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Glenwood Canyon Interstate 70: A Preliminary Design Process That Worked

RICHARD A. PROSENCE AND JOHN L. HALEY

After nearly 20 years of study and 10 years of controversy and debate about the construction of an Interstate highway through picturesque Glenwood Canyon, the Colorado Department of Highways has developed and implemented a unique design development process that has won widespread public support for the project. The way seems to now be clear for closing one of the last gaps in the Colorado Interstate system. The open planning and design process allowed maximum public participation not only in the identification and preservation of critical environmental and recreational factors but also in the selection of technical design concepts for the highway. The design development process demonstrated how a politically and publicly controversial project received endorsement from virtually all elements of the public. The process is unique in that it used sports enthusiasts, naturalists, artists, and many lay persons to evaluate and decide on highly technical design issues and asked scientists, engineers, architects, and construction contractors to identify and demonstrate environmental, recreational, social, and aesthetic impacts. Modern design tools, including computer graphics, electronic survey instruments, and photo montages, were mixed with artists' drawings, sketches, models, and field mock-ups to suggest final alignments and grades, locate structures, pinpoint roadside facilities, and resolve safety issues. Completion of the design development process has culminated in the holding of a design public hearing, issuance of a design report that has been accepted by the vast majority of interested agencies and individuals, and design approval by the Federal Highway Administration. Use of the methodology developed in this process will provide an alternative to delays, repetitious studies, public and political controversy, increased costs, and litigation.

The purpose of this paper is to describe a unique design development process that culminated in a favorable consensus on a controversial Interstate highway project in western Colorado. Design elements were a key issue in discussions about the Glenwood Canyon segment of Interstate 70 (I-70). The usual post-environmental impact statement (EIS) approval process would be to hold a formal design public hearing and proceed with design development. However, in the case of Glenwood Canyon, many people thought that a poorly designed freeway would impose unacceptable impacts on this scenic wonder.

So cardinal was this concern that the Federal-Aid Highway Act of 1976 allowed variations from Interstate standards for this segment of I-70. Without the implementation of an open process to provide much opportunity for participation by the public, the prospects of advancing this project to construction appeared bleak in 1976, when route location approval was granted.

GLENWOOD CANYON

Glenwood Canyon, located in west-central Colorado about 240 km (150 miles) west of Denver, is the result of a gradual uplift of the Rocky Mountains over the past 30-40 million years. The Colorado River has downcut through younger metamorphic rocks into the pre-Cambrian formation to expose steep and majestic canyon walls that rise up to 915 m (3000 ft) above the river. The nearly vertical rock faces are accented and complemented by steep talus slopes, covered with multicolored vegetation. The grandeur and unblemished beauty of the upper reaches of the canyon make this transportation corridor one of the nation's most spectacular traveling experiences.

The floor of the canyon reflects its natural and man-made history as a transportation corridor. On one side of the Colorado River, US-6, a two-lane, low-capacity, and unsafe highway that carries up to 11 000 vehicles/day runs parallel to the river. On the opposite shore is the single-track, 24 trains/day, Denver and Rio Grande Western Railroad that is scheduled for expansion to two tracks to accommodate the ever-increasing coal train traffic.

Shoshone Dam, located midway through the canyon, backs up the Colorado River into a 6.4-km (4-mile) long reflective pool that adds tranquility to the canyon's beauty but restricts the available space for further construction considerably. From Shoshone Dam a 3.2-km (2-mile) long tunnel carries part of the river through the north canyon wall to a hydroelectric generating plant owned by Public Service Company of Colorado. From the power plant, transmission lines follow the canyon both east and west and transport much needed energy out of the area.

Limited vegetation and sheerness of the canyon walls restrict its use by wildlife, although numerous mule, deer, and other animals and birds use the canyon for grazing, hunting, and nesting.

Recreation uses are heavy and increasing. Rafters, kayakers, fishermen, and picnickers use the river. Hikers, bicyclists, and campers compete for the limited space along with recreation vehicles, rock climbers, photographers, and other tourists.

The importance of Glenwood Canyon as an Interstate highway corridor is readily matched by its many important natural, social, recreational, environmental, and economic benefits. The unique characteristics of the canyon made it appropriate for the Colorado Department of Highways to develop and implement a special process for planning the design and construction of I-70 through this spectacular canyon.

PROJECT BACKGROUND

The project began more than 20 years ago when Congress authorized the extension of I-70 west from Denver to I-15 in Utah. Although Glenwood Canyon was incorporated along the initial corridor for the highway, the obvious engineering and environmental problems encountered along this narrow, twisting 21-km (13-mile) route caused the Colorado Department of Highways to undertake a detailed investigation of potential alternative routes.

Beginning during 1970, alternative locations, including north and south bypasses of the canyon, were studied in detail. The Colorado Department of Highways initiated and completed a comparative evaluation of the social, economic, and environmental impacts of highway construction along each of these alignments. After release of the draft EIS, a corridor public hearing was held. The final EIS, issued in 1972, recommended that Glenwood Canyon be the location of I-70 and that the other alignments be eliminated from further consideration.

Although the Glenwood Canyon corridor was firmly established as the most logical location for I-70, much concern remained as to whether a highway could be designed that would meet both Interstate standards and the mandates of the Colorado legislature, which had passed a resolution that stated, "The interests of the people of this state will be best served by a highway so designed that...the wonder of human engineering will be tastefully blended with the wonders of nature."

To help find the most acceptable solution, the Colorado Department of Highways retained three nationally recognized consulting firms for a design comparison. This was an unusual departure for highway design even though it is a well-established technique for important architectural projects. The resulting concepts developed in the comparison were as follows:

1. A double-decked structure with truck traffic restricted to the lower level,
2. An Autostrada-type highway partly located high up on the canyon walls to leave portions of the canyon floor free of highway traffic, and
3. A more conventional design confined to the platform along the canyon floor.

Although none of the design concepts was accepted in total, the comparison illustrated that careful design techniques could enable an Interstate highway to be constructed without severe permanent impacts on the visual and recreational elements of the canyon.

In 1976 the U.S. Department of Transportation (DOT) officially designated Glenwood Canyon as the I-70 corridor. In the same year Congress passed the Federal-Aid Highway Act with an amendment that stated as follows (PL 94-280, Section 152):

> The Secretary of Transportation is authorized upon application of the Governor of the state, to approve construction of that section or portions thereof of Interstate Route 70 from a point 3 miles [4.8 km] east of Dotsero, Colorado, westerly to No Name Interchange, approximately 2.3 miles [3.7 km] east of Glenwood Springs, Colorado, approximately 17.5 miles [28.2 km] in length, to provide for variations from the number of lanes and other requirements of said Section 109(b) in accordance with geometric and construction standards whether or not in conformance with said Section 109(b) which the Secretary determines are necessary for the protection of the environment, and for preservation of the scenic and historic values of the Glenwood Canyon. The Secretary shall not approve any project for construction under this section unless he shall first have determined that such variations will not result in creation of safety hazards and that there is no reasonable alternative to such project.

During the spring of 1976, the Colorado Department of Highways officially initiated an open planning and design process and specified in a published work program the techniques and procedures to be followed in carrying out the project. Four groups of participants were identified to be involved in conceptual studies with the department. The groups consisted of a citizens advisory committee (CAC), whose seven members were designated by the highway commission to work directly with the department of highways staff; two citizen workshop groups, which consisted of residents of the study area working separately and independently of each other; and a technical review group (TRG), which consisted of federal and state agencies, two private companies, and professional organizations that have a direct interest in I-70.

Project objectives adopted to be implemented in the design and construction programs were as follows:

1. The highway must be safe and provide a reasonable level of service;
2. The highway must not destroy or preclude important recreation potentials related to the canyon;
3. The visual quality of the highway should not detract from the landscape and should be of high design quality;
4. The highway should afford a pleasant driving experience;
5. The construction procedures must be carefully planned and continuously monitored to avoid undue environmental damage; and
6. Environmental protection measures, revegetation programs, and landscaping plans must be fully implemented.

From the start, an extraordinary effort was made to get people involved. Bus tours were conducted through the canyon by Colorado Department of Highways representatives, who described conflicts and challenges. An information office was kept open at night with a telephone line so that people could call in and have questions answered. Canyon Echos, a newsletter, was printed and distributed periodically. A community attitude survey was completed by a professional survey firm. Workshop participants were assisted by professionals and furnished with numerous graphic materials during their deliberations.

During December 1976, the CAC made its first report to the state highway commission. The report recommended that a narrow, four-lane highway be constructed in the canyon. It also recommended that, for comparison purposes only, a two-lane design be developed for critical areas of the canyon.

In February 1977, the preliminary design process was begun. Later that year the CAC filed its final report with the state highway commission. They had reached a consensus that a four-lane facility was generally acceptable.

Following the holding of a design public hearing in March 1978, work began on the formal design report, which was transmitted to the Federal Highway Administration (FHWA) during March 1979. On September 17, 1979, FHWA informed the Colorado Department of Highways that the design was approved and final design was authorized.

Conceptual Design Conclusions

The special consideration given Glenwood Canyon by Congress found in the Federal-Aid Highway Act of 1976 coupled with the mandate from the Colorado legislature and the extensive planning process carried out by the Colorado Department of Highways established two basic issues as being of equal importance and complexity:

1. Safety and capacity and
2. Recreational, environmental, and visual impacts.

In setting the preliminary design objectives, the Colorado Highway Commission wisely and prudently accepted the theme and thrust set by CAC and TRG during the conceptual planning process.

Preliminary Design Parameters

Based on the conclusions of the conceptual planning process, a number of formal and informal design objectives were accepted and implemented for the preliminary design phase of the project. Certain of these were very precise and technically defined; others were deliberately left flexible and general in definition to allow maximum public participation, uninhibited discussion, innovative design approaches, and thorough understanding and acceptance by all participants.

The number, definition, and priority of the design objectives were never precisely documented or spelled out because it was understood and accepted that each participant brought to the process a specific set of goals.

Among the numerous individual design objectives were the following generally accepted goals that guided the daily activities and specific alignment, grade, and structural decisions:

1. Interstate standards for 80-km/h (50-mph) design speed;
2. Four-lane divided, separated, or barrier-protected cross section;
3. No relaxation of minimum safety standards;
4. Minimal intrusion of the highway on the existing features and vistas of the canyon;
5. Maximum adaptation of the design to the natural characteristics of the rocks, vegetation, waterway, wildlife, recreation, and appearance of the canyon;
6. Design to repair, expand, enhance, and improve the benefits to travelers, recreation users, property owners, and the existing facilities whenever practical;
7. Design process to encourage maximum participation and understanding by all agencies, businesses, special interest groups, and individuals;

8. Design process to provide maximum exposure and disclosure of all pertinent facts and considerations during the procedures;

9. Presentation at the design public hearing of all identified benefits and adverse impacts and an easily understood display of precise alignment, grade, structures, schedules, costs, and economic factors; and

10. Publication of a design report that would fully describe and document the design process, alternative designs, the selected design itself, the effects of the project, the public hearing comments and responses, and all other information required to obtain the approval of the Colorado Highway Commission, FHWA, and the Secretary of Transportation.

DESIGN DEVELOPMENT PROCESS

Objectives

The basic objective of the design development process was to refine conceptual designs to a point of having sufficient detail to allow the holding of the design public hearing. The intent was not to have all design decisions finalized at the time of the hearing. However, it was intended that all recommended solutions would be sufficiently developed to present a complete and factual statement to the public of major details, costs, and impacts, and to be prepared to answer all pertinent questions. A secondary objective was to enable the orderly and logical scheduling and accomplishment of all subsequent final design and construction activities.

This was to be accomplished in accord with the conclusions of the conceptual planning phase in an open, well-publicized process that invited public participation and kept all interested parties fully informed at all times.

Structure

The chart in Figure 1 shows the management format and relationships between the various elements of the design team, CAC, TRG, the consultants' staffs, and the design review panel (DRP).

Each step in the design process was followed by a presentation for review and concurrence to the CAC, TRG, and the Colorado Department of Highways prior to completion and development of the next step. Only after a thorough screening and review by the entire design team and DRP was an alternative allowed to progress to the next step.

The Process

The basic process was for the design consultants to develop options and alternatives in sufficient detail to enable them to discuss each with members of the design team (and frequently representatives of CAC and TRG). Critical comments and suggested modifications would be made by the design team and DRP along with requests for additional information or verification of data. Frequently, new concepts or approaches would be developed in the field and the process would begin again for further refinement or the start of a new cycle.

Each consultant project manager was responsible for the development, presentation, and defense of the alternatives developed by staff in his or her particular area and, as a member of the DRP, was also responsible for review, evaluation, and constructive criticism of other areas. This dual role as designer-critic demanded the utmost in cooperation and coordination.

Each design consultant was responsible for the development and presentation of support materials to demonstrate the visual, environmental, and technical benefits of his or her design alternative. In addition to the usual plan-profile sheets, typical cross sections and computer printouts, artists sketches, photo montages, scale models, and field staking were used to demonstrate a particular alternative. When even these proved inadequate, the designers constructed full-scale models and cloth curtains in the canyon cut to actual structural dimensions, along the canyon walls and across the river to simulate proposed alignment, elevations, and grades. Designated tunnel portals were outlined on the rock faces, limits of construction were flagged, and individual trees scheduled for removal were marked in the field.

These visual aids were used by both the designers and the reviewers to develop and test alternatives, identify conflicts, pinpoint environmental losses, illustrate the visual impacts, and establish the future driving and viewing experience for both the motorists and the pedestrian users of the canyon. These were not marketing displays designed to convince the public of the values of a particular solution; these were design tools used by the designers to forecast and prove (or disprove) specific design alternatives.

By January 1978, the design team, TRG, and CAC had generally agreed on an optimum design to be taken into the final design process for much of the canyon. It was a hybrid solution that used a variety of concepts carefully fitted to the diverse character of the canyon. As a result of the various design, display, and informational techniques, and the very active participation of all the CAC and TRG members, a consensus was reached and a design public hearing was scheduled for March 1978. Alternative designs would be presented to the public for five specific locations as well as the recommended design for the remainder of the canyon.

Public Hearing

A prehearing display at a local hotel was opened to the public 10 days prior to the public hearing. The display was well attended and attracted 840 registered visitors. A team member was on hand to explain the exhibits and answer any questions. The positive response at the public hearing was attributed, in part, to the prehearing display and the candor of the responses to questions asked. The prehearing display educated the public concerning the project. All inquiries concerning the design process, reasons for the selection of various alternatives, and the factual results of the recommended solution, were responded to in detail.

Special exhibits were prepared for the public hearing held at Glenwood High School on March 21, 1978. These exhibits included five scale models and thirty-one 4 x 8-ft panels that clearly illustrated the recommended solutions and the unresolved alternatives. Special care was taken to explain the design process, design guidelines, selected alignments, recreation and landscape provisions, estimated costs, and the anticipated schedule of events to follow.

CAC publicly recommended endorsement of the designs and the process and documented their consensus where alternatives were still under consideration. In their report the CAC applauded the design process and stated (1):

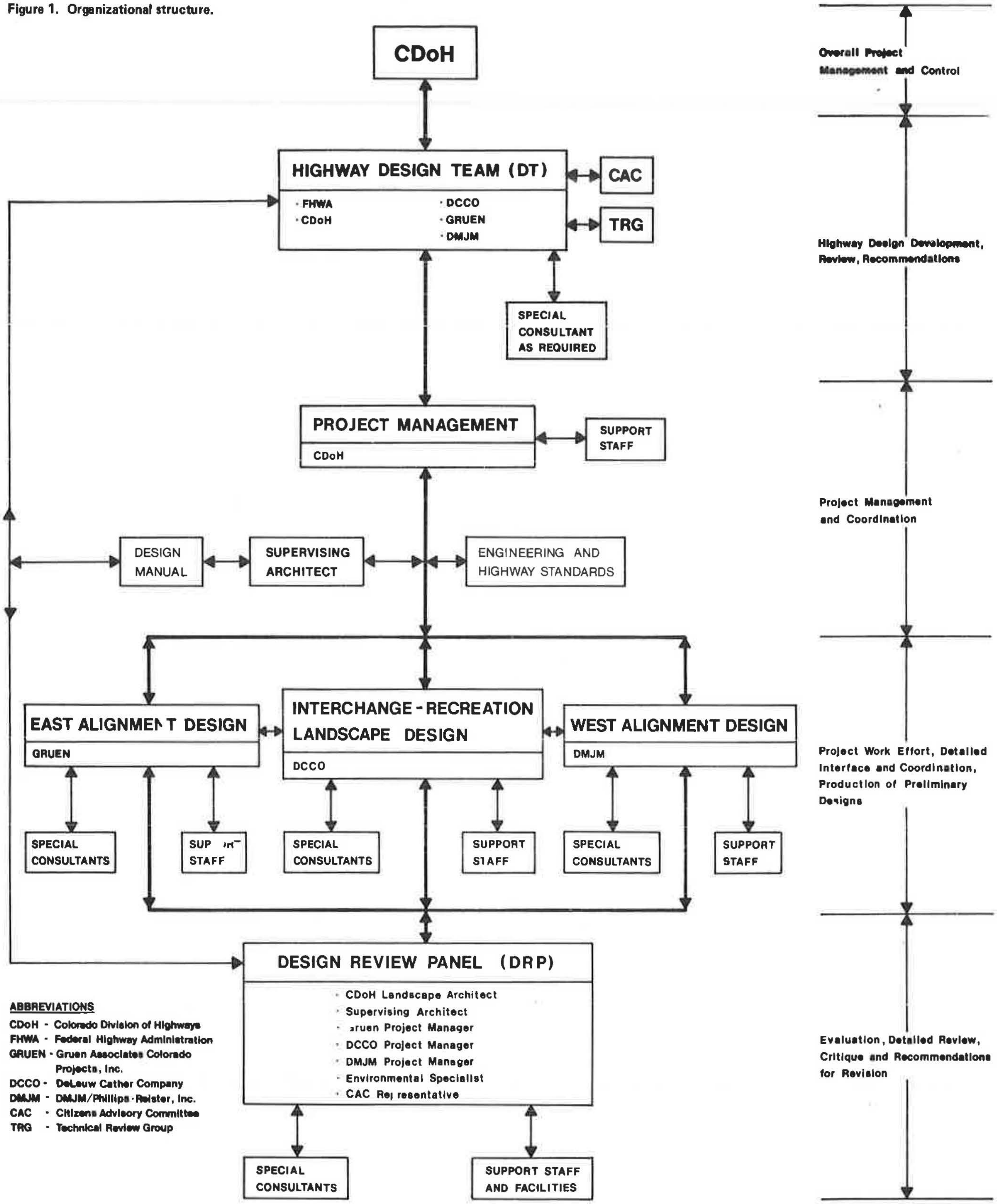
> The decision that was made to involve, educate, and bring along a lay oversight committee like the CAC was a daring and courageous one. It admits the shortcomings of traditional hearing and impact statement processes which cater to confrontation instead of conciliation and compromise. It places a great deal of trust in a social process that must be in conflict with the traditionally developed technical skills within the highway fraternity. And it exposed all the decision-making steps to a public press that is frequently more interested in highlighting the conflicts than in the resolution of tough issues.

More than 300 persons attended the public hearing. Of these, 35 gave direct testimony and an additional 81 provided letters either in support of or objection to various recommended solutions. At the public hearing there were many statements made to compliment, commend, and support the preliminary design process and its extensive public involvement. No speaker voiced opposition to or criticism of the process. This is particularly significant in light of the highly controversial and emotional history of the project. Before, the Colorado Department of Highways had been criticized for its lack of environmental sensitivity and disregard for public concerns; at the public hearing their methods received much praise.

Participants

Four groups of participants were involved with the Colorado Department of Highways in both the conceptual planning phase and the preliminary design process.

Figure 1. Organizational structure.



CAC

A CAC, which consisted of seven members nominated by various governmental and public agencies, was appointed by the state highway commission. The CAC was drawn in the following manner: one representative each from the four adjacent counties (as nominated by their respective county commissioners), one member from Club 20 (a west-slope organization of 20 counties), one member from the Colorado Open Space Council, and one member from the American Institute of Architects. The CAC was directed to work directly with the staff of the highway department and its consultants and report directly to the state highway commission their conclusions, recommendations, and concerns regarding alternative social, environmental, and economic impacts of various proposals, to suggest changes, and to critically review all suggestions and design alternatives proposed by others.

Citizen Workshops

To assist the CAC and to broaden the base of citizen input, 37 local citizens were appointed to two workshops that met separately and independently each week to study issues and produce reports to the CAC that expressed their concerns and recommendations. The Wednesday night workshop and the Thursday night workshop committees held many meetings, involved hundreds of hours of volunteer effort, and produced numerous reports, suggestions, and constructive criticisms. Their membership consisted of business persons, homemakers, sports enthusiasts, ranchers, males and females, young and elderly, and teachers and students. The CAC was structured to be truly representative of all the people concerned with the development of I-70 in Glenwood Canyon.

TRG

A TRG, which consisted of representatives from the following organizations was created to study, evaluate, make technical decisions, and advise on technical impacts of all design alternatives: U.S. Forest Service; U.S. Bureau of Land Management; U.S. Bureau of Outdoor Recreation; Colorado Division of Wildlife; Public Service Company of Colorado; Denver and Rio Grande Western Railroad; Architects, Planners, Engineers, Surveyors (a local professional society); U.S. Environmental Protection Agency; Colorado Contractors Association; U.S. Geological Survey; and Colorado Department of Health. Each of these agencies had a direct interest in the development of I-70 and the future of Glenwood Canyon. Their representatives, either as members of the committee or as individuals, spent varying amounts of time reviewing proposals and advising on their area of expertise or special interest.

Consultants

During the preliminary design process, additional participants were involved either as consultants or as individual participants on special panels or teams. Assisted by the CAC, the Colorado Department of Highways interviewed several nationally recognized design consultants to survey, inventory, design, and develop detailed alternative solutions for the entire project. The firms of DMJM/Phillips, Reister, Inc., of Denver, and Gruen Associates of Los Angeles, were chosen as design consultants to work with DeLeuw Cather and Company, the supervising architect, and all the other participants during the preliminary design process.

Highway Design Team

From the above groups and the staffs of the Colorado Department of Highways and FHWA, Colorado division, a highway design team was created. The team included designers and engineers from each of the major consultants and was supported by the staff and resources of each organization. In addition, the design team selected and employed special consultants, as needed, to assist the design process in general.

This design team worked together, but each designer was primarily responsible for an assigned segment of the project. The design team developed technical answers and detailed plans for each alternative.

DRP

To review and critique the work of the design team, a DRP was created. It consisted of a representative of the CAC, one member of the TRG, the project manager from each of the major consultants, and representatives from the Colorado Department of Highways and FHWA.

CHRONOLOGY AND EFFORT

In January 1977, the state highway commission approved proceeding with the preliminary design of a four-lane roadway through the canyon. CAC and TRG were asked to continue their involvement in the process and funds were allocated to support the necessary consulting services.

In February 1977, the Colorado Department of Highways published a detailed work program for the preliminary design phase. This specified basic design standards, organizational responsibility and relationships, an outline of key tasks to be performed by consulting designers, and processes to be followed in the design, screening, review, and acceptance of solutions.

In March 1977, the consulting roadway designers were brought on board to work with the highway department, FHWA, CAC, TRG, and the supervising architect to carry out the work program. In May 1977, approved planning and design guidelines and the Architectural and Planning Design Handbook were distributed to the design team, CAC, and TRG, and the actual field work commenced.

From May 1977 to February 1978, the design team, DRP, CAC, and TRG held regularly scheduled weekly, biweekly, and monthly meetings. During this time approximately 20 formal meetings were held that involved CAC and TRG. Several walking field trips to the canyon were made to view site-specific problem areas, review design alternatives, and inspect full-scale models, mock-ups, or field conditions pertinent to a design.

Several special meetings were held to brief the state highway commission, Colorado's governor, the press, and the public on the progress of the designs. All meetings were well advertised in advance and specific invitations were made to the press and individuals or groups that held a special interest in a particular design or problem area.

Each month 1000 copies of Canyon Echos, a special news bulletin, were distributed to a mailing list of 400, and the remainder were distributed free through stores and shops in the area. Each issue of Canyon Echos gave an up-to-date progress report and showed examples of studies and techniques being used to find optimum solutions.

In February 1978, the design team met to determine which alternatives were to be recommended to the public in the design public hearing. Decisions were made on which alternatives were to be presented and how the information should be displayed and explained.

In March 1978, the prehearing display was set up, and staff was on hand for 10 days prior to the actual public hearing. Special care was taken to advertise the project, including a special issue of Canyon Echos, which was included as a supplement in a local paper. The design public hearing was held on March 20, 1978.

In April 1978, the design team and DRP met to review the record of the public hearing and the comments received and to decide on proposed design solutions to be presented in the design report.

Several meetings were held in May, June, and July 1978 between the Colorado Department of Highways, FHWA, CAC, TRG, the design team, and the state highway commission to review the proposed design prior to completion of the preliminary design drawings and publication of the design report.

In July 1978, the preliminary design drawings were completed; the draft design report was prepared and circulated for review by all interested parties. In August 1978 the draft design report was officially submitted to the state highway commission for review and transmittal to other agencies.

In October 1978, the draft design report was officially circulated for review, as required by the Office of Management and Budget Circular A-95, to ensure that federal funds were not being spent for conflicting projects. Although the Colorado Department of Highways action plan does not require a period for public circulation and comment for a design report, it was considered appropriate in this case due to the magnitude and sensitivity of the project.

The draft design report was made available for a 45-day public-comment period beginning October 6, 1978. Answers to review comments received were published and made a part of the design report.

In March 1979, the design report, including the two volumes of the draft design report published in October 1978, was published and forwarded by the state highway commission to FHWA and the Secretary of Transportation for review and approval.

COSTS

Although the preliminary design process is properly an integral part of the total design and should not be evaluated as an independent cost item, it is important to know the actual cost of the process. Route studies, EIS preparation, and the preliminary design process cost the Colorado Department of Highways approximately \$3.5 million including all consultant fees, printing costs, administrative costs, and miscellaneous expenses. This represents approximately 1.65 percent of the estimated \$212 million construction cost (1977 dollars). When this is compared with the highway department's current estimate of 11 percent/year rate of inflation, it becomes apparent that the cost of the preliminary design process is minimal. The preliminary design process would be paid for if it merely reduced the completion time by only eight weeks. It is expected to expedite the work by several years.

RESULTS

Unfortunately, the preliminary design process does not lend itself to measurement of precise results. If the process completely eliminated all opposition to construction, then maybe it could be said that the process was 100 percent successful. However, that is not the case. After all comments were in and analyzed, a spokesman for the Colorado Department of Highways stated that the "project was supported by a better than three-to-one margin in responses received." This could be considered as a more than adequate statistical justification for calling the process successful. However, the project's opponents use these same statistics to point out that 25 percent of the people who responded are still not satisfied with at least some portion of the design.

Perhaps the best way to measure the results is to look at the CAC and evaluate its posture before, during, and after the design process. This seven-person committee was specifically appointed because of their proven concern about the many social, economic, environmental, recreational, and safety issues involved in the project. They each had voiced strong reservations about whether or not an Interstate highway could, or should, be constructed in the canyon. They were appointed as a lay oversight committee and, in the beginning, functioned and were viewed as adversaries by the technical designers. As the process developed, however, these attitudes and relationships began to change. The lay citizen and the engineer technician began to work together harmoniously. As one of the CAC members stated in a published endorsement of the process (2):

> This process allowed the establishment of confidence in each of the issues being considered and the relevance and completeness of the data base which was being provided. In addition, to be successful it would be necessary to establish a "safe place"--a place where all assumptions and past criteria could be questioned and reevaluated for their applicability in this new condition. It would have to encourage technicians to express their technical opinions but would allow them to presume an equal relevance for their personal, social, cultural, and aesthetic opinions as well. And it required the same honesty of expression of social, cultural, and aesthetic experts, as well.

This same CAC author, in comparing the preliminary design process used on Glenwood Canyon with the normal EIS process, stated (2):

> Therefore, the CAC's recommendations did not land like a bombshell in the division of highways' lap. The EIS fireworks of confrontation was missing. In place of that dubious excitement was the less dramatic but far more effective semiweekly effort of a group of laymen. There were confrontations, compromises, and losses scattered throughout the CAC's two years of deliberations. Yet the final solution as recommended offers gains as well as losses for each interest.... The CAC has agonized over each special recommendation and in some respects is still not fully satisfied with the results. But what we hoped for ever since citizen groups and environmentalists began voicing their concern has finally occurred in this Glenwood Canyon experiment: Reasonable people differed and clashed, but reason prevailed.

The CAC voted six to one to support the recommended solutions, and that has to be called a positive result.

SUMMARY AND CONCLUSIONS

After years of controversy, discussion, and often emotional debate, a design solution has been developed that meets most of the expressed concerns of the environmentalist and the road builder, the recreationalist and the trucker, the local community, and the national interest. The solution has been reached as a result of a new and unique process that amalgamated the former adversaries with the physical problems, provided a minimum of arbitrary constraints, and challenged them to collectively find solutions.

Early efforts to develop a design that complies with normal Interstate standards and the traditional EIS process had not worked on less-demanding projects and offered no promise of success in Glenwood Canyon. Although the EIS was designed to address conflicting needs, it has not worked successfully where multiple value systems have been involved. The adversary approach it uses appeared counterproductive in an area where the issues cannot be directly measured one against the other.

In Glenwood Canyon, the aesthetic and environmental values could not be measured on the same scale as the cost and safety values or even the values of recreational pleasures and convenience. It appears to be the nature of the EIS process that it can only protect against the worst alternatives and cannot encourage the best solutions. In recognition of this, the state highway commission, with the encouragement of the governor, adopted a more flexible and creative process for arriving at solutions that were reasonably well balanced among the many interests involved.

The preliminary design process described in this paper has worked, at least to the point that it received substantial endorsement from the members of the public who took the time to record their views. Many concerns remain about costs, scheduling, funding, and construction priorities;

however, the major conflicts seem to have been resolved. The design meets nearly all the minimum standards for an Interstate highway; there are few deviations from safety standards, and the consensus is that the highway can be built without major permanent environmental, visual, or recreational impacts.

Since FHWA and DOT have indicated agreement with the conclusions of the design team, the CAC, and the state highway commission, by issuing design approval, the project has proceeded into final design, with construction anticipated at a pace that will complete one of the last gaps in the 68 400-km (42 500-mile) Interstate system by 1986. In addition, the Glenwood Canyon 20-km (13-mile) segment should, when completed, be a testimonial to the inspired blending of engineering with nature.

REFERENCES

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Methodology for Evaluating Geometric Design Consistency

CARROLL J. MESSER

This paper presents a methodology for evaluating and improving the geometric design consistency of rural nonfreeway highways. The methodology is based on driver behavior principles, a sound conceptual approach, and empirical evidence collected during a recent Federal Highway Administration-sponsored research project. Factors that contribute to potential geometric inconsistencies include basic feature type, design attributes, sight distance, separation distance, operating speed, and driver familiarity. The methodology may be applied to proposed or existing two-lane and four-lane highways in flat or rolling terrain. Design speeds may range from 80 km/h (50 mph) to 129 km/h (80 mph). The basic objectives of geometric design and application procedures are presented to aid the engineer in the design or evaluation of a design for geometric consistency.

The basic goal of the highway design engineer has always been to design a facility that will satisfy expected transportation needs safely, efficiently, and in a cost-effective manner. To satisfy public demand for better facilities, design engineers developed a vast highway system that reflects the needs, technology, and resources of the times. Design standards progressively changed to accommodate increasingly greater traffic volumes, increased speeds, larger trucks, and higher safety standards.

From 1920 to 1970, when most of the rural highway system was built, the evolutionary development process has had a major effect on rural driving experience and resulting driving behavior. During the earlier portion of this period, drivers had little experience with any long-distance, high-speed driving on rural highways. A high percentage of all roads were low-design and poorly coordinated. Drivers expected bad roads. World War II and the following 15 years also continued this variation in driving experience as highway conditions and vehicle performances continued to change rapidly. The driving experience of the 1960s stabilized to a great degree and motorists undoubtedly began to expect good roads everywhere. Perhaps the Interstate highway system created this illusion.

THE PROBLEM

Highway design engineers should recognize that existing high-design rural highways have produced a built-in set of high-design standards that cannot be safely ignored. Major changes in design speed, cross section, or alignment standards between adjacent sections along a rural highway may not be expected by today's motorist. Abrupt geometric changes may be so inconsistent with the driver's basic expectations that delayed driving responses and incorrect decisions may occur and result in unsafe driving (1).

The following sections present a methodology for evaluating the highway geometric design consistency of existing or proposed rural, nonfreeway highway facilities. Design concepts and procedures cover a wide range of design situations. Sound engineering evaluation and judgment still will be required to apply the methodology routinely to specific real-world design problems.

