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Foreword

The seven papers in this Record are sponsored by the Committees on Geometric Design, on Operational Effects of Geometrics, and on Photogrammetry and Aerial Surveys. They are related by their coverage of geometric design or the operational and safety effects of design issues.

The Record begins with a paper by Fwa, who describes an optimization program that he developed to produce vertical highway profiles from preselected horizontal alignments. The program can model most of the critical cost items for highway construction and the effects of different geometric design alternatives on project costs.

In the next paper, Chang discusses the feasibility of using an expert system with LISP programming and AutoCAD(TM) for the typical intersection design process. An expert system interfaced with the normal drawing functions of a CADD package can assist engineers in the decision making and design processes, taking into consideration the operational effectiveness and trade-offs among a number of design factors.

Two papers examine the safety effects of left-turn lanes or medians. McCoy and Malone evaluate accident experience at signalized and unsignalized intersections on urban four-lane roadways to assess the safety effects of left-turn lanes. Their analysis indicates that the left-turn lanes reduced rear-end, sideswipe, and left-turn accidents significantly at both signalized or unsignalized intersections. At uncontrolled intersections, however, right-angle accidents were significantly increased. Squires and Parsonson provide an accident rate comparison of raised medians and continuous two-way left-turn lanes used as medians on four- and six-lane roads and develop regression equations to model the expected accident experience for each.

Rymer and Urbanik describe the use of the TRANSYT-7F model to develop a methodology, based on an evaluation of the reduced delay benefits to the cost of a grade separation, that can assist traffic engineers and planners in choosing proposed grade separation improvements.

Batz evaluates the effectiveness of a new painted gore design for the beginning of passing zones. His results indicate that the gore design actually decreased passing efficiency in the short length passing zone (less than 0.5 miles) but also produced a slight increase in efficiency for the longer zone (greater than 0.5 miles).

In the final paper, Berg et al. describe the development of computer software that can transform digitized photolog data into a highway alignment data base. This data base can then be used for inventory studies, deficiency analysis, and preliminary design studies. The program can also be used to determine maximum speeds and provide estimates of stopping or passing sight distance restrictions due to horizontal or vertical alignment.

Highway Vertical Alignment Analysis by Dynamic Programming

T. F. FWA

In this paper, an optimization program developed to produce an optimum vertical highway profile for a preselected horizontal alignment is described. The aim of the program was to provide local highway engineers and planners in Singapore with a practical aid for highway geometric design and location analysis. A dynamic programming formulation was adopted to minimize the overall cost, which includes earthwork, land acquisition, materials, and vehicle operating costs. The program's versatility allows it to model most of the cost items important in highway construction and can also be used to provide quick information on the effects of different geometric design parameters on project costs and to determine the relative importance of different cost categories. A real-life numerical example is presented to illustrate program application.

In the process of constructing new highways and relocating existing highways, a highway location study is usually performed to position the proposed highway within a corridor according to engineering, social, economic, and environmental considerations. Such an analysis typically involves a number of conflicting construction and operational requirements. A highway designer is required to achieve an economical design that is adequate operationally.

Vertical alignment design, a subproblem of the broad three-dimensional highway location problem, is of great practical significance because its solution has direct bearing on highway construction and vehicle operating costs. Vertical alignment design has attracted much research interest in the last two decades, particularly in the area of highway vertical alignment optimization. Comparisons with conventional manual designs have shown that an average savings of 15 percent of construction costs could be achieved by using computer-derived optimum alignments (*1*,*2*).

The optimization techniques that have been used for vertical road alignment studies include linear programming (*2*), quadratic programming (*3*), dynamic programming (*4*), and relaxation methods (*5*). Methods of search—direct search, random search, and gradient search (*6*,*7*)—have also been used.

In this paper, an optimization program that was developed to produce a preliminary vertical highway profile for a preselected horizontal alignment is described. The aim was to provide local highway engineers and planners in Singapore with a practical aid for highway geometric design and location analysis. A dynamic programming formulation was adopted to minimize the overall cost, which included earthwork, land acquisition, materials, and vehicle operating costs. Constraints were carefully selected and modeled to suit local conditions and design and construction practices. Parametric and sensitivity studies were conducted, based on a real-life example, to illustrate the salient features of the program.

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MODELING OF ALIGNMENT PROBLEM

Three basic elements can be identified in modeling the problem of vertical alignment optimization of a highway: (a) representation of input ground profile and output road alignment, (b) formulation of objective function, and (c) definition of construction constraints and operational requirements. Detailed descriptions of these aspects of the optimization program are included below.

Ground Profile and Road Alignment

The natural ground profile is established along the central axis of the proposed highway, the horizontal alignment of which has been predetermined and fixed. This profile is input as a series of straight-line segments, each spanning an equal horizontal distance measured along the central axis of the highway. These equal horizontal intervals, which are determined by the grid size selected for a particular problem, also determine the line segment intervals over which the output road alignment will be represented. Smoothing of the piecewise-linear alignment by a design engineer is required to give a practical curved profile.

The dynamic programming optimization process uses a set of vertical data grid lines spaced at the horizontal grid intervals mentioned above, as shown in Figure 1. Both the piecewise-linear input ground profile and output road alignment pass through one grid point each on each vertical grid line. The optimization algorithm determines one point on each grid line to represent the output alignment that satisfies specified constraints and yields the lowest overall cost.

Objective Function

The objective function is the overall cost that represents the sum of the following four cost components: (a) earthwork cost, (b) pavement cost, (c) land acquisition cost, and (d) vehicle operating cost.

Earthwork Cost

Earthwork cost may be divided into two major components: (a) cost of cutting when the computed road alignment level

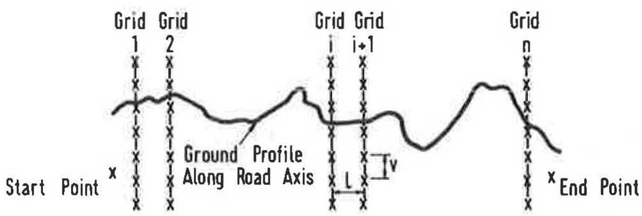

FIGURE 1 Data grid for vertical alignment optimization.

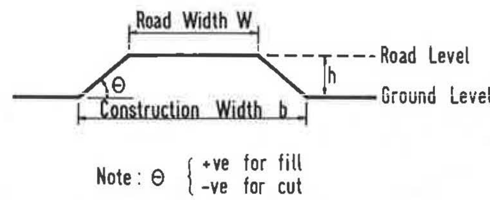

FIGURE 2 Simplified cross section for earth volume computation.

is lower than the existing ground level and (b) cost of filling or embankment construction when the required road alignment lies above ground level. These two cost components can be further subdivided into excavation or backfilling cost, transport cost, and roadbed preparation cost according to the size of project.

Excavation and backfilling costs are basically functions of the volume of earth involved, although the unit rate of excavation may vary with the depth of cut and the type of soil. Transport costs can be computed once the hauling distance is known. Locations of dumping sites and borrow pits are required as input to the optimization program. Roadbed preparation costs vary with the type of soil upon which the proposed road is to be constructed.

For the purpose of cut- or fill-earth volume calculation, the cross sections of finished roadbed were simplified (see Figure 2). The cross-sectional area of cut or fill at any grid point is completely defined by three variables: (a) roadbed width, w; (b) cut or embankment slope represented, θ; and (c) the difference between computed road alignment level and ground level, h. End area method was used to compute the volume of cut or fill required between any two grid lines.

Pavement Cost

Pavement cost, computed from pavement thickness and unit costs of pavement materials used, is basically a function of the quality of roadbed soil. In most highway projects, soil type and strength are found to vary along the length of the route. Pavement thickness requirement is also likely to vary between fill and cut sections, as well as among different depths of cut at a given point.

Because the type and strength of soil is a function of grid line location, as well as the relative vertical position of roadbed with respect to ground surface, it is necessary to compute unit pavement cost for each grid point. The pavement cost for each road segment between two vertical grid lines was computed by multiplying the length of the road segment by the arithmetic mean of the unit pavement costs at the two grid points which defined the road segment.

Land Acquisition Cost

Land acquisition cost was defined as the product of unit area land cost multiplied by the area of land requirement for highway right-of-way. The unit cost of land may vary along the length of the proposed route. Depending upon the type of construction (cut or fill) and the nature of land development on the two sides of road, right-of-way width may also vary along the length of the route.

When the unit land cost and width of right-of-way requirement are known, the land acquisition cost at each grid point can be computed. Figure 2 shows that the right-of-way width at a grid point depends on the construction width, b, at the ground level. The magnitude of width b is, in turn, related to the difference between road alignment and ground level. To obtain the land acquisition cost for any line segment, the arithmetic mean of land acquisition costs at the two corresponding grid points was computed, and the mean value was multiplied by the road segment length to calculate the desired land acquisition cost.

Vehicle Operating Cost

Two elements that influence the cost incurred by road users were considered—length and gradients. These factors can be conveniently combined into a single input variable expressed in terms of total user cost/percent gradient/kilometer (or mile) of road length. This cost was computed based on a period equal to the design service life in the highway. The total highway traffic volume expected over this period must be estimated for computing the unit user cost.

Constraints

The following constraints were included in the optimization program: (a) a maximum gradient, (b) curvature requirements, and (c) control points with fixed levels. The numerical control values for each constraint were based on the design standards and requirements provided by the Singapore Public Works Department (*8*).

Gradient control is a traffic operational requirement to ensure smooth vehicle movement. The maximum allowable gradient is a function of design highway speed, as well as the types of vehicles included in the design traffic stream. The constraint on gradient can be dealt with in the following manner:

$$|Y_i - Y_{i-1}| \leq G\,d \qquad i = 1, 2, \cdots, N \tag{1}$$

where

Y_i = road alignment level at grid line i;
G = maximum allowable gradient;
d = horizontal distance between two successive grid lines; and
N = total number of grid intervals.

Curvature requirements were achieved by controlling the magnitude of algebraic change in gradient between two consecutive line segments. Adopting the recommended practice of Singapore Public Works Department (*8*), the absolute value of gradient change, g, which was derived from considerations of highway safety, aesthetics, and comfort of ride, were specified separately for crest and sag vertical curves as follows:

For crest vertical curves:

$$g \leq 425/(2S - L) \qquad \text{when } L \leq S \tag{2}$$

$$g \leq 425L/S^2 \qquad \text{when } L > S \tag{3}$$

For sag vertical curves:

$$g \leq (122 + 3.5S)/(2S - L) \qquad \text{when } L \leq S \tag{4}$$

$$g \leq (122 + 3.5S)L/S^2 \qquad \text{when } L > S \tag{5}$$

where

g = absolute algebraic difference in gradient (%);
L = length of vertical curve (m); and
S = sight distance (m).

The sight distance, S, is a function of design speed, vehicle type and roadway gradient. The allowable gradient change, g, would therefore also vary with the geometric parameters selected by the designer.

The third constraint, control points with fixed levels, is commonly encountered in actual highway design problems. The levels of the start and end points of a new highway are typically fixed. Intermediate fixed-level control points are needed where a new highway intersects existing roads.

DYNAMIC PROGRAMMING FORMULATION

The dynamic programming formulation for the vertical alignment problem is summarized in this section. The objective was to minimize the total sum of selected costs incurred in the construction of a new highway. The objective function was

$$\text{Min} \sum_{k=0}^{N-1} [C_1(U_k) + C_2(U_k) + C_3(U_k) + C_4(U_k)] \tag{6}$$

Subject to the following constraints:

Slope

$$|U_k| \leq G\,d \qquad k = 0, 1, \cdots, N - 1 \tag{7}$$

Curvature

$$|U_{k+1} - U_k| \leq 2d(g/100) \qquad k = 0, 1, \cdots, N - 1 \tag{8}$$

System equations

$$U_k = Y_{k+1} - Y_k \qquad k = 0, 1, \cdots, N - 1 \tag{9}$$

For boundary conditions, Y_o and Y_N are prescribed.

All the variables and symbols in the constraint equations 7, 8, and 9 have been explained in the preceding section and are graphically represented in Figure 3. $C_1(U_k)$, $C_2(U_k)$, $C_3(U_k)$, and $C_4(U_k)$ represent earthwork, pavement, land acquisition, and vehicle operating costs, respectively, for the line segment k bounded by grid lines k and $k + 1$. Each of the four cost components is a function of the positions of ground level $f(x)$ and road alignment $y(x)$, where x is the horizontal distance measured from the start point along the roadway central axis. The four cost components can be computed once the relative position of road alignment with respect to ground surface is known, as represented by $h_k = y(x_k) - f(x_k)$ in Figure 3.

APPLICATION

The work reported in this paper was a response to a need to provide local highway engineers and planners in Singapore with a practical aid for highway location analysis and preliminary geometric design. The program can be used to provide quick solutions concerning the impacts of different geometric design parameters on project costs and to determine the relative importance of different cost categories. A real-life numerical example is presented here to illustrate program application.

Numerical Example

The optimization program was used to conduct a preliminary construction cost analysis for a proposed four-lane highway connecting an industrial town to a residential area with a population of 500,000. The horizontal alignment of the proposed highway had been fixed. The highway would shorten the one-way traveling distance between the two locations from 11.5 km to 5.9 km. Figure 4 shows the ground surface profile along the central axis of the proposed route. The total horizontal length was 5875 m (19,275 ft).

The cost data used in the optimization analysis are summarized in Table 1. The proposed road was located within government land reserved for highway construction. Because sufficient right-of-way width had been reserved, land acquisition constraints of cost and width became irrelevant and were not considered in this example. The following discussion presents the results of vertical alignment analysis in the fol-

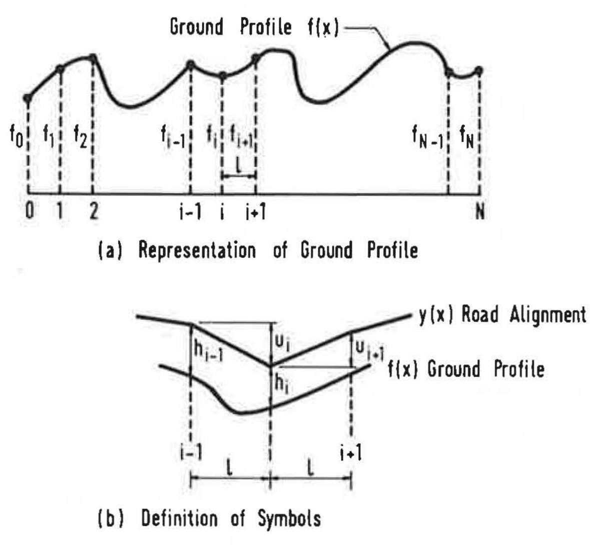

FIGURE 3 Dynamic programming model.

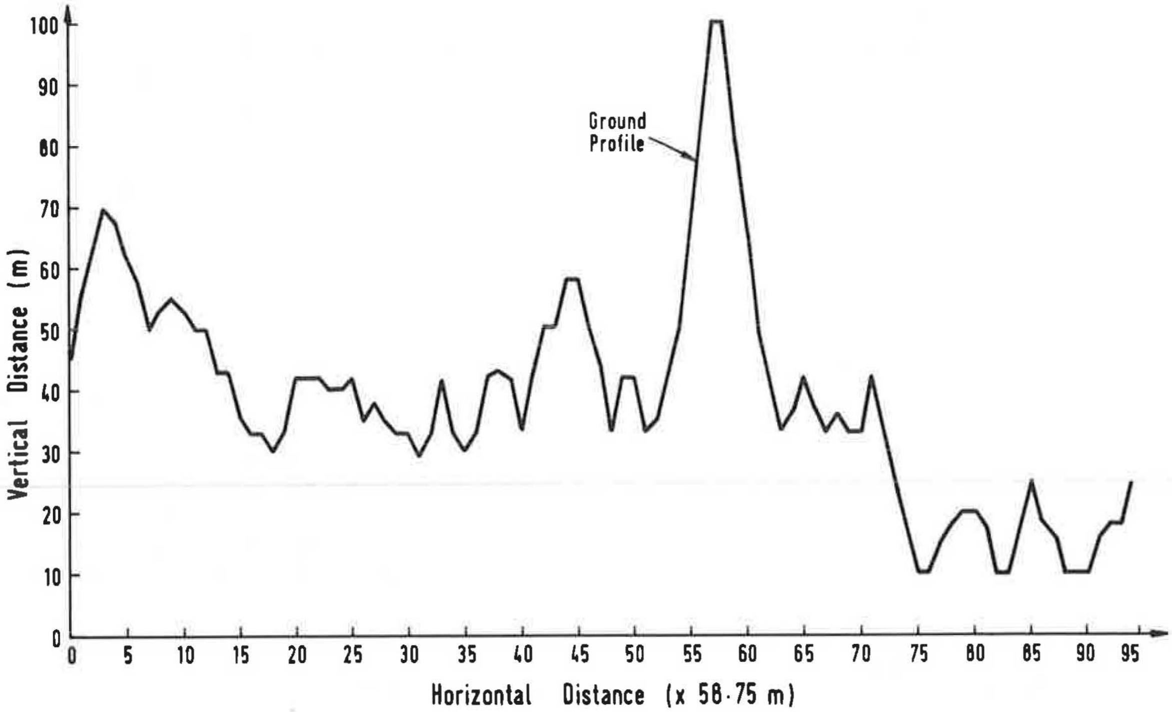

FIGURE 4 Ground profile input for example problem.

TABLE 1 COST DATA AND CONSTRAINT CONDITIONS FOR EXAMPLE PROBLEM

Parameter	Value
Vertical gradient control	4% maximum
Vertical curvature	(See equations 2–5)
Filling cost	S\$10.00/m³
Cutting costs	
Depth < 1.5 m	S\$10.00/m³
1.5–3.0 m	S\$14.40/m³
3.0–4.5 m	S\$18.20/m³
4.5–6.0 m	S\$25.00/m³
6.0–7.5 m	S\$30.00/m³
> 7.5 m	S\$50.00/m³
Pavement cost	S\$80/m²
Vehicle operating costs	Tangent running costs on grades recommended by AASHTO (*9*) for passenger cars on multilane highways.

NOTE: One Singapore dollar (S\$) is approximately 0.5 U.S. dollar.

lowing aspects: (a) sensitivity of results in terms of total cost with respect to horizontal and vertical grid sizes; (b) effect of maximum gradient constraint on optimum total cost; (c) effect of minimum curvature control on optimum total cost; (d) effect of earth filling cost on optimum total cost; (e) effect of earth cutting cost on optimum total cost; and (f) effect of vehicle operating cost on optimum total cost.

Following the local practice of project cost computation, the first five analyses did not consider vehicle operating cost. The sixth analysis was performed to give an indication of the effect of including vehicle operating cost. Because no information was available on the magnitude of vehicle operating costs in Singapore, values for passenger cars recommended in an AASHTO Manual (*9*) were used as examples only. Only tangent running costs on grades were included in the analysis.

Selection of Horizontal and Vertical Grid Intervals

The input ground profile and the final computed road alignment are each represented by a series of line segments passing through one grid point on each vertical grid line. As shown by the data grid in Figure 1, the accuracy of the computation depends to a great extent on the horizontal grid interval, d, as well as the length of each division, v, on the vertical grid lines.

A sensitivity analysis of the computed total cost with respect to the grid variables v and d was performed. The results of this analysis are presented in Tables 2 and 3. Data in Table 2 indicate that with a horizontal grid interval, d, fixed at 62.5 m, sufficiently accurate solution could be obtained with vertical division spacing v equal to 0.5 m or less. When fixing v at 0.25 m (Table 3), acceptable results were obtained for values of d equal to 62.5 m or less.

Analyses conducted on highways in other regions of Singapore also showed similar ranges of acceptability for horizontal and vertical grid intervals. For Singapore terrain it was, therefore, recommended that the horizontal and vertical grid intervals be not more than 60 m and 0.5 m, respectively.

It is of importance to impose another control on the values of horizontal and vertical grid intervals selected. The values of d and v used must be such that their ratio (v/d) is less than the maximum gradient constraint specified. The results in Tables 2 and 3 indicated that a (v/d) ratio of less than 1.0 percent would provide an acceptable solution for a maximum vertical gradient control of 4 percent. Because the allowable

TABLE 2 EFFECT OF VERTICAL GRID DIVISION ON COST ANALYSIS

Vertical Division, v(m)	Horizontal Interval, d(m)	Total Cost (S$)	$(v/d) \times 100\%$	Remarks
1.0	62.5	18,106,219	1.60	No good
0.75	62.5	17,030,170	1.20	Marginal
0.5	62.5	16,244,489	0.80	Satisfactory
0.25	62.5	16,239,328	0.40	Satisfactory

NOTE: Input data and constraint conditions are given in Table 1. One Singapore dollar (S$) is approximately 0.5 U.S. dollar.

TABLE 3 EFFECT OF HORIZONTAL GRID INTERVAL ON COST ANALYSIS

Horizontal Interval d(m)	Vertical Division v(m)	Total Cost (S$)	$(v/d) \times 100\%$	Remarks
250.0	0.25	11,907,072	0.10	No good
125.0	0.25	14,421,626	0.20	No good
62.5	0.25	16,239,328	0.40	Satisfactory
31.25	0.25	16,423,647	0.80	Satisfactory

NOTE: Input data and constraint conditions are given in Table 1. One Singapore dollar (S$) is approximately 0.5 U.S. dollar.

range of maximum vertical gradient is between 4 and 8 percent in Singapore (*8*), it has been suggested that a (v/d) ratio of not more than 1 percent be used.

Effect of Maximum Gradient Constraint

A highway designed with a stricter gradient control—by imposing smaller values of maximum vertical gradient constraint on a hilly terrain—allows for a more comfortable ride and a smoother flow of traffic. Savings on vehicle operating costs could also be achieved by using stricter gradient control, a direct consequence of which is the increase in highway construction cost due primarily to the increased earthworks required. Cost computation of highway construction for different maximum gradient controls could therefore provide useful information for a benefit/cost analysis to aid in the selection of gradient control in highway planning and geometric design.

Figure 5 shows the variation of construction cost (excluding vehicle operating costs) with different values of maximum allowable vertical gradient. As expected, highway construction cost decreased as higher maximum gradient was allowed. It should be noted that the computed road profiles were different for different values of gradient control imposed. Figure 6 shows the profiles for maximum gradient control of 4 percent and 6 percent. The 6-percent gradient profile conformed more closely to the ground surface, resulting in a saving on the amount of earthwork cutting and filling.

Although considerable changes in construction costs were observed when vertical gradient control was changed in this example, it is clear that the sensitivity of computed construction costs against gradient control is a function of ground terrain. The construction cost of a highway to be built on a flat terrain would not vary much with changes in vertical gradient control.

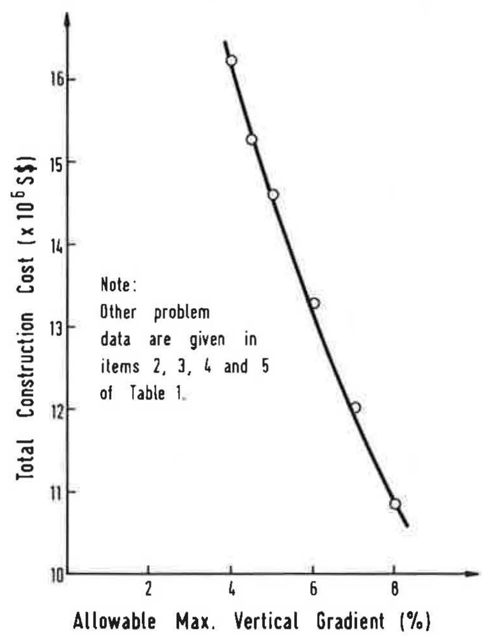

FIGURE 5 Effect of vertical gradient control on highway construction cost.

Effect of Minimum Curvature Control

Curvature control was achieved by setting an upper limit on the value of allowable gradient change, g, which is defined in equations 2 through 5. Small g values allow gradual changes of vertical road alignment, leading to design of highways with smooth riding profile. Construction costs, however, tend to be higher with smaller g values because of the higher quantity of earthwork needed to satisfy the constraint.

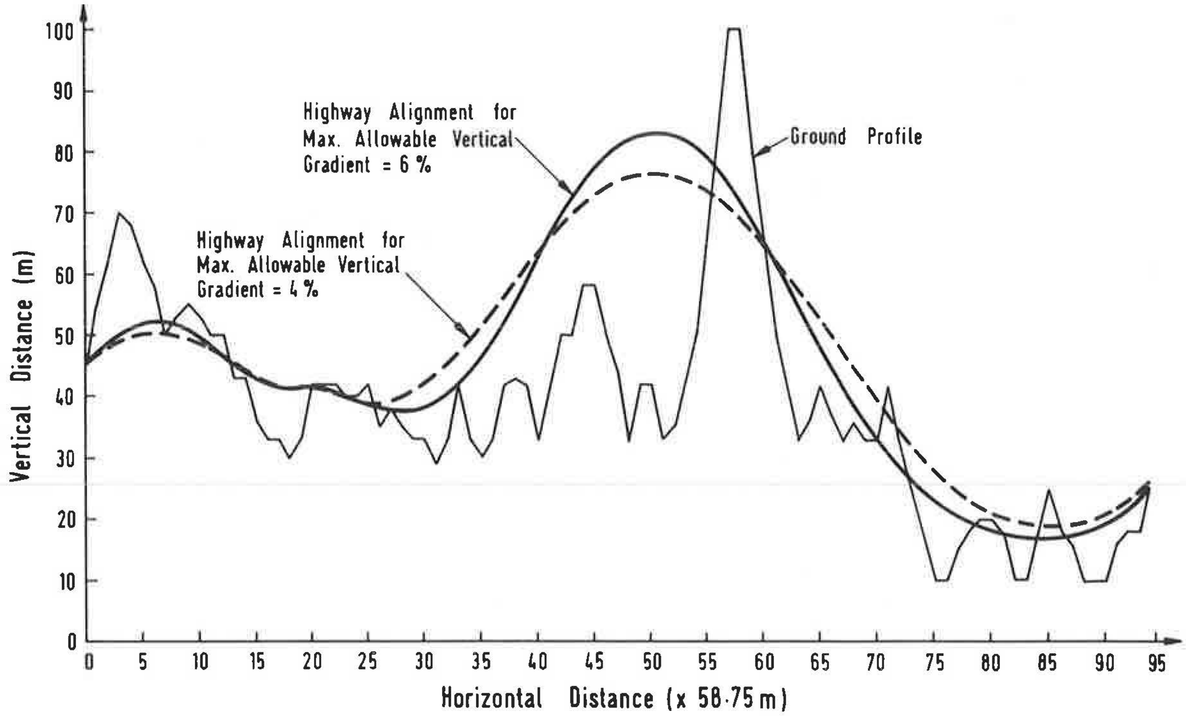

FIGURE 6 Effect of maximum vertical gradient control on highway alignment in example problem.

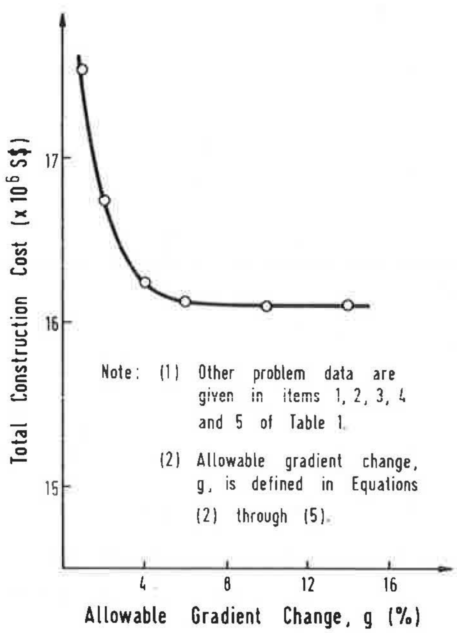

FIGURE 7 Effect of curvature control on highway construction cost.

The relationship between construction cost and curvature control was studied using the example problem, and the results were plotted in Figure 7 for the range of *g* values commonly used in highway design. The changes in construction costs caused by varying the value of *g* were relatively small compared with the changes caused by varying vertical gradient control. This finding has also been found true for highways constructed in other regions of Singapore.

Figure 8 shows the effect of curvature control on the computed optimum road alignment. The alignment computed with a higher value of minimum curvature value, *g*, conformed better to the natural ground profile.

Effect of Earth Filling Costs

Earth filling cost may vary with the type of filling material and hauling distance, when more than one borrow pit is available for a construction project. Construction cost analyses for different borrow materials and hauling distances are useful for highway designers and project planners.

Figure 9 shows the impact of varying earth filling costs for the example problem. The construction cost increased with rising earth filling cost. The amount of increase for each unit rise in filling cost became lower at higher filling costs as the optimum alignment moved toward a profile with less and less volume of fill. Figure 10 illustrates this trend by comparing the optimum profiles for two cases of different earth filling costs.

Effect of Earth Cutting Costs

The effect of varying earth cutting costs on highway construction cost is similar to the effect of earth filling costs in many aspects, as illustrated by Figures 11 and 12. The curve in Figure 11 tended to level off as cutting costs were increased, reflecting the smaller unit impact on construction cost when cutting costs became higher. This trend could be explained by the fact, as depicted in Figure 12, that the volume of cut diminished as cutting costs escalated.

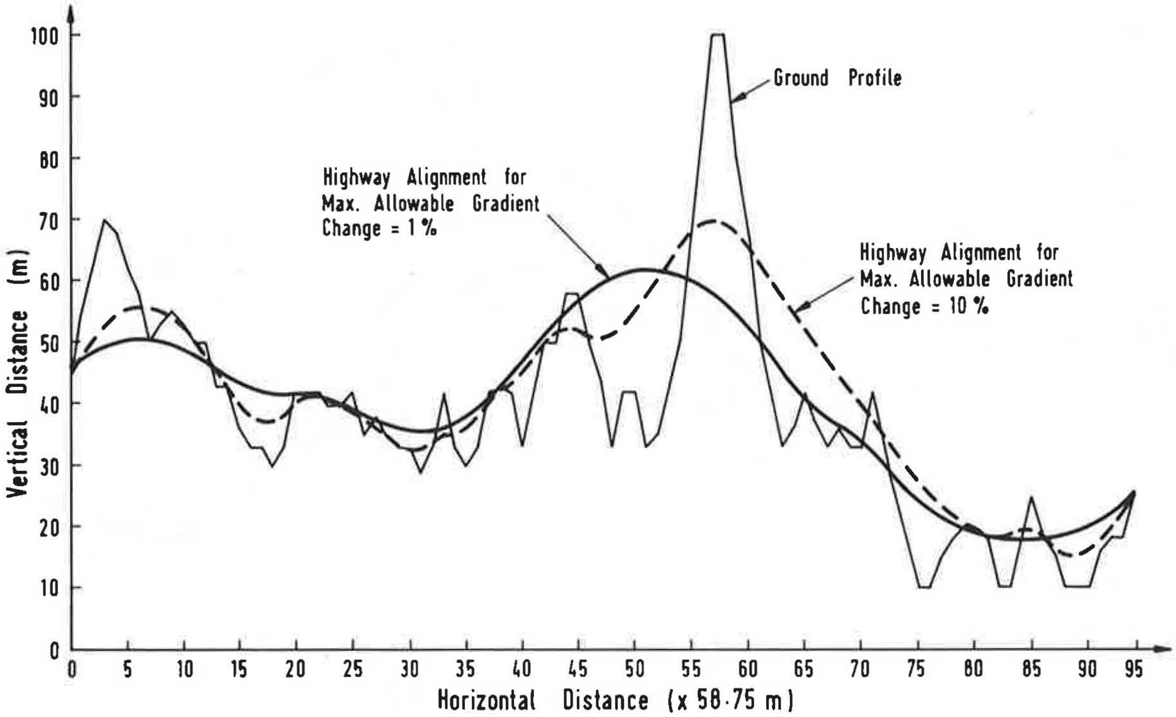

FIGURE 8 Effect of gradient change constraint on highway alignment in example problem.

