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Isometric Projections and Other Study and Display Methods Used in Preliminary Design of I-70 Through Glenwood Canyon

JOSEPH R. PASSONNEAU

The designers of I-70 through Glenwood Canyon, Colorado, were responsible to reviewers from many backgrounds. All wanted to find a good fit between highway and natural landscape, but their definitions of "good fit" varied. Accurate graphic descriptions of the canyon and of alternative highway proposals were important to designers and reviewers. These were needed to show conventional highway plans, profiles, and sections; to show the appearance of the highway in the natural landscape; and to show the precise relation between highway alternatives and landforms and plant communities taken or impinged upon. Canyon and highway alternatives were described by using conventional artist's sketches, colored slides and black-and-white photographs, surveyed cross sections combined with perspective backgrounds, environmental maps, isometric drawings, composite drawings or "story boards", computer graphic cross sections at close intervals, highway representations painted into photographs, scale models, full-size mock-ups, and even diagrams and cartoons. An isometric projection technique developed for the project was particularly helpful in the design of the highway in the western half of the canyon. Both landscape and highway alternatives can be drawn in isometric with engineering accuracy and combined, with much of the realism of an architectural perspective. Highway designers and reviewers have traditionally come from similar backgrounds and have communicated through familiar methods. The Glenwood Canyon work was typical of recent projects in which unusual proposals were reviewed in a process more like a New England town meeting. In such a setting, the ability to communicate lucidly, in words and images, is an important aspect of highway engineering.

In the design of I-70 through Glenwood Canyon, Colorado, alternative proposals were examined in a series of iterations, each of which lasted about a month. The work was reviewed frequently by the district engineer and the project engineer for the Colorado Division of Highways (CDH), by representatives of other agencies and a Technical Review Group, and, finally, by representatives of all of these groups and the seven members of the Citizens Advisory Committee for Glenwood Canyon.

The design of a highway in mountain terrain is the problem of fitting a ribbon of concrete and asphalt, shaped by its own precise geometric rules, to a natural landscape shaped by the very different rules of geology. Whereas each of the reviewers was searching for a good fit between highway and landscape, their definitions of "good fit" varied dramatically and sometimes conflicted. The participants in this process brought many points of view to the work, and the designers were asked to investigate a large number of unusual alternatives. The presentation of these alternatives had to meet the following requirements:

1. Horizontal and vertical location, superelevation, and cut and fill were to be shown in conventional plans, profiles, and sections.
2. The appearance of the road was to be shown in its surroundings, unambiguously, to many people who were skeptical about both artists' renderings and engineering drawings.
3. The relation of each proposal to rock formations, talus slopes, watercourses, plant communities, and individual trees was to be shown precisely, and drawings were to identify each natural feature displaced or impinged upon.
4. Because the work, from start of design through the design public hearing, took just nine months, alternatives were to be described not through elaborately contrived presentations but by using drawings peeled from the drafting boards at the beginning of each meeting.

This paper first summarizes study and display techniques used in the design of I-70. It then describes in detail an isometric method of drawing highway alternatives in a natural landscape that was developed for this project.

TECHNIQUES USED IN DESIGN OF I-70

Conventional Sketches of Existing Landscape

Conventional sketches of the existing landscape were used by the designers to educate themselves. Figure 1 shows an example of such a sketch of the canyon, looking west from just above the location of the recommended alignment of the westbound roadway. These sketches emphasized the plant communities typical of different slopes and elevations and showed, in ways the camera could not reveal, that the canyon is a sequence of great outdoor rooms with transitions that could be dramatized by highway design. The sketches also showed that the appearance of the canyon changes with the elevation of the viewer and that, from as little as 100 ft above the river, the traveler's view can be breathtaking. Sketches emphasized the damage that has been done to the river's edge by earlier road building and that a well-designed new road could partly correct this damage.

Colored Slides and Black-and-White Photographs

Colored slide transparencies and black-and-white photographs also helped designers to understand the canyon and were essential in illustrating issues for reviewers and laypersons. They were also used in preparing perspective drawings of alternative designs. Figure 2 shows black-and-white reproductions from color slides used in the design public hearing, showing the different character of the western and eastern halves of the canyon.

Terrain Cross Sections

Terrain cross sections, with the natural background drawn in perspective, were prepared from a viewpoint about 25 ft above the existing road. These sections were taken at stations 1000 ft apart, at more than 60 locations, as well as at intermediate stations that presented special problems. The sketches were drawn from life and from photographs taken from a high lift. These perspective cross sections were used as base drawings for preliminary studies of highway alternatives. It was an advantage of these drawings that a large number of proposals could be sketched very rapidly and the alternatives that were most interesting could be examined at each location in the canyon. These drawings could be made precisely accurate at the cross-section line, but beyond this line accuracy depended on the skill of

Figure 1. Sketch of Grizzly Creek in Glenwood Canyon.

Figure 2. Black-and-white photographs of Glenwood Canyon showing (left) deep, V-shaped western section and (right) shallower, U-shaped eastern section.

Figure 3. Perspective cross sections used in preliminary studies: (top) immediate separation design and (bottom) maximum separation (Shoshone Powerhouse).

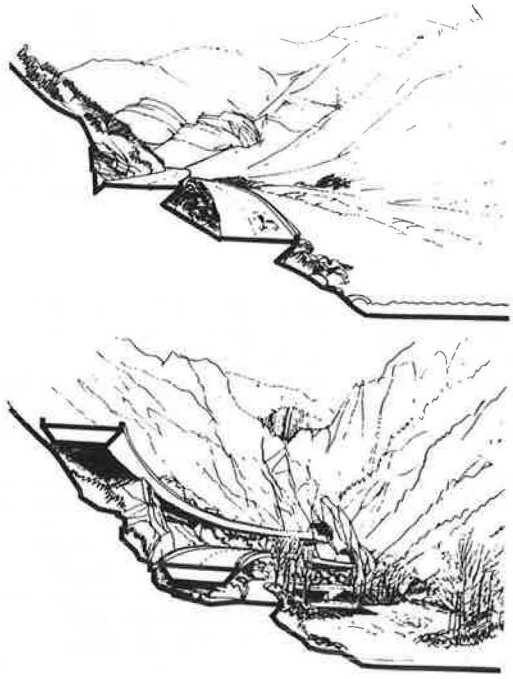


Figure 4. Section from environmental strip map just west of Shoshone Dam.

the artist. This technique was therefore useful for preliminary studies but not for design development. Figure 3 shows examples of these drawings.

Environmental Strip Map

An environmental strip map was prepared, in color (at 1 in = 100 ft), on dimensionally stable mylar. This map showed the four distinctly different kinds of plant communities found in the western half of the canyon, three kinds of rock formations, talus slopes, rock rivers, watercourses, and all man-made elements (buildings, roads, railroads, and the dam) plus scars in the landscape (cut talus slopes, cut rock faces, trails, and mine tailings). A section of this map near the Shoshone Dam is shown in Figure 4 at reduced scale. Highway alternatives were then drawn on transluscent mylar (at 1 in = 100 ft); cut and fill slopes were shown in plan and overlaid on the strip map. The composite plan drawing showed, for each alternative, the effect of highway construction on landforms and plant communities.

I consider such strip maps to be, in urban work, the essential reference documents; one client described them, not entirely facetiously, as the "magic maps" and the "sacred scrolls". But in the Glenwood Canyon work the design problem was so emphatically three-dimensional that these two-dimensional reference drawings were not very helpful. This was unfortunate, since the amount of work involved in locating so much information so accurately is formidable.

Isometric Drawings

Isometric drawings of existing conditions were used as base drawings on which were overlaid isometric drawings of highway alternatives. These composite drawings, which were also helpful in describing alternatives to reviewers, were to the designers the

most useful of all graphics tools used in studying the western half of the canyon. (The low investment return on time spent in preparing the environmental strip maps was offset by the high return on time spent in preparing isometric base maps at 18 different locations in the canyon.) These drawings are described in the final section of this paper.

Composite Presentations

Composite presentations were used to describe particular issues. Figure 5, for instance, shows a section of a large "mural" study of the Colorado River's edge. This mural showed existing conditions of both natural and damaged landscape, as well as possible relations between highway edge and riverbank, and examined possibilities for at least partly restoring the river's edge to its original natural condition. The drawing combined sketches from nature; diagrams; plans, profiles, and sections; text; and perspective drawings of various alternatives. Such a drawing is a useful "story board," summarizing the thinking of designers and presenting a complex subject.

Computer Graphic Cross Sections

Computer graphic cross sections were useful--essential, in fact, in the western section--in the final stages of design development and in "fine tuning" the selected alignment. The initial preparation of computer graphic cross sections is laborious, time-consuming, and expensive. Terrain cross sections must be entered, highway templates must be prepared and entered, and each alignment to be printed out must be mathematized and entered. But, once initial preparation is complete, a number of iterations can be examined accurately and more rapidly than sections can be prepared by hand. The earthwork programs used as the basis for the Glenwood Canyon printouts were also adapted to printing computer perspectives, on either an oscilloscope or a Cal-Comp plotter. Figure 6 shows two typical sections.

Composite Photograph-Drawings

Composite photograph-drawings, in which highway alternatives are painted in tempera into a photograph, can be made to look, to a layperson, like a photograph taken from "life". Because computer perspectives are unforgiving, the combination of photographs and computer perspectives is particularly reliable. The photograph and the perspective must be taken from the same point in space. This technique was most effective in presenting alternatives in the eastern half of the canyon, where particular views from particular locations were important in alignment comparisons. Many of the reviewers considered views from the driver's eye level to be the most significant views. The composite photograph-drawing is a good technique for displaying such views. This procedure has the disadvantage of being too laborious for study purposes and is most useful in comparing well-defined alternatives. Two alternatives for Blue Gulch are shown in Figure 7.

Full-Scale Mock-Ups

Full-scale mock-ups, one of which is shown in Figure 8, were constructed to test critically important proposals in difficult sections of the canyon. To many of the reviewers, these were the most useful of all of the study techniques. Such mock-ups are obviously expensive in money and time.

Small-Scale Sectional Model

A small-scale (1 in = 100 ft) sectional model was

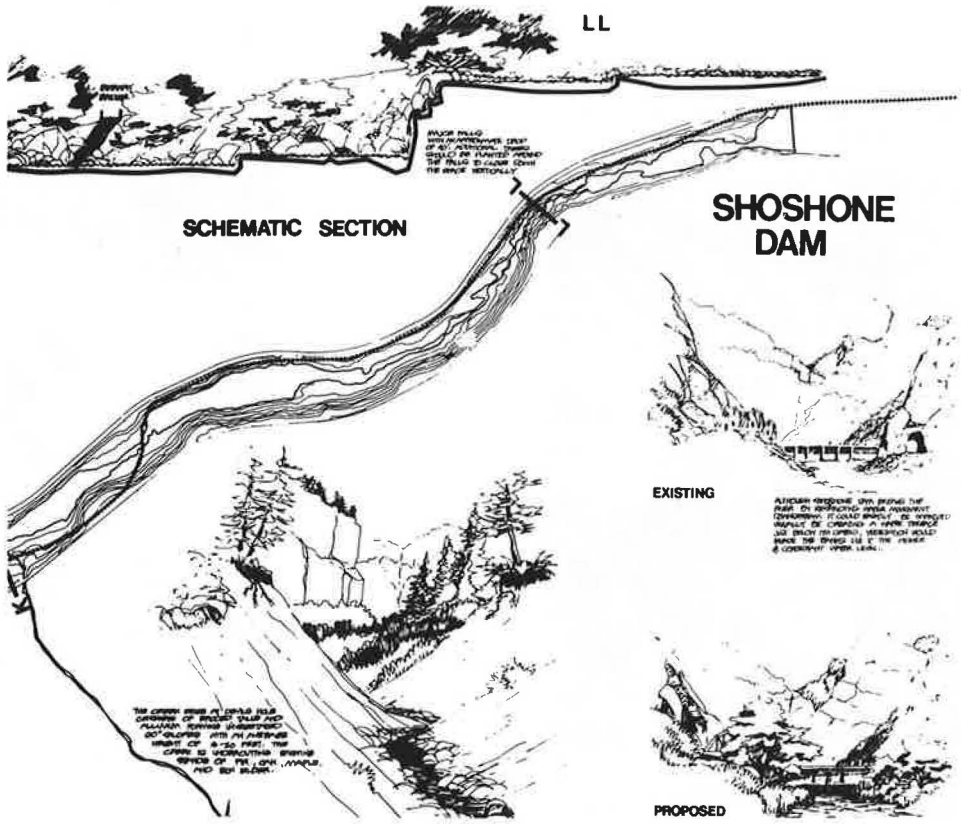


Figure 5. Story board from river's edge study.

Figure 6. Computer graphic cross sections of recommended alignments: (top) near No Name Creek and (bottom) near West End.

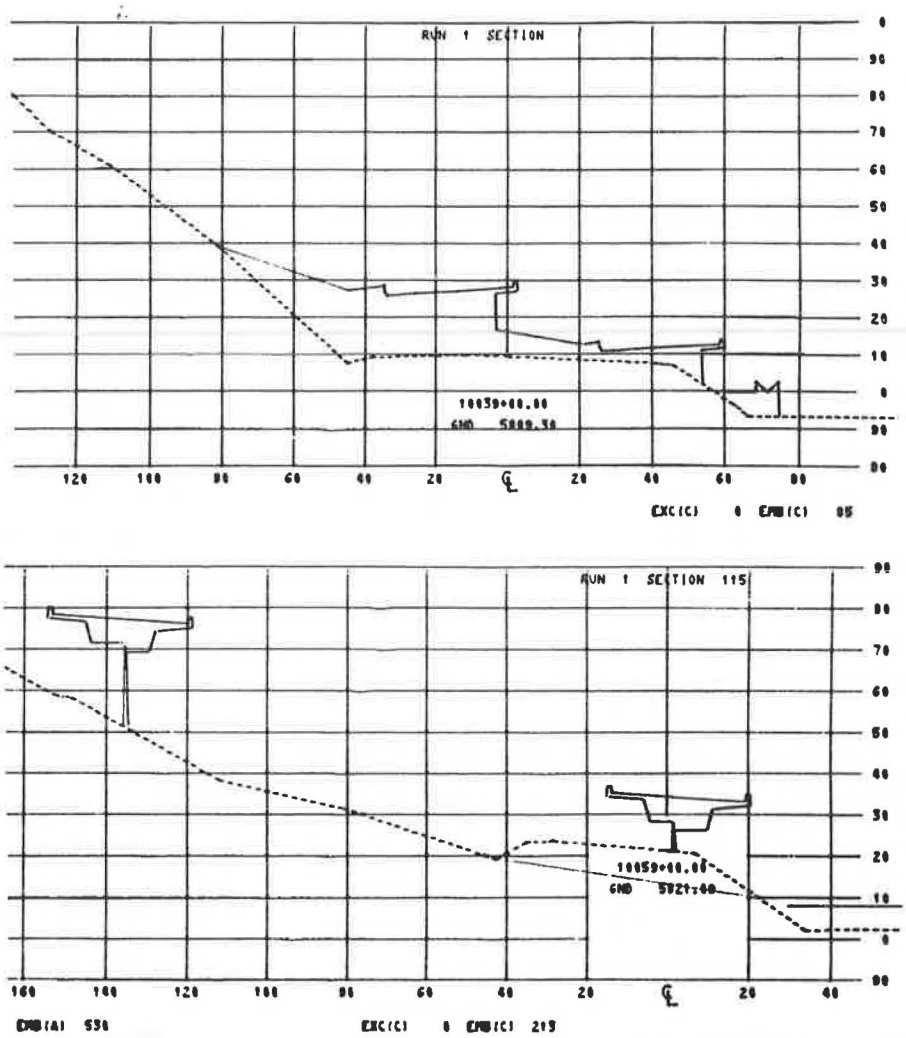


Figure 7. Composite photograph-drawings of Blue Gulch alternatives.

Figure 8. Full-scale test mock-up east of Hanging Lake.

prepared by the eastern design group to illustrate alternative proposals at Hanging Lake, just east of Shoshone Dam. The model could be assembled in three different ways to show the particularly complex solutions for this important section of the canyon. Viewers (who would get down and squint) could see the appearance of the canyon from many eye-level viewpoints; the model also showed the overall character of each proposal. Such scale models and the study models that precede them are helpful to both designers and reviewers. They often provide

Figure 9. Diagrams of highway alternatives west of Shoshone Dam.

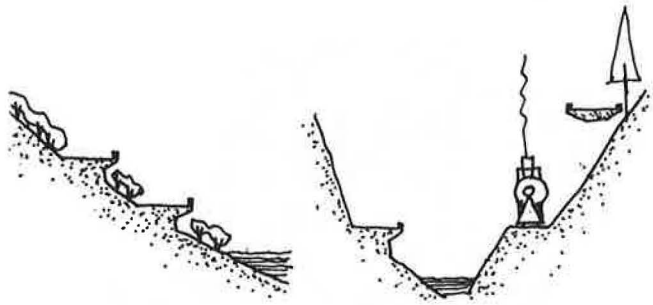


Figure 10. Three ways of drawing a rectangular solid: (top) descriptive geometry, (middle) perspective, and (bottom) isometric.

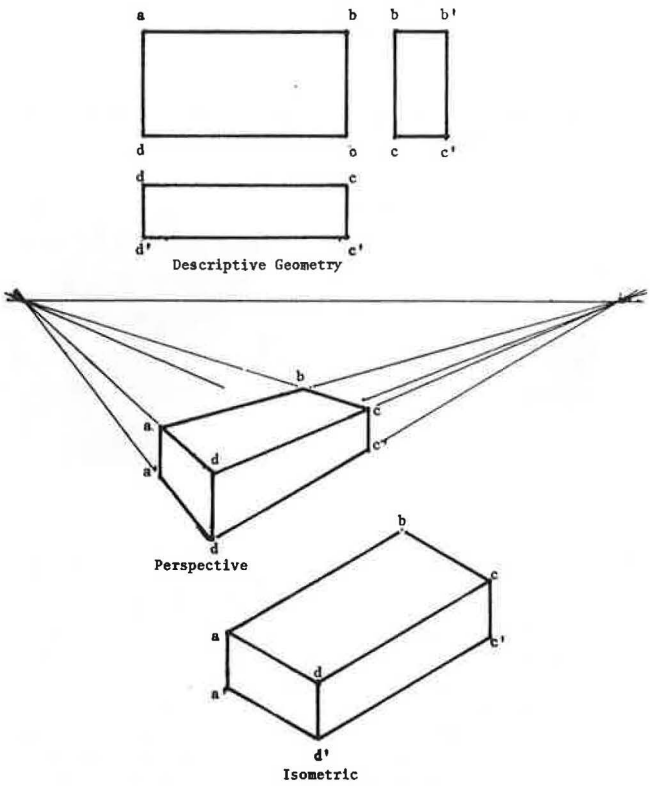


Figure 11. Isometric armature of Bear Creek: (top) terrain and (bottom) existing conditions.

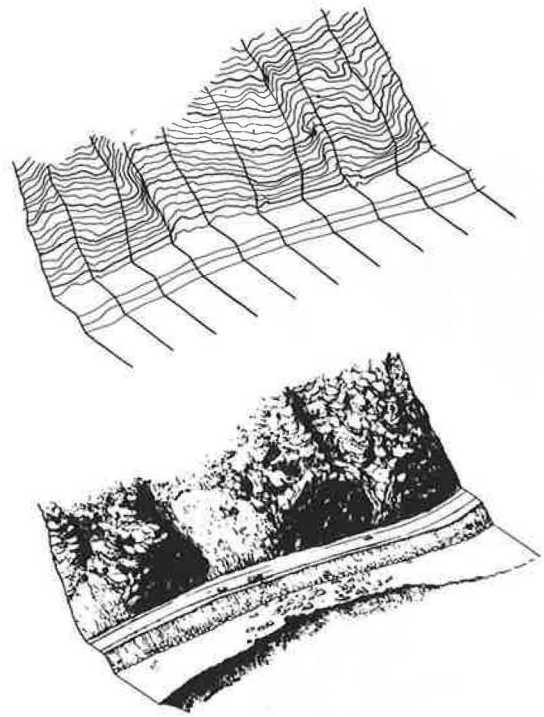
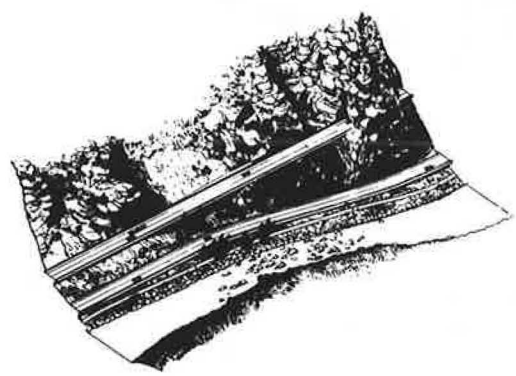


Figure 12. Isometric projection of alignment at Bear Creek recommended in design public hearing.



the most effective way to present complex three-dimensional problems to laypersons. They are generally extremely expensive.

Cartoons

Diagrams, or "cartoons", summarizing and simplifying complex issues were presented frequently, in chalk on blackboards and colored markers on sketch pads. This simple technique was used naturally in meetings in which ideas were exchanged and proposals criticized in round-table seminars. Such diagrams were also used in more formal meetings. Figure 9 shows sketches used in the design public hearing to summarize highway location possibilities in the western half of the canyon.

ISOMETRIC DRAWINGS AS A TOOL FOR STUDYING HIGHWAY ALTERNATIVES IN A NATURAL LANDSCAPE

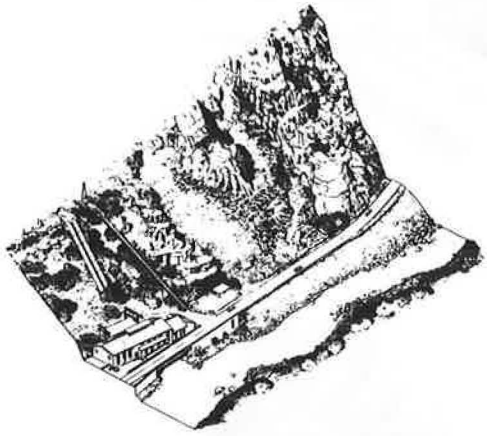
Isometric drawings have the following characteristics:

1. Highway and landscape can be drawn with engineering accuracy.
2. Such drawings have much of the realistic character of architectural perspective sketches.
3. The preparation of base drawings of existing conditions is a laborious process but, once base drawings are completed, prints can be made for as many alternative studies as the problem demands.
4. The extent of the area to be studied is not limited by the projection technique (as with perspective drawings); because the scale remains unchanged throughout the drawing, it can be extended as far as the size of the paper and the energy of the draftsman permit.

Figure 10 shows (a) a rectangular solid drawn in plan and elevation by using the basic technique of descriptive geometry, (b) the same solid drawn in perspective, and (c) the solid drawn in isometric projection. In this particular isometric projec-

Figure 13. Isometric projection of original conditions near Shoshone Powerhouse.

Figure 14. Isometric projection of existing conditions near Shoshone Powerhouse.



tion, angles b a d and b c d, which are right angles in reality, are drawn as 60° angles. Angles a d c and a b c, which are right angles in reality, are drawn as 120° angles. Therefore, the rectangular cube is distorted in the drawing to a diamond shape, somewhat as it appears in perspective. However, all dimensions scaled along lines parallel with the x, y, and z axes are true scalar dimensions.

In an isometric projection, all lines that are parallel in reality remain parallel in the projected drawing; in a perspective drawing, they converge.

Steps in Development of Isometric Drawings

The development of isometric drawings for highway studies proceeds in three steps.

In step 1, an isometric armature is prepared. Terrain cross sections are drawn in isometric convention, parallel with the x y plane of the projection. Contours are then drawn in isometric convention, parallel with the x z plane. The result, shown at the top of Figure 11, is the isometric armature of the terrain as surveyed. In the Glenwood Canyon isometric drawings, sections and contours were taken at 50-ft intervals and intermediate contours at 10-ft intervals.

In step 2, an artist sketches an isometric base drawing of the existing terrain over the isometric armature. In the Glenwood Canyon work, the artist climbed up and sat on a rock on the opposite wall of

Figure 15. Isometric projection of alignment alternative near Shoshone Powerhouse.

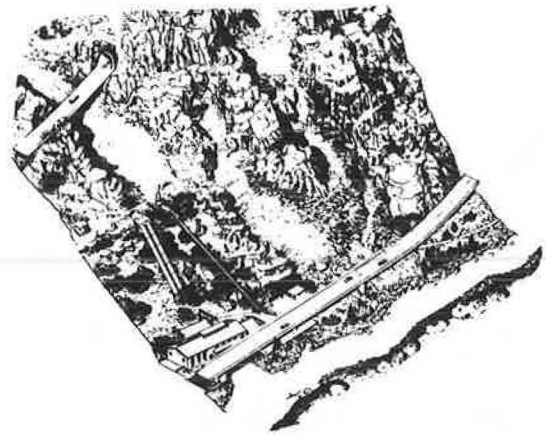
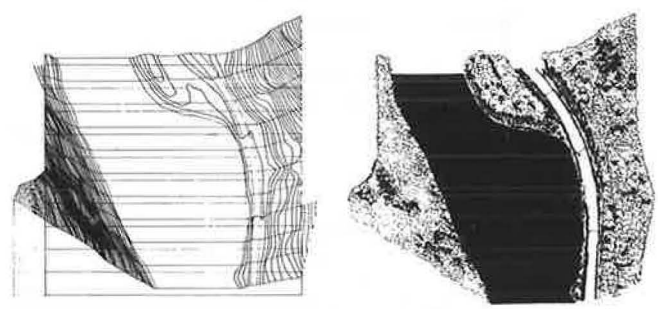


Figure 16. Profile Lake, Franconia Notch, New Hampshire: (left) axonometric armature and (right) axonometric projection.



the canyon, approximately 60° in plan and section from the center of the area covered by the armature. From here, he sketched the topography and vegetation over a print of the armature. Because the arid canyon vegetation is sparse, talus and rock surfaces could be accurately displayed. Stakes in the ground at the intersections of grid lines were used where the relation between terrain and armature seemed ambiguous. The bottom portion of Figure 11 shows the base drawing prepared over the armature. Prints were made from the base drawing as needed for the study of various alignment alternatives.

In step 3, isometric drawings of highway alternatives can then be prepared to the same scale as the base drawings, from conventional highway plans, profiles, and cross sections. These can be printed, cut out, and stripped into prints of the base drawing. Figure 12 shows the alignment alternative at Bear Creek that was recommended in the design public hearing.

Figure 13 shows an isometric drawing of the vicinity of the Shoshone Powerhouse, in which the original character of the canyon has been reconstructed with the help of late 19th century photographs. Figure 14 shows existing conditions at the powerhouse. These drawings emphasize that the choice is not between a primeval wilderness and a wilderness scarred by man but between two different combinations of wilderness and artifacts. Figure 15 shows one of the alternative alignments at the Shoshone Powerhouse.

An isometric projection is a perspective projection from a particular point of view--in the words of one of the reviewers, "from the viewpoint of an

eagle at infinity" (with superior eyesight). Therefore, the computer program could be modified to prepare the isometric armatures and the isometric roadway drawings from terrain data, mathematized alignments, and highway templates.

The 60°-120°-60°-120° isometric was tailored to studies of the western section of Glenwood Canyon because the highway alternatives were built on a canyon wall that sloped about 60° from the vertical. However, other conventions could be selected to fit other problems. For instance, in preliminary work on the Franconia Notch Parkway in New Hampshire, an axonometric armature was drawn (at 1 in = 50 ft) for the entire 12-mile length of the Notch. An axonometric drawing is an isometric in which the angles b a d, b c d, a d c, and a b c are right angles in reality (see Figure 10) and remain right angles in the drawing; that is, plan sections remain plan sections. Because there is a mantle of trees over the entire floor of Franconia Notch, a drawing convention was needed that would permit the viewer to look down into the channel cut through the woods to make room for the road. Figure 16 shows an axonometric armature of the terrain at Profile Lake, looking north, and a base drawing drawn over that armature, showing existing terrain and vegetation.

SUMMARY

Designers and reviewers of roadway designs have traditionally come from similar backgrounds, with homogeneous values and objectives. The Glenwood Canyon process was not conventional in that sense. It was typical of more recent projects to which a variety of participants have brought a variety of interests. In the Glenwood Canyon process, which has been described elsewhere from several points of view (1-3), design review was more like a New England town meeting. In such a setting, the ability to communicate lucidly, in words and images, is an important aspect of highway engineering.

My son, who is deaf, has grown up in the middle of a controversy between those educators intent on teaching deaf children to speak orally and those who favor manual communication by the deaf. One participant in this heartrending debate has put forward this proposition, which also summarizes one of the arguments of this paper: "People communicate in many ways. People communicate with sounds in many languages, with their hands, with facial expressions and body movements, and through writing and drawing--all in a myriad of subtle combinations. The important thing is simply to communicate in whatever way best serves the message, the sender, and the receiver, and the more ways that are available for communication, the more likely it is that the message will get through."

There is a second argument in favor of a "kit" of graphic techniques for designers wrestling with unusual highway problems. How can various locations of that ribbon of asphalt be described? How can they even be imagined? Fleeting images are not good enough. Alternative highway locations in relation to landforms must be postulated, recorded, and reflected on, with enough facility so that they can be discarded without regret, while the designer pushes on toward a better alternative. The need is so fundamental that it is often overlooked, and designers may at times be content with ways of thinking that are not up to the job.