CONCEPTUAL MODEL

Certain driving tasks must be performed by a motorist in order to safely and comfortably follow a preselected route to the destination. The driver must control the vehicle in a manner that tracks a safe path along the highway at a safe speed for the conditions at hand (1). The driver continually updates vehicle control actions as new information is obtained from the driving environment. This information is handled in a decision-making process, and these decisions are translated into control actions (i.e., appropriate speed and path). The roadway itself serves as the primary source for information inputs to the driver and correspondingly imposes work-load requirements on the driver.

Driver Work Load

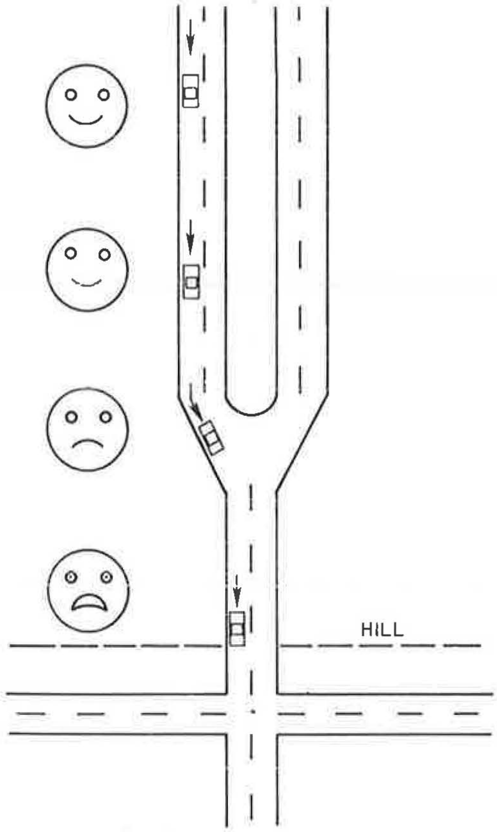
Driver work load is the time rate at which drivers must perform a given amount of work or driving tasks. Driver work load increases with increasing geometric complexity of those highway features perceived as potentially hazardous situations in the driving environment. Driver work load also increases with speed and reductions in sight distance for a given level of work to be performed over a section of highway. In addition, driver work load may increase dramatically for those motorists who are surprised by the unexpected occurrence or complexity of a set of geometric features. These motorists will require more time and mental effort to decide on an appropriate speed and path.

Driver Expectancy

Driver expectancy relates to the readiness of the driver to perform routine driving tasks in a particular manner in response to perceived situations and circumstances in the driving environment. Driver expectancy is primarily a function of the driver's memory and driving experience. The past experience that is relevant to the present task of driving a given section of highway is (a) the driver's immediate memory of the prior roadway and (b) long-term driving experience with similar facilities.

Driver performance is directly affected by driver

Figure 1. Example of compound geometric inconsistency.



expectancy. Driver performance tends to be error-free when an expectancy set is met. When an expectancy is violated, longer response times and incorrect driver behavior usually result (1).

Geometric Inconsistency

A geometric inconsistency in rural highway design is defined as a geometric feature or combination of adjacent features that have such unexpectedly high driver work load that motorists may be surprised and possibly drive in an unsafe manner. The unfamiliar motorist is more likely to be surprised by geometric feature inconsistencies.

The concept of a geometric inconsistency is illustrated through the use of an extreme example in Figure 1. An unfamiliar driver is traveling along an apparently well-designed four-lane divided rural highway. Suddenly and unexpectedly, the median ends and the road narrows to two lanes near the crest of a hill. The driver was performing at a low work-load level but, suddenly, the demand created from oncoming traffic, traffic to the left, and maneuvering to a new lane at a lower speed is much higher. The driver begins to respond to this new information and task loading since it demands a greater amount of work. As the driver crests the hill, he or she encounters an unexpected intersection, which may overtax the motorist's capabilities to deal with the situation.

PROCEDURES FOR ENSURING DESIGN CONSISTENCY

The goal of this federally sponsored research was to develop procedures for ensuring geometric design consistency by the development of a generalized methodology applicable to all rural nonfreeway facilities. Due to the necessary reliance on subjective ratings, expert opinion, and limited empirical evidence, the procedures presented should be interpreted as being a methodology for evaluating the geometric consistency of rural highways. The methodology approaches the evaluation from the viewpoint of designing the geometric features, such as lane drops, intersections, or curves so that unfamiliar motorists should be able to perform successfully the resulting driving tasks based on driver expectancy considerations described earlier. The influence of traffic control devices is not specifically considered.

Criticality Factors

The probability that a particular geometric feature may be inconsistent in a particular situation depends on numerous factors. The more influential factors that relate to the feature itself include the following:

1. Type,
2. Relative frequency of occurrence,
3. Basic operational complexity and criticality in the driving task, and
4. Overall accident experience in general.

Other important design variables that will affect the apparent criticality of a feature include (a) time available, (b) sight distance to the feature, (c) separation distance between features, (d) operating speed, and (e) prior roadway design features. Driver familiarity, traffic, topography, and roadside environment effects, among other factors, also will influence the resulting criticality of the individual geometric feature.

Criticality and Work-Load Ratings

Average criticality ratings were developed for nine basic geometric features by using a seven-point rating scale developed for identification of hazardous locations based on driver expectancy considerations (2). In this seven-point rating scale, 0 is no problem and 6 is a critical problem situation. A group of 21 highway design engineers and research engineers who have expertise in highway design, traffic engineering, and human factors rated each of the features. Each feature was assumed to be located along a high-quality rural highway. Average operating speed was 93 km/h (58 mph) and the sight distance to the feature was 244 m (800 ft). Some unfamiliar motorists drove the route. The engineers were asked to rate the nine basic geometric features, projected one at a time in schematic on a screen, according to their judgment of the average criticality of the feature (2). Engineers from Arkansas, Georgia, Illinois, Oklahoma, and Texas, in addition to the Texas Transportation Institute, were represented in the rating session.

The results of the rating session are presented in Table 1. The nine basic features are rank ordered from worst to best case. Ratings of different designs within a given feature category also were made as shown. Ratings for mediocre two-lane roads and undivided four-lane highways were determined from the basic ratings and other study results (3). In all studies made, divided highway transitions, lane drops, and intersections scored relatively high (more critical). Shoulder-width reductions, alignment changes, and lane-width reductions rated lower (less critical).

These criticality ratings were then used as anchor points on the criticality scale for each feature from which further study was conducted to estimate the range of probable criticality ratings for various specific cases that might exist. These calculated expectancy criticality scores were defined as work-load ratings and are used to evaluate the geometric design consistency of rural highways.

General Design Objectives

The following set of recommended general geometric design objectives, if thoroughly understood and practiced by design engineers, would eliminate many of the geometric feature inconsistencies that otherwise might appear in routine design.

Table 1. Summary of geometric feature ratings for average conditions on various classes of rural nonfreeway highway conditions.

Geometric Feature	Two-Lane High	Two-Lane Mediocre	Four-Lane Divided	Four-Lane Undivided
Bridge				
Narrow width, no shoulder	5.4	5.4	5.4	5.4
Full width, no shoulder	2.5	2.5	2.5	2.5
Full width, with shoulders[a]	1.0	1.0	1.0	1.0
Divided highway transition				
4-lane to 2-lane			4.0	
4-lane to 4-lane			1.8	
Lane drop (4-2 lanes)				3.9
Intersection				
Unchannelized	3.7	2.8	2.4	2.1
Channelized	3.3	2.5	2.1	2.4
Railroad grade crossing	3.7	3.7	3.7	3.7
Shoulder-width change				
Full drop	3.2	2.4	2.1	2.1
Reduction	1.6	1.2	1.0	1.0
Alignment				
Reverse horizontal curve	3.1	2.3	2.0	2.0
Horizontal curve	2.3	1.7	1.5	1.5
Crest vertical curve	1.9	1.4	1.2	1.2
Lane-width reduction	3.1	2.3	2.0	2.0
Crossroad overpass	1.3	1.0	0.8	0.8
Level tangent section[a]	0.0	0.0	0.0	0.0

Notes: Ratings of two-lane mediocre road (i.e., surface treatment pavement without paved shoulders) and all four-lane highways are usually assumed to equal 0.75 and 0.65 of two-lane high-design highway ratings.
Value system: 0 = no problem, 6 = big problem.

[a]Assumed.

Design to Give the Driver What Is Expected

The approaching road conditions, including the geometric design and resulting traffic operations, should be designed to be expected by the driver. Drivers tend to build up an expectation of what the upcoming roadway will be like based on their previous driving experiences. Some geometric features basically are unexpected at any location because of their limited frequency of use in rural highway design. Other features, such as horizontal curves and intersections, may have rare or unusual design attributes or operational demands that are unexpected. A 95 percentile horizontal curvature level (e.g., a 6° curve) is one example of a common feature (a horizontal curve) with an uncommon attribute.

Avoid Creating Compound Features

A compound geometric feature is one that contains two or more of the basic geometric features listed in Table 1 at the same location or in close proximity to one another. Close proximity is defined as a separation distance between the centers of the adjacent features of 457 m (1500 ft) or less. This distance may be reduced to 305 m (1000 ft) where 85 percentile speeds are less than 80 km/h (50 mph) or the compound feature is composed of two of the lower-valued basic features (less critical, i.e., less than 2.0) listed in Table 1. Tangent sections are excluded from compound feature analysis.

The designer should separate features by the proximity distance to provide the unfamiliar motorist time to recover from the experience of driving the first unexpected feature before being required to begin perceiving a subsequent surprise feature. The time a motorist needs to recover, perceive, and react to a subsequent unexpected feature is estimated to range from 5 to 10 s or more. Since the viewing (and maneuvering distance) to the next feature is also in the same range (5-10 s), an overall separation time of 10-20 s is desired.

Provide Feature Visibility in Proportion to Criticality of Work-Load Rating

The designer should seek to provide as much sight distance as is practicable on roadways that approach geometric features. The greater the work-load rating (i.e., the more unexpected and complex the feature is), the greater the sight distance needed. Adequate sight distance and feature visibility provide the time unfamiliar drivers will need to correct false expectations, decide on the appropriate speed and path, and make the required traffic maneuver.

The effective design speed from which sight distances would be determined is calculated from the following formula:

$$V_e = V_0 + 8W \quad W \geq 2 \tag{1}$$

where

V_e = effective design speed (km/h),
V_0 = original design speed (km/h), and
W = work-load potential rating in Table 1 ($W \geq 2$).

Thus, a geometric feature that has a work-load rating of 2.0 would need a sight distance given by a design speed effectively 16 km/h (10 mph) greater than the existing original design speed of the highway. The 85 percentile vehicle operating speed ($V_{85\%}$) on the approach prior to the feature should also be estimated. The higher speed value of V_0 or $V_{85\%}$ should be used to determine the needed sight distance from standard sight-distance design tables.

Provide Adequate Transitions

In addition to providing adequate visibility and separating basic geometric features in new construction to reduce driver work load and to improve traffic safety, adequate transitions also should be provided in all improvement programs and new designs to improve traffic operations. These transitions are desired for those geometric features that require vehicle path adjustments to achieve safe vehicle control. Common geometric features that need an acceptable transition include the following:

1. Lane drops,
2. Lane-width reductions,
3. Shoulder drops (greater than 50 percent reduction), and
4. Divisional channelization at divided highway transitions and channelized intersections.

The transition for the feature may be developed by using a straight taper transition in the departure direction along the roadway that goes from the higher-standard design to the lower-standard cross section. A similar transition in the opposite direction may be provided for symmetry if desired. The longitudinal transition taper ratio should not be less than the design speed to one or the latest recommendation of the Manual on Uniform Traffic Control Devices (MUTCD) (4).

Good transitions should be provided at the job terminals, even if plans exist to continue with the improved design in the near future. Plans can change rapidly and unexpectedly. The new project may not be finished for years. The transition requirements from a multilane divided highway to a two-lane highway without paved shoulders are large. It would appear unreasonable to continue these improvement programs to some arbitrarily selected location, such as at a major intersection located on the crest of a hill, then use a temporary reverse horizontal curve throughout the transition zone to connect the roadways, and finally expect motorists to negotiate the resulting compound feature safely until funds can be obtained to complete the work.

Provide Forgiving Roadside

When the previously recommended design objectives and practices are not feasible or cannot be implemented

Table 2. Work-load potential ratings (R_C) of horizontal curves.

Degree of Curvature (D°)	Deflection Angle (Δ°)				
	10	20	40	80	120
1	0.5	1.0	2.1	4.1	6.2
2	1.2	1.5	2.0	3.0	4.1
3	2.1	2.3	2.6	3.3	4.0
4	3.1	3.2	3.5	4.0	4.5
5	4.0	4.1	4.3	4.7	5.2
6	5.0	5.1	5.3	5.6	6.0
7	6.0	6.1	6.2	6.5	6.8
8	7.0	7.0	7.1	7.4	7.7

Note: All ratings are for two-lane, high-design highways. Ratings for two-lane mediocre roads (i.e., surface treatment pavement without paved shoulders) equal 0.75 of rating shown. Ratings for all four-lane highways equal 0.65 of rating shown.

effectively, the problem feature should be designed to have especially forgiving roadsides for possible errant vehicle operations. Estimate the more probable path guidance errors that might be made by drivers when exposed to the geometric feature susceptible to expectancy violations. Allow for these potential driver-control errors in design by providing clear recovery areas sufficient in space, slope, and surface stabilization to permit safe recovery of vehicle control. Paved shoulders may be an appropriate recovery addition for some situations. Hazardous roadside obstacles should be removed, relocated, or softened in areas where severely out-of-control vehicles might travel. One may wish to review the latest roadside safety literature, especially the American Assocation of State Highway and Transportation Officials' (AASHTO's) Yellow Book (5), before selecting a specific forgiving roadside design.

SPECIFIC FEATURE RATINGS AND DESIGN PROCEDURES

The following section presents material critical to the geometric consistency evaluation of specific geometric features and procedures for designing consistent features. The material is more technically detailed to more specifically identify trade-offs that exist among the related design variables.

Each geometric feature will have a set of estimated work-load potential ratings provided. These ratings serve three purposes. The ratings identify the estimated driver work load for the primary design variable; the ratings can be used to estimate the sight distance needed if an individual feature is being designed (in lieu of the average values presented in Table 1); and the ratings will be used in the roadway system-evaluation procedures to be described later in this paper.

The limits of these design and evaluation aids must be recognized by the user. The design-speed range is from 80 to 129 km/h (50-80 mph). Topography can be flat or rolling. Low speeds and mountainous terrain are not included because driver expectancy and driving experience are greatly different from those on routine rural highways.

Horizontal Alignment

Research and the literature review have shown that sharper curves are generally more troublesome to drivers. Accident rates were reported to increase significantly on curves greater than 8° (6). These curves were observed in this research study to be very rare curves in the normal rural highway driving experience (3). The most frequently used curve in design was noted to be about 2°. Illinois noted that excessively long curves are accident prone and are to be discouraged. The frequency of horizontal-curve central deflection angles (Δ°) was also measured. A central deflection angle of 20° was found to be about the average angle (3). An average horizontal curve was assumed to be a 3° curve with a 20° deflection angle. A work-load potential rating of 2.3 for an average horizontal curve had been previously established (Table 1) for a two-lane high-design road. Other work-load potential ratings (R_C) were estimated over the ranges of degree curvature based on the relative magnitude of side-force levels expected from the results of other operational speed studies reported earlier. Excessively long curves were rated proportionally higher. The resulting work-load potential ratings for a wide range of horizontal curve conditions are presented in Table 2.

AASHTO design procedures regarding horizontal alignment (7) (and in combination with vertical alignment) address several objectives, including geometric consistency of alignment. Selected procedures imply driver expectancy considerations. Aesthetic qualities are also reflected.

In addition to the AASHTO general horizontal alignment design procedures, the following horizontal alignment design procedures are recommended to maintain consistent geometrics:

1. The maximum increase in curvature between horizontal curves should not exceed 3° and
2. Horizontal curves that exceed 3° should be avoided in compound geometric features.

Vertical Alignment

The primary effect of vertical alignment on driver expectancy of criticality of geometric features is in the restriction to sight distance. Sight distance impacts were judged on an average basis during the evaluation of basic geometric features. Secondary effects are the unexpected frequency and duration of limitations on passing over a section of two-lane highway. Approximations of these passing limitations are provided. The impact of grade is believed to be negligible in flat and rolling topography. Speed losses are approximately regained on the downhill side, and little overall speed increase would be expected. Speed differentials between automobiles and trucks that exceed 16-24 km/h (10-15 mph) may create operational conflicts and unsafe driving practices. These differences estimated between vehicle speed profiles may be evaluated by using Leisch's method (8). No specific rating is provided for this situation. Sag vertical curves do not appear to contribute significantly to potential geometric inconsistencies when designed to prevailing operating speeds.

The work-load potential rating (R_V) for a given crest vertical curve can be determined from the table below by knowing the number of crest vertical curves in the prior 1500 m (5000 ft), including the one being analyzed.

No. of Crest Vertical Curves in Prior 1500	Work-Load Potential Rating (R_V)
≤2	1.9
≤3	3.0
≤4	4.0
≤5	5.0
>5	6.0

The vertical point of intersection (VPI) of the crest curve is used to determine the location of the curve. The work-load ratings were developed from the previously estimated average crest vertical-curve work-load potential rating of 1.9 (Table 1) and distributed to other conditions based on approximate probabilities of not being able to pass. For all multilane highways, assume the work load of a crest vertical curve is 1.2 regardless of the frequency.

AASHTO design procedures for providing aesthetics and consistency in vertical alignment design are recommended in general (7).

The following additional design consistency guidelines are offered based on the results obtained and observations made in this research.

1. Increase the design speed of a highway toward 113 km/h (70 mph) if few crest vertical curves exist and if the roadway is similar to local 113-km/h designs.

Table 3. Work-load potential ratings (R_i) for intersections.

Highway	Type of Approach to Intersection	Approach Stop or Yield Controlled	Approach Not Controlled: Crossroad Average Daily Traffic			
			≤100	<400	<1000	>1000
Two-lane high-design	Unchannelized	4.0	1.5	2.4	3.7	4.0
	Channelized	4.0	2.5	3.0	3.3	4.0
Four-lane undivided	Unchannelized	4.0	1.0	1.6	3.0	4.0
	Channelized	4.0	1.6	2.0	2.8	4.0
Four-lane divided	Unchannelized	4.0	0.6	1.2	2.7	4.0
	Channelized	4.0	0.5	1.0	2.5	4.0

Note: Mediocre two-lane road (i.e., surface treatment without paved shoulders) values equal 0.75 of two-lane high-design values.

2. Isolated crest vertical curves in flat topography will probably be driven at an apparent design speed of 113 km/h if the pavement surface quality and traffic volumes permit.

Intersections

Work-load potential ratings for channelized and unchannelized intersections located on high-design, two-lane facilities and on multilane highways are presented in Table 3. Only a few general classification parameters are used for practicality. Channelized intersections refer primarily to whether the approach has a protected left-turn bay. Stop- or yield-controlled approaches are treated separately from noncontrolled approaches.

It is difficult to design a major intersection in rural areas so that it will be expected. Stop-controlled intersection approaches on main highways are usually troublesome. When horizontal or vertical curvature is present, stop-controlled approaches are seldom satisfactory. Multiple or otherwise complex route numbers also create unexpected decision problems for motorists unfamiliar with a junction. No-passing zones and channelization may also confuse motorists.

The sight distance provided along the roadway that passes through the intersection should be increased above minimum required stopping sight distance in relation to the total work-load potential rating of all features within proximity distance or 457 m (1500 ft) of the center of the intersection. That is, sight distances for the resulting compound geometric feature should be provided to AASHTO requirements (7) for an adjusted design speed higher than the base design speed of the facility. The increased speed is calculated from the following formula:

$$V_a = V_0 + 8R_\Sigma \quad R_\Sigma \geq 2.0 \tag{2}$$

where

V_a = adjusted design speed (km/h),
V_0 = existing base design speed of roadway (km/h), and
R_Σ = sum of work-load ratings of all features within proximity distance 457 m (1500 ft) (≥2.0).

At least 16-km/h (10-mph) increases in adjusted design speed are desired. No adjustment is needed if R_Σ is less than 2.0.

In general, horizontal curvature greater than 3° in intersections should be avoided on through roadways. Reverse curvature within the intersection area is also undesirable and should not be used on crest vertical curves.

Lane-Width Reductions

The criticality of lane-width reductions is primarily a function of vehicle lane placement, placement variability, and initial lane width. The average work-load potential rating of lane-width reductions was estimated to be 3.1. Evaluations of research on lane placement variability with horizontal curvature (9) and this research were used to develop the work-load potential ratings (R_ℓ) for two-lane high-design highways shown in the table below (1 m = 3.3 ft).

Reduction in Lane Width (m)	Work-Load Potential Ratings (R_ℓ) for Initial Lane Widths: 3.35 m	3.66 m	3.96 m
0.30	2.3	2.0	1.8
0.61	3.8	3.0	2.5
0.91	No good	No good	4.7

The ratings in the table were established for tangent alignment. Ratings on two-lane mediocre roads (i.e., surface treatment without paved shoulders) equal 0.75 of value shown. Ratings for all four-lane highways equal 0.65 of value shown. Average lane placement data suggest that vehicles drive closer to the edge of the pavement when traveling on the inside of the curve and drive farther from the edge when traveling on the outside.

Reduction in lane width must be carefully designed with good visibility of the pavement surface provided since all drivers will be exposed to the feature. Liberal transition tapering should be used to achieve the lane-width reduction. Paved shoulders may be substituted for the transition taper if the work-load potential rating is 2.0 or less. If the rating is greater than 2.0, transition tapering and all-weather stabilized shoulders should be provided to provide a forgiving roadside.

Divided-Highway Transitions and Lane Drops

The work-load potential ratings for lane drops and divided-highway transitions depend on the feature characteristics and direction of travel. Average ratings have been established as 1.8 for a four-lane to four-lane divided-highway transition (median drop), 4.0 for a four-lane to two-lane divided-highway transition (median and lane drop), and 3.9 for an undivided-highway four-lane to two-lane lane drop. No ratings are available for larger facilities. Observations indicate that short sections of divided-highway transitions, such as at a roadside park, and multilane passing sections would not surprise motorists when they terminate. A longer multilane facility might. This premise is reflected in the work-load potential ratings (R_t) for these features presented in Table 4.

Design procedures are presented by AASHTO for the design of divided highway transitions (7). The provision of a divided-highway transition on a horizontal curve that has a suitable alignment standard would provide satisfactory results. On long tangents, changes in median width cannot be effected readily except by reverse curves on one or both pavements.

The previous AASHTO design procedures generally seem satisfactory. The suitable alignment standard for a simple horizontal curve is probably about 3.0°. Reverse horizontal curvature of 2.0° or less should prove operationally satisfactory since this is approximately the most frequently used curve in highway design and motorists are accustomed to driving it.

Table 4. Divided highway transitions and lane drop work-load potential ratings (R_t).

Geometric Feature	Lane Direction		Work-Load Potential Ratings (R_T) for Prior Section Lengths		
	From	To	≤3.2 km	≤8.0 km	≥8.0 km
Divided-highway transition	4	2	3.0	3.3	4.0
	2	4	2.0	2.0	2.0
	4	4	1.0	1.5	1.8
Lane drop	4	2	2.5	3.0	3.9
	2	4	1.5	1.5	1.5

Note: 1 km = 0.62 mile.

The design need often arises to combine a divided highway transition with an intersection. This situation may have arisen due to a large drop in volume along a rural four-lane road as it intersects a state highway. This intersection most likely will be one that has high turning movement volumes (since the volume level drops significantly at this location). The intersection may also have some (and possibly significant) cross traffic. It follows that the divided highway's turning traffic will be slowing while the through traffic will be trying to maintain a constant speed. A large speed differential between vehicles and a high potential for traffic conflict will exist. In general, a high accident frequency can be expected.

The mixing of divided highway transitions or lane drops with intersections should be avoided. Unfamiliar motorists are never expecting this complex situation. If one must be constructed, no reduction in design speed should be permitted through the section, visibility should be maximized, only natural flowing horizontal alignment of 3° or less should be used, and paved shoulders should be maintained throughout the features.

Lane drops in rural highway design are typically a feature that results when a four-lane undivided roadway is reduced to a two-lane highway. Other lane-reduction features that should be considered as lane drops include the following:

1. Termination of passing (or truck climbing) lanes greater than 3.2 km (2 miles) long and
2. Termination of an intersection of a through or climbing lane.

In general, a lane drop should be considered as severe a potential geometric inconsistency as a divided highway transition (four-lane to two-lane), except for the likelihood that motorists may become entrapped by divisional channelization.

Lane drops designed according to AASHTO standards are satisfactory when not brought into combination with other geometric features. Several research reports discuss the options of which lane to drop and what pattern to use. Review of these documents is suggested for developing optimum design configurations. Stabilized shoulders should be maintained throughout the transition section.

EVALUATING GEOMETRIC DESIGN CONSISTENCY

The engineer may wish to evaluate either proposed designs or existing highways for geometric consistency. Routine safety reviews may have identified the existing highway as being accident prone. The evaluation process may consider only one problem geometric feature or an extended section of highway. The study procedure is directional in nature and treats one highway direction at a time. The analyst should begin by reviewing the features of the highway prior to the study area. To begin the evaluation, find the most featureless section of highway (e.g., level, tangent) approximately 2 km (1.25 miles) in advance of the study area. Estimate the driver work load of this section of roadway. With design plans in hand and photographs of the existing design (if available), begin the following evaluation procedure.

Identify Geometric Features

Use Table 1 as a guide in identifying the types of geometric features along the highway in the study area. Determine the following items for each feature:

1. Type,
2. Station,
3. Work-load potential rating (R_f),
4. 85 percentile speed [km/h (mph)],
5. Sight distance [m (ft)] to geometric feature, and
6. Separation distance [m (ft)] from last geometric feature.

Obtain Work-Load Potential Rating (R_f)

The basic work-load potential rating for the next geometric feature along the highway may be read directly from Table 1 to evaluate average conditions if this estimation level of accuracy is sufficient. Otherwise, determine the more specific work-load ratings for those features identified in the preceding tables. Note that vertical curvature initially must be considered separately on a systems basis to estimate the impacts of no-passing-zone restrictions.

Estimate 85 Percentile Speed

The 85 percentile speed ($V_{85\%}$) on the approach to the feature should be estimated. In essence, an 85 percentile operating speed profile is required along the highway for each direction of travel. Do not rely totally on the speed limit or design speed in estimating the 85 percentile speed. The 85 percentile speed is approximately 11 km/h (7 mph) above the estimated average speed. The allowable range of 85 percentile operating speeds is from 80 to 113 km/h (50-70 mph).

Determine Sight-Distance Factor

Estimate the maximum sight distance (S) to each feature by using the same measurement criteria as for safe stopping sight distance. Check both horizontal and vertical alignment restrictions. A motorist can be assumed to look through features to see other features downstream if they are visible. Use the midpoint (or obviously most critical location) of the feature for evaluation purposes, including the determination of separation distances. Having estimated the sight distance to the feature and the 85 percentile speed, read the sight-distance adjustment factor (S) from Figure 2. This factor adjusts rating values from average speed and sight distance levels to specific site conditions.

Determine Carry-Over Factor

Once the separation distance from the last feature and the V_{85} speed are known, determine the work-load carry-over factor (C) from Figure 3. This factor adjusts conditions from isolated conditions to specific circumstances. It accounts, in general, for driver memory loss, decision-sight-distance requirements, and average viewing distances (not sight distance) used in the driving task.

Calculate Feature Expectation Factor

The feature expectation factor (E) adjusts for the potential confirmation of driver expectancy where the prior feature is similar to the current feature. If the feature is similar to the prior feature, $E = 1.00 - C$. If the new feature is not similar to the last one, $E = 1.00$. Horizontal curves that have curvature more than 3° greater than the preceding horizontal curve may be considered not similar to the previous curve (3). Flatter curves are always similar when immediately following a sharper curve.

Figure 2. Sight distance factor (S) due to sight distance to next feature as related to 85 percentile speed.

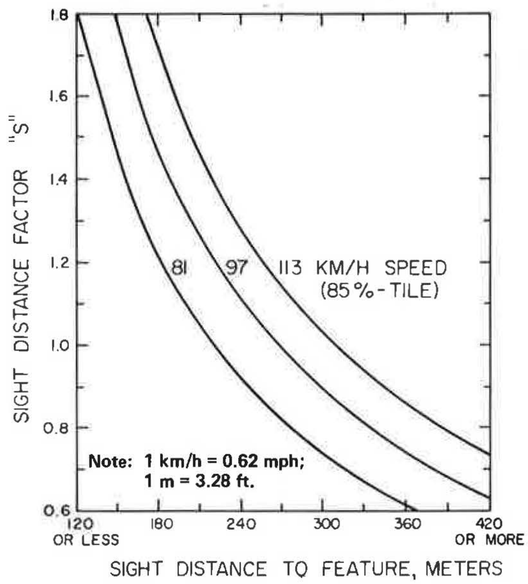
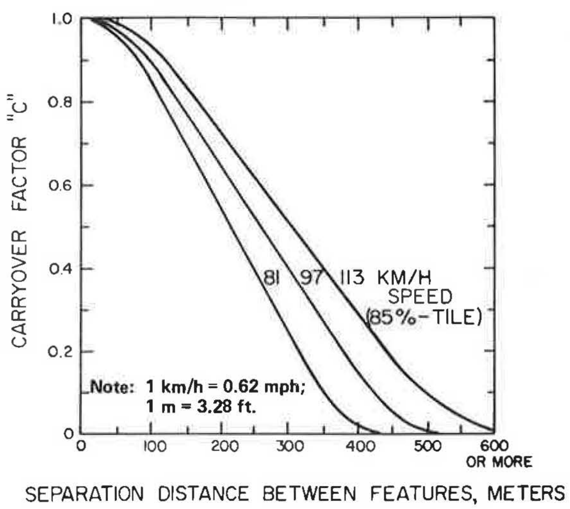


Figure 3. Carry-over factor (C) due to separation distance between features as related to 85 percentile speed.



Estimate Driver Unfamiliarity Factor

The higher the percentage of motorists unfamiliar with the highway (U), the higher the probability of drivers being surprised by relatively unusual geometric features. Use the table below as a guide for estimating U.

Classification System	Examples	Factor U
Rural principal arterial	Major U.S. highway, Interstate	1.0
Rural minor arterial	U.S. route, major state highway	0.8
Rural collector road	State highway, major farm-to-market road	0.6
Rural local road	Farm-to-market road, county road	0.4

Calculate Driver Work-Load Value

Evaluation of the potential for the geometric feature to be inconsistent by using these procedures is based on the calculated driver work-load value (WL_n). WL_n refers to the work-load value being calculated for the next feature, whereas WL_ℓ refers to the work-load value previously calculated for the last feature. The work-load value (WL_n) is determined from the following equation, which uses the previously described factors and the work-load value calculated in the last feature (WL_ℓ).

$$WL_n = U \cdot E \cdot S \cdot R_f + C \cdot WL_\ell \quad (3)$$

Estimate Level of Consistency of Feature

The previous factors have been combined to provide information that could indicate which unexpected geometric features are creating a problem. But at what value of work load can this conclusion be drawn? At present, this decision is subjective. However, in an effort to standardize the process and to allow relative comparisons, the criteria presented in the table below are suggested.

Driver Expectation	Level of Consistency	Work-Load Value, (WL_n)
No problem	A	≤1
expected	B	≤2
Small surprises	C	≤3
possible	D	≤4
	E	≤6
Big problem possible	F	>6

One may conclude that a $WL_n > 6$ is defined as an apparent geometric inconsistency. Thus use the above table to estimate the level of consistency (LOC_n) of the geometric feature given the calculated work-load value.

The designer would use this procedure to minimize both the absolute level of geometric work-load values and also the jump between features. Ways to improve the level of consistency include the following:

1. Improve geometrics,
2. Increase spacing between features, and
3. Increase sight distance to the feature.

CASE STUDY

The following brief case study is presented to illustrate the basic methodology of applying the geometric design consistency procedures to an existing highway. Few, if any, modifications are necessary to apply the procedures to a proposed new design. The basic differences are that a specific problem site may not have been identified based on accident statistics and that estimates of operational variables rather than field measurements would be required. In the case study evaluated, a complete set of calculation results will be provided. Subsequent level-of-consistency evaluations will be presented.

A few years ago, an existing two-lane, primary highway was improved to a four-lane divided facility for 2.4 km (1.5 miles) to expedite the construction of a new railroad bridge overpass. The northern terminus of the four-lane section is located on a rather featureless stretch of the highway. However, the southbound terminus is just beyond the crest of a 97-km/h (60-mph) vertical curve. Other geometric features found along the four-lane divided section include a bridge and a flat, horizontal curve. The three-year accident history indicates that seven accidents have occurred in the southbound direction at the southern divided-highway transition. The average daily traffic (ADT) along the highway is 5200 vehicles/day. The 85 percentile vehicle speeds in the four-lane section vary between 100 km/h (62 mph) and 105 km/h (65 mph), depending on the vertical alignment.

The southbound direction only will be evaluated for consistency. A drive through in the southbound direction

Table 5. Evaluation of geometric consistency of divided-highway transition case study.