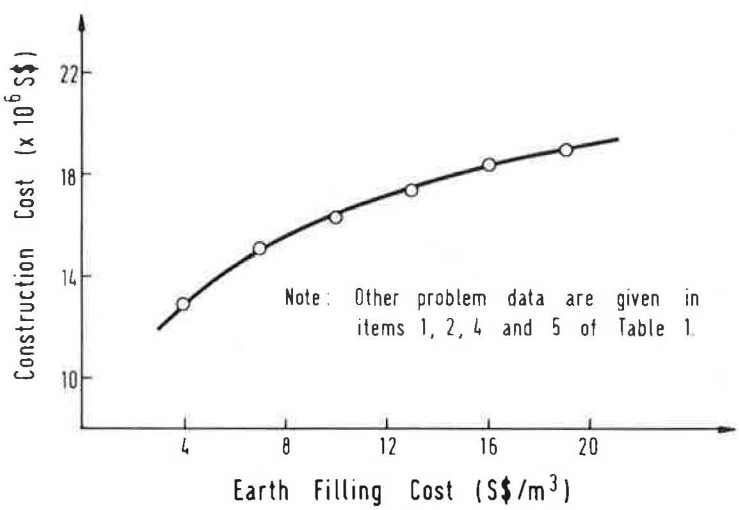

FIGURE 9 Effect of earth filling cost on highway construction cost.

Effect of Vehicle Operating Costs

Although vehicle operating costs have not been commonly considered in cost analyses for highway construction projects in Singapore, examination of how the inclusion of vehicle operating costs would affect engineering designs and project costs is instructive.

For the purpose of illustration, the cases analyzed in Figure 5 were recomputed with the addition of vehicle operating costs defined in item 6 of Table 1. The results of this analysis were plotted in Figure 13. Vehicle operating cost increased as higher values of vertical gradient were permitted in highway design, which had the effect of offsetting the savings in construction cost caused by reduced earthwork when higher vertical gradients were used. As a result, in contrast to the trend in Figure 5, the total costs shown in Figure 13 were relatively insensitive to the changes in allowable maximum vertical gradient used in highway geometric design.

The impact on total highway cost of construction prices and vehicle operating costs depends very much on the relative magnitudes of these two major cost items. In countries where vehicle operating costs are very high, it is possible on certain terrain that the total highway cost might increase as a higher maximum vertical gradient is allowed in design.

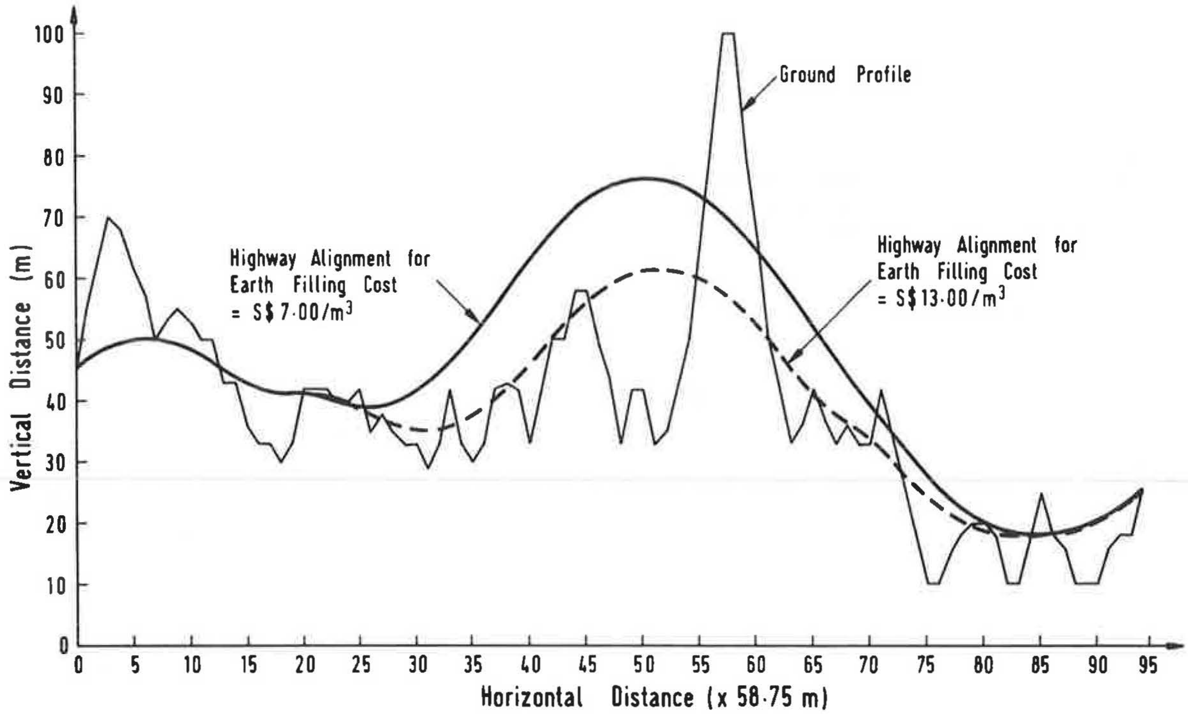

FIGURE 10 Effect of earth filling cost on highway in example problem.

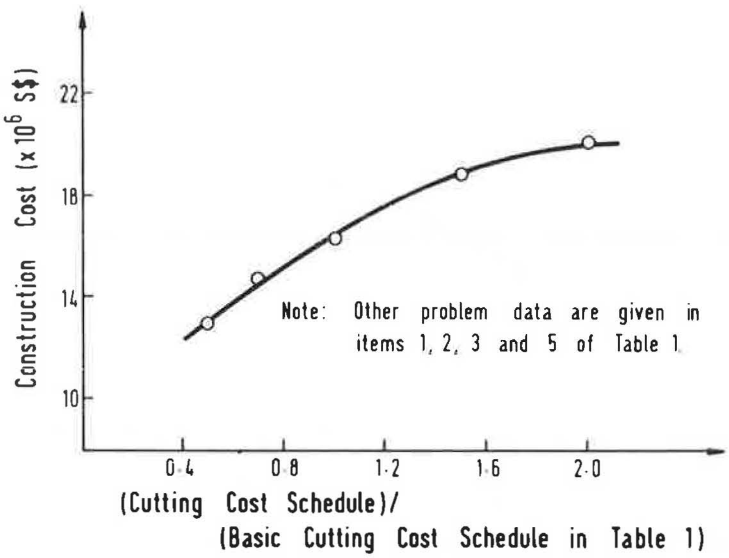

FIGURE 11 Effect of earth cutting cost on highway construction cost.

CONCLUSIONS

Highway vertical alignment analysis is useful for highway designers and planners in the location study, benefit/cost analysis, and geometric design of new highways. The dynamic programming formulation developed in Singapore has been found to be a valuable tool for local highway planners and designers. The formulation of the optimization program is simple in concept, yet has the versatility of modeling practically all of the cost items important in highway construction. The numerical example presented, based on a real-life ground profile and cost data, illustrated these points.

ACKNOWLEDGMENTS

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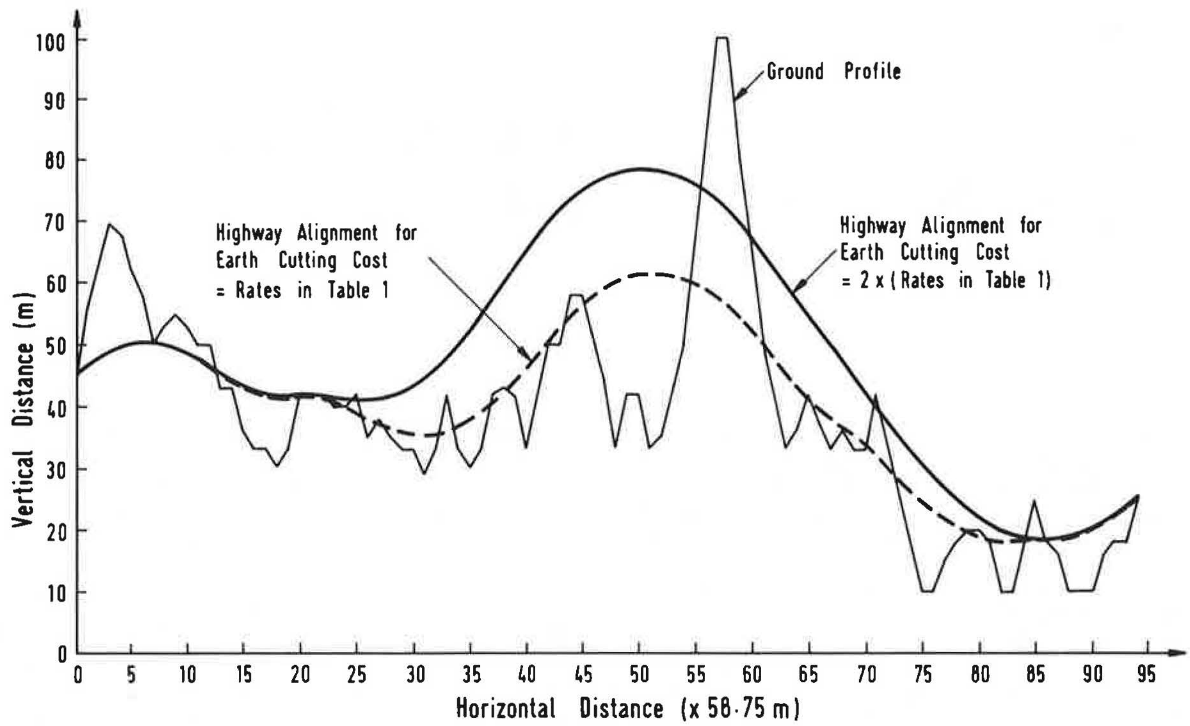

FIGURE 12 Effect of earth cutting cost on highway alignment in example problem.

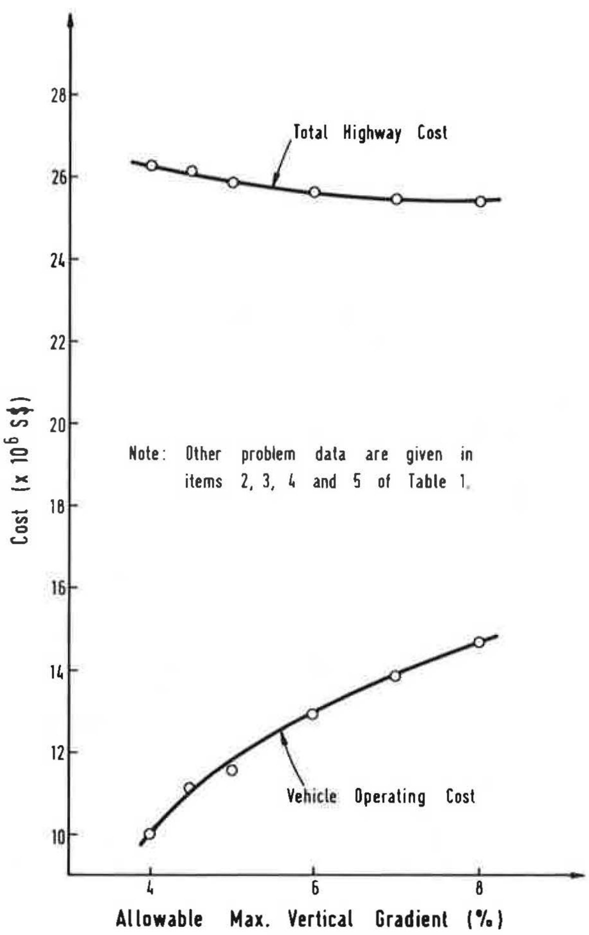

FIGURE 13 Effect of vehicle operating cost on total highway cost.

REFERENCES

1. OECD Road Research. *Optimization of Road Alignment by the Use of Computers*. Organisation of Economic Co-operation and Development, Paris, 1973.
2. I. Shacke. Optimization of Vertical Alignment—A Case Study. *Proc. Conference on Computer Systems in Highway Design*, Copenhagen, 1972, pp. 280–90.
3. V. Calogew. *Computer-Aided Highway Design*. Ph.D. thesis. Institute of Computer Science, University of London, 1969.
4. P. Antoniotti. APOLLON: A New Road Design Optimization Procedure. PTRC Symp. on Cost Models and Optimization in Road Location, Design, and Construction, London, 1969, pp. 236–41.
5. J. L. Groboillot and L. Gallas. Optimization d'un Project Reutier par Recherche de plus Court Chemin dans un Graphe a Trois Dimensions. R.I.R.O., Vol. 2, Paris, 1967, pp. 99–121.
6. R. W. Haymon. Optimization of Vertical Alignment for Highways through Mathematical Programming. In *Highway Research Record 306*, HRB, National Research Council, Washington, D.C., 1970, pp. 1–9.
7. R. Robinson. A Further Computer Model for Defining the Vertical Alignment of a Road: Programme VENUS II. *TRRL Report LR 469*, Department of the Environment, Crowthorne, Berkshire, U.K., 1972.
8. Public Works Department. *Manual for Geometric Design of Highways and Streets*. Ministry of National Development, Singapore, 1975.
9. *A Manual on User Benefit Analyses of Highway and Bus-Transit Improvements*. AASHTO, Washington, D.C., 1977, p. 51.

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Interactive Intersection Design Using an Expert Systems Approach

EDMOND CHIN-PING CHANG

The development of "expert systems" for microcomputer applications has received increasing attention in the transportation engineering field. Expert systems are computer programs that include simulations of the logical reasoning and problem-solving processes of human experts for specific applications. Proper implementation of this concept offers a possible means to effectively use the specific knowledge and experience of recognized professionals, efficiently use limited highway resources and revenue, and provide safe, efficient transportation programs. The major advantage of expert systems is that they permit specific human knowledge used in the decision-making process to be systematically examined, organized, and applied to particular engineering problems. During the typical intersection design process, highway engineers make many decisions concerning the operational effectiveness and trade-offs among a number of design factors. These important decisions may be bounded by either the planning budget, the potential total project costs, the maximum lane width of each typical highway lane, or the potential traffic demand volumes. This study illustrates the prototype applications of expert system design and LISP programming in the highway design process using the AutoCAD(TM) package. AutoLISP(TM), a version of LISP supported by AutoCAD, was used to create a small-scale expert system to interface with the normal drawing functions. This study demonstrates the feasibility of implementing built-in functions of the AutoCAD system through AutoLISP programs to assist end users in completing the decision making for potential highway design applications.

This study illustrates an application of expert systems through the LISP programming using the AutoCAD package in the highway design process (*1*). The AutoCAD(TM) drafting package is a powerful engineering tool, which allows engineers to prepare drawings that look virtually identical to drawings prepared by hand. AutoCAD supports AutoLISP, a version of the LISP computer programming language commonly employed in the field of artificial intelligence. AutoLISP can be used to create an expert system that can implement normal drawing functions (*2*). Functions are applied in AutoCAD through AutoLISP to assist end users in completing the decision-making process during the simplified highway designs application. These decisions may be bounded by either the planning budget, the potential total project costs, the maximum lane width of each typical highway lane, or the potential traffic volumes (*3,4*).

STUDY APPROACH

An expert system is a computerized decision-making assistance system. It can be designed to handle complex real-world problems through an expert's interpretation and solves these engineering problems by using a computer representation of the human reasoning process. If the system is properly designed and implemented, the computer will be able to assist users by offering suggestions, the way that human experts make recommendations in comparable situations.

This study was started to examine the feasibility of creating a highway design expert system with the built-in features of AutoCAD and AutoLISP. The programs apply some typical decision-making rules on the basis of information supplied by users and output suggestions or conclusions for users. A simplified set of production rules was defined to guide the computer program in assisting the user in reaching a viable conclusion. The simplified knowledge depends on compromises among the different highway design elements, such as the construction budget or environmental factors affecting the highway. The system applies a set of production rules to the input and gives its decisions back to the user. The data can be input interactively, retrieved from the existing input data file, or obtained from the existing design specifications.

STUDY BACKGROUND

The development of "expert systems" for microcomputer applications has received increasing attention in the transportation engineering field. Expert systems are computer programs that include simulate logical reasoning and problem-solving processes of human experts for specific applications. Proper implementation of these programs effectively uses specific knowledge and experience of recognized professionals, efficiently uses limited highway resources and revenue, and provides safe and efficient transportation programs. Expert systems permit users to systematically examine, organize, and apply specific human decision-making knowledge to particular engineering problems.

In addition to the basic requirements, the acceptance and operational use of this estimation process depends on the simple to use but nevertheless highly sophisticated user interfaces to assist in the complicated calculations. Historically, the information required for the highway user benefit and cost evaluations has been fairly complex (*3,4*). Familiarity with and training in the analysis procedures are often required of users. Usefulness of the design in the operating agencies is often constrained by lack of incentive for performing operational reviewing that can further improve the data quality. To improve user implementation, computer and graphics interfaces are increasingly used. This effectively lowers resistance to the new applications and reduce a user's training

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requirements. Through these changes, an improved, overall system functional design can then be built. Different types of user interface have been used in many highway management systems, although the software necessary for implementation has not yet been fully developed. Consequently, effective use of the expert system in design relies on efficient implementation of available techniques and tools.

A useful highway design system must provide the end user with a fast, powerful, and easy information management method to interactively access, accumulate, update, and assimilate road information. Such a system can take advantage of the effective, graphics oriented environment which local authorities or divisional offices can download and operate in a familiar visual display format. The data base information may be represented through a combination of graphic displays similar to maps, general layouts of their road system, and associated installations and facilities. This paper explores the necessary interface and data requirements for such a system. Consequently, this prototype could be expanded further to enhance the interaction with different interface from the potential user's perspective. In addition, changes can easily be made to provide a good prototype which avoids potential problems encountered during user testing and product upgrade before final system implementation.

Design Considerations

During the typical highway design process, highway engineers make certain decisions to determine the operational trade-offs among a number of design factors. Some important traffic engineering design considerations may include expected traffic demand volumes, number of traffic lanes, and necessary lane width.

One of the important underlying design objectives is to evaluate the required highway capacity that not only accommodates the current traffic demands but also provides the necessary near-term traffic demands. However, the maximum number of traffic lanes and achievable lane widths are usually bounded by the maximum available right-of-way road. Therefore, the design of highway lanes and lane widths is often constrained to certain working ranges.

In addition to these design considerations, engineers also need to consider the operating budget constraints and potential allowable total costs in certain highway improvement projects. The cost of building highway facilities can usually be broken down into fixed portions of the construction costs as well as the variable costs of constructing an additional lane. The variable cost for constructing each traffic lane may depend on the number of additional lanes, lane widths, and some other traffic related factors.

AutoCAD System

AutoCAD®, a commercially available computer-aided design (CAD) tool, is one of the most powerful CAD packages used on IBM PC/XT/AT/386-compatible microcomputers. It is also available for many other minicomputers and engineering workstations. The system operation requires the basic computer system, which includes the processor, keyboard, text display screen, disk drives, and a graphics monitor capable of reasonably high resolution. A plotter, or printer plotter, can be connected to the system to provide a hard copy of the drawing files. A number of graphics input or pointing devices can be used, such as the mouse, digitizing tablet, or TouchPen®.

On some computer systems AutoCAD may use two display monitors—one for command prompts and text output and the other for graphics. On other systems a single monitor may be used for both graphics and text purposes. In these situations, three lines at the bottom of the screen are reserved for command entry and prompts, and the right edge may contain a screen menu. When AutoCAD runs on a single-monitor system, it remembers a full 24 lines (or more) of text, just as it does on the regular text display. AutoCAD automatically switches to the text display when it outputs a large amount of information and automatically returns to the graphics display when drawing graphically.

When users enter AutoCAD's Drawing Editor, a screen menu will appear along the right edge of the screen, as shown in Figure 1. Users can move the mouse or digitizer cursor to select items in the menu and perform actions as needed. They can also extend AutoCAD's capabilities and customize them for particular application by designing their own menus and submenus to complement those supplied with the program.

Program Operation

AutoCAD operates on two shell levels to reduce both the effort required to generate a drawing and the time needed to learn the system. At the outer level, AutoCAD provides a menu-driven interface, the Main Menu, that allows it to initiate various working tasks, such as creating new drawings, modifying previously stored drawings, and producing plots. The main menu is displayed on the screen at the beginning of AutoCAD execution, and users can use this screen to end an AutoCAD session. It provides quick access to various parts of AutoCAD, such as the interactive Drawing Editor and the plotter interface.

The Drawing Editor provides easy access to the drawing, as a text editor accesses a document. When users create a new drawing or edit an existing one, AutoCAD automatically loads the Drawing Editor, which displays the drawing and provides commands to create, modify, view, and plot drawings. When users have finished working with a given drawing,

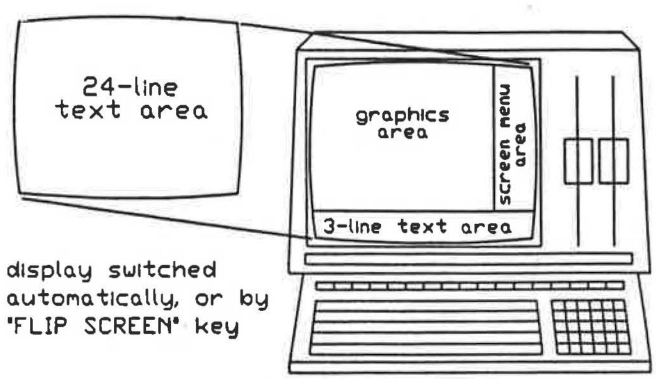

FIGURE 1 Typical single-screen configuration of the AutoCAD system.

any changes made can be saved or discarded before returning to the Main Menu. All information on the drawing—the size and position of every element, the size of the drawing itself, and its display characteristics—is automatically updated with each command. Also, users can zoom, move, pan, or rotate the drawing automatically with each command. This information will be stored when the AutoCAD system is exited.

A user can specify different points in the drawing in a variety of ways. From the keyboard, points can be designated by typing in absolute coordinates or coordinates relative to the last point specified. A user can use keyboard control keys to move the cursors around the graphics monitor and visually locate the desired point(s). A graphics input device can also be used to designate points, which can be locked or snapped to a user-defined grid.

Commands can be entered in several ways. A command can be typed in directly or selected from any of the menus. Customized menus can be constructed through the screen, tablet, and button menus. The screen menu can be displayed on the graphics monitor while the Drawing Editor is active and allows command entry by simply pointing to the command on the display screen with a pointing device or with the keyboard.

A tablet menu includes up to four menus of AutoCAD commands on the digitizing tablet, permitting a command to be entered by pointing to it with the stylus and pushing a button. If the tablet stylus or mouse has multiple buttons, the button menu can be used to enter often used commands.

An auxiliary function box can also be used. It has buttons for command selection, but it cannot be used as a pointing device. A user can also plot a hardcopy of the drawing at any stage in its development. Check-plots can also be generated while the drawing is in progress to check for positioning and dimensioning errors that might not be immediately apparent on screen. When the drawing is complete, the final plot is done to produce the finished drawing.

AutoLISP is an implementation of the LISP programming language embedded within the AutoCAD system. A popular computer programming language used in artificial intelligence programs, LISP is a very powerful language suitable for the expert system development on engineering designs because of its ability to interface with AutoCAD. AutoLISP allows users and AutoCAD developers to implement macro programming in a very powerful high-level programming language format well suited to the interactive graphics applications.

EXPERT SYSTEM DESIGN

Concept Design

To maximize flexibility, a dual design approach has been used to provide interactive intersection design in this prototype expert system. As shown in Figure 2, the design analysis approach can be specified through the objectives required by the users, according to their functional requirements and evaluation priorities. Users may wish to find the desired intersection configuration and total construction costs according to the traffic demands, or they may want to use the system to find the best design on the basis of budgetary concerns.

The first approach allows users to evaluate costs on the basis of traffic demand inputs and specified design requirements. Users input the number of lanes and lane widths to be used, and the program evaluates estimated costs for constructing the intersection. Another approach is for users to suggest the number and widths of traffic lanes on the basis of available financial resources and input the total budget and traffic volumes. The output suggests the total cost, number of traffic lanes, and lane widths to be used.

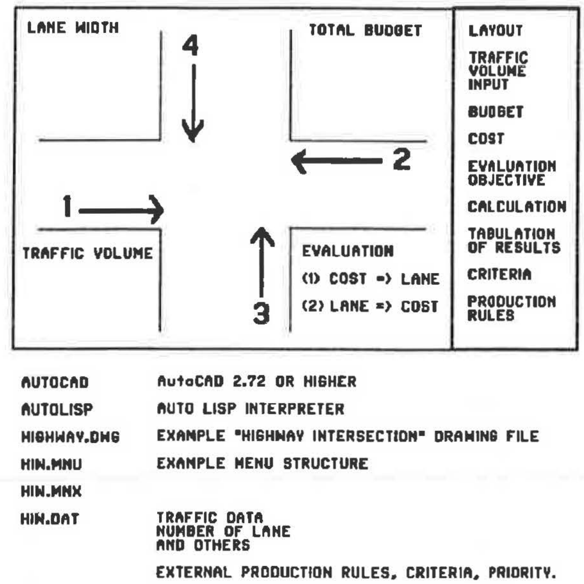

FIGURE 2 Functional design of the interactive intersection design system.

Design Parameters

The estimated costs of designing the intersection can be broken down into fixed and variable portions, which can be used by the program and stored in existing data file. During program initialization, the computer can retrieve these design parameters from historical files and save the values in some memory variables. These design parameters can be modified at any time during the program evaluation. When finished, the computer will automatically update the external parameter file with the new values.

In addition, a user can also review or modify the data file directly and store design parameters outside the graphics system. In this way, interactive design systems can use the same set of parameters for consistent design evaluation each time. The program runs continuously until values are modified. The design parameters used in this prototype program development include (a) maximum capacity of a traffic lane, (b) maximum and minimum width of a traffic lane, (c) fixed cost per lane, and (d) variable cost per lane-foot.

Production Rules

Several production rules are also implemented. The set of rules is preset and stored externally outside the AutoCAD system. The decision rules can be turned on or off during program evaluation. Similar to the design parameter file, these production rules can also be stored or updated by editing the disk file of production rules directly. Three production rules are illustrated in this program:

1. If the total evaluated cost is greater than the total budget, either adjust the lane width to lower the total cost or ignore the budget and stick with the evaluated cost.
2. If the number of lanes input by the user does not meet the requirement of the traffic volume, increase the number of lanes.
3. If the lane width input by the user or evaluated by the program is not a whole number, either truncate or round the number off.

System Development

Because a fairly large amount of memory is usually required in the AutoCAD operations, development of the interactive design system through interfacing with other languages is limited. This is due to the standard memory of the IBM PC/XT/AT/386-compatible microcomputers under the MicroSoft Disk Operating System (MSDOS) environment. Therefore, AutoCAD allows only a limited memory size for loading AutoLISP or other high-level languages on the IBM AT under MSDOS version 3.2 with 1024K main memory.

Compared with the commercial GCLISP system, a popular version of the common LISP program available on microcomputers, AutoLISP is not as powerful in system building, data and knowledge analysis, and logical reasoning. GCLISP provides many more built-in functions. However, AutoLISP is a programming language that is embedded within the AutoCAD system, designed to provide interfacing capability directly with the AutoCAD graphics functions. It can activate all available functions in AutoCAD directly, such as drawing a circle or setting the system variables. Therefore, AutoLISP is more appropriate for building a prototype expert system for interactive intersection design.

Program Implementation

To run AutoCAD and load the interactive intersection design system, the user needs only to run the batch file HIW.BAT, which loads the AutoCAD main program, drawing files, and the AutoLISP environment program automatically. As shown in Figure 3, after the user points the mouse cursor to "START" on the drawing screen and activates it by clicking the button on the mouse, AutoCAD will load HIW1.LSP and HIW2.LSP. The following are brief descriptions of these program files.

Drawing Files

As shown in Figure 4, the drawing file "HIW.DWG" interactively displays the physical layout of the study intersection, illustrates detailed design requirements, shows active design objective, tabulates evaluation results in the table, and displays descriptions of the recommended system design. At the end of the evaluation, the expert system automatically outputs the results and draws suggested lane lines onto the screen.

AutoLISP Programs

Two AutoLISP programs were developed in this prototype intersection design to perform the necessary data input, decision making, output interpretation, and graphics illustration support. The first system component provides global system initialization and user's input. The second system component applies the linkage to the user-defined AutoLISP functions and built-in AutoCAD system drawing functions.

The first system component ("HIW1.LSP") provides the initialization and interpretation of the LISP program. It reads the prestored parameter file, production rules, number of input lanes, and traffic volume input and keeps all user-selected values as accessible memory variables. The following example illustrates the system initialization:

```
; RESET SYSTEM VARIABLES
(setvar "cmdecho" 0)        ; NO COMMAND ECHO
(setvar "blipmode" 0)       ; NO BLIP MODE
(setvar "expert" 1)         ; NO CONFIRM QUESTIONS
(setvar "menuecho" 3)       ; NO MENUS ECHO

; BLANK THE MENUS AREA
(menucmd
"S=BLANK")
(command "layer" "off" "help" "on" "wait" "")
; INITIALIZE BUDGET, COST AND WIDTH AS WELL
; AS VARIOUS FLAGS
(setq budget 0)             ; PRE-SET BUDGET
(setq cost 0)               ; COST OF HIGHWAY
(setq width 8)              ; LANE WIDTH
(setq chg_made nil)         ; ANY GLOBAL
                            ; VARIABLES CHANGE
```

HIGHWAY	-	TAMU.TTI TRAFFIC OPERATIONS
Layout	-	Draws the intersection
Rules	-	Display the production rules
Criteria	-	Display the criteria
Vol In	-	Input the Traffic Volume
Budget	-	Input the Project Budget
Calc	-	Do the calculation
Results	-	Summarize results in a table
HELP	-	Display the HELP Information
Exit	-	Exit this program

FIGURE 3 Main menu of the interactive system.

