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Relations Between Geometric Design and Operations

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Subject Areas

22	Highway Design
51	Highway Safety
53	Traffic Control and Operations

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FOREWORD

The seven papers and one abridgment in this RECORD are concerned with the relationships between selected geometric design features and resultant costs in terms of operations and safety. The design considerations range from decision-making among major interchange types to such specific design elements as median configurations, bridge geometrics, freeway ramps and lane drops, intersection turn lanes, and special lanes for transit and automated vehicles. Most traffic engineers, a wide range of design specialists, and safety specialists can expect to find material of value in the papers in this RECORD.

In the opening paper, Garner and Deen report the findings of their study of accident histories as related to median widths, slopes, and configurations. They conclude that wider medians are generally safer medians, that raised medians have a number of undesirable characteristics, and that medians with irregular width should be designed only with adequate clear zones. Median shoulder requirements are also discussed.

Balachandra and Dais propose an analytical method for the design of automatically controlled vehicle guideways. They present equations describing the levels of discomfort for motions along horizontal and vertical curves, including effects of superelevation. They show that the analytical approach that includes only the centripetal component of acceleration and jerk can lead to considerable error in measuring discomfort.

In a California Division of Highways study, Martin, Newman, and Johnson looked at density (vehicles per lane-mile) as a measure of effectiveness of operations resulting from a number of geometric design alternatives. Different ramp types and spacing, collector-distributor roads, auxiliary lanes, and lane drops are included, and they concluded that freeway designs offering freedom of choice to drivers will result in smoother and more efficient operation.

Using a freeway traffic evaluation model, May and James looked at total passenger time as a measure of effectiveness of various freeway design alternatives. Combining these measures with annual costs for the various designs permits determination of the cost-effectiveness of each design alternative. An example problem is discussed.

Concerned with the number and severity of accidents involving vehicles striking bridges, Hilton collected accident history data on several bridges in Virginia. Geometric factors determined to contribute to the accident experiences included bridge roadway width, curving road approach alignment, curving bridge alignment, adjacent intersections and lane drops or transitions, surface condition, grade, and others. A number of remedial measures are suggested.

Next Taylor reviews present practices in trade-offs between costs and operational efficiency in the design process for major interchanges. He describes a level-of-merit design concept aimed at separating the operational and safety considerations from costs in the decision-making process. He also suggests that trade-off analyses cannot be isolated but must be interwoven with the entire design process.

King and Plummer used laboratory and field tests to study traffic signal indication needs for the intersection case where the left-turning movement is terminated while the through movement continues. Starting in the laboratory with 19 possible indications, four were selected for field evaluation; of these four, one was recommended after analysis of driver performance data.

In an abstract of a larger work, Robertson and Diewald present results of an analysis of an express-bus-on-freeway demonstration project in the Washington, D.C., area. Their analysis led them to conclude that the project was feasible and acceptable to the public.

ELEMENTS OF MEDIAN DESIGN IN RELATION TO ACCIDENT OCCURRENCE

Gordon R. Garner and Robert C. Deen, Kentucky Department of Highways

The purpose of this study was to compare the accident histories of different median types and to provide verification of generally recommended median widths and slopes. A major limitation of the analyses was the small number of possible combinations of median width and cross slope available for study. The analyses reported provided evidence from accident histories to support the general assumption that wider medians are safer medians. It was indicated that medians should be a minimum of 30 to 40 ft wide for high-speed facilities and that flat slopes should be provided; 4:1 slopes are inadequate for medians less than 60 ft wide. There was an indication that 6:1 or flatter slopes should be used. Raised medians provided an unsuitable vehicle recovery area on rural highways and were also undesirable from the standpoint of roadway surface drainage. The irregular Interstate highway medians that result from independent roadway alignment should be used only with adequate clear zones in the median. Shoulders 12 ft wide should be provided where guardrail is to be used.

•HIGHWAY DESIGN is a dynamic process. Design standards are continually being revised and modernized. Consequently, new highways of today are safer, longer lasting, and more efficient than ever before. However, as traffic volumes and the number of accidents increase, many design features once considered adequate have proved to be inadequate. Changes are constantly being made to provide safer highways.

The divided roadway was first conceived as a safety measure. It was hypothesized that roadways separated by a median of some sort would reduce head-on accidents. Medians can be found that are raised, depressed, traversible, nontraversible, earth, concrete, with and without barriers, with and without plants, and so on. Median widths vary from 2 to more than 100 ft.

In studies by Hurd (1), Telford and Israel (2), Crosby (3), and Billion (4), no definite relationship between accident rates and widths of various types of medians was found. Although the overall superiority of wider medians could not be shown, it was apparent that cross-the-median, head-on collisions were reduced by increasing the width (1, 3). Largely for this reason, the use of wider medians became commonplace.

Hutchinson (5), in a comprehensive study of encroachments on several medians, found that steep (4:1) slopes cause driver overreaction and vehicle control problems. He concluded that an absolute minimum median width of 30 ft is required under ideal conditions of mild slopes and no median obstacles. Evidence indicated that any irregularities in the median due to crossovers, drainage structures, bridge piers, or other appurtenances could destroy the effectiveness of the median. Stonex (6) concluded from tests conducted at the General Motors Proving Ground that slopes of 6:1 are the minimum required for off-the-road safety.

It was thus generally accepted that wide, gently sloping medians were superior. The current Interstate System standard 60-ft wide median with 6:1 slopes is an example of this type. However, many roads are still being built with narrower medians. Although widths may exceed the minimum urged by Hutchinson, the mild cross slope requirements have not always been met.

The purpose of this study was to provide information concerning the accident histories of various median types to verify minimum requirements for width and cross section. Previous accident studies failed to disclose significant relationships between median width and accident rates. Those studies did not recognize or control several important variables that were controlled in this study. The efforts here are to compare median types on rural, four-lane, fully controlled-access facilities with similar geometrics other than median types. This study gives information on the operational performances of several medians and offers persuading analyses with respect to the design or styling of medians.

PROCEDURE

Previous median accident studies (1, 2, 3, 4) used data bases involving very short study sections, generally less than 5 miles and frequently less than 1 mile in length. Such short road sections were used in an effort to obtain larger sample sizes. However, the results obtained from such a data base are subject to suspicion due to the sensitivity of accident rates to a single accident occurrence and the inability to obtain reasonably accurate volume information for such small sections. The only variable between locations should be median type, but this is not the case. Thus, local roadway peripheral and environmental factors have a greater effect on short sections.

The effects of roadway geometric features must not be ignored when accident rates of different road sections are compared. Things such as pavement width, shoulder width, grades, curves, coefficient of friction, sign location, access control, and other design standards could have a greater effect than the variables under study, i.e., median type and width. The geometric features of all road sections in the study should be as similar as possible.

As previous research has shown (7), great care must be exercised when accident records are used for evaluation purposes. When different agencies are involved in patrolling a given road, variations in reporting practices, training of personnel, and amount of surveillance can produce incomplete and inconsistent accident records. Inadequacies found in individual reports involve inaccurate locations, poor sketches, and the like. There can be frequent variations in the number, type, and percentage of accidents reported. The natural variability of accident records can, therefore, make any results obtained from accident studies extremely unreliable, especially in determining the causality of any particular accident.

Experience with accident records provided by the Kentucky State Police indicated a high quality and consistency in reporting methods, especially when compared to other agencies in the state. It was, therefore, decided to select road sections patrolled exclusively by the Kentucky State Police. This would allow a certain degree of uniformity in reporting methods not present in previous studies.

In summary, it was desirable that study sections in an accident study be as long as possible, have a similar degree of access control, have similar roadway geometric features, and be patrolled exclusively by one agency.

The toll road and Interstate System in Kentucky made it possible to select long road sections with these characteristics. More importantly, a variety of median types could be studied. The characteristics of the road sections selected are given in Table 1. The similarity in geometric features other than the median should be noted. Figures 1, 2, and 3 show details of the median types studied.

Four years of accident data were secured for those roads opened in 1965 or earlier. Only 3-year data were obtained for the Bluegrass Parkway and I-65 in Simpson County, both of which opened in 1966. Two-year data were used for the section of I-75. Traffic volume data were available for 2 or 3 of the study years for the Interstate roads. Complete monthly summaries for all toll roads were used. Missing volume data for the Interstate road sections were extrapolated from the available data.

To produce results that would indicate a valid comparison between median types required a strict definition of what constituted a "median-involved accident." Some accidents involving the median were not representative of whether the median was a cause or contributor to the accident. Specifically, there were two types of median-involved accidents that were not considered to be median accidents. Accidents occurring at median crossovers were, in a sense, "caused" by the crossover, considered to be a geometric feature separate from the median. Therefore, accidents at median cross-

Table 1. Characteristics of study road sections.

Road	Length (miles)	Median Type	Median Width (ft)	Access Control	Speed Limit (mph)	Pavement Width (ft)	Pavement Cross Slope (in./ft)	Width of Outside Shoulders (ft)
I-64, Clark County	35	Depressed	60	Full	70	24	3/16	12
I-64, Shelby County	12	Depressed	60	Full	70	24	3/16	12
I-64, Franklin County	17	Irregular	Varies	Full	70	24	3/16	12
I-65, Hardin County	27	Depressed	60	Full	70	24	3/16	12
I-65, Simpson County	26	Depressed	60	Full	70	24	3/16	12
I-75, Scott County	19	Irregular	Varies	Full	70	24	3/16	12
Kentucky Turnpike	39	Raised	20	Full	70	24	3/16	12
Western Kentucky Turnpike	127	Raised	30	Full	70	24	3/16	12
Mountain Parkway	43	Deeply depressed	36	Full	70	24	3/16	12
Bluegrass Parkway	75	Deeply depressed	36	Full	70	24	3/16	12

Figure 1. Details of Interstate highway medians.

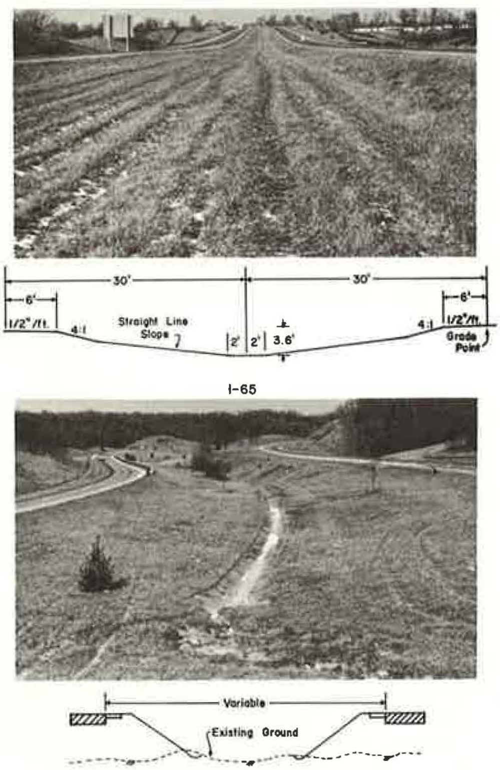
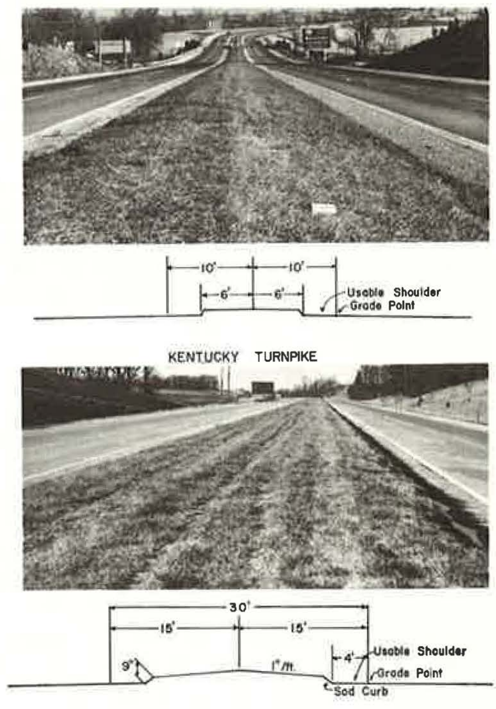


Figure 2. Details of raised medians.



overs were separated and subjected to special analysis (8). There were also a few accidents that involved collisions with fixed objects in the median, specifically bridge piers and bridge ends. These collisions generally resulted in fatal or severe injury accidents and would, therefore, prejudice the results where otherwise the median may have performed satisfactorily. This type of accident was also not considered a median accident. Generally, all other accidents involving the median were included.

Accident events per 100 million vehicle-miles were used as a basis for comparison. Stewart (9) reported that the use of accident rates based on vehicle-miles assumes that all driving involves some exposure to accident hazards, the exposure to accident hazards is proportional to miles driven, and the degree of exposure is the same for all drivers. For the long, rural road sections in this study, these assumptions were generally valid, and accident rates were used for comparison purposes with some confidence.

RESULTS AND DISCUSSION

Any given accident is the result of a complex interaction among roadway, driver, and vehicle. The contribution of any given factor to the causality of the accident will vary with the conditions. Dart and Mann (10) suggested that the driver is a major cause in 80 to 90 percent of accidents, the highway in 40 to 50 percent, and the vehicle in 10 percent. There is widespread disagreement on the relative percentages of each factor. A concept suggested by Bellis (11) would support a much higher contribution by the roadway and off-the-road environment. Humans cannot be improved on very much as drivers, Bellis maintains. Thus, accidents can only be prevented by removing the source of impact. The improved roadway and off-the-road environment provided by Interstate highways and the resulting low accident and severity rates (12) support this view. Thus, it would be logical to assume that the roadway contributes to as many as 75 to 80 percent of all accidents in rural situations.

However, given that roadway geometrics cannot explain all the variability of accident rates, this study attempts only to indicate the influence and importance of two geometric features, median width and cross section. The influence of other variables will be indicated where possible.

Effects of Median Width

The results of this study do support the premise that wider medians are safer medians. Figure 4 shows total accident rate versus width of median. There is a general decline in accident rate with increasing median width. Total accident severity rate (Fig. 5) also decreases with increasing median width. A breaking point or "leveling off" seems to occur between 30 and 40 ft.

Another indicator of median effectiveness in providing a recovery area for out-of-control vehicles is shown in Figure 6. There is a statistically significant decrease in the percentage of total median-involved accident vehicles that crossed the median as median width increased. Wider medians provide a more adequate recovery area and a greatly reduced potential for head-on accidents. Hurd (1) found a similar relationship.

Hutchinson's study (5) of vehicle encroachments on the median concluded that medians should be a minimum of 30 ft wide with gentle cross slopes and no obstacles. Hurd (1) concluded that a median should be at least 40 ft wide to reduce the possibility of head-on collisions. Webster and Yeatman (13) found that at least 33 ft of separation was needed to eliminate disability glare from high-beam headlights. The results obtained here support a minimum width of 40 ft; however, other elements of the median, such as cross slopes and the presence of obstructions and irregularities, can have a greater effect on safety of a median than width.

Effects of Median Cross Section

The beneficial effects of wide medians can be completely negated by steep slopes. Figure 7 shows median accident rate versus width of median. The adverse effects of steep 4:1 and 3:1 cross slopes of 36-ft, deeply depressed medians are clearly indicated by the high median accident rate. The cross slopes of the 20-, 30-, and 60-ft medians

Figure 3. Details of deeply depressed medians.