Feature	Divided-Highway Transition, Two-Lanes to Four-Lanes	Bridge, Full Width, No Shoulders	Horizontal Curve 2°, 15°	Railroad Bridge Overpass	Crest Vertical Curve	Divided-Highway Transition, Four-Lanes to Two-Lanes
	Calculations of Geometric Features in Southbound Direction of State-16					
Station	701+10	727+90	734+90	740+00	776+50	780+00
R_f	2.0	2.5	0.9	0.8	1.2	3.0
V_{85} (km/h)	105	105	105	105	100	100
Sight distance (m)	600	321	606	545	909	144
Separation distance (m)	636	812	212	155	1106	106
R_f	2.0	2.5	0.9	0.8	1.2	3.0
S	0.69	0.90	0.69	0.69	0.69	1.80
E	1.00	1.00	1.00	1.00	1.00	1.00
U	0.80	0.80	0.80	0.80	0.80	0.80
C	0.00	0.00	0.68	0.73	0.00	0.90
WL_ℓ	0.0	1.1	1.8	1.7	1.7	0.7
WL_n	1.1	1.8	1.7	1.7	0.7	5.0
LOC_n	B	B	B	B	A	E

Note: 1 m = 3.3 ft; 1 km/h = 0.62 mph.

would suggest that conditions are good except near the southbound terminus of the four-lane section where the crest vertical curve severely limits the sight distance to the divided highway transition for the existing operating speeds. The route is classified as a rural minor arterial since it is a major state highway and, therefore, the U factor is 0.8.

Table 5 presents a summary of the initial geometric feature data, resulting calculations, and level-of-consistency evaluations. The calculations solve the work-load equation. Level-of-consistency evaluations are determined based on the criteria presented in the preceding table. As indicated in the last line in Table 5, the four-lane roadway is very consistent (i.e., the level of consistency is B, B, B, B, A, E) over much of the four-lane section except at the southern (last) divided-highway transition. To solve this problem the divided-highway transition should be moved farther away from the crest of the vertical curve until the sight distance is again unrestricted.

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Abridgment

Use of Total Benefit Analysis for Optimizing Lane Width, Shoulder Width, and Shoulder Surface Type on Two-Lane Rural Highways

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The relationships between safety and design for lane width, shoulder width, shoulder surface type, and accidents have been developed. The objective of this paper is to demonstrate how these and other relationships may be employed to obtain optimal design specifications for lane width, shoulder width, and shoulder surface type. A manual procedure is presented for basic design problems. As the complexity of the problem increases, some computerized optimization procedure, such as dynamic programming, is recommended.

The primary purpose for conducting research to establish the relationships between the various roadway design features and accidents (frequency and severity) is to enable safer roadways to be constructed under the current funding limitations faced by the states. A critical element in the policy-determination process is often omitted, which leaves valuable research with no means for practical application. This is the step of translating established relationships into quantitative economic models that can be used for optimizing roadway designs. The objective of this paper is to present one such model for consideration for general use.

We will assume that a reliable set of relationships can be obtained through standard research procedures. A detailed review of past research efforts has been conducted (1). Further, a set of relationships, developed especially for this type of optimization process, has been developed for lane width, shoulder width, and shoulder surface type for the National Cooperative Highway Research Program (NCHRP) project 3-25 (1). Although the development of such relationships is not new (2), the special procedures employed in this project to account for interactions between design features makes these relationships particularly appropriate for an optimization design process.

The use of a total benefit approach to the allocation of high-hazard funds for spot roadway improvements was documented as early as 1973 (3). This technique was adopted as an ongoing operating procedure within Alabama (4), and the computer software employed has been well documented (5,6). In addition, these procedures have been published in two independent technical journals (6,7), and a simultaneous implementation was made in Kentucky (8).

The total benefit approach, as implemented by using dynamic programming for optimization, has generally been accepted as (9) "the best procedure identified as currently being used in any state." Although the same concept of maximizing the total benefit obtained subject to budgetary constraints is employed, the procedure discussed here varies from those described above in two important respects: (a) it applies primarily to new construction and major reconstruction as opposed to spot improvements and (b) it employs a manual procedure for optimization.

STATEMENT OF PROBLEM

Briefly stated, the problem is that of allocating a limited budget to roadway construction in such a way that safety is not compromised. Stated another way, the objective is to balance the various costs of increased design parameters so that the total safety benefit is maximized. Budgetary resources are assumed to be limited; however, no assumption is made that they are known from the outset. That is, the problem is complicated slightly inasmuch as the designer is generally not given a specific amount of money to allocate to the roadway under consideration. Usually the design is used to determine the cost or funding request. Thus, the procedure devised for solving this problem must have enough flexibility to enable an easy consideration of last-minute changes in funding.

In order to further limit the scope of the problem at hand, we will assume that the relationships between the design specification variations and accidents (frequency and severity) have been estimated with some degree of confidence. We will also assume that the construction cost variation is known for the variations in the design specifications under consideration. Considerable additional work is required in these two areas, but they will be considered only briefly below because they fall outside the scope of primary consideration. Note, however, that the economic evaluation methodology considered here could contribute heavily toward both the direction and the increased application of future research efforts.

Table 1. Example subset of alternatives for example tangent section.

Pavement Width (ft)	Shoulder Width (ft)	Shoulder Surface Type	Segment Cost[a] ($000s)	Accident Cost ($000s)
20	4	Unpaved	102	163
	6	Unpaved	106	150
	8	Unpaved	109	137
	4	Paved	110	159[b]
	6	Paved	121	146[b]
	8	Paved	129	132
22	4	Unpaved	149	147[b]
	8	Unpaved	153	137[b]
	10	Unpaved	156	118
	4	Paved	157	144[b]
	8	Paved	168	129[b]
	10	Paved	176	114
24	4	Unpaved	199	131[b]
	8	Unpaved	203	115[b]
	10	Unpaved	206	99
	4	Paved	207	128[b]
	8	Paved	218	112[b]
	10	Paved	227	96

[a]Accident costs were obtained from the relationships developed in NCHRP Report 197 (1).
[b]Eliminated from further analysis due to increasing accident costs.

SOLUTION TECHNIQUE

The solution technique will be called a total benefit technique because the objective function is to maximize the total benefit from a safety point of view. This technique will be exemplified in terms of trade-offs among allocations to lane width, shoulder width, and shoulder surface type. Four major phases are associated with the total benefit methodology:

Phase 1: Determine construction costs,
Phase 2: Determine accident costs,
Phase 3: Determine candidate designs, and
Phase 4: Select the final design.

The first two phases of developing construction and accident costs can be performed concurrently and independently of each other, and they are beyond the scope of this paper. Once this is accomplished, the last two phases can be performed as discussed below.

Determination of Candidate Designs

The first two phases involve calculation of the construction and accident costs for any alternative design. This application is concerned with the combination of pavement width, shoulder width, and shoulder surface type. Because of the large number of possible combinations of these alternatives, the designer must select only those alternatives (or combinations) that are practical in terms of highway agency policy, route continuity, and other nonsafety factors related to the project. After these designs have been selected, construction and accident costs over the service life of the project are calculated for each.

These alternatives are next arrayed by increasing construction costs. Table 1 (1-6,10,11) illustrates a typical array based on example costs output from phases 1 and 2. Note that an ascending construction cost does not necessarily lead to decreasing accident costs. For example, the alternative for 20-ft pavement and 4-ft paved shoulders has an estimated accident cost of $159 000, which is more than the accident cost for the preceding alternative at a lower construction cost. Since an increase in construction costs accompanied by an increase in accident costs could not possibly be a cost-safety-effective design, these alternatives should be eliminated from further analysis.

The candidate design alternatives in Table 1 are for roadway segments within the project limits that have horizontal curvature of less than 3°. Another group of candidate design alternatives must be selected for segments that have 3° or greater horizontal curvature. Practical alternatives from each curvature group are then combined into candidates for the total project design specifications.

Table 2. Cost/benefit data for alternatives.

Alternative Total Construction Costs ($000s)	Marginal Construction Costs ($000s)	Reduced Accident Costs ($000s)	Cumulative Increased Construction Costs ($000s)	Cumulative Reduced Accident Costs ($000s)
2366.9	Minimum cost to construct roadway section			
2375.9	9.0	2.1	9.0	2.1
2382.9	7.0	7.0	16.0	9.1
2391.9	9.0	1.6	25.0	10.7
2447.9	56.0	5.3	81.0	16.0
2463.9	16.0	8.5	97.0	24.5
2510.8	46.9	43.5	143.9	68.0
2519.8	9.0	1.6	152.9	69.6
2527.1	7.3	3.3	160.2	72.9
2536.1	9.0	1.2	169.2	74.1
2591.8	55.7	5.3	224.9	79.4
2608.1	16.3	4.5	241.2	83.9
2657.9	49.8	20.4	291.0	104.3
2666.9	9.0	1.2	300.0	105.5
2674.4	7.5	2.4	307.5	107.9
2683.4	9.0	1.2	316.5	109.1
2738.9	55.5	4.2	372.0	113.3
2755.4	16.5	3.6	388.5	116.9
2805.9	50.5	14.5	439.0	131.4
2814.9	9.0	1.2	448.0	132.6
2831.1	16.2	0.6	464.2	133.2
2886.9	55.8	7.1	520.0	140.3
2903.1	16.2	0.6	536.2	140.9
3033.4	130.3	4.0	666.5	144.9

Selection of Final Design

The final design is selected from the alternatives identified in phase 3. No rigid procedure is specified to force a design on the descision maker. Rather, the methodology of phase 3 produces all of the information required for cost/benefit, marginal cost/benefit, and break-even analyses. At this point the designer must take into account other design characteristics that affect the final selection of pavement width, shoulder width, and shoulder type. The final design selected must represent the agency's highway improvement policy in terms of these design features, as well as the most cost-safety-effective design.

Table 2 summarizes the marginal safety benefits and the total safety benefits from an actual appliction of the NCHRP research made to a road section in Alabama. The marginal cost is the additional construction expenditure, and the marginal safety benefit is the reduction in accident cost when the corresponding construction expenditure is made. All feasible combinations that involve both curved and tangent alternatives over the roadway were considered. The total safety benefit is the cumulative total of reduced accident costs, and the cumulative increased construction cost is the increased construction costs over the lowest-cost alternative. As more funds are expended, their capability to purchase safety benefits is generally reduced. The first $2 366 900 is considered the basic expenditure. It is required to construct the project to a minimal safety level. From that point on, the additional investments are viewed as contributing additional safety benefits.

From the marginal benefits shown in Table 2, for an additional $9000 from the minimum cost of $2 366 900, an additional $2100 in safety benefits can be realized. If an additional $7000 is expended for the next alternative, the additional benefit was estimated to be $7000—equal to the additional investment. However, the cumulative additional construction cost is $16 000 for total additional reduced accident costs of $9000.

On the basis of marginal benefits, expenditures in excess of the minimum design do not return equivalent benefits. This result is only for the example given here and not a general conclusion of this research. Each construction project must be evaluated on its own merits. Also, because the construction-accident trade-off is so dependent on estimated costs of accidents by severity, it is not recommended that this be the sole criterion for selection of the construction expenditure. Rather, each of the policies specified is optimal for the corresponding construction expenditure because all suboptimal alternatives were eliminated. High-level judgment must be made at this point in light of alternative uses of funds by using similar results of other project analyses. The final design must represent the agency's highway improvement policy in terms of specific design features as well as the most cost-safety-effective design.

CONCLUSIONS

One criticism that has been leveled against cost-effectiveness techniques applied to safety is that "dollars are being traded against lives." The technique presented circumvents this problem by changing the analysis from a comparison of costs and benefits to a maximization of benefits given a fixed budget. This is a more practical approach that can lead to greater overall roadway system safety.

Use of the total benefit methodology will permit an agency to select designs of pavement width, shoulder width, and shoulder type that are optimum from a safety standpoint. That is not to imply that it leads to the lowest possible accident costs but rather that it produces the lowest accident costs for the expenditure of funds that is to be made. Depending on other design criteria that may offset the final alternative selected, this concept of tailoring the design for each project will result in lower construction costs than would have been required without it. Any reduction in construction costs for a given project can be applied to the improvement of additional miles of highway that would not have been possible without the availability of these additional monies.

Effective use of the methodology presented above will provide safety benefits in excess of those that would be obtained without its use. Although safety benefits are not maximized for each project, total safety benefits are increased by the improvement of more miles of facilities, rather than a few miles of improvements that are built to designs that are not cost effective.

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Strategy for Selection of Bridges for Safety Improvement

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In order to upgrade traffic safety of existing bridges in a systematic and cost-effective manner, we must have a clear understanding of how safety is measured and controlled. Safety is not an absolute but a relative condition that balances the risk of an event and society's acceptance of that risk. Something is considered safe if its risks are acceptable. Risk is measured by the probability of the occurrence of an adverse event (i.e., bridge accident) and the event's consequences (i.e., collision severity). Based on length alone, a bridge is 50 times more hazardous than the roadway in general. The large number of bridge accidents is attributed to narrow bridges and to obsolete approach guardrail and bridge rail installations. To improve bridge traffic safety, the ideal solution would be to widen all narrow bridges and upgrade barrier installations on all other bridges. Because of cost, this approach is not practical. As an alternative, bridge selection for safety improvement can be based on degree of risk and available funds concentrated on the high-risk bridges. This procedure, which is also applicable to other roadside safety problems, advocates uniform standards for degree of risk rather than uniform standards for design. In fact, design standards will be varied according to site requirements to achieve the acceptable level of risk. Two techniques are presented to identify bridges that have a high degree of risk: (a) adverse accident experience and (b) high traffic volume coupled with substandard highway features. The extent and type of safety improvements are presented.

The term safety is currently a very popular word, judging by its use as a topic in newspapers, books, magazines, and other media. The term safety carries a heavy emotional and political load. There is considerable public confusion about safety, and this confusion is not helping the highway community improve its system.

A simplistic and misleading definition of safe is "free from harm or risk." However, nothing can be absolutely free of risk. Because nothing can be absolutely free of risk, nothing can be said to be absolutely safe. There are degrees of risk, and, consequently, there are degrees of safety (Figure 1). Safety, then, is a judgment of the acceptability of risk; and risk, in turn, is defined as a measure of the probability and severity of harm to human health (1). In other words, something is safe if its risks are judged to be acceptable. Even with a specific measure of risk, the acceptability judgment, which is a value decision made by all or a segment of society, may vary with time and place.

Degree of risk is measured by the probability of an event multiplied by consequences or severity of the event:

$$\text{Degree of risk} = \text{probability of occurrence} \times \text{probability of consequences} \quad (1)$$

The degree of risk can be lowered by causing a decrease in the number of events or collisions or a reduction in the severity of the collisions when they occur. Unfortunately, it costs money to make these changes, so one should be sure the improvement in safety (reduction in risk) is worth the cost.

HIGHWAY SAFETY EFFORT

Traditionally, highways have been constructed or upgraded according to state or federal design standards. For example, highway features (such as typical cross sections, lane widths, maximum horizontal curvatures, maximum shoulder slopes, and minimum roadside clear zones) are consistently high on the Interstate system. Low fatality rates on the Interstate system have proved the effectiveness of the high-design standards. On the other hand, some believe that much of the Interstate system has been unnecessarily built to these high and costly design standards. Thus, safety funds have been spent on highway segments where the degree of risk and, therefore, the potential for reducing fatalities are low.

As the safety upgrading attention is directed away from the Interstate system to the remaining 6 200 000 km (3 838 000 miles) of highways, highway agencies are forced to be more prudent with expenditures.

An alternative to the upgrading of highways to one or more specified uniform design standards is the upgrading of highways to a uniform standard for degree of risk. Such a standard can be quantified in terms of kilometers of highway per run-off-the-road type of fatality. An initial goal can be set at, say, 800 km (500 miles) and then increased as additional safety funds become available. This approach implies a variable design standard that is determined by the degree of risk at a local site and is in contrast to the uniform standard design approach used on the Interstate system. This uniform risk approach is the only strategy that is both effective and affordable. To select a uniform design standard will be either grossly wasteful of public funds or ineffective in reducing fatalities.

The application of risk management to the assessment and implementation of safety is an emerging technique in highway technology. The number of spot safety improvement programs is increasing. The multiple-service-level bridge railing selection procedure, which is based on risk measurement and assessment (2), is another example of the emerging technology. Moreover, considerable research activity is under way in this area. Although it will be a few years before a comprehensive technology is developed, some things can be done now.

THE BRIDGE SAFETY PROBLEM

The table below is based on 1975 data (3,4) (1 km = 0.62 mile).

Category	Length (km)	Fatalities	Kilometers per Fatality
Roadway	6 175 000	11 300	546.5
Bridges	12 400	1 120	11.1

In 1975, 45 850 people were killed; 11 300 of them were involved in a single-vehicle, ran-off-the-road, hit-fixed-object type of collision (5). Moreover, we know that the fixed object involved with at least 1120 of these fatalities was a bridge or bridge barrier. By dividing lengths of roadways and bridges by these fatalities, one can see that a fatality occured for every 546.5 km (340 miles) of roadway and every 11.1 km (6.9 miles) of bridge length. The bridge-to-roadway hazard ratio is 546.5:11.1; that is, based on length alone, a bridge is about 50 times more hazardous than the roadway.

Causes of Bridge Fatal Accidents

The question arises as to why so many fatal accidents occur at bridges. Causation factors and remedial treatments must be identified before we can rationally reduce the degree of risk. Some of the causation factors influence both the number and the severity of the event.

One of the primary causes of the large number of fatal bridge accidents is the relative narrowness of the structure. Of the nation's 564 000 bridges, 75 percent were built prior to 1935, according to the Federal Highway Administration's (FHWA's) national bridge inventory. Many of these structures were designed to carry smaller cars and few trucks. In the intervening years, pavement width has been increased to carry larger vehicles in greater numbers; however, due to expense, bridge width has not been increased. Thus, as shown in Figure 2, we have been left with wide pavement and narrow bridges--an inconsistency for the motorist. Hutchinson states that such inconsistencies violate the driver's expectation and cause the accident (6). The importance of bridge width is seen in Figure 3, where the Arizona bridge accident rate expressed in million vehicle kilometers of travel varies from 0.733 to 0.447 million vehicle-km (1.18-0.72 million vehicle miles) (7). If bridge widths are increased from 9.1 to 12.89 m (30 to 42 ft), the accident rate decreases by 39 percent.

Solutions

If money were not a consideration, the ideal solution would be to replace all narrow bridges with wider structures and replace all obsolete bridge barriers with high-performance systems. If one assumes that bridge barrier safety could approach that of the highways in general [that is, 1 fatality/546 km (340 miles)], then the 1120 bridge-related fatalities would be reduced by more than 98 percent to about 23 per year. This approach would cost more than $100 billion (8). Even considering the benefit of 1000 fatalities forestalled per year for 25 years, the cost would be $4 million/fatality forestalled—an extremely high value with respect to other alternatives. Of course, if a bridge is being replaced for other reasons or if it has experienced numerous accidents, the widening of the bridge may be justified.

Under the Highway Bridge Replacement and Rehabilitation Program, enacted in November 1978, $4.2 billion in funds are available over a four-year period to replace or upgrade some existing bridges (9). Bridges are to be rated by the states according to structural adequacy and safety, essentially for public use and serviceability and functional obsolescence. A simplified decision path for this program is illustrated in Figure 4 and shows when traffic safety is considered in the process.

For cases where traffic safety conditions are inadequate (Figure 4), but the bridge has a low priority for replacement, there are alternatives to widening a narrow bridge that can be used to reduce the number of accidents (10). The effectiveness of these treatments, acting alone or in combination, is unknown:

1. Realign roadway;
2. Change approach grade;
3. Transition shoulder to bridge;
4. Add approach bridge delineation;
5. Place edge lines;
6. Place pavement transition markings;
7. Install narrow-bridge signs;
8. Install stop, yield, or signalization;
9. Place advisory speed signs; and
10. Reroute commercial vehicles.

SEVERITY OF BRIDGE BARRIER ACCIDENTS

Certain remedial actions may reduce the severity of a bridge barrier collision. More than one-half of bridge-related fatal accidents occur at the bridge end or terminal post (Figure 5) (11). The terminal post, or tombstone, was a typical feature of most bridge railing until recently. Its contribution to severity of collisions was recognized some 10 years ago, after which time approach guardrail was used to funnel traffic onto the bridge. In the initial effort, the importance of structurally attaching the approach guardrail to the bridge railing system was not recognized (Figure 6), and this resulted in systems that were completely inadequate. The pocketing of vehicles at the juncture of the approach guardrail and the bridge railing caused a large number of fatalities (Figure 7).

Since 75 percent of bridges were built before 1935, it should not be surprising that the safety performance of bridge railings is obsolete with respect to today's safety standards (Figure 7).

Figure 1. Safety is a relative condition.

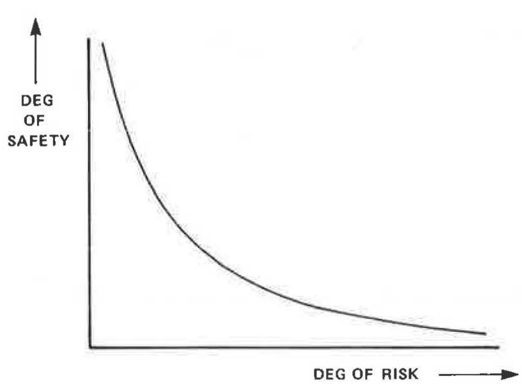


Figure 2. Abrupt constriction of roadway at narrow bridge causes accidents.

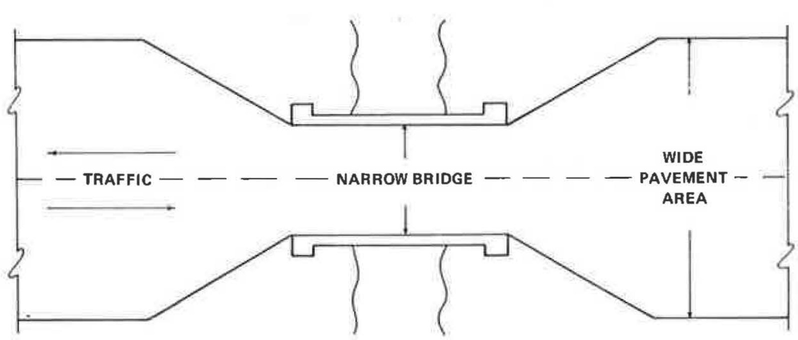


Figure 3. Accident rate as a function of bridge width.

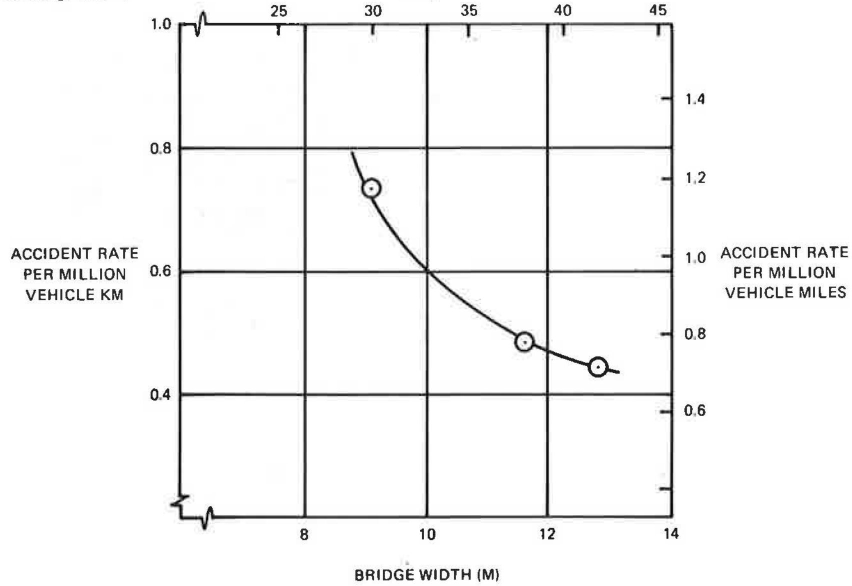
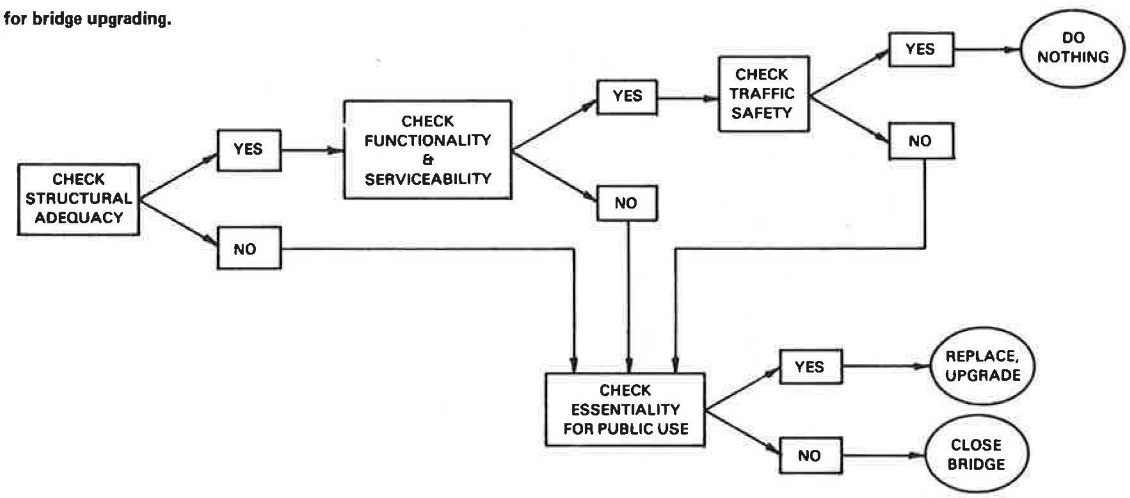


Figure 4. Decision path for bridge upgrading.



Thus, to decrease the potential severity of a bridge barrier collision, it is important to have a good approach rail that funnels the traffic onto the structure and is adequately attached to the bridge barrier (Figure 8). Crash cushions (such as sand drums) have been successfully used in cases where an approach guardrail is not feasible. Then the obsolete bridge barrier should be safety upgraded or replaced by current standard systems. Techniques for upgrading deficient barriers have been developed and are illustrated in Figure 9.

New barrier systems are contained in the American Association of State Highway and Transportation Officials (AASHTO) publications.

BRIDGE SELECTION

Up to this point, we have discussed what can be done to reduce the degree of risk. The problem is which bridges should be upgraded and to what extent, given a restricted amount of funds. An obvious means of identifying hazardous sites is by traffic accidents. Normally, we would filter out single-accident sites as a random event location; however, since bridges are known to have a high accident potential, a single accident at or on a bridge should trigger a design review of the facility.

In the absence of accident records, hazardous sites can be identified on the basis of traffic conditions and geometrics. Contrary to popular belief, single-vehicle ran-off-the-road encroachments and accidents are not completely random incidents but occur with a degree of predictability. Although we cannot predict the exact time and place that an accident will occur, the more hazardous locations can be identified (2). The most important highway feature related to encroachment is traffic volume. The number of encroachments is directly related to traffic volume; that is, the larger the traffic volume, the greater the number of encroachments (13). Other traffic and highway features have important influence on encroachments, but they have not been quantified. These include the following (10):

1. Severe highway curvature, downgrade, and inadequate superelevation;
2. High traffic speed;
3. Adverse prevailing environmental conditions;
4. Inadequate signing, lighting, delineation, and site distance;
5. Low skid resistance; and
6. Route discontinuity and lane drops.

Figure 5. Bridge barrier element involved in 350 fatal accidents.

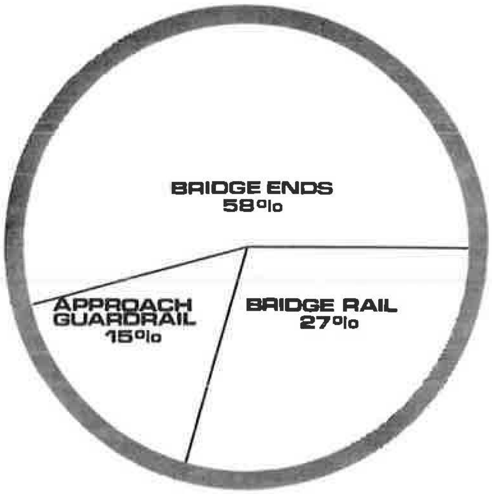


Figure 6. Example of approach guardrail not anchored to bridge railing.

Figure 7. Fatal accidents due to obsolete approach guardrail and bridge railing.

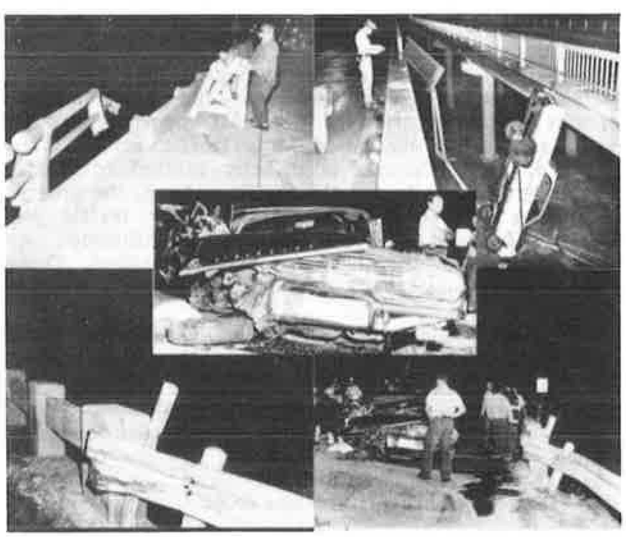
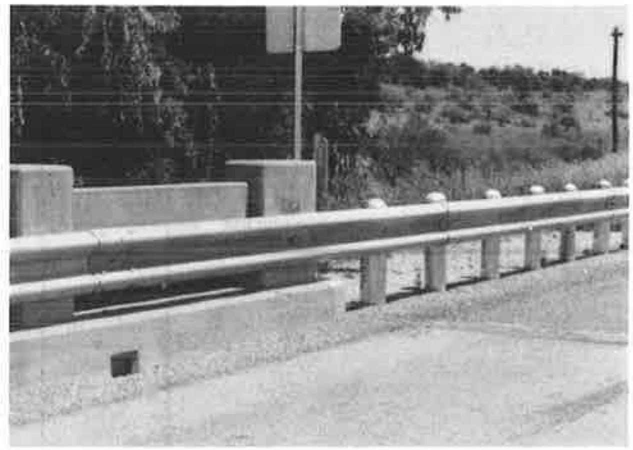


Figure 8. Example of good practice for the transition from approach guardrail to bridge barrier.



Even though it is unknown how each of these features, acting alone or in combination, specifically affect encroachments, general cause-effect relationships have been noted. At highway sites where one or more features are present, the rates of encroachments are atypically high.

EXTENT OF UPGRADING

The extent of safety upgrading can also be adapted to suit the degree and severity of hazard. A range of options that are available include the following:

1. Replace functionally obsolete bridge (most costly),
2. Replace obsolete bridge barrier,
3. Upgrade existing bridge barrier or approach railing,
4. Improve signing and delineation, and
5. Do nothing (least costly).

Based on traffic volume alone, the replacement of a heavily traveled obsolete bridge may be justified economically. On the other hand, it may be justified on a cost-effectiveness basis to do nothing to a functionally obsolete bridge that carries only a few vehicles per day.