The importance of means of communication is underestimated. The essential communication is with oneself. Without methods of communication, a person would have no access to history, could not compare experiences, could not combine abstract ideas in new permutations, and would therefore be cut off from the world of art and invention and design.

ACKNOWLEDGMENT

I was consultant to the prime contractor, Daniel Mann Johnson and Mendenhall, on the Glenwood Canyon I-70 project and was project director for the design group responsible for the western half of the canyon. Leigh Whitehead took the black-and-white photographs. Laurie Olin and Edgar Haag did the landscape sketches, and Edgar Haag drew the perspective cross sections and prepared the "River's Edge Study". The isometric base drawings were sketched by Leavitt Dudley. Gruen Associates was the prime contractor for the design of the eastern section; Edgardo Contini was the project director. DeLeuw, Cather, and Company advised the Colorado Division of Highways in the review of the work and was responsible for interchange and landscape design.

For the work at Franconia Notch, Daniel Mann Johnson and Mendenhall was the prime contractor, and I was project director as consultant to that firm. Kiley Walker was the landscape consultant, Tadihiro Kozawa prepared the isometric armatures, and Alistair McIntosh prepared the base drawings.

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Comparison of Two Integration Methods in Transportation Routing

JOCELYN F.B. SHAW AND BERNARD E. HOWARD

Two methods of integrating the only principle that produces an optimal transportation route are compared, where c is the criterion function whose path integral is to be minimized. (All quantifiable factors, including environmental factors, may be included.) The optimum curvature principle is a necessary condition that an optimal route (however obtained) must satisfy in any region where c is smooth. Classical routes such as linear, parabolic, and circular splines are approximations to optimal routes. An intrinsic-equation algorithm that may have the necessary smoothness is introduced and is compared with a previously presented arc-of-circle algorithm. In the example with known analytic solution, the arc-of-circle algorithm is an adequate approximation to the preferable intrinsic-equation algorithm, the latter of which reduces to the former in the case of constant curvature. The intrinsic-equation algorithm is an order of magnitude more accurate in the example and is preferable because it is easy to use and because the other algorithm does not satisfy the smoothness hypotheses. Discontinuities of the criterion function can be allowed.

This paper compares two integration schemes for establishing optimum transportation routes (referred to in this paper, for brevity, as highways) by the Optimum Curvature Principle (OCP). The OCP was described by Howard, Bramnick, and Shaw (1) and applied to a practical example by Howard and Shaw (2). For convenience, this paper describes the OCP. The first integration scheme uses a sequence of circular arcs joined together (as is sometimes done in highway routing), and the second uses a Taylor series expansion through cubic terms in which the path segments are joined together and there may be continuous curvature at the joints. The comparison reported in this paper is performed to investigate the practical significance of the possibly higher degree of smoothness of the second method. The hypothesis underlying the derivation of the OCP necessary condition by means of the calculus of variations implies the greater degree of smoothness. Error analyses are carried out in each case.

STATEMENT OF THE PROBLEM

The problem can be stated in two parts:

1. The mathematical problem is the numerical solution of a two-point boundary value problem in ordinary nonlinear differential equations, which represents the mathematical statement of the OCP described below. The practical problem is plan optimization of a highway between two given locations.
2. This paper is also concerned with studying a criterion field with a known solution for the optimum routes in order to establish the sensitivity and accuracy of the numerical integration schemes considered. Two integration algorithms are used: the new intrinsic-equation method introduced in this paper and Howard and Shaw's arc-of-circle method (2). Error analyses have been carried out (with the known analytic solution as standard of reference) to establish the correctness and the error bounds of the methods.

NOTATION

The following notation is used in this paper:

C^0 = segmented curve with discontinuous tangent;
C^1 = curve with smooth, continuous tangent;
C^2 = curve with smooth, continuous curvature;
c = $c(x, y)$ = criterion function at point x, y;
c = $\exp(0.05y - 0.2x)$ = equation of the exponentional cost function;
c_x = $\partial c/\partial x$ = partial derivative of c with respect to X;
c_y = $\partial c/\partial y$ = partial derivative of c with respect to Y;
c_{xx} = $\partial^2 c/\partial x^2$ = partial second derivative of c with respect to X;
c_{xy} = $\partial^2 c/\partial x\, \partial y$ = partial second derivative of $\partial c/\partial y$ with respect to X;
c_{yy} = $\partial^2 c/\partial y^2$ = partial second derivative of c with respect to Y;
c' = dc/ds;
$dc/ds_{\theta+\pi/2}$ = directional derivative of c perpendicular to route;
$E_d = \alpha h^k$ = discretization or truncation error in terms of step size h;
$E_r = \beta/h$ = round-off error in terms of step size h;
h = distance between adjacent points (step size);
K = $d\theta/ds$;
K_0, K_0' = K, K' values at point 0;
K' = $d^2\theta/ds^2$;
k = slope of log absolute error versus log step size on the coarse side of the mesh (large step sizes);
R = radius of curvature of optimum route;
s = distance along optimum route;
x, y = coordinates of the criterion function along X and Y axes;
x', y' = dx/ds, dy/ds;
δs = small increment in s corresponding to a small increase $\delta\theta$ in θ;
$\delta x, \delta y$ = small increments in x, y corresponding to increments δs and $\delta\theta$;
$\delta\theta$ = small increment in θ corresponding to a small increase δs in s;
θ = angle of the route with the positive X axis (rad);
θ_s, x_s, y_s = values of θ and coordinates at a point s along the optimum route;
θ_0, x_0, y_0 = initial values of θ_s, x_s, y_s;
θ_1, x_1, y_1 = values for the next point of θ_s, x_s, y_s; and
θ' = $d\theta/ds$.

OPTIMUM CURVATURE PRINCIPLE

Description

The OCP is illustrated in Figure 1. Let us take a case of a route (e.g., a highway) to be built between points C and D. The criterion field is as shown and represents construction costs per mile of highway. There is a high-cost area to the left between the two end points and a low-cost plateau to the right of the figure. Such a situation could exist where a depression in the ground (with marshy conditions) is in the high-cost area; this is surrounded by rising ground, the condition of which

Figure 1. General illustration of optimum curvature principle.

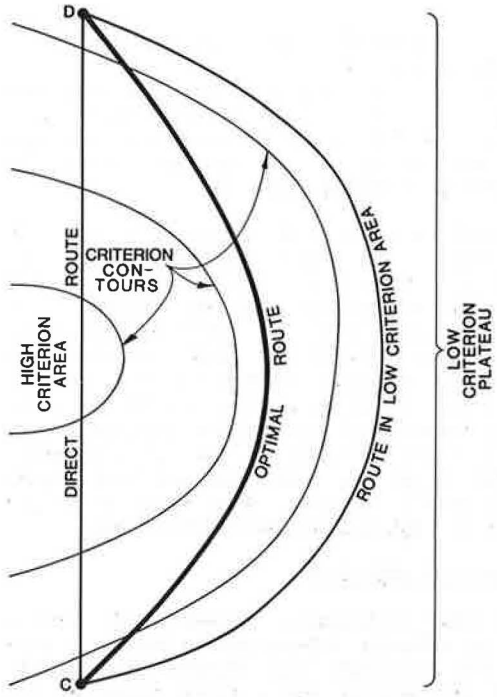
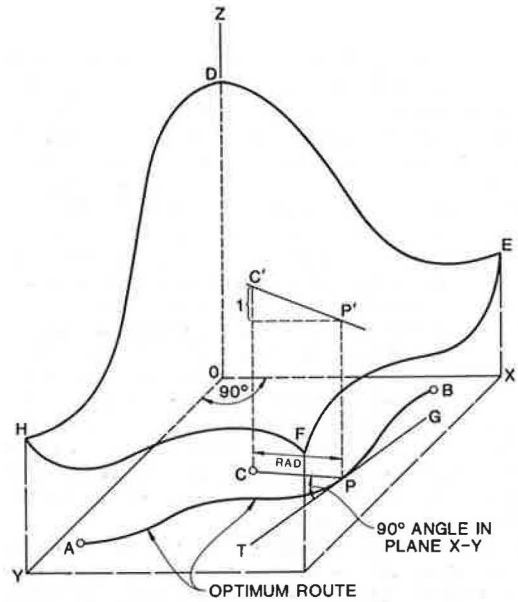


Figure 2. Specific illustration of optimum curvature principle.



gets gradually better, resulting in higher, drier ground. The low area would need an embankment to raise the roadway above flood criteria as well as suitable treatment for the marshy ground; this is surrounded by ground at gradually lower cost and eventually by good ground at constant low cost. There is a straight-line route passing through the high-cost area and a circuitous route in the low-cost area. Somewhere in between, there is an optimal route that is a compromise and that optimizes the path integral (total cost, in this case) of the route. The optimal route will be output by an OCP computer program that implements the OCP, provided that certain smoothness requirements of the criterion field are followed. Discontinuities of the criterion field can be allowed.

The OCP is described by Howard, Bramnick, and Shaw (1, Appendix 1). We may express the OCP as follows: At each point of an optimal route, the curvature is equal to the logarithmic directional derivative (or the percentage rate of change) of the local criterion function perpendicular to the route. That is to say that, at any point on the highway, the forward curvature is obtained by projecting up to the criterion surface at that point, measuring the slope of the criterion surface perpendicular to the local direction of the highway, and obtaining the curvature of the new highway path by means of the fundamental OCP equation. For any point on a route obtained by any method, the OCP can be applied to check whether the segment considered is part of an optimum route. If it is not, then a better route exists. This could be useful where a route is established for nonoptimal reasons and certain segments can be optimized. Figure 2 is adapted from Howard, Bramnick, and Shaw (1). The optimal route goes between points A and B; A and B are in the XY plane. HDEF is the ln[c(x,y)] function surface. Further details are given by Howard, Bramnick, and Shaw (1).

Derivation

Suppose that in three-dimensional space there are two points separated by layers of air that have different refractive indices. There are an infinite number of different paths that the light rays can take. If only the rays that travel between the two points are considered, there will be a number of paths that will locally minimize the time of travel and one global minimum that will be the shortest path possible. The calculus of variations has been applied elsewhere--for example, by Bliss (3)--to solve this problem.

The OCP is derived in the paper by Howard, Bramnick, and Shaw (1). Imagine a plastic medium formed in such a way that its refractive index at each point is proportional to the local cost function for the highway. Then, if a narrow beam of light is introduced at, say, the south end point of the highway and swept around a semicircle, the light rays will trace through the plastic, in accordance with Fermat's principle of least time, paths that correspond to our optimal routes. The equivalent thing is done on the computer by simulation. The mathematical expression of the OCP (plus the geometric relations required to complete the principle) is as follows:

$$d\theta/ds = (1/c)(dc/ds_{\theta+\pi/2}),\ dx/ds = \cos\theta,\ dy/ds = \sin\theta \qquad (1)$$

where

θ = direction of the route,
$d\theta/ds$ = route curvature, and
c = local criteria function.

The subscript means that the direction of the derivative is taken in a direction perpendicular to the route direction. The integration methods are described later.

One characteristic of the OCP is that the optimal paths curve toward the area of the maximum criterion function. This can be understood if it is realized that the optimal route between two points separated by a high-criterion area will be better off if it skirts the high-criterion area and spends more of its path in the low-criterion area (1). The result is that the extrema tend to align themselves along

the gradient to the criterion surface.

Example

1. From the exponential cost function in the example given by Howard, Bramnick, and Shaw (1), the optimal routes obtained by two different integration formulas were compared with the analytic solution. This enabled the error bounds to be studied. From the local criterion function surface generated on the computer, the OCP was used to determine the optimal routes between the two end points. To do this, a one-parameter family of optimal routes was generated by varying the starting angle from one of the end points, numerically integrating the OCP to determine the optimal route for each starting angle, and selecting the paths that terminate on the other end point. The optimal route is the best of this discrete set. A shooting method, outlined by Howard and Shaw (2), was used to narrow down the correct starting angle and successively diminish the increments in the starting angle, wherein the angle increments start at 0.1 rad and decrease by a factor of 10 for each successive graph.

2. The illustrative example given by Howard, Bramnick, and Shaw (1) was used to verify that the method did indeed give the optimal-path routes. This example uses an exponential criterion field of $c = \exp(0.05y - 0.2x)$. The optimal route for any starting angle--say, 0.8 rad--was calculated. [The details are not essential to an understanding of this paper, but the interested reader is referred to Equations 5-7 of Howard, Bramnick, and Shaw (1).] This was also used to get the error bounds for the new intrinsic-equation algorithm and the original arc-of-circle algorithm. Figure 3 is the computer printout for the optimal paths for which the intrinsic-equation algorithm was used. The computer printout for the arc-of-circle algorithm is so similar that it has been omitted. Each optimum starts at the same starting angle (say, 0.8 rad) but uses different step sizes. The error is the difference between the experimental and analytic value of the y-intercept when x is zero. Figure 4 shows the graph of log absolute error versus log step size for both algorithms.

INTEGRATION ALGORITHMS

The equations of the two integration algorithms compared in this paper are presented below. To save space, the method of derivation of each is described in sufficient detail that the results may be reproduced by anyone skilled in the art without listing all of the equations of the intermediate steps.

Arc-of-Circle Algorithm

The OCP gives the curvature of an optimal route at each point. A natural engineering approach is to form the route by joining small arcs of circles of curvature. The analytic equivalent of this geometric operation is the computation of the position and the direction of the route at the end of the small arc of length δs by means of the following equations:

$$\delta\theta = \delta s\,(c_y \cos\theta - c_x \sin\theta)/c \tag{2}$$

$$\delta x = \delta s \cos\theta\,(1 - \delta\theta^2/6) - 0.5\delta s\delta\theta \sin\theta \tag{3}$$

$$\delta y = \delta s \sin\theta\,(1 - \delta\theta^2/6) + 0.5\delta s\delta\theta \cos\theta \tag{4}$$

This algorithm was derived and applied by Howard and Shaw (2). The main steps in the derivation are as follows:

1. $\delta\theta$ is obtained from the OCP and definition of directional derivative (4).
2. δx and δy are obtained by resolving the chord of Figure 5 in the X and Y directions, respectively, by use of trigonometric identities and approximating $\sin\delta\theta$ and $\cos\delta\theta$ by their MacLaurin expansions through terms of order $\delta\theta^2$.

Joining small arcs of circles gives a route of class C^1 (smooth, continuous tangent) but discontinuous second derivative (curvature) at the joints. However, an optimal route (by derivation of the OCP) is of class C^2. The question arises as to whether (a) the intuitively clear arc-of-circle algorithm (vastly superior to the linear segmented C^0 approximation) is a sufficiently good approximation to the theoretical optimum for practical purposes or (b) it is possible to develop a smoother algorithm that is not too cumbersome and that yields significantly better results. This paper addresses this question through the following algorithm.

Intrinsic-Equation Algorithm

It is known from differential geometry (5) that a space curve is uniquely determined to within a congruence by its curvature and torsion (two scalar functions of arc length) and a plane curve by its curvature alone. The congruential ambiguity is resolved by specifying initial position and direction.

It is customary to expand the intrinsic equations in MacLaurin's series about a given point on the curve, referred to the intrinsic trihedral (tangent, normal, and binormal vectors). This gives useful local information, but to generate the entire curve it is necessary to resolve coordinates and directions back and forth between the moving trihedral system and the inertial system and to integrate the Frenet-Serret differential equations to obtain the rotation of the intrinsic trihedral as its origin moves along the curve.

In the two-dimensional problem of determining an optimal highway plan, it is more convenient to work directly and exclusively in the inertial frame of reference of primary concern. The algorithm consists of two steps:

1. Compute the curvature K and rate of change of curvature K' for the given point x_s, y_s, and direction θ_s from the following three equations:

$$K = \theta' = (c_y \cos\theta - c_x \sin\theta)/c \tag{5}$$

$$c' = c_x \cos\theta + c_y \sin\theta \tag{6}$$

$$K' = [c_{xy} \cos 2\theta + 0.5(c_{yy} - c_{xx}) \sin 2\theta - 2Kc']/c \tag{7}$$

where subscripts denote partial differentiation and the prime denotes total derivative with respect to arc length along the curve.

Equation 5 is the OCP, derived by Howard, Bramnick, and Shaw (1); Equation 6 is the chain rule of differentiation (4) plus the geometric relations $x' = \cos\theta$, $y' = \sin\theta$; Equation 7 results from differentiating and simplifying Equation 5.

2. Compute the coordinates (x_1, y_1) of the next point on the optimal route and the direction θ_1 of the route at that point, in terms of the distance $h = \delta s$ along the route, from the following three equations:

$$\theta_1 = \theta_0 + K_0 h + K_0' h^2/2 \tag{8}$$

Figure 3. Computer printout of optimal routes for intrinsic-equation algorithm.

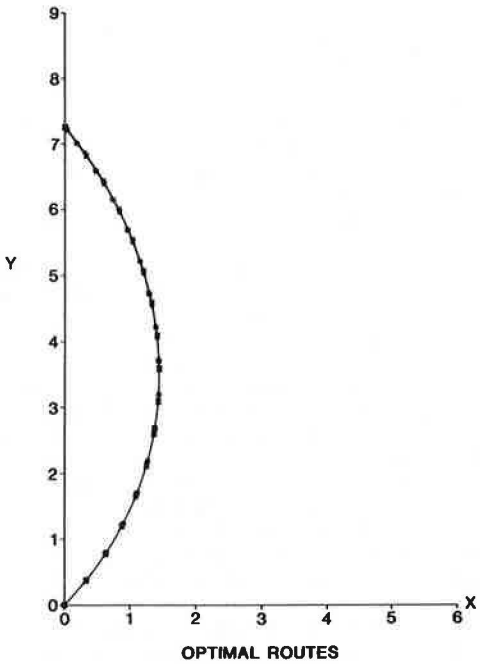


Figure 4. Error analysis.

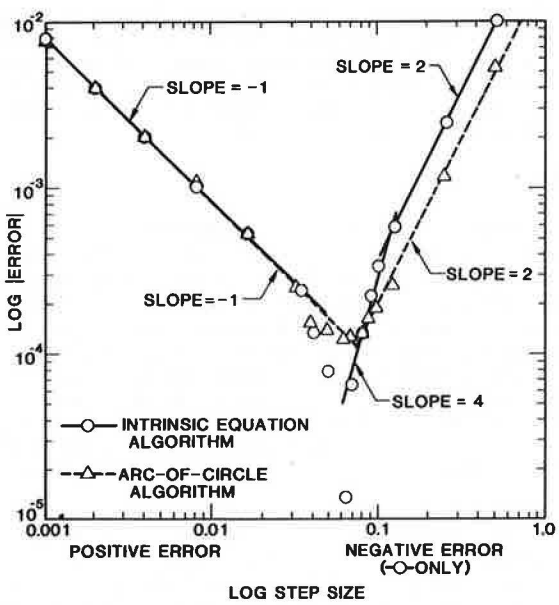
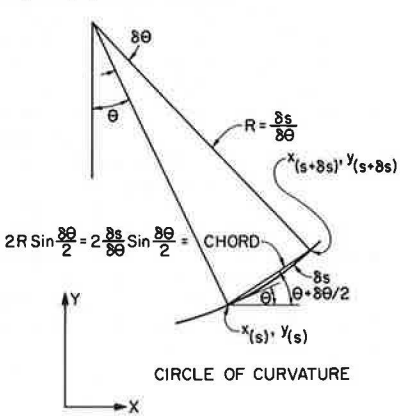


Figure 5. Circle of curvature.



$$x_1 = x_0 + h\cos\theta_0 - 0.5h^2K_0\sin\theta_0 - h^3(K_0'\sin\theta_0 + K_0^2\cos\theta_0)/6 \quad (9)$$

$$y_1 = y_0 + h\sin\theta_0 + 0.5h^2K_0\cos\theta_0 + h^3(K_0'\cos\theta_0 - K_0^2\sin\theta_0)/6 \quad (10)$$

Equations 8-10 are the Taylor series expansions of the respective functions θ_h, x_h, y_h about the values θ_0, x_0, y_0; previously derived relations are used to obtain expressions for the coefficients in the Taylor expansions. They are the practical equivalent of the intrinsic equations of the curve, referred to inertial axes.

Although it appears that x and y are computed to one higher order of accuracy in δs than is θ, they all are computed to the same order of accuracy in $\delta\theta$ and in K--namely, through K'. In fact, if K' = 0 (implying constant K), then the intrinsic-equation algorithm reduces to the arc-of-circle algorithm. Thus, the difference between the two is that the intrinsic-equation algorithm takes into account rate of change of curvature whereas the arc-of-circle algorithm does not.

ERROR ANALYSIS

It is known that the computational error in numerical integration on the digital computer is the sum of two components: (a) the discretization error and (b) the round-off error. These errors behave as follows:

1. The discretization error dominates on the coarse side or large step sizes. It is known that

$$E_d = \alpha h^k \quad (11)$$

where

h = numerical integration step size,
k = order of the method, and
α = factor that depends on the problem being solved.

2. The round-off error dominates on the fine side or small step sizes. It is known that

$$E_r = \beta/h \quad (12)$$

where β is a factor that depends on computer word size.

For the present example, with known analytic solution, the error was calculated by subtracting the computed value from the experimental value. The absolute value of the error is plotted against step size on log-log paper in Figure 4.

The arc-of-circle algorithm produces the neater curve, showing slope 2 where discretization error is dominant; a slope of -1 occurs where round-off error is dominant; the optimal step size (h) is about 1/16. The total error does not change sign, and there is a true minimum error.

The intrinsic-equation algorithm is a case where the error changes sign and has a zero value. Since log 0 = $-\infty$, there is a sharp dip in the curve. The slope is -1 where the round-off error dominates and 2 where the discretization error dominates. The optimal step size is between 0.05 and 0.075. Theoretically, there is an h when the error is zero, but it is not practical to identify the exact value. If one examines the area of the sharp dip, the intrinsic-equation algorithm is an order of magnitude better than the arc-of-circle algorithm for optimal step sizes of each.

DISCUSSION OF RESULTS

1. The results show that the intrinsic equation procedure and the various error analyses carried out, together with the application to a problem with a known answer, verify the validity and accuracy of the method.

2. From the application of the OCP (with the two integration algorithms considered) to the illustrative example given by Howard, Bramnick, and Shaw (1), the methods were found to give correct results, with reasonable error bounds for the value of y when x = 0 as checked by the analytic solution in the same paper. For the intrinsic-equation integration, the error bound is about ±0.000 013 in a y of about 7.2. For the segment of a circle integration, the error bound is about +0.000 12 for the same y at about the same step size.

The error analyses, log absolute error versus log step size, are shown in Figure 4; they use, respectively, the arc-of-circle and the intrinsic-equation algorithms. The superiority of the latter is shown by the fact that it changed sign at a step size of about 0.06. A slope of -1 was clearly shown, for a step size of less than 0.03, on the fine mesh side of the graph as well as unusual stability for the random round-off error, which may be irregular. On the coarse side of the mesh, a slope of 2 was clear for step sizes greater than 0.13. For step sizes between 0.03 and 0.13, because of proximity of the change in sign, and mixing of the effect of round-off and discretization error, there was a steepening of the slopes. This steepening of the slopes and the change in sign show the superiority of the intrinsic-equation algorithm over the arc-of-circle algorithm in this example.

FUTURE RESEARCH

Since the OCP is a new optimizing tool, there are many areas in which further work can usefully be done. These areas include generation of the criterion function, integration of the OCP, and generalization of the OCP in various directions, including discontinuities in the criterion function, generalized end point conditions, and the like. The work of Howard, Bramnick, and Shaw (1) has been drawn on for some of the suggestions that follow.

Local Criterion Function

Revenue is lost to the community when taxable land is lost to right-of-way. If this factor is to be usefully included, some planning projections are needed of the future possible use the land might have had if no expressway had been built. The impact of the expressway itself on future land use could be included in the planning study.

The local criterion field could be generalized to include the effect of discontinuities such as sudden variations in right-of-way costs and environmental factors or sudden change from four to six lanes. Howard and Shaw (2) treat this problem together with the Weierstrass-Erdmann Corner Condition (which we hope to present in later papers).

When the OCP has been generalized for practical application to three-dimensional problems, factors that depend on direction and location relative to factors immediately adjacent to the point considered could be included. Some of these factors are cut and fill and other factors dependent on vertical alignment.

User costs and environmental and ecological factors can be included in (and can dominate) the criterion function. The method encompasses whatever factors are considered.

Theoretical Problems

The application could be extended to three-dimensional problems such as hilly country where cut and fill becomes important. The OCP remains valid, as shown by Howard, Bramnick, and Shaw (1), but, for simultaneous plan and profile optimization, a torsion principle [developed by Shaw (6)] is needed. It is planned to present this in a later paper.

The class of admissable arcs could be extended from continuously differentiable to piecewise smooth, and the boundary conditions could be extended by using such lemmas as the Weierstrass-Erdmann Corner Condition; the transversality condition could be included to allow for variable end points, such as cities being two-dimensional regions rather than points, or to find a route to a major river or political boundary.

The intrinsic-equation method, together with its error bounds, could be studied to improve the application of the OCP, particularly if some of the research suggested above were carried out. The theory of splines appears to have potential application in this regard.

Sufficient conditions for a minimum could be investigated. The OCP is one of several necessary conditions that make possible constructive determination of local extrema. Theorems discussed by Bliss (3), Akheizer (7), and others provide sufficient conditions for a weak extremum (extremum over the class of differentiable arcs) and sufficient conditions for a strong extremum (extremum over the larger class of piecewise smooth arcs), but the practical usefulness of these theoretical conditions is not clear without further study.

CONCLUSIONS

Howard, Bramnick, and Shaw (1) established the principle and theoretical feasibility of the OCP. The OCP is a practical aid in the location and optimization of highways in certain cases. An improved intrinsic-equation integration process is originated in this paper, and its usefulness is demonstrated. An error analysis has been carried out to study the error in cases where the true end value is known.

Bounds can be established on the accuracy of the optimal routes in a practical way. It is shown that the error bounds are well within engineering tolerances.

The reasons for comparing the two integration algorithms are theoretical and practical. The Hilbert differentiability condition (satisfied in optimal routing problems) proves that optimal routes must be at least as smooth as the criterion function, which must be as smooth as class C^2 for the derivation of the Euler-Lagrange equation and the OCP. But the joins of circular arcs, parabolas, and straight lines often used in highway design are only of class C^1 and thus cannot be optimal routes, whereas the intrinsic equations of a curve can be carried to as many terms as necessary to ensure any required degree of smoothness. On the other hand, in this one analytic example, the circular arcs approximate the true optimal route to a sufficient degree of engineering accuracy. This is comforting since, once the optimal route (or routes) has been established and accepted, the highway engineer can approximate it with curves and straight lines in his or her usual way. The same program can then be used to find the cumulative effect of this departure (or any other departure) from the optimal route. But it must be emphasized that the optimal route that does

satisfy all of the hypotheses must be obtained first as a standard of reference.

ACKNOWLEDGMENT

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Effect of Increased Truck Size and Weight on Rural Highway Geometric Design (and Redesign) Principles and Practices

OGILVIE F. GERICKE AND C. MICHAEL WALTON

A summary is presented of a study of the effects that an increase in legal truck limits would have on highway geometric design elements and of the cost implications should various segments of the Texas highway system require redesign and modification to facilitate their safe and efficient operation. The paper includes (a) a review of past and current research concerning the effects of a possible change in legal vehicle dimensions and weights on the geometric design elements of rural roads, (b) an identification of those geometric elements most affected by a change in truck dimension and weight, (c) an assessment of the effects a change in legal truck size and weight will have on these geometric design elements for a variety of operating conditions, and (d) an estimate of the cost required to redesign and modify the highway section.

A set of issues surrounding the legal limits on sizes and weights of motor vehicles has become a primary policy concern of government and the trucking industry. Such concern is reflected by current federal initiatives (stemming from the Surface Transportation Act of 1978), related study activities, and actions of several state transportation agencies.

Fuel shortages and rapidly increasing fuel prices have provided an impetus for resolving many of the problems associated with vehicle sizes and weights. The underlying notion is frequently reflected in the following simple relation: Large vehicles can carry more freight per unit of fuel. However, although fuel conservation is important, it is only one of many measures that may be used in an analysis of size and weight issues.

Today's highway network is the result of an evolutionary process that represents, among other things, a mix of geometric design principles and practices. Any significant change in vehicle operating characteristics should require an assessment of geometric design practices and the impact on the existing highway system in terms of operational aspects and safety. Also needed would be an estimate of the cost required to redesign and modify the current network or segments of the network to accommodate the larger vehicles.

In Texas, a study is under way to evaluate some of the effects of operating larger and heavier vehicles on the highway system. Initial results, determined by using a study technique modified from the National Cooperative Highway Research Program (NCHRP) (1), showed estimated pavement costs, bridge costs, truck operating cost savings, and fuel savings that would result from increases in limits on axle weight and gross vehicle weight (GVW) coupled with corresponding changes in vehicle unit length and width. No change in the height of vehicles or trailers is considered in this study. The work reported in this paper focuses on the costs of the geometric design and redesign requirements associated with increases in vehicle size (length and width) as well as weight.

SCOPE OF THE RESEARCH

As an initial assumption, four different vehicle combinations (2) and two highway classification schemes (cases 1 and 2) are considered. The four vehicle scenarios are shown in Figures 1 and 2.

In case 1, the three functional rural highway systems are considered in the analysis: (a) the Interstate highway system, (b) the U.S. and state highway system, and (c) the farm-to-market (FM) road system. Case 1 represents a traditional approach that fits the Texas highway network of about 60 000 miles.

Case 2 differentiates on the basis of road use. In case 2, the following rural functional classes, or combination of classes, are also considered in

Figure 1. Vehicle configurations for scenarios A and B.