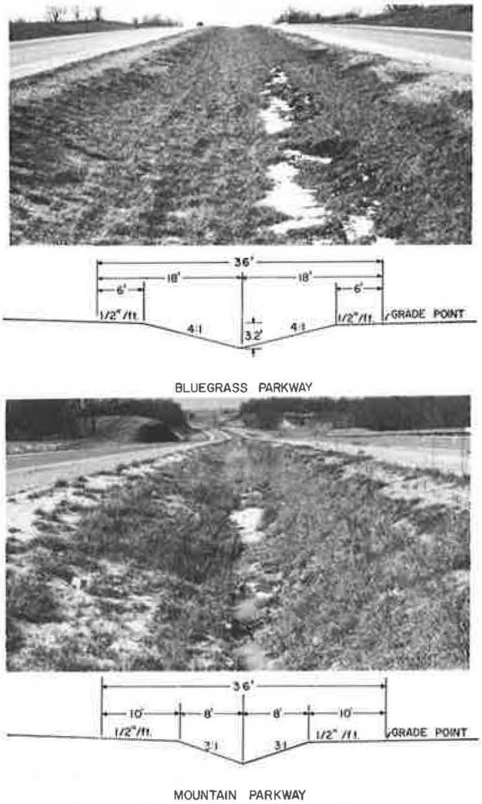


Figure 4. Total accident rate versus median width.

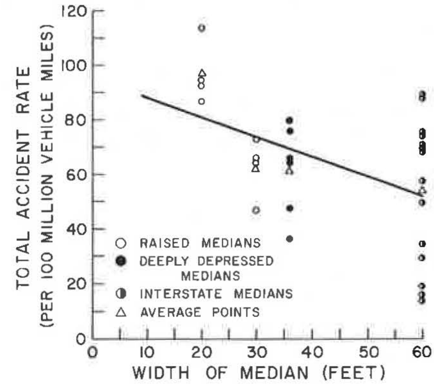


Figure 5. Total accident severity rate versus median width.

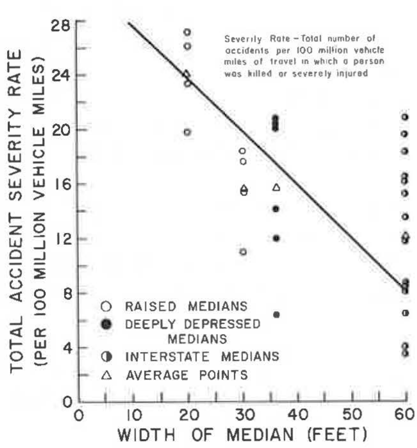
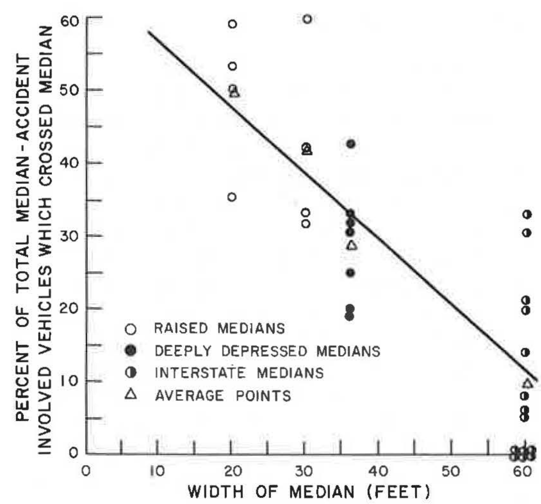


Figure 6. Percentage of total median-involved accident vehicles that crossed the median versus median width.



are relatively mild when compared to the 36-ft medians. Medians with steep slopes do not provide reasonable recovery areas and are often a hazard in themselves. The higher median accident severity rate for these deeply depressed medians is shown in Figure 8.

The deeply depressed median results in a disproportionate number of vehicles overturning. The rate of median accidents resulting in one or more vehicles overturning is much greater for the Bluegrass Parkway and Mountain Parkway (Table 2). These roadways have deeply depressed medians with 4:1 and 3:1 slopes. Figure 9 shows that the severity of accidents for depressed medians is related to whether the vehicle overturns.

Reported studies wherein mild cross slopes are recommended are many. Hutchinson (5) found that steep (4:1) slopes had an adverse effect on vehicle encroachments and estimated that a 40-ft depressed median with 10:1 slopes would allow more than 90 percent of all encroaching vehicles to recover safely. Stonex (6) recommended 6:1 slopes as being minimal from his GM Proving Ground tests. Figure 10 shows the percentage of grade change at the centerline for various slopes. The 4:1 slopes involve a 50 percent grade change, whereas the 6:1 slopes now used on Interstate roads involve a 34 percent grade change. The curve begins to level off at 10:1 slopes. The results from this study strongly support the previous recommendations for mild cross slopes.

The raised medians in this study (20 and 30 ft wide) were found to have several disadvantages not entirely explained by narrower width. Raised medians seemed to have a higher number of cross-median accidents. Both raised median types have a sod curb a few feet from the edge of the pavement. Many drivers were found to hit this curb and overreact, causing an accident. Table 3 gives the rate of hit-median, lost-control accidents by type of median. Raised medians also do not provide storage area for snow removal purposes. Moisture will "bleed" from raised medians onto the roadway for days. In cold weather, this allows hazardous ice spots to form.

There are many sections of Interstate highway where a separate, independent roadway is provided in each direction. These sections have a median of varying width and highly irregular nature. Figures 11 and 12 show that the sections of Interstate highway with an irregular median have much higher median and total accident rates and severity rates. The median shoulders are only 6 ft wide. This places the guardrail only 6 ft from the edge of pavement versus the 12 ft provided on the right side. Whereas the typical section of Interstate highway has a relatively flat, gently sloping recovery area, the divided sections in many cases provide no recovery area at all. In the future, use of independent roadway sections, clear zones, and recovery space should be provided. Also 12-ft shoulders should be used where guardrail is to be installed.

Effects of Volume

A synopsis of studies concerning the effect of traffic volume on accident rates (14) indicates that a correlation does exist between volume and accidents. In general, accident rates will increase with increasing volume. However, the increases are obvious only when very large differences in volume are being considered. For the volume ranges considered in this study, there is no obvious correlation of total and median accident and severity rates and volume (expressed as average daily traffic). Other variables have more effect than volume.

That accident rates may increase with increasing volume can be partially explained by the increase in multivehicle collisions with increasing volume. The data from this study are shown in Figure 13. There is an increasing trend showing that multivehicle accidents, as a percentage of the total, increase with volume. Such a relationship was previously reported by Belmont (15).

Other factors that may account for any increase in accident rate with volume include enforcement levels and age of roadway as related to road roughness and skid resistance. It is general practice for enforcement levels to be adjusted to traffic volumes. In other words, high-volume roads are more heavily patrolled than are low-volume roads. Thus, it is more likely that minor accidents will be reported on high-volume roads.

It has been shown by Burchett and Rizenbergs (16) that skid resistance decreases with accumulated vehicle passes for most pavements. Road roughness has also been

Figure 7. Median accident rate versus median width.

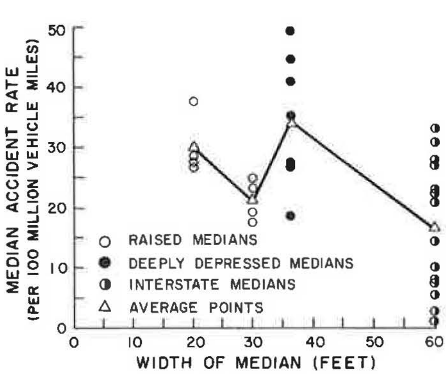


Figure 8. Median accident severity rate versus median width.

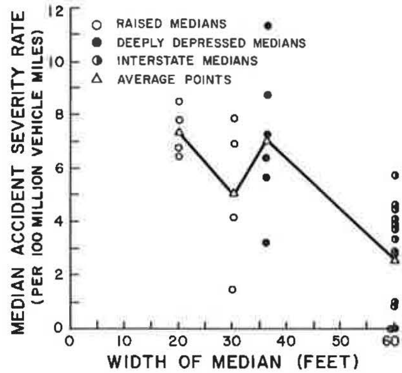


Table 2. Median accidents involving vehicles that overturn.

Road	Median Width (ft)	Median Type	Percent	Rate[a]
Kentucky Turnpike	20	Raised	10.7	2.88
Western Kentucky Turnpike	30	Raised	24.0	4.75
I-64 and I-65[b]	60	Depressed	20.1	2.42
Bluegrass Parkway	36	Depressed, 4:1 slopes	34.7	10.31
Mountain Parkway	36	Depressed, 3:1 slopes	46.0	16.47

[a]Number of accidents per 100 million vehicle-miles. [b]Average.

Figure 9. Number of fatal and severe injury accidents versus number of median accidents in which vehicle overturned.

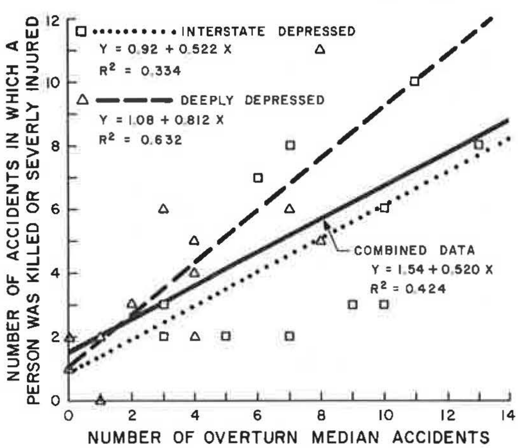


Figure 10. Percentage of grade change at the centerline for various slopes.

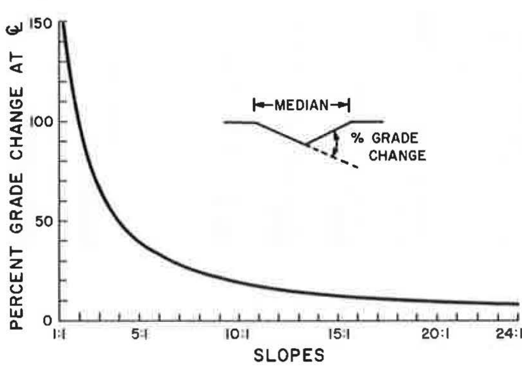


Table 3. Median accidents involving vehicles that hit the median and lost control.

Road	Median Width (ft)	Median Type	Left Shoulder Width (ft)	Percent	Rate[a]
Mountain Parkway	36	Depressed	10	4.8	1.70
Bluegrass Parkway	36	Depressed	6	11.2	3.34
I-64 and I-65[b]	60	Depressed	6	16.5	1.99
Kentucky Turnpike	20	Raised	4	19.2	5.16
Western Kentucky Parkway	30	Raised	4	30.2	5.99

[a]Number of accidents per 100 million vehicle-miles. [b]Average.

Figure 11. Total and median accident rates for Interstate highway medians.

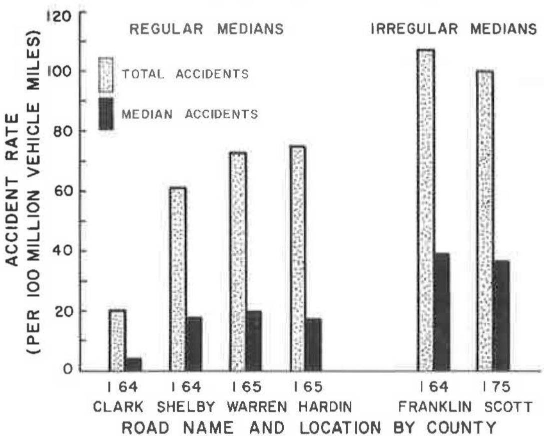


Figure 12. Total and median accident severity rates for Interstate highway medians.

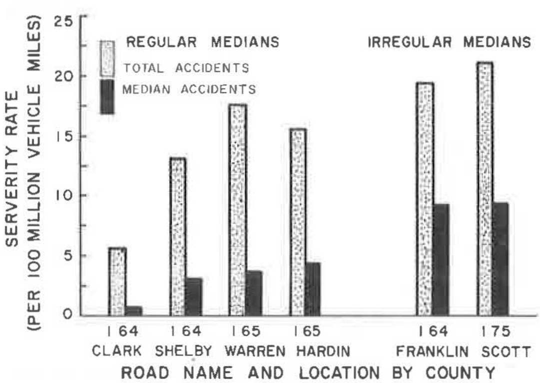
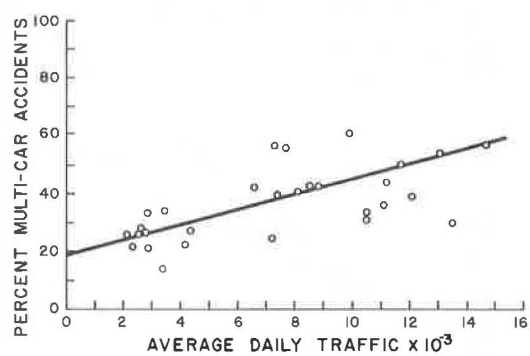


Figure 13. Relationship of multivehicle accidents and traffic volume.



shown to increase with years since construction. The lower skid resistance and higher roughness index are as likely to account for an increase in accident rates as is volume.

The results of this study appear to be unaffected by differences in traffic volume. That accident rates do generally increase with increasing volume may be explained by volume effects such as the increase in multivehicle accidents or by volume and age-related phenomena such as the decrease in skid resistance and the increase in road roughness.

Effects of Other Variables

The number of variables that can influence the occurrence of accidents has been shown to be very great. There are any number of variables that can affect accident rates, but the relative effects of each cannot be accurately determined. These variables are likely to account for much of the deviation of accident statistics. A few of these variables will be discussed for illustrative purposes. Weather, bearing of roadway, and enforcement levels are three such factors.

That weather should influence the occurrence of accidents is intuitively obvious. However, few studies have given this full consideration. Hutchinson (17) found good correlation between rainfall and intersection accidents in Lexington, Kentucky. An attempt was made herein to correlate accidents with the occurrence of precipitation, but no apparent correlation was found. The inherent precipitation variables (intensity and duration), coupled with the variability in length of road sections affected and traffic volume at the time of rainfall, were probably responsible for the inability to obtain significant findings. More precise data collection methods need to be established to accurately determine the effects of weather on accidents on long, rural road sections.

The bearing of the roadway was found to have a significant effect on the occurrence of accidents in a given direction. In all cases except one, the majority of accidents occurred in the southbound direction. Figure 14 shows a directional analysis of each of the road sections. Given is the percentage of total median accidents that occurred in a direction. That these percentages are different from the expected 50-50 split is significant at the 95 percent level using a t-test. The actual geographical orientation of the study roads is shown in Figure 15. The probable explanation for this phenomenon is related to visibility and glare. Drivers heading into the sun are more likely to be affected by glare and thus are exposed to a greater accident risk.

The variation in patrolling levels found on Kentucky's Interstate and toll roads is given in Table 4. In 1968, all troopers who patrol Interstate or toll roads were given a questionnaire to complete. The values given in Table 4 were calculated from state troopers' estimates of actual time per week spent in patrolling each road. Generally, high-volume roads are more frequently patrolled than low-volume roads. This could result in the reporting of a greater number of minor accidents on high-volume roads.

Evaluation of Medians by Function

The functions of medians on divided highways with complete control of access have been listed (18). An evaluation of median types included in this study is presented in Table 5. The narrow, raised medians satisfy very few of the necessary functions of medians. Deeply depressed medians do not provide an adequate recovery space, and this has been shown to be a significant failing. Only the wide, gently sloping Interstate medians adequately satisfy all functions.

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study was to compare the accident histories of different median types and to provide verification of generally recommended minimum widths and slopes. The major limitation of this analysis was the small number of possible combinations of median width and cross slope available for study. For example, only one width of median with a 4:1 side slope was available for inclusion in the sample. The individual effects of width and cross slope were therefore not determined. However, all combined effects evident in the results of this analysis support the contentions from previous research that wider, flatter medians are safer.

Table 4. 1968 levels of enforcement on Interstate and toll roads.