SUMMARY

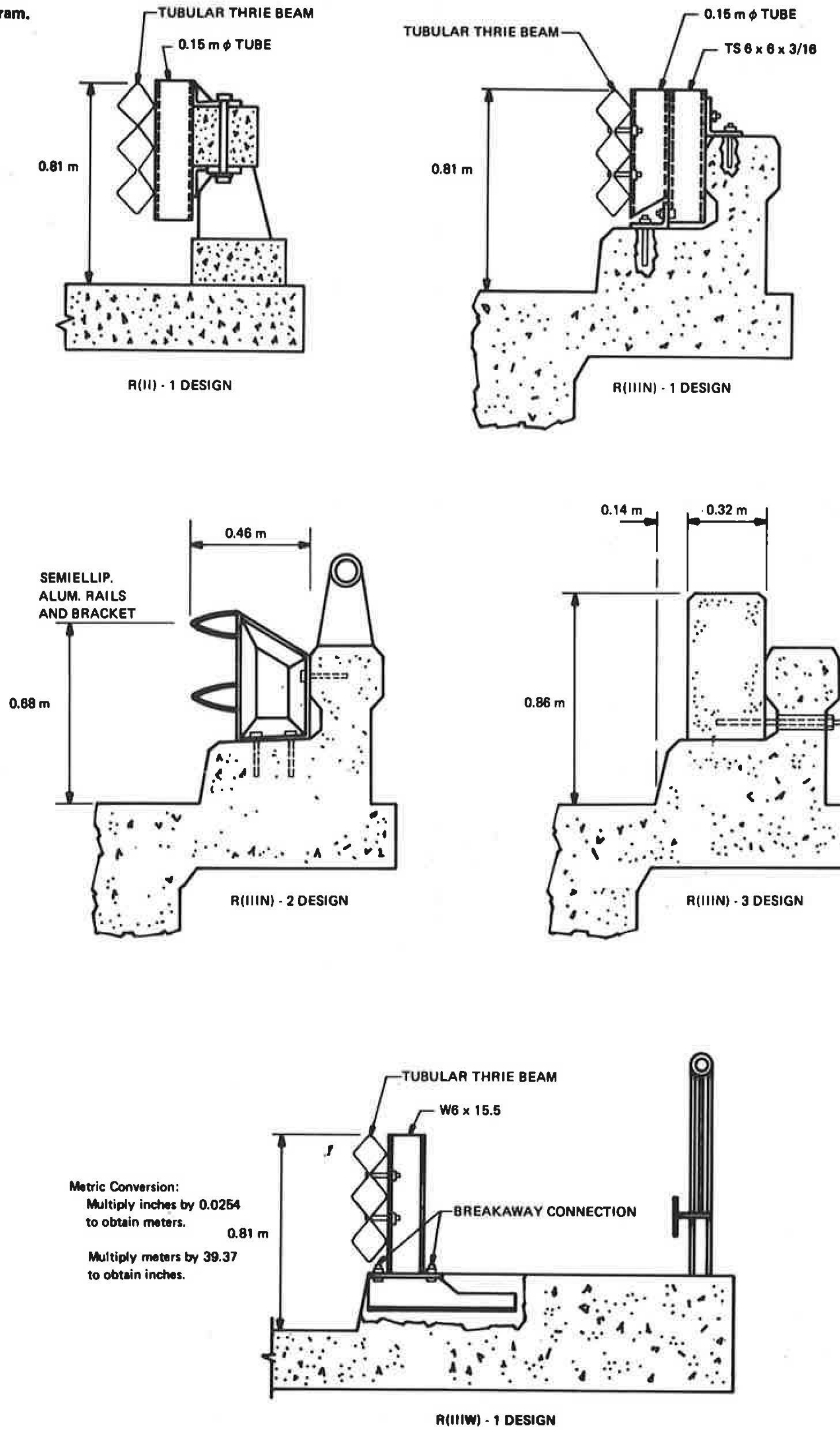
Safety is a societal judgment of the acceptability of risk. Safety is an ever-changing value judgment that balances the degree of risk against costs to reduce these risks. Risk is measured by the probability of an event and its severity. Uniform safety and risk can be achieved by varying the level-of-design standard to suit local site conditions.

Based on length alone, a bridge is 50 times more hazardous than the roadway. The disproportionately large number of bridge fatal accidents is attributed to the narrowness factor. Widening the bridge will reduce this accident rate. Other traffic control techniques may also reduce this rate. Severity of bridge barrier accidents is attributed to the following obsolete design features:

1. Tombstone terminals,
2. Inadequate or no approach guardrail, and
3. Inadequate bridge railing.

Because of funding limitations, a highway agency must be selective in identifying bridges for upgrading. A bridge should be selected based on accident records. Also, bridges selected should have high traffic volume and encroachment-causing features. Selectivity in the extent of upgrading is important.

Figure 9. Retrofit designs evaluated in program.



In conclusion, bridges represent an important safety problem. Although the solution approach is indicated, it will not be cheap or easily accomplished. It will require a considerable amount of patience, persistence, and good sound engineering work.

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Highway Alignment and Superelevation: Some Design-Speed Misconceptions

JOHN C. HAYWARD

Horizontal alignment and superelevation of curves have an impact on the traffic safety performance of highway sections. Research that relates traffic safety to roadway horizontal alignment has consistently shown that traffic accidents increase with increasingly sharper curves. Sharp curves in segments that otherwise have good alignment tend to surprise drivers and create even more hazardous situations. Consistency in design speeds along significant sections of highways has been advocated by some as a means of controlling the incidence of surprise curves in otherwise gentle alignments. However, design speeds for horizontal curves are a function of the maximum superelevation policies adopted by a design agency. Therefore, a single curve design may be regarded as having different design speeds by agencies that have different maximum superelevation policies. For this reason, the use of design-speed criteria for identifying potentially hazardous horizontal alignments would not appear to be appropriate. This finding is discussed in relation to the resurfacing, restoration, and rehabilitation projects proposed by the American Association of State Highway and Transportation Officials.

In recent years the highway design community has focused its attention on the development of geometric standards for the rehabilitation of existing highways. One important element in the improvement of roadways is the elimination of horizontal curves that, because of their geometric design, have created hazardous situations for the motorist. This paper outlines some of the research that has related safety to horizontal alignment of roadways and examines differences in current design policies of the states. Emphasis is placed on nonfreeway locations so that the resultant material would be relevant to resurfacing, restoration, and rehabilitation (3R) improvements of two-lane rural roadways.

The literature relative to alignment and superelevation shows that the highway research community is in basic agreement that roadway alignment is a key factor in unsafe vehicular operation. Increasing degrees of curvature cause more accidents. Single sharp curves in a highway system, generally characterized by long tangents and flat curves, create hazardous situations. Horizontal curvature may have the highest correlation with accident rates of major geometric characteristics for two-lane rural roads.

An examination of design practices in various states indicated a substantial difference in the manner in which horizontal alignment and superelevation is provided for the driver. Some states employ transition or spiral curves normally in design, others do not. Treatment of superelevation runout or transition also varies from state to state.

Perhaps the most significant variation in state design practices, however, is the assumption employed by various states regarding the maximum allowable superelevation on curves. This assumption has a direct bearing on the meaning of the term design speed for a curve and hence could have significant impact on any national 3R program for highways.

The following pages support the contention that highway alignment is related to safety performance. The issue of design speeds and 3R improvements will be touched on and some problems pointed out with respect to current definitions of design speed for specific curves. A review of basic highway curve formulas will be given and an analysis of how design speed changes with respect to maximum superelevation will be presented. Finally, some conclusions will be offered that relate 3R improvements to some general misconceptions about what design speeds really mean and how they relate to the dynamics of vehicles on curves.

SAFETY RESEARCH AND HIGHWAY ALIGNMENT

Research into the relationship between accident rates and highway curvature has been consistent in the finding that increasing curvature causes increased accident rates. Several studies have been summarized by Leisch (1) in the chart reproduced as Figure 1. A recent National Cooperative Highway Research Program (NCHRP) report by Jorgensen (2), which used information developed by Coburn (3), arrives at identical conclusions for rural roads. An extensive study by Taragin (4) on driver performance on horizontal curves noted that the sharper the curve, the closer drivers will operate their vehicles at speeds that approach the safe speed. Therefore, the margin for error for sharper curves is less than for flat curves. These findings led to the adoption of American Association of State Highway Officials (AASHO) policies as early as 1954 that specify that (5, p. 79) "Every effort should be made to use as high a design speed as practicable to attain a desired degree of safety, mobility, and efficiency."

The research literature offers some evidence that the frequency of curves within a roadway section also affects

Figure 1. Accident rate related to horizontal curvature.

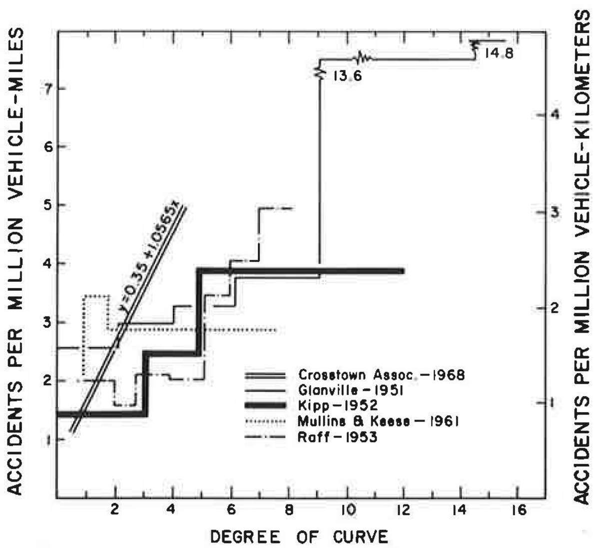
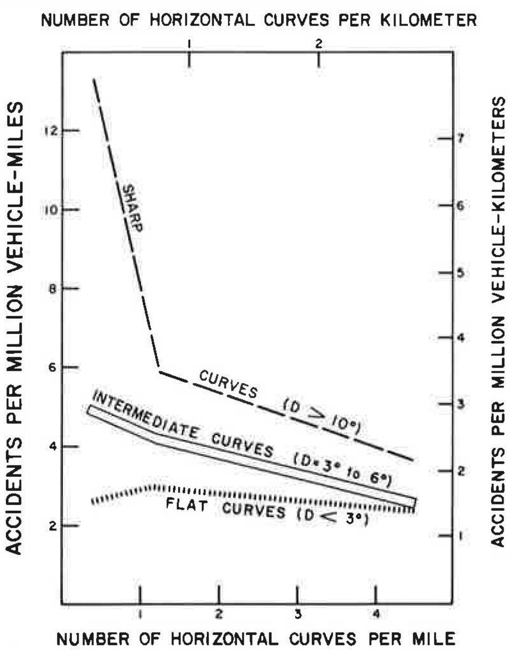


Figure 2. Accident rate related to curve frequency.



accident rates. The work presented by Baldwin (6) and summarized in Figure 2 demonstrates that sharp curves at infrequent intervals are much more dangerous than frequent applications of the same class of curves. Raff (7) has supported this basic finding in his study of Interstate system accidents.

ALIGNMENT AND DESIGN SPEED

Specific decisions on highway alignment (degree of horizontal curvature and superelevation) are based on assumptions about design speed. Therefore, it is useful to review the definition of design speed and its subsequent application to curve design.

AASHO defines design speed as follows (8, p. 283), "Design speed is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern."

This definition differs from that offered in a 1940 AASHO publication (9, p. 8), which stated that

> The assumed design speed of a highway is considered to be the maximum approximately uniform speed which probably will be adopted by the faster group of drivers but not, necessarily, by the small percentage of reckless ones.

The proposed rules issued by the Federal Highway Administration (FHWA) on August 23, 1978, that govern 3R design standards offer some additional information on design speed with the following sentence (10): "The purpose of a design speed is to correlate those physical features of a highway that influence vehicle operation."

The choice of what design speed to use for a highway section is a function of the type of highway and the terrain. This basic assumption for the entire highway section is used in the design of most highway elements to achieve a balanced design. The alignment features of a roadway (i.e., horizontal curvature and superelevation) are directly related to (and change significantly with) the design speed.

Essentially, the design speed, when combined with a maximum allowable superelevation, fixes the maximum degree of curvature that may be employed in a highway section. The maximum degree of curvature employed in a highway section has a profound effect on section costs and, as noted in the research literature, a significant impact on operating safety. It seems obvious that any major rehabilitation program for a length of highway would be initiated by relating inconsistencies in design speed to traffic accidents in an attempt to provide a balanced design and improve safety.

3R IMPROVEMENTS AND HIGHWAY ALIGNMENT AND SUPERELEVATION

The American Association of State Highway and Transportation Officials (AASHTO) 3R guide (11) recognizes the need for improvements to highway alignment and superelevation. A primary objective listed in the guide is the improvement of superelevation on curves. This manual also classifies the improvement of an isolated curve as a 3R project that could result in considerable traffic operational improvement. The guide further states that (11) "Every attempt should be made to maintain a uniformly safe running speed for a significant segment of highway."

Rules proposed by FHWA echo the AASHTO guidelines on this point. In addition, FHWA-proposed rules suggest the collection of field data on average running speeds to determine how the existing or proposed design speed relates to actual operations. The rules note that (10) "Application of an ideal design speed that has no relationship to the speeds actually found on an existing highway would be arbitrary."

DESIGN-SPEED PROFILES

One way to identify problem alignments within a highway system would be to display the design speed of each component graphically and look for discontinuities in the design-speed curve.

On the surface, such a procedure would seem to be a quick way to spot problem areas in existing design by using readily available information (design drawings). For analysis of horizontal curvature, the analyst takes curve parameters (degree of curvature and superelevation) and solves for design speed by using standard curve design tables. The relation of the design speeds of individual highway elements to the entire system ought to give some indication as to where drivers are surprised and consequently have less of a safety margin.

This procedure is suggested in the proposed FHWA rules and some limiting values given as to the permissible disparities between specific highway components (curves) and the generally assumed design speed (10). If a difference

Table 1. Comparison of design speeds for identical curves under different e_{max} values.

Degree of Curvature	Actual Superelevation (m/m)	Design Speed (km/h)			
		e_{max} = 0.06	e_{max} = 0.08	e_{max} = 0.10	e_{max} = 0.12
3	0.05	89	79	72	69
8	0.05	56	47	45	43
10	0.05	50	43	42	40
3	0.06	113	89	80	77
8	0.06	72	55	48	47
10	0.06	64	48	45	43
3	0.08	NA	121	98	92
8	0.08	NA	79	63	56
10	0.08	NA	68	55	51

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

of less than 24 km/h (15 mph) exists between the calculated design speed for a curve and the designated design speed of adjacent sections, the curve ought to be signed and marked accordingly. If a difference of more than 24 km/h exists for horizontal curves, corrective work should be undertaken.

Problems with the Design-Speed Concept for Horizontal Curves

The design speed for a curve is perceived by most designers to represent the maximum speed of safe vehicular operation. This is probably true because most textbooks or geometric guidelines begin their discussion of horizontal alignment with a presentation of the basic formula that governs the dynamics of vehicles on curves:

$$e + f = V^2/127.5R \qquad (1)$$

where

- e = rate of roadway superelevation (m/m),
- f = side-friction factor,
- V = vehicle speed (km/h), and
- R = radius of the curve (m).

From this basic formula and assumptions regarding safe side-friction factors and maximum superelevation rates, tables of acceptable curve geometrics have been developed and adopted for use in highway designs. For a given design speed and maximum allowable superelevation, the designer can easily determine the appropriate range of curve radius (or degree of curve) and the superelevation rate. One would normally assume that the geometrics of these curves are related in some consistent manner to the initial formula that governs the dynamics of vehicles on curves.

The problem is that they are not consistently related. Design speeds on curves are not representative of the maximum permissible safe speed as expressed by the formula. In fact, identical curves located in two different states can have different design speeds.

Put more precisely, a curve with a fixed degree of curvature and superelevation rate can be considered to have different design speeds, depending on the state criteria that have been used to design the curve.

Curve Design Speeds Differ by State

The theoretical design speed for a given curve geometry is also a function of the maximum superelevation rate permitted in that state. Each state chooses what maximum superelevation rate is appropriate to its particular terrain and condition. Generally, the maximum allowable superelevation is chosen after consideration of the climatic condition of the state. States that have a high incidence of snow and ice conditions typically adopt low maximum superelevation rates. States that have more temperate climates opt for higher rates. A range of current state practice is shown in the following table:

State	Maximum Superelevation Permitted (%)
California	8-12
Florida	10
Illinois	8
Indiana	8
Kentucky	10
New York	8
Ohio	8.3
Pennsylvania	8
Texas	8-12
Washington	10
Wisconsin	8

The maximum superelevation rate has an impact on the curve geometrics because of the manner in which superelevation and side-friction factors interact to keep the vehicle from leaving the curve. The maximum allowable degree of curvature for a specific design speed can be computed by using the maximum allowable superelevation and the maximum side-friction factor. The formula can be expressed as follows:

$$D = 222\,480(e + f)/V^2 \qquad (2)$$

where

- D = the degree of curvature,
- e = superelevation (m/m),
- f = the side-friction factor, and
- V = the design speed (km/h).

The maximum side-friction factor is assumed to vary with speed according to the following:

$$f = 0.19 - 0.000\,62V \qquad (3)$$

where V is speed in km/h. Therefore, to solve for maximum D for a specific design speed (V), one uses the following expression:

$$D_{max} = 222\,480(e_{max} + 0.19 - 0.000\,62V)/V^2 \qquad (4)$$

The problem of different design speeds for identical curves comes about because of the assumptions employed by AASHO about the relation of e and f for curves below the maximum degree of curvature for that speed, for the geometric design of rural highways (12). The assumption is made that friction factors vary in curvilinear fashion with the degree of curve between the limits of e equal to zero and e_{max}. Therefore, for different e_{max} values, the curve takes on a different shape and hence affects the curve geometry.

Comparisons of Curve Design Speeds

Table 1 illustrates the magnitude of the difference in design speeds derived from constant curve geometry. The design-speed values are taken from the curves presented in the AASHO rural highway policy (12, pp. 163-166).

An examination of this table shows that differences between design speeds are substantial, depending on the maximum superelevation that is assumed. For differences in maximum superelevation of 0.06-0.12 m/m, the design speed varies by a maximum of 35 km/h (22 mph) (see 3° curve, e = 0.06).

As curves get flatter (D becomes smaller), the differences between design speeds become greater. Also, as the actual superelevation increases, the disparity between design speeds becomes greater.

CONCLUSIONS

The following statements serve to sum up this analysis of alignment and superelevation.

1. Highway alignment is definitely a causal factor in highway accidents: Curves surprise drivers. This leads to driver error and accidents. The sharper the curve, the higher the accident rate. Sharp curves in the middle of long segments that do not have speed-impeding environments are the worst curve-related safety problem.

For 3R programs to be effective, the locations that have alignment discontinuities associated with them should be identifiable. This identification might come from an analysis of highway plans, accident statistics, or over-the-road inventory techniques.

2. Design speed for a curve is not a limiting speed that is indicative of the maximum safe operating speed of the curve: The method used by most states to distribute the maximum superelevation throughout the range of intermediate curve radii has weakened the relationship between design speed and the limiting speeds suggested through the laws of physics. Because different states employ differing rates of maximum superelevation, the same curve can have different design-speed values in different states.

3. Tying 3R improvements to design speeds on curves can lead to inequities between states: Because the same curves can have different design speeds, depending on the maximum permitted superelevation, the adoption of a uniform policy for rehabilitation based on design speeds would be inconsistent. States that have lower e_{max} standards will show higher design speeds for a given curve than those states that have higher e_{max} standards.

Therefore, an analysis of the highway system that compares design speeds of curves to adjacent sections and a standard that attempts to improve situations with large disparities would penalize states that have high maximum permitted superelevation. Those states would show higher deviations from a uniform design-speed policy for an identical roadway section simply by virtue of their design policy.

4. Surprise curves and other geometric conditions that lead to improper average running-speed transitions need to be remedied; however, comparisons of design speeds are not the appropriate measures. The disparity between the maximum safe speeds as derived from the standard curve formula and that of the design speed is large. Therefore, comparisons of design speeds are not appropriate. However, some means of determining the impact of individual geometric elements on average vehicular speed performance must be developed and applied.

ACKNOWLEDGMENT

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Effect of Shoulder Width and Condition on Safety: A Critique of Current State of the Art

CHARLES V. ZEGEER AND DAVID D. PERKINS

A critical review was conducted of available studies on the effect of shoulder width and condition on safety. A set of criteria was established for use in evaluating the reliability of the conclusions reported in past studies on this subject. Most studies based conclusions on the analysis results of pre-1955 accident data and only two of them considered the effect of shoulder width on related accident types (run-off-the-road and head-on accidents). Several studies did not control for the effect of intersections and differing roadway alignment (tangent or curved sections) on rural highway accident rates. Wider shoulders were found to be associated with safer conditions in the studies that were judged most reliable. Shoulder stabilization was effective in reducing accident rates on two-lane roads, particularly on identified high-accident sections. Shoulder widening was found to be cost effective on high-accident sections that had shoulder widths less than 1.2 m (4 ft). In particular, sections of rural two-lane roads that had six or more run-off-the-road or head-on accidents per 1.6 kilometer per year were likely to result in benefit/cost ratios greater than one. Shoulder widening was not cost effective, however, for low-volume roads (less than 1000 vehicles/day) that had a low frequency of accidents. Shoulder paving or stabilization is generally desirable from a safety standpoint, although its cost-effectiveness is not well established. Rural winding highway sections and sharp horizontal curves were recommended as the best

candidates for shoulder improvements, particularly those that have a high incidence of run-off-the-road and head-on accidents. Shoulder widths of 1.8-2.7 m (6-9 ft) are recommended for rural, two-lane roads.

Rural highways typically account for a disproportionately high percentage of injury and fatal accidents. Consequently, rural highways present a continual challenge to highway safety engineers who are responsible for the selection of cost-effective highway safety improvements. Countermeasure selection is usually based on past experience and documented results of project evaluations and research studies.

The effect of most rural highway improvements is generally consistent and well documented. For example, deslicking projects reduce wet-weather accidents, lane-widening projects [i.e., to 3.4 or 3.7 m (11 or 12 ft)] reduce run-off-the-road accidents, and removal of fixed roadside obstacles on horizontal curves results in fewer fixed-object accidents. The effect of such highway improvements is generally accepted when consistent results are documented in the literature.

A considerable amount of inconsistency exists in the literature concerning the safety effects of shoulder width and condition. Several major studies conclude that accidents increase with increasing shoulder width for certain conditions. Other studies report inconclusive results or no relationship between shoulder width and safety. Others report that wider shoulders result in a safer roadway in terms of run-off-the-road and other accident types. Some studies conclude that wide shoulders are necessary for recovery by vehicles that run off the edge of the roadway. Others argue that wide shoulders encourage leisure stops that result in rear-end accidents, particularly at night on Interstate routes. Faced with conflicting results from past research, today's safety engineers must decide which conclusions to believe.

The purpose of this study was to critically review and critique many of the research studies related to highway shoulders to obtain a better understanding of the effect of shoulder width and condition on safety. This knowledge will assist the highway safety engineer in making informed decisions regarding the selection of cost-effective, shoulder-related improvements. First, we reviewed current shoulder design standards. Next, a set of criteria was defined for evaluating past studies. These criteria were used to identify the strong points and deficiencies of each study and to evaluate the reliability of study conclusions.

CURRENT SHOULDER-WIDTH STANDARDS

Design standards for shoulder widths on rural highways are addressed in the 1965 American Association of State Highway Officials (AASHO) Blue Book. AASHO recommends 3.7-m (12-ft) lanes with usable 3.0-m (10-ft) shoulders on two-lane roads. However, because of high construction costs, 3.0-m shoulders are not always feasible, so minimum and desirable standards were developed by AASHO for various ranges of traffic volumes. For very low-volume roads [average daily traffic (ADT) of 50-250], a 1.2-m (4-ft) shoulder is the suggested minimum, and the desirable width is 1.8 m (6 ft), as shown in Table 1 (1). Minimum shoulder widths are 1.8 m for ADT of 400-750, 2.4 m (8 ft) for design hourly volume (DHV) of 200-400, and 3.0 m (10 ft) for higher volumes. Desirable shoulder widths are 3.7 m (12 ft) for a DHV greater than 400 (1).

According to AASHO, shoulders should be usable at all times, regardless of weather conditions. Shoulders on high-volume roads should be stabilized or paved whenever possible. Where the side slopes are steeper than a 4:1 ratio, the shoulder should be 0.6-1.8 m (2-6 ft) greater than the dimensions given in Table 1. Whenever possible, full shoulder widths should be carried across bridges to reduce the chance of a vehicle hitting the bridge structure (1).

Table 1. Design widths for shoulders on two-lane rural highways.

		Usable Shoulder Width (m)	
Current ADT	Design Hourly Volume	Minimum	Desirable
50-250		1.2	1.8
250-400		1.2	2.4
400-750	100-200	1.8	3.0
	200-400	2.4	3.0
	>400	3.0	3.7

Note: 1 m = 3.28 ft.

CRITERIA FOR EVALUATING PAST STUDIES

To evaluate past studies on shoulder improvements, criteria were defined and used as a basis for determining the reliability and validity of the conclusions of each study. The criteria established include the following:

1. Type of data analysis and statistical testing performed,
2. Reliability of the accident data sample,
3. Characteristics of roadway sections used, and
4. Accident types used in the study.

If a study fails to satisfy any one of these criteria, serious questions may arise as to the validity of the study's results.

Analyses and Statistical Testing

Two types of analysis were used in past studies to evaluate the relationship between shoulders and traffic accidents. These include the following:

1. Analysis of traffic accidents before and after a change has been made in the highway shoulder and
2. Comparative analysis of traffic accidents for various shoulder-width characteristics.

Both analyses are valid when used properly. An awareness of potential problems and an understanding of the limitations of each analysis technique are essential to proper interpretation of study results.

The before-and-after analysis is used to determine the cause-and-effect relationship between shoulder improvements and accidents. The effect of a shoulder improvement can be assessed by comparing accident data before and after the improvement only when the shoulder improvement is the sole physical change on the highway section. The change in accident experience can then be attributed to the improvement, all else being approximately equal.

There are several potential problems with the use of before-and-after analysis. For example, accident data at a location are random and several years of both before and after data are necessary to increase the reliability of the accident sample. However, as the analysis period is increased, other factors may be introduced that influence accidents (e.g., changes in traffic volumes and traffic mix). Also, to obtain an adequate sample of highway distance for which the only improvement is a shoulder improvement is very difficult, since improvement projects often include other simultaneous improvements, such as delineation, skid treatment, realignment, and improved drainage—all of which affect accident experience. Finally, some construction-related accidents may result from lane closures or traffic stoppages and should be omitted from the analysis.

Other limitations of the before-and-after technique are that accident experience may change because of (a) random fluctuation in accident experience, (b) a change in the character of the highway system other than the shoulder improvement, or (c) the regression-to-the-mean phenomenon. Properly designed analysis techniques can minimize the adverse effects of these limitations. Problems associated with chance variations in accidents may be minimized by performing statistical tests of significance on the observed change in accident experience between the before and after periods. Statistical tests such as Poisson test and chi-square test (2) may be used to assess whether the accident change is a result of chance or some specific change in the environment (assumed to be the shoulder improvement) at a selected level of statistical confidence. The confounding effect of changes to the highway system (other than the improvement) and regression to the mean can be miminized by the use of control sections when practical.

The second and more common type of analysis involves selection of a large sample of highway sections where both geometric and accident data are known. We refer to such an analysis as a comparative analysis. Sections that have similar geometrics are grouped together and accident data are compared for different shoulder widths and conditions. One advantage of this method is that a large data base may be used without relying on improved sections. One or two years of accident data are usually adequate if a large number of similar sections are combined into each group. Also, volume changes may be minimal, since a shorter analysis period is required than for a before-and-after analysis.

Despite these advantages, there are several disadvantages of this type of analysis. For example, no two highway sections are exactly alike and, therefore, grouping sections of similar characteristics obviously does not consider all possible differences in geometrics or volumes. Another problem involves the difficulty of handling extensive geometric and accident information for large distance samples.

Some of the studies used regression techniques to perform comparative analyses. This involves developing linear or nonlinear relations by using accident measures as dependent variables and shoulder characteristics (with other variables) as independent variables. However, as in the before-and-after approach, statistical tests must be performed to determine the significance of the observed relationship. This includes testing both the slope of the relationship and the correlation between the dependent and independent variables for statistical significance. Researchers have also used a variety of other analysis approaches, which range from correlation techniques to analysis of variance. Each approach must be accompanied by appropriate statistical testing techniques to facilitate interpretation of results and ensure validity of findings.

Neither analysis method is perfect; however, either can produce reliable results if the limitations and potential problems of each method are fully understood and steps are taken to minimize these shortcomings.

Reliability of Accident Data Sample

The reliability of the accident data sample is important in any safety study. Two of the major questions to be answered on data reliability are (a) How current are the accident data? and (b) What is the sample size used? Outdated accident data can give results that may not be totally appropriate when applied under current roadway and traffic conditions. For example, several major shoulder-related safety studies were conducted in the 1950s. Studies that are 25-30 years old may not reflect current driver attitudes, vehicle characteristics, highway speeds, average traffic volumes, delineation characteristics, gasoline availability, or traffic mix. Also, many roadway safety standards have changed considerably in recent years. Such changes have been made in the design of guardrails, shoulder slopes, lane widths, clear zones, pavement striping, highway signing, placement of fixed objects, and other highway features. Older study results may still be valid today in many cases; however, more credibility can be commanded by a properly designed study if recent accident data are used.

The size of the data sample is also important to ensure reliable conclusions. A larger sample size is generally possible for a comparative analysis than for a before-and-after study. However, with either analysis technique, several hundred kilometers may be considered a minimum to ensure consideration of a variety of different highway conditions. One must also remember that sections that had no accidents should not be arbitrarily excluded from an analysis, since this could lead to biased and erroneous results.

Highway Data Characteristics

The reliability of analysis results is improved if sample highway sections are selected that have basic similarities, such as number of lanes, section length, and highway classification. For example, sections of two-lane and four-lane roads should not be combined for analysis purposes. Traffic operations are considerably different on two-lane roads than on four-lane roads, so the effect of shoulder improvements on safety could be different. Comparison of unequal lengths of highway segments could also cause instability in data summaries. A more desirable procedure would be to use sections of equal length, where all geometric and volume characteristics can be recorded separately for each section.

Care should also be exercised in choosing the type of sections. Sections that contain major intersections should not be included, because intersection accident data can distort the effect of shoulders on safety. For example, higher-class roads normally contain wider shoulders and more major intersections than lower-class roads. An analysis of accidents might initially indicate that roads with wider shoulders (higher-class roads) result in higher total accident rates than roads with narrow shoulders. The true explanation may be that the wide-shouldered roads are associated with higher rates of intersection-related accidents and higher traffic volumes.

Highway sections that contain sudden changes in geometrics (transition sections) should also be omitted from the data base because they may adversely influence the accident data. Such transitions include abrupt changes in lane width, shoulder width, median width, pavement type, clear recovery area, area type (suburban, rural, or urban), traffic volume, and the number of lanes (lane drop). Sample sections should generally be homogeneous, so the corresponding accident data for each highway segment represent a single combination of traffic and highway conditions.

The data set should also include representative characteristics of rural roads, since urban streets normally use curbs and gutters instead of shoulders. Representative sections should not include only tangent sections. This is because shoulders are logically more useful for vehicle recovery after the vehicles leave the highway, and vehicles are more likely to leave the highway on curves than on tangents. Thus, the use of only tangent sections will not represent the full benefit of shoulders on rural highway sections.

The purpose of shoulder improvements can largely determine the resulting safety benefits that will occur. If a before-and-after analysis is used, the results may vary greatly, depending on whether or not the shoulder improvement is in response to an observed safety deficiency. If the shoulder is widened on a section primarily for operational reasons and few or no related accidents occur annually before the improvement, then the improvement is not likely to be a cost-effective means of accident reduction. If, however, shoulders are widened in response to a disproportionately high number or severity of run-off-the-road accidents, then the improvement will probably result in an acceptable safety benefit.

Selection of Accident Types

One of the major problems with past studies is that they fail to use accident types that are related to shoulder width and condition. For example, logic dictates that shoulder

Table 2. Summary of information for various major studies.

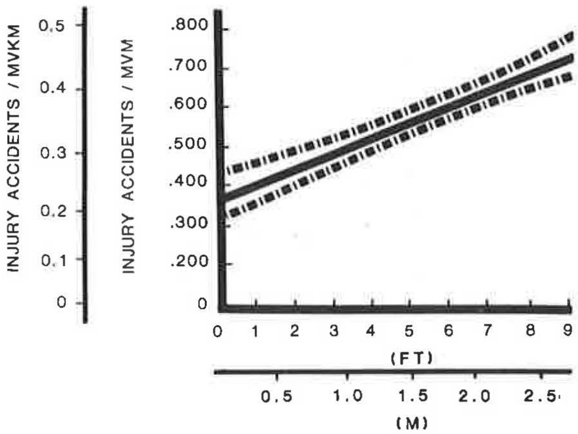
Study	State	Analysis Period	Sample Size	Controlled Variables	Analysis and Statistical Testing	Accident Variables
Billion and Stohner (3)	NY	1947-1955	1753 accidents	Number of lanes, pavement width, speed restrictions, location, intersections excluded, sections with roadside structures excluded, alignment, grade, and shoulder width	Comparison of average accident expectancy with actual accident experience for combinations of shoulder width, grade, and alignment. Chi-square used to test significance of actual accident experience	Ratio of percentage of total accidents to percentage of total travel
Stohner (4)	NY	1952	9299 accidents on 14 075 km	Number of lanes, pavement width, traffic volume, location, and shoulder width	Development of graphical relationship of accidents versus shoulder width by pavement width, no statistical analysis performed	Fatal + injury accidents per million vehicle-km and property-damage accidents per million vehicle-km
Perkins (5)	CT	1951-1954	16 672 accidents	Number of lanes, location, pavement width, shoulder type, and shoulder width	Analysis of trends between accidents and shoulder widths	Total accidents and total accidents per kilometer
Belmont (6)	CA	1948	1333 accidents on 858 km	Number of lanes, pavement type, grades, speed limit, intersections excluded, sections with roadside structures excluded, traffic volume, shoulder type, and shoulder width	Regression analysis, F-test used to test statistical difference of developed relationship	Total accidents and total accidents per kilometer
Head and Kaestner (7)	OR	1952-1954	554 km	Number of lanes, shoulder type, sight restrictions, lane width, speed restrictions, alignment, grade, traffic volume, number of driveways, and shoulder width	Statistical significance of slopes and partial correlation coefficient tested	Total accidents per kilometer, injury accidents per kilometer, and property-damage accidents per kilometer
Blensly and Head (8)	OR	1959	557 km	Number of lanes, lane width, sight restrictions, alignment, grades, shoulder type, traffic volume, number of driveways, and shoulder width	Simple and partial correlation techniques and analysis of variance and covariance, F-test used to test statistical differences	Total accidents, injury accidents, and property-damage accidents
Zeeger and Mayes (9)	KY	1976	16 912 accidents on 25 488 km	Number of lanes, lane width, shoulder width, traffic volume, access points per kilometer, and functional classification	Types of accidents found to relate to shoulder width were run-off-the-road and opposite direction, average accident costs were computed for related accidents, accident rates were computed for various shoulder widths for sections of similar geometrics, and calculation of percentage of accident reduction due to wider shoulders	Property damage, total accident rates, all accident severities, and rates of run-off-the-road and opposite direction accidents
Rinde (10)	CA	1964-1974	230 km, 37 projects	Number of lanes, surface width, traffic volume, and shoulder width	Chi-square statistical distribution testing, comparison of accident rates for similar sections, and before-after study	Property damage, injury, and fatality, specific accident types, accidents by movement preceding collison, and total accident rates
Belmont (11)	CA	1951-1952	1122 sections	Number of lanes, surface width, shoulder width, vehicle speed, level tangents, paved shoulders, and lane width	Least-squares fit and confidence levels computed	Injury accident rate

improvements influence run-off-the-road accidents but probably not right-angle accidents. Many past studies have only considered total accidents in the evaluation of shoulder width and condition. For example, consider a rural highway sample that has 1000 accidents/year before shoulder widening, of which 20 percent (200 accidents) involve run-off-the-road accidents. After shoulder widening, suppose traffic volumes and total accidents increase by 10 percent to 1100 accidents/year, but run-off-the-road accidents decrease to 100. Although run-off-the-road accidents decreased by 50 percent, the total accidents and traffic volume each increased by 10 percent. If the run-off-the-road accidents are not considered, the conclusion is made that the total accident rate did not change. Thus, the true effect of shoulder widening on related accidents may go undetected.

