadway with a three-lane TWLTL because Jomini (7) reported that, on a facility with a lesser traffic volume, no significant increase in delay and no substantial reduction in accidents occurred.

It is apparent from the literature that, although much has been learned about TWLTLs, there are a number of gaps in the knowledge of the operational characteristics of these facilities. Some of the most obvious are (a) lack of understanding of the effects of nighttime conditions on operations, (b) absence of a study on adverse weather effects on TWLTL operations, and (c) comprehensive comparison of TWLTLs with other types of median treatments.

COMPARISON OF ALTERNATIVE GEOMETRIC DESIGNS

A conflict analysis was used in order to evaluate the operational differences of various geometric designs. Although the traffic conflicts technique did not correlate well with many types of accidents (r^2 between 0.21 and 0.70), it appears to have a strong relation to left-turn accidents. Glave and Migletz (8) stated that, "In particular, accidents involving cross traffic and opposing left turns may be correlated to traffic conflicts of an analogous nature." One definition of a traffic conflict is, "An event involving two or more road users, in which one user performs some atypical or unusual action, such as a change in direction or speed that places another user in jeopardy of a collision unless an evasive maneuver is undertaken" (8).

An important aspect to note is that unless two road users are present a conflict cannot occur. Also, an unusual action must occur. For example, stopping at a stop sign does not qualify as a conflict. When an unusual action takes place that does not place another vehicle in jeopardy, it is categorized as a potential conflict. Potential conflicts were included for improper driving maneuvers only. Twelve different traffic conflicts were observed in this study.

Seven study sites were selected--three with five-lane TWLTLs, two with Z-pattern left-turn lanes, one with a three-lane TWLTL, and a four-lane road that does not have a turn lane (see Figure 1). Volumes ranged from 750 to 2,200 vehicles/hr, and speed limits were from 30 to 40 mph. Section widths on the four- and five-lane sections were from 50 to 68 ft, and the three-lane section was 46 ft wide. Turn lanes varied from 10 to 14 ft, and access density ranged from 27 to 165/mile.

Analysis of data from 58 hr of observations was performed in order to isolate specific aspects of driver behavior by type of facility. Visual studies of the data served as the original step in the analysis of these observations. The first step in this visual analysis was to plot conflicts by site under a number of different groupings: percentage of left turns, vehicles per hour, day or night operations, wet or dry conditions, and accesses per mile.

Figure 2 shows that a tight linear grouping exists in the data points when broken down by type of median treatment. These data suggest a linear relation between conflicts and percentage of left turns, as well as a difference in conflicts by treatment type.

Conflicts were then broken down by type. The most prevalent conflict was the rear-end conflict for all treatments, except the three-lane TWLTL section. This conflict reflects the ability of each

Figure 1. Lane geometry.

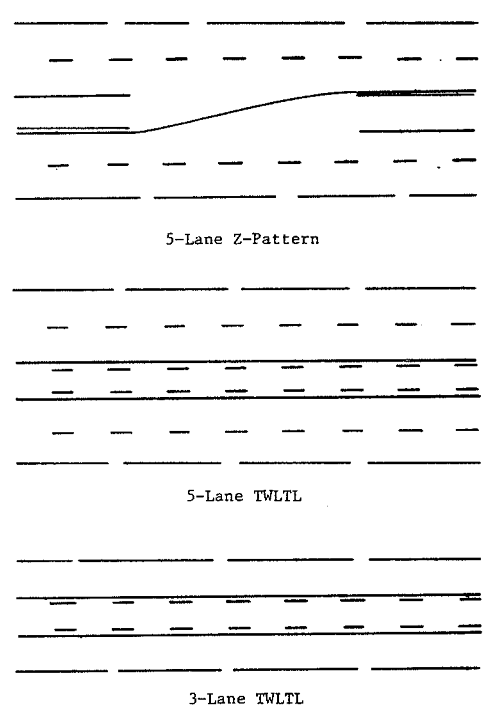


Figure 2. Conflicts versus lane geometry and percentage of left turns.

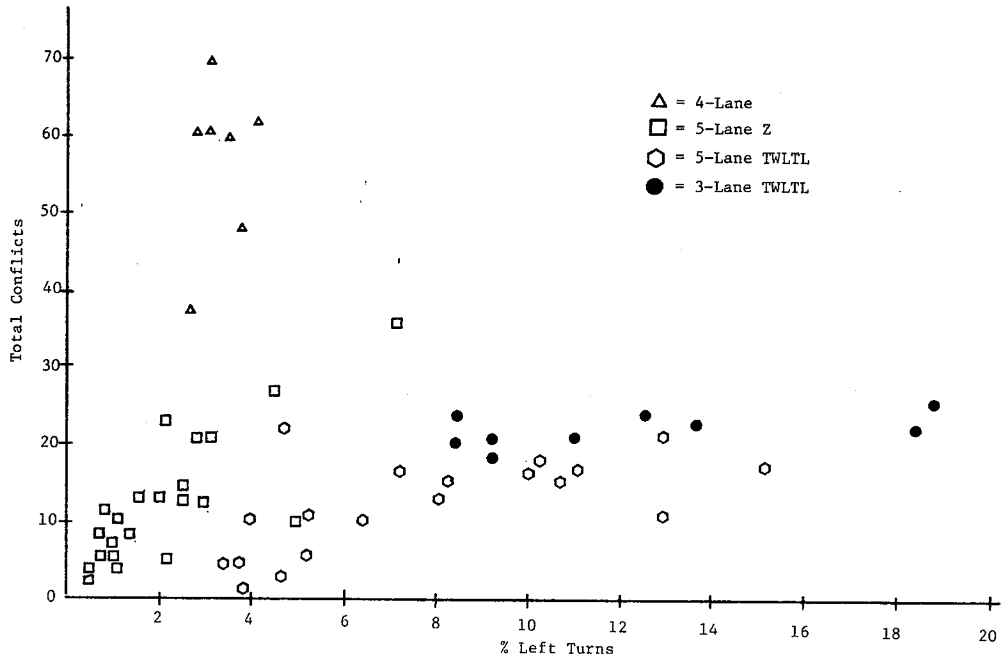
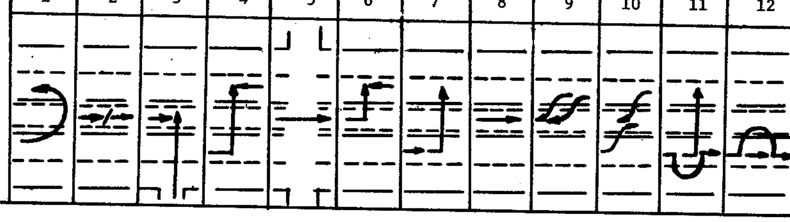