Road	1968 Approximate ADT	Enforcement Level (man-hour/ mile/week)
Western Kentucky Turnpike	2,800	0.9
Mountain Parkway	3,600	1.5
Bluegrass Parkway	4,400	1.0
I-64, Clark County	6,000	2.2
I-65, Simpson County	8,500	5.2
I-65, Hardin County	11,000	7.7
I-64, Shelby County	12,500	8.0
Kentucky Turnpike	13,500	7.7
I-75, Scott County	17,500	6.8

Figure 14. Effect of roadway bearing on accident occurrence in southbound direction.

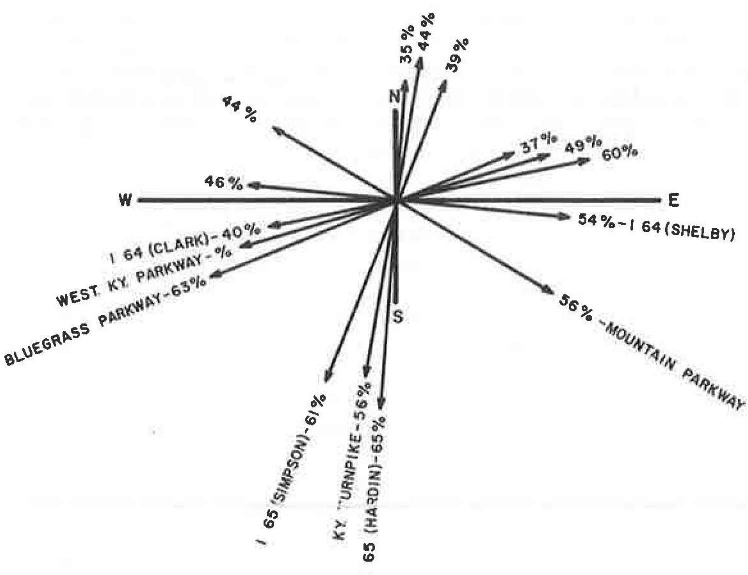
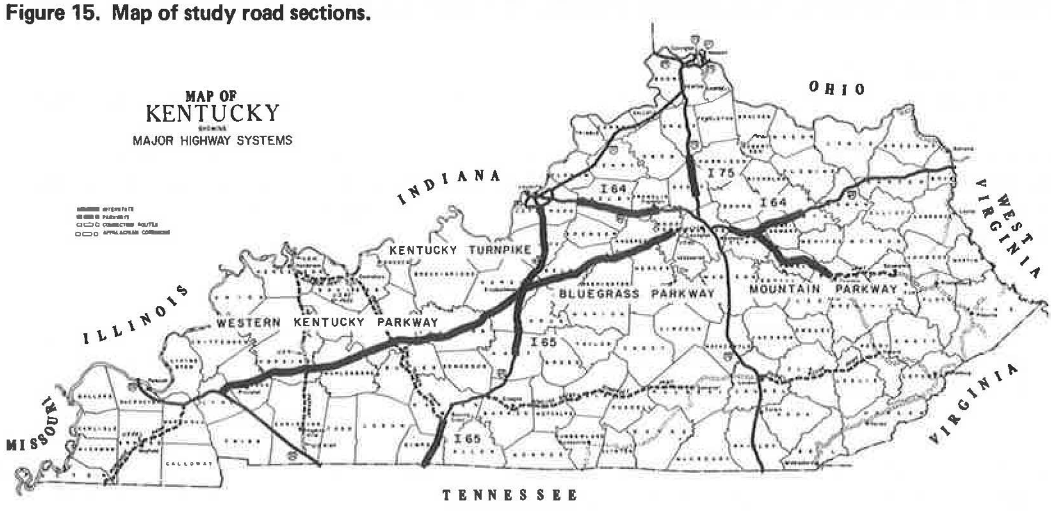


Figure 15. Map of study road sections.



The analysis reported herein provided documentary evidence from accident histories to support the reasonably known and intuitively presumed rule that wider medians are safer medians. It suggested that medians should be a minimum of 30 to 40 ft wide for high-speed facilities.

Factual support was provided for previous research conclusions that indicated that flat slopes should be provided; 4:1 slopes are inadequate. For medians less than 60 ft wide, there was sufficient cause to use 6:1 or flatter slopes. Specifically, 36-ft medians, such as have been used on Kentucky's toll roads, should have 6:1 or flatter slopes, even though this will require some special drainage considerations.

Raised medians provided an unsuitable vehicle recovery area on rural highways and were undesirable from the standpoint of roadway surface drainage. The use of curbed, raised medians in urban areas should be reexamined inasmuch as the deficiencies of raised medians apparent in this study may be applicable.

The irregular Interstate highway medians that result from independent roadway alignment design should be used only with adequate clear zones in the median. Shoulders 12 ft wide should be provided where guardrail is to be used.

This study, because similar roadway environments allowed the effects of median type to be separated and analyzed effectively, has conclusively justified the premise that providing a clear, gently sloping, off-the-road environment is one of the best ways to reduce accidents and accident severity on modern divided highways.

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GEOMETRY OF AUTOMATICALLY CONTROLLED VEHICLE GUIDEWAYS FOR COMFORT

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An analytical method is proposed for the design of guideway elements subject to jerk and acceleration limits. The method generalizes the limits on lateral jerk and acceleration familiar in highway design to the fore-aft and vertical modes. It is proposed that, under combinations of these modes, the magnitudes of the jerk and effective acceleration vectors must be limited. Equations describing the level of discomfort are derived for both constant and variable-speed motion along horizontal and vertical curves. The equations explicitly include the effect of superelevation. A four-spiral switch for use in a personalized rapid transit system is analyzed in detail. It is found that a length reduction of more than 25 percent is possible by using superelevation. A right angle turn composed of two spirals and a circular arc is detailed. It is found that the use of superelevation can reduce the magnitudes of the effective jerk and acceleration vectors by more than 50 percent. Finally, a four-spiral grade change is detailed. Discomfort is investigated and compared for constant speed and zero fore-aft thrust motion. It is shown that the usual analytical approach that includes only the centripetal component of acceleration and jerk can lead to considerable error in predicting discomfort.

•IT IS well known that human beings experience discomfort when subjected to acceleration and jerk. Considerable experimental work has been done to evaluate the effects of particular types of acceleration and jerk and the data have been used in the geometric design of automotive test tracks (1, 2), highways (3), and railroads (4). Physiological mechanisms associated with acceleration discomfort have also been extensively studied (5). Most of the work has been concerned with accelerations and jerks in one mode: lateral, fore-aft, or vertical. There has been very little work describing discomfort levels due to combinations of these modes, although the importance of such work has been noted (5, 6), and the authors are aware of one experimental study on the subject (7).

Evaluation of discomfort in combined modes has not been important for the geometric design of highways and railroads. Only the lateral comfort limits are used for the geometric design of highways and railroads. Curvatures in the vertical plane are limited by visibility requirements and grades by engine power and surface friction limitations. In recent days, however, there has been considerable interest in automatically controlled vehicle systems like personalized rapid transit (PRT) and dual mode transit. In such systems, vehicles run on fixed guideways with off-line stations (6, 8, 9).

Because of electronic sensing, novel types of vehicle suspension (e.g., air or magnetic), and a uniform power-weight ratio, the highway limitations on grade and vertical curvatures would not apply. Thus, the geometric design of guideway elements for these networks will to a large degree be determined by comfort criteria. Furthermore, in networks with closely spaced stations and interchanges, space limitations would be very stringent. Such limitations require that geometric design incorporate quantitative criteria involving combinations of fore-aft, lateral, and vertical accelerations and jerks.

In this paper a mixed mode measure of discomfort is developed by defining an "effective" acceleration vector $\underset{\sim}{a}_e$ as shown in Figure 1. $\underset{\sim}{a}_e$ is the difference between the total acceleration (including gravity) and the acceleration associated with self weight, $-g\underset{\sim}{n}$. In three dimensions, the fore-aft component of acceleration, if present, would be vectorially included in $\mathbf{a}_e$. The magnitude of $\mathbf{a}_e$ is termed the acceleration discomfort index and is assumed to be a direct quantitative measure of the amount of acceleration discomfort. $\mathbf{J}_e$, the time derivative of $\mathbf{a}_e$ in a frame of reference attached to the vehicle, is termed the effective jerk vector. The magnitude of $\mathbf{J}_e$ is assumed to measure the amount of jerk discomfort. From these assumptions, it is possible to derive expressions for the amount of jerk and acceleration discomfort in terms of the vehicle speed, speed change, and guideway centerline geometry and superelevation.

Three examples of geometric design of components of a PRT network are presented as applications of the methods of the paper. The techniques described are also applicable to the geometric design of other transportation systems. The first example is concerned with a four-spiral switch that connects an off-line acceleration or deceleration lane with the main line. Dais used the four-spiral combination horizontally as an unbanked switch for PRT networks (12). In this paper we investigate the possibility of decreasing the length of such a switch by banking the guideway.

A right angle turn using a circular arc with spiral easement at both ends is described next. The discomfort along such a curve and the effect of superelevation are investigated for constant speed motion along the curve.

The use of spiral curves along with a straight inclined line to achieve a grade change makes up the third example of the paper. Motion along such a curve is considered both with zero fore-aft thrust and with constant speed, and it is shown that the two differ significantly in terms of the discomfort caused. The work generalizes previous work (1) by using formulas that include the fore-aft components as well as the normal modes. The error introduced by considering only the centripetal component of acceleration (1) is also investigated.

THEORETICAL DEVELOPMENT

Consider a subject in a moving vehicle in some preferred configuration like sitting or lying down. The basic principle postulated here is that, in the presence of accelerations in addition to that of gravity, the discomfort experienced by the subject is a function of the vectorial difference between the total body force on the subject and a datum corresponding to his self weight. Although this idea has been stated by others (1, 5), its mathematical consequences in connection with transportation guideway design have not been fully explored.

To make the above concept more explicit, consider a triad of mutually perpendicular unit vectors fixed in the vehicle with $\mathbf{f}$ tangent to the guideway centerline in the direction of travel of the vehicle, $\mathbf{n}$ normal to the vehicle, and $\boldsymbol{\ell}$ in the lateral direction of the vehicle. n and $\boldsymbol{\ell}$ are shown in Figure 1. Then we define the effective acceleration vector by

$$\mathbf{a}_e = -g\mathbf{n} - \mathbf{g} + \mathbf{a} \tag{1}$$

where

$\mathbf{a}$ = acceleration of the vehicle and
$\mathbf{g}$ = acceleration of gravity.

It is assumed that $|\mathbf{a}_e|$ determines the level of acceleration discomfort. In equation form,

$$\phi = |\mathbf{a}_e| = (\mathbf{a}_e \cdot \mathbf{a}_e)^{1/2} \tag{2}$$

where ϕ is termed the acceleration discomfort index. Analogously, it is postulated that jerk discomfort is essentially due to the subject's having to adjust to a changing force field and consequently may be analyzed by considering an effective jerk vector $\mathbf{J}_e$ defined by

$$\mathbf{J}_e = \frac{d\mathbf{a}_e}{dt} \tag{3}$$

In Eq. 3 the differentiation is assumed to be performed in the $\underset{\sim}{n}$, $\underset{\sim}{\ell}$, and $\underset{\sim}{f}$ frames of reference. The level of discomfort due to jerk is expressed by means of a jerk discomfort index ψ. This index is assumed to be given by the magnitude of the effective jerk vector. In equation form,

$$\psi = |\underset{\sim}{J}_e| = (\underset{\sim}{J}_e \cdot \underset{\sim}{J}_e)^{1/2} \tag{4}$$

Equations 2 and 4 tacitly assume that all components are equally significant in causing discomfort. That is, a radial jerk or acceleration of a certain magnitude would cause the same level of discomfort as would a normal or fore-aft jerk or acceleration of that same magnitude. [A more general mathematical theory in which this assumption is not made is presented elsewhere (10).] The degree of validity of this assumption is a matter for experimentation, and the answer will depend significantly on the amount of lateral restraint incorporated in the seating. The methods developed in the present paper, however, remain valid even though the assumption is not strictly true. Geometric designs will be obtained by limiting the maximum amount of ϕ and ψ, denoted respectively as ϕ_{max} and ψ_{max}, along a curve. Then one simply sets $\phi_{max} = \min(a_f, a_\ell, a_n)$ and $\psi_{max} = \min(J_f, J_\ell, J_n)$, where a_f, a_ℓ, and a_n are acceleration limits in respectively the fore-aft, lateral, and normal modes and J_f, J_ℓ, and J_n are the respective jerk limits in those modes. It may also be remarked that the present treatment excludes discomfort caused by motion about a body axis (e.g., rolling) and by the simultaneous presence of acceleration and jerk.

Experimental work is needed to determine acceptable levels of ϕ_{max} and ψ_{max}. The present work will present solutions based on $\phi_{max} = 8$ ft/sec^2 and $\psi_{max} = 8$ ft/sec^3. These numbers are consistent with reported experimental work (6) but are somewhat higher than the 0.15 g allowable lateral acceleration suggested by AASHO (3). In any case, it is felt that levels this high would be suitable if passengers were seated and had good lateral restraints.

EFFECT OF SUPERELEVATION

Banking of curves to completely eliminate lateral force on the vehicle is well established in highway, railroad, and automotive test track design. We will show that the bank angle corresponding to zero lateral force also minimizes the discomfort indexes defined by Eqs. 2 and 4. Consider the centerline curve of the guideway to lie in the horizontal plane, and let the angle of bank be θ. Let $\underset{\sim}{k}$ be a unit vector in the vertically upward direction and $\underset{\sim}{p}$ a unit vector in the horizontal direction as shown in Figure 1.

From elementary dynamics (11), the acceleration of the center of mass of the vehicle is given by

$$\underset{\sim}{a} = -v^2\kappa\underset{\sim}{p} + \dot{v}\underset{\sim}{f} \tag{5}$$

where $\kappa = 1/R$ is the centerline curvature, R is the radius of curvature, and v is the speed of the vehicle. From Figure 1 and Eq. 1 it follows that

$$\begin{aligned} \underset{\sim}{a}_e &= -g\underset{\sim}{n} + g\underset{\sim}{k} - v^2\kappa\underset{\sim}{p} + \dot{v}\underset{\sim}{f} \\ &= (-g + g\cos\theta + v^2\kappa\sin\theta)\underset{\sim}{n} + (g\sin\theta - v^2\kappa\cos\theta)\underset{\sim}{\ell} + \dot{v}\underset{\sim}{f} \end{aligned} \tag{6}$$

and therefore

$$\phi = |\underset{\sim}{a}_e| = [2g^2(1 - \cos\theta) + v^4\kappa^2 - 2gv^2\kappa\sin\theta + \dot{v}^2]^{1/2} \tag{7}$$

It may be readily verified that ϕ is minimized by the choice

$$\theta = \theta^* = \tan^{-1}\frac{v^2\kappa}{g} \tag{8}$$

By setting $\underset{\sim}{a} = -v^2\kappa \underset{\sim}{p}$ in Figure 1, it also follows that this choice reduces the lateral component of the thrust on the vehicle to zero. Thus, we have shown that a guideway with a bank angle defined by Eq. 8 will result in zero lateral thrust on the vehicle and will also minimize the acceleration discomfort index ϕ.