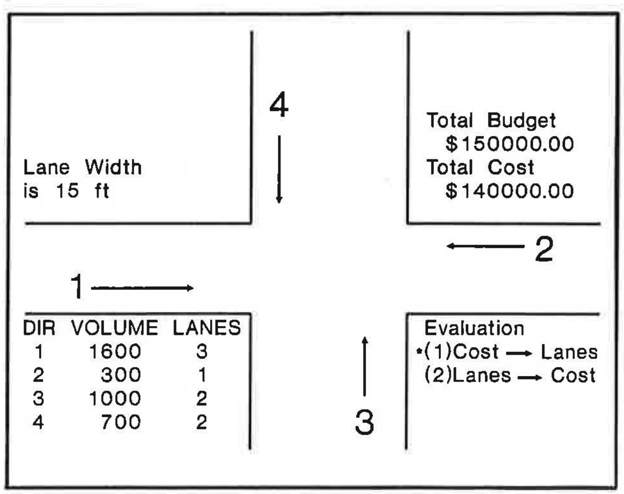

FIGURE 4 Display screen of the interactive system.

```
; FUNCTION TO TRUNCATE A REAL NUMBER
; TO INTEGER
(defun truncate (r)
  (- r (rem r 1)))
```

In addition, the following example program defines initialization of all the traffic input design considerations as default values:

```
; INITIALIZE PRM (VOLUMES, MIN LANE WIDTH, MAX
; LANE WIDTH, FIXED COST AND VARIABLE COST
; PER LINE)
  (setq prm (if (or (null (setq f1 (open "hiw.prm" "r")))
                    (null (setq line (read-line f1))))
                '((vol 500) (min 8) (max 16) (fix 10000.0)
                  (var 500.0))
                (read line)))
  (if (null (assoc 'vol prm))
    (setq prm (cons '(vol 500) prm)))
  (if (null (assoc 'min prm))
    (setq prm (cons '(min 8) prm)))
  (if (null (assoc 'max prm))
    (setq prm (cons '(max 15) prm)))
  (if (null (assoc 'fix prm))
    (setq prm (cons '(fix 10000.0) prm)))
  (if (null (assoc 'var prm))
    (setq prm (cons '(var 500.0) prm)))
```

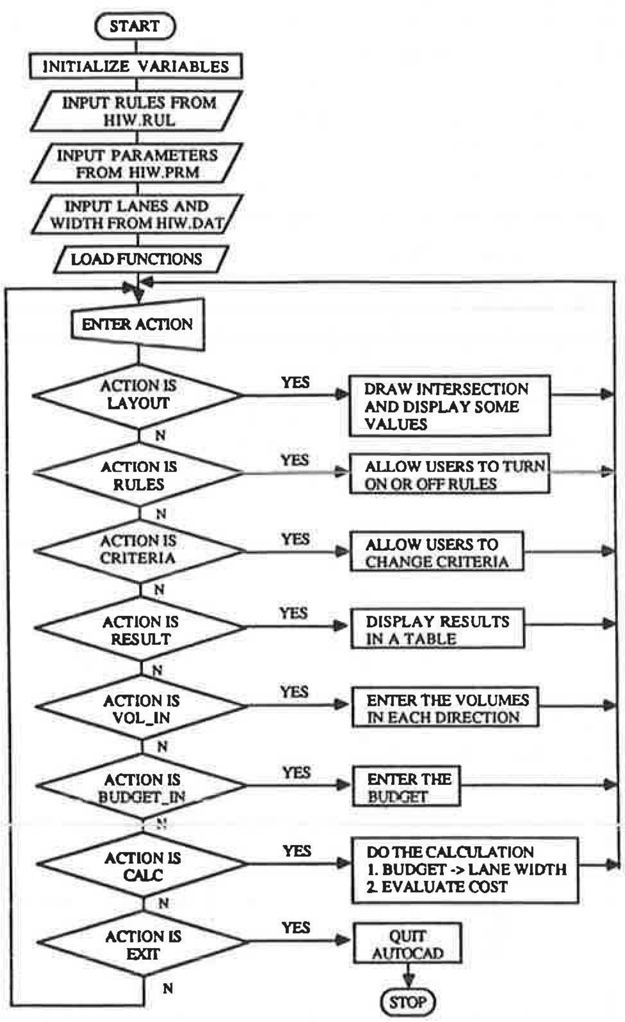

FIGURE 5 Program flowchart of the interactive system.

The second component, or "HIW2.LSP" file, applies the written AutoLISP functions for program execution. Specifically, the program will perform the following functions according to the program flowchart as shown in Figure 5. It illustrates the overall conceptual design of the prototype expert system.

First, the system draws the simplified intersection schematics to illustrate all design elements. Next, the user will be asked to update both the production rules and the evaluation criteria from the historical data files stored previously. Then, the user can select to start the evaluation objectives from the known budget constraints or the preliminary design to purely satisfy the traffic demands. The system will then acquire additional input data based on the approach selected. Basic calculations include the computation of allowable numbers of approach lanes from the available operating budget, computation of potential project costs, and the basic requirements of number of approach lanes from traffic demands. Finally, the system will summarize recommendations and provide the preliminary design configuration.

The following section presents an example of AutoLISP program application to determine the minimum design requirements of various types of inputs when a design compromise needs to be made. A user may encounter three types of decision-making processes during the analysis. According to the production rule design in this prototype system, the program will respond with three responses when determining the lane widths based on the following three conditions. The corresponding program codes being implemented are:

1. If any variable is zero, prompt the user for it;
2. If the result width is too wide, set it to the upper limit; and
3. If the result width is too narrow, no solution.

```
; IF ANY VARIABLE IS ZERO, PROMPT THE USER FOR IT
  (if (<= budget 0)
    (setq budget (getreal "Enter your total budget:  ")))
  (if (equal vol '(0 0 0 0))
    (progn
      (setq vol ())
      (terpri) (setq i 0)
      (repeat 4
        (setq i (1+ i))
        (princ "Enter volume in direction") (princ i)
        (setq vol (append vol (list (getint ": ")))))))
  (setq lanes (findlanes vol))
  (terpri)
  (setq width (truncate (/ (- (/ budget (sumup lanes))
      (cadr (assoc 'fix prm))) (cadr (assoc 'var prm)))))

; IF THE RESULT WIDTH IS TOO WIDE, SET IT TO THE
; UPPER LIMIT
  (if (> width (cadr (assoc 'max prm)))
    (setq width (cadr (assoc 'max prm))))

; IF THE RESULT WIDTH IS TOO NARROW, NO SOLUTION
  (if (< width (cadr (assoc 'min prm)))
    (progn
      (setq cost 0)
      (textscr)
      (terpri) (princ "No solutions!") (terpri)
      (terpri) (princ "Not enough budget!") (terpri)
      (setq width (cadr (assoc 'min prm)))
      (setq cost (* (sumup lanes)
        (+ (cadr (assoc 'fix prm))
        (* (cadr (assoc 'var prm)) width))))
      (terpri) (princ "Hit (RETN) to continue")
      (read-char) (read-char))
```

System Operations

The AutoCAD screen drawing speed is rather slow compared with interpreting and running AutoLISP functions in the overall operation. Therefore, 11 drawing layers have been implemented at the beginning of program initialization. These graphic layers appear to make the program run faster. During program execution, different calculation results and roadway elements are displayed on different drawing layers on the screen. When there is a need to display another screen, the program switches to the corresponding program layer for that particular display instead of erasing the entire screen and drawing the other screen from scratch. Using different graphics layers, the drawing file can be prepared all at one time during program initialization and take much less time to load the drawing file and speed up the screen display.

Eleven graphics layers are used in this program:

1. 0 (display values, such as budget, cost, volumes).
2. DISPLAY (number of lanes, lane width, as shown in Figure 4).
3. HIDDEN (for the purpose of drawing hidden lines).
4. CENTER (for the purpose of drawing center lines).
5. OBJECTIVE (display the evaluation objectives, as shown in Figure 6).
6. CRITERIA (display the criteria, as shown in Figure 7).
7. RULES (display the production rules, as shown in Figure 8).
8. BORDER (draw the border of the screen).
9. RESULT (tabulation of evaluation results, as shown in Figure 9).
10. LAYOUT (display the intersection and the recommended design).
11. LANES (layer for drawing the lanes, as shown in Figure 10).

In addition to the preparation of these 11 drawing layers, a "WAIT" layer has also been implemented to display the "Please Wait" message during program execution. A customized menu has been created for using this system and includes a HELP menu to provide a brief description of each item on the custom menu, as shown in Figure 3. The menu items on the right-hand side of the drawing screen can be activated individually by pointing the mouse or digitizer cursor to a particular item and pressing the button to activate the selection. In this sample run, as illustrated in Figures 6–10, only

Evaluation Objectives

1. Suggest number of lanes in each direction and the total cost according to the input volumes and budget.
2. Calculate the total cost according to the input number of lanes and volume in each direction.

FIGURE 6 Evaluation objectives of the interactive system.

Criteria

1. One lane every 600 vehicles.
2. Lane width between 8–15 ft.
3. Fixed cost each lane $10,000.
4. Width costs $500. per foot.

FIGURE 7 Evaluation criteria of the interactive system.

Production Rules:

*1. If Cost > Budget, Use cost and Update budget. {Otherwise, reduce cost.}
2. If original number of lanes not enough for the volume, overwrite no. of lanes.
3. Round off lane width to nearest integer. {Otherwise, Truncate lane width.}

* – Rule is turned on

FIGURE 8 Production rules of the interactive system.

DIR	VOLUME	LANES
1	1600	3
2	300	1
3	1000	2
4	700	2

COST = $140000.00
WIDTH = 15ft

FIGURE 9 Evaluation results of the interactive system.

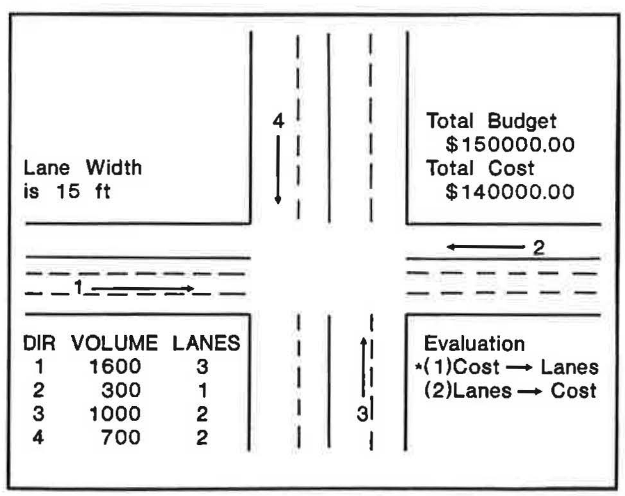

FIGURE 10 Final drawing of the interactive system.

one menu is constructed; however, this prototype system can be expanded further with a number of submenus to provide additional design features. Users can also switch to different submenu items by pointing to the submenu name on the menu, or entering the command, for switching among menu selections.

CONCLUSIONS AND RECOMMENDATIONS

Intersection design involves the interactive evaluation of automobile, truck, public transit, taxi, pedestrian, and bicycle movements through the operating, regulatory, and service policies to achieve maximum efficiency. By properly identifying and reviewing the required traffic design improvements when developing adequate transportation facilities, many low-cost TSM strategies could be applied to improve intersection operations and achieve desired air, environmental, and community qualities, as well as fuel and economic efficiency. Many TSM type strategies may be applied through implementation of the practical, low-cost, short-range planning and programming projects. However, many current highway design and plan review procedures are difficult to implement because of the complex manual analysis process involved (*5,6*).

This study examines the possibilities of integrating analysis procedures and existing review processes into an interactive portable system that can interface with the existing graphics design systems. During a typical intersection design process, highway engineers have to make many decisions and determine the operational trade-offs among a number of design factors. These decisions may be bounded either by the planning budget, potential total project costs, maximum lane width of each typical highway lane, or the potential traffic demand volumes. This study illustrates the prototype applications of expert system design and LISP programming in the highway design process using the AutoCAD package. AutoLISP, the version of LISP supported by AutoCAD, was used to create a small-scale expert system to interface with the normal drawing functions.

This study demonstrated the feasibility of implementing some built-in functions of the AutoCAD system through AutoLISP programs. This implementation will assist end users in the decision making for the potential intersection design and highway planning applications. It is possible that this prototype system can be expanded into an interactive evaluation and plan review system to assist in designing and evaluating many TSM improvements.

The prototype can be used to identify potential problem areas, define study frameworks, develop evaluation criteria, warn of unfeasible alternatives, refine candidate actions, and recommend workable solutions for improving the design and operational analysis of the integrated intersection graphics design. The implementation of these design evaluation and plan review procedures, based on the expert systems designs and graphics design concept, can greatly assist end users in the design and plan review of various strategies, which are recommended for improving the design, planning, plan review, and implementation of the intersection design.

ACKNOWLEDGMENTS

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REFERENCES

1. *AutoCAD Reference Manual.* Autodesk, Inc., June 1986.
2. G. O. Head. *AutoLISP in Plain English.* Ventana Press, Chapel Hill, N.C., 1985.
3. AASHTO. *A Manual for User Benefit Analysis of Highway and Bus Transit Improvements*, Washington, D.C., Feb. 1977.
4. D. A. Curry and D. G. Anderson. *NCHRP Report 133: Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects.* HRB, National Research Council, Washington, D.C., 1972.
5. J. H. Batchelder. *NCHRP Report 263: Simplified Procedures for Evaluating Low-Cost TSM Projects—User's Manual.* TRB, National Research Council, Washington, D.C., 1983.
6. J. M. Mason, Jr., J. D. Blaschke, and D. M. Chang. *NCHRP Report 283: Training Aid for Applying NCHRP 263.* TRB, National Research Council, Washington, D.C., 1986.

The content of this paper reflects the views of the author, who is solely responsible for the opinions, findings, and conclusions presented herein.

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Safety Effects of Left-Turn Lanes on Urban Four-Lane Roadways

PATRICK T. MCCOY AND MICHAEL S. MALONE

As part of research conducted to develop a more definitive guide for the selection of divided and undivided sections on urban four-lane roadways in Nebraska, accident experience at signalized and unsignalized intersections on urban four-lane roadways was analyzed to assess the safety effects of left-turn lanes. Results of this analysis are presented. Multivehicle accidents on intersection approaches with left-turn lanes were compared with those on similar approaches without left-turn lanes. The degree to which left-turn lanes on signalized and on uncontrolled approaches reduced acccidents was computed. The statistical significance of the percent reductions was determined using the chi-squared test. Left-turn lanes at intersections on urban four-lane roadways were found to significantly reduce rear-end, sideswipe, and left-turn accidents. However, on the uncontrolled approaches of intersections on urban undivided roadways, left-turn lanes were found to significantly increase right-angle accidents, as well as reduce rear-end, sideswipe, and left-turn accidents.

The Nebraska roadway design manual (*1*) contains a guide for the selection of typical sections on urban roadways. According to the guide, four-lane undivided sections should be selected for roadways with projected design hourly volumes (DHVs) between 400 and 600 vehicles per hour (vph), and four-lane divided sections should be selected for roadways with projected DHVs between 1,800 and 3,200 vph. For roadways with projected DHVs between 600 and 1,800 vph, the guide suggests using either a four-lane undivided or divided section, depending on the character of the roadway, traffic, and surrounding area.

The experience of the Nebraska Department of Roads (NDOR) with the guide indicates that it is too ambiguous to use for urban roadways with projected DHVs between 600 and 1,800 vph. Frequently, four-lane undivided sections have been selected and later found to be inadequate well in advance of their design years because they do not provide for left-turn lanes at the intersections. Therefore, research was undertaken to develop a more definitive guide to consider the need for left-turn lanes at intersections on urban roadways with projected DHVs between 600 and 1,800 vph.

The need for left-turn lanes was determined on the basis of intersection capacity and the safety effects of left-turn lanes. Intersection capacities were evaluated to determine the traffic volumes at which left-turn lanes would be required in order to provide design levels of service. Accident experience at intersections on urban four-lane roadways was analyzed to assess the safety effects of left-turn lanes. The accident analysis is presented in this paper. The capacity analysis and the section selection guide developed are presented elsewhere (*2*).

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PREVIOUS RESEARCH

Several studies have been conducted of the safety effects of left-turn lanes. Only a few of those conducted at intersections on four-lane roadways, however, have been reported in the literature.

A before-and-after study of 53 left-turn channelization projects at urban and rural intersections in California found that the installation of left-turn lanes resulted in significant reductions in accidents (*3*). Rear-end, left-turn, and total accidents at unsignalized intersections were reduced by 85 percent, 37 percent, and 48 percent, respectively. However, right-angle accidents increased significantly by 153 percent. At signalized intersections, left-turn and total accidents were reduced by 54 percent and 17 percent, respectively. No significant changes in right-angle and rear-end accidents were reported.

Accident experience over a 2-year period on 363 intersection approaches on rural state highways in Ohio was analyzed to evaluate the safety effects of left-turn lanes (*4*). Approaches were classified with respect to signalization, number of lanes, presence of a left-turn lane, and intersection type. Approaches with left-turn lanes were found to have lower accident rates than approaches without left-turn lanes.

On four-lane roadways at unsignalized approaches with left-turn lanes, left-turn and total accident rates were 27 percent and 32 percent lower, respectively, and at signalized approaches with left-turn lanes, left-turn and total accident rates were 39 percent and 9 percent lower, respectively. None of these differences was found to be statistically significant at the 5 percent level of significance. However, the results of the study showed that the number of approach lanes and the type of intersection control must be considered in the evaluation of the safety effects of left-turn lanes.

A study of the relationships between accidents and roadway conditions revealed that there were significantly higher accident rates at intersections with opposing left-turn lanes than at intersections without left-turn lanes (*5*). The addition of left-turn lanes at signalized intersections without left-turn phases was found to increase accident rates, a situation that led to the recommendation that left-turn lanes be used as a means to increase intersection capacity and not as an accident-reduction measure. However, these findings were confounded by the failure to differentiate between one-lane and two-lane approaches.

Five years of accident data for intersections in Lexington, Kentucky, were used to investigate the relationship between left-turn accidents and left-turn lanes (*6*). The study definition of left-turn accidents included three types of collisions: (a) a vehicle turning left into the path of an oncoming vehicle; (b) a left-turning vehicle that is struck from behind while waiting to turn left; and (c) a vehicle that weaves around a vehicle stopped to turn left and is struck by a third vehicle. The left-turn accident rates for intersections with left-turn lanes were found to be substantially lower than those for intersections without left-turn lanes. The left-turn accident rate was 77 percent lower at unsignalized intersections and 54 percent lower at signalized intersections without protected left-turn phases.

The accident reduction factors from the literature cited previously are summarized in Table 1. Except for right-angle accidents at unsignalized intersections and rear-end accidents at signalized intersections, left-turn lanes were consistently found to be associated with fewer accidents. However, only the accident reductions found in the California study (*3*) were reported as statistically significant. None of the accident-reduction factors was reported as being computed exclusively from accident experience at intersections on urban four-lane roadways.

PROCEDURE

Accident experience at intersections on urban four-lane roadways in Nebraska was analyzed to determine the safety effects of left-turn lanes at these locations. The first step was to select the intersections for the study from among urban four-lane roadways with DHVs between 600 and 1,800 vph, the focus of the research. According to NDOR traffic count data (*7*), the DHV on an urban roadway is about 10 percent of the annual average daily traffic (AADT). Therefore, the (NDOR) computerized state highway system inventory was searched to identify all urban four-lane segments with AADTs between 6,000 and 18,000 vpd. The NDOR traffic signal inventory and photolog data were then used to locate and classify the intersection approaches on these segments.

Intersection approaches were classified according to type of control and presence of left-turn lane:

1. Signalized approach without a left-turn lane,
2. Signalized approach with a left-turn lane,
3. Uncontrolled approach without a left-turn lane, and
4. Uncontrolled approach with a left-turn lane.

Signalized approaches with protected left-turn phases, stop-sign controlled approaches, and yield-sign controlled approaches were not included in the study. Only approaches at intersections with AADTs on the crossroads of at least 1,000 vpd were classified.

TABLE 1 ACCIDENT REDUCTION FACTORS FOR LEFT-TURN LANES FROM PREVIOUS RESEARCH

Accident Type	Unsignalized Intersections (%)	Signalized[a] Intersections (%)
Right angle	-153^{b} (*3*)	None reported
Rear-end	85^{b} (*3*)	-15 (*3*)
Left turn	37^{b} (*3*), 27 (*4*)	54^{b} (*3*), 39 (*4*)
	77^{c} (*6*)	54^{c} (*6*)
All	48^{b} (*3*), 32 (*4*)	17^{b} (*3*), 9 (*4*)

[a]Without protected left-turn phases.
[b]Statistically significant at the 5 percent level of significance.
[c]Includes left-turn related rear-end and sideswipe accidents.

A minimum of 10 intersection approaches in each of the four approach categories were to be used in the accident study. Ten intersections from each category were selected initially at random. Photologs, construction records, and traffic volume data were examined to determine if the roadway and traffic conditions had remained the same since 1984 at each of the intersections selected. Approaches at intersections where the conditions had changed were not used as study sites.

The approaches in the two signalized approach categories were compared to ensure that they had similar roadway and traffic conditions and that the only major distinction between them was the presence of left-turn lanes. Likewise, the approaches in the two uncontrolled approach categories were compared. Approaches with conditions that differed from the majority were not used as study sites. If the elimination of some intersections reduced the total number of study sites in any category to fewer than 10, additional intersections were selected at random to increase the number to at least 10.

Current 8-hour turning movement counts and copies of all accident reports for 1984, 1985, and 1988 for the study sites were obtained from NDOR. The turning movement counts were expanded to AADTs, which were used to compute mean accident rates for each approach category.

Accident rates for the approach categories with left-turn lanes were compared with those for the corresponding approach categories without left-turn lanes to compute the reductions in accident rate attributed to left-turn lanes. The statistical significance of the percent reductions was determined using the chi-square test (*8*), which has also been referred to as the Poisson comparison of means test (*9*).

STUDY SITES

A total of 63 intersections were found to have approaches that met site selection criteria. See Table 2 for the number of intersections in each approach category. Ten intersections were initially selected at random from each category. Four of the 40 were eliminated because the roadway and traffic conditions had not remained the same since 1984. These were not replaced, however, because the remaining intersections provided more than 10 study sites in each category. A total of 46 study sites were used. Table 3 presents the number of study sites in approach categories.

All of the study sites were on tangent sections of urban four-lane roadways in outlying commercial areas with streetlights. All were approaches to four-leg, right-angle intersections. Most were on level grades, and the rest were on slight to moderate grades. None was on a hillcrest or had sight distance restrictions caused by the alignment of the roadway.

TABLE 2 TOTAL NUMBER OF INTERSECTIONS IN EACH APPROACH CATEGORY

Approach Category	Number of Intersections
Signalized[a] without left-turn lane	20
Signalized[a] with left-turn lane	15
Uncontrolled without left-turn lane	14
Uncontrolled with left-turn lane	14
Total	63

[a]Without protected left-turn phases.

Posted speed limits of 30 and 35 mph were found at sites in the signalized approach categories; at sites in the uncontrolled approach categories, they were between 35 and 45 mph. Table 4 gives the distribution of study site speed limits.

Sites without left-turn lanes were on four-lane undivided roadways and had two 12-foot lanes—one through/left-turn lane and one through/right-turn lane. Sites with left-turn lanes were on four-lane divided roadways with 16-ft raised curb medians and had three 12-ft lanes—one left-turn lane, one through lane, and one through/right-turn lane. The opposing approach at each site also had a left-turn lane.

ACCIDENT ANALYSIS

Left-turn lanes are intended to reduce multivehicle accidents on intersection approaches, particularly those accidents related to left-turning traffic. Therefore, accident rates were computed for each approach category for the following types of multivehicle accidents: (a) right-angle, (b) rear-end, (c) sideswipe (same direction), (d) sideswipe (opposite direciton), (e) head-on, (f) left-turn, and (g) right-turn. The volumes used to compute each rate were the volumes of the turning movements involved in the particular type of accident. The turning-movement combinations involved in each accident category and the turning-movement volumes used to compute each accident rate are presented in Table 5.

For example, the turning-movement combinations involved in accidents defined as rear-end accidents were:

1. Two left-turn movements on the study approach (movements 1 and 1);
2. A left-turn and a through movement on the study approach (movements 1 and 2);
3. A left-turn and a right-turn movement on the study approach (movements 1 and 3);
4. Two through movements on the study approach (movements 2 and 2);
5. A through and a right-turn movement on the study approach (movements 2 and 3); and
6. Two right-turn movements on the study approach (movements 3 and 3).

Therefore, the volume used to compute the rear-end accident rate was the sum of the left-turn, through, and right-turn volumes on the study approach.

However, only three movement combinations were defined for the left-turn accident:

1. A left-turn movement on the study approach and a left-turn movement on the opposing approach (movements 1 and 7);
2. A left-turn movement on the study approach and a through movement on the opposing approach (movements 1 and 8); and
3. A left-turn movement on the study approach and a right-turn movement on the opposing approach (movements 1 and 9).

Therefore, the volume used to compute the left-turn accident rate was the sum of the left-turn volume on the study approach and the total volume on the opposing approach.

The accident rates were computed using the accidents that occurred on the study approaches during 1984, 1985, and 1986. For each of the four approach categories, each accident rate was computed using the total number of accidents and the total turning-movement volumes on all study approaches in the approach category.

The percent reductions in the accidents associated with the presence of left-turn lanes were computed as follows:

$$R = [(B - A)/B] \cdot 100\% \tag{1}$$

where

R = percent reduction in accidents (%),
B = number of accidents on approaches without left-turn lanes, and
A = number of accidents on approaches with left-turn lanes.

Percent reductions were computed for the signalized and the uncontrolled approach categories. The statistical significance of the percent reductions was checked using the chi-squared test. In Equation 1, the numbers of accidents on approaches with left-turn lanes were computed by applying the accident rates for these approaches to the turning-movement volumes for the approaches without left-turn lanes. Thus the number of accidents, with and without left-turn lanes, were for the same volumes.

TABLE 3 NUMBER OF STUDY SITES IN EACH APPROACH CATEGORY

Approach Category	Number of Study Sites
Signalized[a] without left-turn lane	11
Signalized[a] with left-turn lane	10
Uncontrolled without left-turn lane	12
Uncontrolled with left-turn lane	13
Total	46

[a]Without protected left-turn phases.

TABLE 4 SPEED LIMITS ON STUDY SITES

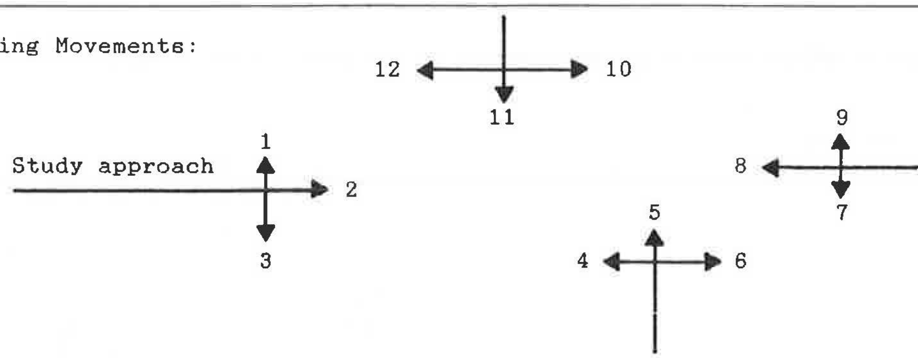
	Signalized Approach[a]		Uncontrolled Approach	
Speed Limit	Without LT Lane	With LT Lane	Without LT Lane	With LT Lane
30	5	3	0	0
35	6	7	6	4
40	0	0	1	2
45	0	0	5	7
Total Study Sites	11	10	12	13

[a]Without protected left-turn phases.

TABLE 5 ACCIDENT TURNING MOVEMENTS[a]

Accident Type	Turning Movement Combinations Involved	Turning Movement Volumes Used to Compute Accident Rates
right-angle	1-4,1-5,1-6,1-10,1-11, 2-4,2-5,2-6,2-10,2-11, 3-10,3-11	1,2,3,4,5,6,10,11
rearend	1-1,1-2,1-3,2-2,2-3,3-3	1,2,3
sideswipe (same direction)	1-1,1-2,1-3,2-2,2-3,3-3	1,2,3
sideswipe (opposite direction)	2-8,2-9,3-8,3-9	2,3,8,9
head-on	2-8,2-9,3-8,3-9	2,3,8,9
left-turn	1-7,1-8,1-9	1,7,8,9
right-turn	3-4,3-5,3-6	3,4,5,6

[a]Turning Movements:



FINDINGS

The accident rates computed for each category are given in Table 6, as well as the number of accidents and turning-movement AADTs used to compute the accident rates. The percent reductions in the accident rates associated with the presence of left-turn lanes are presented in Table 7.

The presence of left turn lanes was never associated with statistically significant reductions in sideswipe (opposite direction), head-on, or right-turn accident rates. This finding was expected because these types of accidents seldom occurred on the study approaches. It is consistent with previous research, which has not reported any relationships between the occurrence of these types of accidents and the presence of left-turn lanes (*3,4,5,6,10*).

The presence of left-turn lanes on the signalized intersection approaches was not associated with any statistically significant change in the right-angle accident rate. However, the presence of left-turn lanes on the uncontrolled approaches was associated with a statistically significant 68 percent increase in the right-angle accident rate. This finding is consistent with the California study (*3*) cited previously, which also found a significant increase in right-angle accidents after left-turn lanes were installed at unsignalized intersections in urban areas. It should be noted, however, that the increase in the right-angle accident rate was determined through a comparison of approaches on four-lane undivided roadways without left-turn lanes and on approaches on four-lane divided roadways with left-turn lanes. The increase in the right-angle accident rate probably reflects the greater degree of difficulty cross-street drivers have determining adequate gaps to accept (when crossing a four-lane divided street with a 16-ft median), as well as the longer distances that cross-street drivers must travel. Therefore, although the installation of left-turn lanes on

TABLE 6 NUMBER ACCIDENTS, TURNING MOVEMENT AADTS, AND ACCIDENT RATES

Accident Type	Signalized Approach[a] Without LTL[b]	Signalized Approach[a] With LTL	Uncontrolled Approach Without LTL	Uncontrolled Approach With LTL
Number of Accidents:[c]				
right-angle	37	25	23	48
rearend	61	26	27	4
sideswipe (same dir.)	14	4	5	3
sideswipe (opp.dir)	2	0	0	0
head-on	0	0	0	0
left-turn	31	11	40	7
right-turn	0	1	0	0
Turning Movement AADTs:				
right-angle	128,550	138,550	95,443	118,620
rearend	94,720	99,287	82,350	103,900
sideswipe (same dir.)	94,720	99,287	82,350	103,900
sideswipe (opp. dir.)	178,870	187,440	154,700	187,380
head-on	178,870	187,440	154,700	187,380
left-turn	100,960	104,860	87,340	109,980
right-turn	28,510	31,712	12,730	13,810
Accident Rates (accidents/million entering vehicles):				
right-angle	.26	.16	.22	.37
rearend	.59	.24	.30	.035
sideswipe (same dir.)	.14	.037	.055	.026
sideswipe (opp. dir.)	.010	0	0	0
head-on	0	0	0	0
left-turn	.28	.096	.42	.058
right-angle	0	.029	0	0

[a]Without protected left-turn phases.

[b]LTL - left-turn lane.

[c]Number of accidents during three-year period.

uncontrolled intersection approaches on four-lane undivided roadways would be expected to increase right-angle accidents, their installation on uncontrolled intersection approaches on four-lane divided roadways would not necessarily be expected to increase right-angle accidents.

Left-turn lanes are intended to reduce conflicts between through and left-turning traffic. As expected, the presence of left-turn lanes on signalized and uncontrolled approaches was associated with statistically significant reductions in rear-end, sideswipe (same direction), and left-turn accident rates. This finding is consistent with the results of previous studies (*3,4,6*) and indicates that left-turn lanes are effective in reducing these types of accidents at intersections on urban four-lane roadways.

CONCLUSION

The results of this research are consistent with those of previous studies. Left-turn lanes are demonstrated to be effective in reducing rear-end, sideswipe (same direction), and left-turn accidents at intersections on urban four-lane roadways with DHVs between 600 and 1,800 vph and cross-traffic AADTs above 1,000 vpd. Contrary to accident experience reported on two-lane roadways (*5,11*), the results of this study show that opposing left-turn lanes on four-lane roadways do not increase left-turn accidents.

However, the results of this research also indicate that left-turn lanes on the uncontrolled approaches of intersections on urban four-lane undivided roadways increase right-angle acci-

TABLE 7 PERCENT REDUCTION IN NUMBERS OF ACCIDENTS

Accident Type	Signalized Approach[a] (%)	Uncontrolled Approach (%)
Right-angle	37	−68[b]
Rear-end	59[b]	88[b]
Sideswipe (same direction)	73[b]	52
Sideswipe (opposite direction)	100	0
Head-on	0	0
Left-turn	66[b]	86[b]
Right-turn	—[c]	0

NOTE: Percent reductios in numbers of accidents associated with the presence of left-turn lanes. Negative percent reductions indicate higher numbers of accidents when left-turn lanes are present.
[a]Without protected left-turn phases.
[b]Percent reduction is statistically significant at the 5 percent level of significance.
[c]Undefined percent reduction, because there were no accidents without left-turn lanes, but a non-zero number of accidents with left-turn lanes.

dents. Consequently, the trade-off between the reductions in rear-end, sideswipe, and left-turn accidents and the increase in right-angle accidents should be considered when evaluating the cost-effectiveness of installing left-turn lanes at these locations.