Figure 3. Conflict occurrence by type of median treatment.



	CONFLICT OCCURRENCE BY TYPE (% OBSERVED)												
	1	2	3	4	5	6	7	8	9	10	11	12	
5-Lane TWLTL	0.8	0.4	3.9	0.8	0.0	18.4	41.0	3.3	2.5	0.0	29.9	0.0	100
3-Lane TWLTL	0.0	1.6	2.2	0.0	0.0	6.9	16.5	8.0	4.3	12.8	0.0	47.9	100
Z-Pattern	1.9	0.0	0.8	1.9	3.8	3.4	61.8	0.0	7.2	0.0	19.1	0.0	100
4-Lane	0.2	0.0	3.7	7.2	0.0	0.0	83.6	0.0	0.0	0.0	5.2	0.0	100
Total Observations	7	4	30	40	10	66	678	23	33	24	151	90	1156

treatment to remove vehicles from the travel lane when needed. The data in Figure 3 indicate that the three-lane section had the lowest percentage of rear-end conflicts, followed by the five-lane TWLTL, the Z-patterns, and finally the four-lane section. The Z-pattern has only limited value in separating turning traffic from the through lane in strip commercial areas because of midblock turns. The high percentage of rear-end conflicts on the five-lane TWLTL section suggests that, even with the turn lane, a large number of vehicles do not make use of it when performing a turn. Also note the high percentage of conflicts from passing in the turn lane (48 percent) in the three-lane section. This is the result of right-turning vehicles blocking the through lane and, consequently, traffic moving around it. This misuse of the TWLTL does not occur in the five-lane section because of the additional lane.

Statistical procedures were used to augment, analyze, and prove this visual inspection, as well as to probe other hypotheses intuitive to engineering experience. Analysis of variance and covariance were the statistical tools used. Conflict data were normalized to reflect 300-ft study sites (except for conflict data) compared to the percentage of left turns, which were already normalized.

It was discussed previously that the seven sites were investigated jointly by median type. The assumption was that the locations that use the same treatment behaved similarly under the same conditions and could be treated as a homogenous group for analysis. By using analysis of variance at the 0.05 level, this hypothesis could not be rejected, which

Table 1. Statistical summary.

Null Hypothesis	Statistical Test	Test Results
Mean conflict rates equal for similar TWLTL	F	$F = 2.112 < F_{1,20,0.05} = 4.35$ Cannot reject Ho
Mean conflict rates equal for Z-patterns	F	$F = 0.00 < F_{1,12,0.05} = 4.75$ Cannot reject Ho
Mean conflict rate equal between median treatments	F	$F = 90.72 > F_{3,54,0.05} = 2.80$ Reject Ho
Mean conflict rate equal between median treatments (adjusted for vehicles per hour and percentage of turns)	F	$F = 51.35 < F_{3,52,0.05} = 2.80$ Reject Ho
Mean conflict rates equal over all observed volumes (adjusted for percentage of turns)	F	$F = 16.119 > F_{4,52,0.05} = 2.57$ Reject Ho
Mean conflict rates equal over all percentages of turns observed (adjusted for vehicles per hour)	F	$F = 7.30 > F_{2,54,0.05} = 3.19$ Reject Ho
Mean conflict rates equal for day and night operation (adjusted for vehicles per hour and percentage of turns)	F	$F = 0.649 < F_{1,54,0.05} = 4.04$ Cannot reject Ho
Mean conflict rates equal for snowpack and clear conditions (adjusted for vehicles per hour and percentage of turns)	F	$F = 0.204 < F_{1,54,0.05} = 4.04$ Cannot reject Ho

indicated that the data may be analyzed by median treatment and not strictly by study site.

Scheffe pairwise comparisons were performed at the 0.05 level on all four treatment means, and it was revealed that each mean was contained in a different subset. A summary of these statistical test results is given in Table 1. The analysis was refined further by using analysis of covariance to factor out the effects of volume and percentage of left turns. The results also indicated a significant difference in conflict rates between sites. The adjusted mean conflict rates were highest for the four-lane section (22.1) and the three-lane TWLTL (17.6), and the Z-pattern and five-lane conflict rates were 9.1 and 4.8, respectively. Although the three-lane TWLTL conflict rate is high, it is a treatment often used to improve Z-lane streets to reduce the left-turn problem. The three-lane section conflict rate is less than the four-lane section, which suggests that the three-lane section may be superior when a high-volume, high-turn rate situation arises and width is not available to restripe to five lanes.

Other hypotheses were applied to test for differences in conflict rate due to volume, access density, percentage of left turns, day or night operation, and adverse weather conditions. It was found that as the volume increased the mean conflict rate increased. The data suggest a relation between conflicts and volume that is similar to the relation between volume and accident rates. As the percentage of turns increased, the conflict rate also increased. The type of treatment was not isolated in this analysis.

Because of large numbers of vehicles that do not use the turn lane and because of the generally adverse conditions while snowpack conditions were present, it was expected that an increase in conflicts would occur; this was not the case. The hypothesis that mean conflict rates are equal for snowpack and clear conditions was not rejected. It can only be reasoned that the poor driving conditions were offset by increased driver awareness.