From Eqs. 3, 4, and 6 it follows that the effective jerk and jerk discomfort index are respectively given by

$$\underset{\sim}{J}_e = (-g\dot{\theta}\sin\theta + 2v\dot{v}\kappa\sin\theta + v^2\dot{\kappa}\sin\theta + v^2\kappa\dot{\theta}\cos\theta)\underset{\sim}{n} + (g\dot{\theta}\cos\theta - 2v\dot{v}\kappa\cos\theta - v^2\dot{\kappa}\cos\theta + v^2\kappa\dot{\theta}\sin\theta)\underset{\sim}{\ell} + \dot{v}\underset{\sim}{f} \tag{9}$$

and

$$\psi = (g^2\dot{\theta}^2 + 4v^2\dot{v}^2\kappa^2 + v^4\dot{\kappa}^2 + v^4\kappa^2\dot{\theta}^2 + \dot{v}^2 - 4gv\kappa\dot{\theta}\dot{v} - 2gv^2\dot{\kappa}\dot{\theta} + 4v^3\kappa\dot{v}\dot{\kappa})^{1/2} \tag{10}$$

We note that bank angle θ does not explicitly appear in the expression for ψ. By equating $\frac{d\psi}{d\dot{\theta}}$ to zero, we obtain

$$\dot{\theta} = \frac{2gv\kappa\dot{v} + gv^2\dot{\kappa}}{g^2 + v^4\kappa^2} \tag{11}$$

as a necessary condition for ψ to attain a minimum. It may then be checked that $\dot{\theta}$ as defined by Eq. 11 is precisely the time derivative of θ^* defined by Eq. 8. This means that, if the bank angle is given by Eq. 8 at all points of the centerline curve, then Eq. 11 will be satisfied. Thus we have shown that a guideway bank angle defined by Eq. 8 will minimize the jerk discomfort index ψ.

Thus we conclude that the bank angle defined by Eq. 8 not only reduces the lateral thrust on the vehicle to zero but also minimizes both discomfort indexes that we have defined. [It is possible to obtain this conclusion for the more general case in which the centerline curve is not restricted to horizontal. Furthermore, a more general form of the function than that assumed in Eq. 2 is possible. The results are presented in a separate report (10).] This bank angle will be referred to hereafter as the optimal bank angle. The discomfort along an optimally banked curve may be obtained by substituting from Eqs. 8 and 11 for θ and $\dot{\theta}$ in Eqs. 7 and 10. The corresponding expressions for the effective acceleration and jerk vectors and the discomfort indexes are given as follows:

$$\underset{\sim}{a}_e^* = [(v^4\kappa^2 + g^2)^{1/2} - g]\underset{\sim}{n} + \dot{v}\underset{\sim}{f}$$

$$\phi^* = \{[(v^4\kappa^2 + g^2)^{1/2} - g]^2 + \dot{v}^2\}^{1/2}$$

$$\underset{\sim}{J}_e^* = (v^4\kappa^2 + g^2)^{-1/2}(2v^3\kappa^2\dot{v} + v^4\kappa\dot{\kappa})\underset{\sim}{n} + \ddot{v}\underset{\sim}{f}$$

$$\psi^* = (v^4\kappa^2 + g^2)^{-1/2}[(2v^3\kappa^2\dot{v} + v^4\kappa\dot{\kappa})^2 + \ddot{v}^2(v^4\kappa^2 + g^2)]^{1/2}$$

In practice, the bank angle could be limited by other considerations, and the value obtained from Eq. 8 would be too high to be practical.

FOUR-SPIRAL SWITCH

The first example we consider is that of a four-spiral curve that can be used as either a merge or diverge switch at stations and interchanges in PRT networks. The four-spiral switch is shown in Figure 2. The equation for the spiral curve is

$$\beta = cs^2 \tag{12}$$

If β is small, then Eq. 12 may be approximated by

$$y = \frac{c}{3} x^3 \tag{13}$$

The spiral set shown in Figure 2 uses four spirals of the form of Eq. 13. The spirals are matched for position, slope, and curvature at junction points 2, 3, and 4. The first spiral winds, the second unwinds, the third winds, and the fourth unwinds. The curvature at points 1, 3, and 5 is zero. Maximum curvature is attained at points 2 and 4. The slope of the spiral set is zero at both ends. The use of the spiral set as a switch permits acceleration and deceleration lanes to be packaged parallel and near to the main traffic lane. It can be shown (1, 12) that

$$c = \frac{16h}{L^3} \tag{14}$$

where h and L are as shown in Figure 2. It follows further from Eqs. 13 and 14 that

$$\kappa \simeq \frac{d^2y}{dx^2} = \frac{32hx}{L^3} \tag{15}$$

We next consider the banking of the switch. It will be assumed that the bank angle varies linearly along each of the four spirals; is zero at points 1, 3, and 5; and attains its maximum value θ_o at the points 2 and 4. It follows immediately that between points 1 and 2 θ is given by the equation

$$\theta = \frac{4\theta_o x}{L} \tag{16}$$

Furthermore, if a vehicle is traveling at speed v, then

$$\dot{\theta} = \frac{4v\theta_o}{L} \tag{17}$$

We next consider the problem of finding the discomfort experienced by traveling at constant speed along the switch. By symmetry it suffices to study the problem only in the first sprial, between 1 and 2. By using Eqs. 13 through 17, it follows that Eqs. 7 and 10 become

$$\phi = \left\{2g^2\left[1 - \cos\left(\frac{4\theta_o x}{L}\right)\right] + 1{,}024\,\frac{h^2v^4x^2}{L^6} - 64ghv^2\,\frac{x}{L^3}\,\sin\left(\frac{4\theta_o}{L}\,x\right)\right\}^{1/2}$$

$$\psi = \left(16g^2\theta_o^2\,\frac{v^2}{L^2} + 1{,}024\,\frac{h^2v^6}{L^6} + 16{,}384h^2\theta_o^2\,v^6\,\frac{x^2}{L^8} - 256gh\theta_o\,\frac{v^4}{L^4}\right)^{1/2} \tag{18}$$

If on the other hand the four-spiral curve is optimally banked, the bank angle as given by Eq. 8 is

$$\theta^* = \tan^{-1}\left(\frac{32hv^2x}{gL^3}\right) \tag{19}$$

and the discomfort indexes given earlier take the form

$$\phi^* = g\left(1 + 1{,}024\,\frac{h^2v^4x^2}{g^2L^6}\right)^{1/2} - g$$

$$\psi^* = 1{,}024\,\frac{h^2v^5x}{gL^6}\left(1 + 1{,}024\,\frac{h^2v^4x^2}{g^2L^6}\right)^{-1/2} \tag{20}$$

Equations 18 and 20 were programmed on a digital computer to permit numerical investigation. In all computations, h = 8 ft and v = 60 ft/sec were chosen. For selected values of L and θ_o, ϕ and ψ were computed over the range $0 \leq x \leq L/4$. In every case, both ϕ and ψ attained their maximum values, denoted respectively as ϕ_{max} and ψ_{max}, at $x = L/4$. Figure 3 shows the dependence of ϕ_{max} and ψ_{max} on the switch length. The curves show that a substantial reduction of discomfort can result from banking.

It is of interest to graphically depict the switch length reduction possible with banking as shown in Figure 4. The figure is a design curve based on the design assumptions of $\phi_{max} \leq 8$ ft/sec^2 and $\psi_{max} \leq 8$ ft/sec^3. The curve was obtained by solving Eq. 18 by trial and error with $x = L/4$. The procedure was to fix θ_o and vary L by small increments over a wide range. The value of L corresponding to $\psi_{max} = 8$ ft/sec^3 was then found. In every case, ϕ_{max} was less than ψ_{max}. By doing this for several values of θ_o, we obtained the data shown in Figure 4. Values of θ_o in the range $0 \leq \theta_o \leq \theta^*$ were selected. It follows by setting $x = L/4$ in Eq. 19 that $\theta^* = 3/8$ radian (21 deg 30 min).

RIGHT ANGLE TURN

A right angle turn may be accomplished by means of a circular arc blended with the straight at both ends through spiral curves as shown in Figure 5. We shall design the right angle turn with no banking to keep $\psi \leq 8$ ft/sec^2 and then investigate the effect that banking has in reducing discomfort. Vehicle speed in the right angle turn will be taken as constant.

Because the angle β_o over which the spiral extends need not be small, we discard the approximation introduced in Eq. 13 and use Eq. 12 exactly for the spiral curve. From Eq. 12 we have

$$\kappa = \frac{d\beta}{ds} = 2cs = 2c^{1/2}\beta^{1/2}$$

$$\dot{\kappa} = 2cv, \quad \dot{\beta} = 2csv = 2c^{1/2}v\beta^{1/2} \tag{21}$$

Furthermore, if the bank angle θ varies linearly along the spiral, it follows that

$$\theta = \kappa s = \kappa c^{-1/2}\beta^{1/2}$$

If the maximum bank angle is θ_o, then

$$\theta_o = \kappa c^{-1/2}\beta_o^{1/2} \tag{22}$$

and thus, substituting for κ,

$$\theta = \theta_o(\beta/\beta_o)^{1/2} \tag{23}$$

Substituting from Eqs. 21 and 22 in Eqs. 7 and 10 gives the following discomfort indexes:

$$\phi = [2g^2(1 - \cos\theta) + 4cv^4\beta - 4gc^{1/2}v^2\beta^{1/2}\sin\theta]^{1/2}$$

$$\psi = \left(g^2c\theta_o^2\frac{v^2}{\beta_o} + 4c^2v^6 + 4c^2v^6\theta_o^2\frac{\beta}{\beta_o} - 4gc^{3/2}\theta_o\frac{v^4}{\beta_o^{1/2}}\right)^{1/2} \tag{24}$$

In particular, for the unbanked curve putting $\theta_o = 0$, we have

$$\phi = 2c^{1/2}v^2\beta^{1/2}$$

$$\psi = 2cv^3 \tag{25}$$

If the maximum allowable values for ϕ and ψ are ϕ_o and ψ_o respectively, we have

$$c = \frac{\psi_o}{2v^3}$$

$$\beta_o = \frac{\phi_o^2}{2\psi_o v} \tag{26}$$

Furthermore, because the curvatures are to be matched at the point where the spiral and circular curves blend and the curvature of the spiral at this point is $\kappa_o = 2c^{1/2}\beta_o^{1/2}$, we obtain the radius of the circular arc R as

$$R = \frac{1}{\kappa} = \frac{1}{2\sqrt{c\beta_o}} = \frac{v^2}{\phi_o} \tag{27}$$

Equations 26 and 27 completely determine the geometry of the right angle turn. To get an idea of the space taken up by the curve, we define two terms L_o and D as shown in Figure 5. The distance D is of importance when how the interchange fits in with existing road patterns and structures is considered. For the spiral we have

$$\frac{dx}{ds} = \cos\beta \quad \frac{dy}{ds} = \sin\beta$$

Using Eq. 12 and integrating these equations approximately with the help of Taylor series expansions for $\cos\beta$ and $\sin\beta$, we get

$$x = c^{-1/2}\left(\beta^{1/2} - \frac{1}{5}\beta^{5/2}\right)$$

$$y = c^{-1/2}\left(\frac{1}{3}\beta^{3/2} - \frac{1}{21}\beta^{7/2}\right) \tag{28}$$

If (x_o, y_o) are the coordinates of the point P where the spiral and circular arcs blend, a little geometric analysis shows that

$$L_o = x_o + y_o + 2R\sin(\pi/4 - \beta_o)\cos\pi/4$$

$$D = y_o\sec\pi/4 + R\sin(\pi/4 - \beta_o) - R[1 - \cos(\pi/4 - \beta_o)]$$

Substituting from Eq. 28 gives

$$L_o = c^{-1/2}\left(\beta_o^{1/2} + \frac{1}{3}\beta_o^{3/2} - \frac{1}{5}\beta_o^{5/2} - \frac{1}{21}\beta_o^{7/2}\right) + \sqrt{2}\,R\sin(\pi/4 - \beta_o)$$

$$D = \sqrt{2}\,c^{-1/2}\left(\frac{1}{3}\beta_o^{3/2} - \frac{1}{21}\beta_o^{7/2}\right) + R[\sin(\pi/4 - \beta_o) - 1 + \cos(\pi/4 - \beta_o)] \tag{29}$$

The parameters for the unbanked right angle turn obtained from Eqs. 26, 27, and 29 are summed up as follows:

$$\beta_o = 1/4 \text{ rad} = 14 \text{ deg } 20 \text{ min} \qquad L_c = 40.2 \text{ ft}$$
$$R = 32 \text{ ft} \qquad D = 13.7 \text{ ft}$$

(variable values: $v = 16$ ft/sec, $\phi_o = 8$ ft/sec^2, $\psi_o = 8$ ft/sec^3)

To investigate the effect of banking this curve on discomfort, we programmed Eq. 24 on a digital computer, and the values of ϕ and ψ were obtained for several values of θ_o, the maximum bank angle. Numerical values were found at several points along the curve. In every case, the discomfort indexes attained their maximum value at the point P. The maximum value of each discomfort index for motion along the curve at a constant speed of 16 ft/sec is plotted against the bank angle in Figure 6, which brings out the considerable reduction in discomfort possible by introducing banking.

GRADE CHANGE

To analyze a grade change, we first analyze discomfort along a curve with zero superelevation in the vertical plane. If the tangent to the curve is inclined at γ to the horizontal, the acceleration components for motion along such a curve are

$$\underset{\sim}{a}_e = -g\underset{\sim}{n} - \underset{\sim}{g} + \underset{\sim}{a} = -g\underset{\sim}{n} - (g \sin \gamma \underset{\sim}{f} - g \cos \gamma \underset{\sim}{n}) + (-v^2\kappa\underset{\sim}{n} + \dot{v}\underset{\sim}{f}) \qquad (30)$$

We consider two possible methods of traversing a curve in the vertical plane, namely, constant speed ($\dot{v} = 0$) and zero thrust. In the latter case, vehicle acceleration is solely due to gravity and $\dot{v} = g \sin \gamma$. The expressions thus obtained for the effective acceleration and jerk and the discomfort indexes are presented as follows, where the subscript c denotes constant speed motion and subscript z denotes zero thrust motion.

$$(\underset{\sim}{a}_e)_c = [-g(1 - \cos \gamma) - v^2\kappa]\underset{\sim}{n} - g \sin \gamma \underset{\sim}{f}$$

$$\phi_c = [2g^2(1 - \cos \gamma) + v^4\kappa^2 + 2gv^2\kappa(1 - \cos \gamma)]^{1/2}$$

$$(\underset{\sim}{J}_e)_c = (-g\dot{\gamma} \sin \gamma - v^2\dot{\kappa})\underset{\sim}{n} - g\dot{\gamma} \cos \gamma \underset{\sim}{f}$$

$$\psi_c = (g^2\dot{\gamma}^2 + v^4\dot{\kappa}^2 + 2gv^2\dot{\gamma}\dot{\kappa} \sin \gamma)^{1/2}$$

$$(\underset{\sim}{a}_e)_z = [-g(1 - \cos \gamma) - v^2\kappa]\underset{\sim}{n}$$

$$\phi_z = g(1 - \cos \gamma) + v^2\kappa$$

$$(\underset{\sim}{J}_e)_z = (-g\dot{\gamma} \sin \gamma - v^2\dot{\kappa} - 2gv\kappa \sin \gamma)\underset{\sim}{n}$$

$$\psi_z = |-g\dot{\gamma} \sin \gamma - v^2\dot{\kappa} - 2gv\kappa \sin \gamma|$$

A grade change may be accomplished by means of a straight sloping line connected to the horizontal at either end by two spirals as shown in Figure 7. If the straight segment is eliminated, the grade change just becomes a vertical version of the four-spiral curve of Figure 2. Such a curve was used as a grade change for an automotive test track (1). We will compare the discomfort in traversing the grade change for the zero thrust and constant speed cases.

In the zero thrust motion, maximum speed is attained at point 5 and maximum discomfort is attained in the lower half of the grade change. For the constant speed case the discomforts in the lower and upper halves are equivalent. Therefore we consider only the bottom two spiral curves in Figure 7. Taking the origin at point 5 as shown in Figure 7 and matching slope and curvature at points 3 and 4 yields the following equations for two spirals:

Figure 1. Banked guideway and acceleration.

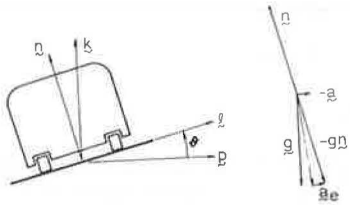


Figure 2. Four-spiral switch.