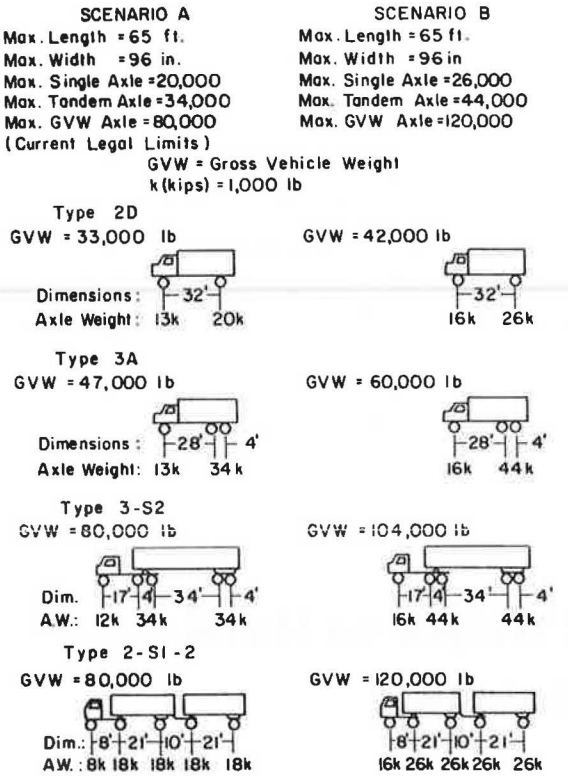
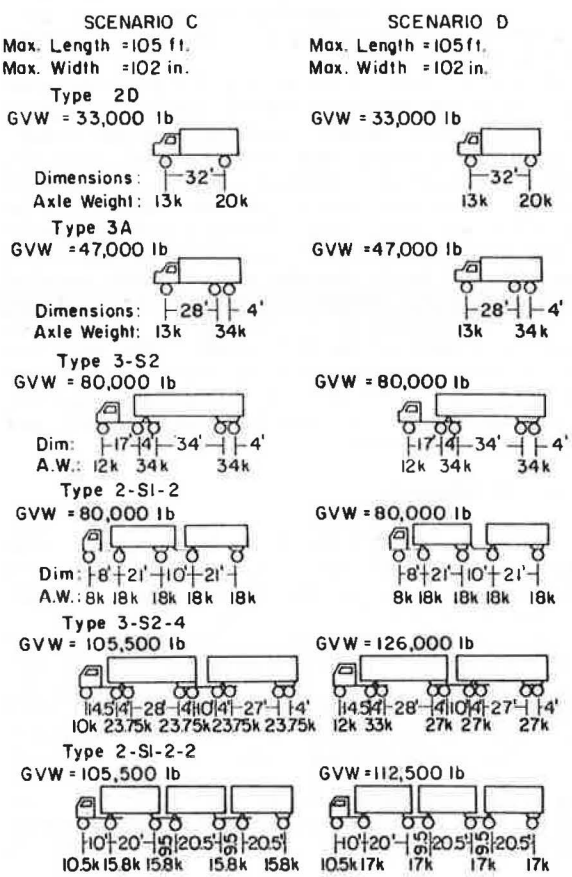


Figure 2. Vehicle configurations for scenarios C and D.



the analysis: (a) the Interstate highway system, (b) principal arterial systems (including Interstate), and (c) a combination of "all classes" (Interstate, other principal arterials, minor arterials, and major and minor collectors, excluding county roads that may be part of the roadway types mentioned above).

It was desirable to examine highway upgrading costs for these rural systems according to differences in their use, design standards, and vehicle composition. Four alternative scenarios were developed to provide a framework for analyzing a significant change in truck dimensions and weight patterns. Scenario A represents the current statutes and assumes that these weight and dimension limits will remain the same over the 20-year analysis period. The other three scenarios represent an array of changes in GVW, single- and tandem-axle weights, and vehicle length and width.

ELEMENTS AFFECTED BY SIZE AND WEIGHT CHANGES

From an evaluation of present geometric design principles and practices, the following elements were identified as those that may be affected by a change in vehicle dimension and weight:

1. Design elements--(a) Stopping sight distance, (b) passing sight distance, (c) pavement widening on curves, and (d) critical lengths of grades;
2. Cross-section elements--(a) Lane width and (b) width of shoulder; and
3. Intersection design elements--(a) Minimum design for sharpest turns, (b) widths for turning roadways, (c) sight distance at at-grade intersections, and (d) median openings.

Stopping Sight Distance

Design stopping sight distance is, according to the American Association of State Highway and Transportation Officials (AASHTO) (3,4), "the minimum distance required for a vehicle traveling near the design speed to stop before reaching an object in its path."

The minimum stopping sight distance is calculated according to the following formula (3,4):

$$SSD = 1.47 \times V \times 2.5 + V \times V/30\,(f \pm g) \quad (1)$$

where

SSD = stopping sight distance,
V = vehicle speed (miles/h),
2.5 = value assumed to represent perception and reaction times (s),
f = coefficient of friction between the tires and the roadway surface, and
g = percentage grade divided by 100.

The first part of the formula (1.47 x V x 2.5) gives the distance traveled during perception-reaction time. The second part [V x V/30 (f ± g)] gives the distance required to stop after brake application.

In measuring stopping sight distance, the following assumptions are made by AASHTO (4): (a) that the height of the operator's eye is 3.50 ft above the road surface and (b) that the operator must be able to detect an object 6 in high in his or her path.

The above formula and measuring criteria for minimum stopping sight distance were derived for passenger car operations. But AASHTO (4) states that, although trucks require a longer stopping distance from a given speed, the additional braking distance

is balanced by the higher eye position of the truck operator. The Federal Highway Administration (FHWA) Motor Carrier Safety Regulations specify a deceleration rate of 14 ft/s/s for truck combinations and 21 ft/s/s for passenger cars. This indicates that cars should stop in two-thirds the distance required for trucks (5).

Full-scale tests have been conducted by the transportation departments of California, Utah, and the province of Alberta, Canada, to assess the braking performance of trucks (5-7). Figure 3 shows the results obtained by these agencies. All of the dry-pavement results are well under the FHWA curve. A theoretical evaluation was performed by the Illinois Institute of Technology (8), and their results, based on analytic studies, computer simulation, and examination of experimental data, confirmed the results obtained in California, Utah, and Alberta.

Maximum vehicle height remains the same for the four scenarios, and no change in operator eye height is expected. This will therefore effect no change in stopping sight distance.

Passing Sight Distance

AASHTO states that, although most rural highways are two-lane highways, vehicles must frequently use a lane that is regularly used by opposing vehicles in order to overtake slower-moving vehicles. Passing sight distance is the length needed to safely complete this passing maneuver on two-lane highways (4) with an operator eye height of 3.50 ft and an object height of 4.25 ft:

$$PSD = d(1) + d(2) + d(3) + d(4) \quad (2)$$

where

PSD = passing sight distance;
d(1) = initial maneuver distance (ft)
= 1.47 x t(V - m + a x t/2) (3,4), where
t = initial maneuver time (s),
V = average speed of passing vehicle (miles/h),
m = speed difference between the two vehicles (miles/h), and
a = average acceleration (miles/h/s);
d(2) = distance traveled in the left lane by the passing vehicle (ft)
= (Lf + Ls + 150)V/Vi, where
Lf = length of faster vehicle (ft),
Ls = length of slower vehicle (ft),
V = speed of faster vehicle (miles/h),
Vi = speed difference between the vehicles (miles/h), and
150 = additional distance between the two vehicles before and after the passing maneuver (ft);
d(3) = distance at the end of the passing maneuver between the passing vehicle and an opposing vehicle (ft); and
d(4) = distance traversed by an opposing vehicle (ft).

Whereas an increase in vehicle weight and width will have no effect on the elements defined above, an increase in vehicle length will have a pronounced effect on d(2) and d(4). This was confirmed by tests in Utah and Alberta (5,6).

Design values of AASHTO and the Texas State Department of Highways and Public Transportation (TSDHPT) (3,9) are based on requirements for a passenger car passing a passenger car. Since it is common practice for cars to overtake trucks, additional length will be needed or more abortive passing maneuvers will result when the truck length is increased. This does not include consideration of the relative change in engine power associated with today's vehicles. The increase in abortive movements may have a detrimental effect on safety.

The following assumptions were made in calculating the extra passing sight distances required because of increased truck length:

1. Car length = 19 ft (3).
2. Truck length = 65 ft for scenarios A and B and 105 ft for scenarios C and D.
3. The speed difference between the two vehicles is 10 miles/h (3,4).
4. Values for t and a are assumed according to observed AASHTO values (3,4).
5. Overtaken vehicles travel at a uniform speed.
6. The passing vehicle slows down and trails the overtaken vehicle on entering the passing zone.
7. Values for d(3) are in the suggested range of 100-300 ft (3,4).
8. d(4) = 0.666 x d(2).

The values obtained are given in Table 1. It can be seen that passing sight distance will increase considerably with an increase in vehicle length. Pavement markings that prohibit passing maneuvers are warranted according to the Manual on Uniform Traffic Control Devices (MUTCD) (10). These values are also given in Table 1.

Pavement Widening on Curves

AASHTO (3,4) states that "pavements on curves are sometimes widened to make operating conditions on curves comparable to those on tangents." The justifications are based on truck operating characteristics: (a) that the rear wheels track inside of the front wheels (off-tracking) and (b) the difficulty of steering the vehicle.

The following formula gives maximum off-tracking values that were experimentally found to be close to the real, measured off-tracking (4,11):

$$MOT = R(1) - SQRT[R(1) \times R(1) - SUM(L \times L)] \quad (3)$$

where

MOT = maximum off-tracking (ft),
R(1) = turning radius of outside front wheel (ft),
SQRT = square root (here and as shown in other formulas), and
SUM(L x L) = L(1) x L(1) + L(2) x L(2) + ..., where
L(1) = wheelbase of tractor (ft),
L(2) = wheelbase of first trailer (ft),
L(3) = distance between rear axle and articulation point (ft),
L(4) = distance between articulation point and front axle of next trailer (ft), and
L(5) = wheelbase of next trailer (ft).

Extra width to compensate for the difficulty of driving on curves can be computed from the following (3,4):

$$Z = V/SQRT(R) \quad (4)$$

where

Z = extra width (ft),
V = design speed (miles/h), and
R = radius of centerline (ft).

The width of the overhang can be computed as follows (3,4):

Figure 3. Braking distance.

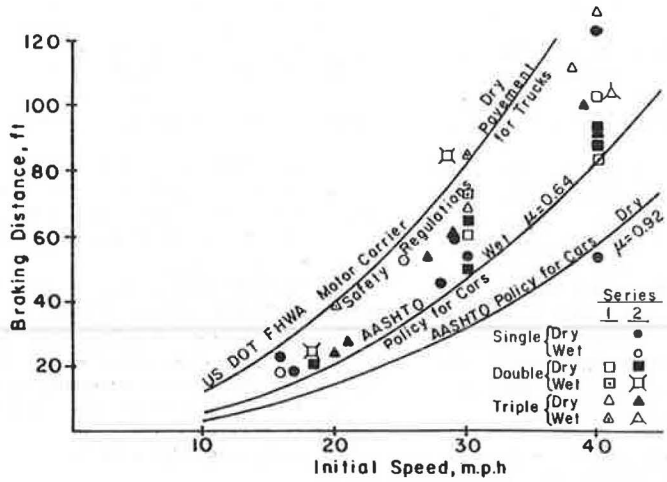


Table 1. Minimum passing sight distance for two-lane highways for scenarios A-D.

Design Speed (miles/h)	Assumed Speed (miles/h)		Avg a (miles/h/s)	t (s)	Passing Sight Distance (ft)			
					Calculated		AASHTO (4)	MUTCD (10)
	Passed Vehicle	Passing Vehicle			L=65 ft	L=105 ft		
30	26	36	1.40	3.6	1700	1900	1100	500
40	34	44	1.41	3.8	2100	2400	1500	600
50	41	51	1.45	4.1	2500	2800	1800	800
60	47	57	1.48	4.4	2800	3200	2100	1000
65	50	60	1.50	4.5	3000	3400	2300	–
70	54	64	1.50	4.5	3200	3600	2500	1200
75	56	66	1.50	4.5	3300	3700	2600	–
80	59	69	1.50	4.5	3400	3900	2700	–

Note: L = maximum vehicle length.

Table 2. Calculated values for pavement widening on two-lane pavements.

Degree of Curve	Design of Vehicle	Widening by Width of Pavement (ft)					
		30 ft	40 ft	50 ft	60 ft	70 ft	80 ft
1	3-S2-4	0.0	0.0	0.5	0.5	0.5	1.0
	2-S1-2-2	0.0	0.0	0.0	0.5	0.5	1.0
2	3-S2-4	0.5	1.0	1.5	1.5	2.0	2.0
	2-S1-2-2	0.5	0.5	1.0	1.0	1.5	1.5
3	3-S2-4	1.5	1.5	2.0	2.5	2.5	3.0
	2-S1-2-2	1.0	1.0	1.5	2.0	2.0	2.5
4	3-S2-4	2.0	2.5	2.5	3.0	3.5	
	2-S1-2-2	1.0	1.5	2.0	2.5	3.0	
5	3-S2-4	2.5	3.0	3.5	4.0		
	2-S1-2-2	1.5	2.0	2.5	3.0		
6	3-S2-4	3.0	3.5	4.0	4.5		
	2-S1-2-2	2.0	2.5	3.0	3.5		
7	3-S2-4	3.5	4.0	4.5			
	2-S1-2-2	2.5	3.0	3.5			
8	3-S2-4	4.0	5.0	5.5			
	2-S1-2-2	2.5	3.5	4.0			
9	3-S2-4	5.0	5.5	6.0			
	2-S1-2-2	3.0	3.5	4.5			
10-11	3-S2-4	6.0	6.5				
	2-S1-2-2	4.0	4.5				
12-14.5	3-S2-4	7.5	8.5				
	2-S1-2-2	5.0	6.0				

$$Fa = SQRT[R \times R + A(2 \times L + A)] - R \quad (5)$$

where

Fa = width of overhang (ft),
A = overhang (ft), and
L = wheelbase of unit (ft).

The width of a two-lane pavement on a curve can then be computed from

$$W1 = 2(U + C) + Fa + Z \quad (6)$$

where U is the vehicle track width (in feet) and C is the lateral clearance per vehicle (2, 2.5, or 3 ft for 20-, 22-, or 24-ft pavement widths, respectively).

From the above formulas, it can be seen that vehicle configuration and length will have an effect on pavement widening. The maximum vehicle width proposed for scenarios B, C, and D is 8.5 ft, which is the same as the maximum for the AASHTO design vehicles but 6 in wider than the TSDHPT maximum. New widths for pavement widening on curves were calculated for vehicle types 3-S2-4 and 2-S1-2-2. Some of the results obtained are given in Table 2.

Critical Lengths of Grades

According to AASHTO (4), climbing lanes should be provided on the upgrade side of a two-lane rural highway when

1. The length of upgrade causes a speed reduction of 10 miles/h or more or
2. The added cost is justified by the volume of traffic and the percentage of trucks.

The size, power, gradeability, and entrance speed of trucks contribute to the performance of trucks on a grade. Their combined effect then will lead to the maximum allowable speed reduction of 10 miles/h (4,9).

AASHTO (4) uses the nationally representative truck with a ratio of GVW (pounds) to net horsepower of 300:1 to evaluate the performance (acceleration and deceleration) of trucks on grade. It seems reasonable to assume that vehicles with a GVW of as much as 126 000 lb will have a ratio of 300:1 (4,12). The current availability of engines big enough to provide the 300:1 ratio underlines this assumption (12).

Lane Width

AASHTO states (3,4) that, on rural two-way highways, hazardous conditions exist on pavements that are less than 22 ft wide when even a moderate volume of mixed traffic is present because of inadequate body clearance.

Body and edge clearances for meeting or passing vehicles were identified as critical factors in judging the adequacy of pavement width (1). In experiments conducted in the earlier days of highway construction, two important observations were made (13):

1. Only on 24-ft pavements were drivers apparently satisfied with both edge and body clearance.
2. Drivers of passenger cars prefer a body clearance of about 5 ft when meeting other passenger cars. This cannot be attained on pavements that are less than 22 ft wide.

These observations make it clear that only vehicle width will have an impact on lane width. The following AASHTO design vehicles all have a current width of 8.5 ft: the SU, the WB40, the WB50, the WB60, and the BUS. No change in vehicle width from the existing AASHTO standards is proposed in scenarios B, C, and D, but the proposed width will differ from the allowable TSDHPT standard of 8.0 ft (4,9). Should Texas or other states adopt a wider vehicle width, the following should be borne in mind.

Although a 10-ft lane width may be an acceptable minimum on arterials that carry a few commercial vehicles (4,13), it is difficult to control the number and movement of commercial vehicles. Although substantial lane flow is accommodated, driving on such lanes causes undesirable tension and strain for drivers, especially at other than low speeds (3).

The average body clearances of 2.6 and 3.5 ft for passenger cars meeting commercial vehicles on 18- and 20-ft pavements, respectively, appear to be inadequate for safety (13).

The question of minimum lane width for safe operation of 102-in-wide trucks is a difficult one, especially for multilane highways. According to R.J. Hansen Associates (14), there is no evidence to indicate that an increase in width of 6 in would result in an increased number of accidents. It seems practical to allow for a gradual modification of lane width to 12 ft for the operation of 102-in-wide trucks. AASHTO (4) did not specifically address this issue; however, the lane width that it recommends is 11-13 ft. During an initial period, the operation of 102-in-wide trucks could, for instance, be allowed on multilane divided highways that have 11-ft-wide lanes. These lanes should gradually be widened to allow for the safe and tension-free operation of 102-in-wide trucks.

Width of Shoulders

Shoulders are mainly provided to accommodate stopped vehicles, for emergency use, and for lateral support of the pavement base and surface courses (3,4).

In order to accommodate stopped vehicles, AASHTO recommends that vehicles should clear the pavement edge by at least 1 ft and that a 2-ft working space should be provided (3,4). Widths of the standard AASHTO vehicles vary from 7.0 to 8.5 ft (4). By using the standard widths and clearances required, AASHTO recommends that, for heavily traveled and high-speed highways, the usable shoulder width should be at least 10 ft but preferably 12 ft (4).

The following relations between shoulder width and accident frequency have been found (15):

1. On tangents, as the right-shoulder width increases beyond the width necessary to accommodate a parked vehicle, the safety benefits become insignificant.
2. As the right-shoulder width increases on curves, the accident rate decreases.
3. Paved right shoulders produce fewer accidents than unpaved right shoulders.

The capacity of a highway is reduced if there are restrictive lateral clearances (4). For obstructions farther than 6 ft away from the pavement edge, no reduction in capacity is experienced (4). By considering capacities, accident costs, construction costs, and other relevant costs for various shoulder types and widths, a cost-effective design can be obtained.

Minimum Design for the Sharpest Turns

According to AASHTO (4), it is sometimes necessary to provide for the turning of vehicles within minimum space, such as at unchannelized intersections. Then minimum turning paths of the design vehicle become highly significant. It is assumed that the vehicle is positioned 2 ft from the pavement edge at the beginning and the end of the turn. The inner wheel should at no point be closer than 9 in to the pavement edge during the turn.

The paths that the 2-S1-2-2, 3-S2, and 3-S2-4 vehicles are expected to follow are shown in Figure 4. These paths were obtained by using a model built according to the description of the "tractrix integrator" (11) and the vehicle configurations shown in Figures 1 and 2.

Width for Turning Roadways

The widths required for turning roadways are classified according to three types of operation (4). The three cases are

1. One-lane, one-way operation with no provision for passing;
2. One-lane, one-way operation with provision for passing; and
3. Two-lane operation, either one-way or two-way.

The formulas used to compute the width for cases 1-3, respectively, are as follows:

$$W = U + C + Z = U + 6 \quad (7)$$

$$W = 2(U + C) + Fa + Fb = 2U + Fa + Fb + 4 \quad (8)$$

$$W = 2(U + C) + Fa + Fb + Z = 2U + Fa + Fb + 10 \quad (9)$$

where

U = track width of vehicle (out-to-out tires) (ft),
Fb = width of rear overhang (ft),
C = total lateral clearance per vehicle (ft), and
X = extra width allowance due to difficulty of driving on curves (ft).

To compute U, Fa, and C, the same formulas used earlier in the discussion of pavement widening on curves were used (4).

It can be seen from these formulas that vehicle width, configuration, and length will have an effect on roadway width while weight and height do not. The maximum vehicle width proposed for scenarios B, C, and D is 8.5 ft, which is the same as the maximum width used for some of the AASHTO design vehicles but is 6 in wider than the TSDHPT standard. When the above formulas were used, new widths were calculated for the 3-S2-4 and 2-S1-2-2 vehicles. The results obtained from these calculations are given in Table 3.

Sight Distance at At-Grade Intersections

AASHTO (4) considers three general cases of required sight distance at intersections, and the designer must ensure that for the different assumptions there will be an unobstructed view along both roads. The three cases are

1. Enabling vehicles to adjust speed, in which case only reaction plus perception time and one additional second for acute braking is considered;
2. Enabling vehicles to stop, in which case safe stopping sight distance plays a role; and
3. Enabling the stopped vehicle to cross a major highway, in which case the formula used to obtain the required sight distance is

$$d = 1.47V\,(J + Ta) \tag{10}$$

where

- d = minimum sight distance along the major highway (ft),
- V = design speed of the major highway (miles/h),
- J = sum of perception time and time required to shift to first gear or actuate an automatic shift (s),
- Ta = time required to accelerate and traverse the distance S required to clear the major roadway (s), and
- S = D + W + L, where
- D = distance from the near edge of the pavement to the front of the stopped vehicle,
- W = width of pavement along the path of the crossing vehicle, and
- L = overall vehicle length.

From the above, it can be seen that only case 3 will be influenced by vehicle length and acceleration capability. If it is assumed that the acceleration capability of the 3-S2-4 and 2-S1-2-2 vehicles will be at least the same as that of the WB50 vehicle, then longer sight distances will be needed due to the increase in vehicle length. This assumption is affirmed by truck acceleration tests made by the Western Highway Institute (12). For scenarios C and D, additional sight distance along the major highway will be needed for the 3-S2-4 and 2-S1-2-2 vehicles (see Figure 5).

Median Openings

The design of median openings depends on the type of turning vehicle and the traffic volumes (4). The opening must accommodate the off-tracking characteristics of the design vehicle at slow speeds. The previous discussion of minimum design for the sharpest turns deals with the expected wheel paths of the 3-S2-4 and 2-S1-2-2 vehicles. Figure 6 was obtained by using the off-tracking characteristics obtained in that section of this paper. Figure 6 shows the minimum median opening for various widths of the median. An 85-ft control radius was used, since this fits the path of the turning vehicle without undue encroachment of the vehicle on the adjacent lane. A left turn from the major divided highway can be made without any encroachment.

While entering the divided highway from a left turn, the 3-S2-4 vehicle will encroach on the adjacent lane about 4 ft, but this can be minimized by swinging wide at the beginning of the turn.

Figure 4. Off-tracking for a 65-ft radius.

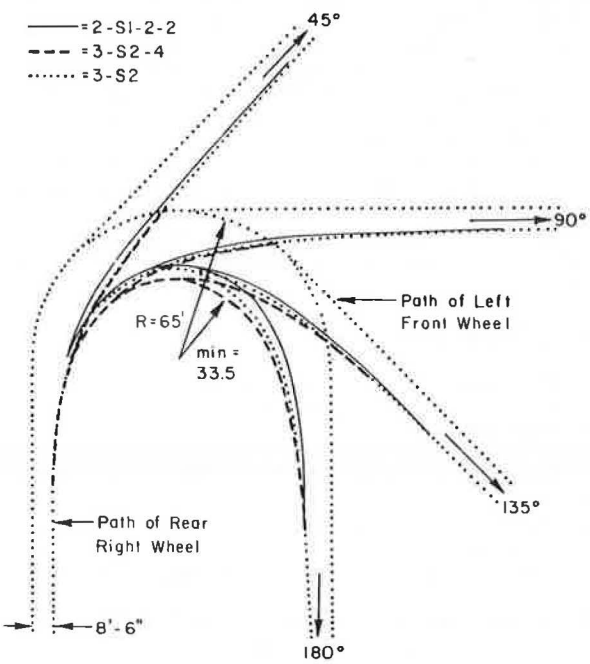


Table 3. Derived pavement widths for turning roadways for various design vehicles.

	Case 1			Case 2			Case 3		
Radius (ft)	WB50	3-S2-4	2-S1-2-2	WB50	3-S2-4	2-S1-2-2	WB50	3-S2-4	2-S1-2-2
50	26	–[a]	32	44	–[a]	57	50	–[a]	63
75	22	34	25	36	61	43	42	67	49
100	21	29	21	34	50	37	40	56	43
150	19	24	19	29	40	32	35	46	38
200	17	21	17	27	35	29	33	41	35
300	17	19	17	25	31	27	31	37	33
400	16	18	16	24	28	25	30	34	31
500	16	17	16	24	27	25	30	33	31
Tangent	15	15	15	21	21	21	27	27	27

[a]The 3-S2-4 vehicle cannot negotiate a 50-ft radius.

Figure 5. Required sight distance along major highways.

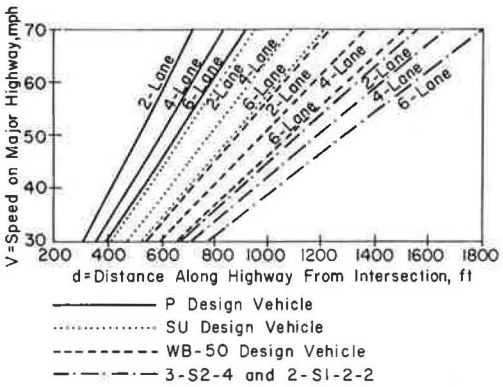
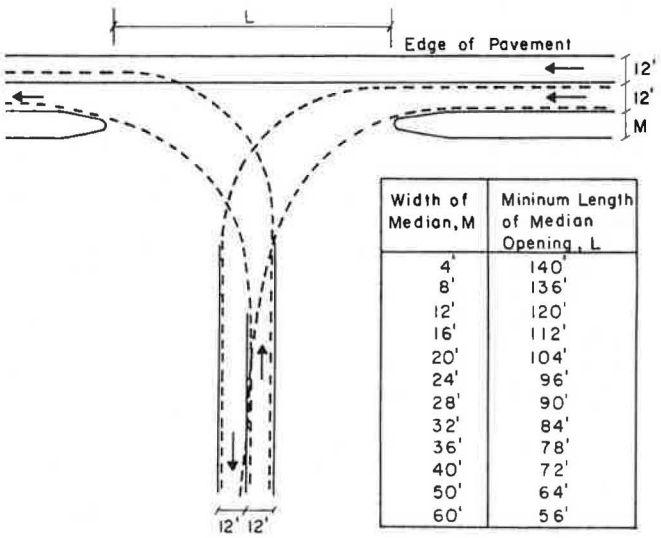
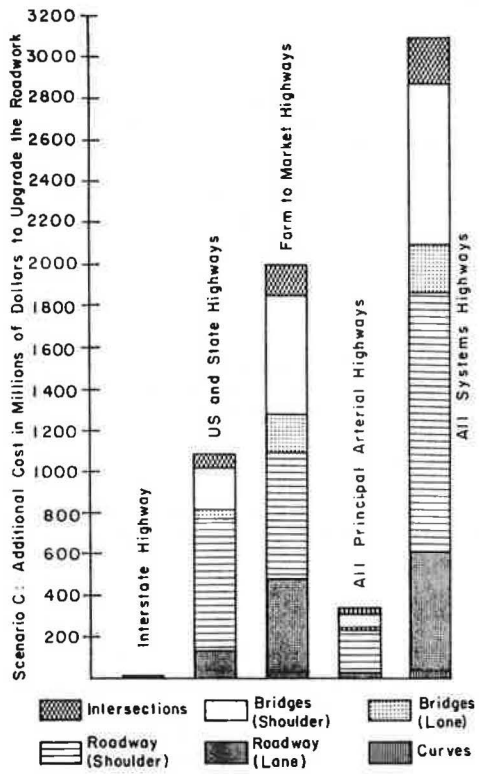


Figure 6. Minimum median openings.



Width of Median, M	Mininum Length of Median Opening, L
4'	140'
8'	136'
12'	120'
16'	112'
20'	104'
24'	96'
28'	90'
32'	84'
36'	78'
40'	72'
50'	64'
60'	56'

Figure 7. Additional cost to allow for implementation of scenario C.



COST ESTIMATES

In order to derive cost estimates for the various elements with an acceptable interval of confidence, it was necessary to obtain information on a representative group of each road functional or system class. This information was obtained either by collecting data manually from "as-built" plans and doing a statistical test on the confidence interval obtained from the sample or by using information provided by TSDHPT.

FHWA, through the Highway Performance Monitoring System (HPMS), required a diversity of information from TSDHPT concerning the following rural functional road classes (16): Interstate highways; principal arterials; other, minor arterials; major collectors; and minor collectors. The sample sizes required for the HPMS were based on "a 90-5 precision level for the volume groups of the principal arterial system, 90-10 for the minor arterial system, and on an 80-10 precision level for the collector system" (16).

This information was made available for use in this study and proved to be invaluable. Whenever use was made of this information (hereafter referred to as the HPMS information) or of the extended form to derive a cost estimate, no statistical testing on the adequacy of the sample size was done (this had been done by TSDHPT prior to the collecting of the required information). For all other estimates, statistical testing was done to ensure an adequate sample size.