The comparison of day and night operation with the hypothesis that mean conflict rates are equal for day and night operation also could not be rejected. This may be partly explained by the generally excellent lighting found along the study sites and along strip commercial areas in general.

SUMMARY AND CONCLUSIONS

The provision of a safe, efficient means of accessing strip commercial areas (or other locations that have a high density of unsignalized intersections and accesses) is a challenge to the transportation engineer. Left turns have the potential to significantly increase delay, reduce capacity, and increase accident rates. With this in mind, the continuous TWLTL was observed and compared to two other common treatments: the five-lane Z-turn pattern and the four-lane section that does not have turn lanes. Seven study locations were evaluated by conflict analysis, speed surveys, lateral placement, turning movements, and other techniques. Statistical techniques were used to evaluate conflicts with respect to median treatment, day or night operation, and snowpack or clear condition.

The results of this research indicate that a properly installed TWLTL functioned efficiently on urban arterial streets with high roadside development. The following specific conclusions have been reached during the course of this investigation.

1. The TWLTL had a substantially lower mean conflict rate than either the four-lane or Z-pattern treatment. Conflicts were one-fifth those obtained on the four-lane treatment and nearly one-half those on the Z-pattern treatment.
2. The five-lane TWLTL lowered conflict rates over a wide range of volumes, speeds, and turning frequencies. The results of this study suggest that TWLTLs are warranted under much lower turning frequencies and volumes than previously had been suggested. However, under low opposing volumes (<400 vehicles/hr), TWLTLs will probably be ignored by a large number of drivers. The advantage of spatial separation should not be overlooked.
3. The TWLTLs virtually eliminate U-turn conflicts. The suggested use of barrier medians to control conflict locations and prevent left turns often results in increased U-turns. Only seven U-turns were observed during this study (out of 75,000 total vehicles).
4. The five-lane TWLTL alternative is recommended over the four-lane section without any median treatment when left turns are permitted on arterials that have strip commercial development.
5. The three-lane TWLTL may be superior to the four-lane section that does not have median treatment, especially when the width of the four-lane section is limited and left turns are permitted on arterials that have established strip commercial development.

ACKNOWLEDGMENT

We wish to express our appreciation to the Wyoming Highway Department, and in particular to the Traffic Operations and Traffic Design Divisions, for their valuable help in the course of this research, and to the FHWA for their cooperation in the project (Wyoming HPR-17-070NFE), from which this paper originated.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Accident Implications of Shoulder Width on Two-Lane Roadways

JAMES C. BARBARESSO AND BRENT O. BAIR

Previous studies regarding the accident implications of shoulder width have been inconclusive and their results contradictory. Engineering guidelines concerning shoulder width have been established, but emphasis is placed on the minimum shoulder width necessary for emergency parking and not on the effects of shoulder width on accident experience. The accident implications of shoulder width on two-lane roadways in an urban county in Michigan are investigated. Some liability claims against the county road agency have alleged that shoulders, which are at variance with shoulder-width guidelines, are hazardous because they do not adhere to the suggested guidelines. One intent of this paper is to determine whether these allegations are substantiated. Analyses were performed to determine whether there is a significant difference in accident frequency between two-lane roadways that meet shoulder-width guidelines and those that do not meet the guidelines. The results of this research do not support the premise that roadways with wider shoulders have significantly fewer accidents than roadways with narrow shoulders. No significant difference in accident frequency was found between roadways that meet shoulder-width guidelines and those that do not meet the guidelines. Accident data reviewed in this study reveal that shoulder width is not related to the frequency of overturn accidents, head-on type accidents, or to accident frequency in general, even after traffic volume and other variables are considered. A relation was discovered between the frequency of fixed-object accidents and shoulder width, but the findings indicate that fixed-object accident frequency is significantly lower on roadways with shoulders <7 ft wide than it is for roadways with wider shoulders. It was concluded from this research that (a) projects to reduce accident frequency should focus on factors that exhibit greater influence on accident frequency than does shoulder width; and (b) although it is desirable to adhere to current guidelines wherever possible, when undertaking certain types of construction projects it may be acceptable to retain existing shoulders of <8 ft in width unless a review of accident data for the project location indicates otherwise.

Previous studies regarding the accident implications of shoulder width have been inconclusive and their results contradictory (1-7). Engineering guidelines concerning shoulder widths have been established (8,9), but these guidelines emphasize the minimum shoulder width necessary for emergency parking and not the impact of shoulder width on accident experience. It would be advantageous to provide adequate facilities for emergency parking, but to adhere to these guidelines on all roadways would not be financially feasible.

It is more practical to investigate the effect of shoulder width on accident occurrences and pinpoint locations where accident experience can be related to shoulder width or a combination of shoulder width and other roadway factors. If such locations can be determined, then countermeasures to alleviate the accident situation could be implemented.

The accident implications of shoulder width on two-lane paved roadways are investigated in this paper. Analyses were performed to determine

1. Whether there is a significant difference in accident frequency between two-lane roadways with shoulder widths that meet the guidelines and those that do not meet the guidelines, and
2. Whether there is a relation between certain accident types and shoulder width.

The primary purpose of this research was to determine the relation between accident characteristics and shoulder width on two-lane roadways in Oakland County, Michigan. The research did not address the liability exposure of the residents of Oakland County or the Oakland County Road Commission (OCRC) due to shoulder widths less than those recommended by current engineering guidelines. Nevertheless, some liability claims against the Road Commission have cited narrow shoulders as contributing factors in certain accidents because the shoulders do not conform to the guidelines. If shoulder width is a contributing factor in certain accident types or the frequency of accidents, corrective action by OCRC may be justified.