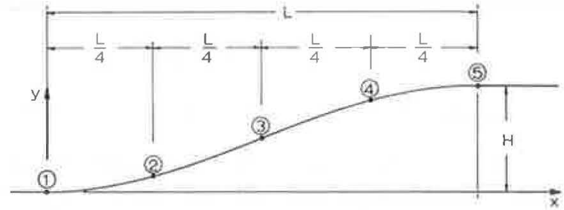


Figure 3. Discomfort indexes versus switch length.

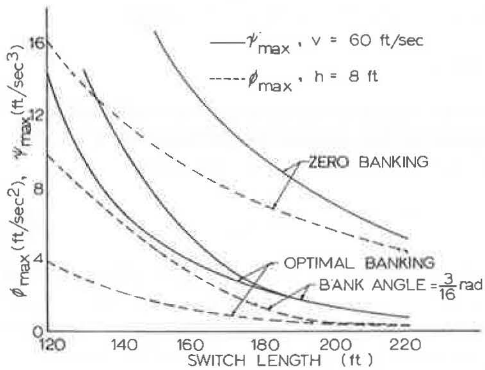


Figure 4. Switch length versus bank angle.

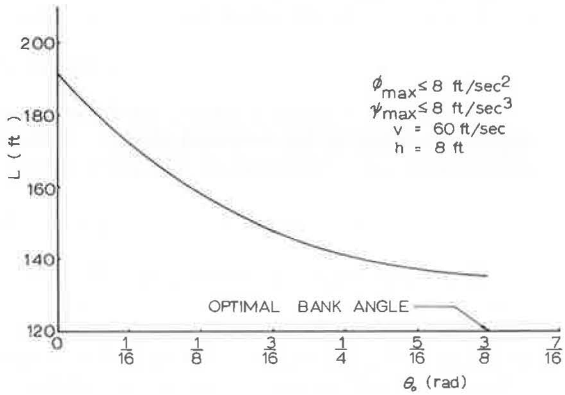


Figure 5. Right angle turn.

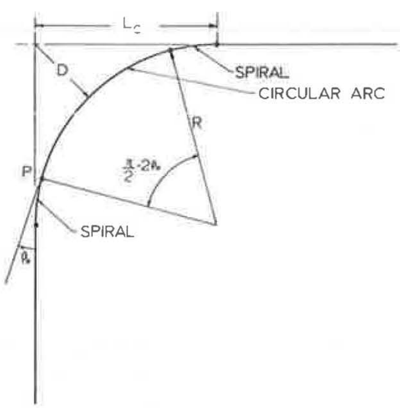
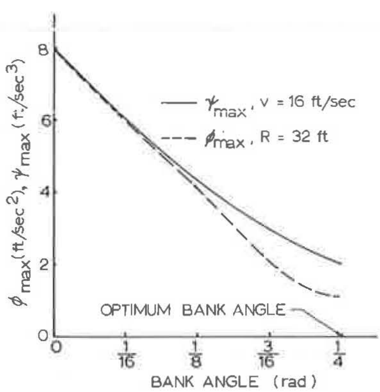


Figure 6. Discomfort in a right angle turn.



$$\beta = \frac{2\alpha}{L'^2} x^2 \quad 0 \le x \le L'/2$$

$$\beta = -\frac{2\alpha}{L'^2} x^2 + \frac{4\alpha}{L'} x - \alpha \quad L'/2 < x \le L' \tag{31}$$

where, because of the small angles involved, we have replaced s by x.

By substituting these expressions for β in the formulas for effective acceleration and jerk and discomfort indexes, we obtain the expressions for ϕ_z, ψ_z, ϕ_o, ψ_o, and the discomfort indexes for zero thrust and constant speed motions for each of the two bottom spiral curves of the grade change. The expressions are given in the Appendix and were programmed on a digital computer to find the discomfort indexes at several points from 3 to 5. Because the speed along the grade change varies in the zero thrust case, the speed for constant speed motion may be taken equal to either the maximum speed or the minimum speed of the zero thrust case. For numerical computations the following values were chosen:

$$\begin{aligned} H &= \text{level difference} = 10 \text{ ft} \\ v &= \text{minimum speed} = 16 \text{ ft/sec} \\ \alpha &= \text{maximum slope} = 0.20 \text{ radian (11 deg 27 min)} \end{aligned} \tag{32}$$

The corresponding maximum speed for zero thrust motion turns out to be 29.93 ft/sec. The maximum values of the discomfort indexes were found for zero thrust motion and for motion at constant speeds of 16 ft/sec and 29.93 ft/sec and are shown in Figure 8. It is observed that the discomfort for constant speed motion at the higher speed is considerably more than for zero thrust motion.

McConnell (1) considered only the centripetal component of the acceleration $v^2\kappa$ and the corresponding jerk in evaluating discomfort. Denoting the corresponding discomfort indexes by ϕ_M and ψ_M we have

$$\phi_M = v^2\kappa$$

$$\psi_M = |v^2\dot{\kappa}| \tag{33}$$

The expressions obtained on substituting for κ and $\dot{\kappa}$ in Eq. 33 are also included in the Appendix. It is of interest to investigate the degree of error introduced by this assumption. To do this, we compared the maximum discomfort as given by Eq. 33 with the values obtained for $(\phi_o)_{max}$ and $(\psi_o)_{max}$ from the expressions in the Appendix. This was done for two values of the speed, v = 16 ft/sec and v = 60 ft/sec, and two values of the maximum inclination, $\alpha = 0.20$ radian (11 deg 27 min) and $\alpha = 0.40$ radian (2 deg 17 min). The results are given in Table 1.

The effect of the speed is readily observable. A study of the expressions given earlier indicates that at higher speeds the centripetal component provides the major contribution to ϕ_o and ψ_o and therefore ϕ_M and ψ_M provide better approximations to ϕ_o and ψ_o at higher speeds than at lower speeds. As for the effect of inclination at a given speed, the expressions indicate that, as the inclination increases, the approximate expressions (Eq. 33) would become less accurate. However, for any realistic angle of grade, the inclinations would still be fairly small, and therefore the error in Eq. 33 is not very sensitive to variations of the inclination. These conclusions are borne out by the numerical values given in Table 1.

ACKNOWLEDGMENT

This work was supported in part by the Urban Mass Transportation Administration, U.S. Department of Transportation.

Figure 7. Grade change.

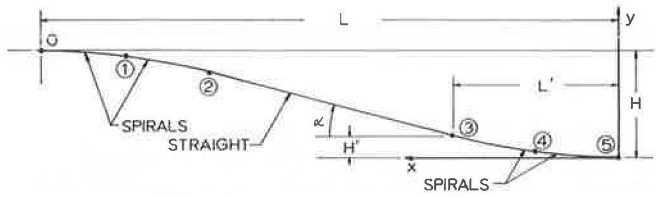


Figure 8. Discomfort on a grade change.

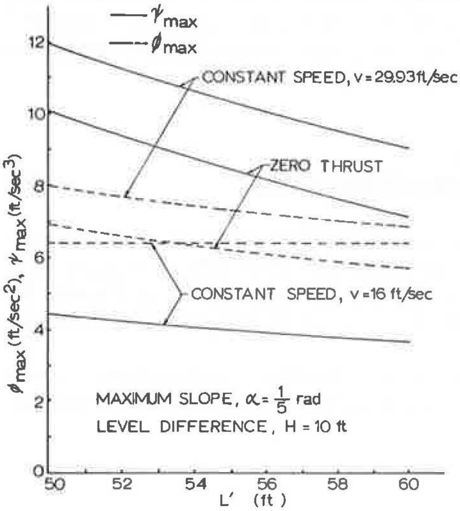


Table 1. Contribution of fore-aft component to discomfort.

Maximum Slope (rad)	Speed (ft/sec)	$(\varphi_c)_{max}$ (ft/sec^2)	$(\varphi_H)_{max}$ (ft/sec^2)	$(\psi_c)_{max}$ (ft/sec^3)	$(\psi_H)_{max}$ (ft/sec^3)
α = 0.20	v = 16	6.39	0.68	1.39	0.15
	v = 60	10.27	9.60	9.65	7.68
α = 0.04	v = 16	1.28	0.14	0.28	0.03
	v = 60	2.03	1.92	1.86	1.54

Note: L' = 150 ft.

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APPENDIX

DISCOMFORT EXPRESSIONS FOR FOUR-SPIRAL GRADE CHANGE

Spiral 1: $0 < x < L'/2$

$$\beta = \frac{2\alpha}{L'^2} x^2 \qquad \kappa = \frac{4\alpha}{L'^2} x \qquad \dot{\kappa} = \frac{4\alpha}{L'^2} v$$

$$\phi_z = g(1 - \cos\beta) + \frac{4\alpha}{L'^2} v^2 x$$

$$\psi_z = 12 \frac{g\alpha}{L'^2} vx \sin\beta + \frac{4\alpha}{L'^2} v^3$$

$$\phi_c = \left[2g^2(1 - \cos\beta) + \frac{16\alpha^2}{L'^4} v^4 x^2 + \frac{8g\alpha}{L'^2} v^2 x (1 - \cos\beta)\right]^{1/2}$$

$$\psi_c = \frac{4\alpha v}{L'^2} (g^2 x^2 + v^4 + 2gv^2 x \sin\beta)^{1/2}$$

$$\phi_M = \frac{4\alpha}{L'^2} v^2 x$$

$$\psi_M = \frac{4\alpha}{L'^2} v^3$$

Spiral 2: $L'/2 < x < L'$

$$\beta = -\frac{2\alpha}{L'^2} x^2 + \frac{4\alpha}{L'} x - \alpha \qquad \kappa = \frac{4\alpha}{L'}\left(1 - \frac{x}{L'}\right) \qquad \dot{\kappa} = -\frac{4\alpha v}{L'^2}$$

$$\phi_z = g(1 - \cos\beta) + \frac{4\alpha}{L'} v^2 \left(1 - \frac{x}{L'}\right)$$

$$\psi_z = \left| - \frac{12g\alpha}{L'} v \left(1 - \frac{x}{L'}\right) \sin\beta + \frac{4\alpha}{L'^2} v^3 \right|$$

$$\phi_c = \left[2g^2(1 - \cos\beta) + \frac{16\alpha^2}{L'^2} v^4 \left(1 - \frac{x}{L'}\right)^2 + \frac{8g\alpha}{L'} v^2 \left(1 - \frac{x}{L'}\right)(1 - \cos\beta) \right]^{1/2}$$

$$\psi_c = \frac{4\alpha v}{L'} \left[g^2 \left(1 - \frac{x}{L'}\right)^2 + \frac{v^4}{L'^2} - \frac{2gv^2}{L'} \left(1 - \frac{x}{L'}\right) \sin\beta \right]^{1/2}$$

$$\phi_M = \frac{4\alpha}{L'} v^2 \left(1 - \frac{x}{L'}\right)$$

$$\psi_M = \frac{4\alpha}{L'^2} v^3$$

EVALUATION OF FREEWAY TRAFFIC FLOW AT RAMPS, COLLECTOR ROADS, AND LANE DROPS

Darryl B. Martin and Leonard Newman, California Department of Public Works; and Roger T. Johnson, California Division of Highways

This study was designed to evaluate freeway traffic flow characteristics for several high-standard geometric design features. The study evaluates and compares the effect of different ramp types, spacing, and volumes on freeway capacity and operation. Collector-distributor roads, auxiliary lanes, and lane drops are included. The primary measure of effectiveness used for this study was density (vehicles per lane-mile). Density was chosen rather than speed because it is a better indication of driving conditions. The major finding of this study is that freeway designs that offer greater flexibility (freedom of choice to the drivers) will result in smoother and more efficient operation. For example, a freeway with auxiliary lanes has greater flexibility than a freeway with a collector road system with the same total number of lanes. It also has greater capacity and more efficient operation.

•SINCE the first freeway was built, an evolution in design has been in progress that has caused standards to be continually changed. With better standards, such as wider lanes, wider medians, longer ramp tapers, special weaving areas, large radius curves, increased lateral clearance, and reduction in grades, newer freeways carry larger volumes of traffic faster and safer than ever before.

In densely populated urban areas, freeways tend to become overloaded during peak periods of the day. These freeways carry such high volumes of traffic that geometric configurations become critical controls of their operation. Improper design causes increased congestion and excessive delay to the motorist.

As always, the adequacy of present design standards is being questioned. What freeway geometric designs, from an operational point of view, will give the highest level of service (least congestion) and carry the maximum volume of traffic? This study was designed to help answer this question by making an evaluation of traffic flow characteristics for several high-standard geometric design features. Traffic flow characteristics are simply volume, lane distribution, and average speed or density as well as variability in speed or density. The study evaluates and compares the effect of different ramp types, spacing, and volumes on freeway capacity and operation. Collector-distributor roads, auxiliary lanes, and lane drops are included in the analysis.

STUDY LOCATIONS

Eleven study sites were used in the Los Angeles area on the San Diego, Santa Monica, and Hollywood Freeways (Fig. 1). The design characteristics of each are as follows:

1. Medium-volume multiple on-ramps,
2. A high-volume standard on-ramp,
3. Medium-volume multiple off-ramps,
4. High-volume multiple off-ramps,
5. A high-volume standard off-ramp,
6. Merge of collector-distributor road onto freeway,
7. Slip-ramp from collector-distributor road to freeway,
8. A high-volume off-ramp with parallel auxiliary lane,

9. A high-volume on-ramp with parallel auxiliary lane,
10. Freeway merge from four to three lanes, and
11. A medium-volume two-lane on-ramp.

At all of the off-ramp locations, an upstream straight pipe section was observed, and, at most of the on-ramp locations, a downstream straight pipe section was observed.

METHODOLOGY

The primary measure of effectiveness used for this study was that, for a given rate of flow, one section of freeway operates better than another if the traffic density is less or if it is more uniform at the same average density. Density was chosen as the primary measure of quality of operation rather than speed because it is thought that density (which, if volume is known, can be translated into average speed) is a better indication of driving conflicts or tension than speed per se.

Time-lapse photography was used to obtain density and volume by individual lanes at every location. Photographs were taken at the rate of one frame per sec. Spot densities were determined by counting the number of cars in a known length of road in every fifth frame. Knowing density and volume by lane permitted analysis of different sections of roadway with varying volumes and geometric conditions.

The findings of this study are based on observations of freeways carrying capacity or near-capacity volumes. This is the volume level when the quality of flow tends to be unstable and geometric conditions have a major effect on the operation of the freeway.

The 90th percentile density (a value that density is equal to or less than 90 percent of the time) was used as a paramter for the quality of flow. The 90th percentile density can be used to show the variance in operation at a location or between locations. The general method or form this analysis took can be illustrated in the following example taken from the data.

Say we are concerned with sections of freeway with four lanes in one direction with a total 5-min flow rate of 7,500 vehicles per hour (Appendix). To get a statistical sample, we dealt with 5-min periods that had flow rates between 7,400 and 7,600 vph.

Then, on a four-lane section with no ramps in the vicinity or any other unusual features, a typical 5-min time-slice, representing 60 data points for a given time of day on one of the graphs, would be as given in Table 1. This table basically shows that, at this section when the 5-min flow rate is 7,500 vph, the mean density is 125 vehicles per mile, which is an average calculated speed of 7,500/125 or 60 vph. (This is a calculated speed for a short duration of time.) It also shows that 10 percent of the time density in lane 3 exceeds 48 vehicles per mile. This represents queuing effects and is an adverse feature.

Next we look at a merging section where there is a single on-ramp and where the total freeway 5-min flow rate is 7,500 vph (Table 2). Data given in Table 2 show that, for this circumstance with a total flow rate of 7,500 vph and a ramp flow rate of 925 vph, mean density is 126 vehicles per mile, which is an average calculated speed of 59.5 mph. The table also shows that 10 percent of the time the merging density is greater than 55 vehicles per mile. (It is entirely possible to have equal mean densities or speed; then the variability factor becomes more important in determining operational difference.) It can be seen that the merging section does not operate so well as the through section to the extent shown.