RESEARCH FINDINGS: SHOULDER WIDTH VERSUS SAFETY

Past research to investigate the relation between highway shoulder width and traffic accidents has resulted in a variety of conclusions. Some research findings indicate that accidents increase with increasing shoulder width; others conclude that accidents decrease with increasing shoulder widths. Other studies conclude that no detectable relation exists or that relation exists only for certain ranges of traffic volume. This section provides a brief description and critique of selected research publications. The validity of each study was measured against the criteria discussed previously. To facilitate the evaluation of past research efforts, information on several of the research studies is summarized in Table 2.

Figure 1. Injury accident rates for various shoulder widths.



The studies are classified into three general categories:

1. Studies that indicate adverse safety effects of wider shoulders,
2. Studies that indicate unclear or no effects of wider shoulders, and
3. Studies that indicate improved safety effects of wider shoulders.

Such classification schemes are not totally appropriate for several of the studies because some study results give different conclusions for various volume ranges or number of lanes.

Studies That Indicate Adverse Safety Effects of Wider Shoulders

One of the first major research studies that concluded that accidents increase with increasing shoulder width was a 1954 report by Belmont (6) in California. Three ranges of shoulder widths were tested against total accident frequency: shoulders less than 1.8 m (6 ft), 1.8-m shoulders, and shoulders greater than 1.8 m. The study concluded that accident rates were significantly lower with paved 1.8-m shoulders than with wider-paved shoulders for traffic volumes greater than 5000 vehicles/day. Accident data included about 1300 accidents (1948 data) on 858 km (533 miles) of two-lane tangents (6).

A critical review of this study resulted in the following weaknesses:

1. Roadway sample consisted of only tangent sections,
2. Accident types were analyzed without testing related accident types,
3. Accident data are now outdated (more than 30 years old), and
4. Most volume ranges were limited with respect to roadway sample sizes.

The regression equations developed in the study resulted in r^2 values that were quite high (0.82-0.90), and a considerable number of variables were controlled. However, the weaknesses mentioned above limit the reliability of the conclusions.

Another study that reported adverse safety effects of wider shoulders was a 1960 study by Blensly and Head in Oregon (8). Based on simple correlation procedures, the authors concluded that total and property-damage accident frequency increased with increasing shoulder widths for all volume ranges studied. Another analysis approach (partial correlation procedures) resulted in a similar finding in the 2000-2999 ADT range. Analysis of variance and covariance also yielded similar results. The sample included 557 km (346 miles) of rural two-lane tangents.

Although rather sophisticated analyses were performed, several study limitations were observed. For example, only tangent sections were used. Comparisons were performed on only two groups of shoulder widths [1.2 m (4 ft) or less and 2.4 m (8 ft) or greater]. The effects of shoulders between 1.2 and 2.4 m were not reported. Also, no consideration was given to specific, related accident types, and the accident data are now outdated.

A later study by Belmont in 1956 used 1951 and 1952 accident data to develop an equation of the relationship between shoulder widths and injury accident rates as shown in Figure 1. The figure shows that wider shoulders are associated with higher injury accident rates (11).

Several observed study limitations include the following:

1. Use of only straight and level tangent sections,
2. Outdated accident data, and
3. Failure to analyze specific related accident types.

Also, only injury accident rates were used and may result in limited usefulness of study conclusions, since accident severity is usually related to vehicle speeds at the time of impact and may not be a good substitute for shoulder-related accidents.

Studies That Indicate Mixed or No Effects of Wider Shoulders

Several studies reported mixed effects or no effect of shoulder width on accidents. One such study was completed in 1956 by Perkins (5) in Connecticut by using a sample of more than 16 000 accidents for 1951-1954. His analysis considered accident numbers on roads that have pavement widths of 4.3-7.3 m (14-24 ft). Control variables included pavement width, shoulder width and type, number of lanes, and other locational information. No significant relation was found between accident rate and shoulder width for any volume category.

The analysis included a large accident sample but failed to consider several important factors, such as (a) effect of related accident types, (b) influence of volumes on accidents, and (c) other important geometric variables that affect accidents. Also, accident data used are nearly 30 years old.

A study completed in 1956 by Head and Kaestner (7) provided mixed results on the effect of shoulder width on accidents. A sample of 554 km (344 miles) of highway in Oregon that has gravel shoulders was analyzed by means of accident data from 1952 to 1954. The statistical significance of regression coefficients and partial correlation coefficients was tested for total, injury, and property-damage accidents per kilometer for various ADT groups. Accident frequency was found to be unrelated to shoulder width for low-ADT groups (less than 3600). However, for ADT groups of 3600-7500, total accidents were reduced for wider shoulders, as shown in Figure 2 (7).

This study included an extensive statistical analysis and controlled for 10 variables. Intersection accidents were omitted and various accident severities were considered, all of which add credibility to the results of the study. The possible weaknesses of the study were that (a) specific accident types were not considered, (b) the accident data are outdated, and (c) accidents per kilometer were used instead of accidents per million vehicle kilometers.

Studies That Indicate Positive Effects of Wider Shoulders

Several past studies conclude that shoulder widening reduces various types of accidents. One study, by the Institute of Transportation Engineers (formerly Institute of Traffic Engineers), was completed in California in 1955 and showed an accident rate of 213 (accidents per hundred million vehicle kilometers) on roads that have no shoulder and 165 on roads that have shoulders of 2.4 m (8 ft) or more, as shown in Figure 3 (12,13). Details of the study were not

Figure 2. Predicted total accidents from shoulder width and ADT.

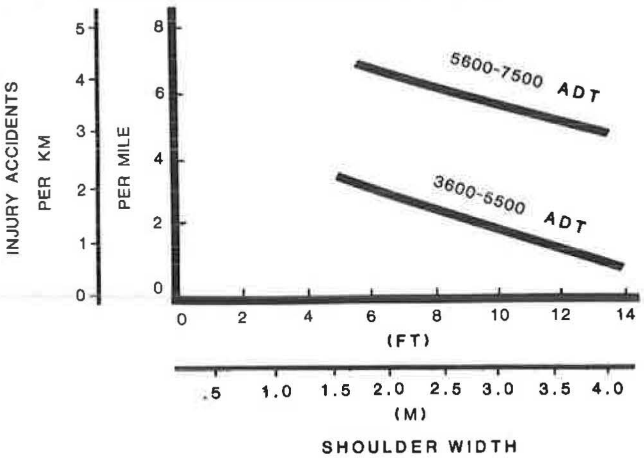
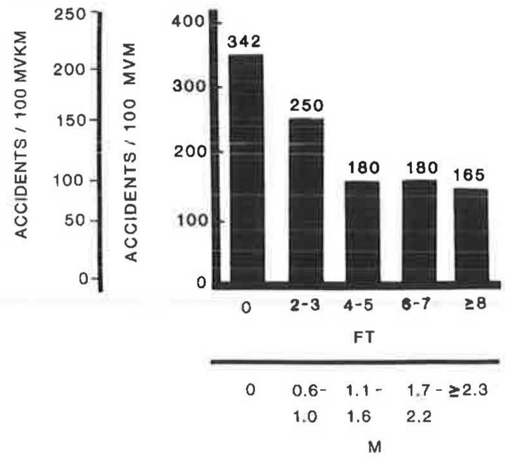


Figure 3. The effect of shoulder width on accidents.



available for review and critique.

A study by Stohner was completed in 1956 for 14 081 km (8746 miles) of two-lane rural highways in New York State that had more than 9000 accidents (1952 data). Rates were used for property-damage and injury plus fatal accidents. Results showed that the accident rate decreases with increasing shoulder width, particularly for property-damage accidents (4).

This study analyzed a large highway-length sample and used a classification scheme for grouping similar highway types for analysis purposes. Although intersection accidents were included in the data, the author noted that the study results were dependent on an equitable distribution of intersections (and other geometric features) in each grouping. Related accidents were not analyzed, and the data base is now quite old.

In 1957, Billion and Stohner published a paper that included 1753 accidents that occurred from 1947 to 1955 in New York State (3). Numerous variables were controlled for rural two-lane highways that have 6.1-m (20-ft) pavements. Shoulders of 1.5-2.1 m (5-7 ft) in width were found to be safer than 0.9- to 1.2-m (3- to 4-ft) shoulders under all conditions of vertical and horizontal alignment. Wide shoulders [2.4 m (8 ft) or more] had a lower accident incidence than did narrow- or medium-width shoulders on poor alignment. No statistically reliable relationships were found for level tangents or grades of more than 5 percent.

The study controlled for several variables. The control for 6.1-m lane width isolated the analysis to sections where wide shoulders are probably more likely to be beneficial. The study does not include analysis of specific related accident types but does include various accident severities. Also, the accident data are now outdated.

One of the more prominent studies on the effect of shoulder width on accidents was a study by Rinde in 1977 (10). The before-and-after technique was used to evaluate 37 shoulder-improvement projects on rural two- and three-lane roads in California, which included 230 km (143 miles) of shoulder widening on existing alignment. The accident rates were reduced by 16 percent for shoulder widening widths of 8.5 m (28 ft) [less than 3000 annual average daily traffic (AADT)], by 35 percent for 9.8 m (32 ft) (less than 5000 AADT), and 29 percent for 12.2 m (40 ft) (more than 5000 AADT). Reductions for 9.8 and 12.2 m were statistically significant at the 95 percent confidence level.

Summaries of accidents were also made for specific accident types as shown in Table 3 (10). Head-on accidents decreased by 50 percent, and hit-object accidents were reduced by approximately 25 percent. Significant accident reduction was not observed for rear-end, overturn, and sideswipe accidents. The total accident rates were higher for wider pavement widths due to the greater number of intersections and driveways on sections that have wide pavements (10). This finding is consistent with studies that report adverse effects of wider shoulders.

Generally, this study presents a very good analysis of the effect of shoulder widening on safety. Some of the strong points of this study include the following:

1. Various related accident types (run-off-the-road and head-on incidents) were analyzed,
2. The analysis used relatively recent accident data (1964-1974 data),
3. Control of several influencing variables, and
4. An analysis to determine which accident reductions were statistically significant due to shoulder widening (at the 95 percent confidence level).

The length of highway was somewhat small [230 km (143 miles)] but, as was discussed earlier, very large samples are usually not possible in a before-and-after analysis, since samples are selected from highways where widening has been completed.

A study of the effect of lane and shoulder widening on safety on rural two-lane roads was performed by Zegeer and Mayes in Kentucky in 1979 (9). A comparative analysis was conducted on more than fifteen thousand 1.6-km (1-mile) sections for which geometric data, traffic information, and accident data (including numbers, severity, types, and rates) were available. The roadway sections were classified by AADT, functional class, the number of access points per kilometer, lane width, and shoulder width. No sections were used that contained major intersections or other transitional characteristics. Sections were compared where the only known difference was in shoulder width. Optimal shoulder widths were found to be 2.1-2.7 m (7-9 ft), and wide shoulders were found to be associated with fewer run-off-the-road and opposite-direction accidents. On wide shoulders, accidents were observed to be 6-21 percent lower for these two accident types, depending on the width of shoulders, as shown in Table 4 (9). The average accident costs (National Safety Council costs in terms of 1976 dollars) for the run-off-the-road and opposite-direction accidents were $5569/accident, compared with $2199 for other accident types.

The strong points of the study included the following:

1. A large sample size was used [more than 24 000 km (15 000 miles) of data and about 17 000 accidents],
2. Specific accident types were used in the analysis (including run-off-the-road and head-on accidents),
3. Numerous important classification variables were controlled (including lane width, access control, ADT groups, functional classification, number of lanes, and area type),

Table 3. Effect of shoulder widening on various accident types.

Collision Type	8.5-m Pavement; AADT<3000			9.8-m Pavement; AADT<5000			12.2-m Pavement; All AADT		
	Before	After	Change (%)	Before	After	Change (%)	Before	After	Change (%)
Head on									
Frequency	3	2		32	19		29	14	
Rate	0.10	0.5	-50	1.04	0.50	-52[a]	0.14	0.6	-57[a]
Rear end									
Frequency	2	2		10	4		80	71	
Rate	0.6	0.5	-17	0.32	0.10	-69[a]	0.37	0.29	-22
Hit object									
Frequency	37	35		34	20		137	112	
Rate	1.19	0.87	-27	1.10	0.52	-53[a]	0.64	0.46	-28[a]
Overturn									
Frequency	13	18		10	18		61	41	
Rate	0.42	0.45	+7	0.32	0.47	+47	0.29	0.17	-41[a]
Sideswipe									
Frequency	1	8		14	14		43	37	
Rate	0.03	0.20	+567[b]	0.45	0.37	-18	0.20	0.15	-25

Notes: 1 m = 3.28 ft.
Accident rates are expressed in terms of accidents per million vehicle miles (1.6 million vehicle kilometers).

[a]Statistically significant decrease.
[b]Statistically significant increase.

Table 4. Percent reduction in related accident types due to wider shoulders.

Shoulder Width (each side): Before Widening (m)	After Widening (m)	Reduction in Run-off-the-Road and Opposite-Direction Accidents (%)[a]
0	0.3-0.9	6
0	1.2-1.8	15
0	2.1-2.7	21
0.3-0.9	1.2-1.8	10
0.3-0.9	2.1-2.7	16
1.2-1.8	2.1-2.7	8

Note: 1 m = 3.28 ft.

[a]Opposite direction includes head-on accidents and sideswipes between vehicles of opposing direction.

4. Recent accident and geometric data were used, and
5. Intersections, nonhomogeneous sections, and transition sections were eliminated.

Expected accident reductions were also used to determine accident benefits for various degrees of shoulder widening.

ECONOMIC ANALYSIS OF SHOULDER WIDENING

The economic effectiveness of shoulder-widening projects is a function of improvement costs and derived accident benefits. If shoulder widening has no effect or a negative effect on safety, the expected benefits will be zero or negative. Therefore, the only studies that might be expected to include a meaningful economic analysis are those where shoulder widening was found to improve safety.

One study of the cost-effectiveness of various accident countermeasures was the 1976 National Highway Safety Needs report by the U.S. Department of Transportation (14). In this report, 37 types of safety improvements were listed in priority order by cost-effectiveness, as given in Table 5 (14). For each improvement type, the corresponding fatalities forestalled were given with corresponding improvement costs and dollars per fatality forestalled.

The most cost-effective improvement was found to be mandatory safety-belt usage, which would only cost an estimated \$500/fatality forestalled. The least cost-effective project was improvement of the roadway alignment and gradient, at a cost of \$7.7 million/fatality forestalled. Paving or stabilizing shoulders was found to be next to last in terms of cost-effectiveness, at \$5.8 million/fatality forestalled. Based on this study, shoulder improvements do not appear to be a cost-effective improvement in terms of reducing traffic fatalities per dollar spent (14).

The information from this study is based on nationwide estimates; however, only fatalities were included as benefit items. Shoulder improvements were apparently not found to have much effect on fatalities in this study. Because of the rare, random nature of fatal accidents, such an accident sample is probably not the most desirable for comparing the relative merits of various improvement types. Further, information was not available concerning the author's assumptions to adequately evaluate the study results.

Very different results were found in another study of safety benefits from improvements by Hall in 1978 (15). A total of 23 different improvement types were ranked from best to worst by benefit/cost ratio, as shown in Table 6 (15). The top-priority improvement was shoulder widening or improvement, which had a benefit/cost ratio of 28.83. The least cost-effective project was bridge widening, which had a benefit/cost ratio of 0.41. The computed annual reduction in accidents for shoulder widening or improvement was 29 percent for all accidents, 20 percent for injuries, and 41 percent for fatalities.

Although the details of the data were not readily available, the shoulder-improvement projects evaluated for this study were possibly high-accident sections before improvement, since benefit/cost ratios were of such a high magnitude. Such high benefit/cost ratios may not be possible for shoulder widening on a random sample of highway sections. However, this study illustrates that shoulder improvements can be very cost effective, dependingl8n the sections selected for widening.

A third study by Zegeer and Mayes in Kentucky in 1980 included an economic analysis related to various shoulder widths (9). Costs for shoulder widening were computed based on a large number of past statewide construction costs and adjusted to 1976 dollars. For every 0.6 m (2 ft) of widening on each side of the road, average costs were nearly \$24 000/km (\$38 000/mile). Costs for widening shoulders by 1.8 m (6 ft) (each side of road) were found to be about \$56 000/km (\$90 000/mile). All costs were itemized and represent average values for the generally rolling and hilly terrain found in Kentucky.

As discussed earlier, wider shoulders were found to be associated with from 6 to 21 percent lower rates for run-off-the-road and opposite-direction accidents (depending on amount of widening). This information was used to compute expected benefit/cost ratios from shoulder widening. The expected benefit/cost ratios for such improvements were a function of annual number of related (run-off-the-road and opposite-direction) accidents. Plots were made of benefit/cost ratios for shoulder widening projects that have from 1 to 20 such accidents per year [Figure 4 (9)]. For example, for a 1.6-km (1-mile) section of road that has 10 accidents/year, the widening of 0.6-m (2-ft) shoulders to 1.5 m (5 ft) (each side of road) would be expected to result in a benefit/cost ratio of about 1.8. Benefit/cost ratios of greater than 1.0 were expected for the widening of sections that have narrow shoulders and 6 or more related accidents per 1.6 km per year. The magnitude of such benefit/cost ratios appears to be quite reasonable for high-accident sections, and the expected economic effectiveness of shoulder improvements from this study was found to lie between the results of the Highway Needs study (14) and the Hall study (15).

SHOULDER STABILIZATION AND SAFETY

The effect of shoulder stabilization on safety has been addressed in several studies, with somewhat different results. Accident data were collected before and after shoulder stabilization in Ohio and Oregon, as reported by

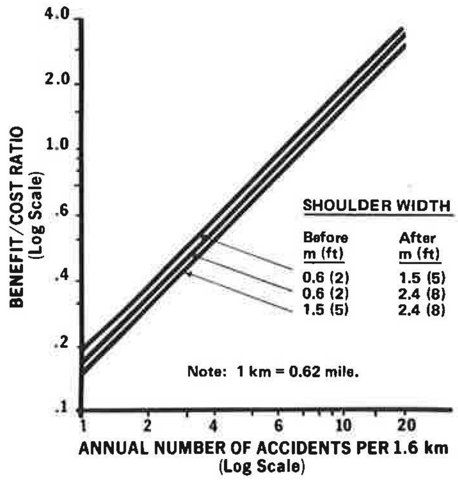
Table 5. Ranking of countermeasures by decreasing cost-effectiveness in present-value dollars—10-year total.

Countermeasure	Fatalities Forestalled	Cost ($000 000s)	Dollars per Fatality Forestalled
Mandatory safety belt usage	89 000	45.0	506
Highway construction and maintenance practices	459	9.2	20 000
Upgrade bicycle and pedestrian safety curriculum offerings	649	13.2	20 400
Nationwide 88-km/h (55-mph) speed limit	31 900	676.0	21 200
Driver improvement schools	2 470	53.0	21 400
Regulatory and warning signs	3 670	125.0	34 000
Guardrail	3 160	108.0	34 100
Pedestrian safety information and education	490	18.0	36 800
Skid resistance	3 740	158.0	42 200
Bridge rails and parapets	1 520	69.8	46 000
Wrong-way entry avoidance techniques	779	38.5	49 400
Driver improvement schools for young offenders	692	36.3	52 500
Motorcycle rider safety helmets	1 150	61.2	53 300
Motorcycle lights-on practice	65	5.2	80 600
Impact-absorbing roadside safety devices	6 780	735.0	108 000
Breakaway sign and lighting supports	3 250	379.0	116 000
Selective traffic enforcement	7 560	1010.0	133 000
Combined alcohol safety action countermeasures	13 000	2130.0	164 000
Citizen assistance of crash victims	3 750	784.0	209 000
Median barriers	529	121.0	228 000
Pedestrian and bicycle visibility enhancement	1 440	332.0	230 000
Tire and braking system safety critical inspection, selective	4 591	1150.0	251 000
Warning letters to problem drivers	192	50.5	263 000
Clear roadside recovery area	533	151.0	284 000
Upgrade education and training for beginning drivers	3 050	1170.0	385 000
Intersection sight distance	468	196.0	420 000
Combined emergency medical countermeasures	8 000	4300.0	538 000
Upgrade traffic signals and systems	3 400	2080.0	610 000
Roadway lighting	759	710.0	936 000
Traffic channelization	645	1080.0	1 680 000
Periodic motor vehicle inspection, current practice	1 840	3890.0	2 120 000
Pavement markings and delineators	237	639.0	2 700 000
Selective access control for safety	1 300	3780.0	2 910 000
Bridge widening	1 330	4600.0	3 460 000
Railroad-highway grade crossing protection, automatic gates excluded	276	974.0	3 530 000
Paved or stabilized shoulders	928	5380.0	5 800 000
Roadway alignment and gradient	590	4530.0	7 680 000

Table 6. Safety benefits of improvements.

Improvement	Annual Reduction (%)			Benefit/Cost Ratio
	Accidents	Injuries	Fatalities	
Shoulder widening or improvement	29	20	41	28.83
Installation of striping or delineators	13	20	46	26.49
Skid treatment and grooving	48	30	74	20.12
Installation or upgrading of traffic signs	23	33	27	15.03
Signing or marking	0	42	35	14.94
Installation or improvement of median barrier	3	6	91	13.73
Localized lighting installation	9	9	73	13.24
Installation or improvement of road edge guardrail	13	15	59	10.97
Flashing lights replacing signs only, railroad crossing	94	93	99	9.41
Signs and striping combination	24	26	27	8.60
Breakaway signs or lighting supports	35	44	100	7.25
Traffic signals installed or improved	18	32	49	6.36
Skid treatment and overlay	17	27	30	6.09
Automatic gates replacing signs only	99	99	100	5.44
Channelization, including left-turn bays	23	29	65	3.94
Pavement widening, no lanes added	25	38	87	3.68
Sight distance improved	31	38	36	2.97
Traffic signals installed or improved and channelization, including left-turn bays	31	35	50	1.78
Automatic gates replacing active devices	81	75	96	1.13
Horizontal alignment changes (except to eliminate highway grade crossing) and vertical alignment changes	21	32	69	0.91
Replacement of bridge or other major structures	44	60	47	0.90
Lanes added without new median	17	11	31	0.80
Widening existing bridge or other major structures	65	74	33	0.41

Figure 4. Expected benefit/cost ratios from widening 0.6-m shoulders.



Jorgensen (16). Stabilization of shoulders in Oregon was conducted as routine improvements on highways that have low accident rates, and accidents were found to actually increase on two-lane roads (91 percent) and also on roads that have more than two lanes (52 percent). The number of injuries and fatalities also increased, although accident variances were quite high (16).

In Ohio, the results were quite different for two-lane roads. This was due primarily to the fact that shoulders were stabilized on sections that had high numbers of accidents, high accident rates, and high percentages of run-off-the-road and head-on accidents. Accidents were reduced by 38 percent, and there was a 46 percent reduction in injuries and fatalities (16).

Based on the results of data from Ohio and Oregon, the effectiveness of a shoulder stabilization project (or perhaps any improvement project) depends on the need for improvement from a safety standpoint. Reductions in accidents are not likely to result when such improvements are implemented on sections that had few or no accidents before improvement. When stabilization projects were selected where the greatest needs existed (as in the Ohio sites), then a reduction in accidents and injuries is very likely.

Another study was conducted by Heimback and others in North Carolina in 1974 (which used 1966 to 1969 accident data) to investigate the cost-effectiveness of paved shoulders on rural primary highways (17). Accident experience was compared between highway sections that were similar in all respects, except for the presence or absence of a paved shoulder. A sample of 3054 homogeneous roadway sections on rural, two-lane roadways was used. Results showed that paved shoulders of 0.9-1.2 m (3-4 ft) were the safest. Shoulder paving was found to sometimes be cost effective (benefit/cost ratio of 1.0 or greater) on two-lane roads but not on four-lane roads. The study assumed paving costs of \$1200-\$8800/km (\$2000-\$14 000/mile) (both sides of the road), service lives of 7-21 years, an economic rate of return of 6-12 percent/year, and a traffic growth rate of 5-8 percent/year.

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study was to determine the effect of shoulder width and condition on highway safety through a critique of past research studies. A set of criteria was established for use in evaluating each study in terms of reliability and validity. Studies were classified according to their general findings of the effect of shoulder width on accidents. Three studies were evaluated where wider shoulders were associated with increased accidents. These studies dealt primarily with tangent sections and the results should not be generalized for all alignments.

Numerous studies were found where accidents were reduced due to wider shoulders, particularly for moderate- to high-volume sections. Wider shoulders were found to reduce run-off-the-road and head-on accidents considerably. Wider shoulders were generally found to be effective on curves and winding sections.

Shoulder widening was found to be cost effective for sections identified as high-accident sections but probably would not be cost effective for random shoulder-widening projects. Shoulder stabilization was also found to reduce accidents where shoulders were stabilized for safety reasons.

Based on a critique of numerous research studies related to shoulder width and condition, the following recommendations were made.

1. Shoulder-widening projects should not be selected randomly but should be based primarily on the incidence of run-off-the-road and head-on accidents or on the presence of obvious roadway safety problems. Widening should be given more consideration on moderate- and high-volume roads and where related accident numbers are abnormally high.
2. Higher priorities for shoulder widening should be given to horizontal curves and winding sections than to straight, level tangent sections.
3. The potential benefits and costs for each shoulder-widening project should be carefully estimated to select projects that have the greatest potential cost-effectiveness.
4. On rural two-lane roads, the optimal shoulder widths are 1.8-2.7 m (6-9 ft).
5. If cost-effectiveness is of primary concern, the best candidate sections for shoulder widening are those rural, two-lane roads that have shoulder widths less than 0.9 m (3 ft) and six or more related accidents (run-off-the-road or head-on) per 1.6 km per year.
6. Shoulder paving or stabilization is generally desirable from a safety standpoint if conducted properly. Locations that have unstabilized shoulders and a history of shoulder-related accidents should be considered for paving or stabilization.

Not all accident-related research studies can be taken at face value. Some may contain unreliable data or questionable analysis techniques. The four criteria developed in this paper (as well as other appropriate criteria) may be useful in the review of all types of safety-related studies.

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Design of Left-Turn Lanes for Priority Intersections

JOE LEE AND THOMAS MULINAZZI

There is general agreement that a left-turn lane should be warranted on a benefit-cost basis. However, existing documents do not provide accurate techniques for the prediction of the two items that are needed for such an approach—the reduction of delay and the length of the left-turn lane. This study shows that the problem can be solved by using the results of two simulation models. These two models attempt to duplicate the traffic of an uncontrolled approach at a two-lane by two-lane priority intersection. A priority intersection is an intersection at which only the two minor approaches are controlled by stop or yield signs—in other words, the major flow has been assigned priority. One model represents a without-left-turn condition and the other represents a with-left-turn condition. Design charts and tables were produced from these models. These charts and tables are presented in this paper to give the user a systematized guide to design problems for the left-turn lane. Application of the study results are intended for use in Kansas and are limited to a two-lane priority intersection. Although the approach and methodologies reported in the study are considered applicable to other locations and for other purposes, users are cautioned to observe the limits of the study results.

A priority intersection is an intersection at which only the two minor approaches are controlled by stop or yield signs. In other words, it is an intersection at which the major flow is assigned priority. Highway engineers involved with the design of left-turn lanes for priority intersections are confronted by two major design consideration issues. The first issue is to determine the conditions (i.e., approach volumes, left-turn percentages, and accidents) under which a left-turn lane is warranted. The second issue is to determine the appropriate length of the left-turn lane. The questions involved in these two issues are complex because of the randomness with which vehicles arrive at an intersection to make left turns and the incidental number of vehicles that turn left at one time when a left-turn lane is provided. Past research efforts regarding these two issues are relatively inadequate.

REVIEW OF LITERATURE

Failmezger (1) developed a warrant for left-turn-refuge construction based on ratings of many geometric and traffic parameters. However, no analytical rationale was provided. Harmelink (2) calculated the arrival and release rate of a combination of through and left-turning vehicles. He proposed that construction of a left-turn lane is warranted when the probability of having more than one of the vehicle combinations waiting in the system is less than 0.005. However, he failed to consider all the other numerous vehicle combinations, such as two consecutive left-turning vehicles, one left-turning vehicle followed by two through vehicles, and two left-turning vehicles followed by one through vehicle, and he did not explain the rationale behind the selection of the 0.005 probability level.

Hammer (3) suggested that a left-turn lane is warranted from an accident consideration point of view but neglected to consider delay. Shaw and Michael (4) as well as Ring and Carstens (5) employed a more-comprehensive approach for the left-turn-lane problem. Both teams considered the reduction in delay and accidents to be the benefits of a left-turn lane. They then compared the benefits with the construction cost of the left-turn lane to see whether the left-turn lane was justified. The approach was undoubtedly rational for an isolated intersection; however, because they assumed that the delay varied linearly with approach volume, opposing volume, and left-turn volume, they underestimated delays for high-volume ranges. This shortcoming would make their findings applicable only to low and moderate volumes.

Numerous studies of delay caused by left-turning vehicles at signalized intersections (6) have shown that delays increase curvilinearly with increases of left-turn, approaching, and opposing volumes. Delay approaches infinity when volumes are so high that left-turning vehicles could not find enough acceptable gaps in the opposing traffic stream. This characteristic of the delay function seems to point out the need for an accurate method of predicting delay if the use of the benefit-cost approach is to be expanded.

An important problem associated with the consideration of stopped delay is capacity. Once vehicles must stop and wait for their release from an intersection, the lane that they have occupied is temporarily blocked. The longer the delay, the shorter the time that the lane would be open for vehicles to go through the intersection, and the greater would be the reduction in capacity. Because delay varies curvilinearly with volumes, the capacity of the lane may be reduced to less than that of the approaching volume (a total breakdown of traffic) sooner than many people have believed. Even if the critical condition has not been reached, the reduction of capacity would cause the volume-capacity ratio to rise. This would result in the reduction of the level of service for the lane. For many lightly traveled highways, capacity may not be a serious problem. The level-of-service consideration, however, would certainly be of interest to highway engineers.

Because of the emphasis on safety and safety improvements, some left-turn lanes have been warranted based only on a consideration of accident reductions, and the reduction of delay is just an added benefit. Methods for handling this have been documented comprehensively in a National Cooperative Highway Research Program (NCHRP) report (7).

A benefit-cost approach is probably the most desirable way to handle a left-turn-lane design problem at a priority intersection. In order to effectively implement this approach, an estimate of the three most important quantifiable parameters (reduction of accidents, reduction of delays, and the length of the designed left-turn lane) with an acceptable accuracy appears to be essential.