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REFERENCES

1. *Road Design Manual.* Nebraska Department of Roads, Lincoln, March 1984.
2. P. T. McCoy and M. S. Malone. *Criteria for Left-Turn Bays: A Guide for Selection of Divided and Undivided Sections on Urban Four-Lane Roadways.* Research Report TRP-02-18-87. Proj. RES 1 (0099) P409. Nebraska Department of Roads, Lincoln, Feb. 1988.
3. C. G. Hammer, Jr. Evaluation of Minor Improvements. In *Highway Research Record 286,* HRB, National Research Council, Washington, D.C., 1969, pp. 33–45.
4. T. J. Foody and W. C. Richardson. *Evaluation of Left-Turn Lanes as a Traffic Control Device.* Proj. 148210. Bureau of Traffic Control, Ohio Department of Transportation, Columbus, Nov. 1973.
5. N. A. David and J. R. Norman. *Motor Vehicle Accidents in Relation to Geometric and Traffic Features of Highway Intersections.* Vol. 2—Research Report FHWA-RD-76-129. FHWA, U.S. Department of Transportation, July 1975.
6. K. R. Agent. *Development of Warrants for Left-Turn Lanes.* Research Report 526. Bureau of Highways, Kentucky Department of Transportation, Lexington, July 6, 1979.
7. *1986 Continuous Traffic Count Data and Traffic Characteristics on Nebraska Streets and Highways.* Nebraska Department of Roads, Lincoln, April 1987.
8. R. M. Michaels. Two Simple Techniques for Determining the Significance of Accident-Reducing Measures. *Traffic Engineering,* Vol. 36, No. 12, Sept. 1966, pp. 45–48.
9. J. C. Laughland, L. E. Haefner, J. W. Hall, and D. R. Clough. *NCHRP Report 162: Methods for Evaluating Highway Safety Improvements.* TRB, National Research Council, Washington, D.C., 1975.
10. *Synthesis of Safety Research Related to Traffic Control and Roadway Elements.* Vol. 1, Chapt. 5—Intersections. FHWA-TS-82-232. FHWA, U.S. Department of Transportation, Dec. 1982.
11. P. T. McCoy, W. J. Hoppe, and D. V. Dvorak. Cost-Effectiveness Evaluation of Turning Lanes on Uncontrolled Approaches of Rural Intersections. Research Report TRP-02-15-84. Proj. HPR 79-3. Nebraska Department of Roads, Lincoln, Oct. 1984.

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Intersection, Diamond, and Three-Level Diamond Grade Separation Benefit-Cost Analysis Based on Delay Savings

BRUCE RYMER AND THOMAS URBANIK II

A method for determining when traffic flow should be grade separated would be an invaluable tool for the traffic engineer/planner. The results of this study facilitate choosing proposed grade separation improvements on the basis of an evaluation of the reduced delay benefits to the cost of a grade separation. This methodology can assist decision-makers in determining when grade separations are appropriate. The analysis is centered on the Federal Highway Administration's TRANSYT 7F model. An economic analysis that presents the benefit/cost methodology for ranking a grade separation project is included.

Transportation engineers and planners are often required to rank intersection-to-interchange improvement projects on the basis of a minimal amount of input. The objective of grade separation is to enhance total overall traffic movement, to rank traffic movement on one functional class of roadway over another, or to perform both of these functions. Grade separation increases operational efficiency and therefore improves the overall traffic movement at the junction of the roadways by increasing the amount of traffic the roadway junction can accommodate, lowering overall delay, and decreasing certain types of accidents.

No guidelines currently exist for warranting a grade separation at a roadway intersection. The possible operational improvement that the grade separation will have on the intersection has not been adequately evaluated. Typical measures of operational improvement are the delay savings and the increased capacity of the interchange versus the intersection. Delay can be used for a relative comparison of the improvement and also in an economic analysis by assigning a value to this delay time. This study was conducted to establish a procedure for evaluating grade-separation projects based on quantifying vehicle-delay improvement. Vehicle delay can then be used as one of the criteria for considering grade separation at an intersection.

A grade separation, prompted by the desire for a gain in operational efficiency, can be accomplished by many different types of interchanges. One set of through movements can be grade separated by a single-point urban interchange, a conventional diamond, a three-point diamond, or a split diamond. Two sets of through movements can be grade separated by a three-level diamond or a stacked diamond. There are many other interchange configurations, with varying levels of operational efficiency. The fully directional interchange serves as the upper limit in efficiency and cost. This analysis focuses on delay improvements gained by grade separation from a high-type intersection to a conventional diamond interchange to a three-level diamond interchange.

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STUDY PROCEDURE

The study was not an attempt to acquire data for estimating delay for every possible variety of intersection and interchange. Rather, its purpose was to identify major characteristics to permit comparison of one type of improvement with another.

A major portion of potential project benefits can be attributed to delay reductions. At an interchange, traffic consists of two components: the grade-separated vehicles and the vehicles that are operating at grade and passing through the signal system. Separate procedures are necessary for evaluating the at-grade and grade-separated portion of interchanges. This report explores the at-grade signalized portion of interchanges. For purposes of this study, grade-separated through volumes less than or equal to 1,800 vph/lane will contribute a negligible amount to the system delay.

After evaluating a variety of alternatives, the TRANSYT 7F computer model was selected for developing relationships among various at-grade configurations. TRANSYT 7F is capable of optimizing the signal controls at intersections, diamond interchanges, and three-level diamond interchanges.

Setting Input Variables

TRANSYT 7F is a macroscopic deterministic traffic model. The required input data for the TRANSYT 7F model (*1*) include geometrics, phasing, clearance intervals, saturation, and traffic volumes. There are an infinite number of combinations of these variables. The comparison presented here is for planning purposes and is intended to be as equitable as possible for evaluation of the operational upgrades from intersection to diamond interchange to three-level diamond interchange.

To simplify the evaluation, all of the at-grade intersections considered had separate left-turn and right-turn bays. Figures 1–3 show the geometric layout of the various types of at-grade signalized intersections. The saturation flow rate was estimated to be 1,700 vph for left turns and 1,750 vph for through and right-turning traffic. Right-turning traffic was phased with its corresponding through movement. Phasing at

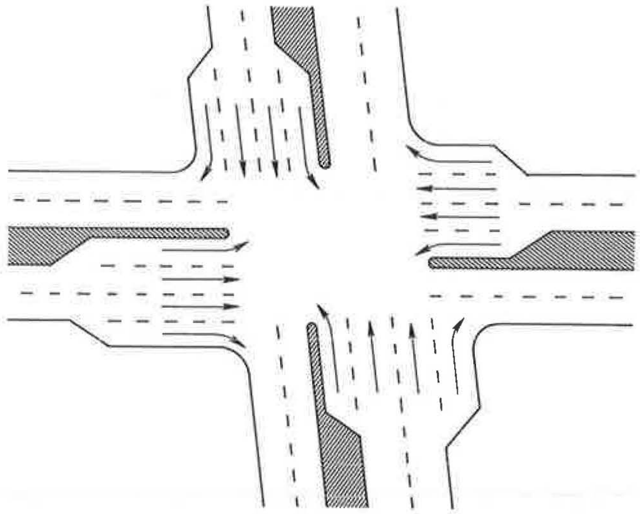

FIGURE 1 Intersection geometrics used in T7F simulation.

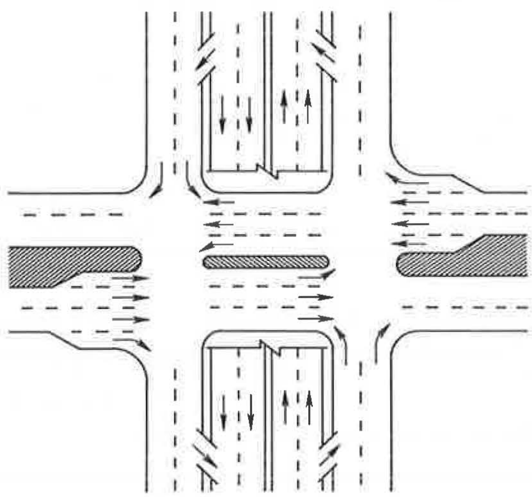

FIGURE 2 Diamond geometrics used in T7F simulation.

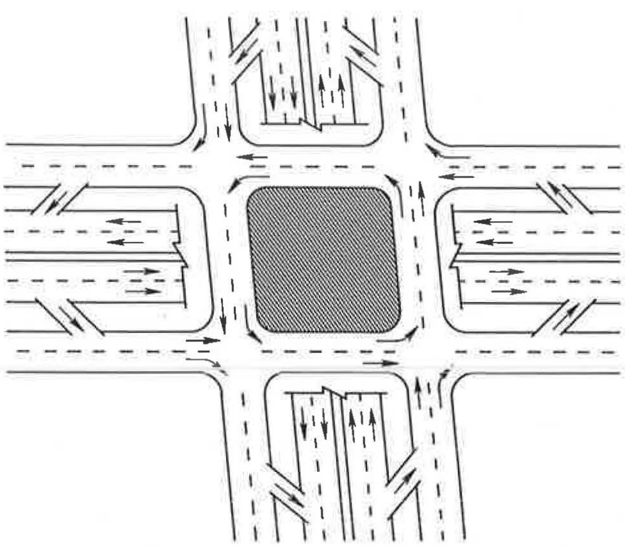

FIGURE 3 Three-level diamond geometrics used in T7F simulation.

the high-type intersection consisted of four phases with leading left turns. The diamond interchange operated on three phases, with an appropriate offset between the two intersections. The three-level diamond ran on a coordinated two-phase pattern.

A minimum cycle length of 30 seconds was used on the three-level diamond interchange, and a minimum cycle length of 40 seconds was used on the intersection and diamond interchange. A clearance interval of 3 seconds was used throughout the simulations. Another simplifying assumption was that the cross-road directional volume split was 1 to 1, or 50/50. Each right- and left-turn movement was light (10 percent) or heavy (20 percent) on each approach. This provided two scenarios on each configuration: light turning movements (right + left = 20 percent of through movement) and heavy turning movements (right + left = 40 percent of through movement).

STUDY RESULTS

Figure 4 presents the total system delay (stopped delay + approach delay) calculated by TRANSYT 7F at the intersection on the basis of hourly volume and turning movement percentages and the other assumptions made with the geometrics, phasing, and clearance intervals. The curves were obtained by starting with a low initial traffic volume and incrementally increasing the volume in each succeeding computer simulation until oversaturation occurred.

The plots in Figure 4 appear to approach a vertical asymptote, much like the underlying TRANSYT 7F delay function. When the total of all four approaches is 6,000 vph, average delay per vehicle is approximately 60 seconds, making the overall system delay 100 vehicle hours.

A diamond, in essence, removes two through movements from the at-grade intersection and replaces one signal with two coordinated signals. When interpreting the delay calculations of TRANSYT 7F, the overall delay of the two-signal diamond interchange system will be compared with the overall delay of the one-signal at-grade intersection system. The same methodology is used when comparing the system delay of the four-signal, three-level diamond with the two-signal diamond and the one-signal at-grade intersection. As a result, the system delay on the ordinate represents a summation of all of the intersection(s) delay within the system. This step was taken to provide an equitable operational comparison of the different grade separation options.

In Figure 5, which is a plot of the diamond interchange simulation results, the same asymptotic relationship is evident. The abscissa is marked with three different scales; the top scale reflects the total number of vehicles in the interchange system. From this total, two of the through movements have been removed by the grade separation, leaving the accompanying turning movements to negotiate the at-grade signals. The bottom two scales reflect the actual number of vehicles entering the at-grade intersections. The two-diamond interchange curves are very similar in shape to the at-grade intersection curves once the abscissa is rescaled or compressed. The three-phase, coordinated signals of the at-grade portion of the diamond interchange approach 100 vehicle hours of delay when total at-grade entering volume is 6,000 vph.

In Figures 1–3, the initial assumptions is that each movement would have at least one lane to travel through the at-

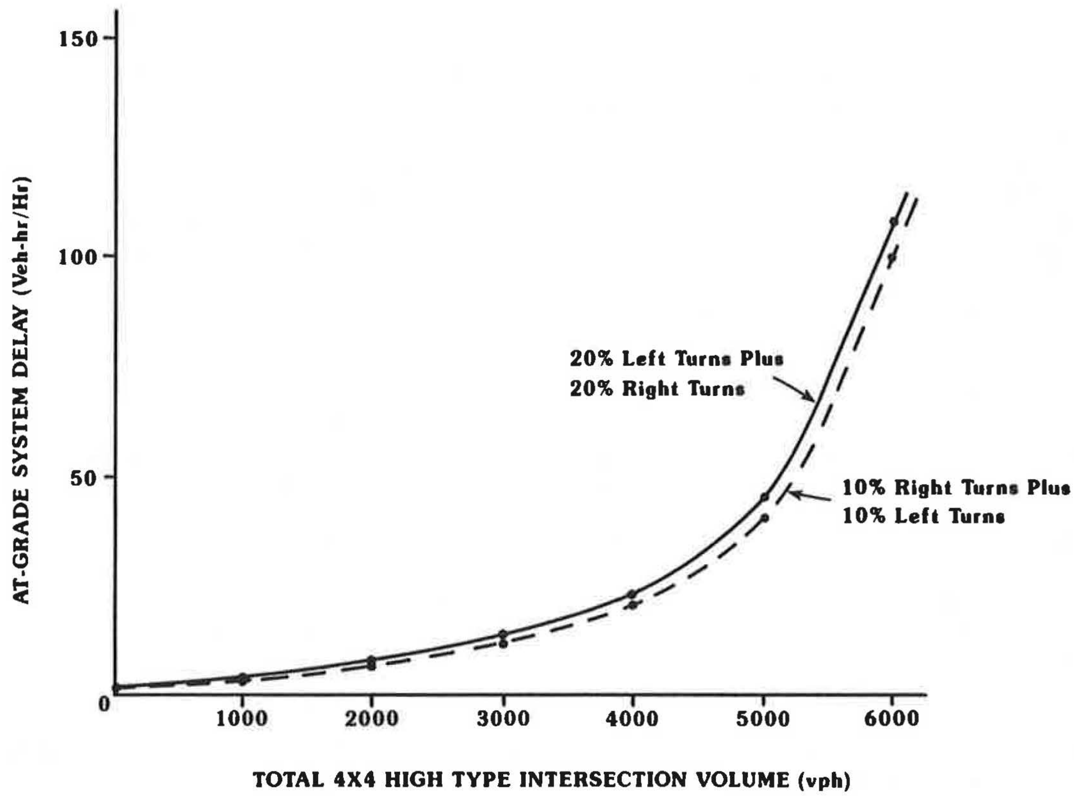

FIGURE 4 System delay for high-type interchanges.

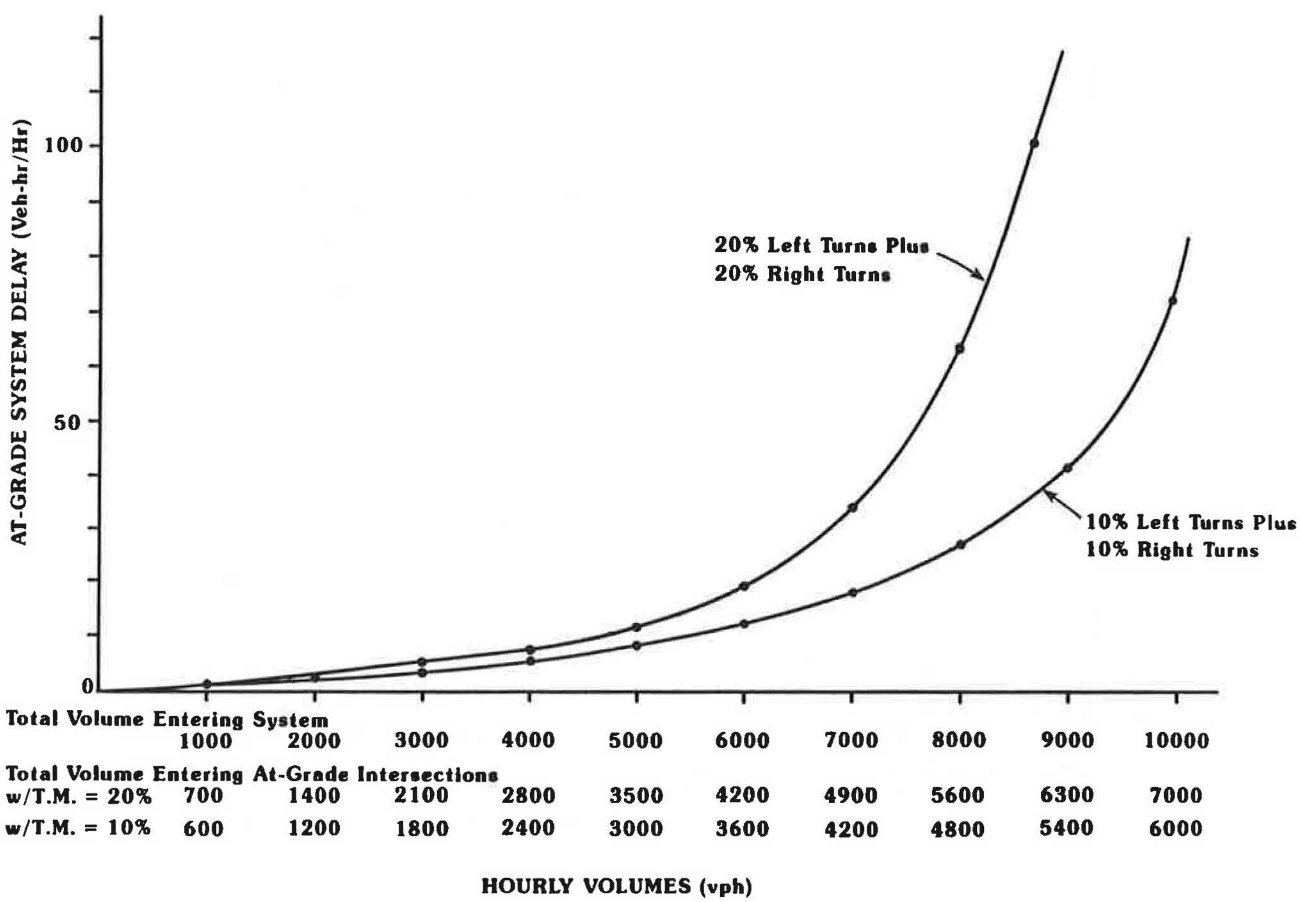

FIGURE 5 System delay for diamond interchange.

grade portion of the system. Both frontage road traffic and u-turning traffic were negated to provide a consistent comparison throughout. This is an appropriate assumption for an arterial-arterial interchange, where no frontage roads would exist. Also, frontage road and u-turning volumes are rarely known at the planning stage.

Figure 6 demonstrates that the same asymptotic relationship exists in the three-level diamond interchange. The top abscissa scale is the total number of automobiles in the three-level diamond system. Four through movements have been grade separated or removed from the at-grade intersection. The remaining turning movements must negotiate the at-grade signals. The two lower abscissa scales reflect the residual of the through movements and are a combination of the right turns plus the left turns. With two through lanes in each direction (Figure 3), the total system capacity for this roadway junction is 4 directions × 2 lanes/direction × 2,000 vph/lane = 16,000 vph. Therefore, the maximum volume that can enter the 4 × 4 system is 16,000 vph.

A three-level diamond interchange would probably have three or more lanes on each at-grade approach. Figure 3 represents the geometrics assumed for this analysis only. Each turning movement had a separate lane while it negotiated a signal controlled intersection. Note that with 40 percent (left + right) of the grade separated through movements exiting, the exit ramp is approaching its capacity [(0.40 × 2 lanes × 2,000 vph/lane) = 1,600 vph]. This factor will act as a constraint on the at-grade capacity of the system. Once the two lower abscissa scales are compressed, a delay relationship very similar to the intersection and diamond relationships is formed. The at-grade, signalized portion of the three-level diamond reaches its capacity at 6,000 vph entering volume when the system is approaching its maximum entering volume of 16,000 vph and there are 20 percent left turns and 20 percent right turns on all approaches.

A family of curves has been developed for the three different geometric scenarios. Figure 7 shows a relative comparison of system delay with total vehicles in the system and a comparison of the amount of hourly traffic that each system can accommodate. Each roadway junction type has an upper and lower limit that is actually set by the severity of the left-turning movement demand and the number of through lanes on each roadway.

Figure 7 also plots the range of intersection delays within each system. This analysis neglects any delay on the free-moving through lanes. By definition, the delay on the grade-separated portion should also be included with the overall system delay if the free-moving through lanes become congested. However, with at least 20 percent or more of the traffic (10 percent right turns and 10 percent left turns) negotiating the at-grade portion of the interchange, this leaves 1,800 vph/lane on the through lanes on the three-level diamond. The grade-separated through lane delay has been omitted and could best be computed by a speed/density analysis. The grade-

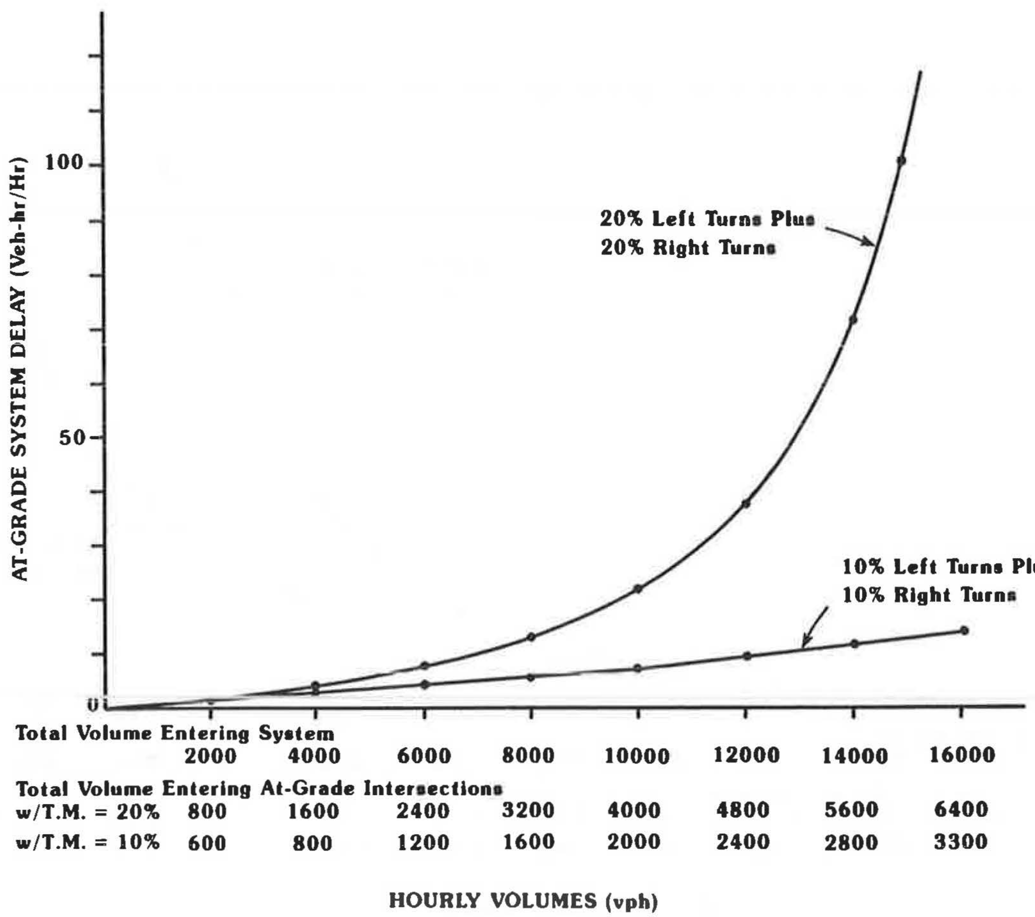

FIGURE 6 System delay for three-level diamond interchange.

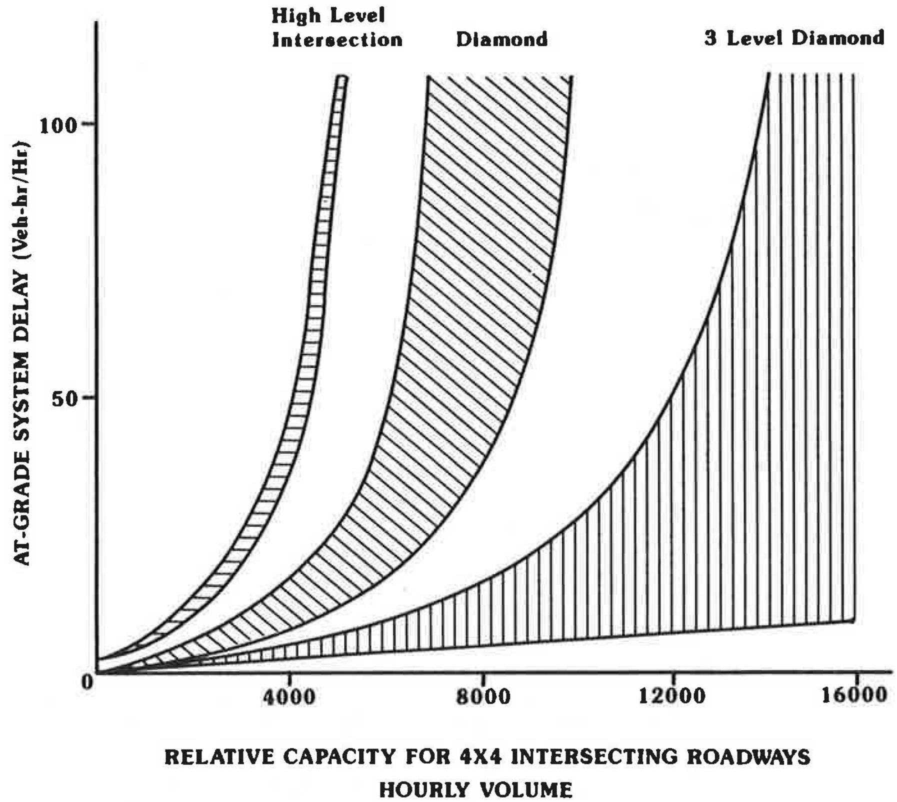

FIGURE 7 At-grade delay versus system capacity.

separated lanes on the diamond interchange are operating at 1,000 vph/lane when the at-grade delay reaches 60 seconds per vehicle, incurring a small amount of delay on the grade-separated through lanes and adding very little delay to the overall diamond system. There are additional volume benefits unaccounted for on the through lanes of the diamond interchange.

The underlying asymptotic delay relationship, as demonstrated in Figures 4–6, follows the same general shape for each at-grade signalized portion of the three types of roadway junctions (with these assumed geometrics). The approximate capacity of all three at-grade intersection systems is 6,000 vph. At the transitions from an at-grade intersection to a diamond interchange to a three-level diamond interchange, it appears that any efficiencies gained by losing a phase and removing two through volumes are counterbalanced by increasing the number of coordinated traffic signals. Therefore, for planning purposes, a single delay equation can be developed for evaluating the delay incurred on the signalized, at-grade portion of these three types of roadway junctions.

DEVELOPMENT OF A DELAY EQUATION

A delay relationship based on assumed geometrics and an hourly volume has now been established. The delay calculation routine can be greatly shortened by fitting an equation to the relationship and using hourly traffic as the only independent variable for calculating delay. This equation can then be used in an hour-by-hour, day-by-day, year-by-year economic planning analysis for evaluating a grade separation under the assumed geometrics.

The similarity among the intersection, diamond, and three-level diamond at-grade delay curves can be used to an advantage. This similarity in shape means that direct comparisons can be made from intersection to diamond, diamond to three-level diamond, and intersection to three-level diamond. Therefore, any amount of traffic removed from the at-grade portion of the intersection becomes a benefit.

By using the SAS curve fitting routine, an equation was derived for the observed delay relationship (*2*). An r^2 of 0.92 was obtained. For a 4 × 4 high-type intersection (four through lanes by four through lanes), the at-grade delay equation is:

$$\text{Delay } 4 \times 4 = 1.1778\ e^{v(.00072452)}$$

where delay is in vehicle hours per hour and v = total volume entering at-grade intersection (veh/hr).

By using the same procedure, a similar equation may be developed for a 6 × 6 high-type intersection. Only an additional through lane has been added to the geometrics; all other variables remain the same.

$$\text{Delay } 6 \times 6 = 1.2662\ e^{v(.00056726)}$$

Delay and v have been defined previously.

In Figure 8, the 6 × 6 delay function yields a higher amount of capacity. The extra capacity comes from the additional through lanes. The delay also goes up in a corresponding manner. An upper limit was placed on the curves. It is rec-

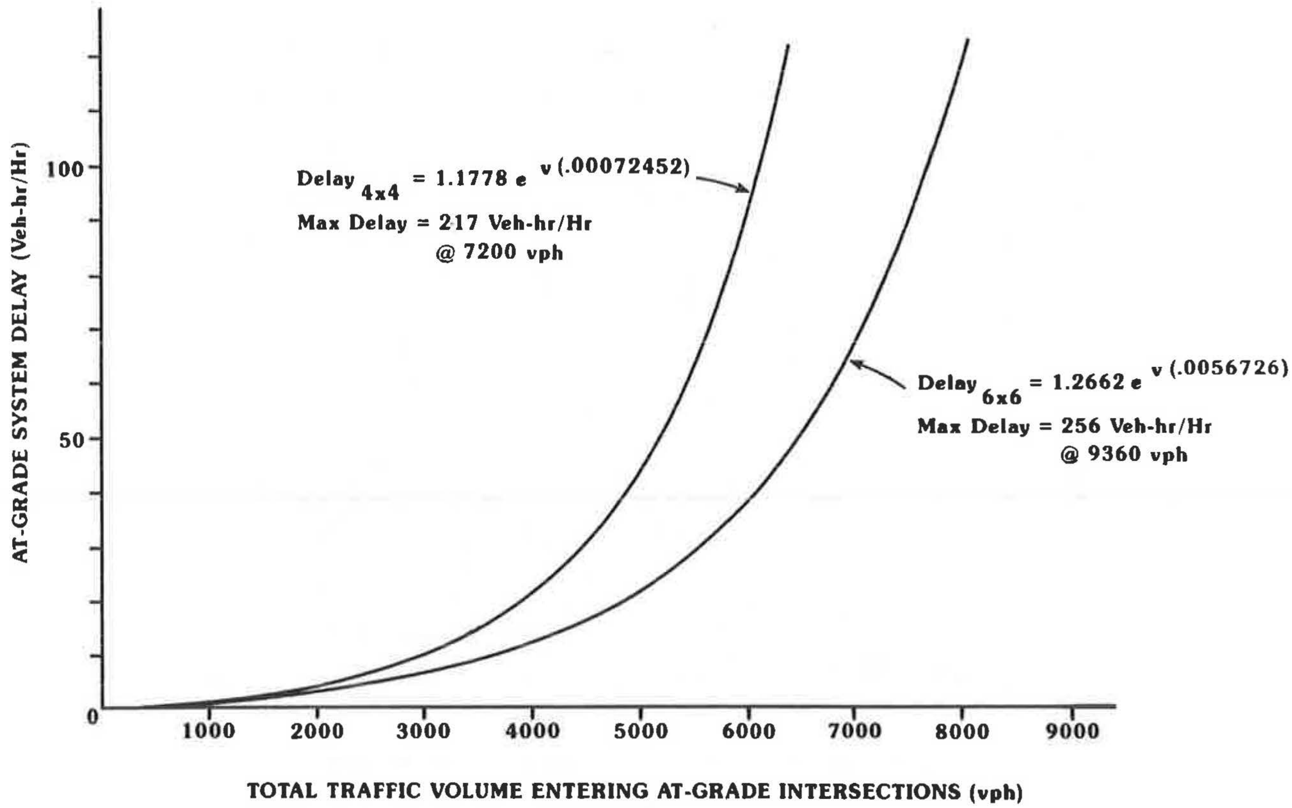

FIGURE 8 Derived delay equations.

ommended that the derived delay relationships be used only for projected demands that are no greater than 20 percent in excess of capacity because it is likely that traffic will divert to other routes. The following economic analysis limits delay when the capacity reaches 120 percent of the at-grade capacity.