Graphs have been developed (Fig. 2) that show the relationship between volume and density with respect to time for the different locations. Speed was calculated with volume and density and shown on the graphs also. Figure 2 shows volume, lane distribution, density, and speed all with respect to time of day. This makes it possible to compare the quality of operation for different locations at similar flow rates. The graphs also obviate the difficulty in interpreting the meaning of standard "q-k" curves, which are plotted without relation to time and which fail to distinguish upstream from downstream of bottlenecks.

Figure 2 vividly shows when operation breakdown occurred. Using the term "breakdown" is really a misnomer. Even though the drivers are experiencing stop-and-go

Figure 1. Study sites.

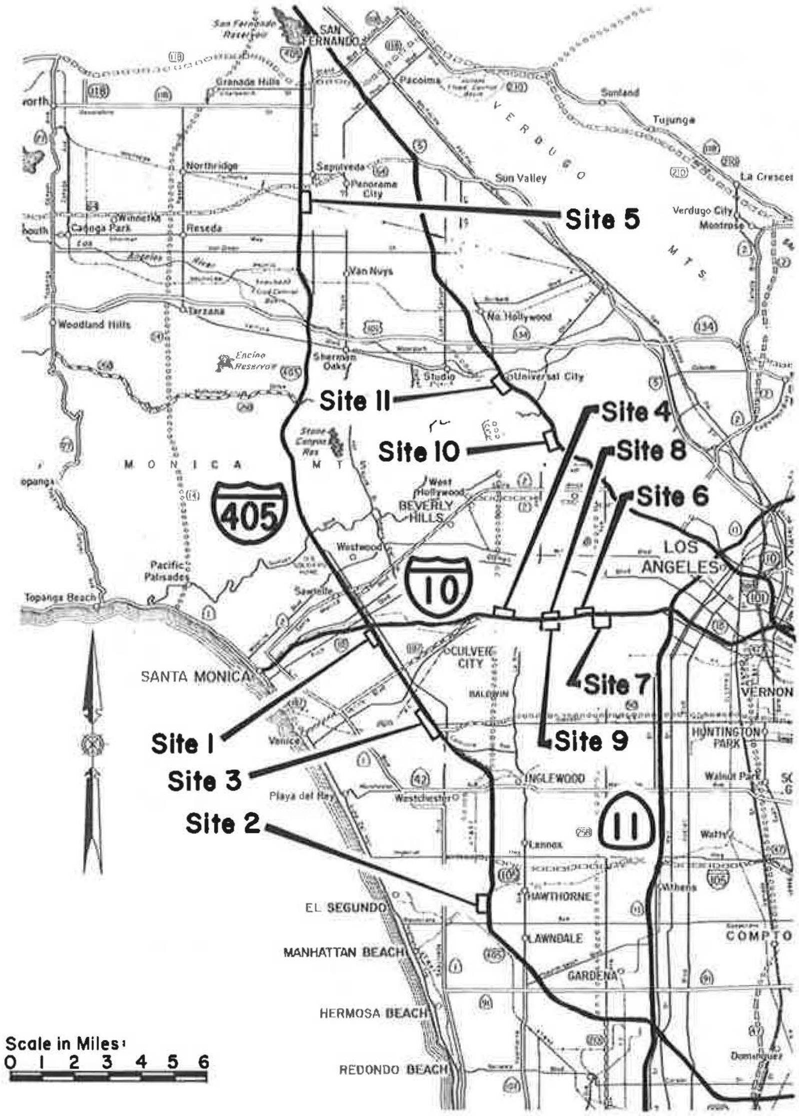


Table 1. Flow rate and density on four-lane section with no ramps.

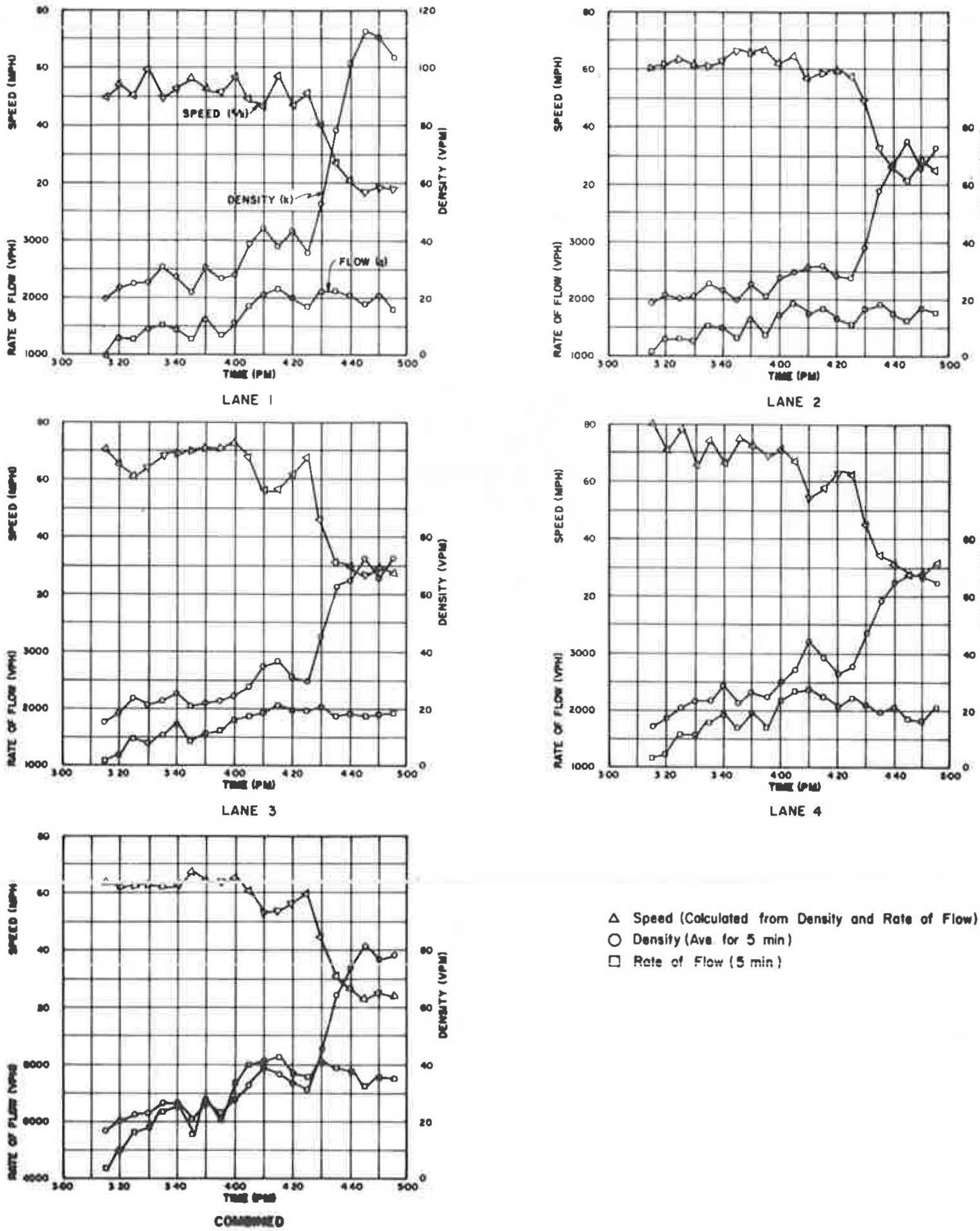
Lane[a]	5-Min Flow Rate (vph)	Mean Density[b] (vehicles/mile)	90th Percentile Density (vehicles/mile)
1	1,340	24	40
2	1,900	34	40
3	2,050	34	48
4	2,210	33	39
Total	7,500	125	

[a]Highway Capacity Manual nomenclature is used; lane 1 is on right side of roadway.
[b]Mean of 60 instantaneous densities observed during 5 min; 90th percentile is density equaled or exceeded in six of these observations.

Table 2. Flow rate and density on four-lane section with on-ramp.

Lane	5-Min Flow Rate (vph)	Mean Density (vehicles/mile)	90th Percentile Density (vehicles/mile)
Ramp	925		
1	895	35	55
Subtotal	1,820		
2	1,560	27	39
3	1,920	29	40
4	2,200	35	46
Total	7,500	126	

Figure 2. High-volume single-lane on-ramp (San Diego Freeway southbound at El Segundo on-ramp–site 2).



driving and are averaging speeds of approximately 30 mph, the freeway is still allowing a near-capacity rate of flow through the section. This can be seen on any of the graphs that show excessive densities and low speeds. The volume will stay at or near capacity.

All of the sites were analyzed in a similar manner to what has been discussed, and conclusions were reached from this analysis. (Detailed analysis and discussion of each study site have been omitted from this publication in the interest of brevity.)

SUMMARY OF CONCLUSIONS

The major findings of this study are as follows:

1. Where ramp layout provides adequate acceleration and deceleration tapers as well as adequate capacity on ramps and at terminals with the local street system, it was observed that off-ramps operate more smoothly and cause less congestion to the freeway than on-ramps. Bottlenecks most frequently occur downstream from an on-ramp where traffic is added to the freeway without addition of extra lanes. Because traffic signals are usually present at the ramp terminals with the city streets, traffic frequently enters the freeway in platoons, which causes severe sporadic overloading of the shoulder lane. Off-ramps relieve the freeway by taking traffic off. The vehicles usually leave the freeway at a more uniform rate; therefore, the platooning problem is greatly reduced. Platooning and overloading of the right lane do occur for off-ramps, but it takes higher volumes for platooning and overloading to become a problem.

2. High-volume single-lane standard on-ramps create operational problems on the freeway by overloading lane 1 and causing poor lane distribution of volumes on the freeway. A more efficient operation of the freeway can be obtained either by splitting the same volume into two lower volume ramps, say 1,200 to 1,500 ft apart, or by adding an auxiliary lane (as a continuation of the on-ramp) that extends to the next off-ramp or, in the absence of a high-volume off-ramp within a mile or so, for a minimum distance of 2,500 ft. Either alternative will allow the freeway to carry higher volumes with less congestion. The effect of platooning is greatly reduced.

3. If off-ramps have adequate capacity, operation will usually be smooth on the freeway near the off-ramp nose. Congestion, if present, will usually occur upstream of the off-ramp due to lane changing and overloading of lane 1 by vehicles desiring to use the off-ramp. High-volume single-lane off-ramps create operational problems to the freeway upstram of the off-ramps, i.e., excessive congestion and poor distribution of lane volumes. The use of an auxiliary lane upstream of the off-ramp will relieve congestion by allowing existing traffic to move out of the through lanes of traffic. If a capacity problem exists at the throat of the off-ramp, it can be relieved by enlarging the exit into a two-lane off-ramp and giving the vehicles in lane 1 an exit option. (A two-lane exit throat must be preceded by an auxiliary lane.) Another alternative to relieve congestion would be to split the same volume of traffic into two lower volume off-ramps, say 1,500 to 1,700 ft apart. This results in better lane distribution and reduces congestion in lane 1.

4. The use of multiple on- or off-ramps gives high volumes with less congestion to both the freeway and ramps because of better lane distribution and a more balanced use of the freeway. Multiple ramps also create a better distribution of traffic to the city street network. Merging problems are reduced.

5. The section of freeway studied that has collector-distributor roads is actually a 12-lane facility, but it is not capable of carrying the volume of a 12-lane freeway. Lane volumes on the collector roads are consistently lower than lane volumes on the freeway, which shows that full utilization of the capacity of the collector roads is not being accomplished. The separation of lanes between the freeway and collector road causes inflexibility in handling traffic fluctuations because even distribution of traffic in all lanes is prevented. The capacity of the collector road is also reduced because of design standards lower than those of the freeway; i.e., merging lengths are shorter.

6. On any design where a continuous collector-distributor road is proposed, it can be assumed that speeds will at times be high. Design standards including weaving and merging distances, acceleration and deceleration lane lengths, and sight distances should be consistent with these conditions. Generally speaking, the same design

criteria should be applied to a continuous collector-distributor road as are applied to the main line freeway.

7. Collector systems create high-volume slip-ramps. These ramps usually cause a capacity problem on the freeway during peak periods. Auxiliary lanes should be added to give these ramps a free entrance to or exit from the freeway. The auxiliary lanes should extend to the next interchange or at least 2,500 ft. This principle should apply to all high-volume ramps.

8. The use of auxiliary lanes between closely spaced interchanges increases the capacity of both the freeway and the ramps. This is considered the major benefit of auxiliary lanes. Auxiliary lanes also reduce weaving problems. Weaving occurs away from the mainstream of flow, therefore causing a minimum of disturbance to the freeway. Congestion upstream of an off-ramp with an auxiliary lane is minimized.

9. Auxiliary lanes should have special delineation to differentiate them from the through lanes of the freeway. Contact treatment is not necessarily the answer, but possibly use of special lane striping or dots may be.

10. The highest flow rates recorded in this study occurred downstream from bottleneck sections during periods when operational "breakdown" existed at or upstream of the bottleneck. Bottlenecks usually occur downstream of on-ramps or lane drops.

11. Operational "breakdowns" are manifested at lane densities between 40 and 50 vehicles per mile. In every case when the density was 50 vehicles per lane-mile or greater the operation was poor. During periods of operational breakdown, volume levels are near capacity.

Most of the findings of this report can be summed up by the following statement: Freeway designs that offer greater flexibility (freedom of choice to the drivers) will result in smoother and more efficient operation. For example, a freeway with auxiliary lanes has greater flexibility than a freeway with a collector road system with the same total number of lanes. It also has greater capacity and more efficient operation.

APPENDIX

SPACE-MEAN SPEED CALCULATED FROM LOW DENSITIES

In this report, the reader will note some incredibly high calculated space-mean speeds at fairly high volumes. These result from measured densities in the range of 20 to 40 vehicles per mile. At densitites greater than 40, the calculated speeds are more realistic. Extensive rechecking or "auditing" of the original data transcription failed to reveal any systematic error that would account for this phenomenon. Further analysis was made in an attempt to show that at least a portion of the anomaly can be attributed to the skewed distribution of small samples, even when a large number of samples is used for each data point. This attempt was unsuccessful, but it is still felt that there is something mystic about the effect of small numbers.

Density was determined by counting cars in a 400-ft trap every 5 sec and taking the mean of 60 such counts for the 5-min data point. A density of 33 vehicles per mile represents a mean count of 2.5 vehicles in the 400-ft trap. This means that a lot of zeros and ones are included in the 60. At first we suspected that, because zero could represent any intervehicle space from 400.1 ft to infinity, and knowing that the true value was a lot closer to 400 than infinity, some value higher than zero should be assigned to the zero readings. This would have had the effect of increasing the density, which in turn would result in more realistic space-mean speeds. However, it was soon discovered that this "suspicion" was founded on false reasoning; if the true mean space between vehicles is, say, 800.2 ft, there will be twice as many zeroes among the 60 samples as there would be if it is 400.1 ft.

Because of the anomaly (incredible speeds at low densities), this report was shelved for several years. However, in 1970 System Development Corporation (SDC) performed a study for NCHRP in which space-mean speeds were measured in a short (but much longer than 400 ft) trap, using 1-sec time-lapse aerial photography. Aerial photography eliminates the principal source of potential error in reading the raw data; i.e., the number of cars in the trap is not subject to doubt because of foreshortening effects or difficulty of determining whether a car is in or out of the trap.

The SDC study also showed very high space-mean speeds at low densities (75 mph in lane 4, average of 70 mph for all lanes). The authors of that report have no explanation either, except that Los Angeles area drivers drive very fast. We do not accept that exaplanation, but at least we are not the only ones who had trouble with the process.