PREVIOUS STUDIES

The multiplicity of studies concerning the effects of shoulder width on accident occurrences has resulted in an array of contradictory and often inconclusive findings. Transportation professionals have been forced to choose among these varied results for years.

In a critique of these past research attempts, Zeeger and Perkins (10) concluded that studies that found wider shoulders associated with safer conditions were the most reliable. They based their con-

Figure 1. Accidents by shoulder width.

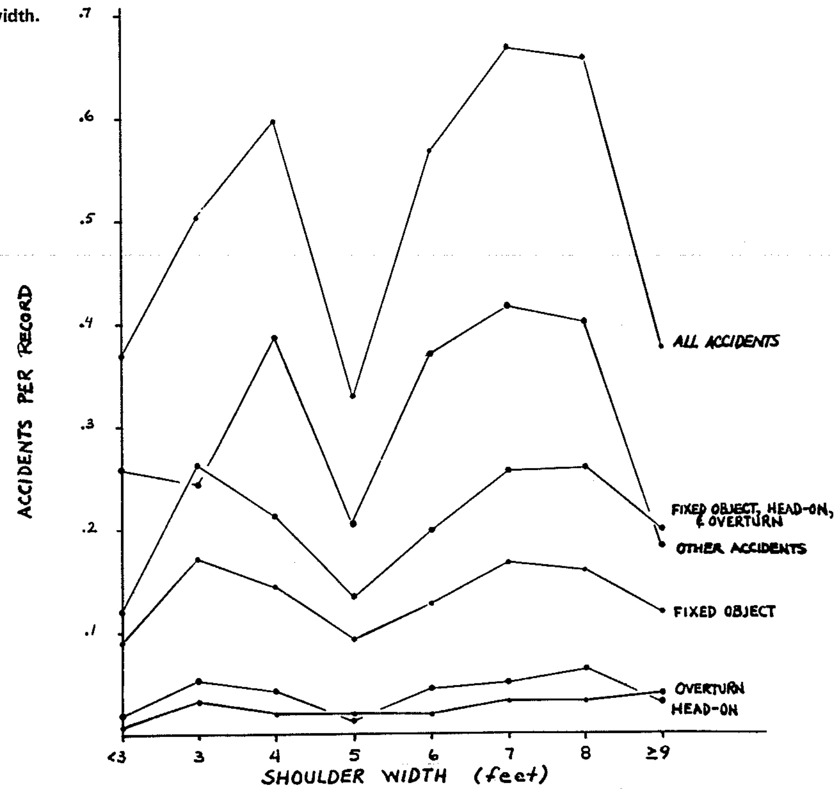
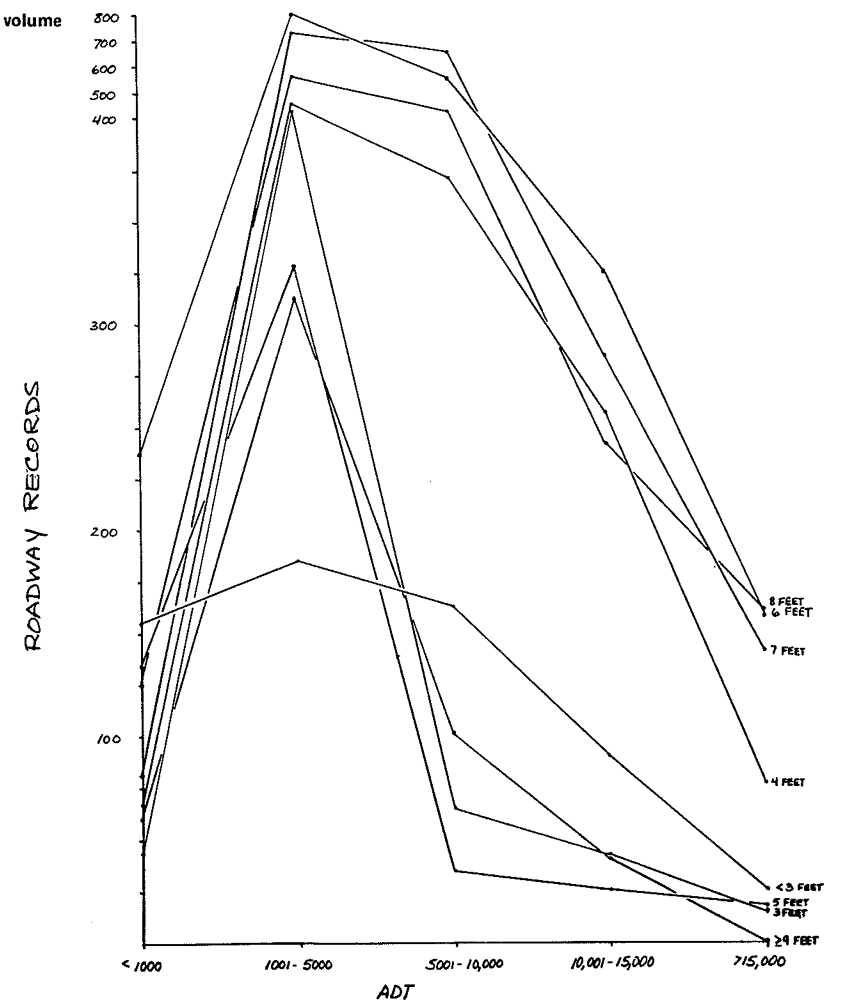


Figure 2. Roadway records by traffic volume and shoulder width.



clusion on a set of criteria concerning the type of analysis used, the reliability of the data, the sample size, and the importance of categorizing shoulder-width effects by accident type.

Attempts were made in this study to integrate the conclusions of Zeeger and Perkins in order to improve the reliability of the findings of this paper. A comparative analysis of accident experience by shoulder width on all paved two-lane major roadways (637 miles) in Oakland County was conducted. Roadway geometric data and accident data were derived from computer files established in 1980 and 1981. More than 5,000 accidents, categorized by various types, were reviewed for almost 9,000 computer records. The control of variables other than shoulder width was facilitated by the statistical methods used to test the research hypotheses.