A manual identification of the HPMS section identities was performed for the following road systems because it was necessary to distinguish between them: Interstates, U.S. and state routes, and FM roads.

Only the following items were taken into account when the cost estimates were made:

1. Widening of the existing pavement (excluding such items as grading, median barriers, curbs, guardrails, sign relocation, earthworks, additional right-of-way, culvert extension, or pavement markings) and
2. Widening of existing bridges.

Figure 7 shows a breakdown of the different element upgrading costs. Although the variation between the three scenarios is small, Figure 7 is approximately representative of all three scenarios.

SUMMARY

Assuming that one of scenarios B, C, and D is implemented, and assuming that the reasoning and assumptions made to establish the effect of these scenarios on design elements, cross-section elements, and intersection design elements are reasonable, then the following can be expected.

Stopping Sight Distance

No change from the current policy on stopping sight distance is foreseen due to the ability of the 2-S1-

2-2 and 3-S2-4 vehicle combinations to stop within the FHWA braking distances.

Passing Sight Distance

Although the implementation of any one of scenarios B, C, and D will require additional sight distance, the current pavement marking policy remains unaffected and no upgrading costs are required. This element is only applicable to two-lane, two-way operations and, if the current pavement marking practice is maintained, an adverse effect on safety can be expected.

Pavement Widening on Curves

Due to the increased off-tracking characteristics of the 3-S2-4 vehicle, additional pavement width will be needed if scenario C or D is implemented.

Critical Lengths of Grades

No adverse effect on the climbing ability of trucks is expected should scenario B, C, or D be implemented.

Lane Width

Although no change in the TSDHPT policy on lane width is expected, a 6-in increase in vehicle width will necessitate strict adherence to the current desirable standards. This will have a pronounced cost effect for either scenario B, C, or D. Although this is the existing policy and is being strictly adhered to, the cost estimates should not be considered as "over and above" that for scenario A because the same costs will be necessary if the TSDHPT road network is upgraded to the current policy.

Width of Shoulder

For shoulder width, as for lane width, no change in the current TSDHPT policy is expected, but a strict adherence to that policy is recommended. This will be very costly for some of the road classes. This cost should not be considered as over and above that for scenario A, for the same reason as that given for lane width.

Minimum Design for the Sharpest Turns

Due to increased off-tracking characteristics and decreasing turning ability, especially for the 3-S2-4 vehicle, additional pavement width will be needed in confined spaces to allow for the implementation of scenario C or D. Although it is assumed that the existing intersections on all road classes are designed to allow for the operation of scenario A, this is not so, especially for the FM roads. Estimates for all four of the intersection design elements are included because of their close relation.

Width for Turning Roadways

For width for turning roadways, as for minimum design for the sharpest turns, additional pavement width will be needed to accommodate the 3-S2-4 vehicle if either scenario C or D is implemented.

Sight Distance for At-Grade Intersections

Additional sight distance will be needed because of the increase in truck length and the additional time required to cross an intersection. No cost estimate was made to allow for scenario C or D due to the fact that insufficient information was available on the existing sight distances or the restriction on sight distance at intersections.

Median Openings

Due to the increased off-tracking characteristics of the vehicle combinations in scenarios C and D, additional pavement area will be needed to accommodate the 3-S2-4 and 2-S1-2-2 vehicles without undue encroachment on adjacent lanes.

CONCLUSIONS

Results of the Study

It can be concluded that, if any one of scenarios B, C, and D is implemented, some alterations to the Texas highway network may be necessary. There is little difference in the cost of modifying the geometrics of the highway system under these three scenarios, but other considerations, such as pavement and bridge effects, will have a bearing on the evaluation of changes in the legal size and weight of motor vehicles.

Need for Future Research

The existing procedure used by AASHTO to calculate required passing sight distance considers only the case of a passenger car overtaking a passenger car. Because of the serious safety implications, future research involving the relation between passing sight distance and passing maneuvers that involve trucks and truck lengths needs more attention.

The performance of trucks on grades (acceleration and deceleration) needs attention because the proposed AASHTO standards are based on a 300:1 weight-to-power ratio. If larger trucks are introduced, there may be a shift back toward the 400:1 ratio, and this will need future monitoring.

The questions of lane width, safety, and vehicle width also need additional attention so that a definitive standard for lane width can be established. A move toward a cost-effective design can be accomplished only if additional safety implications are known and a cost assessment is made in relation to the trade-offs of safety and lane width.

As for lane width, a more conclusive study of shoulder width, safety, and vehicle width is needed.

This study represents one element of a broad set of issues surrounding the legal size and weight of motor vehicles, principally trucks. One concern has been the cost of redesign or required modifications to the existing highway network to accommodate a range of possible vehicle types, sizes, and configurations. It is intended that this study, coupled with other ongoing studies in Texas and elsewhere, will assist in developing the necessary data on which future decisions can be based.

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This paper reflects our views, and we are responsible for the contents, facts, and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of TSDHPT. This paper does not constitute a standard, specification, or regulation.

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Simulation of Highway Traffic on Two-Lane, Two-Way Rural Highways

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A summary is presented of research undertaken to develop a rural two-lane, two-way computer simulation model that could be used by the highway design practitioner to measure and evaluate traffic-flow consequences for various alternatives considered during the roadway design process. To this end, a microscopic computer simulation model written in FORTRAN was developed. For the simulation roadway, the model can incorporate vertical grades, intersecting side roads, climbing lanes, and no-passing zones. Traffic and speed data used by the model include driver desired speeds, overall posted speed for the highway, localized speed restrictions, individual main-road traffic lane volumes, side-road traffic volumes, vehicle composition in five categories, and vehicle acceleration and deceleration characteristics. Throughput statistics for use in design evaluation include distributions of space mean speed and speed change and a traffic-flow quality index. Effects on traffic flow of spot improvements in roadway geometry or traffic control can be obtained by placing windows in the program at specified locations. Output data are summarized and reported at user-specified time intervals. By using a FORTRAN level H compiler, the simulation model has been run on an IBM 370/165 computer. For an 8000-m (4.9-mile) long roadway and a real-time simulation of 3600 s, as two-way traffic volume varied from 400 to 800 vehicles/h, the actual computer time varied from 28.3 to 62.3 s. Model validation tests were performed and the results were found to be consistent with actual traffic-flow patterns. In addition, the model was applied to an actual field site, where the base condition and three redesign alternatives were simulated.

Highway engineers normally develop a number of preliminary design alternatives. These alternatives are then evaluated on the basis of environmental impact, cost, and traffic operation. The impact of an alternative on the environment is analyzed by comparing the "before highway location" situation with the "after highway location" situation. The cost study is a general economic analysis and involves an estimation of construction costs and vehicle operating costs. Traffic operation studies for preliminary design alternatives include estimation of traffic performance resulting from vehicle-roadway interactions. Due to its complexity and its often random nature, traffic flow on highways cannot be characterized in a straightforward manner. The highway engineer resorts to empirical relations based on real-world observations. Even though these relations provide a general idea of the nature of traffic operations, they are not sensitive enough to detect either roadway traffic-flow interactions for any individual design alternative or the differences in these interactions between two or more alternative designs.

Since the development of large-scale, high-speed computers, engineers have had available a technique for simulating those systems that require empirical study. In 1954, the first traffic simulation model in the United States was processed on a digital computer. Since then, many computer simulation models have been developed to describe traffic flow at either the macro or micro level. None of the models developed in the past, however, is able to simulate traffic flow on a rural two-lane roadway without major restrictions on the input roadway.

The object of the research reported in this paper was to develop a computer model for microsimulation of traffic flow on two-lane, two-way highways for the roadway and traffic volumes that are normally found on this class of highway. The following functional capabilities were deemed to be desirable in the computer model:

1. The model should be able to vary the directional distribution of traffic volumes and to accom-

modate a number of different types of automobiles and trucks.

2. The headways assigned to vehicles should represent those found on two-lane highways.
3. The model should have the potential to incorporate minor crossroads where vehicles can either enter or leave the main simulation roadway.
4. The model should have the capability to permit individual vehicles to overtake and pass each other.
5. The user of the model should be able to define no-passing zones where vehicle overtaking and passing are prohibited.
6. The model should permit the user to place spot-speed restrictions at specific locations due to sharp curves, narrow pavement, or minor road intersections or at any other location where safety or vehicle operation requires a reduced speed.
7. The vertical grades on the simulation roadway should have the capability to interact with the performance characteristics of vehicles and affect vehicle operation where appropriate.
8. The model should be able to handle simulation roadway lengths normally found in roadway design.

It is anticipated that the model developed in this study will provide engineers with a useful and efficient method for evaluating roadway design alternatives. After the preliminary design stage, when design alternatives are formulated, engineers can use the model to simulate traffic operations on each alternative. The output from the model can provide statistics for the evaluation of traffic operations, level of service, air pollution, and road user costs. This output information can be used to evaluate each alternative by itself as well as in comparison with other alternatives. This evaluation process can be used to provide additional information for the decision maker who is choosing among alternatives.

REVIEW OF LITERATURE

Gerlough (1) has suggested that there are two methods available for simulating traffic flow. The first method is a physical representation in which each vehicle is portrayed as a binary one and the roadway is portrayed as a group of memory cells. Rules are interjected to regulate the movement of the vehicles. The second method is the memorandum method, in which each vehicle on the simulation roadway carries a file that contains all the physical information on the vehicle, such as position coordinates, speed, gap, and traveling time. These files are periodically updated. Since the second method requires less computer time than the first, it was the one chosen for this study.

Computer Simulation Models for Two-Lane, Two-Way Highways

Janoff and Cassel (2) of the Franklin Institute Research Laboratories developed a computer traffic-flow model that simulated vehicle movement on a two-lane highway. The roadway configuration includes no-passing zones, sight-distance restrictions, and grades for each traffic lane at any given location along the simulation roadway. Vehicle speeds and headways are generated according to volume-speed and volume-headway relations taken from the 1965 Highway Capacity Manual (HCM) (3). Using roadway and traffic data as input, the model simulates traffic movement according to the conditions surrounding a particular vehicle. Output data from the model can be summarized for any desired time interval during the simulated real-time period.

Heimbach and others (4) modified the Franklin Institute simulation model and developed the North Carolina State University (NCSU) model for the purpose of investigating the no-passing-zone configuration on rural two-lane highways in relation to throughput volumes. Two subroutines, designated Truck-On-Grade and Car-Exit, and one main routine, called Speed-Headway, were added to the Franklin Institute model. The Truck-On-Grade subroutine makes it possible to duplicate the existing range of grades on two-lane rural primary roadways in North Carolina. The Speed-Headway program resulted from a need to generate speed and headway distributions for simulation that would match those found in the field. After comparing headway field data collected from several sections of primary highways in North Carolina with output data from calibrated headway distribution models such as the Negative Exponential, Pearson Type III, Schuhl, Schuhl-Pearson Type III, Schuhl-Negative Exponential, and Modified Schuhl models, they found the Schuhl model best fitted the data collected from the field. This study indicated that, when the NCSU model is used, field and simulation data can be closely matched.

Boal (5) presented a paper at the Seventh Conference of the Australian Road Research Board in 1974 on his computer simulation model for a two-lane highway. In his model, the simulation roadway is assumed to be straight and flat. The model is able to simulate the passing-overtaking maneuver.

Stock and May (6) used a simulation model to evaluate the capacity of a two-lane road. The model they used is able to handle a simulation road that has high-design standards only. For example, speed restrictions due to design features cannot be simulated. Because the only interaction between the two opposite traffic flows is the passing maneuver, the passing gap acceptance is a function of the mean speed of the opposite flow. To simplify the simulation process, only one direction of traffic is explicitly simulated. The other direction of traffic moves at a constant speed with random headways.

Driver-Vehicle-Highway Relations

To support the development of a microsimulation model, there must be sufficient knowledge regarding vehicle behavior and performance on real-world highways. In this respect, the journals are replete with articles indicating a number of driver-vehicle-highway relations that are useful in the development of a simulation model.

Schuhl (7) has suggested that headways on two-lane highways appear to consist of two subsets, one related to free-flowing cars and the other related to cars whose performance is constrained by the traffic ahead. Driver-vehicle behavior in a traffic platoon where there is no passing has been characterized as a stimulus-response situation. In stimulus-response models, the response of any driver in units of instantaneous acceleration is hypothesized to be a function of the difference in velocity between his or her vehicle and the vehicle immediately ahead (8). Leong (9) studied the free-flow speed of vehicles on two-lane, two-way highways in New South Wales and found that desired speeds could be represented by a normal distribution density function. St. John and Kobett (10) analyzed the acceleration rate used by drivers and found that, if acceleration is not constrained by vehicle performance limits, drivers seem to choose an acceleration that can be expressed as a linear function of the difference between the driver's desired speed and current speed. Gerlough and Hubert (11) have noted that, at a "stop-controlled" intersection, the driver's gap-acceptance distribution can be char-

acterized by either an Erlang or lognormal distribution. Finally, the work of Taragin (12) suggests that the relation between highway operational speed and the degree of a horizontal curve is approximately linear.

Conclusions from Literature Review

From the literature reviewed, it can be concluded that, although much effort has been devoted to the development of computer models for the simulation of traffic flow since 1954, very few of these models are applicable to two-lane, two-way highways. The few exceptions are the Franklin Institute model, the NCSU model, the Boal model, and the Stock and May model. These models are able to simulate traffic on a rural two-lane highway with varying grades and no-passing-zone configurations. These models are limited, however, to minimal traffic entering or leaving the main roadway.

Finally, there does appear to be sufficient knowledge concerning real-world discrete vehicle behavior on a roadway to make it possible to model this behavior mathematically.

TRAFFIC-FLOW SIMULATION MODEL

In setting about to develop a computer simulation model for two-lane roadways, we developed a more explicit set of functional specifications. It was felt that the model should

1. Be capable of being understood well enough by the roadway design practitioner that he or she would feel comfortable in using it to test design alternatives;
2. Permit the user to locate speed restriction zones, no-passing zones, vertical grades, horizontal curves, minor side-road intersections, and passing bays or climbing lanes at any point along the simulation;
3. Accept a simulation roadway up to 13 km (8.1 miles) in length;
4. Be able to accommodate traffic leaving and entering the main simulation roadway via minor side roads;
5. Be able to simulate maximum hourly traffic volumes and directional distribution by traffic lanes that are found in the field;
6. Be able to accommodate vehicle overtaking and passing maneuvers;
7. Be able to simulate a number of different types of passenger cars and trucks, each with different acceleration and deceleration characteristics;
8. Permit the user to input typical speed and headway distributions found in the field;
9. Provide for interaction between the vehicle acceleration and deceleration characteristics and the horizontal and vertical alignment design and traffic control specified for the simulation roadway;
10. Have the capacity for at least 75 min of real-time simulation;
11. Provide real-time simulation that is efficient in terms of consumption of actual computer time;
12. Express throughput data characterizing simulation in statistics that are readily understood and usable by the roadway design practitioner in evaluation of design alternatives;
13. Make summarized, aggregate throughput data available for a few seconds' duration to at least 1 h of time; and
14. Enable the user to output simulation data for a number of spot locations throughout the simulation roadway.

Characteristics

By using the functional specifications outlined above, a computer model was developed that simulated traffic flow on a general two-lane, two-way roadway on a vehicle-by-vehicle basis. The Simulation of Vehicular Traffic (SOVT) model is written in FORTRAN and has been tested on an IBM 370/165 computer. The model permits vehicles to follow each other in the same direction in an orderly fashion and also permits vehicles that are moving faster to overtake and pass slower-moving vehicles. In the latter case, the decision to pass is based on the oncoming-traffic situation.

The upper limit for simulation traffic volumes is a function of traffic density and roadway length. For example, assuming a two-way traffic volume of 2000 vehicles/h and a throughput speed of 88 km/h (60 miles/h), the simulation roadway can be as long as 12 km (7.4 miles). Any directional distribution of traffic volume is acceptable. Any percentage distribution of five vehicle types is also acceptable. Acceleration and deceleration characteristics for these vehicles are defined by the user. Individual input speed distribution for each type of vehicle is also defined by the user.

With respect to the simulation roadway, the model accepts roadway lengths of 2-12 km (1.25-7.5 miles). At any point along the roadway, the user is able to specify for each traffic lane the location of speed-restriction zones. These restrictions may be due to sharp horizontal curves, narrow roadway widths, sight-distance restrictions, or school zones. The user is also able to specify the magnitude of vertical gradients, both positive and negative, and no-passing zones. The latter can be a section where sight distance is less than the minimum passing sight distance or an area where passing is prohibited to ensure safe vehicle operation. The model provides for interaction between the vehicle and the simulation roadway where individual vehicles encounter speed restrictions, vertical grades, or no-passing barriers.

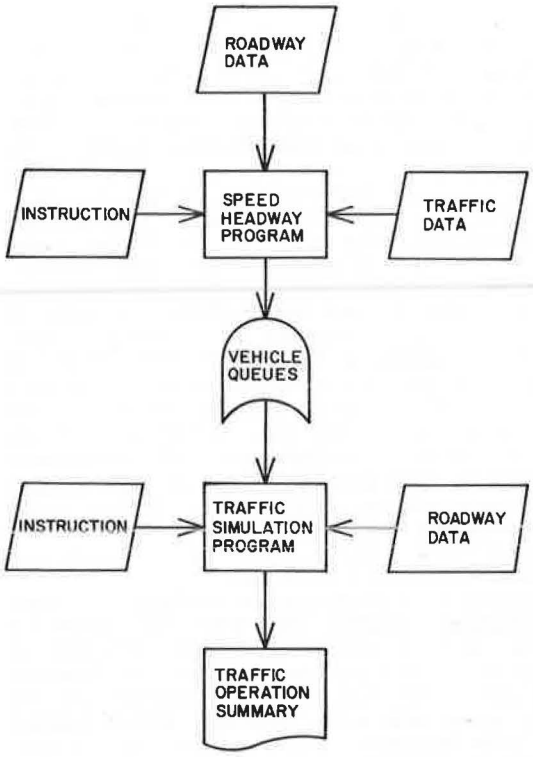
The user is also able to designate as many as eight minor stop-controlled crossroads along the simulation section. Vehicles from the major roadway are able to exit onto the minor roadway by executing either a right or left turn. Vehicles from the minor roadway can also enter the major roadway by executing a right or left turn. The user can specify the total volume and vehicle composition of all vehicles entering and leaving the roadway as well as the percentage of directional turning movements at each minor intersection. Within the simulation roadway, the user has the option of designating the location of any passing bays or climbing lanes that permit traffic in one direction to operate over two traffic lanes in the same direction.

Total simulation time for the model is defined by the user. Due to the variation of hourly traffic distribution, prolonged real-time simulation time is unrealistic. A total real time of not more than 1.5 h is reasonable. Within the total real time being simulated, the user is able to specify intermediate time intervals for which aggregate throughput statistics can be summarized and reported.

Description

There are two main steps in the simulation process. Figure 1 shows a simplified diagram that illustrates the functional processing steps for the SOVT model. In the first step, three sets of data are input into the Speed-Headway Program. These data include traffic data (traffic volume, maximum headway, and vehicle composition), roadway data (length of simu-

Figure 1. Processing steps for SOVT model.



lation roadway, number of minor crossroads, and turning movements), and instructional data (simulation time and an initial random number). For each traffic lane, a headway distribution is generated in a manner that corresponds to the Schuhl model. The sequence of these headways is then arranged randomly to form an entering queue. Desired speed, which is generated by a normal distribution generator, and type of vehicle are assigned to each of the vehicles in the entering queue. One entering queue is generated for each lane. These queues are then pooled into one entering queue according to entering time in chronological order. This queue is stored in a temporary file for later use.

In the second step, two types of data are input to the Traffic Simulation Program: (a) instruction data, including the number of times that simulation is to be repeated, the output printout interval, the maximum simulation time, the length of the simulation section, the throughput percentile speed for output data, and the number of no-passing zones, vertical grade sections, crossroads, speed-restriction zones, climbing lanes, and check stations; and (b) roadway data, including the coordinates of speed-restriction zones, no-passing zones, vertical grades, turning zones, climbing lanes, crossroads, and check stations. Vehicle information, generated by the Speed-Headway Program, is read, and vehicles that have entering times equal to zero are placed on the simulation roadway and the clock is set to zero. The program will proceed with the simulation routine until maximum simulation time is reached and the program is terminated. At each one of the user-defined output printout times, the program summarizes and prints out speed-change cycles, headway distribution, speed distribution, passing and overtaking information, average traveling speed, travel time, instantaneous speed at every check station, and the instantaneous configuration of the traffic stream.

By using FORTRAN H-level compiler, this model has been run on an IBM 370/165 computer. On an 8000-m (4.9-mile) roadway and with a simulation time of 3600 s, as traffic volume varies from 400 to 800 vehicles/h, the actual computer time varies from 28.3 to 62.3 s.

MODEL VALIDATION

The purpose of model validation is to verify that the computer model developed is so structured that the movement of vehicles on the simulation roadway approximates that of vehicles on an actual roadway. Because of financial limitations, no field data were collected to validate this model. Instead, a hypothetical 8000-m (26 250-ft) roadway section is used to check the reasonableness of simulation output. Different sets of data are input into the model to simulate traffic flow. The results of these simulation runs are compared to verify that the model does indeed generate traffic-flow data that have a pattern similar to that normally found on actual highways. Four check stations are located at 1000, 3000, 5000, and 7000 m (3280, 9843, 16 404, and 22 966 ft), respectively, from the beginning of the roadway section.

Desired Speed Versus Throughput Speed

To analyze desired speed versus throughput speed, the hypothetical roadway section is assumed to be flat (0 percent grade) and to have neither speed-restriction zones nor no-passing barriers and to carry one type of passenger car that is equal to one American full-sized (type 1) car. The experiment is a 3x4 factorial design that consists of three levels of mean desired speed and four levels of standard deviation of speed.

The mean desired speed varies from 87.66 to 102.06 km/h (54.5-63.4 miles/h), and the standard deviation of speed varies from 9.97 to 15.37 km/h (6.2-9.6 miles/h). Traffic volumes are 400 vehicles/h on lane 1 and 200 vehicles/h on lane 2. The results are shown in Figure 2. As deviation of speed (σ) among the vehicles on the road decreases, the overall travel speed (space mean speed) increases. When σ approaches 0--i.e., the desired speeds for all vehicles on the road are equal--the space mean speed reaches its maximum value, which approaches the mean desired speed.

Input Volume Versus Throughput Speed

In the analysis of input volume versus throughput speed, it is again assumed that the simulation section is flat and devoid of passing barriers and that it consists of type 1 cars only. Lane 2 volume is held constant at 200 vehicles/h. Lane 1 volume is allowed to vary from 200 to 600 vehicles/h in increments of 100 vehicles/h. Input volume versus throughput speed is shown in Figure 3.

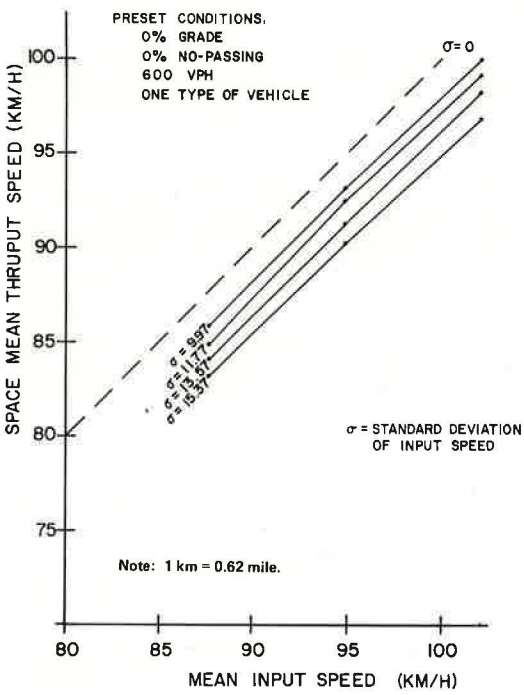
When one compares the space mean speed with operating speed from the 1965 HCM (3), it is found that the model generates throughput data similar to those values shown in the HCM, as shown in Figure 3.

Grade, No-Passing Barriers, and Truck Percentage Versus Throughput Speed

To investigate the throughput effect of grade, percentage of no-passing barriers, and the composition of vehicle types, a 3x3x3 factorial experiment was performed. Three levels were designated for each of these three factors. To simplify the effect of the vehicle types, only two types of vehicles were defined: the full-sized car and the semitrailer truck.

The three levels of vehicle composition of cars and trucks were 100 percent cars and 0 percent trucks, 95 percent cars and 5 percent trucks, and 90 percent cars and 10 percent trucks. The three levels of vertical grades were 0, 2, and 4 percent, and the three levels of no-passing barriers were 0, 20, and 40 percent. Input traffic volumes were held constant throughout these simulations. Lane 1 volume was set at 400 vehicles/h and lane 2 volume at 200 vehicles/h. The results of these simulation runs are shown in Figure 4. From the middle and right-hand graphs, it is obvious that the interactions between grade and percentage of no-passing barriers and between percentage of no-passing barriers and percentage of trucks are minor. The left-hand graph shows that, as grades become steeper, the effect of truck percentage on throughput speed is greater. This can be explained by the restriction of truck acceleration capability due to steep grade.

Figure 2. Space mean throughput speed versus mean input speed.



Crossroad Traffic Volume Versus Throughput Speed

To investigate the effect of crossroad traffic, the same 8000-m hypothetical roadway is used. The following input data are used for these test runs:

1. Lane volume is 400 and 200 vehicles/h for lanes 1 and 2, respectively.
2. The three levels of crossroad traffic volume are 0, 50, and 100 vehicles/h.
3. Mean desired speed is 87.66 km/h (54.5 miles/h), and standard deviation of desired speed is 9.97 km/h (6.2 miles/h) on the main roadway.
4. Four internal check stations were located at 1000, 3950, 4050, and 7000 m (3280, 12 963, 13 283,

Figure 3. Input volume versus output speed.

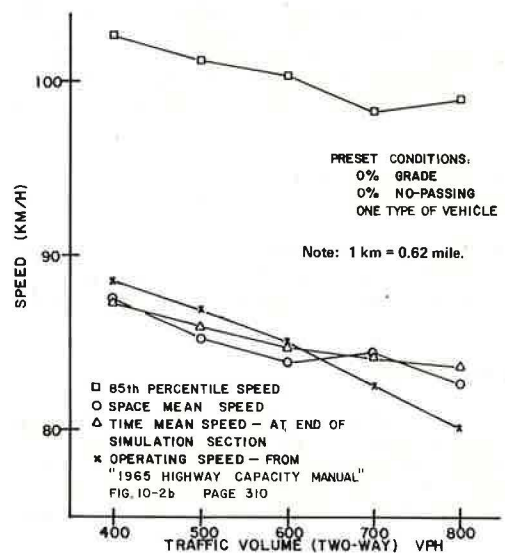
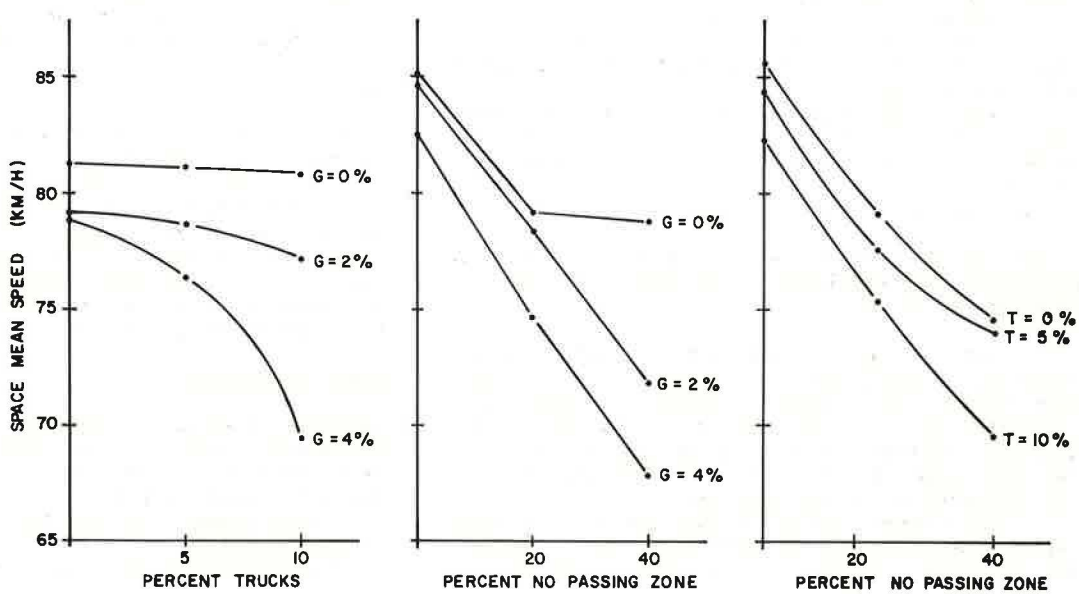


Figure 4. Space mean throughput speed versus (left) percentage of trucks and vertical grade, (middle) percentage of no-passing barriers and vertical grade, and (right) percentage of no-passing barriers and percentage of trucks.



and 22 966 ft) from the beginning of the simulation section.

5. The vertical grade is 0 percent.
6. The traffic stream includes only one type of car.
7. A no-passing barrier is specified at 3930-4000 m (2894-13 123 ft) from the beginning of the section.
8. A crossroad is located 4000 m from the beginning of the section.