STUDY APPROACH

For the purpose of predicting delay reduction, an experimental approach requires that delay data be collected before and after a left-turn lane is installed. Because delay data before installation are not generally required in a left-turn-lane construction, an intersection that has left-turn lanes would provide delay data only for the after-installation condition. If we could choose some intersections that are scheduled to have their left-turn lanes constructed in the future, it would be possible to collect both before and after delay data. Nevertheless, a long period of time would be required to accumulate an adequate number of cases to make the experimental results statistically significant. Certainly, an adequate number of intersections that do and do not have left-turn lanes could be located and their delay data collected to derive statistical trends on delay reductions. This method is costly, however, because a large amount of data are needed to discount the effect of local geometric and traffic conditions. In addition, existing facilities may be clustered within a small range of traffic conditions so that results developed from their data might not be applicable when traffic conditions outside of the range have to be dealt with. As for the estimation of the length of the left-turn lane, the experimental approach would face the same kind of difficulties as the delay-reduction consideration, even though only intersections that have left-turn lanes would be involved.

In view of the difficulties encountered by the experimental approach and since a simple deterministic formula cannot be developed to handle the probabilistic nature of the left-turn-lane design problem, simulation becomes the only logical solution. We have developed two computer-simulation programs for traffic on an uncontrolled approach at a two-lane by two-lane priority intersection. These programs can accurately predict vehicular delays caused by left-turn vehicles and the reduced capacity of an uncontrolled approach with and without a left-turn lane. By comparing the results from the two models, one will be able to see the improvement, in terms of delay and capacity, by providing left-turn lanes at an uncontrolled approach. Therefore, the benefits of building a left-turn lane could be established. Because outputs of the simulation model for the left-turn-lane condition show the queuing characteristics of the left-turning vehicles (i.e., lengths and frequencies of queues), the use of the model alone will provide the needed information on the length requirement for left-turn lanes. The main thrust of this paper is to use results generated from the two simulation models as a base and systematically look into the left-turn-lane design problem so that guidelines may be developed.

Many study results are available for predicting accident reductions due to construction of a left-turn lane. We, therefore, do not actively pursue this topic in this study.

SIMULATION MODELS

Although the simulation models were documented in several articles (8,9), a brief presentation is provided as a quick and usable reference. The first model is an attempt to duplicate the traffic operating characteristics of an uncontrolled approach at a priority intersection without a left-turn lane. The second model attempts to simulate the traffic conditions of the same intersection approach when a left-turn lane of infinite length is available. The conceptual flow of the two models is presented in Figures 1 and 2, respectively. In an attempt to validate the simulation models, delay data were collected in two Kansas locations. Since these two locations did not have left-turn lanes, only the first model was used to generate simulated results to compare with the collected data.

Table 1 is the result of this comparison. Five computer runs were used to generate simulated data so that the average and the standard deviation of simulated results could be developed. A significance test has been conducted by using a normal approximation. This approximation test had a significance level of about 5 percent. The test results showed that, in 12 of the 16 sets compared, the simulated results are not significantly different from the observed data. In view of the complexity of traffic behavior and the widely varied headway patterns that actual traffic has exhibited, it is felt that the simulation models have an acceptable accuracy.

SAMPLE SIMULATION RESULT

Various assumed traffic conditions were used as input to the simulation models to generate needed information for developing design guidelines for left-turn lanes. The results were summarized into the following graphs and tables. Figures 3, 4, and 5 indicate the volume-capacity ratio of an approach if no left-turn lane is available. Figures 6 and 7 illustrate the savings in delay due to the construction of a left-turn lane. Figure 8 specifies the length requirement of an approach if a left-turn lane is warranted. Graphs 3-7 were derived by using a 4.5-s critical gap for all left-turning vehicles and assuming that the opposing volume is either equal to or one-half of the approach volume. Delay-saving adjustments for conditions other than those specified are suggested in the tables below. The adjustment factors for the reduction of delay for various critical gaps are as follows:

Critical Gap (s)	Adjustment Factor
4.0	0.80
4.5	1.00
5.0	1.25
5.5	1.56

The adjustment factors for the reduction of delay for the difference between actual opposing volume and the opposing volumes shown in Figures 6 and 7 are as follows:

Difference (vehicles/h)	Adjustment Factors
+500	2.49
+400	2.07
+300	1.73
+200	1.44
+100	1.20
0	1.00
-100	0.83
-200	0.69
-300	0.58
-400	0.48
-500	0.40

Figure 8 is derived from simulated results of using a negative experimental headway distribution and a 4.5-s critical gap. However, adequate safety margins were included so the figure would be suitable for general use. In graphs 3 through 7, the symbol α represents the percentage of traffic that is assumed to be nonfree flowing in a composite headway-distribution model. The formula $\alpha = 1 - e^{-0.001\,55V}$ (where V is the volume of a traffic flow) was suggested by Lewis (10). The formula $\alpha = 1 - e^{-0.000\,39V}$ was derived from data collected at two Kansas locations.

These tables and graphs were the simulated results of the various described traffic conditions. They were used to

Figure 1. Conceptual flow of the simulated model without a left-turn lane.

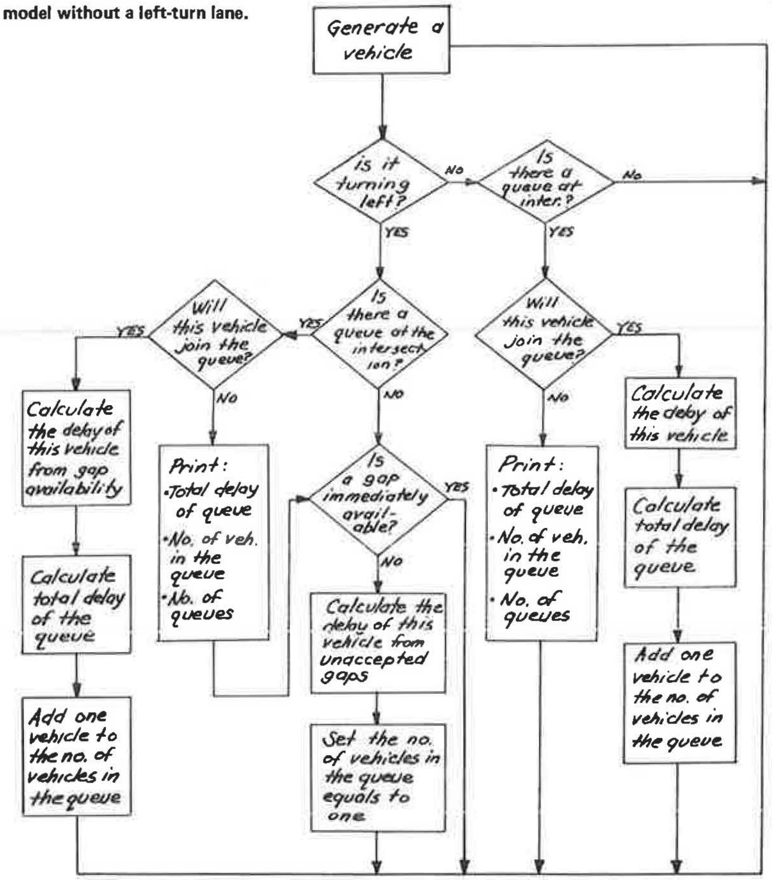
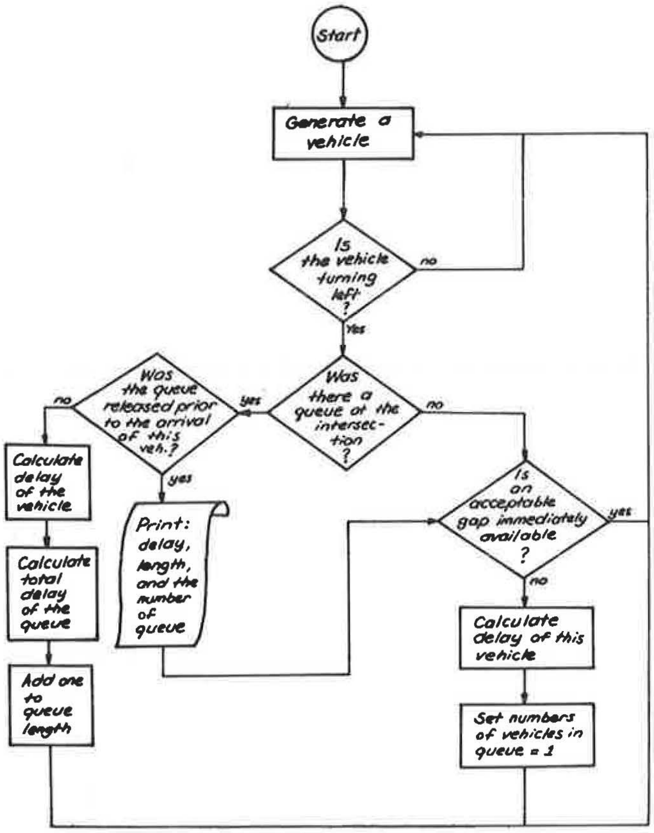


Figure 2. Conceptual flow of the simulation model with a left-turn lane.



illustrate the capability of the simulation models. If traffic conditions other than those described are of interest or greater accuracies are required, direct use of the models to obtain needed information would be desirable.

SUGGESTED DESIGN PROCEDURES FOR LEFT-TURN LANES

The overall approach for designing left-turn lanes at a priority intersection described in this paper is based on the concept of the benefit-cost ratio. The two developed traffic-simulation models are the basis for estimating many of the needed quantitative values for a benefit-cost analysis if such a design is considered. Design charts and tables derived from the simulated data were provided for normal traffic conditions (Figures 3-8 and the tables above). These charts and tables can help designers find needed information faster and more efficiently than can the direct use of the simulation models.

A conceptual model that illustrates the overall left-turn-lane design process on a systems basis is presented in Figure 9. Note that a precondition for using the process is that the intersection under consideration be a two-lane by two-lane priority intersection. However, the simulation models, design charts, and tables are considered applicable to some other situations if minor modifications are made. The user of this process should judge whether it is applicable to his or her particular case. Once the precondition is met and a designer must decide whether left-turn lanes should be built for the uncontrolled approaches, he or she should follow the steps outlined below.

1. Collect or estimate the following information about the traffic: (a) directional hourly volumes, (b) directional truck percentage, (c) directional right-turn

Table 1. Comparison of actual and simulated delays.

Approach Volume (vehicles/h)	Left Turn (%)	Opposing Volume (vehicles/h)	Observed Delay (s)	Simulated Results (s): Delay Run 1	Run 2	Run 3	Run 4	Run 5	Avg	SD	Significant Difference
216	0.9	234	0	0	0	0	0	0	0	0	No
234	20.5	216	73.7	88.5	65.6	32.4	48.4	92.4	65.5	25.9	No
209	4.3	332	59.9	39.5	10.5	4.3	11.9	45.5	22.4	17.7	Yes
332	29.0	209	101.07	285.8	158.0	166.2	126.9	197.7	186.9	30.4	Yes
314	2.8	360	44.4	43.0	6.6	9.0	21.0	74.1	30.7	29.0	No
360	25.9	314	467.3	469.9	241.1	234.7	230.6	261.7	287.6	102.9	No
314	3.5	423	65.4	38.4	9.1	8.4	38.4	70.8	33.0	26.8	No
423	39.5	314	418.7	820.8	527.8	550.6	577.1	519.3	599.1	129.6	No
166	2.6	204	0	11.1	33.0	5.5	2.7	28.9	16.2	13.0	No
204	31.6	166	154.4	111.0	46.9	69.6	69.1	76.1	74.5	27.6	Yes
207	3.0	211	0	3.8	11.0	26.5	7.2	4.6	10.6	9.8	No
211	24.8	207	23.6	43.9	89.9	60.0	34.1	56.8	56.9	23.9	No
236	5.3	376	0	10.7	36.6	80.9	9.8	13.4	30.3	30.5	No
376	20.2	236	543.8	95	214.3	183.2	165.7	100.9	151.8	51.3	Yes
313	3.0	249	0	8.7	20.4	51.6	2.6	7.9	18.2	21.1	No
249	10.7	313	48.9	41.4	60.6	73.4	56.2	82.9	64.1	17.8	No

Figure 3. Variation of the volume-capacity ratio due to changes of approach volume and percentage of left turns (negative exponential headway distribution).

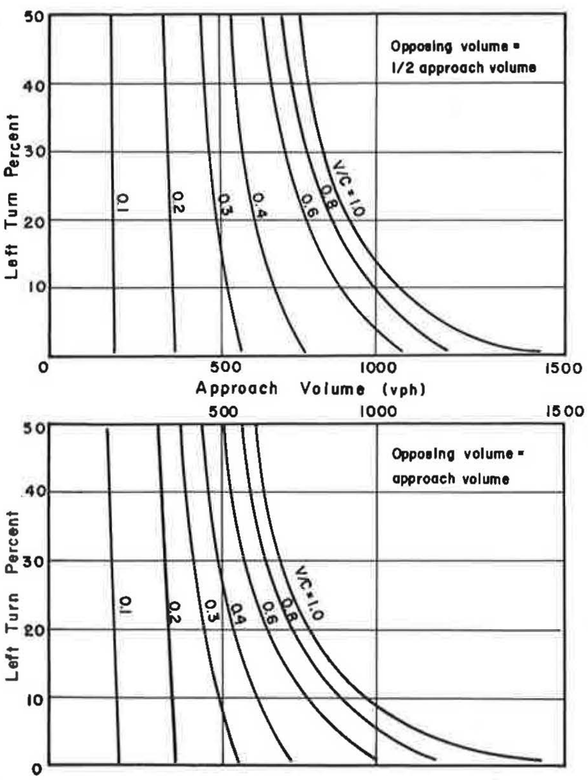
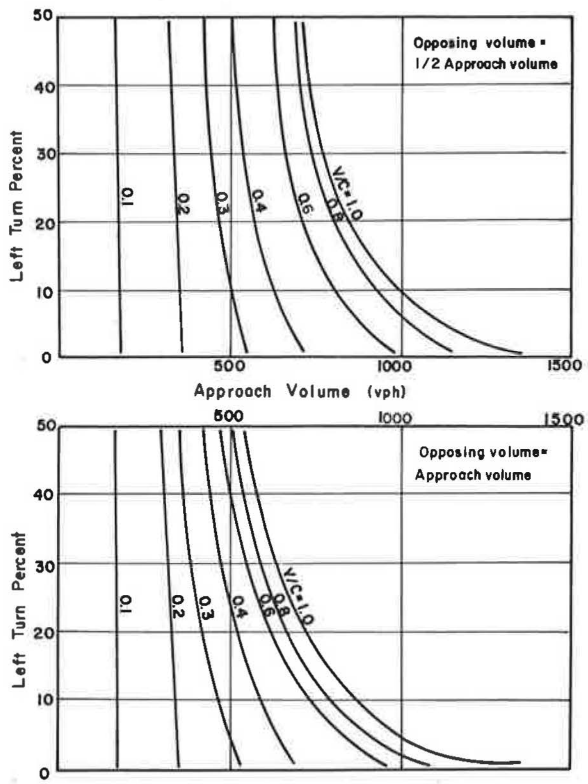


Figure 4. Variation of the volume-capacity ratio due to changes of approach volume and percentage of left turns (composite exponential headway distribution with $a = 1 - e^{-0.000\,39V}$).



percentage, (d) directional left-turn percentage, and (e) approach width.

2. Assume the traffic is composed of a group of free-flowing vehicles and a group of restrained vehicles. The percentage of restrained vehicles (α) is assumed to be equal to $1 - e^{-0.000\,39V}$ unless otherwise proven by collected data (V is volume in vehicles/h).

3. Assume the traffic has a critical gap equal to 4.5 s unless a different value is obtained from actual traffic data.

4. Use Figures 3, 4, or 5 with the design-hour values [expressed as average daily traffic (ADT)] defined above and read the corresponding volume-capacity values for the critical direction (the one with a higher directional volume).

5. Determine the capacity-adjustment factors for trucks, right turns, and approach width from the Highway Capacity Manual (11). The capacity adjustment factor for trucks, right turns, and approach width is given by the following equation:

$$F_c = F_t \times F_r \times F_w \quad (1)$$

where

F_c = total capacity adjustment factor,
F_t = truck adjustment factor,
F_r = right-turn adjustment factor, and
F_w = approach width adjustment factor.

Figure 5. Variation of the volume-capacity ratio due to changes of approach volume and percentage in left-turn lane (composite exponential headway distribution with $\alpha = 1 - e^{-0.00155V}$).

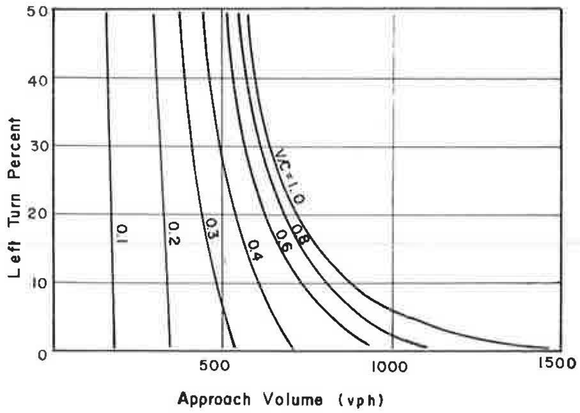
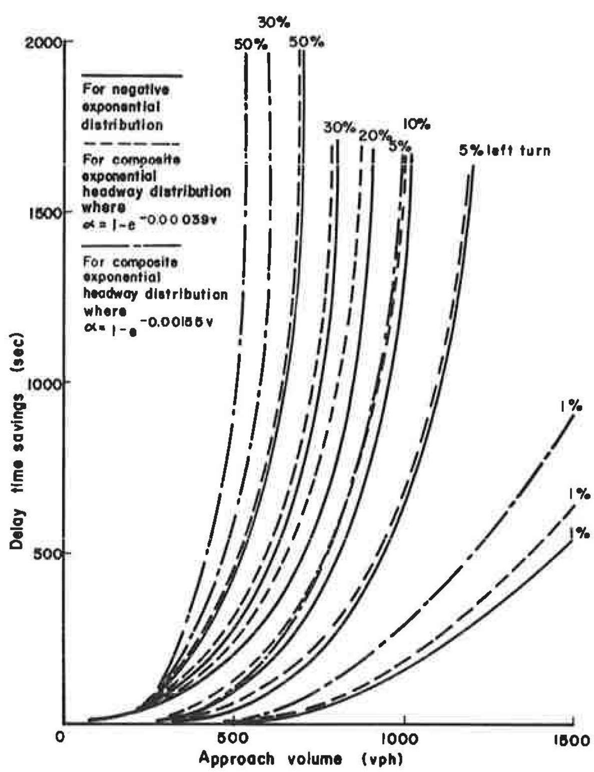


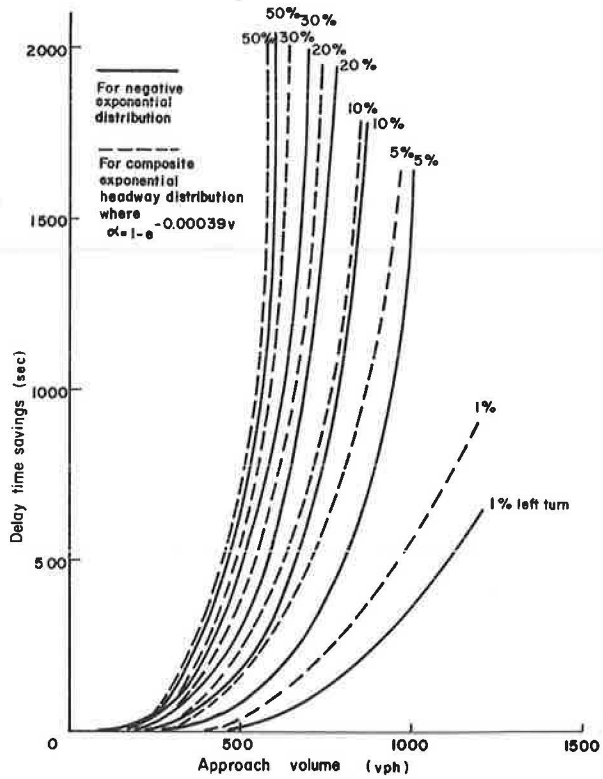
Figure 6. Delay time savings due to the construction of a left-turn lane for varied approach volume and percentage of left turns (opposing volume = half the approach volume).



The table below gives the values for F_t (11).

Trucks (%)	F_t	Trucks (%)	F_t
0	1.00	11	0.89
1	0.99	12	0.88
2	0.98	13	0.87
3	0.97	14	0.86
4	0.96	15	0.85
5	0.95	16	0.84
6	0.94	17	0.83
7	0.93	18	0.82
8	0.92	19	0.81
9	0.91	20	0.80
10	0.90		

Figure 7. Delay time savings due to the construction of a left-turn lane for a varied approach volume and percentage of left turns (opposing volume = approach volume).



The table below gives the values for F_r (11).

Right Turns (%)	F_r	Right Turns (%)	F_r
0	1.00	14	0.80
1	0.98	15	0.79
2	0.97	16	0.78
3	0.95	17	0.78
4	0.93	18	0.77
5	0.92	19	0.76
6	0.90	20	0.75
7	0.88	22	0.74
8	0.87	24	0.73
9	0.85	26	0.73
10	0.83	28	0.72
11	0.83	30+	0.71
12	0.82		
13	0.81		

The table below gives the values for F_w (1 m = 3.28 ft) (11).

F_w	Approach Width (m)
0.82	3.05
0.91	3.35
1.00	3.66
1.09	3.96
1.18	4.27

6. Calculate a modified volume-capacity value by considering the correction factors obtained above. If this modified volume-capacity value is greater than one or represents an unacceptable level of service, left-turn lanes should be built for the intersection and no more analysis is needed. If this modified volume-capacity value is less than one, proceed to the next step.

7. Obtain hourly time savings for every hour of the

Figure 8. Length requirement for left-turn lane for varied opposing volumes and numbers of left-turning vehicles.

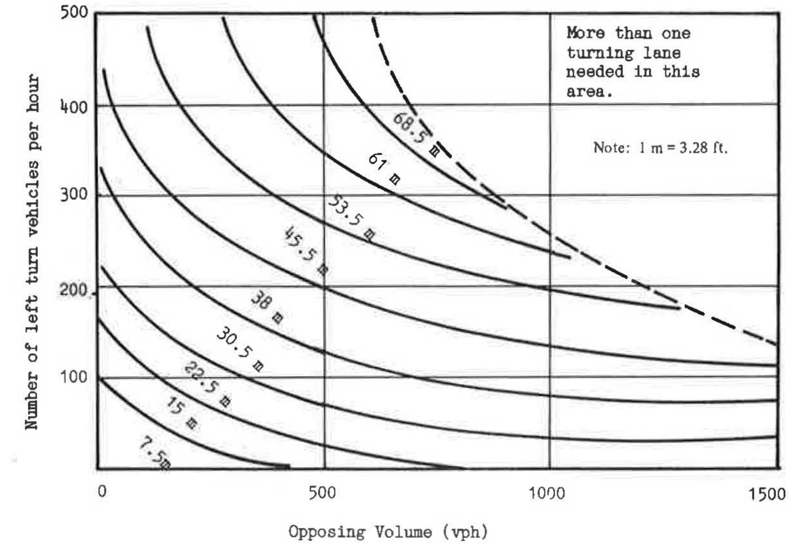


Figure 9. Systems model for designing left-turn lanes for a priority intersection.

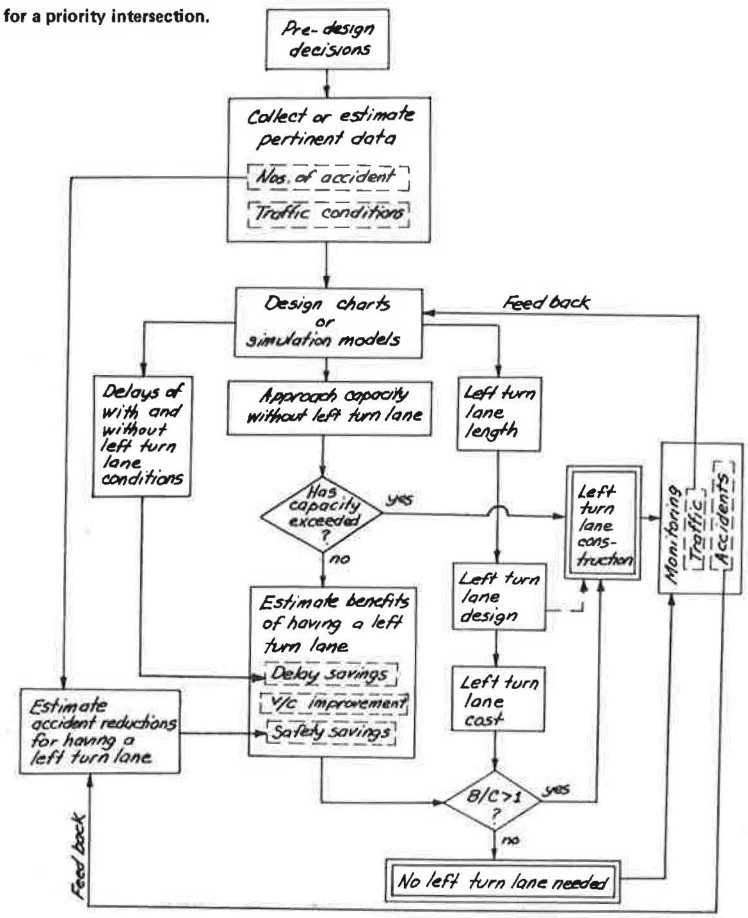


Table 2. Accident reduction forecast as a result of adding a left-turn lane without a signal.

		Accident Reduction (%)		
Area	Number of Lanes	All Accidents	Fatal-Injury Accidents	Property-Damage Accidents
Urban	2	1.9[a]	80[a]	
Urban	2+	6	54[a]	18[a]
Rural	2+	-6	-1[b]	

[a]Rough estimate; accurate percentage is in a range of 30-70 percent of this figure.
[b]Very rough estimate; accurate percentage is in a range of 70-150 percent of this figure.

day from Figures 6 or 7, assuming that a left-turn lane is available. Use the simulation models directly to obtain the hourly time savings if conditions specified for Figures 6 and 7 are not met.

8. Obtain total daily time savings by adding the hourly time savings.

9. Obtain the number of accident reductions from Table 2 (7, p. 140) and the calculation and tables below, which use methods derived from various existing sources. Be cautious in selecting the method used.

For suburban areas, the following calculation can be used (4):

Reduced numbers of accidents per million vehicles = 3.6203 - 1.1407 (number of approach lanes) + 1.2446 (approach ADT) - 0.7723 (opposing ADT) + 0.0371 (total intersection ADT).

For accident reduction due to left-turn channelization, use the table below (3):

Channelization Type	Accident Reduction (%)
Paint	32
Physically protected	64

Ring and Carstens (5, p. 71) found that construction of a left-turn lane prevented about one property-damage accident each year and one personal-injury accident every five years at each of four rural intersections studied.

The accident reduction forecasts used by the California Division of Highways (7, p. 141) show the following reductions (as a percentage of all accidents) for new left-turn channelization of unsignalized intersections:

Type of Channelization	Average Accident Reduction (%)
With curbs or raised bars	
Urban area	70
Suburban area	65
Rural area	60
With painted channelization	
Urban area	15
Suburban area	30
Rural area	50

10. Convert the time and accident savings into dollar values based on state economic analysis policies. American Association of State Highway and Transportation Officials (AASHTO) and National Highway Traffic Safety Administration (NHTSA) policies can be used if no state policies are available.
11. Obtain the left-turn-lane length requirement from Figure 8.
12. Design the left-turn-lane arrangement.
13. Compute the cost of installing the left-turn lanes.
14. By using benefit values obtained from step 10 and the cost value obtained from steps 11-13, conduct a benefit-cost analysis. An annual computation of the benefit-cost ratio is suggested.
15. If the calculated benefit-cost ratio is greater than one, the building of a left-turn lane is warranted. If the calculated benefit-cost ratio is less than one, the building of a left-turn lane may not be warranted. When the calculated benefit-cost ratio is close to one, redesign or recomputation is suggested for reaching a final decision.

For conditions that are not included in the charts and tables presented in this paper, the developed simulation models are suggested for use for left-turn-lane design purposes. Since the designer is likely to have a set of traffic parameter values different from those used in producing the charts and tables, he or she is urged to study the computer models carefully before making the necessary modifications. Lee (8,9) has a detailed description of model logics and other technical details.

CONCLUSION

This study has pointed out a new approach to highway design. Two simulation computer models for two-lane by two-lane priority intersections, one with and the other without left-turn lanes, were used as decision tools. The computer models tend to indicate that they have an acceptable degree of accuracy in duplicating the actual traffic condition. The models enable highway engineers to predict reduction of delays due to the construction of a left-turn lane if needed. The results enable us to develop design guidelines for priority intersections. Guidelines suggested in this paper are an attempt to systematize design procedures for left-turn lanes. The information presented should be a great improvement over the existing design methods. The more notable contributions of this study to the left-turn-lane design area can be summarized as follows:

1. Opposing volumes can be more adequately considered;
2. More realistic and complicated headway distributions can be accommodated;
3. Reduced delay, not the delay of without-left-turn-lane conditions alone, can be considered;
4. Left-turn-lane length recommendations are more realistic; and
5. Traffic conditions not suitable for simple queuing theories are more easily dealt with.

The result of this study is also further evidence that simulation is a vital and useful tool for highway designers. The quickness of computers makes them much more efficient for obtaining needed design information than are field observations. Four hours of traffic data collected were simulated on the computer (Honeywell 66/60 at the University of Kansas) in about 2.5 s.

ACKNOWLEDGMENT

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Characteristics of Crashes in Which a Vehicle Overturns

J. W. HALL

The objective of this study is to identify the characteristics of overturning crashes that might be susceptible to correction or amelioration through the application of highway and traffic engineering principles. The study first analyzed information contained in the Fatal Accident Reporting System and then analyzed data from New Mexico, which has one of the nation's highest rates of fatal overturning crashes. National statistics report that overturning is involved in 4 percent of all crashes but in 10 percent of fatal crashes. This study found that, in 11 states, more than 20 percent of the fatal crashes involved overturning. By use of appropriate statistical techniques, the study determined that, in comparison with other crash classifications, overturning occurs with significantly higher frequency under adverse geometric, weather, and lighting conditions. Overturning crashes are also more likely to involve nonlocal drivers and vehicles other than passenger cars. The analysis showed that these crashes had significantly different characteristics than those associated with fixed objects. Therefore, many remedial actions undertaken to reduce fixed-object crashes will have minimal impact on overturning. It is hypothesized that better application of delineation and warning devices could have a positive effect on overturning crash experience in addition to improvements in roadway geometrics. Some roadside design standards may need modification to accommodate the special requirements of certain vehicles. A field study of overturning crash sites is being conducted to obtain more detailed roadway and environmental information on these locations.

Although countless studies of crash occurrence are reported in the technical literature, virtually no attention has been given to the highway-related aspects of the set of crashes grouped into the category of noncollision. This group includes the extremely rare events that involve single-vehicle fires and explosions; however, its principal component consists of crashes in which a vehicle overturns. The crashes predominantly involve a single vehicle and, in fact, according to most schemes for categorizing crashes, a multivehicle crash coded as overturning is probably misclassified. National statistics indicate that only 4 percent of all crashes are classified as overturning, although they are cited in a disproportionate share (10 percent) of fatal crashes. Some western states report that more than 30 percent of their fatal crashes involve overturning.

Despite their substantial contribution to highway fatalities, overturning crashes have not been studied extensively. A recent bibliography of rollover accidents (1) lists several multidisciplinary accident investigation studies of specific overturning crashes but identifies only one study (2) that deals with the highway-related features at single-vehicle crash sites, and the emphasis in this study was fixed objects rather than overturning. A recent Federal Highway Administration (FHWA) study (3) of run-off-the-road crashes addresses some of the problems associated with overturning crashes.

Some researchers have suggested that overturning and fixed-object crashes have generally similar characteristics. The implied assumption is that the roadway, driver, vehicle, and environmental characteristics associated with a vehicle running off the roadway are the same in either case and that the difference is simply whether the roadside at the run-off-the-road location happens to have fixed objects. The logical extension of this assumption is that road-related improvements to reduce fixed-object crashes will also have a favorable effect on overturning crashes. The intent of this study is to examine the characteristics of overturning crashes and to compare them with other types of traffic accidents to establish the validity of this assumption.

The data bases used in the following analyses were obtained from computerized record systems. The classification of overturning was that assigned by the investigating officer. In some instances, such as a vehicle that strikes an embankment and overturns, the officer must make a judgment as to whether the crash should be categorized as fixed object or overturning. The officer's classification, modified to exclude multivehicle overturning crashes, was accepted. It is assumed that classification is reasonably consistent from year to year and among the states.

FATAL ACCIDENT REPORTING SYSTEM

The 1975 Fatal Accident Reporting System (FARS) was used to examine national characteristics related to overturning crashes. The system provides two methods of identifying fatal overturning crashes. A selection based on those crashes where the principal impact point was the top of the vehicle identified 1734 crashes, 18 percent of which were multivehicle. An alternative identification technique, based on those crashes for which the first harmful event is classified as overturning, identified 3038 crashes, only 14 percent of which had a principal impact point on the top of the vehicle. Despite previously noted deficiencies regarding crash classification, the latter group was judged to be more suitable for an analysis of single-vehicle fatal crashes that involve overturning. This set of crashes forms the basis for statistics cited in the following discussion.