BACKGROUND

In 1978 OCRC initiated a highway risk management program (11) in order to identify elements of risk to Oakland County's traveling public and to the Road Commission and to treat those risks in a systematic, cost-effective manner. In order to effectively manage the risk, information concerning the county road network and traffic accidents was a necessity.

The OCRC maintains almost 1,500 miles of county primary and local roads and approximately 1,100 miles of subdivision streets. In 1980 and 1981 OCRC completed an inventory of roadway features for all major county roadways and compiled the information on computer tape. To supplement the roadway inventory, the Statistical Package for the Social Sciences (SPSS) (12,13) was obtained to allow the manipulation of data concerning roadway features, roadside obstacles, and accident characteristics. The development of this comprehensive roadway information system (CRIS) enhanced OCRC's research capabilities and allowed detailed analysis of the relation between shoulder width and accidents.

Approximately half of the 1,500 miles of major county highways are paved, and 637 miles of this total are two-lane uncurbed roadways. The data in Table 1 give the number of miles of two-lane paved roadways by shoulder width. The mean shoulder width on these roadways is 5.9 ft, with a standard deviation of 2.32.

Shoulders are defined in this study as the maintained portion of the roadway between the paved, traveled surface and the roadside ditch. In most cases the shoulder surface consisted of gravel, although minimal paved shoulders (i.e., 3 to 4 ft) existed at a negligible number of selected spot locations. Shoulder widths were derived from a review of photologs, wherein a grid overlay, which facilitated lateral measurement, was placed over the photolog viewing screen.

ACCIDENT TRENDS

The frequencies of various accident types by shoulder width are given in Table 2. These accident data were derived from 1980 accident files available from the Traffic Improvement Association of Oakland County. It would be advantageous to analyze accidents over a 3-yr period, but because of the recent completion of the CRIS, accident data for only 1 yr were available for the analysis. Nevertheless, the size of the accident and roadway samples compensated for this situation. Although all nonintersection accidents for 1980 were reviewed in this analysis, primary emphasis was placed on accident types that are commonly associated with shoulder width (i.e., fixed-object, overturn, head-on, and opposite-direction sideswipe accidents). Another accident type commonly associated with shoulder width--vehicles striking other vehicles stopped on the road--was not analyzed because of the low frequency of accidents of this type.

The distribution of accident frequencies per roadway record for each accident type and shoulder width is shown in Figure 1. Because of the small number of roadway records for extremely narrow shoulder widths and for shoulder widths ≥10 ft, new shoulder width categories were formed by combining records for shoulder widths <3 ft and ≥9 ft.

As indicated by the data in Figure 1, the relations between the frequencies of various accident types and shoulder widths appear to follow a general pattern. Accidents tend to increase with increasing shoulder widths up to 5 ft. For 5-ft shoulders, a decrease in accident frequency is indicated for most accident types. For shoulder widths >5 ft, accidents again increase. For shoulder widths ≥9 ft, there is a decrease in accidents.

These fluctuations may be attributed, in part, to the variation in the number of records for each shoulder-width category. For shoulder-width categories where the number of records is small, the accident frequency per record decreases. Nevertheless, other factors, such as traffic volume [i.e., average daily traffic (ADT)], may also influence these relations.

The relation between shoulder width and traffic volume is shown in Figure 2. As indicated by the data in Figure 2, the number of records for roadways with shoulder widths ≤3 ft, 5 ft, and ≥9 ft drop dramatically as traffic volume increases to greater than 5,000 vehicles/day. The relation between accident frequency and traffic volume has been accepted for many years (14), and this relation may account for the low accident frequencies for roadways with these shoulder widths.

A review of the data in Figure 3 reveals that a positive relation between accident frequency and

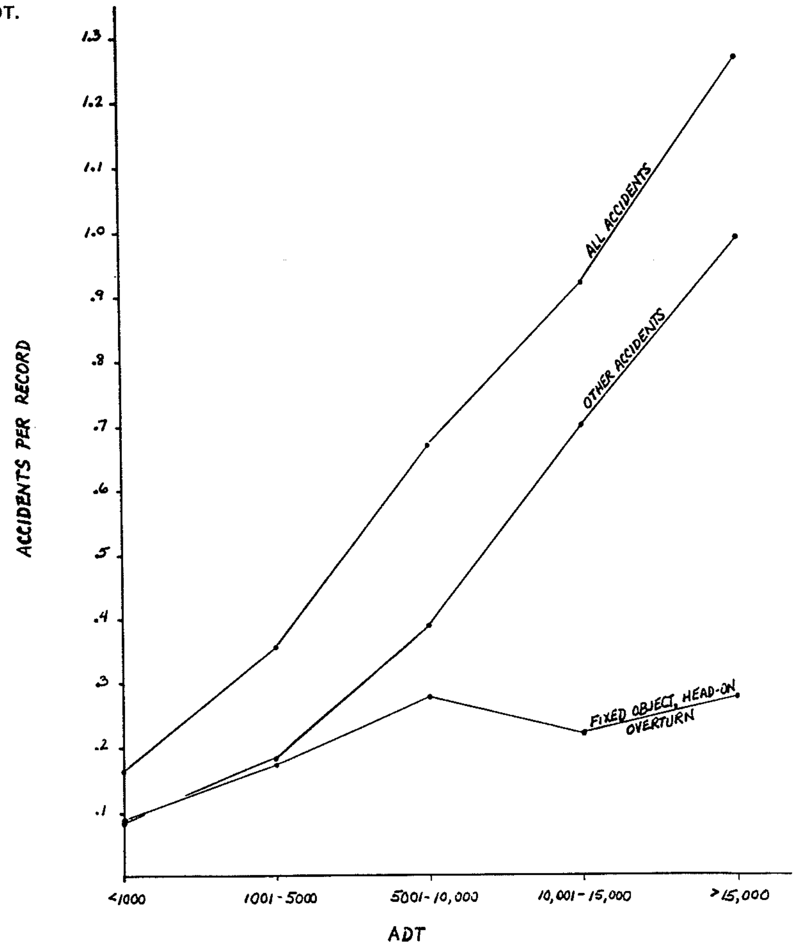
Table 1. Roadway records and mileage by shoulder width.