The simulation results are summarized below (1 km = 0.62 mile):

Crossroad Volume (vehicles/h)	Throughput Speed (km/h)
0	85.62
50	85.58
100	83.29

Throughput speed showed very little difference while crossroad traffic increased from 0 to 50 vehicles/h. However, there was a more noticeable difference in throughput speed when crossroad traffic increased from 50 to 100 vehicles/h. The results of a Kolmogorov-Smirnov test indicated that the speed distribution at the crossroad intersection is not significantly different for crossroad traffic of 0 and 50 vehicles/h. Yet it is significantly different for crossroad traffic of 50 and 100 vehicles/h. These results can be interpreted as follows:

1. When traffic volumes are relatively low, the effect of crossroad traffic on the throughput speed is very minor.
2. As traffic volume from the crossroad increases, the effect of crossroad traffic becomes greater and the throughput speed is significantly reduced.

Effect of Other Factors

Speed-restriction zones, passing bays (climbing lanes), and crossroads can be defined in the SOVT model. The effectiveness of these parameters was tested by using the same 8000-m hypothetical roadway. The following input values were used for these test runs:

1. Lane volume is 400 and 200 vehicles/h for lanes 1 and 2, respectively.
2. Roadway length is 8000 m.
3. Mean desired speed is 87.66 km/h (54.5 miles/h), and standard deviation of desired speed is 9.97 km/h (6.2 miles/h).
4. Four check stations were located at 1000, 3000, 5000, and 7000 m (3280, 9843, 16 404, and 22 966 ft) from the beginning of the simulation section.
5. Only two vehicle types were defined.

For the above data, two sets of with and without conditions were simulated. Input data for these two sets were as follows:

1. With climbing lane and without climbing lane--(a) vertical grade of 4 percent, (b) 10 percent trucks, (c) no-passing barriers at 900-1100 m (2953-3609 ft) and 7100-7300 m (23 294-23 950 ft) from the beginning of the section, and (d) a climbing lane added at 1100-7100 m (3609-23 294 ft) from the beginning of the section; and
2. With speed restriction zone and without speed restriction zone--(a) vertical grade of 0 percent, (b) 0 percent trucks, (c) no-passing barrier at 4000-6000 m (13 123-19 685 ft), and (d) speed-restriction zone at 4000-6000 m with mean input speed of 72 km/h (44.7 miles/h) and standard deviation of input speed of 7.2 km/h (4.5 miles/h).

The results of these comparison simulation runs are summarized as follows:

1 Without a climbing lane, the space mean speed is 82.51 km/h (51.6 miles/h), and average speed at check station 3 (a point two-thirds of the way up the 4 percent grade) is 81.62 km/h (50.7 miles/h). After a climbing lane was added, space mean speed increased 2.4 km/h (1.5 miles/h) to 85.1 km/h (52.9 miles/h), and average speed at check station 3 increased 5.5 km/h (3.4 miles/h) to 87.14 km/h (54.1 miles/h). If one uses the Kolmogorov-Smirnov significance test with 95 percent confidence, the difference between these two sets of data is significant. These results show a definite improvement in traffic flow when a climbing lane is added to the steep grade.

2. With a speed-restriction zone, space mean speed is 79.32 km/h (49.3 miles/h), and average speed at check station 3 (a point inside the speed-restriction zone) is 68.34 km/h (42.5 miles/h). After eliminating the speed-restriction zone, simulation results showed a space mean speed of 83.7 km/h (52.0 miles/h) and an average speed at check station 3 of 83.28 km/h (51.7 miles/h). The results of the Kolmogorov-Smirnov significance test show that the speed distribution at the end of the speed-restriction zone is significantly different. These results show a negative effect on speed when a speed-restriction zone is imposed on the simulation section. These results also demonstrate the effectiveness of the speed-restriction-zone mechanism in the SOVT model.

MODEL APPLICATION

In order to test the different features of the model, an actual field site was chosen. The test section was a two-lane highway approximately 8-10 km (5-6.2 miles) long that had the following characteristics:

1. No signalized or major intersections,
2. At least one minor stop-controlled intersection,
3. Sections where speed is restricted due to either geometry or regulation,
4. Numerous no-passing barriers, and
5. A variety of vertical grades.

Test Site

A section of US-64 between Raleigh and Pittsboro in Chatham County, North Carolina, was chosen for the test run. This site begins at the end of a four-lane divided section west of the New Hope Reservoir and runs to a point approximately 200 m (655 ft) east of the Pittsboro city limits. It was chosen because it is a typical rural highway with very few driveways and fulfills the required conditions. The total length of this section is 10 782 m (6.7 miles).

Existing Condition

Vertical grades, horizontal curves, crossroads, no-passing barriers, and speed restriction zones were measured in the field. The field data were checked against records contained in a road inventory sheet prepared by the North Carolina Department of Transportation.

This section of US-64 is located in rolling ter-

Figure 5. Plan and profile of US-64 test section.

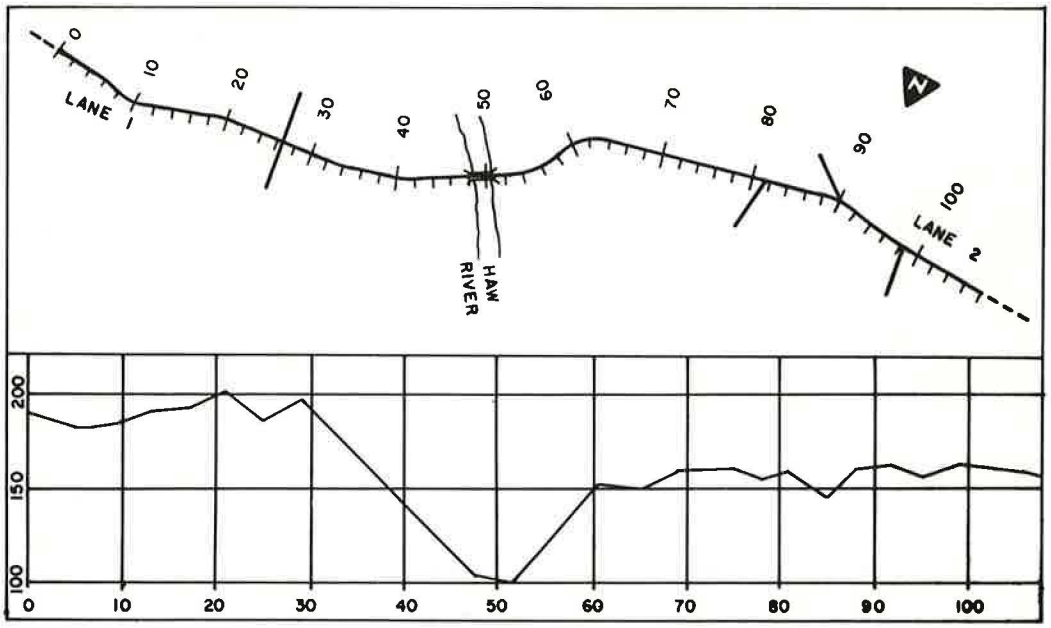
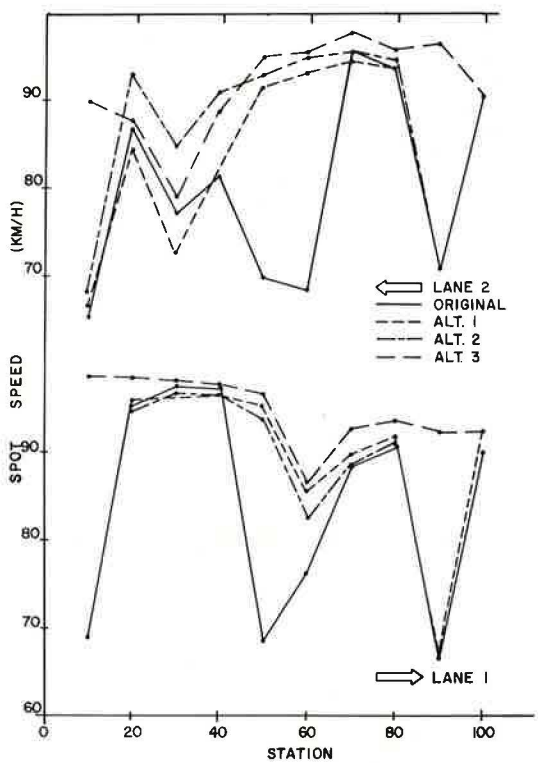


Figure 6. Speed profiles produced by SOVT model traffic-flow simulation.



rain. There are five speed-restriction zones. Three of these zones are attributable to sharp curves, one is a narrow bridge, and the last is the transition zone from the rural speed of 88 km/h (55 miles/h) to the urban speed of 32 km/h (20 miles/h). In addition, there are numerous no-passing barriers in this section of highway because sight distance is restricted by vertical and horizontal alignment and there are various vertical grades that exceed the maximum of 5 percent. Figure 5 shows the plan and profile of the test section.

Improvement Alternatives

US-64 is an alternative corridor for diverting traffic from the already congested I-85 corridor. To improve the capacity and level of service on this section of highway, several alternatives were developed. These alternatives can be described as follows:

1. In alternative 1, the existing narrow bridge is replaced by a bridge 13.5 m (44 ft) wide. This will eliminate the speed-restriction zone and the no-passing barrier at the bridge. In addition, with minimum construction work, the sharp curves at station 59+00 can be flattened; i.e., the speed-restriction zone and the no-passing barriers from station 59+55 to station 65+18 can be eliminated.
2. In alternative 2, besides the restrictions listed in alternative 1, it is found that the long upgrade for eastbound traffic from station 48+00 to 29+00 is too long and too steep for heavy trucks to climb. To improve this situation, a climbing lane is proposed.
3. In alternative 3, it is proposed that all sharp curves be flattened and that all speed-restriction zones, except one that has been posted with a lower speed limit because of the transition from rural to urban speed limit at the west end of the test section, be eliminated.

Result of the Simulation

Traffic flow for the base condition and the three improvement alternatives was simulated by the SOVT model. Speed profiles for all alternatives are shown in Figure 6. The simulation results, which are summarized in Table 1, show increasing speed and reduced delay as the quality of highway improvements increases. These results also indicate the changes in traffic flow that an engineer can expect as a result of different levels of roadway improvement.

When all of this traffic operational information is used in conjunction with engineering cost estimates, roadway design engineers can determine trade-off points and recommend a proposed design.

CONCLUSIONS

From the preceding analysis, it can be concluded that

Table 1. Summary of SOVT model simulation results.

Item	Existing Condition	Alternative 1	2	3
Space mean speed (km/h)	80.47	83.49	85.16	90.25
Spot mean speed at end of roadway (km/h)	77.80	78.77	78.71	80.52
85th percentile speed (km/h)	93.65	95.74	96.30	98.26
Number of completed passes	127	124	173	175
Time for vehicles forced to travel at less-than-desired speed (s)	41 701	39 288	32 407	29 188
No-passing barriers (%)	47	30	26	18
Quality index[a,b] (10^3)	31.52	58.80	64.55	109.30

[a]The larger the number, the higher is the quality of flow.
[b]From Greenshields (13).

1. The microscopic computer traffic-flow simulation model developed, which incorporates general highway features including climbing lanes, speed-restriction zones, no-passing-zone barriers, stop-controlled crossroads, varying vertical grades, and overtaking and passing of vehicles, can successfully simulate traffic flow on two-lane roadways;
2. The model is able to generate throughput that is consistent with actual traffic-flow patterns; and
3. The throughput data can be used to evaluate roadway design alternatives.

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Applying the Level-of-Service Concept to Climbing Lanes

ABISHAI POLUS, JOSEPH CRAUS, AND ITZHAK GRINBERG

A summary is presented of one phase of a multiphase study that deals with heavy-vehicle flow on downgrades and is concerned with geometric and flow warrants for the inclusion of truck climbing lanes on two-lane rural highways. One criterion for the introduction of climbing lanes on two-lane rural highways for both upgrade and downgrade directions is based on the level-of-service concept; another criterion is based on speed reduction. A suggested set of volume graphs is devised for the level-of-service criterion, and extensive use is made of a previously developed model for truck equivalency factors. A scheme for the relative location and signing of climbing lanes is also presented.

Current traffic composition on most highways includes an increasing number of trucks and other heavy vehicles, the relatively poor performance of which greatly affects the freedom of operation of traffic, particularly on grades. The result is an increase in the potential for accidents and a reduction in level of service. The winding two-lane rural road with steep grades and significant truck volume is familiar to many designers and to drivers, who suffer from the greatly restricted capacity of such sections. These developments have heightened the concern of highway engineers over the severity of accidents on rural roads. Since it is fairly well recognized that most accidents between private cars and heavy vehicles are severe in nature, it can readily be understood why certain preventive design measures are necessary.

The provision of appropriate climbing lanes on different grades along rural highways is considered one possible measure for improving safety and capacity. The need for such measures increases when volume computations indicate that an upgrade or downgrade experiences a low level of service or causes capacity to be exceeded.

This paper, which describes one phase of a multiphase study dealing with heavy vehicles on two-lane rural highways, is concerned with geometric and flow warrants for the inclusion of truck climbing lanes

for both upgrade and downgrade situations. The criteria presented are based on the concept of level of service, which reflects the average operating conditions for the highway section under consideration. In order to study the effect of grade on traffic flow and capacity, the usual upgrade speed curves were adopted (1). For capacity calculations, however, the actual performance characteristics of heavy vehicles on grades are also needed. Therefore, previously reported field speed observations made at six downhill sites in Israel (2) were used. This paper describes a truck equivalency factor model that was also used for the analysis. This model, which differs from the current model given by the 1965 Highway Capacity Manual (HCM) (3), has been described in detail elsewhere (4).

BACKGROUND OF STUDY

The influence of heavy vehicles becomes much greater on grades. The HCM, which discusses the effect of trucks, buses, and grades on the service volumes of two-lane highways, states that most typical grades affect operations only when trucks are present and that the effect varies with the length and steepness of the grade as well as the level of service under consideration. The HCM does admit, however, that knowledge of these effects on service volumes is limited.

In current practice, two principal alternatives exist when one considers climbing lanes. One, the policy of the American Association of State Highway Officials (AASHO) (5), suggests that climbing lanes are required where the critical length of grade is exceeded, given that the traffic volume and the percentage of heavy trucks justify the added investment. When the reduction in speed of loaded typical trucks does not exceed 15 miles/h, AASHO does not suggest climbing lanes, regardless of traffic volume, since the delays caused by trucks in such cases are within reasonable limits.

The AASHO policy further recommends that, where critical length of grade is exceeded, justification for climbing lanes should be considered from a highway capacity standpoint. The suggested general rule is that the design hourly volume (DHV) should not exceed the design capacity on an individual grade "by more than about 20 percent". Up to this limit, traffic volume is considered not to exceed the capacity of the highway and the delay to traffic not to be too excessive.

In the other alternative, Stimpson and Glennon (6) reduce the AASHO first criterion to 10 miles/h and suggest the appropriateness of eliminating the design capacity criterion; climbing lanes are then justified if the DHV equals or exceeds the design capacity of the grade.

This alternative, which is stricter than the AASHO approach, seems to be unjustified from the standpoint of both economics and driver behavior. The resulting increased cost may be unreasonable, especially since a driver's anticipation of a certain level of service is reduced on grades.

AUTOMOBILE EQUIVALENCIES

An earlier phase of the overall study (2) dealt with the detailed development of a revised method for the determination of automobile equivalencies, which were evaluated by the ratio of the average delay caused by one truck to the average delay caused by one automobile. This proposed method is rather different from the regular (HCM) approach, which is based on the ratio of the theoretical number of passings of one truck to the average theoretical number of passings of one automobile.

Based on the cumulative distribution of spot speeds of automobiles and on the calculated average speed of loaded trucks at all sites where truck speed data were gathered, automobile equivalencies were calculated by using the delay measure for all levels of service [the analysis is discussed in detail elsewhere (4)]. The developed upgrade automobile equivalency values are shown graphically in Figure 1, and the downgrade values are given in Table 1, as a function of length and rate of grade. The upgrade and downgrade values were used for calculations of the truck adjustment factors, which were substituted later for the service-volume formula.

CRITERIA FOR CLIMBING LANES ON UPGRADES

The objective of modern highways is to provide good, efficient, safe service for the volume of traffic expected. The level-of-service concept, as accepted today, uses operating speed and volume-to-capacity ratio as its two major components. The difference between truck speeds on grades and automobile speeds causes a reduction in the traffic volume carried by the highway. The HCM states that platooning and grouping sometimes characterize vehicle operation and movement on a rural highway and a truck very often leads the platoon. These platoons adversely affect service volume, since the speed of the platoon is reduced considerably on grades. The effect becomes more pronounced as volume increases, for the probability of passing decreases. Werner and Morrall (7) realized that another factor was also involved: The larger the space that trucks occupy in the traffic stream, the more reduced is the capacity of the highway. It is clear, therefore, that there is a strong interrelation between the percentage of trucks, the rate and length of grades, and the service volume for all levels of service.

Figure 1. Suggested automobile equivalency values by level of service.

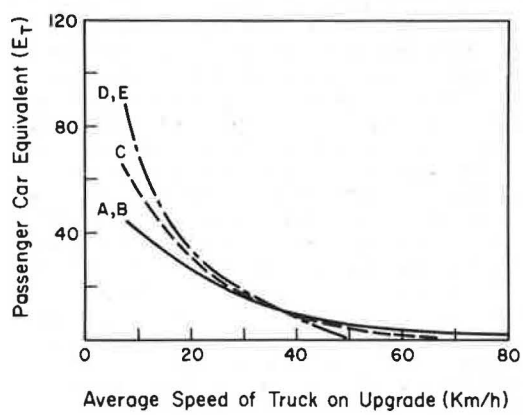


Table 1. Truck equivalency factors for downgrade sections.

Grade (%)	Truck Equivalency Factor by Length of Downgrade							
	250 m	500 m	750 m	1000 m	1250 m	1500 m	1750 m	2000 m
3	2.5	2.8	3.1	3.1	3.2	3.3	3.4	3.5
4	2.9	3.2	3.4	3.5	3.6	3.7	3.8	3.9
5	3.3	3.5	3.7	3.9	4.0	4.1	4.2	4.3
6	3.7	4.0	4.3	4.4	4.5	4.7	4.8	5.0
7	4.1	4.6	5.2	5.5	5.9	6.4	6.8	7.4
8	5.0	6.3	7.4	8.0	8.6	9.3	9.6	10.0
9	7.1	8.8	10.1	11.1	12.0	12.5	13.4	14.9

Note: Truck equivalency factor for 0-2 percent grade = 2.5.

In order to increase the sensitivity of the current approach (to the introduction of climbing lanes) to considerations of volume and operation, two new criteria were devised during the course of this research: (a) The speed of trucks on a given upgrade is reduced, relative to level speed, by more than 20 km/h, and (b) the expected or design volume exceeds the specific grade service volume for a level of service that is one degree lower than the level of service for a level section of the highway. Both criteria should be fulfilled in order to warrant the inclusion of a climbing lane on a specific upgrade. The reason that the two criteria must be met simultaneously is that, when the length of the grade does not exceed the critical length, the delay to traffic may be tolerable at any volume; similarly, when there is a long grade but low volume, the delays are both infrequent and random.

The reasons for using a poorer level of service stem from both economic and behavioral factors:

1. Budget constraints usually do not permit the overdesign approach.
2. Drivers usually expect less favorable operating conditions on grades than on level sections.

The analogy to some of the concepts incorporated into the determination of design speed may further support the use of a poorer level of service. The choice of a design speed is influenced primarily by the character of the terrain: Mountainous terrain justifies a lower design speed than does open country.

In order to check the fulfillment of the first criterion, the regular upgrade speed-distance curves are used (5). For the second criterion, however, a more complex procedure must be pursued. The service volume is usually given by multiplying the ideal volume (2000 vehcles/h) by the following factors: volume-to-capacity ratio (for a given level of service and a given passing sight distance), an adjustment factor for lane width and lateral clearance (W), and an adjustment factor for the presence of trucks (T). All adjustment factors are readily available except for the T factor, which must be derived indirectly and is based on the percentage of trucks (P_T) and on the weighted truck equivalency factor (E_T), which is accepted by multiplying the downgrade values by the appropriate truck distribution fractions for the downgrade and upgrade, accordingly. The particular relation among the P_T and E_T variables is commonly accepted. The upgrade values of the truck equivalency factors, which were developed during the course of the other phases of the research, are taken from Figure 1, and the downgrade values are taken from Table 1; this is done in order to calculate the truck adjustment factor and the service volume at a one-degree-lower level of service, as discussed above.

In order to simplify the calculations needed for a determination of the second criterion, a set of graphs was developed during the course of this research. The two examples shown in Figure 2 give the relation between the length and rate of grade and the service volume for a given level of service: levels B and C. The design speed is 100 and 90 km/h, and the other controlling factors--percentage of passing sight distance greater than 450 m, percentage of trucks, directional distribution of traffic, and the existing level of service for the entire highway stretch--are specified in each graph. A set of similar relations may be devised for all combinations of controlling factors. Once this task is completed, one can check whether the expected DHV exceeds the service volume--for a level of service that is one degree lower, as discussed above--for a given grade.

The maximum service volume depends, of course, on all controlling factors and varies accordingly. For instance, for 1000 m, a 6 percent grade, service levels B and C, and design speeds of 100 and 90 km/h, the maximum service volume is 500 and 390 vehicles/h, respectively. The values leading to those volumes are marked in the appropriate figures. If the expected DHV exceeds these service volumes, the second criterion for the inclusion of a climbing lane is fulfilled.

INCLUSION OF ADDITIONAL LANES ON DOWNGRADES

Simultaneous consideration of both the upgrade and the downgrade situation may represent more closely the real-world situation. For this, the downgrade values of truck equivalency factors are needed. A suggested set is given in Table 1, as discussed previously.

The introduction of an additional lane for the downgrade direction should be considered on the basis of the three criteria that follow.

First, since the directional distribution of traffic on most rural highways is about equal and the effect of trucks (and, consequently, the truck equivalency factor) on upgrades is more pronounced as compared with downgrades, the justification for an upgrade climbing lane has first to be made; then the two other criteria are examined.

The second criterion is based on the downgrade speed findings of a previous phase of this research (2) and other studies (8), which revealed that a certain amount of reduction in heavy-vehicle speeds exists on downgrades. This tendency was found to be particularly pronounced for long and steep downgrades. In fact, it was also found that this reduction is directly related to the downgrade truck equivalency factors, as given in Table 1, and that the rate of grade (i) contributes more than does the length of grade (L) to its magnitude. This explains the use of the product $G = L.e^i$, where L is the length of grade in kilometers, e is the base of natural logarithms, and i is the percentage rate of slope, as a controlling variable for the joint effect of the geometric elements (length and slope) on the equivalency factors on downgrades, as shown in Figure 3. It can be observed here that, for values of G greater than 400, the equivalency factors rise sharply; hence, the second criterion for the inclusion of an additional lane on downgrades is fulfilled.

The third criterion stems from volume considerations and should be considered simultaneously with the other two criteria, as explained for the upgrade situation. Here, two basic situations are encountered: (a) a solid separation line along the grade, a situation in which motorists in a single lane are not permitted to pass, and (b) the minimum passing sight distance along the entire grade, where passing is not prohibited. For both situations, the suggested criterion uses, again, the level-of-service concept. When passing is permitted, a situation that may represent a relatively straight downgrade, an additional downgrade lane is justified if a reduction in one level is encountered. On the other hand, when passing is prohibited--on winding downgrades, for instance--this criterion is modified to be similar to the upgrade case: Namely, a reduction in one level of service is permitted, and an additional downgrade lane is justified, when a reduction in more than one level exists or, as formulated previously, when the DHV exceeds the service volume for a level of service one degree lower than the service level of a level section of the highway concerned.

Figure 2. Service volumes versus length and rate of grade for determination of second criterion: levels of service B and C for entire highway.

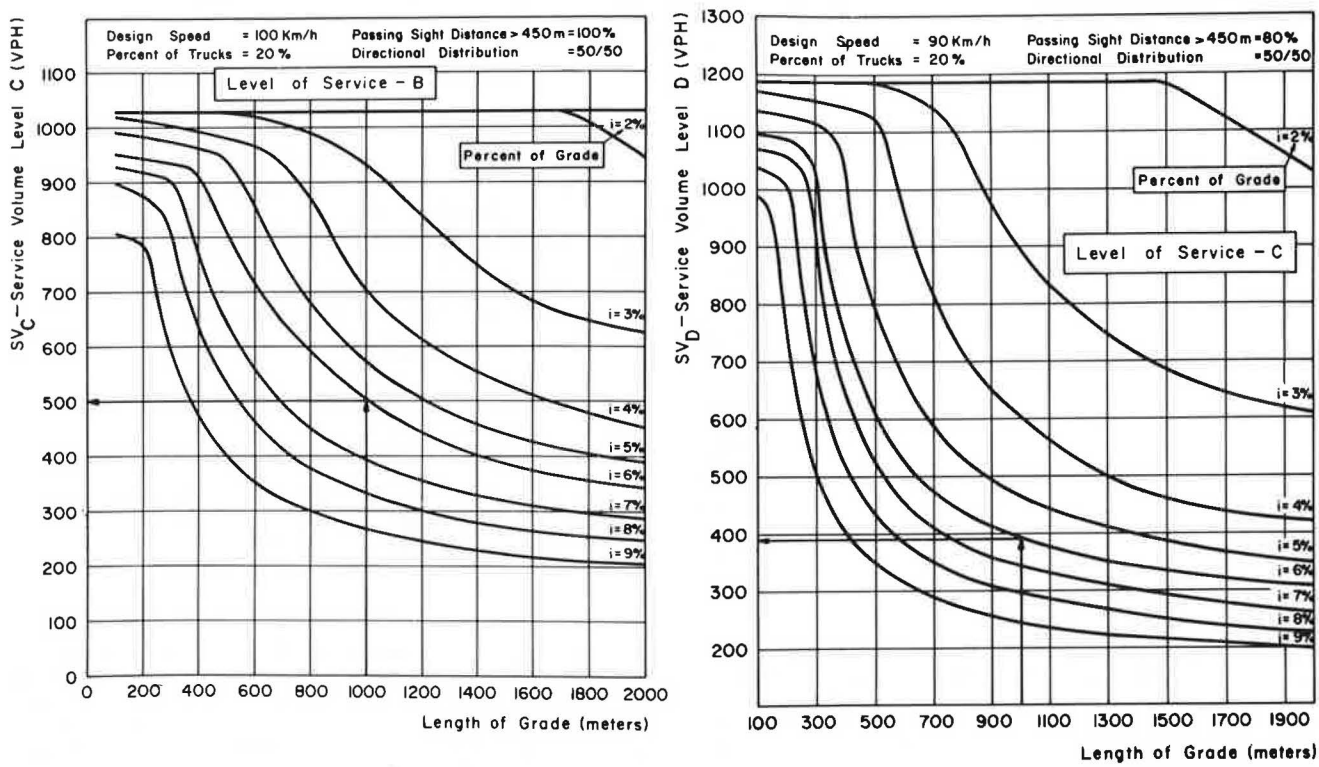
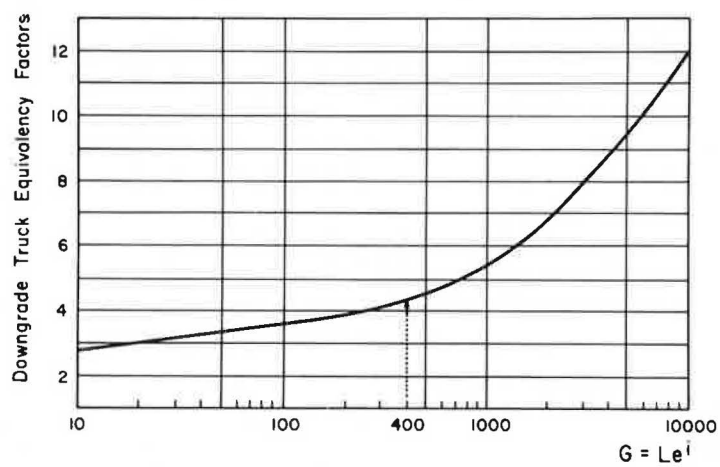


Figure 3. Suggested model for downgrade truck equivalency factors.



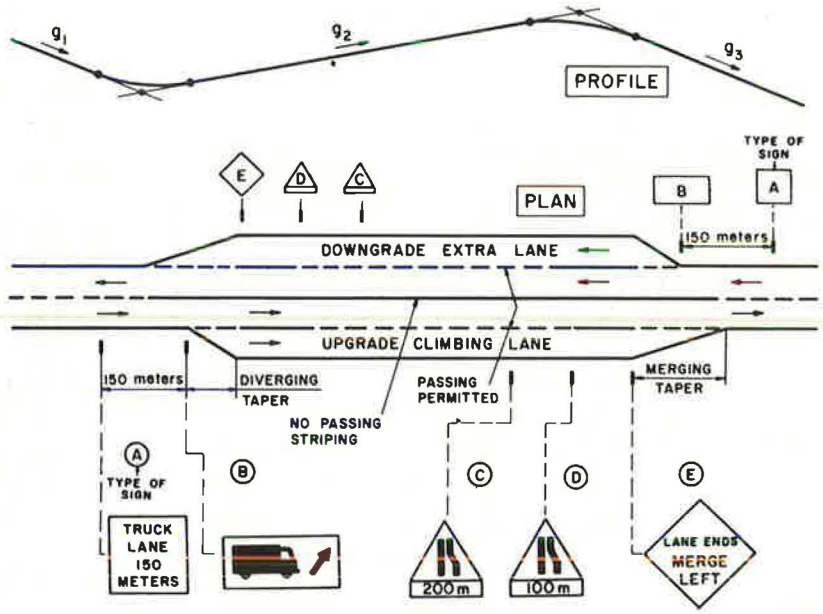
Service-volume computations needed for the analysis of the third criterion for the inclusion of a climbing lane on downgrades are based on the regular method, as outlined above. For both situations, it is assumed that heavy vehicles use the upgrade climbing lane. When passing is permitted, the situation for which calculations should be carried out is that of a two-lane, two-way road with no trucks on the upgrade through lane. When passing is prohibited, as might be the case on many downgrades, service volume should be calculated separately for the two lanes in the upgrade direction and the single downgrade lane. Then, if it is found to be justified, an additional lane should be introduced for the downgrade.