ECONOMIC ANALYSIS

The derived equation(s) can be used in an economic analysis to determine if the benefits to the motorists of reduced delay will offset the cost of a grade-separated structure. The procedure is to take an average daily traffic (ADT) volume and an assumed hourly distribution of vehicles and calculate the delay using the derived delay equation. The delay is then summed over the year. A monetary value is assigned to the delay time and a delay cost calculated. The ADT is increased to reflect an average yearly growth rate, and the process is repeated. A net present worth can then be computed and a relative comparison made.

Grade separations cost somewhere between \$3 and \$5+ million, depending on site-specific conditions. If the public's delay reduction over the project's life is equal to or exceeds the construction cost of a grade separation, then the grade separation is warranted.

The economic evaluation assumed a Texas urban and rural traffic distribution developed by Urbanik (*3*). These specific distributions were obtained from a previous study of urban and rural facilities, and the *k* factors are 7.63 percent and 8.78 percent respectively. For purposes of this example, occupancy of each automobile was set at 1.25 persons. A value of \$7.80 per vehicle-hour was allotted for the delay time. The value of commercial truck time was estimated as \$19.00 per vehicle-hour. These values reflect the value of time to the motor vehicle occupants and associated vehicle operation costs (*4*). Yearly delay was based on 250 working days. A net present worth approach with 5 percent interest rate and a 20-year project life was used to assess the current economic value of delay. Truck traffic was assumed to be 10 percent. Traffic growth was assumed to be 2 percent per year during the 20-year project life.

Oversaturated conditions in any signal system will yield extremely high delay numbers. For planning purposes, a maximum saturation ratio of 1.2 was arbitrarily designated. Therefore if the assumed capacity of a junction were 6,000 vph, the maximum capacity that could pass through the junction would be 6,000 vph $\times$ 1.2 = 7,200 vph. This limits the amount of benefits that a planner can take by putting a maximum upper limit on the hourly volume. No excess volume is carried over into the next hour. It is believed that this is a more conservative procedure, and no undue delay credit is taken.

The following tables were generated with the derived delay relationships. Tables 1 and 2 apply only to high-level, 4 $\times$ 4, and 6 $\times$ 6 roadway junctions. Any combination of grade separations may be evaluated, for example, intersection to diamond, intersection to three-level diamond, or diamond to three-level diamond. These comparisons can all be made because the benefits are a function of the volume of traffic removed from the at-grade signalized portion of the interchange only.

Tables 1 and 2 represent the total delay costs to the motoring public. To determine if grade separation is warranted on the basis of a savings of delay, the existing at-grade ADT must be known. The benefits are found by determining the amount of traffic removed from the at-grade volume and taking the difference between the delay costs of the existing ADT and the remaining at-grade ADT. The following three examples illustrate this procedure.

- Urban upgrade from 4 × 4 high-type intersection to diamond interchange.

  – Known: Existing at-grade volume = 50,000 ADT
  Will remove 20,000 ADT from intersection
  Remaining at-grade ADT = 30,000
  – Net present worth of delay reduction benefits (millions) = \$7.536 − \$3.497 = \$4.039.

A saving of \$4,039,000.00 in delay to the motoring public is achieved over a 20-year period by building a diamond interchange to replace the intersection. The delay saving benefit for this example is roughly equivalent to the cost of building a diamond interchange.

- Rural upgrade from 4 × 4 high-type intersection to a three-level diamond interchange.

  – Known: Existing at-grade volume = 60,000 ADT
  Will remove 40,000 ADT from at-grade ADT
  Remaining at-grade ADT = 20,000
  – Net present worth of delay reduction benefits (millions) = \$19.874 − \$2.220 = \$17.654.

A saving of \$17,654,000.00 in delay to the motoring public is achieved over a 20-year period by replacing the intersection with a three-level diamond interchange. The delay reduction benefits for this example exceed the cost of building a three-level diamond interchange.

- Urban upgrade from a diamond interchange to a three-level diamond interchange (on a 4 × 4 roadway junction).

  – Known: Existing at-grade volume = 60,000 ADT
  Will remove 20,000 ADT from at-grade ADT
  Remaining at-grade ADT = 40,000
  – Net present worth of delay reduction benefits (millions) = \$17.826 − \$5.831 = \$11.995.

A saving of \$11,995,000 in delay to the motoring public is achieved by building a three-level diamond interchange to replace the diamond interchange. This delay reduction benefit exceeds the cost of building a three-level diamond interchange.

When a diamond is upgraded to a three-level diamond, the number of through lanes on the at-grade portion of the roadway intersection determines which of the two tables to select. The variables that have an impact on the net present worth calculations are value of delay time and operating costs, occupancy of the vehicles, interest rate, ADT, hourly distribution of ADT, yearly growth rate of ADT, project life, and percentage of commercial trucks. All of these variables are used with the delay equation(s) and can easily be incorporated into a computer spreadsheet program.

TABLE 1 NET PRESENT WORTH DELAY EVALUATION

	Total Delay Costs, 4 × 4 High-Type Intersections(s) (\$ × 10^6)	
Average At-Grade Daily Traffic	Rural	Urban
10,000	1.421	1.414
20,000	2.220	2.178
30,000	3.642	3.497
40,000	6.232	5.831
50,000	11.051	7.536
60,000	19.874	17.826
70,000	33.728	30.542
80,000	51.724	45.977

TABLE 2 NET PRESENT WORTH DELAY EVALUATION

	Total Delay Costs 6 × 6 High-Type Intersections(s) (\$ × 10^6)	
Average At-Grade Daily Traffic	Rural	Urban
10,000	1.396	1.392
20,000	1.954	1.930
30,000	2.822	2.749
40,000	4.194	4.014
50,000	6.390	4.554
60,000	9.957	9.135
70,000	15.824	14.181
80,000	24.884	22.386

CONCLUSIONS

For planning purposes, the operational efficiency of a given geometric intersection and its corresponding grade-separated improvements can be quantified by a single delay equation. This equation may be used for estimating the operational effectiveness of a grade separation project for use in a benefit/cost analysis.

REFERENCES

1. *TRANSYT-7F User's Manual (Release 5)*. Transportation Research Center, University of Florida, Gainesville, July 1987.
2. *SAS User's Guide: Basics and Statistics*. SAS Institute, Inc., Cary, N.C., 1985.
3. T. Urbanik II. *Speed/Volume Relationships on Texas Highways*. Texas Transportation Institute, Texas A&M University, College Station, April 1984.
4. M. K. Chui and W. F. McFarland. *The Value of Travel Time: New Estimates Developed Using a Speed-Choice Model*. Texas Transportation Institute, Texas A&M University, College Station, May 1986.

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Accident Comparison of Raised Median and Two-Way Left-Turn Lane Median Treatments

CHRISTOPHER A. SQUIRES AND PETER S. PARSONSON

It is accepted that the installation of a median will reduce accident occurrence along a previously undivided road. This report provides an accident comparison of raised medians and continuous two-way left-turn lanes used as median treatments on four- and six-lane roads. A statistical comparison of accident rates for the two section types and regression equations to model expected accident experience for each section were developed. Four- and six-lane roadway study sections in Georgia were analyzed separately. The accident rate of raised medians was found to be lower than the rate of two-way left-turn lanes for both four- and six-lane roadway sections. Regression equations were developed for raised median and two-way left-turn lane sections, four- and six-lane sections, total and midblock accidents, and accidents per million vehicle miles and accidents per mile per year. Tables of expected accident rate values were developed from the regression equations. On the basis of expected total accidents per million vehicle miles the tables indicated that for four-lane sections, raised medians had a lower accident rate over the range of data studied. Results from six-lane sections were mixed. The regression equations indicated that raised medians would have lower accident rates for most conditions. However, two-way left-turn lanes had a lower accident rate where few concentrated areas of turns, such as signalized intersections and unsignalized approaches, existed.

This study was intended to provide a basis of comparison between two median treatment types frequently used on arterial roads. Both raised medians and continuous two-way left-turn lanes (TWLTLs) are often used on high-volume four- and six-lane roads. Implementing either type of median treatment will reduce the number of accidents experienced on an undivided road that has the same number of through lanes. This study compares the relative safety of these two median treatments.

A TWLTL is a lane in the center of a road that is dedicated to left turn movements by both directions of traffic. TWLTLs provide excellent service to the land adjoining the roadway by offering an area for deceleration and stopping before a left turn from the road. As a result, TWLTLs reduce the frequency and severity of rear-end collisions and allow drivers additional perception time in making left turns. These lanes are also used by vehicles turning from cross streets and driveways onto the arterial. TWLTLs allow more flexible use of the entire roadway because, for example, temporary work zones can easily be established.

Raised medians facilitate the movement of through traffic along a roadway. Turning movements are concentrated at relatively few points, where they can better be accommodated. This concentration reduces both the total number of conflict points for vehicles turning onto or off of the roadway and the number of driveway maneuvers allowed. Raised medians may also be used for their aesthetic qualities.

The purpose of the study was to provide a quantitative basis for determining whether raised medians or TWLTLs would have a lower accident rate for a given situation. As many study sites as possible throughout the State of Georgia were identified for the study. The study was undertaken in conjunction with the Georgia Institute of Technology School of Civil Engineering. The Georgia Department of Transportation (GaDOT) provided information that could not be readily collected in the field.

The study was limited to roads with either four or six travel lanes. Data for these two types of sites were analyzed independently. Accident data were obtained for fatal, injury, and property-damage-only accidents occurring along each section. Full data analysis was performed for both total and midblock accident occurrence.

PREVIOUS RESEARCH

A Federal Highway Administration (FHWA) report by Azzeh et al. (*1*) advocated the use of either TWLTLs or raised medians to reduce accidents and delay caused by an undivided roadway. When accident reductions for raised medians and those for TWLTLs are compared, it appears that TWLTLs would be safer for low and moderate levels of development (measured as having fewer than 60 commercial driveways per mile). Raised medians would be considered safer for high levels of development. The relative safety of the two median types remained constant for all average daily traffic (ADT) levels studied (fewer than 5,000, 5,000 to 15,000, and more than 15,000 vehicles per day).

The same report included general comments about each median type. A TWLTL is attractive because it keeps left turning vehicles from through traffic while providing maximum left-turn access. A TWLTL should be used, in lieu of an undivided road, when there are frequent rear-end conflicts caused by left-turning vehicles and on moderate- to high-volume highways that have few cross streets and many driveways.

Raised medians reduce the number of conflicting vehicle maneuvers at driveways. However, there will be some increase in other conflicts because of indirect left-turn maneuvers when

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drivers move vehicles into minor driveways. Raised medians are used on major arterials with a moderate to high number of driveways per mile. A cross-street spacing of one-half mile or greater is desired.

Perhaps the most often quoted report is Parker's 1983 Virginia study (*2*). Regression equations were produced for the accident occurrences of raised median, traversable median (including TWLTL), and undivided highway sections. General guidelines were also presented for using the various median types. The report indicated that if stopping sight distance is less than AASHTO standards, a TWLTL should not be used. A raised median should not be used where speeds exceed 45 mph unless the curb face is mountable. Raised medians are desirable when access points are limited to major intersections, there are large pedestrian volumes, or a grid pattern permits circuitous flow of traffic without disrupting residential traffic. Additionally, TWLTLs should not be used when access is required on only one side of the street.

Harwood and St. John (*3*) listed characteristics and appropriate implementations of raised medians and TWLTLs. Raised medians discourage new strip development, whereas TWLTLs may encourage such development. However, raised median sections increase travel time for drivers who wish to turn left if median openings are not provided. They also reduce operational flexibility, such as allowing for emergency vehicle operations, lane closures, and work zones. Raised medians are best suited to major arterials with a high volume of through traffic and limited access points and are also appropriate when a highway agency makes a conscious choice to favor the traffic movement function through an area.

Two-way left-turn lanes generally reduce delay to left-turning vehicles and enhance operational flexibility. However, they do not provide any refuge area for pedestrians. Inappropriate use of TWLTLs by drivers may cause vehicular conflicts. Harwood and St. John indicated that TWLTLs should be used when there are low to moderate volumes of through traffic.

DATA COLLECTION

Site Selection

Roadway sections that had a continuous TWLTL or a continuous curb-and-gutter raised median were considered for the study. Other than the following restrictions, there were no predefined limits on the range of data to be expected from these sites. The parameters used for selection were

- ADT at least 9,500 vehicles per day,
- Location on a state route,
- A constant four- or six-through lane cross section, and
- Free access to the road at grade (uncontrolled access).

To ensure that the study incorporated only urban type sections, ADT values were kept above 9,500 and there was free access to the road. Sites located on a state route enabled collection of accident data that were uniformly reported.

Some of the sites chosen were suggested by Vargas (*4*). The remaining sites were determined through computer searches of the GaDOT road inventories. These inventories provided the preliminary information needed to identify potential sites, including number of through lanes, ADT, access control, type of median treatment, and lane widths.

In the Atlanta metropolitan area, 16 suitable TWLTL sites were identified; however, only 4 raised median sites were located. Several potential sites were eliminated because of depressed or flush medians along portions of the site length. Broadening the search area to encompass the entire state resulted in the addition of 15 raised median sites and 4 TWLTL sites.

The 20 of the TWLTL sites have a total length of 74.86 miles. The 19 raised median sites have a total length of 47.60 miles. Each site was subdivided into sections wherever possible. Sections for analysis were established for lengths greater than 0.75 miles to ensure that the data for all sections would be representative of actual conditions. Short analysis sections would tend to yield highly fluctuating data. The researchers also wanted to define the analysis sections so that reported ADT values would remain constant through the section. Analyzing sections with a relatively constant ADT was a secondary consideration in establishing the analysis sections. Table 1 provides a summary of basic site and section characteristics.

Data Collected

Data for the analysis sections were obtained from three sources:

- Road inventories from GaDOT Planning Data Services,
- Field collection, and
- Accident data from GaDOT Traffic and Safety Division.

TABLE 1 SITE AND SECTION CHARACTERISTICS

	TWLTL	Raised Medians
Number of Sites		
4 Lane sections	17	13
6 Lane sections	3	6
Totals	20	19
Number of Sections		
4 Lane sections	42	15
6 Lane sections	8	17
Totals	50	32
Site Lengths		
4 Lane sections	62.48	24.68
6 Lane sections	12.38	22.92
Totals	74.68	47.60
Million Vehicle Miles per year		
4 Lane sections	691.48	228.25
6 Lane sections	149.05	264.42
Totals	840.53	492.68

Road inventories provided ADT and mileage points reported to the nearest one 1/100th of a mile and were used to further subdivide sites into analysis sections. Accident data were obtained in summary form, which indicated fatal accidents, injury accidents, and total accidents for each analysis section. The data were provided for the total length of the analysis section and for midlock portions of the section. Accident data were available for 1984, 1985, and 1986 on all but two sites; for each of these, data were available for only two years. Data collected in the field for each section consisted of the number of driveways, signalized intersections, unsignalized approaches (streets), and, for raised median sections, median openings other than at signalized intersections.

Data Summary

The accident data obtained from GaDOT were used to calculate accidents per million vehicle miles (MVM) and accidents per mile per year. The number of accidents per million vehicle miles was believed to be the best indicator for comparison between median types because of the great variation of ADT present in the sites analyzed. However, the numbers of accidents per mile per year were calculated for use in comparing this study with other research.

Table 2 summarizes the accident calculations for injury accidents, fatality accidents, and total accidents. No determination was made of the number of injuries or fatalities associated with each section because these numbers are dependent on variables outside the scope of this research.

The summary rates presented in Table 2 were not obtained by averaging the accident rates for individual sections, which would have created an error because the site lengths and ADTs vary. Instead, accidents per MVM were obtained for each section type by summing the number of accidents per year and dividing that number by the total number of million vehicle miles traveled per year. Accidents per mile per year were found by dividing the total number of accidents per year by the sum of the analysis section lengths for each cross-section type.

TABLE 2 SUMMARY OF ACCIDENT DATA

	Total Accidents			Midblock Accidents		
	TWLTL	RM	% Diff	TWLTL	RM	% Diff
Accidents / MVM						
4 Lane sections	8.99	7.67	-14.7%	3.50	1.34	-61.7%
6 Lane sections	10.82	8.15	-24.7%	4.19	1.92	-54.2%
Accidents / Mi / Yr						
4 Lane sections	99.45	70.91	-28.7%	38.78	12.39	-68.1%
6 Lane sections	130.26	94.07	-27.8%	50.46	22.13	-56.1%
Injury Accidents / MVM						
4 Lane sections	2.00	1.70	-15.0%	0.81	0.32	-60.5%
6 Lane sections	3.61	1.90	-47.4%	1.09	0.43	-60.6%
Injury Accidents / Mi / Yr						
4 Lane sections	22.14	15.76	-28.8%	8.91	2.92	-67.2%
6 Lane sections	43.46	21.87	-49.7%	13.14	4.93	-62.5%
Fatal Accidents / MVM						
4 Lane sections	0.01	0.03	-66.7%	0.01	0.01	0.0
6 Lane sections	0.03	0.03	0.0	0.02	0.01	-50.0%
Fatal Accidents / Mi / Yr						
4 Lane sections	0.14	0.29	-51.7%	0.06	0.08	-25.0%
6 Lane sections	0.38	0.39	-2.6%	0.30	0.10	-66.7%

The data obtained from road inventories and field collection were converted to a per mile basis (signals per mile, for example). Table 3 summarizes the site data.

Data were plotted in the form of scatter diagrams. Each of the independent variables was plotted against accidents per MVM and against accidents per mile per year for each of the section types so that the data could be checked for outliers. The relevant scatter diagrams plot total accidents per MVM against the independent variables found to be significant in the regression analysis. Figures 1–4 show that none of the data points used in developing the regression equations (for accidents per MVM) appears to be an outlier.

DATA ANALYSIS AND RESULTS

Comparison of Accident Rates

The accident data were tested to determine the error level at which there was a significant difference between two-way left-turn lane and raised median accident rates. Table 4 lists the alpha error at which the two accident rates were found to be significantly different. The figures indicate the alpha error associated with the conclusion that raised medians are safer than TWLTLs. The last two columns also indicate whether the two rates are statistically significant at different alpha values of 0.10 and 0.05.

The calculations were based on a one-sided student's *t*-distribution. The assumption that $\mu_T = \mu_{RM}$ (mean of TWLTL accident rates equals mean of raised median accident rates) was tested, with the alternate hypothesis being that $\mu_T > \mu_{RM}$ (mean of TWLTL accident rates is greater than mean of raised median accident rates). With the initial hypothesis, any difference in accident rates is due to chance alone. The alternate hypothesis, for which the alpha error has been calculated, states that the difference in rates is not attributable to chance alone and that the mean of TWLTL accident rates is higher than the accident rate for raised median cross sections.

There is never certainty, statistically speaking, that rates of finite sample sizes are definitely different. However, some

TABLE 3 SUMMARY OF SITE DATA

		TWLTL		RAISED MEDIAN	
		6 Lane	4 Lane	6 Lane	4 Lane
ADT	Minimum	23712	9500	20360	10180
	Maximum	47685	52240	47180	59070
	Mean	32769	30542	31994	24605
	Stnd Dev.	8308	9881	7969	10866
Drives/mi	Minimum	36.90	10.08	18.18	5.00
	Maximum	144.34	103.53	106.40	76.74
	Mean	71.29	50.16	45.62	33.75
	Stnd Dev.	33.96	21.67	22.84	19.30
Signals/mi	Minimum	1.07	0.00	0.00	0.00
	Maximum	5.66	7.06	4.76	8.14
	Mean	2.63	2.10	2.25	2.26
	Stnd. Dev.	1.54	1.56	1.08	1.97
Openings/mi	Minimum	---[1]	---	0.00	1.14
	Maximum	---	---	7.43	13.79
	Mean	---	---	2.89	3.98
	Stnd. Dev.	---	---	1.91	3.37

[1] Openings per mile not applicable to two-way left-turn lane sections.

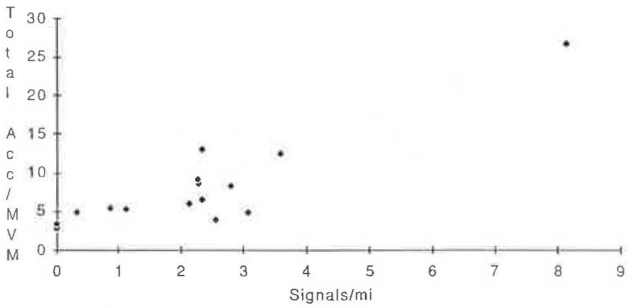

FIGURE 1 Raised median four-lane sections.

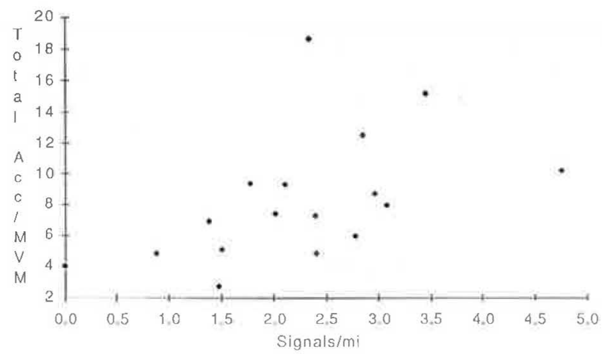

FIGURE 2 Raised median six-lane sections.

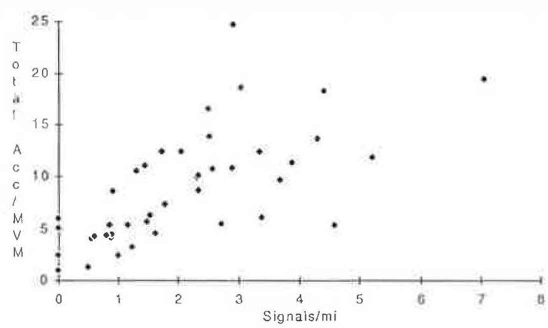

FIGURE 3 TWLTL four-lane sections.

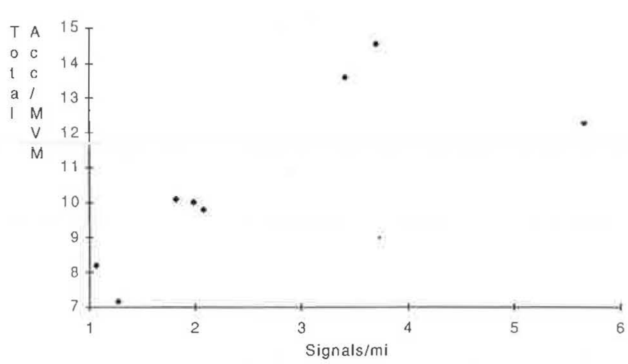

FIGURE 4 TWLTL six-lane sections.

of the extremely low alpha errors found in this study are as close as could reasonably be expected to ascertaining a difference in accident rates for the two cross-section types analyzed.

As expected, raised medians were found to be safer in terms of the number of midblock accidents. However, this determination should not be a decisive factor in comparison of the two median types. Raised medians shift many conflicts from midblock locations to surrounding intersections. Conceptually, the minimization of total accidents, not just midblock accidents, should be important in comparing the effects of median type.

As mentioned, the number of accidents per million vehicle miles (MVM) is preferred to the number of accidents per mile per year as an indicator of relative safety. The use of the accident per MVM rate accounts for differences among sites in traffic volumes and, therefore, in differences in the opportunity for accidents.

Elimination of the study of accidents per mile per year reduces the most useful comparisons to those of total accidents per MVM for four- and six-lane sections. The rates indicate that raised medians had a lower accident experience than TWLTLs for the range of variable data tested. However, the question of determining an acceptable alpha error is crucial because the difference in accident rates for four-lane sections has a high alpha error.

Regression Models

Regression equations were developed to model data obtained for each section type. Four basic section types were analyzed (raised median and TWLTL, each with four- and six-lane sections). Additionally, data were further subdivided by total and midblock accidents and accidents per MVM and accidents per mile per year. This grouping led to the development of 16 regression equations.

Regression equations were found by using three Biomedical computer program (BMDP) statistical software on the Georgia Tech mainframe computer (Cyber B). Data were initially tested with BMDP9R and BMDP2R to determine which variables were significant in the regression analysis. These two programs serve to eliminate variables that are redundant or have a high correlation to significant variables. BMDPIR was then used to find the final regression equation based on the variable sets found by the first two programs.

BMDP9R is an "all-possible-subset" regression program. In other words, the program will test all of the possible combinations of data, from single variables to all of the independent variables. The best set of variables is then chosen from the tested combinations on the basis of Mallow's C_p. This statistic provides a measure of whether the regression equation has enough information in it. Use of this indicator serves to maximize both the squared multiple correlation (R^2) and the F ratio (also called F statistic). Neither of these statistics, when used individually, provides an accurate description of an equation's utility. Although it is desirable to maximize R^2, excess variables in an equation tend to inflate this value. Although the F ratio does not describe the relationship between the regression and residual sum of squares, as R^2 does, this statistic reacts inversely with the addition of unnec-

TABLE 4 SIGNIFICANT DIFFERENCE OF ACCIDENT RATES BETWEEN TWLTL AND RAISED MEDIANS

Section type	Accident type	Alpha-error at point of significant difference	Significant difference at alpha-error =0.10	=0.05
Total Accidents				
4 Lane sections	Acc/MVM	0.2168	no	no
	Acc/mile/yr	0.0980	yes	no
6 Lane sections	Acc/MVM	0.0549	yes	no
	Acc/mile/yr	0.0883	yes	no
Midblock Accidents				
4 Lane sections	Acc/MVM	0.0009	yes	yes
	Acc/mile/yr	0.0128	yes	yes
6 Lane sections	Acc/MVM	<0.0005	yes	yes
	Acc/mile/yr	0.0224	yes	yes

essary variables. The F ratio is used with the R^2 statistic to find the best regression equation.

BMDP2R was then used to find what it considered to be the best set of variables. BMDP2R, a stepwise regression program, attempts to enter a variable into an equation and then seeks to remove a variable based on the equation's F ratio. Often this process results in a smaller variable list than those suggested by other programs.

All of the suggested variables combinations from the two programs were used with BMDP1R (a multiple linear regression program) to find the final regression equation for each section type. When alternate variable lists were compared, the equation that produced the best combination of R^2 and F ratio was chosen.

Table 5 lists the variables selected as significant for regression equations for each section type, along with the corresponding R^2 and F ratio values. Regression equations were found that fit total accidents well for almost all section types. Raised median six-lane section accidents per MVM were the exception. On the other hand, half of the midblock accident models fit poorly, which probably indicates that the type of data obtained was not adequate to explain midblock accidents.

Regression equations developed are linear. That is, they are of the form

$$y = aX_1 + bX_2 + \cdots + f$$

Table 6 lists regression coefficients for the variables. As can be seen all of the total accident equations rely on the number of signals per mile. Further, all of the total accident per mile per year equations (and none of the total accident per MVM equations) incorporate ADT.

Expected Value Tables

Tables of expected accident rates, developed from the regression analysis, list the accident rates estimated by the regression equations. The tables cover only data ranges that were present at the sections studied. This approach has led to different variables value ranges for four- and six-lane sections. For instance, ADTs range from 20,000 to 50,000 for six-lane sections, but four-lane section ADTs range from 10,000 to 50,000.

However, in some places, the tables give rates at combinations of independent-variable values that were not observed at the sections studied. The table of expected accidents per MVM on four-lane sections has no rates that were not covered by the data obtained. This situation results from the limited number of independent variables found to be significant in the corresponding regression equations. On the other hand, the table for accidents per MVM on six-lane sections has several areas that were not found in the study sections. In this table, all of these were predicted TWLTL rates because of the paucity of TWLTL six-lane sections. For all values of ADT, data range combinations that were not found in the field were

- One signal per mile, 30 drives per mile, and 6 approaches per mile;

TABLE 5 VARIABLE SETS USED IN REGRESSION EQUATIONS

Section type		Variable sets	Multiple R^2	F Ratio
TOTAL ACCIDENTS				
TWLTL 6 Lanes	-Acc/mi/yr	ADT, Drives/mi, Signals/mi, Apprch/mi	0.9861	53.088
	-Acc/MVM	Signals/mi, Apprch/mi Drives/mi	0.9572	29.823
TWLTL 4 Lanes	-Acc/mi/yr	ADT, Signals/mi, Apprch/mi	0.6018	19.146
	-Acc/MVM	Signals/mi	0.4443	31.980
R. Med. 6 Lanes	-Acc/mi/yr	ADT, Signals/mi	0.6242	11.629
	-Acc/MVM	Signals/mi	0.2639	5.378
R. Med. 4 Lanes	-Acc/mi/yr	ADT, Signals/mi	0.7670	19.752
	-Acc/MVM	Signals/mi	0.7990	51.661
MIDBLOCK ACCIDENTS				
TWLTL 6 Lanes	-Acc/mi/yr	ADT	0.8294	29.167
	-Acc/MVM	ADT	0.6281	10.131
TWLTL 4 Lanes	-Acc/mi/yr	ADT, Drives/mi, Apprch/mi	0.4772	11.563
	-Acc/MVM	Drives/mi, Apprch/mi	0.3939	12.671
R. Med. 6 Lanes	-Acc/mi/yr	ADT	0.2768	5.741
	-Acc/MVM	Openings/mi, Signals/mi	0.0749	0.567
R. Med. 4 Lanes	-Acc/mi/yr	ADT, Signals/mi	0.7579	18.781
	-Acc/MVM	Drives/mi, Signals/mi	0.7175	15.236

- One signal per mile, 60 drives per mile, and 4 or 6 approaches per mile;
- One or 2 signals per mile and 90 drives per mile;
- Two signals per mile, 30 drives per mile, and 4 or 6 approaches per mile;
- Three signals per mile and 30 drives per mile;
- Three signals per mile, 60 drives per mile, and 6 approaches per mile; and
- Three signals per mile, 90 drives per mile, and 4 or 6 approaches per mile.

The tables use the same variable format—even if some of the variables do not affect the accident rate—to facilitate comparisons and promote clarity. The purpose of the tables is not to show an absolute accident rate; rather, they are intended to present trends in the data and the relative difference between median types.

Tables 7 and 8 present the expected total accidents per MVM for four- and six-lane sections, respectively. Table 7 reveals that raised medians are safer than TWLTLs over the range of data studied for four-lane sections. However, it is significant that the difference between accident rates drops from 26 to 3 percent as the number of signalized intersections increases from one per mile to four per mile. The accident rates were calculated from regression equations that were dependent only on the number of signals per mile.

Table 8 gives the expected total accident rates for six-lane sections. As can be seen, accident rates for TWLTLs and raised medians did not depend on the same variables. In an effort to provide a common basis for comparison, Table 9 was created. Table 9 is similar to Table 8 in all respects, except that raised median accidents were modeled as being dependent on the same variables as TWLTL accidents. A regression equation was derived through computer analysis, as with the

TABLE 6 REGRESSION EQUATION COEFFICIENTS

Total Accidents

Section	Accident Type	ADT	Coefficients Drives per mi	Signals per mi	Apprch per mi	Open. per mi	Constant
TWLTL 6 Lanes	Acc/Mi/Yr	0.0050848	-0.89517	32.37220	6.48221	---	-73.91125
	Acc/MVM	0	-0.08593	3.08711	0.44833	---	7.53150
TWLTL 4 Lanes	Acc/Mi/Yr	0.0038777	0	22.68622	-8.85380	---	-21.86862
	Acc/MVM	0	0	2.29131	0	---	4.01780
Raised Median 6 Lanes	Acc/Mi/Yr	0.00455	0	22.46702	0	0	-96.48022
	Acc/MVM	0	0	1.96196	0	0	3.85559
Raised Median 4 Lanes	Acc/Mi/Yr	0.0019168	0	16.13910	0	0	-14.79288
	Acc/MVM	0	0	2.72091	0	0	1.91835

Midblock Accidents

Section	Accident Type	ADT	Coefficients Drives per mi	Signals per mi	Apprch per mi	Openings per mi	Constant
TWLTL 6-Lanes	Acc/Mi/Yr	0.0033571	0	0	0	---	-60.86993
	Acc/MVM	0.0001292	0	0	0	---	-0.36498
TWLTL 4 Lanes	Acc/Mi/Yr	0.0016209	0.52512	0	-8.74647	---	4.19088
	Acc/MVM	0	0.05632	0	-0.61905	---	3.29801
Raised Median 6 Lanes	Acc/Mi/Yr	0.0009661	0	0	0	0	-8.13549
	Acc/MVM	0	0	0.16643	0	-0.10956	1.86028
							-19.32438
Raised Median 4 Lanes	Acc/Mi/Yr	0.0010357	0	2.51833	0	0	0.98599
	Acc/MVM	0	-0.04567	0.78479	0	0	

other regression models. This model was originally not used because the additional variables do not provide enough information to be statistically significant.