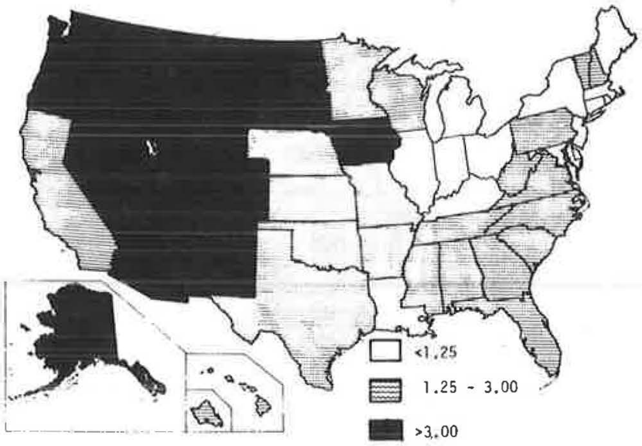
One of the factors that was immediately apparent from an examination of the crash data was the substantial variation in experience with overturning crashes among the states. In most of the primarily smaller, eastern states, overturning crashes constituted less than 6 percent of the annual fatal-accident experience. On the other hand, many western states reported that more than 20 percent of their fatal crashes involved overturning. When experience with fatal overturning crashes is adjusted for travel, a similar pattern is apparent. Figure 1 presents the rate of fatal overturning crashes per billion vehicle kilometers. All of the states that had fatal overturning-crash rates in excess of 3.0/billion vehicle-km of travel report that at least 18 percent of their fatalities are attributable to this manner of collision. Four states (Montana, Nevada, New Mexico, and Wyoming) report that more than one-third of their fatal crashes involved overturning. Although the state rates for fatal overturning crashes are highly correlated with reported fatal accident rates for these states, the unusually high percentages in this category, coupled with the fact that 89 percent of these crashes occur in rural areas, suggest that the western states warrant more attention in this regard.

FARS provides some information on the basic characteristics of these crashes. Reports on almost half the crashes (48.5 percent) for which a roadway alignment was specified indicate that the crash occurred on a curve. In 63 percent of the overturning crashes, the roadway was level. The curvature statistic is in general agreement with the statistics for 7600 run-off-the-road crashes on rural non-Interstate highways (3), although the latter study reports that only one-third of the crashes occur on level roadways. The substantial difference is most likely due to a difference in definitions.

FARS information on light condition indicates that 39 percent of the fatal overturning crashes occur during the day; 53 percent occur during darkness on an unlighted roadway. The portion of fatal overturning crashes during darkness is higher than that for the other fatal crashes.

The statistics also show an unusually high involvement level for pickups and vans. These vehicles are involved in 15 percent of all fatal crashes, and other statistics (4) indicate that this figure is comparable to their proportional share of vehicle registrations in the United States. On the other hand, they were involved in 27 percent of fatal overturning crashes.

Figure 1. Rates of fatal overturning crashes per billion vehicle kilometers.



ANALYSIS OF NEW MEXICO ACCIDENT DATA

FARS analysis confirms the importance, at least for some areas of the country, of overturning crashes in relation to highway safety. In an attempt to further examine the highway and environmental aspects of overturning crashes, an analysis was made of these crashes in New Mexico.

New Mexico has 113 000 km of highway, and annual highway travel is approximately 18 billion vehicle-km. In the past four years, New Mexico has averaged 50 000 accidents/year, and fatalities have averaged 610/year. Although New Mexico's highway fatality rate has decreased in recent years, New Mexico is among the group of states that has the highest fatality and fatal-accident rates. Of relevance to this study is that New Mexico classifies approximately one-third of its fatal crashes as overturning. The 1976 FARS data show that New Mexico has a fatal overturning crash rate of 9.6/billion vehicle-km--the second highest rate in the nation.

The data base used to examine the characteristics of New Mexico's overturning crashes was the 1978 New Mexico accident record system. This system provides information on 55 738 reported accidents, including 580 fatal accidents. The system includes information on urban accidents, including those from Bernalillo County, which accounts for approximately 35 percent of the state's total.

Preliminary analyses of the data were performed by using contingency tables. The comparisons made include the following:

1. A tables--accident classification (i.e., overturn, pedestrian-bicycle, two-vehicle, fixed-object, and other) versus 28 different types of crash characteristics (e.g., severity and lighting);
2. B tables--accident classification as either overturning or other (but excluding pedestrian, bicycle, and single-vehicle motorcycle accidents) versus crash characteristics; and
3. C tables--overturning accident classification only (categorized as either fatal or nonfatal) versus crash characteristics.

The A tables showed the extent to which the characteristics of overturning crashes differed from those that involve fixed objects and the several other crash classifications. The B tables highlighted those areas where overturning crashes differed from the set of other nonpedestrian accidents. Since the contingency table only tests for the independence of the variables, it is appropriate to collapse the table to identify the features that contribute to dependence of the variables. The C tables permitted an analysis of overturning crashes based on their severity. All of the statistical testing performed in this research and reported in this paper used an $\alpha = 0.01$.

A comparison of crash classification versus severity clearly shows the seriousness of the overturning crashes. In 1978, 4213 (fewer than 8 percent) of the crashes involved overturning, but they accounted for 171 (30 percent) of the fatal crashes. In general accord with national statistics, two-vehicle crashes account for 63 percent of the total but for only 34 percent of the fatal crashes. However, the severity statistics are unusual for fixed-object crashes. This crash classification could be viewed as an alternative to overturning in that a vehicle that inadvertently departs from the roadway has the potential for either collision with a fixed object or overturning. National statistics indicate that approximately 13 percent of all crashes involve fixed objects, but they account for 22 percent of all fatal crashes. Statistics from New Mexico indicate, however, that fixed objects are involved in 12 percent of each of the crash severity classifications. A cursory examination of New Mexico roadsides suggests a lower frequency of fixed objects than is found in most other states. To some extent this is reflected by the statistics; for example, only 0.5 percent of New Mexico's crashes involve guardrails. However, the reason for the disparity between national and New Mexico statistics, which exhibit comparable percentages of total fixed-object crashes but a significant difference in fatal crash percentages, is not obvious.

The severity aspects of overturning crashes are even more obvious when the other high-severity classification, pedestrian-bicycle [severity index (SI = 0.93)], is removed. In the tables that follow, pedestrian, bicycle, and single-vehicle motorcycle crashes are excluded. A portion of the B table analysis is presented in the table below to show that the percentage of overturning crashes that result in a fatality or an injury far outweighs their percentage of all crashes. "Other crashes" in the tables that follow include two-vehicle, fixed-object, parked-vehicle, and all other crashes except pedestrian-bicycle and single-vehicle motorcycle accidents.

Severity of Crash	Crash Type (%) Overturning	Other
Fatal	36.8	63.2
Injury	13.1	86.9
Property damage only	5.1	94.9
All	7.4	92.6

The severity of these crashes is also supported by the maximum vehicular damage that results from the crash. More than 67 percent of these crashes result in disabling damage (the most severe code) versus 34 percent for fixed-object crashes and 17 percent for all crashes. One-quarter of the crashes that result in fire are attributed to overturning.

An examination of the roadway alignment characteristic shows that 33 percent of the overturning crashes occur on curves. This percentage is significantly higher than that for all crashes (10 percent) and is significantly higher than that for fixed objects (23 percent). Even when the figures are adjusted to remove pedestrian accidents, the table below shows the unusually high crash experience associated with overturning crashes on curves.

Roadway Alignment at Crash Site	Crash Type (%) Overturning	Other
Straight	5.6	94.4
Curve	23.6	76.4
All	7.4	92.6

However, the percentages are substantially below those obtained from the FARS data. The logical conclusion from this latter comparison, which is supported by an examination of New Mexico's road system, is that a relatively lower percentage of the roadway has curvature.

The New Mexico accident record system classifies roadway grade as level, hillcrest, grade, and dip. By use of this scheme, 85 percent of all accidents reportedly occur on level sections. Since the system does not distinguish between positive and negative grades, the crashes classified in categories other than level were grouped together into an adverse-grade category. Analysis showed that 35 percent of the overturning crashes fell into this category, as opposed to 20 percent for fixed-object crashes and 15 percent for all crashes. These findings are consistent with FARS, but they differ substantially from those reported by FHWA (3), which suggest that only 35 percent of run-off-the-road crashes occur on level roadways. The data presented in the table below show that New Mexico's experience with overturning crashes at adverse-grade locations was significantly higher than would be expected if gradient was independent of crash type.

Roadway Grade at Crash Site	Crash Type (%) Overturning	Other
Level	5.7	94.3
Adverse	17.1	82.9
All	7.4	92.6

A comparison of crash classification versus light condition verifies that a significantly higher than expected number of the overturning crashes occur during other than daylight conditions, although 37 percent occur on dark, unlighted roadways and an additional 10 percent occur on dark, lighted roadways. These figures differ substantially from those for fixed-object crashes, 30 percent of which reportedly occur on dark, lighted roadways. The association of overturning crashes with dark, unlighted roadways, as shown in the table below, is partly a reflection of the rural nature of these crashes.

Lighting Condition at Crash Site	Crash Type (%) Overturning	Other
Day	5.6	94.4
Dawn or dusk	10.9	89.1
Dark, lighted	3.8	96.2
Dark, unlighted	19.9	80.1
All	7.4	92.6

Overturning crashes are found to occur with significantly higher frequencies in New Mexico's less urban counties and with a significantly lower than expected frequency in the principal urban county. Although overturning crashes account for 7.4 percent of the state's total reported accident experience (excluding pedestrians, bicycles, and motorcycles), in one-third of the counties more than 20 percent of the crashes are classified as overturning. On the other hand, fixed-object crashes, which constitute 12.9 percent of the statewide total, represent less than 20 percent of the crashes in each of New Mexico's counties.

The distinction between overturning and other crash types at the county level was examined by using correlation techniques. Both total and fixed-object crashes are found to be highly correlated with county population and population density. On the other hand, the number and rates (based on county population and area) of overturning crashes are poorly correlated with the number or rates of total and fixed-object crashes, population, and area. A linear model to predict overturning crashes based on the number of fixed-object crashes suggests a positive relation but explains only half of the observed variation. This finding detracts from the theory that decreases in the number of fixed-object crashes will lead to an increase in the number of overturning crashes.

An analysis of the weather conditions showed that 88 percent of all crashes occurred during clear weather. The crashes that occur during adverse weather conditions (i.e., rain, snow, fog, dust, and wind) do not show any unusual characteristics, except for those that involve overturning. Actual experience with overturning crashes during rain significantly exceeds what would be expected, and the experience during snow, fog, dust, and wind are each approximately twice the (statistically) expected level. Fixed-object crashes, however, do not exhibit unusual frequency levels for any of the weather categories. Similar findings were obtained from an analysis of road conditions, which showed that a significantly higher percentage of overturning crashes occurred on wet and icy pavement.

The findings of the analyses of weather and road conditions are also supported by the statistics on road defects. Thirty percent of all crashes where a road defect of slippery pavement was reported involved overturning. As shown in the table below, several categories of road defects were much more common at overturning crash sites. Although some lack of consistency may exist in the reporting of road defects, they are cited in only 1 percent of all crashes versus 2.6 percent of fixed-object crashes and 5.3 percent of overturning crashes.

Reported Road Defect at Crash Site	Crash Type (%) Overturning	Other
Slippery	29.5	70.5
Defective shoulder	62.1	37.9
Other	21.4	78.6
All	7.4	92.6

In New Mexico, overturning crashes account for 30 and 2.5 percent, respectively, of the rural and urban crash experience. The record system indicates that approximately 30 percent occur on streets and roadways that are one lane in each direction. However, a substantial number occur on freeways. Not surprisingly, more than 95 percent occur at nonintersection locations.

An analysis of the maximum posted speed at accident sites shows a significant difference between overturning and other crashes. Nearly two-thirds of all overturning crashes, versus 14 percent of other nonpedestrian crashes, occur on roadways where the posted speeds are 80 km/h or greater. The differences shown in the table below are statistically significant and are a reflection of the previously noted high severity.

Maximum Posted Speed at Crash Site	Crash Type (%) Overturning	Other
<55 km/h	1.6	98.4
55-75 km/h	3.3	96.7
>75 km/h	26.5	73.5
All	7.4	92.6

Analysis by day of the week revealed that a significantly higher than expected proportion of overturning crashes (17.3 percent) occurred on Sunday, although the lowest total number of accidents in New Mexico are on Sunday. An even higher (19.6 percent) number occur on Saturday, but the percentage is not significantly different than for other crashes because of Saturday's high accident experience.

With respect to time of day, overturning crashes conform to the patterns reported by others for single-vehicle accidents. They account for more than 15 percent of the crashes that occur between midnight and 7:00 a.m., a figure that is more than twice their proportion of all crashes. However, nearly 30 percent of the crashes during this seven-hour period involve fixed objects--a figure that is even less in line with their 12.9 percent share of all accidents. The lighting analysis showed that fixed-object crashes occur with unusually high frequency on dark, lighted roadways.

To examine the trend suggested by the FARS data that pickup trucks are involved to an unusually high extent in these crashes, an analysis was made of vehicle types. Although passenger cars are involved in two-thirds of all crashes, they constitute only 44 percent of the vehicles in overturning crashes. However, pickup trucks, tractor-trailer combinations, motorcycles, and vans are all significantly overrepresented among overturning vehicles. None of the other crash classifications exhibits a significant variation among vehicle types. The table below, which is based on the crash population excluding pedestrian and motorcycle crashes, shows the unexpectedly high involvement by vehicles other than passenger cars.

Vehicle Involved in Crash	Crash Type (%) Overturning	Other
Passenger car	5.0	95.0
Pickup	11.4	88.6
Van	15.2	84.8
Tractor-trailer	22.9	77.1
Other	8.4	91.6
All	7.4	92.6

Two human-related factors of overturning crashes warrant consideration. The first involves the driver's familiarity with the road and is suggested by the category of local versus nonlocal drivers. More than 71 percent of the drivers involved in overturning crashes were classified as either "nonlocal in-state" (48 percent) or "out-of-state" (23 percent). These figures are significantly different from those for fixed-object crashes, which involved only 34 percent unfamiliar drivers. The remaining crash classifications reported an average of 33 percent unfamiliar drivers. The table below shows the disparity between overturning and other crashes with respect to this characteristic.

Residence of Driver Involved in Crash	Crash Type (%) Overturning	Other
Local	3.1	96.9
Nonlocal, in-state	14.8	85.2
Out-of-state	18.7	81.3
All	7.4	92.6

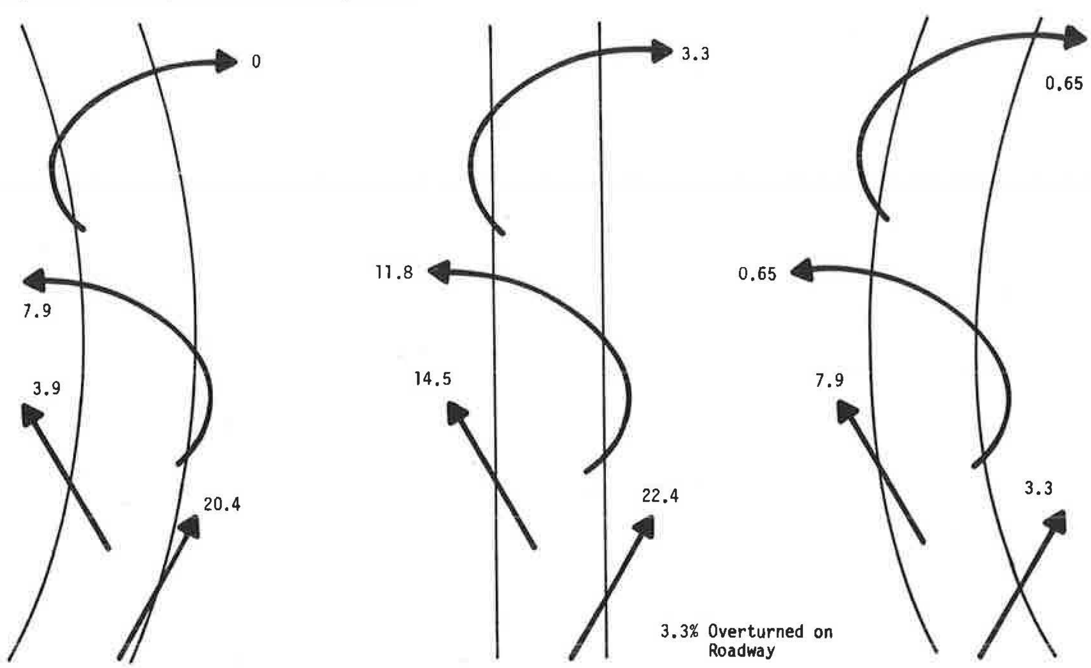
A second characteristic that may be of concern is the principal (reported) factor contributing to the crash. Approximately 16 percent of the overturning crashes reportedly have alcohol involvement (8.9 percent cited for driving while intoxicated) versus 22 percent for fixed-object crashes (15.9 percent driving while intoxicated) and 11 percent for all crashes (7.5 percent driving while intoxicated). Although these figures probably understate the actual involvement of alcohol, it is apparent that alcohol is involved to a considerably lower degree than in fixed-object collisions.

The C tables were used to compare the characteristics of the 171 fatal overturning crashes with the 4042 that resulted in nonfatal injuries or property damage only. In general, few characteristics were significantly different. Alcohol, higher posted speed limits, and dark, unlighted roadways were reported for fatal overturning crashes at moderately higher levels than would be expected. Adverse pavement conditions (primarily snow and the road defect of slippery pavement) were reported at significantly lower levels. A dramatic distinction between the fatal and other overturning crashes is the reported driver use of restraint systems. In fatal crashes, a significantly low number of drivers used seat belts. The analysis found that 63 of the drivers in fatal crashes were ejected, whereas only 4 ejections would have been expected. There were no significant differences with respect to features such as roadway alignment, unfamiliar drivers, and vehicle type, which were earlier found to be different among various crash classifications.

The foregoing analyses pretty much exhaust the information that can be obtained from the computerized accident record system. One factor of importance provided on the accident report form but not contained in the computerized system is the manner in which the vehicle left the roadway. To examine this point, a sample of fatal overturning crash reports was examined. The existence of curvature was determined from the officer's narrative and sketch rather than from the code for road character. It was found that 55 percent occurred on tangents, 32 percent on curves to the left, and 13 percent on curves to the right. Not surprisingly, when the few crashes that involved overturning on the roadway were excluded, half of the vehicles overturned on the right side and half on the left side.

The data showed, however, that 27 percent of the overturnings occurred on the opposite side of the road from which the vehicle initially departed. In virtually all of these cases, the officer stated that the driver left the roadway and overcorrected. Figure 2 shows the percentages of various maneuvers prior to overturning. Statistics that state only the side of the roadway on which the vehicle initially departed or only the side of the roadway on which the vehicle overturned do not reflect the overcorrection factor suggested by the maneuvers in the figure.

Figure 2. Vehicle departures in overturning crashes.



CONCLUSIONS

The analysis of FARS and New Mexico data leads to several conclusions about the nature of overturning crashes. They are seen to be a substantial component of the total accident picture, and their typical classification as apparently minor noncollision accidents seriously understates their importance. In those states where they are responsible for more than 20 percent of the annual highway fatalities, they clearly warrant more attention.

A principal finding of the study is that significant differences exist between the characteristics of overturning crashes and those that involve fixed objects. The specific differences between these two classes are as follows: (a) overturning crashes have higher severity, (b) they are more likely to occur on curves or grades, and (c) they are more closely related to adverse weather conditions. Other characteristics that distinguish overturning crashes from fixed-object crashes are their rural locations (also reflected by the dark, unlighted condition and maximum speed), the higher involvement of unfamiliar drivers, vehicles other than passenger cars, road defects, and the lower rate of alcohol involvement. The significant differences in roadway, environment, vehicle, and driver between these two crash classifications is a strong indication that remedial programs directed toward fixed-object crashes and severity reduction will not necessarily have an effect on overturning crashes.

The analyses suggest several things that are of importance to the transportation engineer. The existence of adverse geometrics at crash sites has been shown to be more common at fixed-object crash sites (2), but it appears to be even more prevalent at overturning crash sites. The excessive involvement by unfamiliar drivers suggests the need for improved positive guidance through the application of better delineation and improved warning. The transportation engineer can do little to control the registration and use of vehicles; however, the significantly higher overturning crash experience associated with certain vehicle types suggests that existing design standards for roadways and roadsides may not adequately address the special characteristics of these vehicles. And finally, the unsuccessful maneuvers that some drivers make, which result in overcorrection, may be susceptible to correction through improved shoulder design and maintenance.

The analyses reported in this paper are based primarily on accident record systems. Many crash-related factors that are of interest to the transportation engineer are not adequately or accurately reflected in the computerized record systems. It is therefore risky to draw far-reaching conclusions simply from an analysis of these systems. To counteract this problem, a program is currently under way in New Mexico to collect detailed information concerning the roadway and roadside characteristics at a sample of the overturning crash sites. Results from this study are anticipated in the spring of 1980.

ACKNOWLEDGMENT

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Operations and Design Guidelines for Facilities for High-Occupancy Vehicles

DEAN BOWMAN, CRAIG MILLER, AND BOB DEUSER

Design guidelines intended to enhance the safety of high-occupancy vehicle (HOV) preferential-treatment projects are proposed. These guidelines reflect the principal findings of a nationwide research program sponsored by the Federal Highway Administration in 1977 that involved the examination of more than 22 HOV projects for safety issues. Virtually every type of HOV technique was investigated, including freeway and arterial separated facilities, concurrent-flow lanes and contraflow lanes, freeway toll-plaza lanes, freeway ramp treatments, and arterial bus-preemption strategies. Cause-and-effect relationships of accident patterns on these projects were investigated and general guidelines formulated. Based on this analysis, HOV treatment-specific recommendations are offered to assist transportation planners and designers in improving the operations and design of HOV facilities with respect to safety.

In the United States, the 1970s were characterized by a proliferation of high-occupancy vehicle (HOV) preferential-treatment projects. Contraflow HOV lanes on arterials and freeways, concurrent-flow arterial freeway lanes, ramp-metering bypasses, separate freeway transitways, toll-plaza priority lanes, downtown transit malls, and signal preemption have all been recently implemented with their own particular design and operational features. The variance in design and operational features even exists among individual applications of the same type of preferential treatment. Without nationally established guidelines, the local project manager has been left to develop project-specific design standards, traffic control devices, and operating strategies. As a result, an extensive experimental base has been established from which local innovations can be analyzed comparatively for safety and operational implications.

In 1977, the Federal Highway Administration (FHWA) initiated such a study to survey existing HOV projects and examine the relationship between project characteristics and accident patterns (1). The research focused on five major areas associated with HOV projects:

1. Examination of accident rates,
2. Analysis of causative factors that influence safety,

Table 1. HOV projects included in FHWA research.

Project	Location	Type of Road	Treatment: Separate Facility	Concurrent-Flow Lane	Contraflow Lane	Toll-Plaza Lane	Ramp Treatment	Bus Preemption
Shirley Highway	Washington, DC	Freeway	X					
San Bernardino Freeway	Los Angeles, CA	Freeway	X					
I-95	Miami, FL	Freeway		X				
Banfield Freeway	Portland, OR	Freeway		X				
Moanalua Freeway	Honolulu, HI	Freeway		X				
Santa Monica Freeway	Los Angeles, CA	Freeway		X				
US-101	San Francisco, CA	Freeway		X	X			
I-495	Hudson County, NJ	Freeway			X			
Long Island Expressway	New York, NY	Freeway			X			
San Francisco-Oakland Bay Bridge	CA	Freeway				X		
Santa Monica, Golden State, and Harbor Freeways	Los Angeles, CA	Freeway					X	
I-5	Seattle, WA	Freeway					X	
North Central Expressway	Dallas, TX	Freeway					X	
I-35W	Minneapolis, MN	Freeway					X	
Nicollet Mall	Minneapolis, MN	Arterial	X					
Washington central business district	Washington, DC	Arterial		X				
Elm-Commerce Streets	Dallas, TX	Arterial		X				
US-1-South Dixie Highway	Miami, FL	Arterial		X	X			
Kalanianaole Highway	Honolulu, HI	Arterial		X	X			
Marquette and Second Avenues	Minneapolis, MN	Arterial			X			
Ponce de Leon and Fernandez Juncos Avenues	San Juan, PR	Arterial			X			
NW 7th Avenue	Miami, FL	Arterial		X	X			X

Table 2. Facility accident rates during peak periods by HOV treatments.

Treatment	Peak Period	Number of Projects	Accident Rate (Accidents/MVM): Average[a]	Highest	Lowest
Freeway related					
Separate facility	Morning and evening	3	1.5	2.2	1.1
Concurrent-flow lane	Morning and evening	4	6.7	8.4	4.2
Contraflow lane	Morning or evening	3	3.1	3.3	2.9
Toll-plaza lane	Morning	1	4.7		
Ramp-metering bypass	Morning or evening	1	17.3[b]		
Arterial related					
Concurrent-flow lane					
Median	Morning and evening	3	6.6	10.5	4.6
Curb	Morning and evening	1	6.5		
Contraflow lane					
Median	Morning and evening	3	8.6	12.4	1.3
Curb	Morning and evening	1	9.2		
Signal preemption	Morning and evening	1	4.1		

[a]This figure is calculated by dividing the sum of the accident rates by the number of projects.
[b]This rate refers to accidents per year for 21 ramps.

3. Identification of difficult maneuvers and potential safety problems,
4. Development of recommendations to improve safety, and
5. Review of the legal authority and legal liability issues faced by HOV projects.

In addition, a second research effort (2) was conducted to explore, among other things, the implications or current design and operating practices on effective enforcement of HOV restrictions and regulatory mechanisms.

The research team visited 22 HOV projects on 16 highway facilities. These projects encompass virtually every type of preferential-treatment strategy currently deployed in the United States on both freeways and uncontrolled-access highways. For each HOV project, data on safety, enforcement, operations, and geometrics were collected and analyzed. These data, when coupled with qualitative information, can be used to describe the current experience relating to contemporary design and operating practice on HOV facilities. The projects investigated are summarized in Table 1 (1,2).

ANALYSIS

Accident data from the different projects were compared by using the number of accidents and injuries per million vehicle miles (MVM) and million passenger miles (MPM) as the primary basis of comparison. Tables 2-4 present a summary of the facility and bus accident rates against various types of HOV priority treatments. The facility accident rates (Tables 2 and 3) describe the significance of the effect that various HOV strategies have on a facility's overall safety. The bus accident rates (Table 4) illustrate the relative safety of vehicles traveling in the HOV lane. Absolute comparisons between HOV priority treatments should not be made because local, site-specific factors can contribute significantly to a facility's safety performance. From Tables 2-4, the following general conclusions can be made.

The introduction of an HOV project on the facilities investigated has tended to increase the facility's accident rate. Based on vehicle miles of travel, six projects experienced a statistically significant increase of peak-period facility accident rates subsequent to the

Table 3. Change in accident rates during peak periods from before condition.

Treatment	Peak Period	Accident Rate Based on Vehicle Miles: No. of Projects That Experienced Change[a]		Accident Rate Based on Person Miles: No. of Projects That Experienced Change[a]	
		Increase	Decrease	Increase	Decrease
Freeway related					
Separate facility	Morning and evening	1^{b}		1^{b}	
Concurrent-flow lane	Morning and evening	2^{c}	1^{b}	2^{c}	1^{b}
Contraflow lane	Evening	1^{b}		1^{b}	
Toll plaza lane	Morning	1^{b}		1^{b}	
Ramp-metering bypass	Morning or evening	1^{a}		1^{c}	
Arterial related					
Concurrent-flow lane					
Median	Morning and evening	2^{d}	1^{b}	2^{b}	1^{b}
Contraflow lane					
Median	Morning and evening	2^{d}	1^{b}	1^{c}	2^{b}
Curb	Morning and evening	1^{c}		1^{c}	
Signal preemption	Morning and evening		1^{c}		1^{c}
Total significant change		6	1	5	1
Total nonsignificant change		5	3	5	4

[a] Some projects do not have comparative before data.
[b] Not significant.
[c] Level of significance is 95 percent or better.
[d] Evening is significant; morning is not.

Table 4. Bus accident rates during peak periods by HOV treatment.

Treatment	Peak Period	Number of Projects	Accident Rate (Accidents/MVM): Average[a]	Highest	Lowest	Change from Before Condition[b]: Increase	Decrease
Freeway related							
Separate facility	Morning and evening	1	4.4			1^{c}	0
Concurrent-flow lane	Morning and evening	3	7.5	18.6	0.0		
Contraflow lane	Morning or evening	3	5.1	8.6	1.7		
Toll-plaza lane	Morning	1	4.8			1^{d}	
Ramp-metering bypass	Morning or evening	1	0.0				
Arterial related							
Concurrent-flow lane							
Median	Morning and evening	3	304.5	851.1	8.9		1^{c}
Contraflow lane							
Median	Morning and evening	3	323.0	535.7	158.5	1^{d}	
Curb	Morning and evening	1	56.4			1^{d}	0
Signal preemption	Morning and evening	1	90.9				

[a] This figure is calculated by dividing the sum of the accident rates by the number of projects.
[b] Some projects do not have comparative before data.
[c] Not significant.
[d] Level of significance is 95 percent or better.

initiation of HOV operations, five projects experienced a statistically insignificant increase, one project experienced a statistically significant decrease, and three projects experienced a statistically insignificant decrease. When the accident rates are based on person miles, there was a small improvement in overall performance compared with the data presented above.

For each priority treatment, the average bus accident rates for freeway projects are slightly higher than the corresponding overall average freeway accident rates, in general. In general, the average bus accident rates for arterial street projects are many times higher than the average bus accident rates for freeway projects.

The statistical procedure applied to determine whether a significant change occurred in accident conditions between the various testing stages was standard hypothesis testing that used the normal approximation to the Poisson distribution as a basis. The Poisson constitutes a reasonable measurement of accident occurrence over time because the nature of accident occurrence is essentially random. Approximation of the Poisson distribution by the normal distribution is valid for sufficient sample sizes. The mean of the population is estimated by $\lambda = N/M$ and the standard deviation by $\sigma = \sqrt{\lambda}$ where N = number of accidents and M = millions of vehicle miles. Since a sample of M million vehicle miles was obtained, it is possible to estimate the mean and standard deviation of the sample by $\bar{x} = N/M$ and $\bar{s} = \sqrt{\bar{x}/M}$. Hypothesis testing can be performed by use of either the t-statistic or the z-statistic.

Since HOV projects are designed to increase passenger throughput and minimize passenger travel time, vehicle miles were also converted to passenger miles wherever sufficient data were available. This enables the planner to assess the safety issue in the context of project goals.

Statistical tests are valid only where a sufficient sample size exists. The sample size has to be large enough to ensure that the confidence interval is small enough for realistic analyses and to ensure that the normal distribution is an appropriate approximation to the Poisson. In general, the sample size has to be greater than $(9/\lambda)$.

From the statistical analyses, trends were observed and related to causative factors, geometric deficiencies, and traffic control features of the various subject projects. In this manner, the local innovations developed for projects prior to the establishment of HOV standards could be examined and evaluated from a safety point of view. The recommendations that have been formulated during the course of the research and summarized here are based on the safety and enforcement experience of actual HOV installations of various HOV treatment strategies. The experiences of other HOV projects should enable future HOV installations to operate in a more safe, enforceable, and efficient manner.

SEPARATE HOV FACILITIES ON FREEWAYS

Separate HOV facilities are roadways or lanes that are physically separated from the general freeway lanes. These facilities are designated for exclusive use by specified HOVs, and all other vehicles are prohibited.

The separation can be a barrier wall or a painted buffer area supplemented by cones or other non-fixed-object traffic control devices. Lanes separated by barrier walls are really independent highways that have no interaction with the general lanes, except at the terminal points. Partially separated lanes can have shared medians or shoulders that reduce right-of-way requirements. In this partially separated design, the restricted lanes are accessible (illegally) from the general lanes and this increases the likelihood of violations. The joint-use shoulder can be penetrated by both violators and HOVs.

Analysis

Two separate HOV facility projects were investigated: the Shirley Highway and the San Bernardino Freeway. Shirley Highway contains an 11.5-mile (18.5-km) HOV facility separated from general lanes by concrete median barrier walls. San Bernardino Freeway contains both completely separated and partially separated sections that were treated individually in the analyses. Hence, three design configurations were analyzed for the separate-facility HOV treatment.

The HOV lanes, in all instances, were considerably safer than the general lanes, as demonstrated by the fact that only 2 percent of total facility accidents occurred in the HOV lanes. Moreover, total facility accident rates experienced a statistically significant decrease with the introduction of HOV provisions on the San Bernardino Freeway (sufficient data were unavailable for Shirley Highway).