A question might be raised about the use of the maximum service volume on one downgrade lane of a rural highway on which no passing is permitted. This study had adopted the value of 1500 vehicles/h as an estimate for this maximum capacity because no other data exist. This value is the average maximum capacity of a one-lane highway, between a multilane and a two-lane highway. The maximum service volume, therefore, is calculated as discussed previously, but a value of 1500 vehicles/h is used instead of 2000 vehicles/h.

RELATIVE LOCATION AND SIGNING OF CLIMBING LANES

The point where the climbing lane should begin depends on the reduction in truck speed and the resulting physical and psychological deterrent to other traffic. Where there are no restricting conditions that cause low approach speed, the full-width extra lane may be introduced on the grade at a point defined by a reduction of 20 km/h from the average running speed on a level section (the average running speed is commonly defined in the literature).

Figure 4. Signing scheme for upgrade climbing lane and downgrade extra lane.



As for the end of a climbing lane, one opinion is that it is desirable to end the climbing lane at a point beyond the crest, where a typical truck could gain some speed (5). As a practical rule, however, the speed value adopted might be the same as that for the beginning of the climbing lane; i.e., the climbing lane should end when trucks again reach the speed as specified above.

The downgrade extra lane, if found to be justified according to the criteria discussed above, should coincide with the upgrade climbing lane. The beginning and end of an additional lane, whether upgrade or downgrade, should be preceded and followed by a tapered section based on the merging and diverging driving habits specified in so many geometric design standards.

There is a definite need to provide proper signing for climbing lanes, although no such intention is found in the guidelines of the United Nations convention. U.S. and Canadian practice (9,10) suggests that, where an extra lane has been provided for slow-moving vehicles, it should be preceded by a proper sign that directs such traffic into this extra lane and also by an advance sign erected at a distance of about 150 m before the beginning of the climbing lane. The legal meaning of these signs should be that a truck must use the extreme right lane (i.e., the climbing lane) for its entire length (no passing is permitted on ascending sections because of the poor accelerating performance of trucks) while other vehicles may use either the extra lane or the through lane. Furthermore, since the end of a climbing lane will usually be beyond the crest of the grade, at least two signs indicating reduction in pavement width should be erected, one 200 m and the other 100 m before the merge-left taper. These signs and their locations are shown in Figure 4 (note the suggested symbol sign to replace the verbal "trucks use right lane" sign). Figure 4 also shows a suggested striping method, both for the upgrade climbing lane and the downgrade extra lane.

CONCLUSIONS AND RECOMMENDATIONS

Climbing lanes are usually needed on two-lane rural highways when slow-moving trucks cause a reduction in the safety level and the possible service volume of the highway.

This paper is concerned with a level-of-service concept for the introduction of climbing lanes on two-lane rural highways, for both upgrade and downgrade directions. A suggested set of criteria is devised for this purpose, and extensive use is made of a previously developed model for truck equivalency factors for upgrades and downgrades. Since highways are constructed to serve for 20 years or more, the future DHV is compared with the service volume, at the proper level of service, and the findings are checked against the volume criterion. An independent speed criterion is also presented, as well as a scheme for the relative location and signing of the extra lanes.

Three final recommendations are suggested for further research and analysis:

1. Additional sets of graphs (or tables), similar to those presented in Figure 2, should be devised for different values of design speed and percentages of trucks for downgrades as well as upgrades.
2. The ideal capacity of a one-lane downgrade stretch should be studied in order to improve the accuracy of service-volume computations for downgrades.
3. There should be more study of the economic values of extra lanes. Safety and traffic aspects, as well as construction priorities for existing highways, should be considered.

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Effect of Lane and Shoulder Widths on Accident Reduction on Rural, Two-Lane Roads

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A study to determine the effect of lane and shoulder widths on accident benefits for rural, two-lane roads and to determine the expected cost-effectiveness of lane and shoulder widening is described. Information concerning geometrics, accidents, and traffic volumes was obtained for more than 25 000 km (15 000 miles) of roads. Run-off-road and opposite-direction accidents were the only accident types found to be associated with narrow lanes and shoulders. Wide lanes had accident rates 10-39 percent lower than those for narrow lanes. Wide shoulders [up to 2.7-m (9-ft) width] were associated with the lower accident rates. Criteria based on a cost-effectiveness approach for selecting highway sections for widening are presented.

A question facing highway engineers is whether to widen lanes and shoulders on existing rural roads to provide improvements in rideability, capacity, and safety. Limited funds require the implementation of those improvements that are most cost effective. Before lane and shoulder improvements are implemented, the relation between width of lanes (and shoulders) and accident experience on different types of roads should be ascertained.

Design standards for pavement (driving lanes) and shoulder width most often depend on traffic volume and design speed (1, p. 91; 2). Standards for the paved surface (pavement plus shoulders) also have been set for two-lane roads on the basis of an economic analysis of construction, maintenance, and accident costs (3).

Previous studies resulted in a variety of findings concerning the effects of pavement width on accidents. Little or no information exists on the economic benefits (if any) expected from wider lanes and shoulders. The purpose of this study was to answer some of the questions regarding the safety benefits of pavement and shoulder widening.

BACKGROUND

Lane Width and Safety

On 5.5-m (18-ft) wide pavements, cars pass oncoming trucks at clearances averaging only 0.8 m (2.6 ft). On 6.1-m (20-ft) wide pavements, average clearances are 1.1 m (3.5 ft). When a truck meets an oncoming truck, clearance distances are less. Trucks overtaking other trucks remain centered in their lanes only when lanes are 3.7 m (12 ft) wide or wider. Clearances for cars overtaking other cars are only 0.7 m (2.3 ft) on 5.5-m-wide pavements and 1.5 m (4.8 ft) on 7.3-m (24-ft) wide pavements (4).

In Illinois, the widening of a 5.5-m-wide pavement to 6.7 m (22 ft) caused a reduction of from 143 to 89 accidents/million vehicle-km (230-140 accidents/million vehicle miles), a 39 percent reduction (5,6). In Louisiana, it was concluded that narrow lanes contribute significantly to injury and fatality accidents and wet-weather accidents. There, accident rates on rural roads decreased from 1.5 accidents/million vehicle-km (2.4 accidents/million vehicle miles) on 2.7-m (9-ft) wide lanes to 1.1 on 3.1-m (10-ft) wide lanes and 0.9 on 3.4- and 3.7-m (11- and 12-ft) wide lanes (6,7).

Shoulder Width and Safety

Several previous studies involving rural, two-lane roads have included correlations of shoulder width with accident occurrence. Considerable variation in findings has been cited. A study in Oregon (8) concluded that total accidents increase with increasing shoulder width, except for roads that have average annual daily traffic (AADT) of 3600-5500. Shoulders wider than 2.4 m (8 ft) experienced significantly more accidents than shoulders 0.9-1.2 m (3-4 ft) wide (9). In Connecticut, all accident types decreased with increased shoulder width for AADTs between 2600 and 4500. In another study (10), a reverse correlation existed for AADTs less than 2600. Only a slight correlation was noted between shoulder width and accidents in Louisiana (7).

Others have found a definite benefit from wide shoulders. In California (for most AADT ranges), about twice as many injury accidents occurred on roads with shoulders 0.3-0.9 m (1-3 ft) wide than for shoulders wider than 1.8 m (6 ft) (11). In New York, reductions in accidents were observed as shoulder width increased, especially in the 2000-6000 AADT range; no correlation was found for AADTs less than 2000 (12). In another study in New York, it was concluded that shoulders 1.2-1.5 m (4-5 ft) wide were adequate on roads with good alignment but shoulders more than 2.4 m (8 ft) wide were preferred on roads with poor geometrics (13).

A number of studies on shoulder widths indicate a lack of correlation with accidents on two-lane roads where AADTs are less than 2000. Wide shoulders appear to be most beneficial where AADTs are between

3000 and 5000. Shoulders 1.2-2.1 m (4-7 ft) wide were preferred to wider ones. Others suggested that shoulders as wide as 3.1-3.7 m (10-12 ft) were the safest.

However, the economic justification for widening shoulders has not yet been determined for rural, two-lane roads. Several geometric variables were found to be significant in accident occurrences in some of the studies. Lane width, access control, conflict points per mile, cross slope of shoulder, traffic volumes, and sight distance were all mentioned as variables that have a greater effect on accident experience than shoulder width.

Shoulder Stability

To derive full benefits from shoulder improvements, it is very important for the shoulders to be stable. Shoulders should support vehicle loads in all kinds of weather. The possibility of a vehicle skidding out of control or turning over is increased when the shoulder is soft or is covered with loose gravel, sand, or mud.

In a study of the cost-effectiveness of paved shoulders in North Carolina, a significantly lower accident experience and severity index were associated with paved shoulders on two-lane roads in comparison with unpaved shoulders on similar highway sections. Shoulders 0.9-1.2 m (3-4 ft) wide were predominant in that study. In some cases, paving of shoulders was cost effective (based only on accident reductions) within 10-20 years, depending on traffic volume (14).

Shoulder stabilization on two-lane roads in Ohio resulted in a reduction of 38 percent for all accidents and 46 percent for injury and fatality accidents. The criterion for stabilizing shoulders was that a minimum of 45 percent of the accidents be run-off-road and head-on collisions (15).

Capacity Considerations

Relations between lane width, shoulder-width lateral clearance, and capacity can be obtained from the Highway Capacity Manual (16). Expected increases in capacity due to wider lanes or shoulders can be estimated from such relations.

PROCEDURE FOR RELATING ACCIDENT OCCURRENCE TO LANE AND SHOULDER WIDTH

To compare accident occurrences for various lane and shoulder widths, two different procedures may be followed. The first would involve conducting a before-and-after analysis of accidents for sections that were widened. This method has several shortcomings:

1. A very limited sample size for such an analysis is normally available.
2. Such improvements often include other improvements such as delineation, skid resistance, realignment, and shoulder leveling, which also affect accident experience to an unknown extent.
3. Additional traffic may be generated by such improvements, and this may affect accidents.

The other procedure may be termed a "comparative analysis", since it compares accident experiences for existing highway sections where geometric and accident data are known. Sections of similar geometrics can be grouped for analysis. Because this technique usually allows for a large data base without relying on improved sections, it was selected for use in this study.

The accident records used consisted of nearly 17 000 accidents reported in 1976 that were investigated by state, county, and city police agencies and stored on computer tape. Highway traffic and geometric data were also obtained from computer tape. Data from both sources were coded by county number, route number, and milepost. Accident summaries were carefully merged with the traffic and geometric data on a third computer tape.

Only rural highways classified as state primary, state secondary, or rural secondary routes were selected. In addition, only two-lane roads were considered, since most four-lane highways did not warrant an in-depth investigation at this time.

Highway sections that include abrupt changes such as major intersections and changes in roadway width or access control were considered undesirable, since they were believed to bias the data. Therefore, all nonuniform sections of road were omitted. By using the above criteria to select a test sample, a total of 25 670 km (15 944 miles) of roads was included in the analysis. A total of eight classifications based on AADT was used (see Table 1).

Information input included the location (county, route, and milepost), lane width, shoulder width, AADT, road classification, pavement type (bituminous or concrete), shoulder type (bituminous, dense-graded aggregate, or other), number of lanes, access control (full, partial, or permit), and number of public approaches (access points). A computer program was then written that matched accident records with each 1.6-km (1-mile) section of highway. The number of accidents for each section was summarized according to several geometric features, weather conditions, severity of accidents, and types of accidents.

Certain other variables were not available, including skid number, shoulder slope, and number and degree of vertical and horizontal curves. Because of the large data sample [about 26 000 km (16 000 miles)], much of the influence of these variables on accidents was minimized when sections were grouped for analysis. In addition, the classification of accidents by type (rear-end, run-off-road, opposite-direction, driveway-related, etc.) allowed for the exclusion of most accidents that were unrelated to lane and shoulder widths.

After accident data were summarized, relations between accidents and various geometric characteristics were determined. Several hundred summary tables were generated that gave cumulative accident numbers for each lane width, shoulder width, AADT, highway classification, access control, etc. This allowed for the use of control variables to determine the true effect of lane and shoulder width on accident experience. All accident rates were expressed as combined averages to ensure data stability.

RELATIONS BETWEEN ACCIDENT RATES AND HIGHWAY CHARACTERISTICS

Lane Width

For this analysis, lane widths were rounded to the nearest 0.3 m (1 ft). Accident and traffic-volume statistics for lane widths of 2.1-4.0 m (7-13 ft) are given in Table 2. Accidents were classified as either run-off-road, opposite-direction (head-on or sideswipe collision between opposing vehicles), rear-end, passing, driveway and intersection, or collisions with pedestrians, bicycles, animals, and trains. The most common accidents, considering all lane widths, were run-off-road, opposite-direction, and rear-end. Rates were the highest for run-off-road and opposite-direction accidents for narrow lanes and decreased steadily as lane width in-

Table 1. Distribution of test sites by traffic volume and route type.

AADT	Number of Test Sites[a]			
	State Primary	State Secondary	Rural Secondary	Total
0 to 500	38	1462	6283	7 783
501 to 1 000	175	1730	1124	3 029
1 001 to 2 500	969	1884	369	3 222
2 501 to 5 000	794	604	47	1 445
5 001 to 7 500	180	124	6	310
7 501 to 10 000	66	47	1	114
10 001 to 15 000	18	13	0	31
15 001 to 20 000	3	7	0	10
Total	2243	5871	7830	15 944

[a]Test sites 1.6 km (1 mile) in length.

Table 2. Lane width and accidents.

Lane Width (m)	Sample Size (km)	Number of Accidents	Accidents per Kilometer	Average AADT	Accidents per Million Vehicle Kilometers
2.1	637	123	0.19	205	2.58
2.4	4 518	1 143	0.25	304	2.28
2.7	13 273	6 652	0.50	729	1.88
3.0	4 082	4 947	1.21	1862	1.78
3.4	1 268	2.017	1.59	3410	1.28
3.7	981	1 743	1.78	3970	1.23
4.0	61	135	2.21	4483	1.35
Total	24 820	16 760	0.68	1099	1.68

Notes: 1 m = 3.3 ft; 1 km = 0.62 mile.
Table was generated before controlling for the effects of traffic and other highway variables.

Table 3. Shoulder width and accidents.

Shoulder Width (m)	Sample Size (km)	Number of Accidents	Accidents per Kilometer	Average AADT	Accidents per Million Vehicle Kilometers
None	17 887	8 790	0.49	751	1.79
0.3-0.9	6 661	6 610	0.99	1578	1.72
1.2-1.8	163	370	2.27	3566	1.74
2.1-2.7	138	188	1.36	3693	1.01
3.0-3.7	553	964	1.74	4088	1.17
Total	25 402	16 922	0.67	1074	1.70

Notes: 1 m = 3.3 ft; 1 km = 0.62 mile.
Table was generated before controlling for the effects of traffic and other highway variables.

creased. Rates for other accidents generally increased as lane widths increased. Thus, the only accidents that would be expected to decrease with lane widening were run-off-road and opposite-direction accidents.

Injury and fatality rates for each lane width were also computed. Rates of property-damage and injury accidents decreased as lane width increased, corresponding to the overall accident rate for various lane widths. No changes in fatality rate occurred as lane width changed. In addition, the percentage of injury and fatality accidents increased slightly and then decreased as lane width increased. No definite relation was found between lane width and accident severity.

Shoulder Width

Of the total sample, about 70 percent of the test sections had no shoulders. Only paved or dense-graded shoulders were considered as shoulders. Grass and soil are not suitable driving surfaces and therefore normally do not function as shoulders.

Table 4. Accident rates for various combinations of lane and shoulder widths on rural, two-lane highways.

Lane Width (m)	Shoulder Width (m)	No. of 1.6-km Sections	Accidents per Million Vehicle Kilometers	
			All	Opposite-Direction and Run-off-Road
2.1	None	286	2.92	3.16
	0.3-0.9	110	1.06	1.21
	1.2-1.8	0		
	2.1-2.7	0		
	3.0-3.7	0		
2.4	None	2460	1.84	2.24
	0.3-0.9	344	2.13	2.52
	1.2-1.8	1		
	2.1-2.7	1		
	3.0-3.7	0		
2.7	None	6032	1.38	1.97
	0.3-0.9	2185	1.19	1.78
	1.2-1.8	9	0.83	1.81
	2.1-2.7	6	0.76	
	3.0-3.7	4		
3.0	None	1384	1.14	1.87
	0.3-0.9	1080	1.01	1.70
	1.2-1.8	23	0.74	1.93
	2.1-2.7	8	0.64	1.84
	3.0-3.7	12	0.64	1.58
3.4	None	382	0.64	1.16
	0.3-0.9	275	0.63	1.37
	1.2-1.8	31	0.50	1.37
	2.1-2.7	21	0.32	0.53
	3.0-3.7	38	0.52	1.37
3.7	None	168	0.48	1.19
	0.3-0.9	87	0.67	1.51
	1.2-1.8	27	0.61	1.40
	2.1-2.7	34	0.44	1.13
	3.0-3.7	26	0.56	1.16

Note: 1 m = 3.3 ft; 1 km = 0.62 mile.

Because of the small sample sizes for some shoulder widths, considerable differences were found in accident rates. Shoulder widths were categorized as no shoulder, 0.3-0.9 m (1-3 ft), 1.2-1.8 m (4-6 ft), 2.1-2.7 m (7-9 ft), and 3.0-3.7 m (10-12 ft), as given in Table 3. The poor relation between shoulder width and all accidents was expected before controlling for other factors such as lane width and volume. The small sample of locations for shoulder widths greater than 0.9 m (3 ft) may also be a factor.

Accident types and rates were summarized for various shoulder widths. As with lane width, the run-off-road and opposite-direction accident rates decreased as shoulder width increased to 2.7 m (9 ft). There was a slight increase in rate for shoulders 3.0-3.7 m (10-12 ft) wide. Rates for other than run-off-road and opposite-direction accidents tended to remain fairly constant or increase slightly as shoulder width increased.

Rates for property-damage, injury, and fatality accidents were calculated. As before, rates for each type generally decreased as shoulders widened, but the percentage of injury and fatality accidents did not show any trends. No reduction in average accident severity, therefore, may be expected from shoulder widening.

Combinations of Lane and Shoulder Widths

An analysis was made of accident rates for various combinations of lane and shoulder widths. For all accidents (see Table 4), rates on roads that had no shoulders decreased from 2.9 to about 0.5 accidents/million vehicle-km (4.6-0.8 accidents/million vehicle miles) as lane width increased from 2.1 to 3.7 m (7-12 ft). For other shoulder widths, accident rates generally decreased with increasing lane width, although the relations were not as pronounced.

For the same lane widths, accident rates tended to decrease as shoulder width increased. Overall, the decrease in accident rate was greater for increases in lane width than for equivalent increases in shoulder width. When only run-off-road and opposite-direction accidents were used (Table 4), more uniform decreases in accident rates were found in most cases than when all accidents were included. Again, increases in lane width resulted in a greater reduction in accident rates than the same increases in shoulder width.

These analyses appear to indicate that a greater reduction in accidents can be realized by lane widening than by shoulder widening. Although little reduction in accidents may be gained by widening a 6.8-m (22-ft) wide road to a 7.4-m (24-ft) pavement, the added width would provide slightly better service to users in terms of capacity and safe driving speed.

Other Highway Features

The previous summaries of accidents by lane and shoulder widths were analyzed to determine the possible influence of other highway features on accident experience. The effect of traffic volume, highway type, and access control on accidents was examined in detail.

This analysis was intended to quantify that portion of the change in accident rates that can be attributed to lane and shoulder width. For example, the average accident rate on roads with 2.1-m (7-ft) wide lanes was 2.58 accidents/million vehicle-km (4.16 accidents/million vehicle miles) compared with a rate of 1.28 accidents/million vehicle-km (2.08 accidents/million vehicle-miles) for lanes 3.4 m (11 ft) wide. This difference may be partly due to the wider lanes and partly to other unidentified causes. For example, narrow roads usually have less access control and lower volumes than wider roads. Both of these factors may be a primary cause of the higher accident rate for narrower roads. Therefore, a separate analysis of the effects of some of these other highway features on accident experience was performed.

Traffic Volume

Accidents per kilometer increased considerably with AADT (see Figure 1). The relation between traffic volume and accident rate is shown in Figure 2 for all sections [more than 24 000 km (15 000 miles)] of rural, two-lane roads. In this case, the rate decreased significantly as AADT increased, particularly for AADTs greater than 1000.

It appears from Figure 2 that lower accident rates are associated with higher traffic volumes. However, higher volumes were also associated with higher classes of roads, which normally have wider lanes and shoulders and less and more gradual curvature than lower-volume facilities. To determine how accident rates were affected by volume alone, summaries were made of rates as a function of volumes for specific highway types and lane widths. To also control other geometric variables, only routes with no shoulder and with <2.5 public approaches (access points)/km (4.0 approaches/mile) were included. No clear relations were found. Rates for each classification and lane width remained roughly the same or fluctuated slightly as AADT increased. This may be expected, since all accident types were included in the calculation of accident rates.

Previous research has shown that single-vehicle accidents are affected differently than multivehicle accidents as AADT increases. This was verified by data reviewed in this study (see Figure 3). Results may be different for test sections containing an intersection. The probable reason that the rate of run-off-road (single-vehicle) accidents decreased as AADT increased is that vehicles tend to be driven slower because passing may not be possible. On low-volume roads, vehicles are not able to caravan (follow each other in groups) and unfamiliar motorists may take curves at excessive speeds, particularly at night or in the rain. At night, motorists sometimes follow taillights ahead of them, which help to warn them of sharp curves.

Since the rate of run-off-road accidents decreases as both lane width and AADT increase, the effect of lane width alone on the rate of run-off-road accidents was determined. The rate of run-off-road accidents was plotted versus AADT for different lane widths (see Figure 4). By controlling for the other variables, the slopes of the lines indicate the effect of AADT on rates, and the vertical distances between lines indicate the effect of lane width on accident rates. Most of the decrease (72 percent) in accident rate was related to volume changes, and 28 percent resulted from wider pavements.

The effect of traffic volume on opposite-direction accidents was also determined with respect to various pavement widths (see Figure 5). The wider pavements were associated with about 76 percent of the decrease in the rate of opposite-direction accidents (Table 3). As can be seen in Figure 5, the greatest reduction in accident rate per foot of widening can be achieved by widening the narrow pavements [4.3-4.9 m (14-16 ft)] to medium width [5.5-6.1 m (18-20 ft)]. The effect of traffic volume on accident rates was determined in a similar manner in the analysis of shoulder widths.

Access Points

Another geometric feature thought to have some influence on accident rates was the effect of access points per kilometer. This is the number of public approaches or minor entrances onto the highway that could adversely affect accident rates.

More access points per kilometer were associated with higher accident rates for virtually all lane-width categories, as shown in Figure 6. However, only about 6 percent [1600 km (1000 miles)] of the sample had 3.1 access points/km (≥5 access points/mile). Those sections were distributed evenly throughout the test sections.

Highway Classification

Another control variable that was studied was the effect of highway classification on accident rate. Rates were compared for each lane width for rural secondary, state secondary, and state primary routes while the other variables were controlled. For lanes 2.7 m (9 ft) wide, accident rates were generally higher for rural secondary routes and lower for state primary routes. For 3.0-m (10-ft) wide lanes with low AADTs, a similar trend was found. However, as AADT increased, rates became highest for state primary routes. This could indicate that 3.0-m-wide lanes are not acceptable for state primary roads that have high traffic volumes. For lanes 3.4 m (11 ft) wide, no obvious differences were found in accident rates between state secondary and state primary routes.

SAVINGS DUE TO ACCIDENT REDUCTION

Savings due to accident reductions were the only benefits included in the economic analysis. Lane and shoulder widths were shown previously to have an

Figure 1. Accidents versus traffic volume: all accidents.

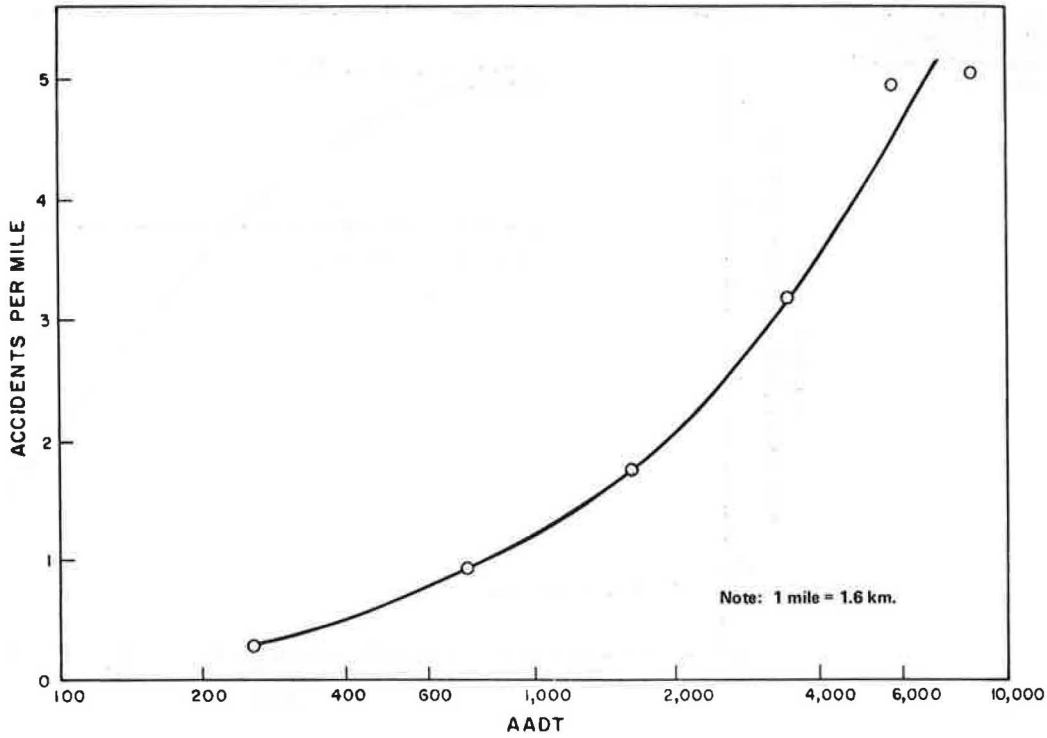
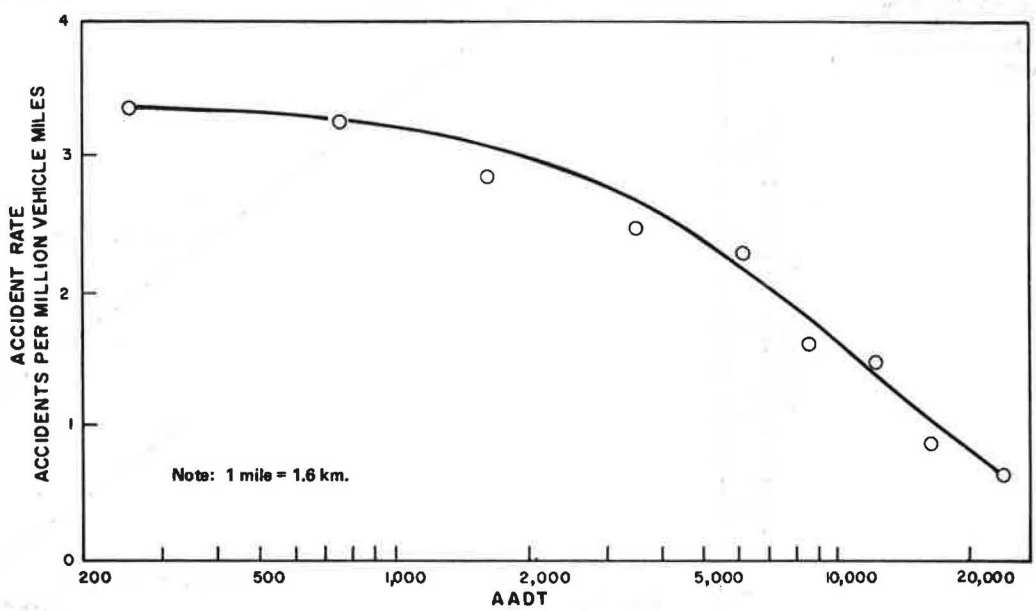


Figure 2. Accident rate versus traffic volume: all accidents.



effect on only run-off-road and opposite-direction accidents. Other types of accidents did not decrease as a function of wider lanes and shoulders. Thus, average costs were computed only for these two categories.

Of all run-off-road and opposite-direction accidents, 40.3 percent involved injuries or fatalities compared with only 19.6 percent for the other types of accidents. The percentage of fatal and A-injury accidents was nearly three times as high for run-off-road and opposite-direction accidents as for all other types.