In any event, whether the anomaly is owing to reading errors, mystic "end-effects" of averaging small numbers, or any other reason (space-mean speed is always less than time-mean speed, so that cannot be the reason), a change in the absolute magnitude of the speeds will not affect the conclusions that were drawn from the graphs and tables in this report. The main thing the reader can be sure of is that, when densities were low enough to result in very high computed speeds, quality of flow was very good.

COST-EFFECTIVENESS EVALUATION OF FREEWAY DESIGN ALTERNATIVES

Adolf D. May and John H. James, University of California, Berkeley

This paper deals with the selection of the optimal series of freeway design improvements in a network where the effects of decisions are nonlinear and inactive. A heuristic algorithm is specified for the formulated problem, and a case study on a California freeway is presented.

•CONGESTION is now encountered during weekday afternoon peak periods on the northbound roadway of the Eastshore Freeway (I-80) in the San Francisco Bay Area. Through driver complaints and preliminary engineering studies, the California Division of Highways has recognized this section of roadway as a problem location and has requested an evaluation of freeway improvements consisting of adding a lane or lanes along portions or the entire length of the study section. The question of what additions are most effective under given design and cost constraints is the problem addressed by this analysis.

The system to be studied will be defined in terms of geographic boundaries and time limits. The system is limited to the northbound roadway and associated on- and off-ramps of the Eastshore Freeway between a point 1,630 ft south of the Powell Street off-ramp to a point 2,560 ft north of the Road No. 20 on-ramp (approximately a 10-mile section). The period of time to be studied is 3:45 to 6:15 on a typical weekday afternoon.

Detailed traffic demand data for the study period have been collected for this section of the freeway. They consist of vehicular volume counts of all on-ramp to off-ramp pairs during consecutive 15-min intervals. The volume counts and their method of collection are described in detail in two earlier project reports (1, 2). Knowledge of these flow demands permits evaluation of the performance of the freeway system for any configuration of lanes.

THE MODEL

For purposes of evaluating the cost and effectiveness of alternative design improvements, the following measures are chosen: (a) annual construction and maintenance costs of the design improvement and (b) annual passenger-hours saved by the design improvement. The basic problem to be solved is formulated in words: Find the design improvement (lane additions) that will maximize annual passenger-hours saved subject to the constraints that the resulting freeway is a feasible design and that a given budget for the improvement is not violated.

Mathematically the problem is stated as follows: The freeway section to be studied is divided into k subsections, each of which is homogeneous in capacity and demand. (The Eastshore Freeway is divided into 30 subsections.) Let X_j be the number of lanes added to subsection j, and let $\overline{X} = (X_1, X_2, X_3, \ldots, X_k)$ be a collection of lane improvements. Define $G(\overline{X})$ as the annual effectiveness of the lane arrangement $\overline{X}$ and $C(\overline{X})$ as the annual cost of lane arrangement $\overline{X}$. The objective, given an investment C, is to find the optimal lane arrangement $\overline{X}^*$ such that $G(\overline{X}^*) = \max G(\overline{X})$ subject to $C(\overline{X}) \leq C$. To solve this optimization problem one must be able to calculate $G(\overline{X})$ and $C(\overline{X})$ for all feasible $\overline{X}$.

A deterministic predictive model (FREEQ, pronounced free queue) has been developed for the analysis of freeway systems (1). Essentially, given the design features of a freeway and the on-ramp to off-ramp flow demands for the study period, the

FREEQ model determines the total passenger time expended in the system. Knowing the total travel time associated with the existing conditions, the savings in passenger-hours induced by a design improvement $G(\overline{X})$ can therefore be calculated.

The original reports (1, 2) contain a complete description of FREEQ and its previous applications. The basic assumptions of the FREEQ model are as follows:

1. Traffic is treated as a compressible fluid where an individual vehicle is regarded as an integral part of the flow and is not considered individually;
2. Traffic demands are assumed to remain constant and do not fluctuate over a given time interval;
3. Traffic is considered to propagate downstream instantaneously, once it is loaded onto the freeway, unless there are capacity constraints; and
4. The capacities of subsections, including weaving sections and merging points, are estimated by using the standard methods defined in the Highway Capacity Manual (3).

The FREEQ model is quite efficient; it takes up 34 K words (octal) of computer core storage and uses about 4 sec of central processor time on the CDC 6400 for the evaluation of one configuration of lanes for this problem (with a cost under 50 cents a run).

In calculating the annual cost of improvements $C(\overline{X})$, we assume that the construction cost of widening a subsection is a piecewise linear function of the number of lanes added, so $C(\overline{X})$ may be taken as

$$C(\overline{X}) = R \sum_{i=1}^{k} \sum_{j=1}^{X_i} C_{ij}$$

where C_{ij} is the total construction cost of adding the j th lane in the i th subsection. (Using a double subscript allows for the possibility of adding lanes to a subsection at increasing cost per lane to reflect the purchase of expensive right-of-way.) If we assume that the additional annual maintenance cost of new lanes is a fixed percentage M of the initial construction cost of the improvement, the scaling factor R is

$$R = CRF + M$$

where CRF is the capital recovery factor given by

$$CRF = r(1 + F)^N/[(1 + r)^N - 1]$$

Here r is the interest rate, and N is the economic life of the improvement in years. For the following analysis, r is taken as 6 percent, N as 10 years, and M as ½ percent of the construction cost.

All design improvements are subject to feasibility in the following manner. The alternatives open to the decision-maker are the lane arrangements made by adding 0, 1, 2, ... lanes to each subsection. A lane improvement plan must be consistent with good highway design practice. This means that the number of lanes downstream of an off-ramp must either be equal to or be one less than the number of lanes upstream of the off-ramp. Likewise, the number of lanes downstream of an on-ramp must be either equal to or one more than the number of lanes upstream of the on-ramp. Finally, only one lane can be added or dropped at a subsection boundary. For the section of the Eastshore Freeway under study, the maximum permitted number of lanes in a subsection is six. These constraints can be represented by a network as shown in Figure 1, and a simple dynamic programming algorithm can be used to generate feasible lane arrangements automatically.

COST-EFFECTIVENESS ANALYSIS

If all feasible lane arrangements were known along with their cost and time savings, they could be plotted on a cost-effectiveness diagram with the X-axis being the annual cost and the Y-axis being the annual savings in passenger-hours. Consider the alternatives shown in Figure 2 in which each point represents the result of a particular lane arrangement. It is easy to see that certain alternatives dominate others; that is, for a given budget, one alternative has a better saving than all others with the same budget. These undominated points for different budgets are represented in the figure by darkened points, while dominated alternatives are shown by squares. The locus of the darkened points defines the optimal cost-effectiveness curve $F^*(C)$. Each of these lane arrangements is optimal, and the collection is referred to as the set S^*. The problem becomes to design a procedure that efficiently identifies the locus and character of the curve and its component points or, at least, some approximation of it.

The decision-maker can utilize the cost-effectiveness diagram in one of three ways to determine the course of action to be taken. One way is to specify the amount of investment C and to select the extreme point of $F^*(C)$ having a cost equal to or less than C. The slope of the line from the origin to this extreme point is numerically equivalent to the number of passenger-hours saved per dollar invested, and its reciprocal is cost-effectiveness in dollars invested per passenger-hour saved. Another approach is for the decision-maker to specify the maximum value of investment per passenger-hour saved, for example, $2.00 per passenger-hour saved, and to select the last extreme point of $F^*(C)$ that lies on or above the sloping line from the origin, which corresponds to the specified maximum cost-effectiveness ratio. The third method is to specify the minimum acceptable annual savings in passenger-hours and to select the extreme point of $F^*(C)$ having an annual savings in passenger-hours that will first satisfy that goal.

MODEL STRUCTURE

The structure of this model has some particular features that have important consequences on the possible forms of analysis. Basically the whole system behaves like a series of queues being processed through a series of service facilities. Here each subsection acts essentially as a service facility serving a traffic queue.

Two important features of the system derive from this representation. First, as is known from queuing theory, delays are a nonlinear function of the percentage of capacity utilization. The result is that improvements in the system, defined in terms of decreased delays, are a highly nonlinear function of investments in capacity. The second important feature is the highly interactive nature of a series of queues. This leads to the realization that the value of improvements in one subsection is not independent of the improvements in other subsections. Moreover, it may well be counterproductive to remove bottlenecks and reduce delays at some points because this may increase the total delay in the system.

As a result of these factors inherent in the structure of this model, the most common optimization techniques are not suitable. Linear programming is out and so is any kind of piecewise linear programming, due to the nonconvexity of the objective function. Nor can more general techniques, such as dynamic programming, be used because of the interdependence between the subsections. Even complete enumeration is impossible due to the huge number of alternatives. There are somewhere between 2^{30} and 3^{30}, i.e., on the order of 10^{10}, alternatives even for the limited problem under study. If a computer could evaluate 1,000 alternatives every second, it would take more than several months of continuous calculations to consider all possibilities.

MARGINAL ANALYSIS

If the objective function were concave and if the effects of investments in each subsection were independent, the cost-effectiveness function could be easily found by marginal analysis. That is, one could simply find which might be the most effective lane to add first, which second, and so on. This requires a minimal computational effort. A maximum of 90 evaluations (30 subsections times a maximum of the three lanes it is possible to add) would be required.

Unfortunately, because the feasible region is nonconvex and the subsections are interdependent, this marginal analysis procedure does not guarantee that one can find the optimal cost-effectiveness curve $F^*(C)$. In fact, in situations such as these, marginal analysis is frequently known as the myopic rule because, by focusing on the value of immediate improvements, one may short-sightedly be led into suboptimal arrangements. Yet, although it is known that using marginal analysis here does not guarantee the calculation of $F^*(C)$, such a method will probably not produce extremely inefficient solutions. And, thus, because an approximate $F^*(C)$ is needed without looking at all 2^{30} or 3^{30} combinations, marginal analysis is used to approach $F^*(C)$. The results, however, must be checked for suboptimality.

A further problem must be dealt with before marginal analysis can be applied. Note that all lane improvements of the optimal lane arrangement may not be built at one time; rather, a plan for sequential construction may be adopted. The general optimal cost-effectiveness curve $F^*(C)$ is no longer applicable because, although a lane arrangement corresponding to any point on $F^*(C)$ can be built, it would be impossible to pass from this point to most higher points without destroying part of the freeway. In short, if the improvement program is to be staged over time, it would be desirable to have each point on the optimal cost-effectiveness curve a subset of higher points, and this is not the case with $F^*(C)$.

To overcome this difficulty requires that a sequential investment cost-effectiveness curve be defined such that each point on the curve corresponds to a lane arrangement that is a subset of the next more costly point. Here lane arrangement A is a subset of lane arrangement B, if the set of improved subsections corresponding to A is a subset of the improved subsections corresponding to lane arrangement B. It is reasonably easy to generate a sequential investment cost-effectiveness curve, but there are many such curves. One sequential curve that reasonably approximates the solution of the general problem is $F_{CE}(C)$ having the property that most of its extreme points are members of S^*, i.e., optimal extreme points of $F^*(C)$. Such a cost-effectiveness curve most closely approximates $F^*(C)$ and thus ensures that any sequential plan using its extreme points is optimal at any stage in its development.

A straightforward way to define the sequential investment cost-effectiveness curve $F_{CE}(C)$ is as follows. Start with the lane arrangement corresponding to the existing freeway; call this arrangement $\overline{X}_0$. Make a FREEQ run for this lane arrangement, and note the subsection(s) where the volume-capacity ratio is equal to one. These subsections are designated as the bottlenecks corresponding to lane arrangement $\overline{X}_0$. In general, the addition of lanes at these subsections will be most effective. For each of these bottlenecks, define a lane arrangement $\overline{X}_i$ such that the i th bottleneck and only those subsections needed to make a feasible lane arrangement are widened. For each $\overline{X}_i$, a FREEQ run is made to obtain the effectiveness $G(\overline{X}_i)$. The most cost-effective improvement is $\overline{X}_A$, defined as the $\overline{X}_i$ for which $[G(\overline{X}_i) - G(\overline{X}_0)]/[C(\overline{X}_i) - C(\overline{X})]$ is maximum. The process of finding $\overline{X}_A$ for a given $\overline{X}_0$ is referred to as a stage in the marginal analysis process.

For purposes of identification, the most cost-effective lane arrangement for a stage is labeled by adding an A to the label of $\overline{X}_0$, the second most cost-effective arrangement is labeled by adding a B, and so on. These steps are repeated from stage to stage by considering $\overline{X}_A$ of a stage as $\overline{X}_0$ of the next stage. That is, suppose lane arrangement $\overline{X}_A$ is built, and then ask the question, What is the next most cost-effective lane arrangement? The answer to this question yields $\overline{X}_{AA}$. The process is continued until the FREEQ runs associated with some lane arrangement $\overline{X}_{A...A}$ results in no bottlenecks. The cost-effectiveness curve $F_{CE}(C)$ is defined by connecting each of the points labeled only with A's by a step function.

The sequential investment analysis described was used to calculate the cost-effectiveness curve $F_{CE}(C)$ for Eastshore Freeway northbound. The successive stages of improvements from the existing freeway conditions to noncongested freeway conditions are shown in block-diagram form in Figure 3 and in cost-effectiveness form in Figure 4.

The block diagram of Figure 3 begins with the existing freeway conditions as shown in stage O. In stage I there are four bottlenecks represented by four blocks. The first bottleneck can be removed by adding a lane to subsections 5 and 6; the second bottleneck can be removed by adding a lane to subsections 20 and 21; the third bottleneck can be removed by adding a lane to subsection 25; and the fourth bottleneck can be

Figure 1. Feasible lane arrangement network.

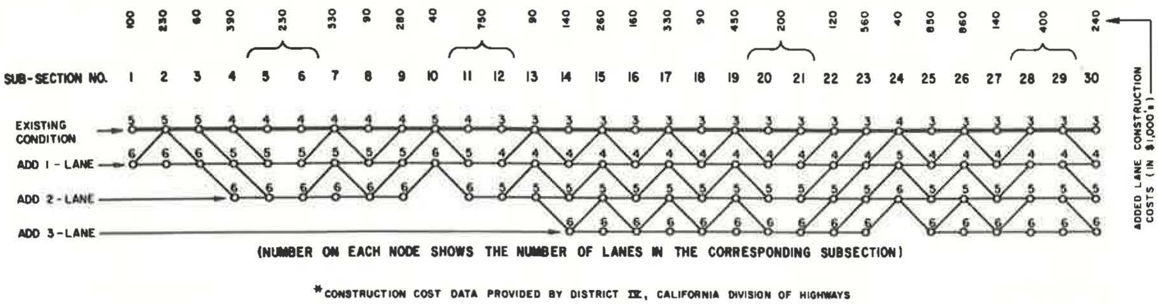


Figure 2. Optimal cost-effectiveness curve.