Recommendations

The research resulted in the identification of certain site-specific safety problems that occurred on the projects studied. The following recommendations are offered in response to these problems.

The ideal terminals to and from separated HOV lanes are exclusive ramps. If this is not possible, the potential exists for a severe accident hazard unless considerable care is exercised at the interface of the HOV lane and the general lane. At the output terminal, it is best to add a lane in order to avoid a left-hand merging condition. At the input terminal, it is best to provide an exclusive concurrent-flow HOV lane upstream of the diverge point, but not of such a distance as to make it attractive to violators as a congestion-avoidance measure.

On the San Bernardino Freeway, a 1-mile (1.6-km) HOV approach lane was provided on the left side of the facility. Violators often used this lane to bypass recurring congestion. A safety problem was created because (a) rear-end accidents resulted from the speed differential between HOVs and violators that enter the lane and (b) violators often became trapped in the lane near the exit and had to stop before being able to merge back into the general traffic lanes. Accident rates more than doubled in this section when the separated HOV lanes were opened. One additional design feature to provide relief for such a condition would be to provide a shoulder for the concurrent-flow HOV lane at the crossover locations to avoid trapping violators.

Totally separated HOV facilities generally require restrictive traffic control devices only at the input terminals to identify the authorized users and times. At outputs it may be necessary to bar wrong-way entry, and this should be accomplished with highly visible gates or barricades, flashing beacons, and NO ENTRY signs. On partially separated sections, HOV lane-use signs should be installed at intervals along the route as a continuous discouragement to violators.

On partially separated HOV lanes, supplemental signing should be provided at inputs to identify the legal exits from the limited-access facility. This should help minimize erratic maneuvers by drivers who need to exit at locations other than the HOV lane terminals. A possible message is RESTRICTED LANE EXITS ONLY AT (location).

On partially separated facilities that have a common shoulder, the shoulder should have distinctive solid white lines on both sides. Double lines are even more forceful. The shoulder should contain chevrons or cross-hatching and word messages to discourage crossing. Flexible tubular markers should be placed at 40-ft (11.9-m) intervals to further discourage crossing.

CONCURRENT FLOW: FREEWAY

Concurrent-flow HOV lane projects on freeways generally involve the designation of the median lanes for use by buses only or by buses and carpools. Access to the restricted lanes is often continuous; that is, there is no physical separation or other barrier between the HOV and general lanes. The lack of physical separation of the HOV from the general lanes permits continuous access and egress, but it is also the cause of several operational and safety problems not experienced in other HOV treatments on freeways.

Concurrent-flow HOV lanes can be created by either reserving an existing lane for HOVs or by constructing new lanes in the median. These two approaches have differing effects on the operation of the facility. The addition of lanes increases capacity but, in order to do so, it often eliminates or reduces median shoulders or refuge areas that could be used by disabled motorists and enforcement operations. Also, the take-a-lane strategy for HOVs will reduce capacity of general traffic and increase the congestion in the general-travel lanes. Public acceptance of the concurrent-flow HOV treatment has been much better when new lanes are added for the HOVs.

Analysis

Four concurrent-flow HOV lane projects were investigated in detail as a part of this research: Moanalua Freeway, Santa Monica Freeway, US-101, and I-95. A general observation was that the implementation of concurrent-flow HOV lanes significantly increased total accident rates for the facility, as evidenced by statistically significant increases in accident rates on all projects except I-95. However, there were no substantial changes in accident types or in the distribution of vehicle types involved in accidents on any of the projects.

High differential speeds between continuously accessible HOV lanes and adjacent general lanes, coupled with merging into and out of the HOV lane, appeared to be the most significant causes of accidents. Weaving across several general lanes to gain access to or leave the HOV lane was a secondary factor. Incidents that blocked any lane, but particularly the HOV lane, were also a significant cause of serious accidents, although it was not possible to quantify the degree of this problem.

Recommendations

Concurrent-flow HOV lanes should be added to a facility rather than taken from existing general use, particularly on heavily congested urban freeways.

The provision of median refuge shoulders is emphatically recommended for this priority treatment. If right-of-way constraints require the compromising of some geometric design standards, the provision of emergency refuge areas in the median should take precedence over such factors as lane width. However, lane width should not be reduced to less than 11 ft (3.3 m).

If the HOV lane is a continuously accessible lane, the lane demarcation between the HOV lane and the general lane should be an extra-wide, broken white line. The Manual on Uniform Traffic Control Devices (MUTCD) now allows a solid line for this purpose (3). However, this can be interpreted as an edge line, and its use for the HOV lane is not recommended except in areas where it is strongly desired to discourage weaving and possibly for bus-only lanes or 24-h HOV lane operations.

The ideal input treatment to a concurrent-flow HOV lane is an added lane on the left. This avoids merging problems because HOVs simply shift into the new lane. If an existing lane must be dropped entirely to create the HOV lane, a right-hand, general-traffic lane should be dropped (preferably at a high-demand exit).

The ideal exit terminal treatment is a continuous lane or, if demand is sufficient, a left-hand exit ramp. If any lane must be dropped at the end of the HOV lane section, it is preferable to drop a right lane at a high-demand exit and have the HOV lane assume general-use status. If the only option is to drop the HOV lane, an extended taper must be provided along with a refuge area for vehicles that have difficulty in merging.

The speed differential between the HOV lane and the general-use lanes provides the travel time savings for HOV traffic but also poses the most severe safety hazard. The resolution of these conflicting goals will require further research to quantify an optimum speed differential that does not adversely affect safety while maintaining the HOV strategy's operational integrity. Metering of general-lane traffic at on-ramps or the use of variable speed control signing on the HOV lane could be used to reduce an excessive speed differential.

Signalization is generally not necessary on concurrent-flow lane treatments. In locations where sight distances are limited, consideration should be given to using either variable message signing or warning beacons to warn motorists of stalled traffic ahead in the HOV lane or other lanes. These could be centrally operated by police officers or by an automated traffic surveillance and control system.

If conventional enforcement techniques are used, the officers should make every effort to minimize disruption to traffic. On-freeway (stationary) monitoring is effective in reducing violations but it can also slow traffic. Weaving across the freeway to apprehend violators is particularly disruptive and should be avoided if possible. Citations should be issued out of the motorists' sight to eliminate gawking. The visibility of issuing citations on the right shoulder has minimal effect since passersby cannot relate the specific violation to the enforcement activity. Legislative action to permit photographic or mail-out citation techniques should also be considered (2).

CONTRAFLOW LANE: FREEWAY

The common practice of installing contraflow HOV lanes is to assign the inside (median) lane in the opposing (off-peak) direction to a special class of vehicles. The contraflow lane is separated from the other travel lanes by flexible tubular markers. If sufficient capacity remains in the off-peak direction, an additional lane can be taken for use as a buffer lane. Thus, the contraflow lane treatment makes use of surplus capacity in the off-peak direction, thereby increasing the vehicle- and person-moving capacity in the peak direction.

Analysis

Three applications of contraflow treatment on freeways were investigated during this research: I-495, Long Island Expressway, and US-101. On all three projects, the accident rates were higher in the off-peak direction than in the peak direction during HOV operations. These differences were statistically significant except on I-495. Only on US-101 were accident data available for both before and after conditions. On this project, the daily accident rate experienced a statistically significant increase with the introduction of the contraflow lane.

The most apparent causative factor related to safety problems on contraflow HOV lanes is the capacity reduction in the off-peak direction. Off-peak decreases in operating speeds and site-specific incidences of congestion resulted in an increased number of rear-end collisions and other congestion-oriented safety problems.

This off-peak safety problem was less prevalent for projects installed on facilities that have superior geometric features. Presumably because of better alignments and fewer geometric constraints, accident rates on US-101 were lower than the comparable accident rates on I-495. Head-on conflicts between the contraflow lane and opposing traffic occurred only on I-495 because of its tight geometrics, although it was not a recurring problem.

Recommendations

Contraflow lanes are generally implemented on existing freeways without substantial modification of the main-line geometrics of the freeway. If possible, contraflow lanes should be implemented on freeways that have high-design standards. Every effort should be made to maximize the safety and quality of the geometric design.

The ideal terminals to and from the contraflow lane are exclusive ramps or toll booth lanes (if the output terminus is to a toll plaza). Where median crossovers are required at the input terminus, a short access lane should be provided upstream of the crossover to allow for deceleration. Terminals should be closed during periods of non-HOV operation.

Where a buffer lane cannot be provided between the contraflow lane and the gereral-use lanes, proper use of the lane should be designated by overhead lane-use control signals displayed over the contraflow lane and the adjacent general-use lane. Spacings should conform to sight distance and MUTCD standards (3).

Where a buffer lane can be provided between the contraflow lane and the general-use lanes, overhead lane-use control signals are not necessary to designate proper lane use if sufficient physical separation and signing is provided.

Signing in the off-peak direction approaching the contraflow section should consist of both advanced-warning and restricted-lane signing along the main line. Messages such as CAUTION--ONCOMING TRAFFIC AHEAD--X FEET (Y KM) and LEFT LANE CLOSED--ONCOMING TRAFFIC, with flashers and merge-right arrows as appropriate, are more positive than the standard MUTCD-restricted lane signing. Blank-out message signs are preferable to specified time periods due to the flexibility in operating hours.

Signing in the off-peak direction at the end of the contraflow section should be the standard MUTCD end-of-HOV-lane sign. A lane-control signal should be placed downstream and all green arrows permanently displayed over each off-peak directional lane.

Signing in the peak direction would depend on the type of terminal treatment. Standard MUTCD signing should be used and emphasis placed on which vehicles may use the contraflow lane.

The demarcation for a contraflow lane should be a double yellow broken line to indicate a reversible lane. Yellow flexible tubular markers should be placed along the lane line. They should be reflectorized and spaced at a maximum distance of 40-ft (11.9-m) intervals. The use of the diamond symbol on the contraflow lane is discouraged, as this implies vehicle classification and not direction.

Use of the contraflow lane should be restricted to experienced and trained operators. In addition to transit operators, operators of other vehicles (charter buses, minibuses, vanpools, taxis, and carpools) could be permitted use of the contraflow lane if special licensing requirements are met. All motorists who use the contraflow lane should be required to use flashers with the vehicle.

Additional restrictions may be desirable on both the contraflow-lane and opposing-lane traffic. Reduction of the speed limit and spatial headways are the most common restrictions.

Quick-reaction incident detection and removal systems should be incorporated into the project. If possible, median cuts should be provided if there is no buffer lane so emergency vehicles can approach in the proper direction; however, these should not be penetrable by general traffic or present a collision hazard themselves. Care must also be taken to minimize pedestrian use of these crossings. Incident management can be greatly enhanced by the provision of freeway surveillance (electronic sensors or television), and warning beacons should be considered as well to alert oncoming traffic of incidents downstream.

Enforcement of contraflow lane use should be directed at the terminals because activity along the main line can be extremely disruptive, if not impossible. Monitoring should be active throughout the project area, especially for violations of the special restrictions suggested above.

Contraflow lanes should not be installed if such action will cause traffic flows in the off-peak direction to deteriorate to levels that induce a significant increase in rear-end accidents.

TOLL-PLAZA LANE: FREEWAY

The establishment of certain toll-plaza lanes for exclusive use by HOVs enables these vehicles to bypass substantial queues and gain access to the toll facility with less delay.

Analysis

On the San Francisco-Oakland Bay Bridge (SFOBB) toll plaza, 3 of the 17 approach lanes are reserved for buses and carpools. The HOV lanes are free flow since carpools pay no toll and bus companies are billed based on scheduled crossings. Further advantage is given to HOVs via a bypass of the ramp-metering station installed to improve flow across the bridge. Thus, the exclusive toll-plaza lanes serve several purposes. They allow HOVs to (a) bypass queues on the approach, (b) move through the toll station with minimal delay, and (c) gain preferential access to the toll facility itself.

Implementation of HOV lanes in the SFOBB toll facility appeared to adversely affect safety on the facility, although this was largely alleviated by the metering system. The most obvious factor that has an effect on safety in the SFOBB toll-plaza area was the congestion pattern that results from the implementation of the HOV lanes. This project had the effect of dividing what was formerly a homogeneous stop-and-move queue, which extended some distance upstream, into two sections separated by HOV lanes in the middle. This resulted in extending the queuing area farther upstream in the two halves of the general roadway lanes and in introducing a speed differential in the center of the facility.

The geometry of the SFOBB was not designed to accommodate the HOV toll-plaza priority treatment. The facility had several problems in this respect:

1. Trucks that enter the facility from the left (Nimitz Freeway) must weave across the lanes to gain access to the right-hand toll-plaza lanes that accommodate trucks;
2. Since HOV lanes are in the center of a 17-lane toll plaza, a large amount of weaving is required;
3. There is a penetrable barrier between the HOV lanes and the general lanes; and
4. There is a rapid narrowing from 17 to 5 lanes in the toll-plaza output section.

These problems do not all result from the HOV priority treatment, but the HOV strategy has, to some extent, compounded the potential hazards.

Recommendations

The following provides a set of recommendations that have been developed in response to the experience of the SFOBB.

The weaving area that provides access to the priority lane should be of sufficient length to minimize conflicts and to permit the distribution of HOVs into the priority lanes well in advance of the queuing area [in order to avoid the unsafe condition of late merges from slower-moving vehicles into the HOV lane where a 15-mph (24-km/h) speed differential exists].

Ideally, the HOV lanes and general lanes should be separated by a physical barrier. Where physical barriers are impossible to implement, some type of lane delineation should be incorporated. Any flexible tubular markers that delineate the HOV lane should be closely spaced to prevent lane-change movements near the toll plaza.

Adequate merging distance should also be provided to the priority lanes where they rejoin the general traffic lanes after passing through the toll booths. HOVs given priority at the toll plaza should be allowed to pass through the toll booths with a minimum amount of delay.

When possible, special refuge areas of shoulders should be provided adjacent to the HOV lanes. Such areas aid both disabled HOVs and enforcement operations.

RAMP TREATMENTS: FREEWAYS

Preferential treatment can also be provided at entry and exit ramps on freeways. There are commonly two types of HOV treatments on ramps:

1. HOV bypass of ramp metering at on-ramps and
2. Exclusive on-ramps or exclusive off-ramps for HOVs.

Analysis

As a part of this research, 21 ramps on the Santa Monica, Golden State, and Harbor Freeways were investigated in detail. All ramps provided ramp-metering bypasses for buses and carpools of two or more persons. Also studied as a part of this research was an exclusive-reversible ramp for buses and carpools that connects the reversible lanes of I-5 with the Seattle central business district (CBD).

The exclusive HOV ramp project in Seattle did not exhibit any accident characteristics that could be directly assigned to the HOV treatment. Indeed, the exclusive use of the ramp probably enhanced the safety of the particular ramp, although comparative data were not available to test this suggestion.

On the Los Angeles area ramp-metering-bypass locations, the installation of HOV provisions increased the number of accidents. These accidents were generally concentrated at or near the interface between the ramp and the surface street. This appears to be directly related to the division of what was formerly a single ramp into two lanes. Because vehicles that enter the ramp from several surface street approaches have to divide into two lanes, some weaving can be expected and accidents can result from the somewhat unpredictable movements associated with entering vehicles. If the metered queue extends back onto the surface street, this safety problem is further compounded. In this event, HOVs trapped in the queue on the surface street may attempt erratic movements to bypass this temporary delay and move directly onto the ramp in the HOV lane.

Recommendations

Ramp-metering-bypass treatment can adversely affect safety. A number of recommendations designed to improve the safety of this HOV strategy are presented below.

Ideally, the HOV lane should be physically separated from the metered lanes, either by being constructed separately (thus having many characteristics of exclusive ramps) or by barriers. This is particularly important at the ramp entry. Shoulders should be provided to enable unintentional violators to pull off the traveled lane.

When separation is not possible, and if the ramp is long and has sufficient storage capacity, the HOV lane should be initiated after the entrance point so there is a single entry lane. This may, at times, delay HOVs, but it would largely eliminate the entry conflicts.

Sufficient distance should be provided for merging on the body of the ramp so that HOVs and general traffic can merge together and assume the same speeds prior to merging on the freeway.

The selection and designation of right or left lanes as the HOV lane is important, particularly at nonseparated ramp-metering-bypass installations. Consideration should be given to access to the ramp, position of signals vis-à-vis the stopped queue, and how the two lanes will merge. Specific guidelines cannot be given because of the diversity of site-specific parameters; however, the most important items to consider are summarized below.

1. The preferred configuration is to have the HOV lane on the left because this allows the slower metered traffic to merge with HOV traffic on the left. This technique provides general traffic with a customary merging situation and eliminates the problem of drivers in the general lane being wary of traffic on both sides (a violation of driver expectancy).
2. If metering signals are pole-mounted, the preferred lane for metering is the left, so that drivers have a better view of the signal. If the right lane is the metered lane, consideration should be given to providing a narrow median with a signal installed both in the median and on the right. Adequate lighting, reflectorization, channelization, and strict application of MUTCD policies are needed to prevent collisions with the median or signal standard during hours of darkness.
3. On curved ramps, the HOV lane should generally be on the outside of the general lane (i.e., the lane having the larger radius). This gives nonstop HOVs a lower degree of curvature but, more importantly, metered-lane traffic has a clearer rear view of the HOV lane and thus the hazard of lane changing is reduced.

Metering rates, queue lengths, and HOV operations should be reviewed on a continual basis to optimize the operation of the ramp and minimize traffic problems.

Although potential safety problems are associated with ramp-metering-bypass installations, exclusive HOV ramps have not been shown to have an adverse impact on safety. Specific recommendations to enhance this position include the following.

Construction of new ramps or conversion of existing ramps is recommended. The addition of ramps generally has a minimal effect, since they do not result in substantially altered traffic patterns. Converted ramps can displace a significant amount of traffic because not all former users can, or will, shift to HOVs. This displacement places a burden on the main-line freeway and ramps at other interchanges. Thus, HOV ramp locations should be carefully selected, and consideration should be given not only to the access needs of the HOVs but also to the resulting adverse impacts.

The intersection with surface streets is of particular concern for HOV ramps. This is especially true if the ramp is reversible. Wrong-way entry can be a problem on these ramps, and traffic controls must be absolutely positive in displaying the proper usage. Changeable message signs, traffic-actuated stop signs, and time-control static signs are generally necessary to identify authorized users and time.

SEPARATE FACILITY: ARTERIAL STREET

Separate facilities on an arterial street system are commonly referred to as transitways because the only type of vehicle that is permitted to travel on such a facility is the transit coach. There are two types of transitways; each serves a distinct objective:

1. A separate facility that serves as a major transit collection-distribution route tends to be located in the CBD in order to provide a high level of transit accessibility in heavily concentrated retail and business districts. Commonly associated with this transitway is some type of pedestrian mall and other aesthetic features. The benefits of this type of transitway are transit accessibility and separation of different classes of vehicles.
2. A separate facility that serves the line-haul portion of transit service tends to connect the CBD with outlying areas. The benefits associated with this type of transitway would be the more-traditional HOV objectives of savings in travel time and increased total person throughput.

The predominant type of arterial-based transitway satisfied the first objective of major transit collection-distribution functions in the CBD. Such transitways exist in Minneapolis, Minnesota (Nicollet Mall); Portland, Oregon (Portland Mall); Chicago, Illinois (Halsted and 63rd Streets); and Philadelphia, Pennsylvania (Chestnut Street) and have been successful in enhancing or reviving downtown vitality.

Analysis

Most arterial transitways have an elaborate pedestrian mall associated with them. Numerous aesthetic features, commercial characteristics, special exhibits, displays, and public entertainment provide visual attractions to the pedestrian.

Access and egress to the separate facility most often occurs only through the facility's terminal points even though the facility will most likely traverse at-grade intersections with cross streets. Access and egress are controlled at the cross-street intersections through both traffic restrictions and possible supportive geometrics such as a low-curvature radius that does not allow for the American Association of State Highway and Transportation Officials' (AASHTO's) 24-ft (7.2-m) minimum turning path for a passenger car. Terminal treatments for a separate facility can vary considerably because the treatments are site specific.

The overall safety experience of arterial-based transitways has been excellent. The general practice of eliminating access and egress to the facility at all intermediate locations greatly enhances safe operation.

The greatest potential safety problem relates to pedestrian conflicts. Pedestrians sometimes unwittingly step into traffic lanes (especially at cross streets) because they become acclimated to a continuous pedestrian mall and become distracted by its attractions.

Recommendations

Specific recommendations that address this and other potential safety problems of arterial-based transitways include the following.

Appropriate pedestrian controls should be instituted. These controls should include highly visible and audible pedestrian signals at locations where cross-street vehicular traffic intersects with the pedestrian walkway.

Cross streets across the transitway should be eliminated whenever possible. When the elimination of cross streets is impossible, the turning movement between the transitway and the cross streets should be restricted. Traffic signals and signs should be standard and easily visible to the motorists. A one-way cross street is preferred to a two-way cross street because of the fewer potential conflicts and traffic operational requirements.

Procedures regarding bus operations on the transitway should include (a) low bus speeds and (b) increased driver awareness and courtesy. A low bus speed should not detract from the bus operations because the prime advantage of the transitway is its accessibility, and that is not affected.

CONCURRENT FLOW: ARTERIAL

Concurrent-flow priority applications on surface streets involve reservation of either the curbside lane or the median lane for HOVs. The different applications have differing operational objectives and requirements. Curbside lanes have historically been installed to provide better transit circulation in the CBD or to improve downtown traffic flow through segregation of buses and automobiles. Median lanes are generally intended to provide HOVs with travel time advantages by bypassing traffic congestion in the general traffic lanes. This type is commonly associated with express bus service that operates in a through or express mode. The concurrent-flow median lane usually operates during the peak period in the peak direction, over a project length of several miles. Carpools may also be permitted to travel in the concurrent-flow median HOV lane.

Analysis

Within the context of this research, four concurrent-flow HOV lane applications on arterial streets were examined, including three median-priority-lane sections and one curbside-priority-lane section. These projects include the Washington, D.C., CBD curbside-concurrent-flow lanes, the US-1-South Dixie Highway median-adjacent concurrent-flow lane, the N.W. 7th Avenue median-adjacent concurrent-flow lane, and the Kalanianaole Highway median-adjacent concurrent-flow lane.

Of the three median-adjacent HOV projects, the N.W. 7th Avenue project experienced a decrease in the total facility accident rate with the introduction of the HOV lane. The primary operational factor that differed between the N.W. 7th Avenue project and other median-adjacent projects was the establishment of the bus-only lane without altering the number of lanes available for general traffic. The other two median-adjacent HOV lane projects (US-1-South Dixie Highway and Kalanianaole Highway) established the HOV lane by taking a lane away from the general traffic, thereby increasing congestion in the remaining lanes. As a result of the congestion, rear-end accidents became more prevalent.

Other safety problems associated with median-adjacent HOV lanes related to left turns from the main line and speed differentials between the HOV lane and general lane. Left turns from a main-line facility that has an HOV lane may create a safety problem due to motorists that stop in the express HOV lane to make the left turn or weave unexpectedly across the HOV lane into a left-turn bay.

A large speed differential between the HOV lane and adjacent general lanes causes slower vehicles to merge into a high-speed HOV lane, or it causes faster vehicles in the HOV lane to decelerate rapidly to merge into the general lane. Either action could result in sideswipe or rear-end accidents.

For curbside HOV applications, buses that use the Washington CBD curb-bus-lanes project experienced parked vehicle (14 percent) and pedestrian (2 percent) accidents in addition to the more common rear-end (27 percent), sideswipe (25 percent), and right-angle (25 percent) accidents. Recommendations useful in alleviating the conditions that contribute to various accident types are listed below.

Recommendations

Taxicabs and other vehicles should be prohibited from stopping in the curb lane to pick up and drop off passengers or to make deliveries. This can be done by posting NO STOPPING OR STANDING regulations and strictly enforcing them.

Parked vehicles should be removed from the curb lane. The technique of putting locked boots on parked vehicles in order to ensure payment of the parking fine has the effect of keeping the parked vehicle in the lane longer.

The potential pedestrian safety problem should be addressed. Several options may be considered:

1. Strict enforcement of jaywalking ordinances,
2. Special visual or audible warning devices installed on the buses,
3. A special yellow stripe of 1-2 ft (0.3-0.6 m) with a warning message painted on the sidewalk adjacent to the curb, and
4. Planting bushes to keep pedestrians away from the curb.

A similar set of recommendations has been compiled for median-adjacent concurrent-flow HOV lanes on arterials.

Left turns should be prohibited at selected locations, if not at all locations. Nonsignalized intersections should be closed by cones, or other implements should be considered to reduce crossing movements across the HOV lane. The operational effect of this recommendation on the cross street or off-line will vary by location.

The speed differential between the HOV lane and general-use lanes should be controlled if necessary and possible. This may be accomplished by using variable-speed-control signing on the HOV lane. Until additional research can be conducted to quantify an optimum speed differential, it is recommended that a 10-mph (16-km/h) maximum speed differential not be exceeded. On each of the concurrent-flow projects studied, the average speed differential did not exceed 10 mph.

Volumes in the HOV lane should be high enough to portray the lane as an operational lane. The higher the volume in the HOV lane, the more keenly alert are motorists to the HOV lane. Increased volumes can be achieved by greater bus use or by permitting carpools to use the HOV lane.

CONTRAFLOW LANE: ARTERIAL STREET

A contraflow HOV lane on an arterial street is commonly a lane in the off-peak direction reserved for HOVs that are traveling in the peak directions. A contraflow HOV lane can incorporate the median lane or the curb lane of a highway facility. The reversible lane is a specialized type of contraflow lane in which the direction of flow for a median lane is always in the peak direction. Left turns are generally permitted from the lane in the off-peak period.

Analysis

The N.W. 7th Avenue reversible-bus-lane section is an example of a reversible lane. The US-1-South Dixie Highway and the Kalanianaole Highway contraflow lanes entailed the dedication of a median-adjacent lane from the off-peak direction for exclusive use by buses. Both the Kalanianaole and US-1-South Dixie Highways are six-lane, divided facilities.

A third type of contraflow operation is exemplified by Ponce de Leon-Fernandez Juncos Avenues. These four- and five-lane, one-way pair arterials contain a contraflow lane along the left-hand curb to provide local bus service along a 13.6-mile (21.9-km) route.

More than 70 percent of the accidents that involve a vehicle from a contraflow lane were associated with a crossing maneuver of some type by the other vehicle involved. These crossing maneuvers may involve (a) a vehicle turning left from the main facility, (b) a vehicle crossing or turning onto the main facility from the side street, and (c) a pedestrian crossing the main facility. The overwhelming causative factor cited by project officials for the occurrence of these contraflow-lane accidents that involve crossing maneuvers is the inability of motorists or pedestrians to recognize a facility's wrong-way operation. Therefore, when performing crossing movements, these individuals may scan for traffic in the direction of the general lane and fail to look for contraflow traffic. These perceptual deficiencies occur because the design of contraflow facilities violates basic driver expectancy based on the following two human factors:

1. The normal symmetrical lane-use distribution, which a driver encounters in nearly all driving experience, is violated by the nonsymmetrical layout that accompanies the contraflow facility and
2. Traffic control devices (signing and marking) used for standard delineation and positive guidance are often superseded by temporary peak-period traffic control measures that define the contraflow lane; however, the motorist or pedestrian may continue to behave in a manner responsive to the permanent traffic control devices.

The omnipresent safety hazard associated with this expectancy phenomenon is dramatically documented on the US-1-South Dixie Highway project. On this project, two contraflow-lane accidents have involved police officers responsible for project enforcement. The officers, while in pursuit of violators of the project's restriction, turned directly into the path of oncoming contraflow-lane buses. The officers, who were very familiar with the operation of the contraflow lane, simply forgot for the moment about the contraflow lane.

There may be an indirect relationship between vehicular volume in the contraflow lane and the accident rate. In other words, the higher the volume, the lower the accident rate. This relationship could result from the fact that motorists are more keenly aware of the contraflow lane due to a higher volume in the contraflow lane. A greater number of vehicles in the contraflow lane provides greater visibility to the motorists of the contraflow-lane operation.

All four contraflow-lane projects experienced bus accident rates that were higher during the early stages than during the later stages of each project. Such accident rate trends may suggest that there is an adjustment period of some duration for the motorists driving the facility to better comprehend the contraflow-lane operation. In other words, driver expectancy may improve with the life of contraflow-lane projects up to a point of optimal driver familiarity.

The contraflow HOV lane treatment is potentially one of the most hazardous priority treatments that can be implemented on an arterial street. On the other hand, it is possible to employ this treatment effectively and safely provided certain precautions are taken.

Recommendations

Specific recommendations that may improve the safety of a contraflow HOV lane on an arterial street include the following.

Left turns should be prohibited at all locations along the contraflow-lane operation. This prohibition should also be considered for the off-peak periods. Any left-turn prohibition should be enforced rigorously. Left-turn prohibitions should be reinforced by physical impediments where possible.

Traffic control devices (signing and pavement markings) should be provided that are highly visible and closely spaced in order to make the motorists more fully aware of any restrictions imposed. The issue of driver expectancy is more pronounced for a median contraflow-lane treatment than for a curb contraflow-lane treatment. In addition, on a median-lane treatment, driver expectancy tends to be greater for a divided facility than for an undivided facility.

The demarcation for a contraflow lane should be a double yellow broken line to indicate a reversible lane. Flexible tubular markers should be placed along the lane line. They should be reflectorized and spaced at a maximum distance of 40-ft (12-m) intervals. The use of the diamond symbol on the contraflow lane is discouraged, as this implies vehicle classification and not direction.

Signing in the off-peak direction approaching the contraflow section should consist of both advanced-warning and restricted-lane signing along the main line. Messages such as CAUTION--ONCOMING TRAFFIC AHEAD--X FEET (Y KM) and LEFT LANE CLOSED—ONCOMING TRAFFIC, with flashers and merge-right arrows as appropriate, are more positive than the standard MUTCD restricted-lane signing. Blank-out message signs are preferable to specified time periods due to the flexibility in operating hours.

Signing in the off-peak direction at the end of the contraflow section should be the standard MUTCD end-of-HOV-lane sign (3). A lane-control signal should be placed downstream with all green arrows permanently displayed over each off-peak directional lane.

Signing in the peak direction would depend on the type of terminal treatment. Standard MUTCD signing should be used with emphasis on which vehicles may use the contraflow lane.

The imposition of additional restrictions on contraflow-lane and opposing-lane traffic may be desirable. Reduction of the speed limit and spatial headways are the most common restrictions. A lower bus headway may make the motorists more aware of the operation of a contraflow lane. A bus headway of 0.5-1 min may be necessary to accomplish this objective. For many express bus operations, it may not be financially feasible to operate with headways of 0.5 min. In view of this and the evidence in support of lower accident rates where HOV lane volumes are higher, consideration may be given to including registered carpools, vanpools, taxis, or other multipassenger vehicles in the HOV lane.

Warning horns or flashing lights could also be used on buses that travel in the contraflow lane. This would improve awareness of the contraflow-lane operation.

Potential provisions that may alleviate, in part, the pedestrian safety problems are as follows:

1. Strict enforcement of jaywalking ordinances,
2. Pedestrian signing and markings that state LOOK BOTH WAYS at designated crosswalks,
3. Special visual or audible warning devices installed on contraflow-lane buses,
4. Special yellow stripe of 1 to 2 ft (0.3 to 0.6 m) with a warning message painted on the sidewalk adjacent to the curb, and
5. For median contraflow projects that have a divided median, application of a combination of fencing and foliage in the median should be provided to obstruct and channel the pedestrian traffic to particular locations equipped with pedestrian signals.

In order to speed up the motorist familiarization process with the operation of a contraflow lane, an intense public education campaign and heavy enforcement of the contraflow-lane restrictions should be undertaken from the onset of the project.

Quick-reaction incident detection and removal systems should be incorporated into the project to minimize the potential for vehicles using oncoming lanes to bypass breakdowns in the contraflow lane.

SUMMARY

This paper has been a summary of an exhaustive nationwide research effort aimed at improving the design and operations of preferential-treatment strategies for HOVs. Potential users of this material are directed to the references for more detailed data and recommendations.

In general, all preferential-treatment strategies can operate in a safe and effective manner. However, since many HOV strategies differ somewhat from normal driver expectancy, considerable effort must be expended during the HOV facility-design phase to ensure safe operation. HOV facility design, traffic operations systems, and enforcement strategies all require careful and comprehensive consideration. Exhaustive attention to operational details as well as to the full range of possible misinterpretations and misuses of the HOV system is esssential.

Certain HOV strategies are inherently more contrary to driver expectations and, hence, pose greater safety problems. Contraflow lanes on arterial streets are an example of potentially unsafe HOV strategy (although for reasons of right-angle collisions rather than the normally feared head-on collisions). Concurrent-flow HOV lanes on freeways are another example.

With proper design, analyses, and attention to details, all forms of exclusive HOV facilities (physically separated HOV systems, contraflow lanes, concurrent-flow lanes, ramp-metering-bypass facilities, exclusive toll-plaza lanes, and bus-priority signalization systems) can be implemented successfully without adversely affecting the safe operation of the transportation facility.

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