With regard to six-lane section total accidents, the expected-value tables indicate that raised medians are safer for all ADT levels except when there is 1 signalized intersection per mile and at least 75 driveways per mile or when there are 2 signalized intersections per mile, at least 80 driveways per mile, and 5 or fewer unsignalized approaches per mile.

These results should be viewed in light of the aforementioned independent variable combinations not covered by study data. Specifically, rates for the conditions where TWLTLs were found to be safer represent an extrapolation from variable combinations present in the study sections. Of course, the same holds true for many of the conditions for which raised medians were found to be safer.

For four-lane total accidents per MVM, raised medians were found to be safer for all conditions.

COMPARISON WITH PAST RESEARCH

Parker's 1983 Virginia study (*2*) presented expected-value tables and a set of general guidelines, all developed from a study of four-lane roads. The expected-value tables in that report indicate that with ADTs from 10,000 to 30,000, TWLTLs have a lower number of accidents per mile when there are fewer than 30 driveways per mile and fewer than 5 streets per mile.

The expected-value table for accidents per mile for four-lane sections in the current study revealed a different relationship. Drives per mile was not found to be significant for either median type. Further, ADT is definitely significant. At an ADT of 10,000, TWLTLs are safer except when the number of approaches per mile is low. At an ADT of 30,000, raised medians are safer except with seven or more approaches per mile and two or fewer signals per mile.

The relative safety of TWLTLs under conditions of few signals per mile and a high number of approaches is probably attributable to the characteristics associated with less developed areas. Under such conditions, there are probably few points of concentrated left-turn vehicle manuevers. Such points seem to adversely affect TWLTL safety. The correlation between ADT and accidents per mile per year is to be expected. As opposing traffic increases, left-turn movements should become safer at concentrated and controlled points such as those found with raised medians.

Parker's general guidelines were also found to apply to the sections studied in this project only when the number of accidents per mile per year was under consideration. Parker recommends a TWLTL median when there are fewer than 12 streets per mile and the number of driveways per mile exceeds 50. Although the results of this project agree with these guide-

TABLE 7 TOTAL ACCIDENTS/MVM EXPECTED: FOUR-LANE SECTIONS

Signals per mile	Drives per mile	Approach per mile	ADT = 10,000 TWLTL	ADT = 10,000 RM	ADT = 30,000 TWLTL	ADT = 30,000 RM	ADT = 50,000 TWLTL	ADT = 50,000 RM
1	25	2	6.31	4.64	6.31	4.64	6.31	4.64
		4	6.31	4.64	6.31	4.64	6.31	4.64
		6	6.31	4.64	6.31	4.64	6.31	4.64
		8	6.31	4.64	6.31	4.64	6.31	4.64
	50	2	6.31	4.64	6.31	4.64	6.31	4.64
		4	6.31	4.64	6.31	4.64	6.31	4.64
		6	6.31	4.64	6.31	4.64	6.31	4.64
		8	6.31	4.64	6.31	4.64	6.31	4.64
2	25	2	8.60	7.36	8.60	7.36	8.60	7.36
		4	8.60	7.36	8.60	7.36	8.60	7.36
		6	8.60	7.36	8.60	7.36	8.60	7.36
		8	8.60	7.36	8.60	7.36	8.60	7.36
	50	2	8.60	7.36	8.60	7.36	8.60	7.36
		4	8.60	7.36	8.60	7.36	8.60	7.36
		6	8.60	7.36	8.60	7.36	8.60	7.36
		8	8.60	7.36	8.60	7.36	8.60	7.36
3	25	2	10.89	10.08	10.89	10.08	10.89	10.08
		4	10.89	10.08	10.89	10.08	10.89	10.08
		6	10.89	10.08	10.89	10.08	10.89	10.08
		8	10.89	10.08	10.89	10.08	10.89	10.08
	50	2	10.89	10.08	10.89	10.08	10.89	10.08
		4	10.89	10.08	10.89	10.08	10.89	10.08
		6	10.89	10.08	10.89	10.08	10.89	10.08
		8	10.89	10.08	10.89	10.08	10.89	10.08
4	25	2	13.18	12.80	13.18	12.80	13.18	12.80
		4	13.18	12.80	13.18	12.80	13.18	12.80
		6	13.18	12.80	13.18	12.80	13.18	12.80
		8	13.18	12.80	13.18	12.80	13.18	12.80
	50	2	13.18	12.80	13.18	12.80	13.18	12.80
		4	13.18	12.80	13.18	12.80	13.18	12.80
		6	13.18	12.80	13.18	12.80	13.18	12.80
		8	13.18	12.80	13.18	12.80	13.18	12.80

lines on the basis of the number of accidents per mile per year, a TWLTL median would not be recommended for a four-lane road on the basis of accidents per MVM.

Harwood and St. John (*4*), within subjective guidelines such as the need to accommodate pedestrians, suggested that TWLTL should be used instead of raised medians when the number of driveways per mile exceeded 45 and when there were low to moderate volumes of through traffic. Some of the expected-value tables developed in this report suggest the same thing. For TWLTLs to be safer, the number of driveways per mile should be high. Although accidents per MVM remained constant with changing ADTs, accidents per mile per year preclude the use of TWLTLs at higher ADT levels.

The relative safety of TWLTLs and raised medians may be inferred from the Azzeh et al. FHWA report (*1*). As discussed previously, the application of the FHWA work is based on anticipated accident reduction from a previously undivided roadway. The accident-rate reductions were determined for a four-lane highway. From the comparison of expected accident reductions for each median type, for all ADT ranges, TWLTLs were expected to be safer when land development was low to moderate. Low to moderate land development was used to describe areas with several concentrated sources of traffic and few low-volume driveways. The implication of the FHWA report is that for high-development areas, which are assumed to have no high-volume driveways and a large number of low-volume driveways, raised medians are safer. Further, when more high-volume driveways and fewer low-volume driveways are present, TWLTLs would be safer. This implication is contrary to the results obtained in the current study and in other studies (*2,3*). The unusual results obtained from the FHWA report most likely mean that the relative safety of median types cannot be inferred from the accident-reduction rates of those median types.

CONCLUSIONS

This study provides a comparison of accident rates occurring in situations with raised medians and two-way left-turn lanes (TWLTLs). Regression equations have also been developed to model accident occurrence for each median type. In all,

TABLE 8 TOTAL ACCIDENTS/MVM EXPECTED: SIX-LANE SECTIONS

Signals per mi	Drives per mile	Approach per mile	ADT = 20,000 TWLTL	ADT = 20,000 RM	ADT = 30,000 TWLTL	ADT = 30,000 RM	ADT = 40,000 TWLTL	ADT = 40,000 RM	ADT = 50,000 TWLTL	ADT = 50,000 RM
1	30	2	8.94	5.82	8.94	5.82	8.94	5.82	8.94	5.82
		4	9.83	5.82	9.83	5.82	9.83	5.82	9.83	5.82
		6	10.73	5.82	10.73	5.82	10.73	5.82	10.73	5.82
	60	2	6.36	5.82	6.36	5.82	6.36	5.82	6.36	5.82
		4	7.26	5.82	7.26	5.82	7.26	5.82	7.26	5.82
		6	8.15	5.82	8.15	5.82	8.15	5.82	8.15	5.82
	90	2	3.78	5.82	3.78	5.82	3.78	5.82	3.78	5.82
		4	4.68	5.82	4.68	5.82	4.68	5.82	4.68	5.82
		6	5.57	5.82	5.57	5.82	5.57	5.82	5.57	5.82
2	30	2	12.02	7.78	12.02	7.78	12.02	7.78	12.02	7.78
		4	12.92	7.78	12.92	7.78	12.92	7.78	12.92	7.78
		6	13.82	7.78	13.82	7.78	13.82	7.78	13.82	7.78
	60	2	9.45	7.78	9.45	7.78	9.45	7.78	9.45	7.78
		4	10.34	7.78	10.34	7.78	10.34	7.78	10.34	7.78
		6	11.24	7.78	11.24	7.78	11.24	7.78	11.24	7.78
	90	2	6.87	7.78	6.87	7.78	6.87	7.78	6.87	7.78
		4	7.77	7.78	7.77	7.78	7.77	7.78	7.77	7.78
		6	8.66	7.78	8.66	7.78	8.66	7.78	8.66	7.78
3	30	2	15.11	9.74	15.11	9.74	15.11	9.74	15.11	9.74
		4	16.01	9.74	16.01	9.74	16.01	9.74	16.01	9.74
		6	16.90	9.74	16.90	9.74	16.90	9.74	16.90	9.74
	60	2	12.53	9.74	12.53	9.74	12.53	9.74	12.53	9.74
		4	13.43	9.74	13.43	9.74	13.43	9.74	13.43	9.74
		6	14.33	9.74	14.33	9.74	14.33	9.74	14.33	9.74
	90	2	9.96	9.74	9.96	9.74	9.96	9.74	9.96	9.74
		4	10.85	9.74	10.85	9.74	10.85	9.74	10.85	9.74
		6	11.75	9.74	11.75	9.74	11.75	9.74	11.75	9.74

TABLE 9 TOTAL ACCIDENTS/MVM EXPECTED USING SAME VARIABLES: SIX-LANE SECTIONS

Signals per mi	Drives per mile	Approach per mile	ADT = 20,000 TWLTL	ADT = 20,000 RM	ADT = 30,000 TWLTL	ADT = 30,000 RM	ADT = 40,000 TWLTL	ADT = 40,000 RM	ADT = 50,000 TWLTL	ADT = 50,000 RM
1	30	2	8.94	5.34	8.94	5.34	8.94	5.34	8.94	5.34
		4	9.83	5.77	9.83	5.77	9.83	5.77	9.83	5.77
		6	10.73	6.20	10.73	6.20	10.73	6.20	10.73	6.20
	60	2	6.36	5.36	6.36	5.36	6.36	5.36	6.36	5.36
		4	7.26	5.78	7.26	5.78	7.26	5.78	7.26	5.78
		6	8.15	6.21	8.15	6.21	8.15	6.21	8.15	6.21
	90	2	3.78	5.37	3.78	5.37	3.78	5.37	3.78	5.37
		4	4.68	5.80	4.68	5.80	4.68	5.80	4.68	5.80
		6	5.57	6.23	5.57	6.23	5.57	6.23	5.57	6.23
2	30	2	12.02	7.15	12.02	7.15	12.02	7.15	12.02	7.15
		4	12.92	7.58	12.92	7.58	12.92	7.58	12.92	7.58
		6	13.82	8.00	13.82	8.00	13.82	8.00	13.82	8.00
	60	2	9.45	7.16	9.45	7.16	9.45	7.16	9.45	7.16
		4	10.34	7.59	10.34	7.59	10.34	7.59	10.34	7.59
		6	11.24	8.02	11.24	8.02	11.24	8.02	11.24	8.02
	90	2	6.87	7.18	6.87	7.18	6.87	7.18	6.87	7.18
		4	7.77	7.60	7.77	7.60	7.77	7.60	7.77	7.60
		6	8.66	8.03	8.66	8.03	8.66	8.03	8.66	8.03
3	30	2	15.11	8.95	15.11	8.95	15.11	8.95	15.11	8.95
		4	16.01	9.38	16.01	9.38	16.01	9.38	16.01	9.38
		6	16.90	9.81	16.90	9.81	16.90	9.81	16.90	9.81
	60	2	12.53	8.97	12.53	8.97	12.53	8.97	12.53	8.97
		4	13.43	9.40	13.43	9.40	13.43	9.40	13.43	9.40
		6	14.33	9.82	14.33	9.82	14.33	9.82	14.33	9.82
	90	2	9.96	8.98	9.96	8.98	9.96	8.98	9.96	8.98
		4	10.85	9.41	10.85	9.41	10.85	9.41	10.85	9.41
		6	11.75	9.84	11.75	9.84	11.75	9.84	11.75	9.84

50 TWLTL and 32 raised median sections were studied, lending stability to the analysis performed.

Comparisons were made for total and midblock accidents, four- and six-lane sections, accidents per million vehicle miles (MVM) and accidents per mile per year, and injury, fatal, and all accidents occurring. Although the comparisons of all of these combinations are interesting, total accidents per MVM is considered to give the best indication of the relative safety of a median type. The comparison of accidents occurring on six-lane sections showed, with a low statistical error, raised medians to be safer than TWLTLs. The accident comparison for four-lane sections also showed raised medians to be safer, but with a higher statistical error.

The relative safety of raised medians probably resulted from the range of ADTs used. With higher volumes of opposing traffic, left-turn movements seem to be safer at concentrated points, such as those provided by raised medians. When turns are concentrated at certain points, the area in which conflicts occur is greatly reduced. The turns may also be better accommodated at concentrated points; by using traffic signals, for example.

Regression equations were developed for 16 conditions: raised medians and TWLTLs, four- and six-lane sections, total and midblock accidents, and accidents per mile per year and accidents per MVM. Total accidents per MVM were used to accurately reflect the relative safety of the sections. The following are regression equations developed for the four sections:

TWLTL 6-lane:

$$\text{accidents/MVM} = 3.087 * S - 0.086 * D + 0.448 * A + 7.532$$

TWLTL 4-lane:

$$\text{accidents/MVM} = 2.291 * S + 4.018$$

Raised median 6-lane:

$$\text{accidents/MVM} = 1.962 * S + 3.856$$

Raised median 4-lane:

$$\text{accidents/MVM} = 2.721 * S + 1.918$$

Where

S = Signals per mile;
D = Driveways per mile; and
A = Approaches per mile.

Tables presenting the expected accident rates were generated for all regression models developed. The expected-value tables for the regression equations produced results comparable to the accident-rate comparison performed earlier.

For four-lane sections, raised medians were always safer than TWLTLs. However, the difference in rates was found to decrease with increasing numbers of signals per mile. For six-lane sections raised medians were, again, found to be safer except under certain conditions. TWLTLs were safer when all the following conditions were met: high numbers of driveways per mile (at least 75), low numbers of signals per mile (2 or fewer), and low numbers of approaches per mile (a maximum of 5 or 6, depending on signals per mile).

Results of this study compared fairly well with those of other research when viewed using the parameters of the other studies. The general guidelines developed in other research appear to be applicable, especially in relation to the six-lane sections studied. For TWLTLs to be safer than raised medians, traffic should be low with few concentrated sources of traffic entering or leaving the road.

ACKNOWLEDGMENTS

The authors would like to offer most sincere thanks to Paul Wright and John Moskaluk for serving as readers of this report. The authors gratefully acknowledge Tim Christian and the Georgia DOT Planning Data Services office in Chamblee for answering questions and providing significant computer time in the early stages of this project and Dick Graves of Georgia DOT Traffic and Safety Division for providing accident data.

REFERENCES

1. J. A. Azzeh, B. A. Thorson, J. J. Valenta, J. C. Glennon, and C. J. Wilton. *Evaluation of Techniques for the Control of Direct Access to Arterial Highways.* Report FHWA-RD-76-87. FHWA, U. S. Department of Transportation, Aug. 1975.
2. M. R. Parker, Jr. *Design Guidelines for Raised and Traversable Medians in Urban Areas.* Virginia Highway and Transportation Research Council, Charlottesville, Dec. 1983.
3. D. W. Harwood and A. D. St. John. *Passing Lanes and Other Operational Improvements on Two-Lane Highways.* Report FHWA-RD-85/028. FHWA, U.S. Department of Transportation, 1985.
4. J. E. Vargas. *Median Treatments: Two-Way Left-Turn Lanes Versus Raised Medians.* Georgia Institute of Technology, Atlanta, June 1987.

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Evaluation of New Passing Zone Gore Design

THOMAS M. BATZ

The purpose of this research was to determine the effects of a new passing zone gore design on traffic characteristics. The new design was a painted gore at the beginning of the passing zone to guide the traffic into the right lane, thereby directing the slow-moving vehicles out of the left lane. The project was to determine whether the new gore design is more efficient than the old design because it allows more vehicles to pass and reduces illegal and erratic maneuvers and whether it has any effect on the safety of the roadway. The study demonstrated that when the new passing zone gore design was used, passing efficiency within the passing zone actually decreased at the 0.4-mile site, although it increased slightly at the 0.9-mile site. The number of illegal or erratic maneuvers, such as passes on the right and platoon leaders staying left or going through the gore, was reduced at the longer site, but increased at the shorter site. Safety did not appear to be affected by either passes on the right or going through the painted gore. From these findings came the following recommendations: (a) the new passing zone gore design should not be implemented at this time on passing zones of one-half mile or less in length; (b) the new passing zone gore design could be implemented on passing zones greater than one-half mile; (c) further research to deal with such factors as total passes and different passing zone lengths should be carried out to determine more specifically the effects of this new design on the traffic characteristics.

In the past, when a sufficient number of safe passing sections could not be afforded on two-lane roadways because of the vertical or horizontal alignment, transportation departments designed and installed additional passing lanes to improve the quality of service of the through lanes by eliminating the interference caused by slower vehicles. This approach prevented the development of bottlenecks. These passing lanes usually extended for less than 1 mile.

The additional lanes have always been added on the right-hand side, as shown in Figure 1. Under this design, slower vehicles should move to the right to allow faster vehicles to pass on the left. At the end of the passing lane, the slower traffic moved back to the left, into the through lane. Therefore, slower traffic must voluntarily make two distinct lane changes for this design to work effectively. However, drivers of some slower vehicles are hesitant to move to the right because they will need to merge back to the left in a short distance. This situation causes a dilemma for drivers of the vehicles in the rear that want to go faster: They must either pass on the right, which is illegal in New Jersey and creates a safety problem, or stay behind the slower vehicle and not pass, which makes the roadway less efficient and defeats the purpose of installing the added lane.

Because of these problems, personnel from the New Jersey Division of Transportation's (NJDOT) Division of Design have introduced a new design (Figure 2) that leads all traffic to enter the added right lane, creating the new lane to be used for passing on the left. The slower vehicles should no longer remain in the left lane, thereby allowing more vehicles to pass properly and improving the efficiency of the roadway. At the end of the passing zone, the right-lane traffic will merge into the left lane, which is the current practice.

A literature search revealed that this design has been used for other purposes. However, it has never been used for the addition of a passing lane on a two-lane roadway. This design project was initiated to determine whether the new passing zone gore design is more effective than the old design because it allows more vehicles to pass, and reduces illegal and erratic maneuvers, and whether it has any effect on the safety of the roadway.

PROCEDURE

Literature Search

A literature search determined that the gore design had been used but not at the beginning of a passing zone. Bennett (*1*) showed how the use of this gore at intersections to introduce left-turn lanes could reduce accidents. The new median lanes of the eight-lane Garden State Parkway in New Jersey, were reserved for high occupancy vehicles (HOVs). To make sure that only HOVs were in these lanes, consultants for NJDOT proposed a striping plan very similar to the new gore design, which brought the new median lane in on the left, so that vehicles had to make a lane change to enter it (*2*). Botha and May developed a computer model for a passing lane with this gore design, but never actually implemented or studied it (*3*). No data were found to describe how the new gore design would affect traffic operations in a passing zone.

Field Study Design

The next step was the design of the field studies. The first phase was to determine what data to collect and the data-collection method. The size of platoons entering the passing zone was an essential element of the evaluation. This variable would provide the number of drivers that wanted to pass the platoon leader and indicate whether the same population would exist when both the traditional design and new gore design were studied. To determine who the platoon leaders were was difficult, however.

Platoons have been defined as groups of vehicles with headways ranging from as little as 2 sec to as much as 5 sec. For

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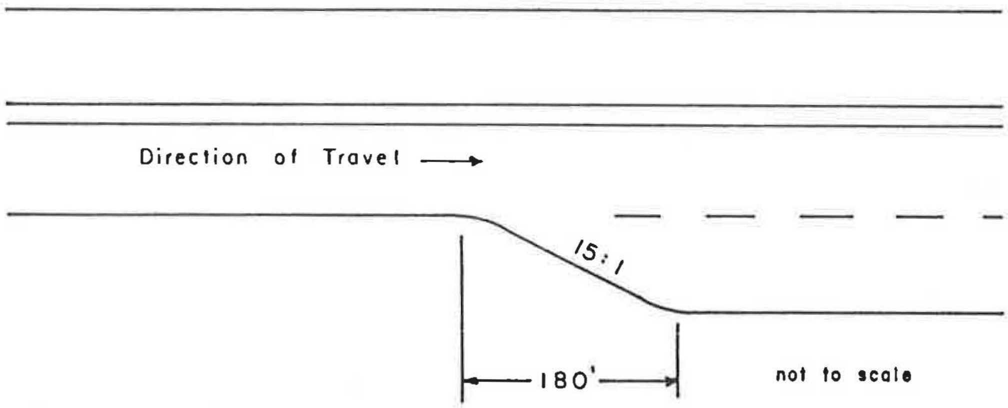

FIGURE 1 Traditional passing zone design.

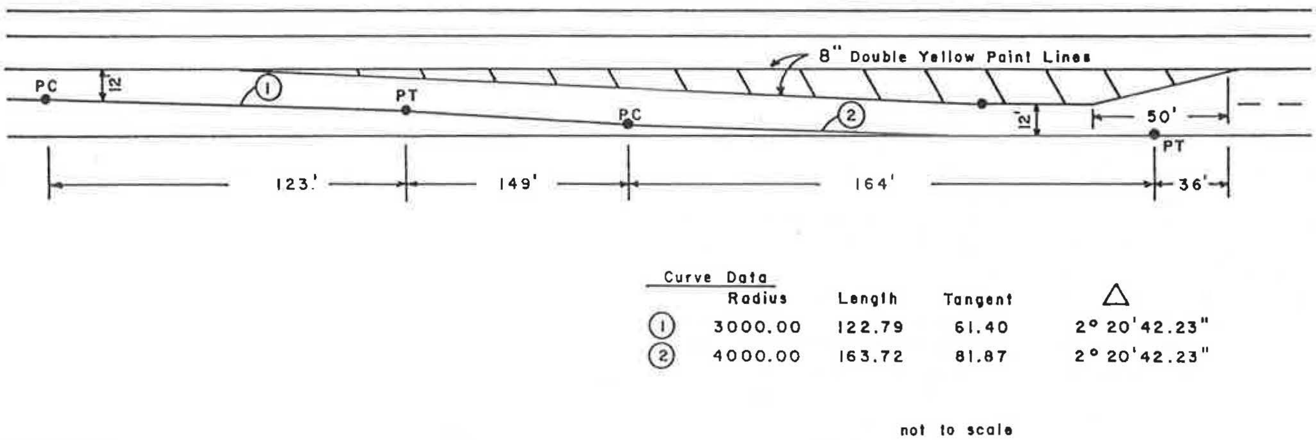

FIGURE 2 New passing zone design.

the New Jersey study, vehicles were included in the same platoon if they were within 3 sec of each other. This definition was based on the fact that at 50 mph, the headway between two cars would be 200 ft, or approximately 10 car lengths. At this distance, the car in the rear would not be influenced by the car that it was following.

A videotape camera, positioned at the beginning of the passing zone, collected data. The time in 100ths of a second was superimposed on the tape. The camera recorded the time at which each vehicle crossed a strip of reflectorized tape on the pavement; from this information the platoon leaders and size of platoon were determined. The reflectorized tape, from all observations, had no effect on traffic characteristics. Figure 3 for NJ-31 and Figure 4 for NJ-206 show the location of both the camera and reflectorized tape. The criteria for selecting these routes will be discussed later. Because the new passing gore was not expected to have an effect on platoon size, this variable was not expected to change.

Determining the speeds of platoon leaders was another way to assure that the same population existed when both traditional and new gore designs were studied. The videotape camera at the beginning of the passing zone recorded the time it took the platoon leader to travel 100 ft between two strips of reflectorized tape. With this time and distance, the speed could be calculated. Figures 3 and 4 show the configuration. Because the new passing zone gore was not expected to have an effect on platoon leader speed, this variable was expected not to change.

The number of platoon leaders that stay in the left lane is a measure of those vehicles that do not obey the "keep right except to pass" law, as well as of the vehicles that cause others to break the "pass only on the left" law. Going through the newly installed gore was illegal because to do so, a vehicle would have to cross a double yellow line. For these reasons, tracking the location of platoon leaders was essential. Two videotape cameras, positioned to capture essentially the entire passing zone area, recorded this information. Camera locations are identified in Figures 3 and 4.

Platoon leaders were classified in four groups:

- Those who moved right;
- Those who stayed left with the traditional design or who traveled through the gore and stayed left with the new gore design;
- Those who stayed or moved left for approximately one-half of the passing zone's length; and
- Those who—using the new design—followed the gore and then immediately returned to the left lane.

The new passing zone gore design was expected to move almost all platoon leaders into the right lane, leading to the expectation of a large decrease in the number of platoon leaders remaining to the left.

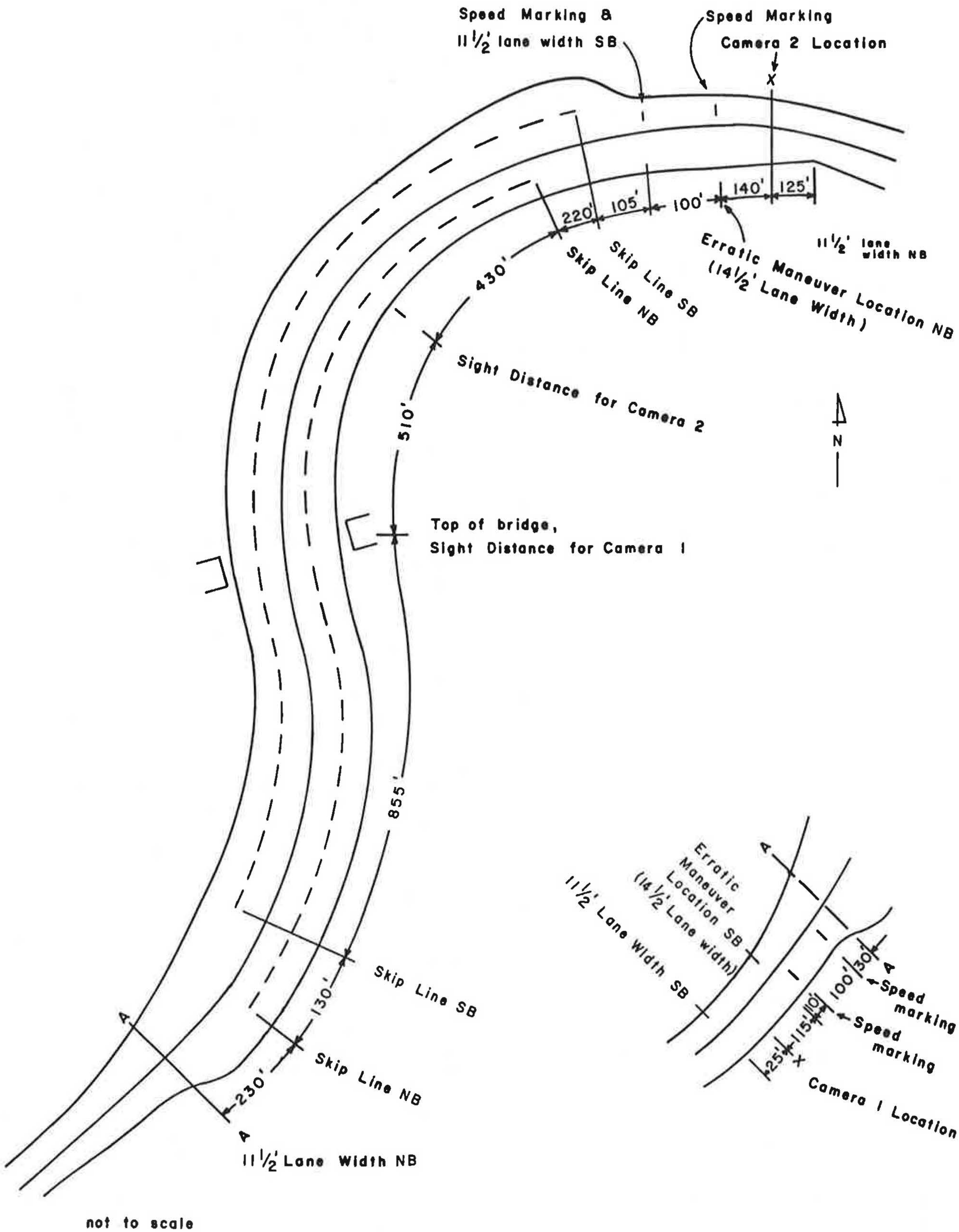

FIGURE 3 Rt. 31 test and control sites; data collection locations.

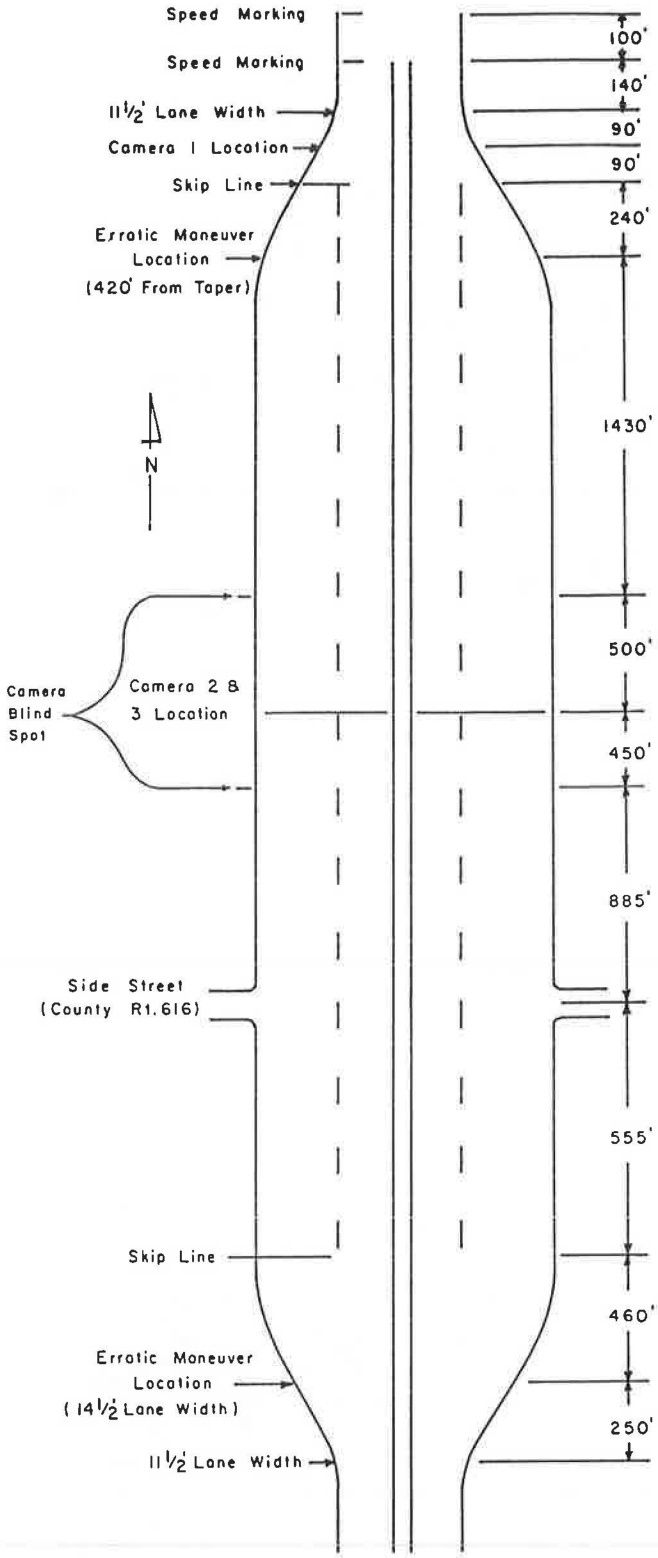

FIGURE 4 Rt. 206 test site; data collection location (not to scale).