Shoulder Width (ft)	No. of Roadway Records	Approximate Roadway Miles
⩽2	617	44
3	588	41
4	1,222	87
5	537	38
6	2,068	147
7	1,886	134
8	1,511	107
9	515	36
⩾10	48	3
Total	8,992	637

Table 2. Accident types by shoulder width.

Shoulder Width (ft)	No. of Accidents: Fixed Object	Overturn	Head-On and Opposite Direction Sideswipe	Other	Total
⩽2	54	4	14	156	228
3	101	21	32	142	296
4	177	27	54	470	728
5	49	13	9	106	177
6	264	46	92	770	1,172
7	321	61	97	785	1,264
8	244	47	99	605	995
9	56	20	13	88	177
⩾10	10	2	4	18	34
Total	1,276	241	414	3,140	5,071

Figure 3. Accidents by ADT.



traffic volume exists. Therefore, the fluctuations in accident frequency for various shoulder widths can be partly attributed to the relation between shoulder width and traffic volume.

Generally, accidents increase as traffic volume increases, but when accident types are segregated, some types reveal a stronger relation to traffic volume than others. As shown by the data in Figure 3, fixed-object, head-on, and overturn accidents increase at a lesser rate than other accident types as traffic volume increases. A review of the data in Figure 4 indicates that the relation between these accident types and traffic volume is strongly influenced by the relation between head-on and fixed-object accidents and traffic volume, whereas overturn accidents remain relatively constant as traffic volume increases. These relations, and how other factors in combination with shoulder width influence accident frequency, are examined in greater detail later in this paper.

METHODOLOGY

The objectives of this study were to

1. Determine whether there is a significant difference in accident frequency between two-lane roadways with shoulder widths that meet the AASHO guidelines and those with shoulder widths that do not meet the guidelines, and
2. Determine whether there is a significant relation between certain accident types and shoulder width.

The first hypothesis to be tested states that roadways with shoulders ≥8 ft wide experience significantly lower frequencies of accidents (especially fixed object, overturn, and head-on) than roadways with narrower shoulders. The critical shoulder width of 8 ft was chosen based on AASHO guidelines (9) for urban arterial streets. Although AASHO recommends 10-ft shoulders, the guidelines state that 8-ft shoulders work "moderately well" when 10-ft shoulders are not feasible. Because the number of roadways with shoulder widths ≥10 ft is too small to allow valid comparisons, the study objective was altered to reflect this situation. Thus the research hypothesis can be stated as follows:

$$H_1 : \mu_1 < \mu_2 \quad (1)$$

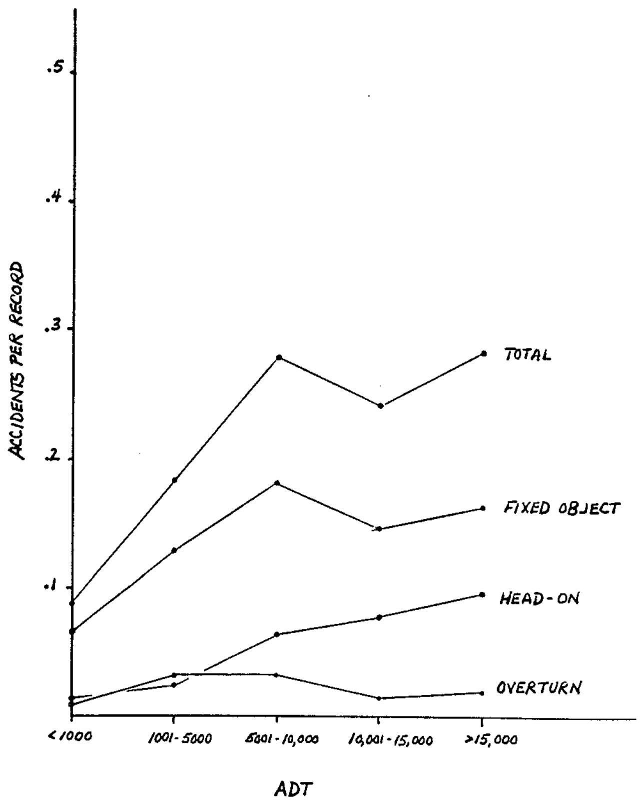
$$H_0 : \mu_1 \geqslant \mu_2 \quad (2)$$

where

H_1 = research hypothesis,
H_0 = null hypothesis,
μ_1 = mean frequency of accidents on roadways with shoulders ≥8 ft wide, and
μ_2 = mean frequency of accidents on roadways with shoulders <8 ft wide.

T-tests were performed to compare the record of mean accident frequencies per roadway on roadways with shoulders <8 ft wide and those with shoulders ≥8 ft wide. A significance level of 0.01 was chosen for the T-tests.

Figure 4. Fixed-object, overturn, and head-on accidents by ADT.



Analysis of variance (ANOVA) was used to determine whether accident types are associated with shoulder width. Accident types were segregated into four categories: all accidents, fixed-object accidents, overturn accidents, and head-on or opposite-direction sideswipe accidents. A significance level of 0.01 was chosen for the ANOVA.

The ANOVA allows the analyst to determine whether there is a significant difference in at least two category means for the frequency of each accident type categorized by shoulder width. The research hypothesis for the ANOVA can be stated as

$$H_1 : \mu_{1a} < \mu_{2a} < \mu_{na} \quad (3)$$

where

μ_{1a} = mean accident frequency of accident type a for the widest shoulder width,
μ_{2a} = mean accident frequency of accident type a for the next widest shoulder width, and
μ_{na} = mean accident frequency of accident type a for the narrowest shoulder width.