The severity index was computed by using a formula developed in a 1973 study (17):

$$SI = [9.5(K + A) + 3.5(B + C) + PDO]/N \quad (1)$$

where

SI = severity index,
K = number of fatal accidents,
A = number of A-type injury accidents,
B = number of B-type injury accidents,
C = number of C-type injury accidents,
PDO = number of property-damage-only accidents, and
N = total number of accidents.

The combined severity index of the run-off-road and opposite-direction accidents was 2.74 compared with 1.74 for the other accidents.

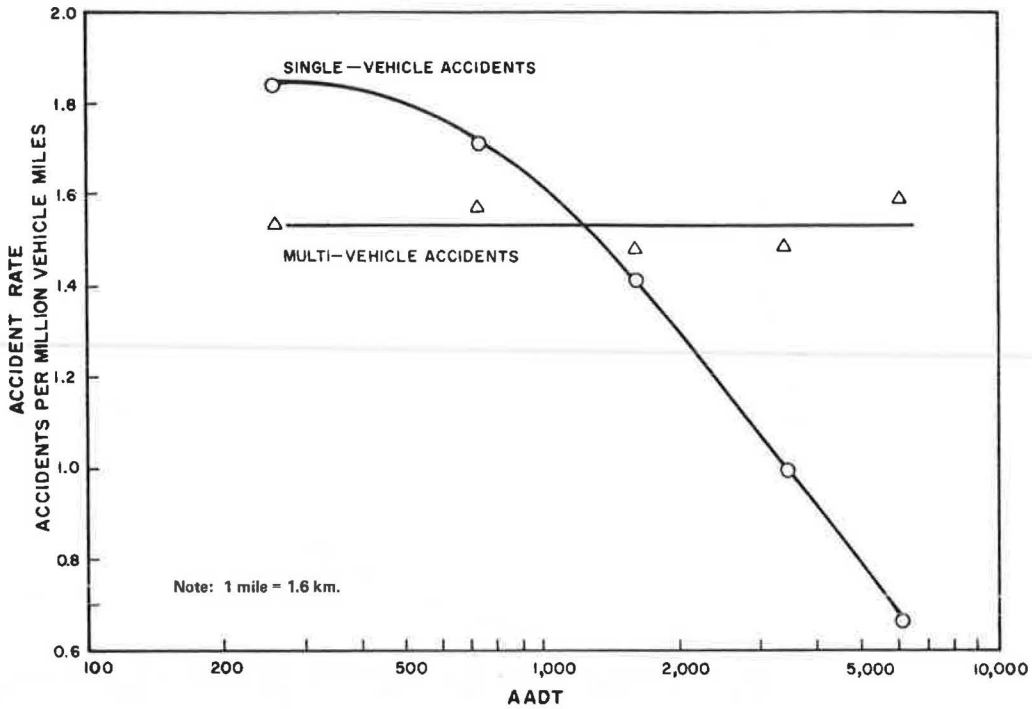


Figure 3. Effect of traffic volume on single- and multiple-vehicle accident rates.

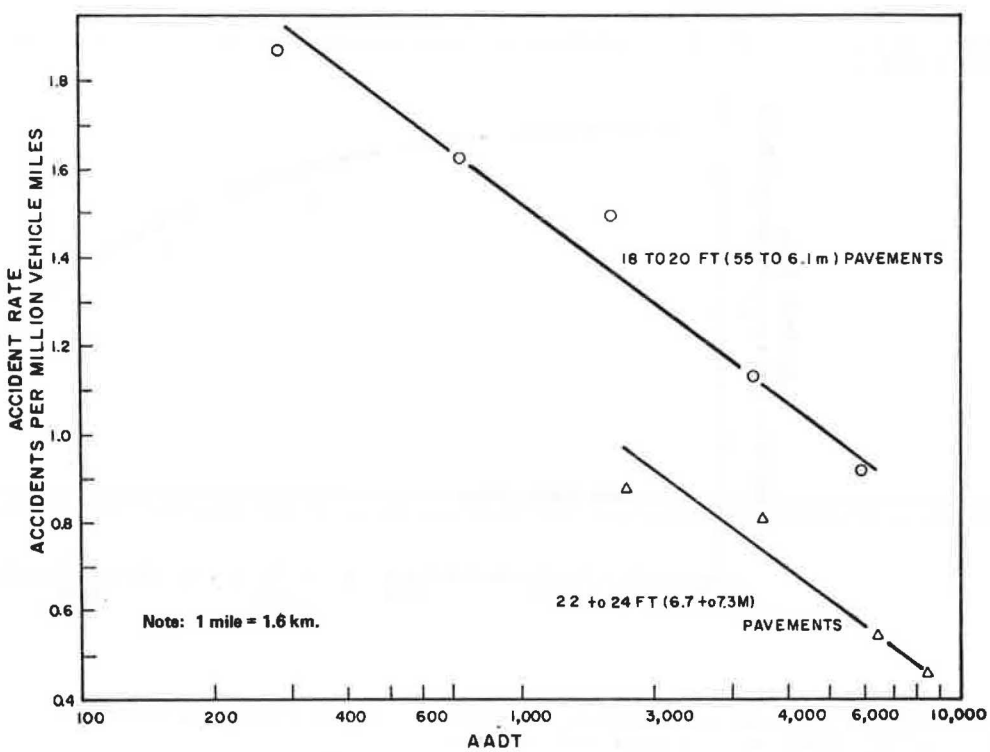


Figure 4. Rate of run-off-road accidents versus traffic volume by lane width.

The average cost per accident was computed for use in the calculation of expected accident savings. The following accident costs reported by the National Safety Council for 1976 (18) were used:

Accident Category	Cost per Accident ($)
Fatality	125 000
Nonfatal, disabling injury	4 700
Property damage only	670

The average cost of a run-off-road or opposite-direction accident was $5569 compared with $2199 for other accident types on rural, two-lane roads.

Lane Width

The expected reduction in accident rate was computed and plotted for various degrees of lane widening (see Figure 7). The values represent reductions in the combined rate of run-off-road and opposite-direction accidents after controlling for other high-

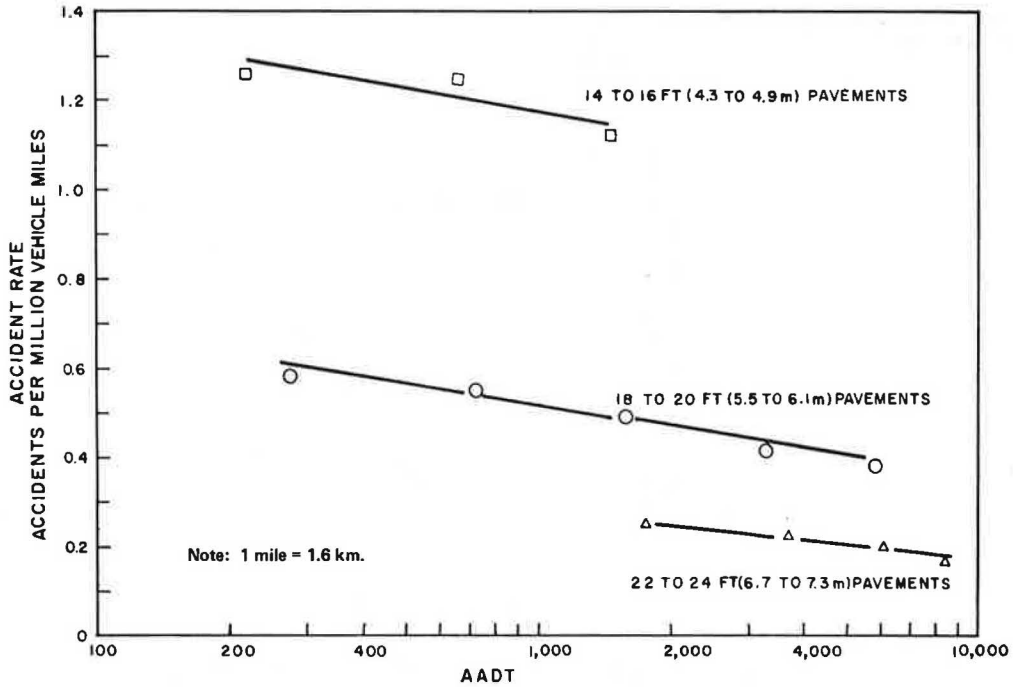


Figure 5. Rate of opposite-direction accidents versus traffic volume by lane width.

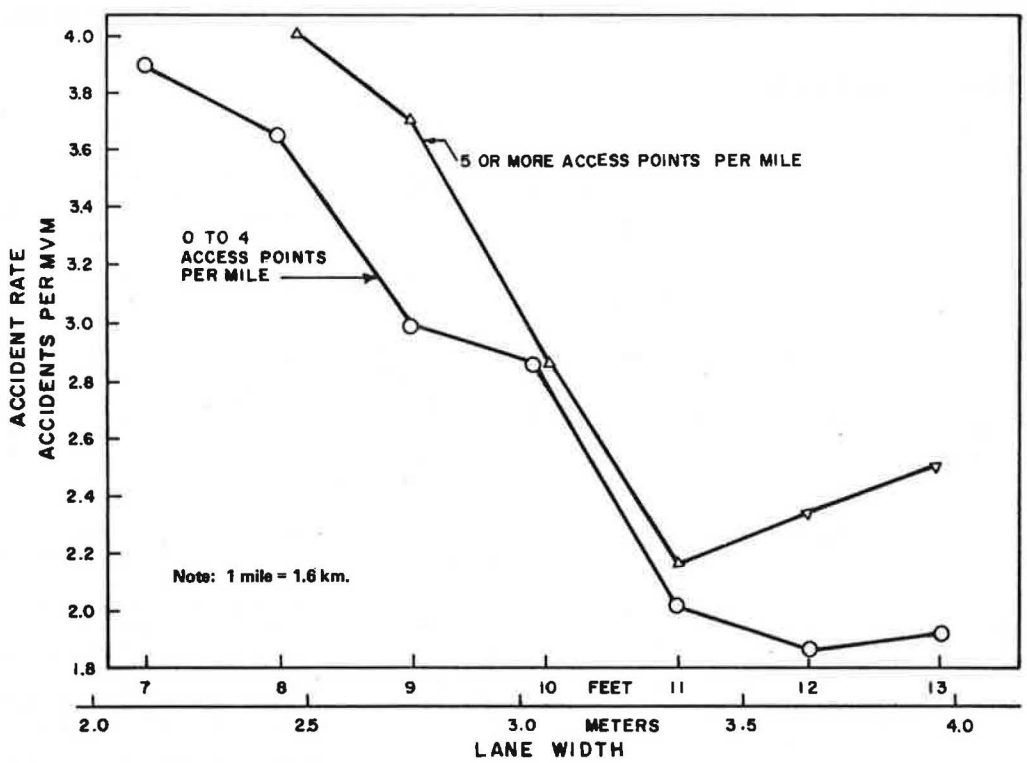


Figure 6. Effect of access points per kilometer on accident rate for various lane widths.

way and traffic variables. Note that very little additional benefit is realized by widening a lane more than 3.4 m (11 ft). The relation for percentage reduction in run-off-road and opposite-direction accidents for various degrees of pavement widening was determined and is given below (1 m = 3.3 ft):

Lane Width (m)		Reduction in
Before	After	Accidents (%)
2.1	2.4	10
2.1	2.7	23
2.1	3.0	29
2.1	3.4	39
2.4	2.7	16
2.4	3.0	23
2.4	3.4	36
2.7	3.0	10
2.7	3.4	29
3.0	3.4	23

For example, on an average section of rural, two-

Figure 7. Reduction in accident rate versus lane widening.

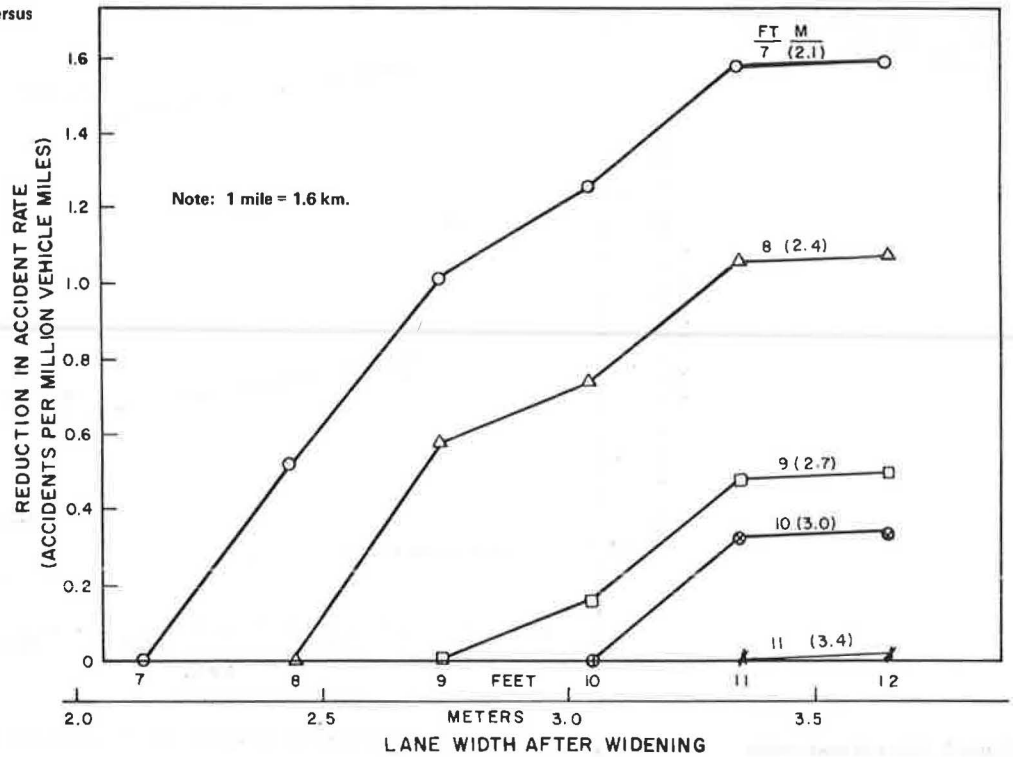


Table 5. Cost of pavement widening.

Pavement Width (m)		Cost per Mile ($)					
Before	After	Grade and Drain	Subgrade	Overlay	Other	Total	Cost per Foot ($)
4.3	5.5	93 943	14 900	19 764	5200	133 807	33 452
4.3	6.1	113 079	22 350	21 960	6647	164 036	27 339
4.3	6.7	132 216	29 800	24 156	8093	194 265	24 283
4.3	7.3	151 352	37 250	26 352	9540	224 494	22 449
4.9	6.1	93 943	14 900	21 960	5529	136 332	34 083
4.9	6.7	113 079	22 350	24 156	6976	166 561	27 760
4.9	7.3	132 216	29 800	26 352	8423	196 791	24 599
5.5	6.7	93 943	14 900	24 156	5858	138 857	34 714
5.5	7.3	113 079	22 350	26 352	7305	169 086	28 181
6.1	6.7	74 807	7 450	24 156	4741	111 154	55 577
6.1	7.3	93 943	14 900	26 352	6188	141 383	35 346
6.7	7.3	74 807	7 450	26 352	5070	113 679	56 840

Note: 1 m = 3.3 ft; 1 mile = 1.6 km; 1 ft = 0.3 m.

Table 6. Cost of shoulder widening.

Shoulder Widening[a] (m)	Cost per Mile ($)				Cost per Foot ($)
	Grade and Drain	Shoulder Stabilization	Shoulder Surfacing	Total	
0.3	19 832	3 568	1 834	25 234	12 617
0.6	26 965	7 136	3 668	37 769	9 442
0.9	34 445	10 704	5 502	50 651	8 442
1.2	42 274	14 272	7 336	63 882	7 985
1.5	50 451	17 840	9 170	77 461	7 746
1.8	58 106	21 408	11 004	90 518	7 543
2.1	65 761	24 976	12 838	103 575	7 398
2.4	73 416	28 544	14 672	116 632	7 290

Note: 1 m = 3.3 ft; 1 mile = 1.6 km; 1 ft = 0.3 m.
[a]Each side.

lane road, widening lanes from 2.4 to 3.4 m (8-11 ft) would be expected to reduce run-off-road and opposite-direction accidents by 36 percent.

Shoulder Width

The expected reductions in combined accident rates for run-off-road and opposite-direction accidents were computed in a similar manner. No additional benefit is obtained on rural, two-lane roads by widening shoulders to more than 2.7 m (9 ft). The percentage reduction in run-off-road and opposite-direction accidents for various amounts of shoulder widening was calculated after controlling for access control, highway classification, AADT, and lane width and is given below (1 m = 3.3 ft):

Shoulder Width (m)		Reduction in Accidents (%)
Before	After	
None	0.3-0.9	6
None	1.2-1.8	15
None	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

For an average section of rural, two-lane highway, widening the shoulders (both sides of the road) from 0.5 to 2.5 m (1.6-8.2 ft) should reduce run-off-road and opposite-direction accidents by 16 percent.

IMPROVEMENT COSTS

Costs (average for Kentucky) associated with pave-

ment widening were determined from historical records of costs (Table 5) (19). Costs per kilometer for 1 m (3.3 ft) of widening ranged widely and depended on the increase in pavement width. All pavements were assumed to require a full-width overlay. Costs for shoulder widening also varied, depending on the amount of widening (see Table 6). All shoulders were assumed to require stabilization and surfacing.

In widening lanes and shoulders, existing right-of-way is normally used. Major reconstruction projects that involve right-of-way acquisition were not considered here. Because of the great variation in terrain and soils throughout Kentucky, the costs differed considerably. Adequate room to widen the pavement for shoulders may be available on some roads and not on others. The costs given here are average values based on past contract prices adjusted to 1976 dollars. It should be noted that such costs were considerably different from similar construction costs in other states because of differences in such factors as terrain and construction techniques.

COST-EFFECTIVENESS ANALYSIS

To determine the cost-effectivensss of lane and shoulder widening, benefit-cost ratios can be used to priority rank the projects. Average statewide costs based on past contract prices in Kentucky (Tables 5 and 6) were used. More exact costs should be used for a particular project whenever available. Benefits should be computed in terms of present worth based on the following formula:

$$B_{pw} = (C_a)(R)(N)(PWF) \quad (2)$$

where

B_{pw} = present-worth benefits expected from a highway improvement ($),
R = annual percentage reduction in opposite-direction and run-off-road accidents due to widening (as given in the two text tables above),
C_a = average cost of each accident affected by the improvement ($5569 for opposite-direction and run-off-road accidents),
N = annual number of accidents affected by improvements, and
PWF = present-worth factor used to convert benefits to present values.

The PWF is based on the interest rate, the AADT growth factor, and the expected service life of the improvement. The interest rate selected was 8 percent. An exponential growth factor of 4 percent was assumed for the AADTs on rural, two-lane roads in Kentucky to reflect recent volume trends. This was also in agreement with traffic growth nationwide from 1975 to 1976 on all non-Interstate routes (20). Laneand shoulder-widening projects were considered to have a 30-year life, assuming proper maintenance. A recent study in Idaho included benefits and costs from pavement widening and assumed a useful service life of 30 years (3). The appropriate PWF (17.62) was selected (21).

Based on the equation given previously, calculated benefits depend on the percentage of accident reduction. Estimates of present-worth benefits can be obtained from Figure 8. To determine how much lanes or shoulders should be widened to obtain the optimal benefits per dollar spent, plots of benefit-cost ratios versus number of accidents, similar to that shown in Figure 9, were developed. Figure 9 shows the benefit-cost ratios expected when 2.1-m (7-ft) wide lanes are widened to 3.4 m (11 ft). As stated before, little if any additional benefits accrue by widening a pavement to more than 3.4 m on rural, two-lane roads. Approximately five accidents per year would prequalify a section in terms of accident benefits (benefit-cost ratio of 1.0). Similar analyses for other initial lane widths were also plotted. Such plots indicate that widening pavements to at least 3.4 m may be optimal, based on cost-effectiveness, for all existing lane widths.

If a two-lane highway with lane widths greater than 3.0 m (10 ft) has at least five run-off-road and/or opposite-direction accidents per year, shoulder widening should be considered. Since shoulder widths were grouped for purposes of accident analysis, average shoulder width in each group was used in the economic analysis.

For pavements without shoulders, the optimal shoulder widening, in terms of benefit-cost ratios, would be 1.5 m (5 ft) (see Figure 10). Slightly more than five accidents per year would be required to result in a benefit-cost ratio greater than 1.0. For 0.6-m (2-ft) wide shoulders, widening to 1.5 m would be more cost-effective than widening to 2.4 m (8 ft).

For this study, all 1.6-km sections with at least two opposite-direction or three run-off-road accidents were selected from the sample data. The average statewide accident rate was then computed for run-off-road and opposite-direction accidents on rural, two-lane roads. For 1976, this statewide average rate was 1.02 accidents/million vehicle-km (1.65 accidents/million vehicle miles) and was used to select highway sections with critically high accident rates.

IDENTIFYING HIGHWAY SECTIONS FOR IMPROVEMENT

The next step involved the identification and ranking of sections of highway for consideration of widening. There were 350 sections (1.6 km each) that had critically high accident rates. A priority listing of the top 631 highway sections based on widening needs was made.

The next step was to determine what improvements, if any, should be recommended at the highest-priority locations. For this, a detailed study of all accident reports was recommended for each section under consideration. A field inspection should follow.

For those sections for which widening is recommended, a benefit-cost analysis will show which improvements would be the most cost effective. Based on the projected benefits and costs for widening each section, priority listings can be prepared for lane-widening and shoulder-widening projects.

It is recommended that, each year, 1.6-km highway sections that have >3.1 accidents/km (>5 accidents/mile) of the run-off-road or opposite-direction type and have narrow lanes or shoulders be identified. These locations should then be analyzed for cost-effectivensss and ranked separately as lane- and shoulder-widening projects. Those that qualify for widening should be investigated in the field; cost estimates should be prepared for all widening alternatives. These projects should then be considered along with other safety improvement projects for implementation.

ACKNOWLEDGMENT

The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Kentucky Bureau of Highways. This paper does not

Figure 8. Present-worth benefits for various accident histories and percentages of accident reduction.

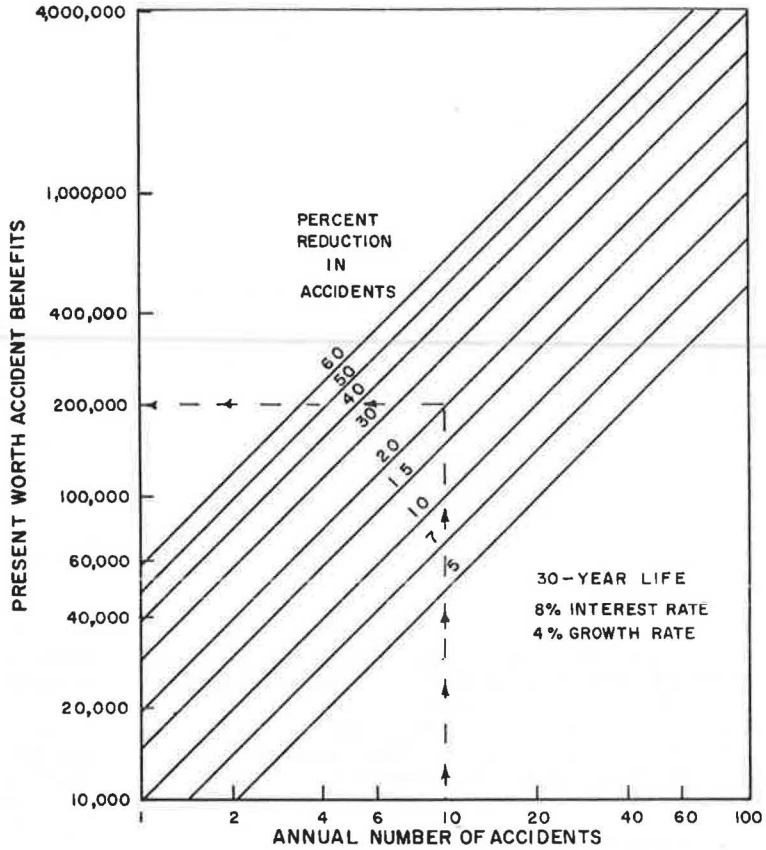


Figure 9. Benefit-cost ratios for widening 2.1-m (7-ft) lanes.

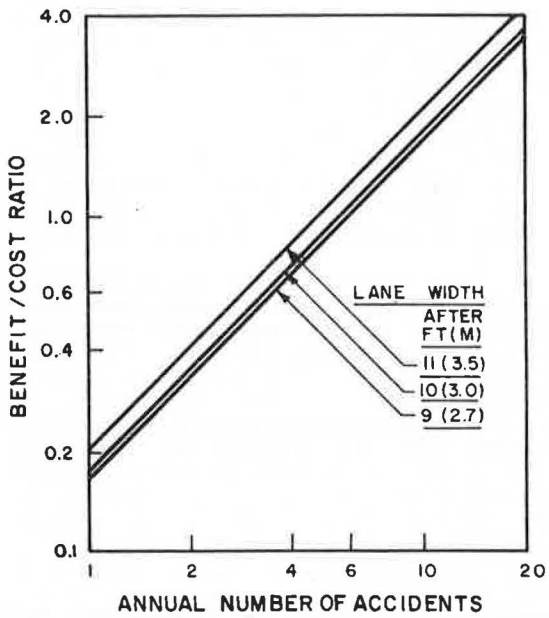
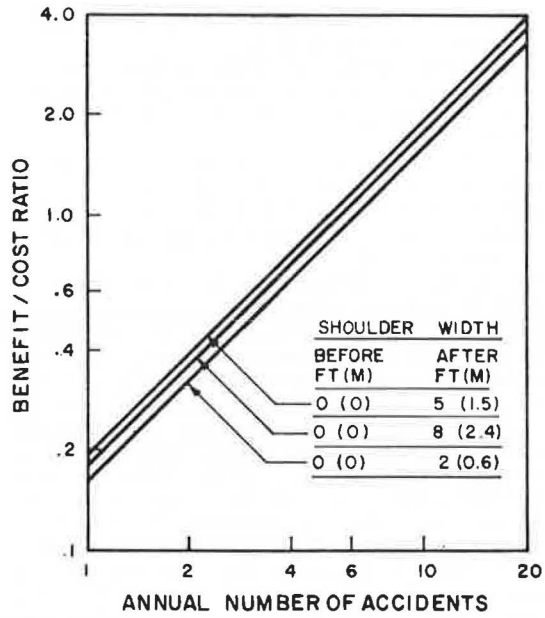


Figure 10. Benefit-cost ratios for adding shoulders.



constitute a standard, specification, or regulation.

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Analysis of Safety Benefits Expected Through Modifications in Drainage Structure Design

JOHN F. NIXON AND DAVID HUSTACE

The problem of providing improved design for roadway culverts is investigated through a twofold analysis: (a) examination of accidents on Texas highways and (b) presentation of a theoretical computation involving societal accident costs and a probability of impacts developed by the American Association of State Highway and Transportation Officials. The study findings indicate that culvert-related accidents are of minor significance in terms of overall accident occurrences. To achieve optimal accident reduction, it is essential that any enhancements to existing culvert designs be carefully considered. Recent research findings offer some insight into designs to mitigate the hazard potential of cross-drain and driveway culvert installations. The acceptance of any remedial measure must compare the cost associated with the recommended design with the cost involved in its required maintenance and the effect on performance due to terrain encountered in typical field installations. A list of proposed modifications to culvert design is provided to assist the designer in optimizing safety expenditures.

Safety on highways is a critical concern to the motorist as well as to state transportation departments. The design engineer must assess the abilities of the reasonable and prudent driver and provide a safe roadway environment while providing for the basic transportation needs of the community. In Texas, the discharge of these obligations is frequently obscured by the many facets involved in planning, constructing, and maintaining more than 72 000 miles of roadway on the Texas highway system. Budgetary limitations have caused severe shortages of funds for needed transportation improvements. All programs must therefore be judiciously compared and selected to achieve and maximize the overall betterment and safety of these highways.

Foody and Long (1) reported that in Ohio almost two-thirds of single-vehicle accidents on the rural, two-lane highway system did not involve a collision with a fixed object. In a comparison of injury-producing accidents, it was found that nonfixed objects were responsible for approximately the same percentage of injury-producing accidents as fixed objects. Terrain or the basic roadway design (or lack of it) represented the greatest hazard to a vehicle leaving the road on a rural, two-lane highway system. Furthermore, it was estimated that any fixed-object improvement program would affect less than 10 percent of the accidents. Therefore, it was concluded that, in Ohio, any major improvement program directed at roadside obstacles would not be economically feasible in comparison with a program in which primary emphasis was placed on improvements to the shoulder and/or roadways.

Similarly, a tabulation by the Texas Department of Public Safety (2) of accidents by type in Texas indicates that accidents involving nonfixed objects amounted to 76.1 and 77.1 percent of the fatalities occurring during 1977 and 1978, respectively. The largest single category of fatal accidents in Texas is the multiple-vehicle collisión. In 1977 and 1978, these accidents represented 38.6 and 38 percent, respectively, of all accidents. Although high speeds are frequently associated with this type of accident, since the opportunity exists for both drivers to exercise evasive action, pavement condition is an important factor that directly affects accident frequency and severity. Similarly, sin-

gle-vehicle accidents that occur both on and off the roadway frequently also initially involve similar pavement conditions to which the driver has not successfully accommodated. Therefore, the importance of focusing primary effort on safe design and maintenance of the roadway is apparent.