Followers passing through the new gore were acting illegally. Therefore, collecting data about this maneuver was very important to the evaluation of the design. Two road tubes (shown in Figure 5) placed on the roadway recorded car movement. It was possible to observe if a vehicle were more than halfway into the left lane or gore area. Data were also collected using the traditional design at the NJ-206 site. Of interest was whether the area now covered by the new gore (420 ft) was previously used by followers to pass. If this pattern were true, the loss of this distance could be critical to short passing zones by reducing the effective distance a vehicle had to pass. The new passing zone gore design was expected to move the followers to the right lane, a maneuver that was not considered to be a future problem.

One of the two possible consequences of platoon leaders' staying to the left is that followers must pass them on the right, which is illegal in New Jersey. Data about this behavior would demonstrate whether the new gore design was meeting its objectives. Information about passes on the right within the platoon among followers was also important. Two videotape cameras that viewed the entire passing zone were used to collect data. Because platoon leaders were expected to be moved into the right lane, passing on the right was expected to decrease substantially.

The total number of passes by platoon leaders was the main measure used to determine whether a passing zone was efficient. Unless a follower passes the platoon leader, it cannot increase its speed. These essential data were also collected by the two videotape cameras that viewed the entire passing zone.

Because the new gore was expected to force the slower platoon leaders into the right lane, the efficiency of the passing zone was also expected to improve. Therefore, total passes of platoon leaders were expected to increase.

When the new design was used, it was anticipated that more of the slower vehicles would be in the right lane. Concern arose that these vehicles would not be able to move into the single lane at the end of the passing lane and, as a result, would cross the shoulder edgeline. Information about this erratic maneuver was collected by using a road tube in the transition area from two lanes to one lane, as shown in Figure 6. This variable was not expected to be affected by the new passing zone gore design.

The major problem with erratic and illegal maneuvers is that they are not anticipated by other drivers and may create conditions that result in accidents. An important element for analysis was if this new gore design had an effect on safety. Accident reports from sites using the traditional and new gore designs were reviewed to determine the causes of accidents at study sites and at the control site. Because of the infrequent occurrence of accidents and the brief study period, conclusions about accidents were not expected.

After the types of data to be collected and hypotheses were determined, a decision was made when to collect data. Peak hours were selected because traffic volumes were the highest during these periods. It was expected that the highest passing rates would occur at these times. In other words, it was assumed that the peak hours are when added passing lanes are needed the most. The decision was made to collect data for three days per study period to avoid any possible 1 day problems, such as an upstream accident, and to obtain a larger sample.

Data were to be collected from 4:00 p.m. to 6:00 p.m. on 3 days at the site with the traditional design and on 3 days at the sites with the new gore design. The data were to be compared to determine it a difference existed. Studies would be performed in July and August in successive years. It was anticipated that the same type of traffic would be present for both designs and that there would be ample time between

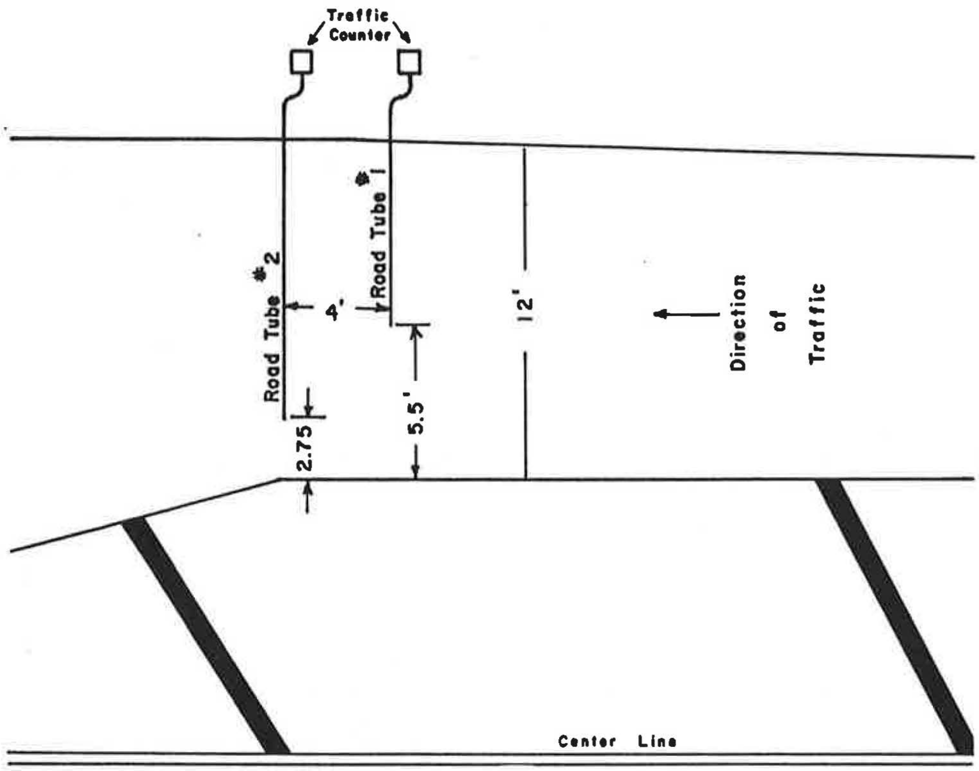

FIGURE 5 Road tube setup for passing gore encroachment study.

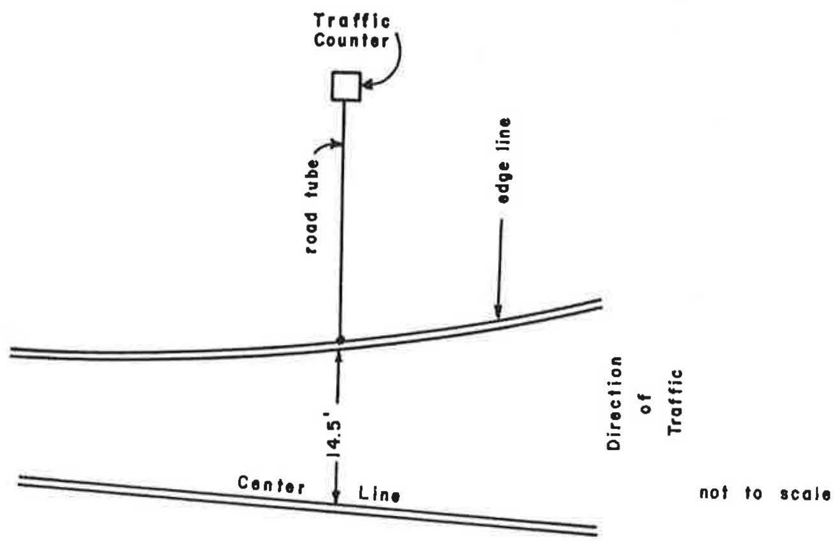

FIGURE 6 Road tube setup for end of passing zone encroachment study.

installation of the new gore design and collection of data to eliminate "newness" of the gore as an influence on the data.

Another decision involved the type of sites to be studied. Major considerations included good sight angles and locations for the videotape cameras that were to collect data, minimal effects from traffic signals upstream of or within the passing zone, different volume levels, and different passing zone lengths. Twenty-five passing zone locations were reviewed. Due to budget limitations, only two sites were to be studied: one approximately 1 mile long and the other approximately one-half mile long. Southbound NJ-206 in Burlington County, from milepost 21.2 to milepost 22.1, and northbound NJ-31 in Mercer County, from milepost 6.8 to milepost 7.2, were chosen as the test sites for the new gore design. Southbound NJ-31 at the same location was chosen as the control site. The NJ-206 site is a straight roadway with one cross street and a few driveways. The NJ-31 site, on the other hand, is an S-curve, with no cross streets and a few driveways.

The study of the traditional design was performed in 1984 for NJ-31; data were collected in 1985 for the new gore design study. The NJ-206 traditional design study was delayed by construction until 1985; data for the new gore design study were collected in 1986. Because the NJ-31 control site data had to be collected for all three years and because the NJ-31 study site was simply the opposite direction, the decision was made to collect and analyze a second year of new gore design data for the NJ-31 study site.

All data collected on videotape were brought back to the office for reduction. The data were then run through the Statistical Analysis System computer package to determine the means and standard deviations for some of the variables.

RESULTS

The literature review yielded no studies concerning the use of the new gore design at the beginning of a passing zone. The New Jersey investigation amassed such data. The results of the data collection and analysis effort are presented here.

Once the data were reviewed, it became clear that certain vehicles should not be included in the analysis. These included any vehicle traveling by itself, all vehicles in a platoon where a turn off or turn on to the highway affected the performance of the platoon, and all vehicles in a platoon in which the platoon leader stayed left to pass a member of another platoon. In this way, only platoons that were not affected by other vehicles or other platoons were analyzed.

Table 1 presents the data by platoons and vehicles for the NJ-31 control site for the three study periods. Tables 2 and 3 present the equivalent data for the NJ-31 study site and the NJ-206 study site. Approximately 45 to 55 percent of the platoons for each site were considered in the analysis that made up approximately 70 to 80 percent of the total volume.

Eight specific variables were analyzed, and an overall evaluation of the effect of the new passing zone gore on traffic characteristics was prepared. Table 4 presents data about platoon size by day and by study period for each of the three sites. At all sites the platoon size increased slightly in successive years. However, it was concluded that these changes in platoon size were not caused by the new gore design and did not have an effect on the number of vehicles passing the platoon leader. This result was as anticipated.

Data concerning platoon leaders' speed, by day and by study period for each of the three sites, are presented in Table 5. The speeds increased slightly at the two study sites, but it was concluded that these changes were not caused by the new gore design and would not affect the passing maneuvers in the platoons. This result was as anticipated.

Platoon leaders' locations by day and by study period for each of the three sites are profiled in Table 6. At the control site the number of platoon leaders staying in the left lane increased in each successive year (1984, 378; 1985, 442; and 1986, 505). However, with the installation of the new passing zone gore at both study sites, the number of platoon leaders going through the new gore and staying in the left lane (NJ-31: 1985, 6; 1986, 7; NJ-206: 1986, 4) combined with those that followed the new gore and then immediately returned to the left lane (NJ-31: 1985, 20; 1986, 25; NJ-206: 1986, 8) was much lower than the number of platoon leaders staying in the left lane with the traditional striping (NJ-31: 1984, 168; NJ-206: 1985, 207). The objective of decreasing these illegal maneuvers was reached.

The number of followers traveling through the new gore for the study sites is displayed in Table 7. Almost 400 of the followers traveled through this gore (new gore studies). The expected result—that the number of illegal maneuvers would not be large—did not occur. This maneuver is potentially dangerous: vehicles that follow the new gore correctly and then start moving to the left to pass do not expect anyone to be going through the gore.

TABLE 1 RT. 31 CONTROL SITE: SIX-HOUR TOTAL PLATOONS AND VEHICLES, BY STUDY PERIOD

	TRADITIONAL - 1984		TRADITIONAL - 1985		TRADITIONAL - 1986	
	Platoons	Vehicles	Platoons	Vehicles	Platoons	Vehicles
PLATOONS ANALYZED	817	3545	947	4082	1008	4451
SINGLE VEHICLES	847	847	750	750	828	828
SINGLE VEHICLES WITH TURNS	7	7	6	6	8	8
OTHER PLATOONS WITH TURNS	8	27	11	53	17	63
SINGLE VEHICLES WITH LEADER PASSING	17	17	24	24	21	21
OTHER PLATOONS WITH LEADER PASSING	12	59	10	39	21	79
SINGLE VEHICLES WITH BOTH	1	1	1	1	0	0
OTHER PLATOONS WITH BOTH	1	2	0	0	1	5
TOTAL	1710	4505	1749	4955	1904	5455
AVERAGE PER HOUR	285	751	292	826	317	909

TABLE 2 RT. 31 STUDY SITE: SIX-HOUR TOTAL PLATOONS AND VEHICLES, BY STUDY PERIOD

	TRADITIONAL - 1984		NEW GORE - 1985		NEW GORE - 1986	
	Platoons	Vehicles	Platoons	Vehicles	Platoons	Vehicles
PLATOONS ANALYZED	915	2932	913	3074	1005	3451
SINGLE VEHICLES	1059	1059	999	999	940	940
SINGLE VEHICLES WITH TURNS	10	10	13	13	9	9
OTHER PLATOONS WITH TURNS	12	36	33	140	27	121
SINGLE VEHICLES WITH LEADER PASSING	25	25	20	20	21	21
OTHER PLATOONS WITH LEADER PASSING	16	53	19	59	13	34
SINGLE VEHICLES WITH BOTH	1	1	0	0	1	1
OTHER PLATOONS WITH BOTH	0	0	2	5	0	0
TOTAL	2038	4116	1999	4310	2016	4577
AVERAGE PER HOUR	340	686	333	717	336	763

TABLE 3 RT. 206 STUDY SITE: SIX-HOUR TOTAL PLATOONS AND VEHICLES, BY STUDY PERIOD

	TRADITIONAL - 1985		NEW GORE - 1986	
	Platoons	Vehicles	Platoons	Vehicles
VEHICLES ANALYZED	707	2615	760	2854
SINGLE VEHICLES	664	664	645	645
SINGLE VEHICLES WITH TURNS	6	6	17	17
OTHER PLATOONS WITH TURNS	17	82	18	82
SINGLE VEHICLES WITH LEADER PASSING	51	51	48	48
OTHER PLATOONS WITH LEADER PASSING	36	122	37	126
SINGLE VEHICLES WITH BOTH	0	0	1	1
OTHER PLATOONS WITH BOTH	0	0	0	0
TOTAL	1481	3540	1526	3773
AVERAGE PER HOUR	247	590	254	629

TABLE 4 SUMMARY, PLATOON SIZE (VEHICLES/PLATOON)

ROUTE 31 CONTROL SITE

	TRADITIONAL - 1984	TRADITIONAL - 1985	TRADITIONAL - 1986
DAY 1	4.26	4.29	4.15
DAY 2	4.21	4.26	4.66
DAY 3	4.53	4.38	4.45
AVERAGE	4.34	4.31	4.42

ROUTE 31 STUDY SITE

	TRADITIONAL - 1984	NEW GORE - 1985	NEW GORE - 1986
DAY 1	3.14	3.39	3.31
DAY 2	3.06	3.29	3.39
DAY 3	3.40	3.42	3.60
AVERAGE	3.20	3.37	3.43

ROUTE 206 STUDY SITE

	TRADITIONAL - 1985	NEW GORE - 1986
DAY 1	3.60	3.64
DAY 2	3.80	3.97
DAY 3	3.70	3.62
AVERAGE	3.70	3.76

TABLE 5 SUMMARY, PLATOON LEADER SPEED (MILES/HOUR)

ROUTE 31 CONTROL SITE

	TRADITIONAL - 1984	TRADITIONAL - 1985	TRADITIONAL - 1986
DAY 1	44.9	45.6	46.7
DAY 2	45.8	46.1	45.9
DAY 3	48.0	45.0	46.3
AVERAGE	46.3	45.5	46.3

ROUTE 31 STUDY SITE

	TRADITIONAL - 1984	NEW GORE - 1985	NEW GORE - 1986
DAY 1	45.8	47.0	46.6
DAY 2	46.2	46.8	47.7
DAY 3	45.6	45.8	47.0
AVERAGE	45.9	46.5	47.1

ROUTE 206 STUDY SITE

	TRADITIONAL - 1985	NEW GORE - 1986
DAY 1	52.6	53.0
DAY 2	52.3	51.9
DAY 3	51.9	52.0
AVERAGE	52.2	52.3

When the area now occupied by the new gore was studied with the traditional design at the NJ-206 site, the results showed that at the point where the gore would be the widest, approximately 50 percent of all the leaders were in the left lane, whereas 75 to 80 percent of the followers were in this lane. This compares with 1 percent of the platoon leaders and 18 percent of the followers that went through the gore area with the new gore installed.

The data show that although a large number of the followers were in the left lane with the traditional striping and in a position to use this future gore area to pass, most of them couldn't because at this same point, more than half of the platoon leaders were also in the left lane. This situation may be caused by drivers who do not follow the shoulder edgeline and take 400 to 500 ft to move to the right lane. Therefore, it is concluded that although the loss of this new gore area (420 ft) from the possible passing distance may have reduced passing opportunities slightly, it did not have a major effect.

Table 8 summarizes data by day and study period at all sites for total passes on the right. These data are in two groups—

TABLE 6 SUMMARY, PLATOON LEADER LOCATION (VEHICLES)

ROUTE 31 CONTROL SITE

	TRADITIONAL - 1984			TRADITIONAL - 1985			TRADITIONAL - 1986		
	A	B	C	A	B	C	A	B	C
DAY 1	131	118	13	142	144	21	140	177	27
DAY 2	113	136	17	153	150	11	135	162	40
DAY 3	141	124	24	149	148	29	126	166	35
TOTAL	385	378	54	444	442	61	401	505	102

ROUTE 31 STUDY SITE

	TRADITIONAL - 1984			NEW GORE - 1985				NEW GORE - 1986			
	A	B	C	A	B	C	D	A	B	C	D
DAY 1	239	57	1	274	3	0	5	290	2	16	8
DAY 2	253	55	1	293	1	0	11	332	4	14	9
DAY 3	253	56	0	312	2	8	4	315	1	6	8
TOTAL	745	168	2	879	6	8	20	937	7	36	25

ROUTE 206 STUDY SITE

	TRADITIONAL - 1985			NEW GORE - 1986			
	A	B	C	A	B	C	D
DAY 1	150	60	11	234	1	17	3
DAY 2	137	57	14	241	3	29	4
DAY 3	180	90	8	212	0	15	1
TOTAL	467	207	33	687	4	61	8

LEGEND

A = Right Lane B = Left Lane or Going Through the Gore
C = Left Lane For Approximately Half the Passing Zone Distance
D = Followed Gore Then Returned to Left Lane

TABLE 7 SUMMARY, FOLLOWERS TRAVELING THROUGH NEW GORE (VEHICLES)

ROUTE 31 STUDY SITE

	NEW GORE - 1985			NEW GORE - 1986		
	A	B	C	A	B	C
DAY 1	126	673	18.7	102	729	14.0
DAY 2	121	698	17.3	141	859	16.4
DAY 3	139	790	17.6	139	858	16.2
TOTAL	386	2161	17.9	382	2446	15.6

ROUTE 206 STUDY SITE

	NEW GORE - 1986		
	A	B	C
DAY 1	133	674	19.7
DAY 2	124	822	15.1
DAY 3	121	598	20.3
TOTAL	378	2094	18.1

LEGEND

A = Followers Through the Gore
B = Total Followers
C = Percent of Followers Through the Gore

****

ROUTE 206 STUDY SITE

TRADITIONAL - 1985

	A	B	C	D	E	F
DAY 1	438	575	76.2	105	221	47.4
DAY 2	451	582	77.4	98	208	47.3
DAY 3	598	751	79.6	155	278	55.7
TOTAL	1487	1908	77.9	358	707	50.6

LEGEND

A = Followers Through Future Gore Area B = Total Followers
C = Percent of Followers Through the Future Gore Area
D = Platoon Leaders Through the Future Gore Area E = Total Platoon Leaders
F = Percent of Platoon Leaders Through the Future Gore Area

TABLE 8 SUMMARY, TOTAL PASSES ON RIGHT (VEHICLES)

ROUTE 31 CONTROL SITE

	TRADITIONAL - 1984		TRADITIONAL - 1985		TRADITIONAL - 1986	
	A	B	A	B	A	B
DAY 1	38	25	48	42	65	58
DAY 2	38	43	41	64	53	83
DAY 3	32	42	38	67	47	91
TOTAL	108	110	127	173	165	232

ROUTE 31 STUDY SITE

	TRADITIONAL - 1984		NEW GORE - 1985		NEW GORE - 1986	
	A	B	A	B	A	B
DAY 1	10	8	2	1	3	0
DAY 2	9	1	2	1	8	1
DAY 3	9	5	0	4	2	0
TOTAL	28	14	4	6	13	1

ROUTE 206 STUDY SITE

	TRADITIONAL - 1985		NEW GORE - 1986	
	A	B	A	B
DAY 1	47	41	0	11
DAY 2	29	25	6	12
DAY 3	53	72	0	6
TOTAL	129	138	6	29

LEGEND
A = Passes of the Platoon Leader
B = Other Passes Within the Platoon

TABLE 9 SUMMARY, TOTAL PASSES OF PLATOON LEADER (VEHICLES)

ROUTE 31 CONTROL SITE

	TRADITIONAL - 1984				TRADITIONAL - 1985				TRADITIONAL - 1986			
	A	B	C	D	A	B	C	D	A	B	C	D
DAY 1	168	38	854	24.1	172	48	1009	21.8	150	65	1082	19.9
DAY 2	141	38	855	20.9	200	41	1025	23.5	217	53	1233	21.9
DAY 3	162	32	1019	19.0	174	38	1101	19.3	195	47	1128	21.5
TOTAL	471	108	2728	21.2	546	127	3135	21.5	562	165	3443	21.1

ROUTE 31 STUDY SITE

	TRADITIONAL - 1984				NEW GORE - 1985				NEW GORE - 1986			
	A	B	C	D	A	B	C	D	A	B	C	D
DAY 1	197	10	637	32.5	187	2	673	28.1	177	3	729	24.7
DAY 2	195	9	637	32.0	194	2	698	28.1	181	8	859	22.0
DAY 3	229	9	743	32.0	203	0	790	25.7	204	2	858	24.0
TOTAL	621	28	2017	32.2	584	4	2161	27.2	462	13	2446	23.5

ROUTE 206 STUDY SITE

	TRADITIONAL - 1985				NEW GORE- 1986			
	A	B	C	D	A	B	C	D
DAY 1	222	47	575	46.8	315	0	674	46.7
DAY 2	223	29	582	43.3	352	6	822	43.6
DAY 3	266	53	751	42.5	306	0	598	51.2
TOTAL	711	129	1908	44.0	973	6	2094	46.8

LEGEND
A = Passes on the Left B = Passes on the Right
C = Total Followers D = Percent of Followers That Passed

TABLE 10 SUMMARY, ERRATIC MANEUVERS AT PASSING ZONE END (VEHICLES)

ROUTE 31 CONTROL SITE

	TRADITIONAL - 1984			TRADITIONAL - 1985			TRADITIONAL - 1986		
	A	B	C	A	B	C	A	B	C
DAY 1	80	1588	5.0	87	1739	5.0	80	1871	4.3
DAY 2	92	1606	5.7	83	1748	4.7	96	1888	5.1
DAY 3	94	1634	5.8	95	1787	5.3	102	2013	5.1
TOTAL	266	4828	5.5	265	5274	5.0	278	5772	4.8

ROUTE 31 STUDY SITE

	TRADITIONAL - 1984			NEW GORE - 1985			NEW GORE - 1986		
	A	B	C	A	B	C	A	B	C
DAY 1	167	1411	11.8	232	1361	17.0	221	1416	15.6
DAY 2	164	1302	12.6	163	1443	11.3	297	1580	18.8
DAY 3	161	1422	11.3	198	1517	13.1	160	1554	10.3
TOTAL	492	4135	11.9	593	4321	13.7	678	4550	14.9

ROUTE 206 STUDY SITE

	TRADITIONAL - 1985			NEW GORE - 1986		
	A	B	C	A	B	C
DAY 1	3	1243	0.2	11	1377	0.8
DAY 2	13	1237	1.1	16	1524	1.0
DAY 3	13	1490	0.9	9	1249	0.7
TOTAL	29	3970	0.7	36	4150	0.9

LEGEND
A = Erratic Maneuvers B = Total Volume
C = Percent Making Erratic Maneuvers

TABLE 11 SUMMARY, ACCIDENTS IN PASSING ZONES

	1984			1985			1986			1-6/1987		
	A	B	C	A	B	C	A	B	C	A	B	C
RT. 31 CONTROL SITE	5	3	NA	1	0	NA	1	0	NA	0	0	NA
RT. 31 STUDY SITE	0	0	NA	2	0	0	2	0	0	1	0	0
RT. 206 STUDY SITE	6	0	NA	9	0	NA	11	0	0	3	0	0
RT. 206, MP. 16.6-17.5	4	0	NA	2	0	0	10	0	0	3	0	0
RT. 206, MP. 11.1-12.5	6	1	NA	5	0	0	3	0	0	3	0	0
TOTAL	21	4	NA	19	0	0	27	0	0	10	0	0

LEGEND
A = Total Accidents B = Passes on the Right Accidents
C = Going Through the Gore Accidents
NA = Not Applicable, No Gore Present

passes of platoon leaders on the right and passes on the right within the platoon among followers. Illegal maneuvers at NJ-31 control site increased in successive years. However, with the introduction of the new passing zone gore design, the number of total passes on the right was reduced greatly at both study sites. Therefore, it is concluded that the objective of reducing this illegal maneuver was met.

Table 9 summarizes the total passes of platoon leaders by day and by study period for all the sites. The number of total passes at the NJ-31 control site increased, whereas the percentage of followers passing their platoon leaders stayed relatively constant. At the NJ-206 study site, both the number and the percentage of followers passing their platoon leaders increased slightly when the new passing gore design was introduced. However. at the NJ-31 study site, both the number and their percentage decreased in successive years after the new gore was introduced. Results for this variable were mixed and were not the expected one—that is, increasing passing efficiency at both study sites.

Table 10 gives the number of vehicles crossing the white edgeline near the transition from two lanes to one lane at the end of the passing zones, by day and study period. The data include all vehicles that went through the passing zone because there was no way to determine original platoons at the end of the passing zone. At the NJ-31 control site, the number of vehicles crossing the edgeline stayed relatively the same in successive years, but the percentage of the vehicles doing it decreased. At each of the study sites, both the number and percentage increased slightly. The large difference between the findings at NJ-31 and at NJ-206 was believed to be caused by the geometrics of the sites. NJ-206 was straight and level, with low edgeline crossings, while end conditions on Rt. 31

TABLE 12 SUMMARY, ILLEGAL MANEUVERS (VEHICLES)

	ROUTE 31 CONTROL SITE		
	TRADITIONAL 1984	TRADITIONAL 1985	TRADITIONAL 1986
PASSES OF PLATOON LEADER ON THE RIGHT	108	127	165
OTHER PASSES ON THE RIGHT WITHIN THE PLATOON	110	173	232
PLATOON LEADERS STAY LEFT OR GO THROUGH GORE	378	442	505
PLATOON LEADERS FOLLOW GORE & RETURN TO LEFT LANE	NA	NA	NA
FOLLOWERS GOING THROUGH THE GORE	NA	NA	NA
TOTAL	596	742	902

	ROUTE 31 STUDY SITE		
	TRADITIONAL 1984	NEW GORE 1985	NEW GORE 1986
PASSES OF PLATOON LEADER ON THE RIGHT	28	4	13
OTHER PASSES ON THE RIGHT WITHIN THE PLATOON	14	6	1
PLATOON LEADERS STAY LEFT OR GO THROUGH GORE	168	6	7
PLATOON LEADERS FOLLOW GORE & RETURN TO LEFT LANE	NA	20	25
FOLLOWERS GOING THROUGH THE GORE	NA	386	382
TOTAL	210	422	428

	ROUTE 206 STUDY SITE	
	TRADITIONAL 1985	NEW GORE 1986
PASSES OF PLATOON LEADER ON THE RIGHT	129	6
OTHER PASSES ON THE RIGHT WITHIN THE PLATOON	138	29
PLATOON LEADERS STAY LEFT OR GO THROUGH GORE	207	4
PLATOON LEADERS FOLLOW GORE & RETURN TO LEFT LANE	NA	8
FOLLOWERS GOING THROUGH THE GORE	NA	378
TOTAL	474	425

NA = Not Applicable, No Gore Present

were downhill and curved to the right, with thus a greater number of edgeline crossings. It was concluded that the increase did not represent a practical change. This result was the one anticipated.

Table 11 presents the actual number of accidents that occurred from January 1984 to the middle of 1987 at the three sites studied. Also included are two other sections of NJ-206 that had the passing zone gore installed but were not studied. Only four accidents associated with passing on the right occurred during this 3.5-year period at the five sites. Three of these occurred in 1 year at the control site. No accidents associated with the new passing gore were found. Such small accident numbers do not justify concerns about the safety problems and possible accidents caused by the two illegal maneuvers.

The best approach to take in determining the effect of the new passing zone gore design on the traffic characteristics is to look at how this study's objectives were met.

The first objective was to determine whether the new gore created a more efficient passing zone by allowing more followers to pass their platoon leaders. The results are mixed. The data in Table 9 indicate that the efficiency increased slightly for the NJ-206 study area but decreased tremendously for the NJ-31 study area.

As for the second objective, reducing the number of illegal and erratic maneuvers, Table 12 summarizes all illegal maneuvers for the three sites by study period. The maneuvers are different for the two striping plans. With the traditional striping, passes on the right and staying left (leaders only) are the illegal maneuvers; with the new gore striping, going through the gore (leaders and followers) are the illegal maneuvers. Again, mixed results were found. The NJ-206 study site improved slightly, but the NJ-31 study site worsened markedly.

Finally, no safety problems were associated with the new passing zone gore. There were hardly any accidents at the sites, and none was associated with the new gore. However, the belief that passing on the right is unsafe was not found to be justifiable from this study either, again because of the low number of accidents.

RECOMMENDATIONS

1. Because of the increase in illegal and erratic maneuvers and the decrease in passing efficiency at the NJ-31 site, the new passing zone gore design should not be implemented at this time on passing zones of one-half mile or less in length.
2. Because illegal and erratic maneuvers and passing efficiency remained relatively the same or improved slightly at the NJ-206 site, the new passing zone gore design could be implemented on passing zones of greater than one-half mile.
3. Further research dealing with such factors as total passes and different passing zone lengths should be carried out to determine more specifically the effects of this new design on the traffic characteristics.

ACKNOWLEDGMENT

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REFERENCES

1. G. T. Bennett. Hatched junction markings on rural roads. *Journal of the Institution of Highway Engineers*, April 1972.
2. *High-Occupancy Vehicles on the Garden State Parkway*. Wilbur Smith and Associates, New Haven, Conn.; Division of Research and Demonstration, New Jersey Department of Transportation, Trenton, 1978.
3. J. L. Botha and A. D. May. *A Decision-Making Framework for the Evaluation of Climbing Lanes on Two-Way, Two-Lane Rural Roads*. Institute of Transportation Studies, University of California, Berkeley, Oct. 1982.

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Development of Highway Alignment Information from Photolog Data

W. D. BERG, J. CHOI, AND E. J. KUIPERS

The Wisconsin Department of Transportation operates an instrumented vehicle that collects both photolog and digitized geometric data on roadway bearing, grade, and cross slope. These data are recorded every 0.01 mile. The objective of the research was to develop the methodology and computer software necessary to transform the digitized roadway data into a highway alignment data base for use in inventory studies, deficiency analyses, and preliminary design studies. The software created locates horizontal curves and provides estimates of the radius, length, superelevation, and maximum recommended speed. A vertical profile can be plotted, and the location of segments with either stopping or passing sight distance restrictions due to horizontal and vertical alignment are identified. The software was tested and validated by using as-built plan and profile data for a case study highway.

The Wisconsin Department of Transportation (WisDOT) operates an instrumented vehicle equipped with a Techwest Photologging System (*1*) to collect both photolog and digitized geometric data for highways under its jurisdiction. This information (referred to in this paper as the datalog file and shown in Figure 1) is recorded at 0.01-mile intervals, or every 52.8 ft. The department's goal is to create a videodisk and computer software system for retrieving and displaying high-resolution photographic images concurrently with corresponding geometric data. This system would provide highway planners and engineers with a high-quality, accessible data base for conducting inventory studies, deficiency analyses, and preliminary design studies.