If a significant difference in at least two of the category means was found, further tests were conducted to determine which of the category means differ significantly.

RESULTS

The data in Table 3 give the mean number of accidents for roadways with shoulders <8 ft wide and for roadways with shoulders ≥8 ft wide. Accident types were broken down into six categories.

At a level of significance of 0.01, the critical T-score is approximately 2.326. The computed T-scores are given in Table 3 for comparison. As indicated by the data in Table 3, none of the accident types demonstrated a significant difference in mean accident frequency between the two shoulder-width categories. Therefore, the research hypothesis is rejected.

The results of the T-test demonstrated that roadways with shoulders ≥8 ft wide do not experience fewer accidents than roadways with shoulders <8 ft wide. A review of the data in Table 3 reveals that most of the computed T-scores are negative, which indicates that the mean accident frequencies for roadways with shoulders <8 ft wide are less than the mean accident frequencies for roadways with wider shoulders.

An ANOVA was conducted to determine which accident types are related to shoulder width. The data in Tables 4-7 give the results of the ANOVA for total accidents, fixed-object accidents, overturn accidents, and head-on accidents. Because traffic volume has been shown to be influential in accident frequency, it has been included as a covariate in the ANOVA. Another variable that has been shown to be influential in the frequency of certain accident types is roadway curvature (15-19). This variable has also been included as a covariate. A significance level of 0.01 was chosen for each test.

The data in Table 4 give the results of the ANOVA for all accidents. When traffic volume and roadway curvature are included in the model as covariates, no significant relation between accident frequency and shoulder width is noted. Most of the explained variation is due to the effect of traffic volume on accident frequency. This finding corresponds to the relation shown in Figure 3.

The data in Table 5 indicate that the frequency of fixed-object accidents is related to shoulder width, even when traffic volume and roadway curvature are included as covariates. The data in Table 6 review the ANOVA for overturn accidents. The results indicate that the frequency of overturn accidents is not related to shoulder width or traffic volume, but it is related to roadway curvature. The data in Table 7 indicate that head-on accident frequency is related to traffic volume and roadway curvature, but not to shoulder width.

It appears that only fixed-object accidents are related to shoulder width. Although this relation was significant at the 0.008 level, the amount of variance in fixed-object accident frequency explained by shoulder width after traffic volume and roadway curvature were considered was not great.

Because only fixed-object accidents were related to shoulder width, T-tests were performed to determine which of the category means differ significantly. The results of these T-tests are given in Table 8.

Traffic volume and sample size appear to influence the significant relation found between fixed-object accident frequency and shoulder widths <3 ft and 5 ft. After traffic volume and sample size were considered, the comparison of means for 6- and 7-ft shoulders demonstrated the only significant difference in fixed-object accident frequency. Further dissection of these results indicates that this finding only applies to roadways that carry 1,000 to 5,000 vehicles/day.

One further test was conducted to determine a range of shoulder widths that provide adequate safety in terms of fixed-object accident frequency when conditions prevent adherence to the guidelines. Shoulder-width categories were combined in order to analyze fixed-object accident frequency between pairs of shoulder-width ranges. T-tests were performed for each pair of ranges in a manner such that fixed-object accident frequency on roadways with shoulders <3 ft wide was compared with the acci-

Table 3. T-test results: shoulders <8 ft wide versus shoulders ≥8 ft wide.

		Accident Frequency per Record				
Shoulder Width (ft)	All Accidents	Fixed-Object, Overturn, and Head-On Accidents	Total of Other Accident Types	Fixed Object	Overturn	Head-On
<8	0.5587	0.2076	0.3511	0.1396	0.0249	0.0431
≥8	0.5815	0.2387	0.3428	0.1495	0.0333	0.0559
T scores	-0.70	-2.09	0.27	-0.85	-1.90	-2.25
Significance	NS	NS	NS	NS	NS	NS

Note: $\alpha = 0.01$, T = 2.326, and NS = not significant.

Table 4. ANOVA: all accidents by shoulder width, with traffic volume and roadway curvature.

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Squares	F Ratio	F Probability	Significance
Covariates	2	924.264	462.132	291.559	0.000	SIG
ADT	1	891.195	891.195	562.255	0.000	SIG
Curvature	1	50.920	50.920	32.125	0.000	SIG
Main effect—shoulder width	13	34.982	2.691	1.698	0.054	NS
Explained	15	959.250	63.950	40.346	0.000	SIG
Residual	8,976	14,227.285	1.585			
Total	8,991	15,186.535	1.689			

Note: $p < 0.01$, SIG = significant, and NS = not significant.

Table 5. ANOVA: fixed-object accidents by shoulder width, with traffic volume and roadway curvature.

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Squares	F Ratio	F Probability	Significance
Covariates	2	23.399	11.700	55.841	0.000	SIG
ADT	1	5.085	5.085	24.270	0.000	SIG
Curvature	1	19.184	19.184	91.563	0.000	SIG
Main effect—shoulder width	13	5.960	0.485	2.188	0.008	SIG
Explained	15	29.360	1.957	9.342	0.000	SIG
Residual	8,976	1,880.642	0.210			
Total	8,991	1,910.002	0.212			

Note: $p < 0.01$, and SIG = significant.

Table 6. ANOVA: overturn accidents by shoulder width, with traffic volume and roadway curvature.

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Squares	F Ratio	F Probability	Significance
Covariates	2	0.526	0.263	8.460	0.000	SIG
ADT	1	0.026	0.026	0.839	0.360	NS
Curvature	1	0.488	0.488	15.706	0.000	SIG
Main effect—shoulder width	13	0.583	0.045	1.442	0.131	NS
Explained	15	1.109	0.074	2.378	0.002	SIG
Residual	8,976	279.184	0.031			
Total	8,991	280.293	0.031			

Note: $p < 0.01$, SIG = significant, and NS = not significant.