Fixed objects off the roadway are another area of concern to the safety engineer. Optimization of design requires an intensive examination of each fixed object with respect to the overall safety of the facility. Basically, in order to select an optimum design, two questions should be answered in considering the treatment of fixed objects off the roadway:

1. Does a problem exist?
2. If a problem exists, what is the optimal method of treating the problem?

INVESTIGATION

Driveway culverts and crossroad culverts represent two types of the many fixed objects adjacent to the roadway. This paper attempts to examine this class of fixed objects to develop a procedure for a systematic evaluation of benefits to be derived from any proposed program for enhancement of these structures.

Four primary factors are involved in considering the relative hazard potential of any fixed object off the roadway: (a) distance of the object from the roadway, (b) frequency of occurrence of the object along the roadway, (c) obstacle size, and (d) traffic volume.

The distance of an obstacle from the roadway will affect the hazard potential of an impact in two ways: Not only is the probability of an impact greatly reduced by distance from the roadway, but also a markedly reduced severity of impact should be possible through driver corrective action in steering or vehicle braking and deceleration.

The frequency of occurrence of a hazard directly influences the probability of impact with that hazard, and probabilistic models have been developed to estimate this occurrence. When impacts with intermittently occurring obstacles are compared with those that occur continuously, however--such as guardfence, pavement edge drop-offs, or poor driving-surface friction resistance--the comparative exposure frequency is slight.

Obstacle size influences both potential impact frequency and impact severity. Fewer and less severe impacts are expected with objects that offer a small, low "target" value.

Finally, traffic volume must be considered in comparing roadway appurtenances that need improvement. Higher-volume facilities, with their proportional increase in frequency of exposure to hazards, should receive priority in improvement scheduling over the comparable low-volume facility.

In this research, two approaches were used to investigate the problem: (a) Historical accident data were analyzed to determine the dimensions of the problem, and (b) a probabilistic model was used to estimate the frequency of impact in order to compute a benefit/cost (B/C) comparison for treated and untreated installations.

ACCIDENT STATISTICS

Statistics comparing culvert accidents with total accidents on Texas highways are given in Table 1 (3). A comparison with culvert accidents of all types indicates that culvert accidents represent only 0.7 percent of the total roadway accidents occurring on state-maintained roadways in Texas and only 1.5 percent of the fatalities, 1.4 percent of the injuries, and 0.4 percent of the property damage. Although any computed percentage will vary from year to year, it is apparent that culvert accidents represent a very low-frequency type of incident. To give an indication of the frequency at which vehicles departing the road collide with culverts, the Federal Highway Administration (FHWA) reports that impacts with culverts represent 3.1 percent of the most common impacts with roadside objects (4).

To evaluate the relative significance of the fixed-object collision, Table 2 gives a ranking of fixed objects based on societal costs. Since rates of fatalities, injuries, and property damage for collisions with a given fixture will vary, by weighting these rates with an estimate of the cost to society associated with each type of accident, a comparison between the severities of each type of fixed-object accident can be made. Societal costs reported by FHWA (5) were $287 175/fatality, $3185/

Table 1. Comparison between culvert accidents and total accidents on Texas highways in 1978.

Accident Category	Culvert Accidents	Total Accidents	Culvert Accidents as Percentage of All Accidents
Fatality			1.5
Accidents	37	2 538	
Fatalities	44	2 987	
Injury			1.4
Accidents	862	59 609	
Number injured	1217	94 545	
Property damage	570	140 135	0.4
All	1469	202 282	0.7

Table 2. Comparison of societal costs for 10 types of single-vehicle, fixed-object accidents on Texas highways in 1977.

Type of Fixed Object	Fatality		Injury		Property Damage		Total Societal Costs ($000s)	Cost as Percentage of All Types of Accidents
	No.	Cost ($000s)	No.	Cost ($000s)	No.	Cost ($000s)		
Guardpost rail	102	29 292	3 339	10 635	6 490	3 375	43 302	3.8
Tree or shrub	95	27 282	1 067	3 398	1 559	811	31 491	2.7
Culvert headwall	62	17 805	1 018	3 242	1 339	696	21 743	1.9
End of bridge	72	20 677	194	618	371	193	21 488	1.9
Fence	43	12 349	1 278	4 070	2 757	1 434	17 853	1.5
Highway sign	42	12 061	1 099	3 500	3 560	1 851	17 412	1.5
Side of bridge	29	8 328	1 155	3 679	2 051	1 067	13 074	1.1
Utility pole	20	5 744	1 482	4 720	2 300	1 196	11 660	1.0
Pier or support	36	10 338	230	733	351	183	11 254	1.0
Luminaire pole	13	3 733	778	2 478	1 434	746	6 957	0.6
All accidents statewide	2 671	767 044	84 386	268 769	227 855	118 485	1 154 298	

injury, and $520 for property damage. When societal costs for the 10 most frequently struck fixed objects are compared with costs for single-vehicle accidents and total accidents, all culvert accidents represent only 1.9 percent of the cost of total accidents. Although these percentages cannot be used to infer the degree of hazard in relation to other fixed objects unless total numbers of fixed objects are known, they do reflect the low significance of total culvert accidents in relation to all accidents. In addition, since a culvert accident can be interpreted to include accidents involving large drainage structures (structures up to 20 ft long in Texas are classified as culverts) as well as small-diameter pipe culverts, these percentages are apt to be considerably overstated when only the typical smaller driveway and cross-drain culverts are considered.

PROBABILITY MODEL

A probability model was next used to establish a measure of the hazard potential of the individual culvert and to further complement the historical data. The American Association of State Highway and Transportation Officials (AASHTO) Guide for Selecting, Locating, and Designing Traffic Barriers (6) provides a procedure for estimating accidental departures from the roadway and expected impacts with a given fixed object, as well as a method for analysis with an improved design to achieve a B/C comparison.

Basically, the procedure used first estimates the expected frequency of accidental departure of a vehicle from the roadway based on the average daily traffic of the facility. Next, by using a nomograph (Figures 1 and 2) for a fixed object of known width, length, and distance from the roadway and an estimated departure frequency, the frequency of collision (C_F) with cross-drain and driveway culverts, respectively, is determined for various roadway densities.

A comparison of the benefits of an "improved" design is then made with the existing design to enable a B/C determination. By means of estimates of the expected severities associated with an impact with existing and improved designs and a computation of the societal cost of the accident, a dollar value can be assigned to each installation. The severity index used was based on a scale of 0-10. Figure 3 shows a graph that was revised from the AASHTO report to reflect the previously cited societal costs for fatalities, injuries, and property damage. Accident costs according to the severity-index scale can be summarized as follows:

Severity Index	Accident Cost ($)
0	520
1	920
2	1 320
3	1 719
4	4 959
5	16 585
6	36 731
7	88 116
8	173 579
9	227 537
10	272 975

Figure 1. Frequency of vehicle collisions with cross-drain culverts.

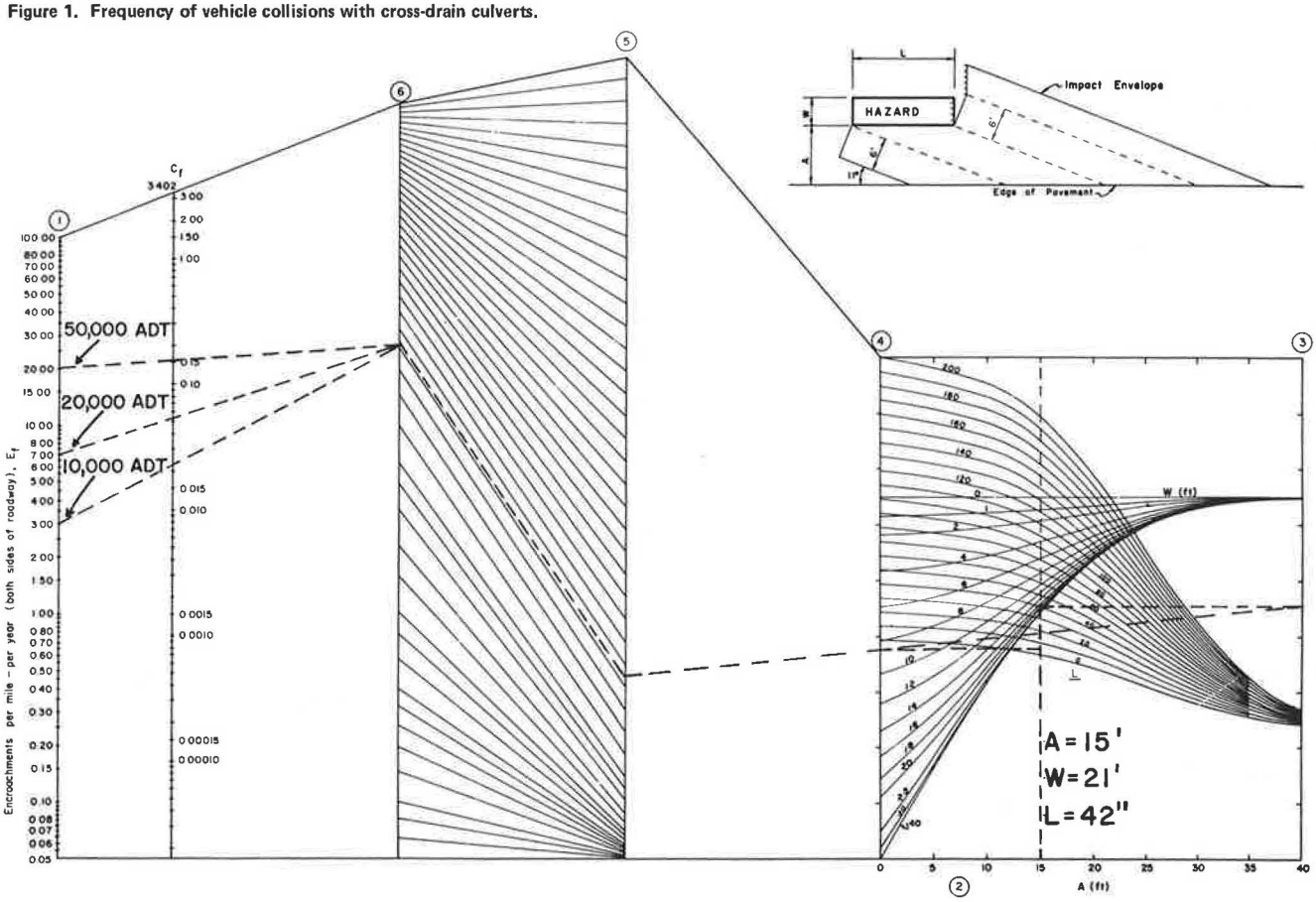


Figure 2. Frequency of vehicle collisions with driveway culverts.

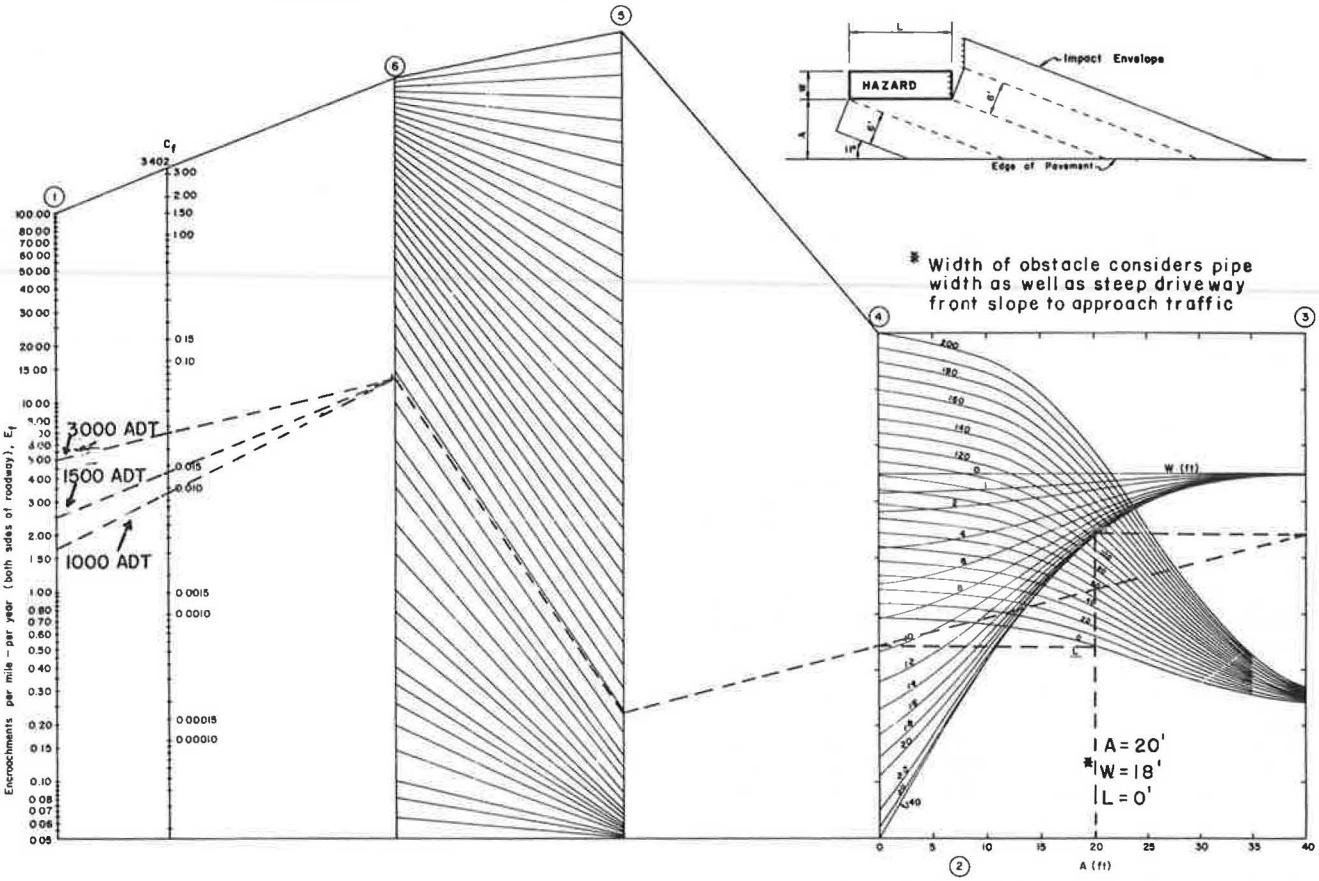


Figure 3. Dollar value of an accident.

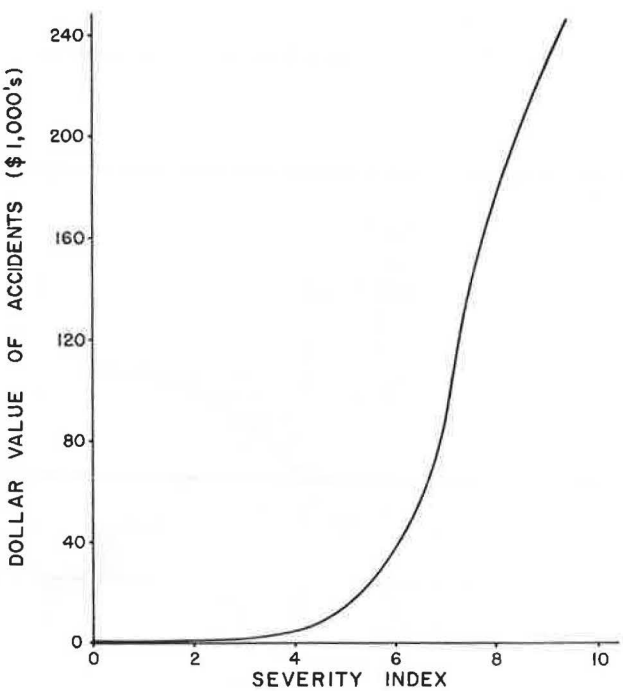


Figure 4. B/C value for cross-drain culverts versus ADT.

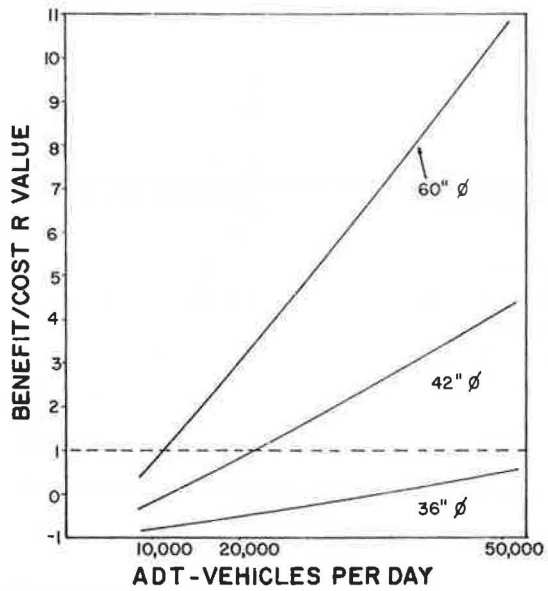
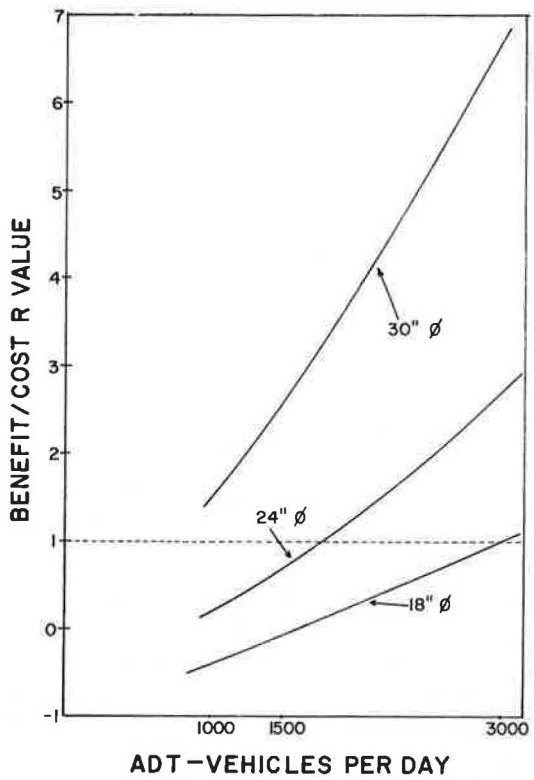


Figure 5. B/C value for driveway culverts versus ADT.



The following equations were used to compute the R, or B/C value, of the existing installation:

$$C_{TU} = C_I + C_D (C_F)(K_T) + C_M (K_T) + C_{OVD} (C_F)(K_T) - C_S (K_F) \quad (1)$$

$$C_{TP} = C_I + C_D (C_F)(K_T) + C_M (K_T) + C_{OVD} (C_F)(K_T) - C_S (K_F) \quad (2)$$

$$C_{TD} = C_I + C_D (C_F)(K_T) + C_M (K_T) - C_S (K_F) \quad (3)$$

$$R = (C_{TU} - C_{TP})/C_{TD} \quad (4)$$

where

C_{TU} = cost of unprotected fixed object,
C_{TP} = cost of protected fixed object,
C_{TD} = cost of installation of protected fixed object,
R = B/C ratio,
C_I = initial cost of fixed object,
C_D = cost of damage through use,
C_F = collision frequency,
C_M = cost of maintenance through use,
C_{OVD} = cost of accidents,
C_S = salvage value (cost), and
K_T, K_F = interest rates based on 20-year life at 8 percent.

FINDINGS

Cross-Drain Culverts

Figure 4 shows B/C values calculated for culvert sizes as large as 60 in in diameter for average daily traffic (ADT) through 50 000 vehicles/day. An interpretation of the B/C ratio is that a negative value and values less than 1 imply that no benefit is expected for the proposed enhancement. It is only when a B/C ratio of 1 is attained or exceeded that a net benefit is realized. For this analysis, unprotected culvert installations assumed cross-drain culverts without headwalls that are sloped to coincide with the side slope of the roadway. The protected culvert design assumed the same culvert with the addition of a bar grate.

B/C ratios for cross-drain culverts indicated that, for culvert sizes through 42 in in diameter on roads with traffic densities of <20 000 vehicles/day, no benefit is expected for a culvert protected by grates.

Driveway Culverts

Figure 5 shows B/C values calculated for 18-, 24-, and 30-in-diameter driveway culverts for roadways with ADT between 1000 and 3000 vehicles/day. Unprotected culverts were considered to be circular corrugated metal pipes that do not have headwalls and have no special end treatments. Protected culverts were assumed to have a pipe with a 6:1 tapered end that has been stabilized with concrete riprap. Neither corrugated arch pipe nor concrete pipe was considered in this analysis because of their higher cost of modification compared with the circular pipe.

B/C ratios for the designs considered indicate that, for an 18-in-diameter pipe culvert, a modified design is not cost effective for traffic densities up to 3000 vehicles/day. Similarly, a modified 24-in culvert is cost effective for traffic densities greater than 1800 vehicles/day. A 30-in culvert shows a B/C ratio of greater than 1 for all traffic densities.

Table 3 illustrates a table of safety benefit countermeasures as an example of some B/C ratios for roadway and roadside improvements on actual construction projects (5). It is noted with interest that many of the highest B/C ratio items are concerned with treatment of the driving surface or adjacent areas. It should also be emphasized that, when improvements are being considered, a B/C ratio is one of the several tools available to the designer to establish a priority of needed improvements to a facility. Other crucial elements that must also be considered in any proposed improvement program are (a) available funds to achieve the needed program without curtailing essential roadway improvements elsewhere and (b) the effect that such improvements will produce with respect to increased maintenance of a facility.

CURRENT RESEARCH

Research into the reduction of the relative degree of hazard of typical highway culvert designs has been initiated by the Texas State Department of Highways and Public Transportation (TSDHPT). A cooperative research study (7) was directed at producing guidance on some of the fundamental questions concerning cross-drain and driveway culvert installations. A full-scale vehicle test program was conducted to investigate the maximum spacing of bars on culvert grates to provide for a safe vehicle traversal with a minimum hydraulic disruption to the system. Also tested was a comparison of high-speed impacts with driveway-culvert slopes. The culmination of this research activity has provided the engineer with a comparative measure of performance constraints that may be used as a guide in the design of future culvert installations.

SUMMARY AND CONCLUSIONS

In any consideration of roadside design safety, primary emphasis must be placed on the driving surface itself. Roadways should be constructed so

Table 3. Benefits of roadway and roadside safety countermeasures.

Rank	Code	Improvement	Service Life (years)	No. of Projects	Avg Annual Accident Experience: Before: Accidents	Before: Injuries	Before: Fatalities	After: Accidents	After: Injuries	After: Fatalities	Annual Accident Reduction Expected (%): Accidents	Injuries	Fatalities	Benefit ($)	Cost per Project ($)	B/C Ratio
1	23	Shoulder widening or improvement	20	46	1917	585	35	1353	465	21	29	20	41	4 754 088	35 200	28.83
2	64	Installation of striping and/or delineators	4	2000	5849	3351	113	5060	2599	60	13	20	46	17 493 150	1 094	26.49
3	25	Skid treatment (grooving)	20	96	1117	395	27	580	275	7	48	30	74	6 372 399	32 385	20.12
4	60	Installation or upgrading of traffic signs	4	775	3727	1538	80	2879	1030	23	23	33	27	8 031 275	2 278	15.03
5	57	Signing and/or marking	10	3046	191	142	34	192	82	22	0	42	35	3 638 462	536	14.94
6	63	Installation or improvement of median barrier	10	23	962	479	48	994	449	4	-3	6	91	12 712 361	270 070	13.73
7	65	Installation of roadway lighting	10	115	1119	546	20	1022	499	6	9	9	73	4 393 559	19 363	13.24
8	62	Installation or improvement of road-edge guardrail	10	1651	2077	844	69	1716	719	29	13	15	59	12 273 743	4 546	10.97
9	50	Replacement of signs only with flashing lights (railroad crossing)	10	56	36	17	7	*	1	0.06	94	93	99	2 014 682	25 655	9.41
10	60,64	Sign-striping combination	4	465	5858	2844	90	4464	2108	65	24	26	27	9 982 024	8 270	8.60
11	61	Breakaway signs or lighting supports	4	527	195	41	2	127	23	0	35	44	100	656 538	569	7.25
12	11	Installation or improvement of traffic signals	10	699	9408	4181	87	7698	2840	43	18	32	49	17 688 205	26 650	6.36
13	26	Skid treatment (overlay)	20	126	3071	1627	37	2552	1194	26	17	27	30	4 747 692	60 796	6.09
14	55	Replacement of signs (only) with automatic gates	10	101	43	17	11	0.2	0.03	0.2	99	99	100	3 100 704	37 872	5.44
15	10	Channelization including left-turn bays	10	612	5815	2618	83	4481	1860	30	23	29	65	17 982 781	50 091	3.94
16	20	Pavement widening, no lanes added	20	241	951	489	26	715	301	4	25	33	87	7 238 042	80 188	3.68
17	13	Improvement of sight distance	10	142	338	205	6	234	127	4	31	38	36	859 826	13 696	2.97
18	12	Combination of codes 10 and 11	10	36	887	333	1	609	215	0.5	31	35	50	620 449	64 846	1.78
19	56	Replacement of active devices with automatic gates	10	166	28	11	3	5	3	0.1	81	75	96	948 528	33 921	1.13
20	42	Combination of codes 40 and 41[a]	20	69	423	219	6	332	150	2	21	32	69	1 350 900	211 055	0.91
21	31	Replacement of bridge or other major structures	30	163	113	84	5	63	33	0	44	60	47	1 548 658	118 475	0.90
22	21	Addition of lanes without new median	20	96	1482	595	7	1224	531	5	17	11	31	900 544	114 987	0.80
23	30	Widening of existing bridge or other major structures	20	354	565	291	3	198	76	2	65	74	33	1 103 632	75 440	0.41

[a]Specified in the complete table contained in the FHWA report (4).

that motorists can achieve their transportation needs in safety and with confidence. Off-roadway fixtures must assume a decidedly subordinate importance to those on-roadway features that confront every motorist. The greater the distance the fixed object is from the traveled way, the less frequent its occurrence, and the smaller its size, the less of a hazard that object becomes.

This investigation into the enhancement of culvert designs has first considered historical accident data on culvert accidents of all types. In a survey of six states and more than 8000 accidents, accidents involving culverts are reported to represent 3.1 percent of the most commonly occurring roadside fixed-object accidents. In Texas, culvert accidents of all types for 1978 represented 0.7 percent of total roadway accidents, 1.5 percent of fatalities, 1.4 percent of injuries, and 0.4 percent of property damage. Historical accident data, therefore, have indicated that culvert-related accidents of all types occur at a low frequency.

Next, a societal cost comparison that weights each accident by the cost involved with fatalities, injuries, and property damage has indicated that for 1977 culvert accidents represent 1.9 percent of the cost of all accidents occurring on Texas highways. This theoretical computation also indicates that culvert accidents of all types are a low-cost item compared with all types of accidents.

Finally, B/C comparisons were computed for protected and unprotected cross-drain and driveway culvert installations to generate some measure that could assist the designer in an evaluation of alternative safety expenditures. Because of the shortage of funds available for needed construction and maintenance, an arbitrary program to slope the ends of driveway culverts and to require grates on many cross-drain culverts does not appear to be in the best interest of the traveling public. Instead, the following recommendations are offered to optimize the overall safety of the highway system:

1. The involvement of cross-drain culverts in fixed-object accidents represents a low-frequency occurrence where full shoulders and flat side slopes are present. Culvert ends do not require special safety treatments for the following sizes and traffic densities based on preliminary B/C computations:

Pipe Diameter (in)	Traffic Density (vehicles/day)
36	50 000
42	20 000
60	10 000

2. For driveway culverts, there is a higher potential for injury to vehicle occupants if the culvert end is untreated than if the design provides a 6:1 sloped end. However, preliminary B/C computation involving impact frequency indicates that treatment of driveway culvert ends need not be provided where full shoulders are present for the following pipe sizes and traffic densities:

Pipe Diameter (in)	Traffic Density (vehicles/day)
18	3000
24	1600
≥30	Treatment warranted if inside clear zone

3. A high priority for safety expenditures should be assigned to on-roadway factors that directly involve the safety of all motorists. A list compiled by FHWA indicates that the top five safety B/C

countermeasures to be applied to the highway system are (a) shoulder widening or improvement, (b) installation of striping and/or delineators, (c) skid treatment and grooving, (d) installation or upgrading of traffic signs, and (e) signing and/or marking.

4. Additional improvements to culvert installations that should be considered are (a) locating driveway and cross-drain culverts as far from the travel way as possible, (b) minimizing cover of driveway culverts to reduce overall height of the obstacle, (c) using ditchline driveways without pipes wherever possible, and (d) deleting concrete headwalls and shaping fill adjacent to pipe ends on driveway culverts to minimize the opportunity for abrupt vehicle decelerations on possible impacts.

5. Since the cost and hazard potential of culverts dictate the use of the minimum pipe sizes able to accommodate expected hydraulic flow conditions, the use of grates to improve chance impact performance should be considered with extreme caution. The reduction in culvert capacity caused by the addition of a grate necessitates larger culvert sizes (and fill heights). This can greatly increase the hazard potential of the installation as the height of the force that results more nearly approaches the vehicle center of mass.

The B/C computations presented in this paper have only considered routine maintenance costs. They have not addressed the potential for extensive damage to the facility and adjacent property or the hazard to the motorist when water overflows onto the highway because of a blockage caused by a reduction in hydraulic capacity during high-intensity rainfalls. The costs associated with these problems are very real and need to be carefully assessed by highway designers whenever a culvert upgrading program is initiated and alternative safety expenditures are considered.

ACKNOWLEDGMENT

The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of FHWA or TSDHPT. This paper does not constitute a standard, specification, or regulation.

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