The objective of this research was to develop the methodology and computer software necessary to transform the digitized roadway data into a highway alignment data base, which was to include information on horizontal and vertical alignment as well as sight distance restrictions.

DATA BASE

When the driver of the instrumented vehicle arrives at a highway segment to be measured, he or she manually sets the highway name, county name, and beginning odometer reading. A plus or minus sign is specified for the odometer reading, according to the direction of travel. If one direction is listed as plus, the opposite direction will have a minus sign. Usually, a one-directional datalog file is created because the northbound and southbound runs will have different file names. This information is stored in the datalog file.

The bearing of the vehicle is automatically determined by a gyro compass. Percent grade is recorded by using an electrically powered gyro as a reference platform. Transverse slope is recorded by using the grade gyro with a sensor perpendicular to the grade sensor. This sensor indicates the total transverse slope of the road, plus any slope contributed by the vehicle due to static or dynamic loading. This bearing and slope information is recorded every 0.01 mile.

DEVELOPMENT OF VERTICAL ALIGNMENT

A two-step methodology was used to develop vertical alignment information from the datalog file. The first step involved adjustment of the original grade readings; the second involved calculation of the elevation of the highway at each record.

Because the raw data collected by the instrumented vehicle contained a substantial amount of noise, due largely to the bouncing motion of the vehicle, it was necessary to smooth these data before attempting to establish the elevation at each record along the highway section. The grade readings from the datalog file were plotted against the true grades, using several case study sites for which a current set of plan and profile drawings were available. Regression analysis was then performed to develop a set of adjustment models that would be able to eliminate much of the noise in the raw data. It was found that the errors were a function of whether the vehicle was operating on a positive or negative grade. The resulting regression models are

$$\overline{G} = 0.0744 + 0.847\,(+G) \tag{1}$$

$$\overline{G} = -\,0.151 + 0.980\,(-G) \tag{2}$$

where

$+G$ = positive grading reading,
$-G$ = negative grade reading, and
$\overline{G}$ = adjusted grade.

The R^2 for the models are 0.937 and 0.926, respectively.

The software uses the regression models to adjust the grade readings, after which the elevations at each successive record are calculated as

$$E(i) = E(i-1) - 52.8 \sin[\overline{G}(i)/100] \tag{3}$$

where $E(i)$ is the elevation at record i and $\overline{G}(i)$ is the adjusted grade at record i. A graphical plot of the highway profile may

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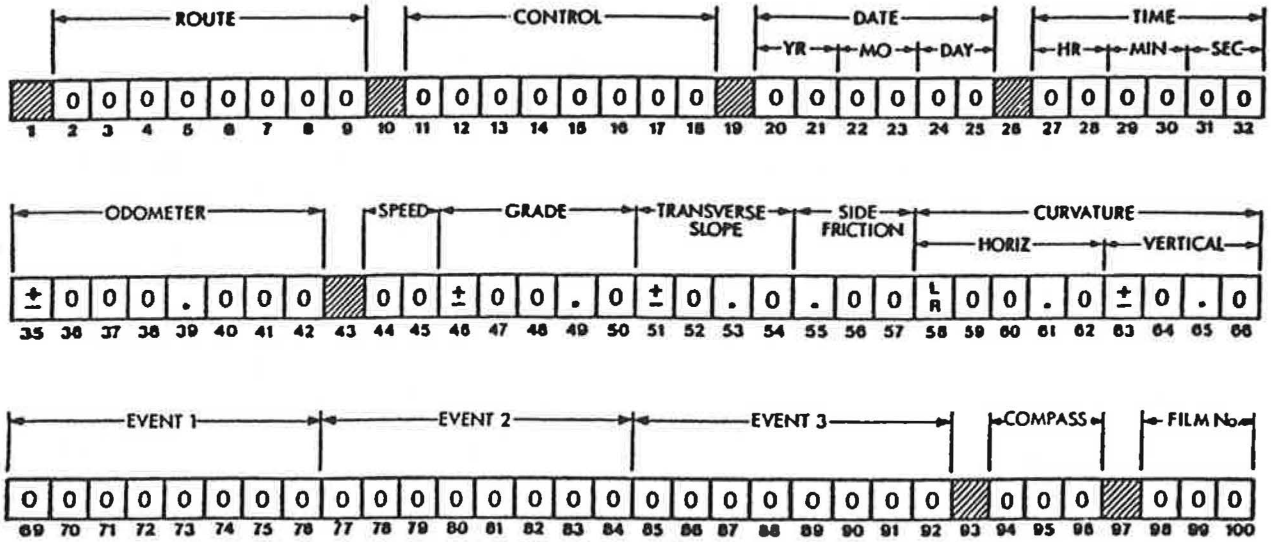

FIGURE 1 Format of the datalog file.

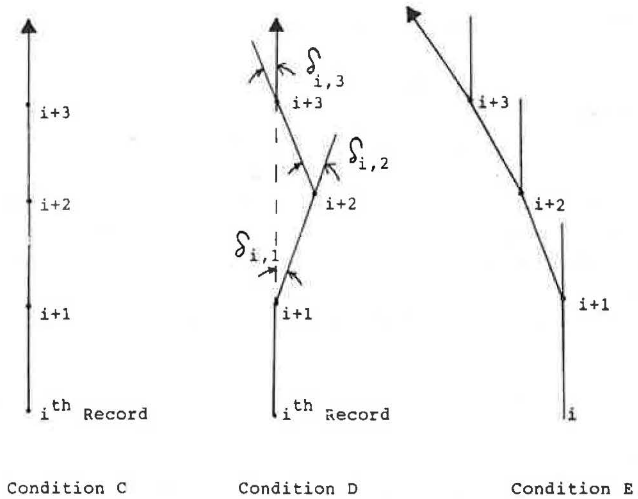

FIGURE 2 Horizontal alignment test conditions.

then be requested. The location of the points of curvature (PC) and tangency (PT) could not be successfully programmed because of the difficulty in separating the inherent remaining noise in the adjusted grades from the gradual grade changes associated with the beginning or end of a parabolic curve. Further research is underway to overcome this problem.

DEVELOPMENT OF HORIZONTAL ALIGNMENT

The algorithm developed to locate the points of curvature (PC) and tangency (PT) for a horizontal curve involves the following steps. The conditions tested in Steps 3 through 5 are illustrated in Figure 2.

1. The program reads the compass bearing at every record i and filters out inconsistent records by checking three consecutive compass bearings: C_i, C_{i+1}, and C_{i+2}. If $C_i = C_{i+2}$, then C_{i+1} is set equal to C_i, and the process is repeated for the next record.
2. The program calculates changes in compass bearings for each record between the current record i and record $i + 3$. These are then referenced with respect to the current record:

$$\delta_{i,1} = C_i - C_{i+1} \tag{4}$$

$$\delta_{i,2} = C_{i+1} - C_{i+2} \tag{5}$$

$$\delta_{i,3} = C_{i+2} - C_{i+3} \tag{6}$$

where C_i, C_{i+1}, C_{i+2}, C_{i+3} are compass bearings for records i, $i + 1$, $i + 2$, and $i + 3$, respectively, and i is the current record.

3. If $\delta_{i,1} * \delta_{i,2} < 0$, then record i is identified as being on a tangent. Select the next record, and return to Step 2. Otherwise, go to Step 4.
4. If $\delta_{i,1} = \delta_{i,2} = 0$, then it is assumed that the driver of the datalog vehicle has made a steering error on a tangent section. Select the next record and return to Step 2; otherwise go to Step 5.
5. If neither condition 3 nor condition 4 is satisfied, it is assumed that record $i + 1$ is within a curve. However, the program does not pick the PC until it encounters the PT of a curve. The program finds the PT first, and it counts the frequency of events, n, where the condition e was satisfied. The program then establishes the PC by subtracting n from the PT.
6. If $\delta_{i,3} = 0$, go to Step 7; otherwise select the next record and return to Step 2.
7. The PT is found at record $i + 3$. Subsequently, the program resets all flags associated with a curve by increasing the curve number. In addition, the highway name, county name, and direction of a curve are reported in this step.

Once the PC and PT for a horizontal curve are found, the program computes the following parameters. The calculation of radius of curvature accounts for the position of the center of gravity of the datalog vehicle that is assumed to be approximately 6 ft to the right of the highway centerline.

$$L = 5280(ODM_{PT} - ODM_{PC}) \tag{7}$$

$$R = [L/(C_{PT} - C_{PC})](180/\pi) + 6 \quad \text{right-hand curve}$$
$$R = [L/(C_{PT} - C_{PC})](180/\pi) + 6 \quad \text{left-hand curve} \tag{8}$$

$$D = 5730/R \tag{9}$$

where

L = length of curve (ft);
ODM_{PT}, ODM_{PC} = odometer readings at the PT and PC;
C_{PT}, C_{PC} = compass bearing at the PT and PC, respectively;
D = degree of curvature; and
R = radius (ft).

The superelevation of the curve, e, is obtained by taking the average of the transverse slope readings from the datalog file:

$$e = \frac{TS}{(ODM_{PT} - ODM_{PC})100} \tag{10}$$

where ODM_{PC} and ODM_{PT} are odometer readings for the PC and PT, and TS is the sum of the transverse slope readings on a horizontal curve.

After these three parameters are calculated, the program determines the maximum recommended speed on the curve. The speed at which a driver will travel around a horizontal curve when not restrained by a vehicle ahead is dependent primarily on two factors: the driver's sense of safety as judged from the sight distance ahead and the comfort as judged by the centrifugal force. On a horizontal curve the centrifugal force tending to keep the vehicle in a straight path is opposed by the side frictional force developed at the area of contact between the tire and the roadway surface. Additionally, the speed may be influenced by roadside markings of the safe speed. Assuming that sight distance is adequate, the maximum recommended speed is governed by the radius of curvature, the superelevation, and the allowable coefficient of friction between the tires and pavement surface:

$$V = [15R(e + f)]^{0.5} \tag{11}$$

where

V = maximum recommended speed (mph),
e = superelevation, and
f = allowable side friction.

The program calculates the maximum recommended speed by using the computed values for radius and superelevation and a representative limiting side friction value of 0.12.

When two horizontal curves are within two records (105.6 ft) of each other, the two curves are merged. The program also identifies concurrent records for a highway segment. A concurrent record is created when the driver of the datalog vehicle encounters a highway segment that carries more than one route designation and for which a datalog file was created during a prior run. The program prints a report of the horizontal alignment data (Figure 3). Only the horizontal curve sections of a highway segment are listed in the horizontal alignment table; therefore, records not included in these curve sections are on tangent sections. A listing of any concurrent records is provided at the bottom of the horizontal alignment information table.

SIGHT DISTANCE EVALUATION

The availability of the horizontal alignment data permitted identification of those locations having either a stopping or passing sight distance restriction. For purposes of this research, the American Association of State Highway and Transportation Officials (AASHTO) definition (*2*) of stopping sight distance and both the Manual on Uniform Traffic Control Services (MUTCD) and the WisDOT definitions (*3,4*) of no-passing zone sight distance were used.

Minimum stopping sight distance (MSSD, in feet) is expressed as

$$MSSD = 1.47\ Vt + V^2/[30/(f \pm G)] \tag{12}$$

where

V = design speed (mph),
t = perception-reaction time (2.5 sec),
f = coefficient of friction, and
G = grade (percent).

Driver's eye height and object height are assumed to be 3.5 and 0.5 ft, respectively.

HORIZONTAL ALIGNMENT INFORMATION

HIGHWAY	DIR	COUNTY	DATE	PC(Odm)	PT(Odm)	D	R(FT)	L(FT)	PI	e	DSN SPEED
STH017	N	LIN...01	84- 4-29	27.52	27.54	1.9R	3025.9	105.6	27.53	.036	84.
STH017	N	LIN...01	84- 4-29	27.73	27.75	1.9R	3025.9	105.6	27.74	.030	83.
STH017	N	LIN...01	84- 4-29	28.07	28.09	1.9R	3025.9	105.6	28.08	.017	79.
STH017	N	LIN...01	84- 4-29	28.35	28.37	1.9R	3025.9	105.6	28.36	.014	78.
STH017	N	LIN...01	84- 4-29	28.99	29.04	1.9L	3025.9	264.0	29.02	.041	86.
STH017	N	LIN...01	84- 4-29	29.06	29.18	2.1L	2793.1	633.6	29.12	.037	81.
STH017	N	LIN...01	84- 4-29	29.76	29.83	1.9L	3025.8	369.6	29.80	.019	79.
STH017	N	LIN...01	84- 4-29	29.85	29.87	1.9L	3025.9	105.6	29.86	.018	79.
STH017	N	LIN...01	84- 4-29	29.92	29.97	1.9L	3025.7	264.0	29.94	.012	77.
STH017	N	LIN...01	84- 4-29	29.99	30.03	1.9L	3025.9	211.2	30.01	.012	77.

FIGURE 3 Typical horizontal alignment information table.

The marking of a no-passing zone at a horizontal or vertical curve is based on the availability of a minimum passing sight distance at the prevailing off-peak 85th percentile speed of traffic (Table 1). The sight distance is evaluated using 3.5-ft eye and object heights.

Vertical Curve Sight Distance Deficiencies

The algorithms that were developed to locate stopping and passing sight distance deficiencies are virtually identical, except for the different sight distances and object heights used. The task was to locate the beginning and ending points of each zone in which there was either a stopping or passing sight distance restriction.

The algorithm used to locate vertical sight distance deficient segments involves the following steps:

1. Specify the design or 85th percentile speed and determine the appropriate sight distance requirement.
2. Calculate the elevation of Station (X + sight distance).
3. Calculate the elevation at each record between Station (X) and Station (X + sight distance).
4. Calculate the elevation of the sight line at each record between Station (X) and Station (X + sight distance).
5. If for any record between Station (X) and Station (X + sight distance) the sight line elevation is less than the profile elevation, then the sight distance is considered to be inadequate. When this condition takes place for the first time, the beginning point of a sight distance deficient segment is established.
6. Keep checking Step 4 until the available sight distance again becomes greater than the minimum value. At a record where this condition is satisfied, the ending point of a sight distance deficient segment is established.

The length of a sight distance deficient segment is calculated on the basis of the above beginning and ending points. The highway name and county name associated with a sight distance deficient segment are found by using the datalog records.

After the passing sight distance deficient segments are identified, a criterion is used to determine whether the distance between two successive segments is so short that the two segments should be connected. For this purpose, the MUTCD criterion of 400 ft is used.

The percentage of no-passing zones is then computed and displayed at the bottom of a no-passing zones summary table. Figure 4 illustrates the output reports.

TABLE 1 NO-PASSING ZONE CRITERIA

85th Percentile Speed (mph)	Minimum Passing Sight Distance (ft) MUTCD	Wis. MUTCD
30	500	
35		528
40	600	
45		686
50	800	
55		845
60	1,000	
65		1,108
70	1,200	

- VERTICAL ALIGNMENT INFORMATION -

* PASSING SIGHT DISTANCE DEFICIENCY SEGMENTS *

HIGHWAY	DIR	COUNTY	FROM(Odomtr)	TO(Odomtr)	L(FT)
STH017	N	LIN...01	27.56	27.74	950.4
STH017	N	LIN...01	27.99	28.06	369.6
STH017	N	LIN...01	28.46	28.61	792.0
STH017	N	LIN...01	28.99	29.06	369.6

% NO PASSING ZONE = 15.61(%)
TOTAL LENGTH DRIVEN = 15892.8(FT)

S STOPPING SIGHT DISTANCE DEFICIENCY SEGMENT S

HIGHWAY	DIR	COUNTY	FROM(Odomtr)	TO(Odomtr)	L(FT)
STH017	N	LIN...01	27.53	27.67	739.2
STH017	N	LIN...01	28.41	28.57	844.8

FIGURE 4 Typical vertical alignment information table.

Horizontal Curve Sight Distance Deficiencies

Sight distance at a horizontal curve is measured along the center of the lane of travel. The sight line is, however, a straight line that connects the driver's position and the end of the required sight distance. An algorithm was designed to check if an obstruction cuts off the sight line. The required middle ordinate, a distance from the sight line to the arc of the vehicle path, varies according to the driver's position and the radius of a horizontal curve. If the calculated middle ordinate, m, is less than a user-specified actual lateral clearance on a curve, then the current highway location is considered to have inadequate sight distance.

As described previously, the computer program calculates the radius of a curve with respect to the centerline of the highway. The user is asked to specify the available middle ordinate, m, with respect to the centerline of the highway. Therefore, to determine horizontal sight distance deficient segments, the program adjusts the radius and the middle ordinate to account for the position of the datalog vehicle in the center of the inside lane. Three sight distance evaluation procedures were designed to reflect the possible roadway geometry downstream from the driver's assumed position: tangent-to-curve, curve-to-curve, and tangent-to-tangent.

The tangent-to-curve situation involves a driver on a tangent section in advance of a curve and the end of the sight line on the curve section. This condition is illustrated in Figure 5. To determine a required m, the program performs a series of computations for a side angle ϕ. Also, two simultaneous equations are solved to find lengths of line segments $\overline{A \cdot D_1}$ and $\overline{A \cdot D_2}$. Once these values are determined, the program calculates m to the center of the vehicle path as follows:

$$\theta = \frac{SD - \overline{A \cdot PC}}{R'} \quad \text{(radians)} \tag{13}$$

$$R' \cos \alpha = R' - \overline{A \cdot D_1} \sin \phi \tag{14}$$

$$\cos \alpha = 1 - (\overline{A \cdot D_1}/R') \sin \phi \tag{15}$$

hence,

$$\alpha = \cos^{-1} [1 - (A \cdot D_1/R') \cdot \sin \phi] \tag{16}$$

$$\delta = \alpha + \theta \tag{17}$$

$$m' = R'[1 - \cos (\delta/2)] \tag{18}$$

and

$$\begin{aligned} R &= R - 6 \quad \text{right-hand curve} \\ R' &= R + 6 \quad \text{left-hand curve} \end{aligned} \tag{19}$$

When both the driver and the end of the sight line fall within a curve section as illustrated in Figure 6, the middle ordinate, m', can be obtained using the following expression:

$$m' = R'[1 - \cos \theta/2] \tag{20}$$

To determine m' under the condition where both the driver and the end of the sight line are on two different tangent sections separated by a horizontal curve, it is necessary to calculate the X and Y positions shown in Figure 7. The length of a long chord that connects X and Y depends on the side angle ϕ. The procedure used to determine the middle ordinate

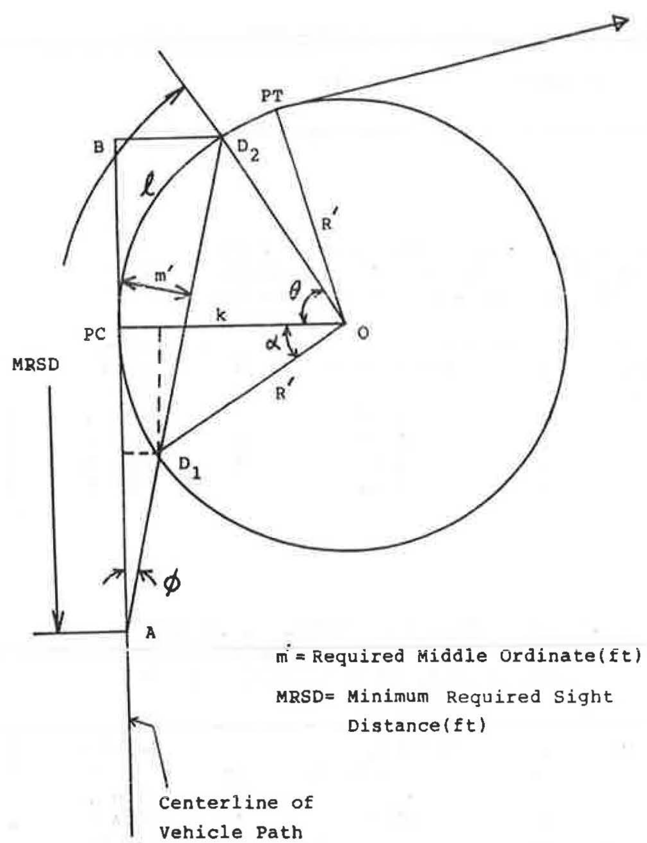

FIGURE 5 Tangent-to-curve sight distance situation.

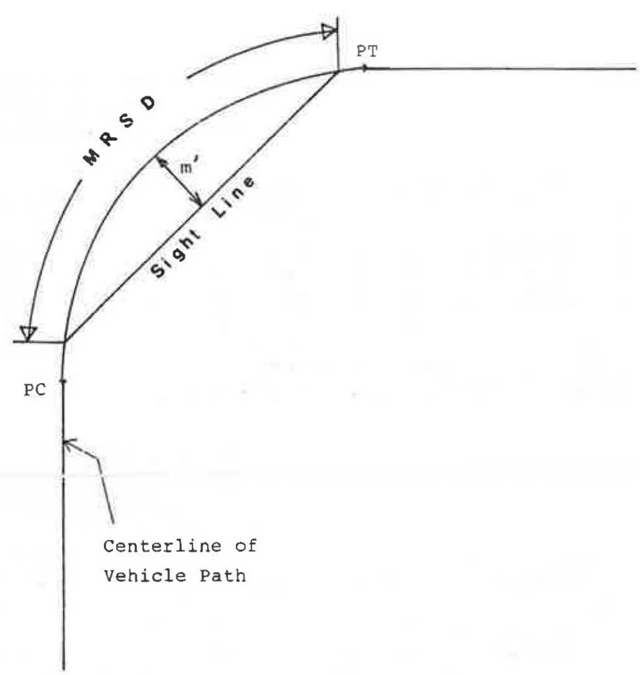

FIGURE 6 Curve-to-curve sight distance situation.

when the driver and the end of sight line are on different tangent sections is as follows:

1. Calculate the central angle of the horizontal curve:

$$\theta = l/r' \quad \text{(radians)} \tag{21}$$

$$x = R' \sin \theta \tag{22}$$

$$y = R'(1 - \cos \theta) \tag{23}$$

2. Calculate the x and y components for the straight line $\overline{\mathrm{PT} \cdot B}$:

$$a = \overline{\mathrm{PT} \cdot B} \sin \theta \tag{24}$$

$$b = \overline{\mathrm{PT} \cdot B} \cos \theta \tag{25}$$

3. Determine the angle ϕ:

$$\overline{DB} = a + y \tag{26}$$

$$\phi = \tan^{-1} \left[\frac{\overline{DB}}{\overline{A \cdot \mathrm{PC}} + \overline{D \cdot \mathrm{PC}}} \right] \tag{27}$$

4. Calculate the angle α:

$$\alpha = \cos^{-1} [1 - (\overline{A \cdot D_1}/R') \sin \phi] \tag{28}$$

5. Calculate the angle γ. This requires the same procedure shown in Steps 1 through 4, except that the driver is assumed to be at point B, as shown in Figure 8:

$$\eta = (90 - \phi) - (90 - \beta) = \beta - \phi \tag{29}$$

$$\overline{B \cdot D_1} = \frac{(2\,\overline{B \cdot \mathrm{PT}} \cos \eta + 2\,R' \sin \eta) + \sqrt{Z}}{2} \tag{30}$$

$$\overline{B \cdot D_2} = \frac{(2\,\overline{B \cdot \mathrm{PT}} \cos \eta + 2\,R' \sin \eta) - \sqrt{Z}}{2} \tag{31}$$

where

$$Z = (2\,\overline{B \cdot \mathrm{PT}} \cos \eta + 2\,R' \sin \eta)^2 - 4\,\overline{B \cdot \mathrm{PT}}^2 \tag{32}$$

Hence,

$$\overline{B \cdot D_2} \sin \eta = R'\,(1 - \cos \gamma) \tag{33}$$

$$\gamma = \cos^{-1} \left[1 - \frac{\overline{B \cdot D_2} \sin \eta}{R'} \right] \tag{34}$$

6. Compute the required middle ordinate, m':

$$\delta - \theta + \alpha + \gamma \tag{35}$$

$$m' = R'\,(1 - \cos \delta)/2 \tag{36}$$

$$\begin{aligned} m &= m' + 6 \quad \text{right-hand curve} \\ m &= m - 6 \quad \text{left-hand curve} \end{aligned} \tag{37}$$

A majority of stopping or passing sight distance deficient segments can be located using these three situations. How-

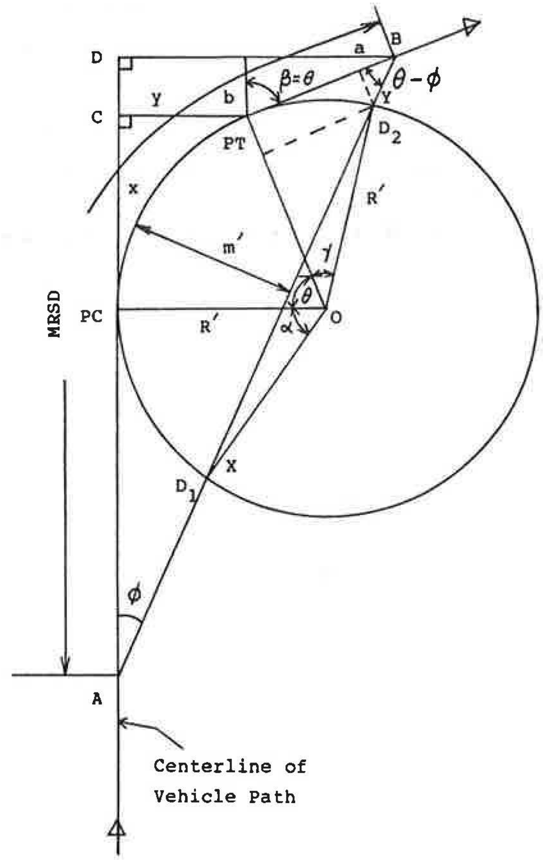

FIGURE 7 Tangent-to-tangent sight distance situation.

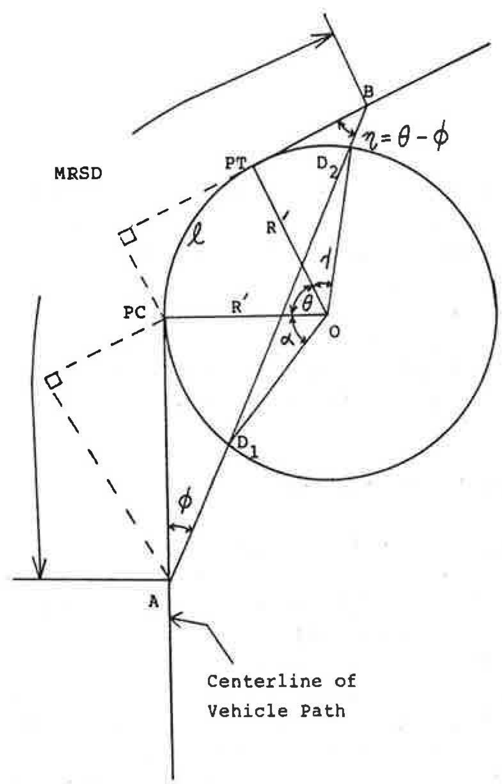

FIGURE 8 Diagram for angle γ.

= HORIZONTAL PASSING SIGHT DEFICIENCY SEGMENT =

HIGHWAY	DIR	COUNTY	FROM(Odmtr)	TO(Odmtr)	L(FT)
STH017	N	LIN...01	28.93	29.06	686.4
STH017	N	LIN...01	29.69	29.92	1214.4

FIGURE 9 Typical horizontal passing distance sight deficiency information.

TABLE 2 COMPARISON HORIZONTAL CURVE DATA

	Manual						Computer Program					
Curve #	PC (odm)	PT (odm)	D (°)	e	R (ft)	L (ft)	PC (odm)	PT (odm)	D (°)	e	R (ft)	L (ft)
1	27.46	28.48	1	.03	5730	5383	27.52	27.54	2	.04	3026	106
							27.73	27.75	2	.03	3026	106
							28.07	28.09	2	.02	3026	106
							28.35	28.37	2	.01	3026	106
2	28.97	29.19	2	.04	2865	1138	28.99	29.04	2	.04	3026	264
							29.06	29.18	2	.04	3026	634
3	29.72	30.09	1.5	.04	3820	1915	29.86	29.83	2	.02	3026	370
							29.85	29.87	2	.02	3026	106
							29.92	29.97	2	.01	3026	264
							29.99	30.03	2	.01	3026	211

ever, when two or more horizontal curves are separated within the required sight distance, the computer program may not produce reasonable results. When two successive horizontal passing sight distance deficient segments are separated by a tangent less than the 400-ft MUTCD criterion, they are combined to form a single no-passing zone. Finally, the program generates a summary of the horizontal passing sight distance deficient segments as illustrated in Figure 9.

SOFTWARE VALIDATION

The computer program was applied to a case study highway section to verify whether the software could accomplish its intended functions. The computer-generated information was compared with manual solutions developed near the as-built plan and profile drawings for the case study highway.

A comparison of the horizontal curve data is provided in Table 2. The relatively long curves were divided into several shorter curves by the computer program. This is probably due to the inherent noise in the data and the fact that such noise would have a more noticeable effect on low-degree curves.

A comparison of the sight distance deficient segments is presented in Tables 3–5. When the computer-generated data were visually compared with photolog images, a very close correspondence was immediately apparent with respect to the beginning and ending points of horizontal and vertical curves, as well as no-passing zones.

CONCLUSIONS

Overall, the evaluation of the software indicated that it was possible to develop reasonably accurate highway alignment and sight distance data from the photolog system used by the Wisconsin Department of Transportation. The data are of sufficient accuracy to be used in an inventory data base or geographic information system and for deficiency analysis and preliminary project development. The system has the further advantage of eliminating the need for an existing set of plan and profile drawings. It is only necessary to operate the instrumented vehicle over the highway of interest and then to process the data using the software package.

TABLE 3 COMPARISON OF STOPPING SIGHT DISTANCE DEFICIENT SEGMENTS

	Manual			Computer Program			Difference		
Zone No.	From (odm)	To (odm)	*L* (ft)	From (odm)	To (odm)	*L* (ft)	From (odm)	To (odm)	*L* (ft)
1	27.57	27.75	950	27.53	27.67	739	−0.04	−0.08	−211
2	28.48	28.62	739	28.41	28.57	845	−0.07	−0.05	+106

TABLE 4 COMPARISON OF NO-PASSING ZONES AT VERTICAL CURVES

	Manual			Computer Program			Difference		
Zone No.	From (odm)	To (odm)	*L* (ft)	From (odm)	To (odm)	*L* (ft)	From (odm)	To (odm)	*L* (ft)
1	27.57	27.75	950	27.56	27.74	950	−0.01	−0.01	0
2	27.95	28.06	581	27.99	28.06	370	+0.04	0.00	−221
3	28.43	28.60	898	28.46	28.61	792	+0.03	+0.01	−106
4	28.97	29.05	422	28.99	29.06	370	+0.02	+0.01	−52

TABLE 5 COMPARISON OF NO-PASSING ZONES AT HORIZONTAL CURVES

	Manual			Computer Program			Difference		
Zone No.	From (odm)	To (odm)	*L* (ft)	From (odm)	To (odm)	*L* (ft)	From (odm)	To (odm)	*L* (ft)
1	28.89	29.10	1,108	28.91	29.08	898	+0.02	−0.02	−210
2	29.62	29.98	1,901	29.67	29.94	1,426	+0.05	−0.04	−475

REFERENCES

1. *Techwest Photologging System Operating Manual.* Techwest Enterprises Ltd., Vancouver, B.C., Canada, July 1984.
2. *A Policy of Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1984.
3. *Manual of Uniform Traffic Control Devices.* FHWA, U.S. Department of Transportation, 1978.
4. *Wisconsin Manual on Traffic Control Devices.* Wisconsin Department of Transportation, Madison, 1984.

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