Table 7. ANOVA: head-on accidents by shoulder width, with traffic volume and roadway curvature.

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Squares	F Ratio	F Probability	Significance
Covariates	2	6.374	3.187	62.088	0.000	SIG
ADT	1	5.993	5.993	116.754	0.000	SIG
Curvature	1	0.534	0.534	10.401	0.001	SIG
Main effect—shoulder width	13	1.310	0.101	1.964	0.020	NS
Explained	15	7.684	0.512	9.980	0.000	SIG
Residual	8,976	460.722	0.051			
Total	8,991	468.406	0.052			

Note: $p < 0.01$, SIG = significant, and NS = not significant.

Table 8. T-tests: fixed-object accidents.

Mean Accident Frequency	Shoulder Width (ft)	Signicance of T-Test by Shoulder Width (ft)							
		<3	3	4	5	6	7	8	≥9
0.087	<3								
0.0912	5								
0.1172	≥9								
0.1277	6								
0.1448	4	SIG			SIG				
0.1615	8	SIG			SIG				
0.1702	7	SIG			SIG	SIG			
0.1718	3	SIG			SIG				

Note: α = 0.01, T = 2.326, and SIG = significant difference.

Table 9. T-tests: fixed-object accidents for combined shoulder-width categories.

Shoulder-Width Range (ft)	Mean Frequency of Fixed-Object Accidents	Variance	No. of Cases	Significance
<3	0.0875	0.106	617	SIG
≥3	0.1478	0.220	8,375	
<4	0.1419	0.177	1,205	NS
≥4	0.1440	0.218	7,787	
<5	0.1434	0.176	2,427	NS
≥5	0.1438	0.226	6,565	
<6	0.1339	0.162	2,964	NS
≥6	0.1485	0.237	6,028	
<7	0.1314	0.163	5,032	SIG
≥7	0.1593	0.279	3,960	
<8	0.1396	0.209	6,918	NS
≥8	0.1495	0.223	2,074	
<9	0.1455	0.215	8,429	NS
≥9	0.1172	0.171	563	

Note: α = 0.01, T = 2.326, SIG = significant, and NS = not significant.

dent frequency on roadways with shoulders ≥3 ft wide; the accident frequency on roadways with shoulders <4 wide was compared with the accident frequency on roadways with shoulders ≥4 ft wide; and so on.

The results of these tests are given in Table 9. The tests demonstrate that roadways with shoulders <3 ft wide have significantly fewer fixed-object accidents than roadways with wider shoulders, and that roadways with shoulders <7 ft wide have significantly fewer fixed-object accidents than roadways with wider shoulders. Although traffic volume and sample size may have influenced the significant difference in fixed-object accident frequency between roadways with shoulders <3 ft wide and those with wider shoulders, the significant difference found between roadways with shoulders <7 ft wide and those with shoulders ≥7 ft wide can be attributed to shoulder width.

CONCLUSIONS AND OPERATIONAL CONSIDERATIONS

The impact of shoulder width on accident experience for two-lane paved roadways in Oakland County has been addressed. The liability exposure of the county road agency has not been discussed, although liability claims against the agency have cited shoulder width, which is at variance with suggested guidelines, as a contributing factor in specific accidents.

Analyses were performed to determine whether there is a significant difference in accident frequency between two-lane roadways with shoulder widths that meet the guidelines and those that do not meet the guidelines, and whether there is a relation between certain accident types and shoulder width. The results of this study do not support the premise that roadways with wider shoulders experience fewer accidents than roadways with narrow shoulders. Although it is advantageous to construct shoulders to the minimum width necessary for emergency parking, the results do not indicate a significant difference in accident frequency between two-lane roadways that meet shoulder-width guidelines and those that do not meet the guidelines. Accident data reveal that shoulder width is not related to overturn accidents, head-on type accidents, or to accident frequency in general. A relation was discovered between fixed-object accident frequency and shoulder width. Nevertheless, the findings indicate that fixed-object accident frequency is significantly lower for roadways with shoulders <7 ft wide than it is for roadways with wider shoulders. These findings are similar to those reported by Blensley and Head (3) and Perkins (4).

Drivers may perceive roadways with wider shoulders differently than they perceive roadways with narrow shoulders. Wider shoulders may give drivers a false sense of security, wherein they are likely to drive at speeds faster than conditions tolerate. Although analysis of this hypothesis is beyond the scope of this paper, factors other than shoulder width were found to influence the frequency of fixed-object accidents more than shoulder width.

A step-wise regression analysis indicated that horizontal curvature, traffic volume, pavement width, vehicle speed, and vertical curvature influence the frequency of fixed-object accidents to a greater extent than shoulder width. Even after all of these roadway-related variables are considered in a multiple regression, only 1.3 percent of the variance (represented by r^2) in fixed-object accident frequency was explained. Shoulder width, by itself, only explained 0.06 percent of the variance in fixed-object accident frequency.

Variables that were not analyzed in this study, but which may also influence fixed-object accident frequency, include the condition of the shoulder, roadway delineation, weather and lighting conditions, the lateral distance of obstacles from the roadway, and driver-related factors (e.g., intoxication, inattention, improper passing, recklessness). Further research is needed to evaluate these factors.

Because of the influence of other factors on fixed-object accident frequency, and because of the substantial costs involved in shoulder alteration, projects designed to reduce fixed-object accident frequency through shoulder alteration would not be cost effective. Therefore, it was concluded that

1. Projects to reduce accident frequency should focus on factors that exhibit greater influence on accident frequency than does shoulder width, and
2. Although it is desirable to adhere to current guidelines wherever possible when undertaking certain types of construction projects, it may be acceptable to retain existing shoulders <8 ft wide

unless a review of accident data for the project location indicates otherwise.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

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