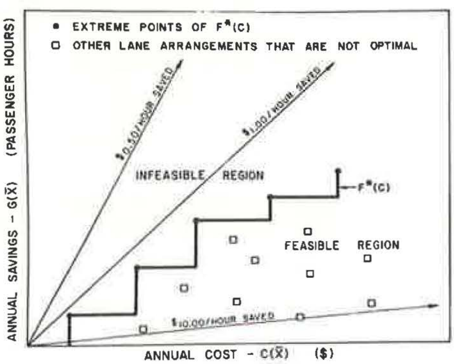
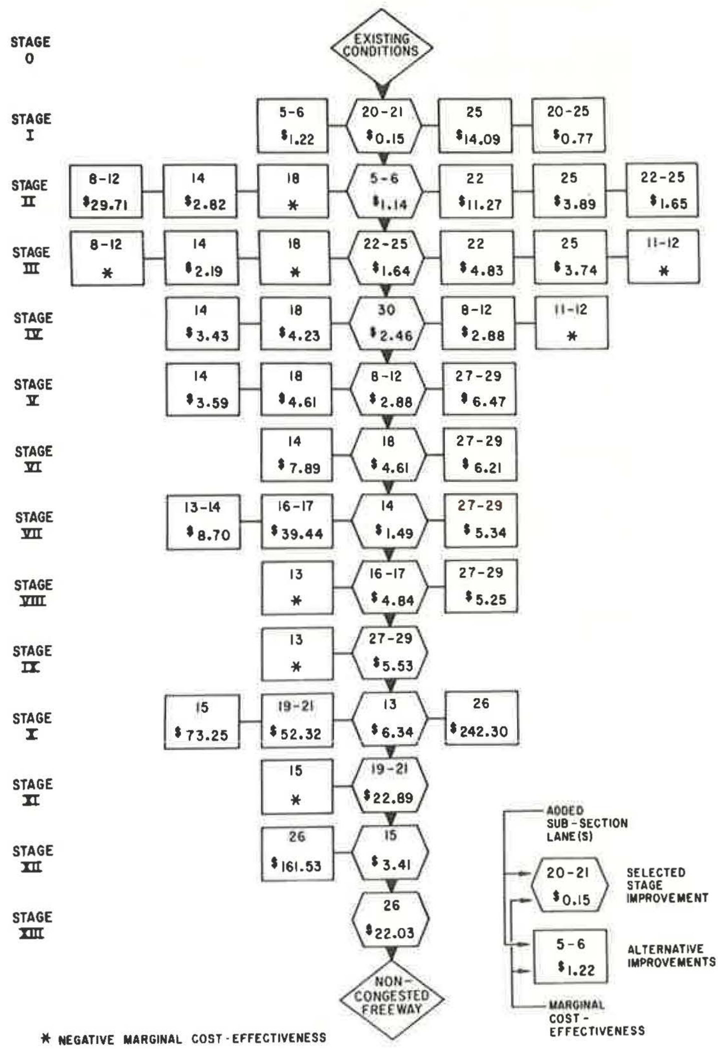


Figure 3. Block diagram of most cost-effective lane arrangement method.



removed by adding a lane to subsections 20, 21, 22, 23, 24, and 25. The marginal cost-effectiveness ratio of each improvement is marked in the corresponding box. The most cost-effective improvement plan would be to add a lane to subsections 20 and 21, and such a plan has a marginal cost-effectiveness ratio of 15 cents per passenger-hour saved.

The process is continued until at the completion of stage XIII the freeway is not congested and the final marginal cost-effectiveness ratio is $22.03 per passenger-hour saved. At each stage the most cost-effective alternative is selected and is included as a subset of all further alternatives.

Results of this investigation of the various improvement plans are shown in cost-effectiveness form in Figure 4. Each point on this diagram corresponds to one of the alternatives shown in Figure 3 and represents a particular improvement that has a specific increased annual cost and savings, in passenger-hours, when compared to the existing freeway conditions. There are a total of 48 alternative improvement plans, and they fall into three groups. The first group consists of the solid dots that lie on the $F_{CE}(C)$ enclosure curves. These are the 13 optimal sequential stage alternatives. The second group is denoted by squares that represent the alternatives that are less cost-effective and lie below the $F_{CE}(C)$ enclosure curve. The alternatives in the third group are denoted by triangles and deserve particular attention. They represent alternatives that lie above the $F_{CE}(C)$ enclosure curve and are for the most part members of the optimal set S^*. However, note that none of the members of this group dominates (i.e., more effective at a smaller cost); this shows the inoptimality of any of the extreme points of $F_{CE}(C)$. Thus all 13 extreme points, as far as these data are concerned, are members of S^* as desired. On the other hand, even though most of the members of the triangle group are optimal, they do not lead to nor are they subsets of later alternatives lying above the $F_{CE}(C)$ enclosure curve.

SENSITIVITY ANALYSIS

Inasmuch as the marginal analysis we used only leads to an approximation of the optimum solution set S^*, it is essential that the goodness of this approximation be explored by sensitivity analysis using alternate procedures. Two underlying philosophies were maintained in these additional analytical procedures. First, it was assumed to be always more cost-effective to add a lane to a bottleneck subsection before considering adding a lane to a nonbottleneck subsection. Second, it was assumed desirable to employ the sequential investment approach, that is, to select a sequence of alternatives in which each stage is a subset of all following stages.

The sequential investment cost-effectiveness curve $F_{CE}(C)$ was essentially generated by a marginal analysis procedure. At each stage the most cost-effective alternative was selected, and all future analyses branch from that alternative only. Because of the nonconcavity of the objective function, it is entirely possible that this marginal analysis is myopic and that a sequence that is ultimately optimal branches from an alternative that is not the most cost-effective at some stage. We can thus consider alternative procedures for automatically generating sets of desirable alternatives.

An alternative, intuitive way to solve the sequential investment problem is as follows. Proceed as in the marginal analysis method, but for each stage select the next improvement plan on the basis of which lane arrangement reduces delays the most rather than on the basis of which is cost-effective. This method was used to calculate a new sequential investment cost-effectiveness curve, labeled $F_E(C)$, for Eastshore Freeway northbound. The successive stages of improvements from the existing freeway conditions to noncongested freeway conditions are shown in block diagram form in Figure 5 and in cost-effectiveness form in Figure 6.

Figure 5 is similar to Figure 3 except that the procedure in the sequence is to select always the most effective lane arrangement rather than the most cost-effective lane arrangement. Stage O represents existing conditions, and the sequence continues until stage XI is completed and no congestion exists along the freeway. At each stage the most effective alternative is selected and is included as a subset of all further alternatives.

Figure 4. Cost-effectiveness of most cost-effective lane arrangement method.

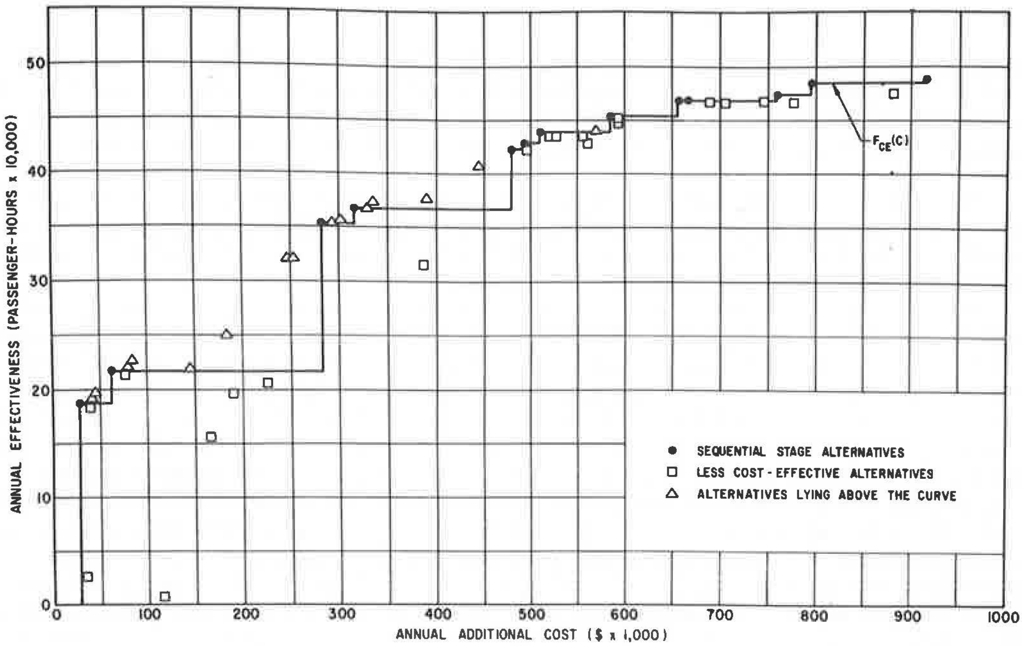


Figure 5. Block diagram of most effective lane arrangement method.

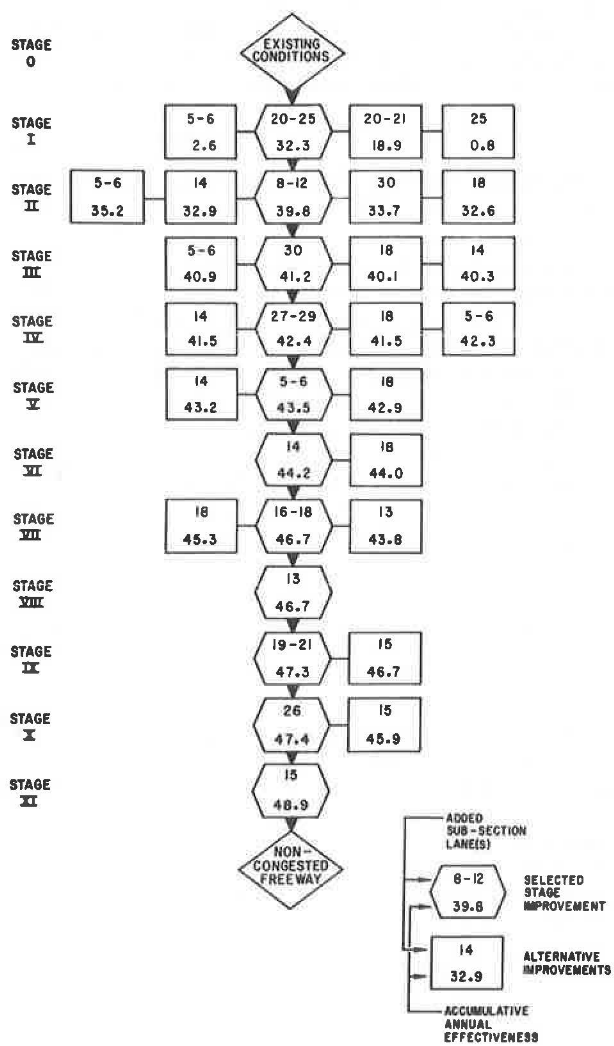


Figure 6. Cost-effectiveness of most effective lane arrangement method.

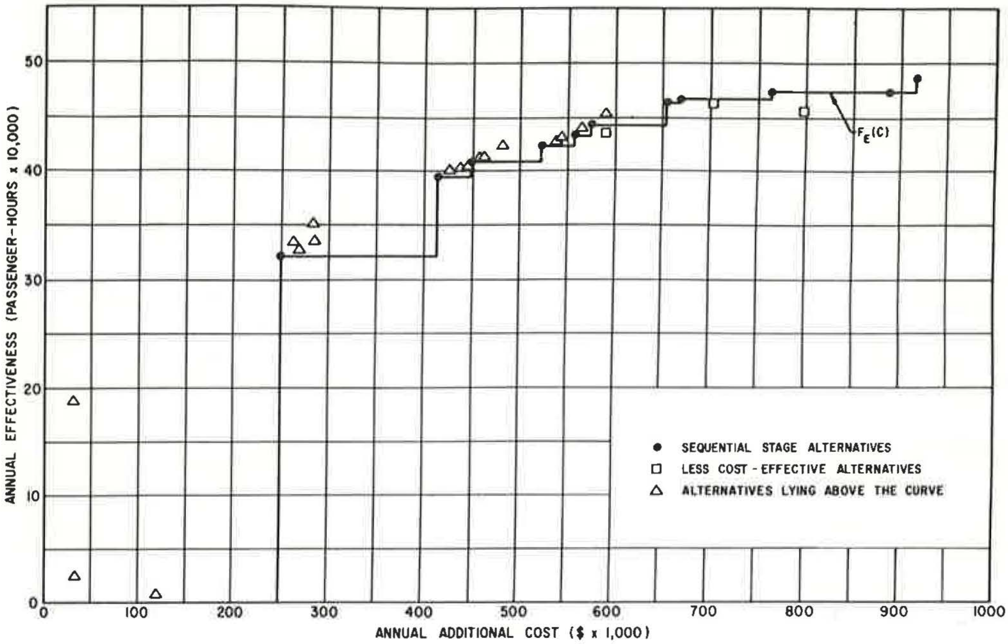
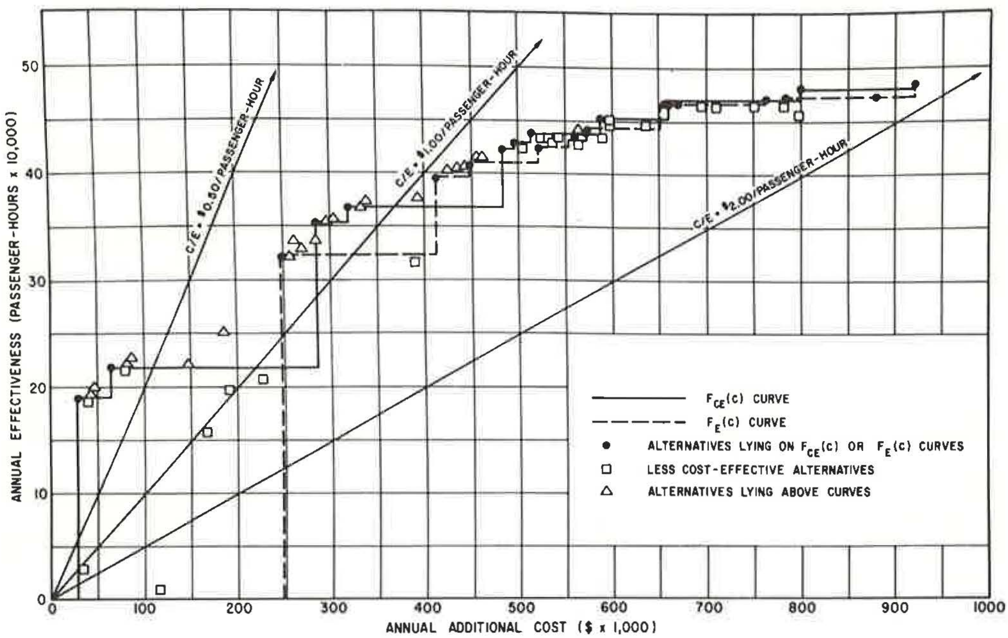


Figure 7. Summary of cost-effectiveness diagrams.



Results of this investigation of the various improvement plans are shown in cost-effectiveness form in Figure 6. Each point on this cost-effectiveness diagram represents a particular improvement that has a specific increased annual cost and savings in annual passenger-hours when compared to the existing freeway conditions. There are a total of 31 alternative improvement plans shown on the diagram of which 11 points lie on the $F_E(C)$ curve, 17 points above the $F_E(C)$ curve, and three points below the $F_E(C)$ curve. In comparison with the results obtained using the most cost-effective lane arrangement method, this method is less desirable because it has fewer extreme points and because some of its extreme points are not optimal. Note upon careful inspection of Figure 7 that the fourth and fifth extreme points of $F_E(C)$ are dominated by the fifth, sixth, and seventh extreme points of $F_{CE}(C)$. Thus, $F_{CE}(C)$ is a better approximation to $F^*(C)$ than is $F_E(C)$.

A second set of sensitivity analyses was based on the experience and judgment of highway designers. During February and March 1972, three 2-day systems analysis workshops were held for the California and Nevada state highway departments. The participants were grouped into three-man study teams and requested to generate the sequential investment cost-effectiveness curve as already discussed. Upon completing the program, the study teams were asked to investigate other alternatives they felt might lie above the $F_{CE}(C)$ curve, based on their experience and judgment. The results of these additional investigations are shown in Figure 7, which also includes the results of the previous investigations as well as the $F_{CE}(C)$ and the $F_E(C)$ curves. Only a few alternatives were found to lie above the $F_{CE}(C)$ curve, and none of these dominated any of the extreme points of $F_{CE}(C)$.

SUMMARY

The sequential investment cost-effectiveness analyses proved to be a practical method of calculating an approximate solution to the problem of optimum expansion of congested freeways. Numerical experience seems to indicate that the resulting $F_{CE}(C)$ curve is nearly optimal. Further, the method is simple, and the solution required only 48 runs out of a possible 2^{30} to 3^{30} alternatives.

From a practical point of view, two specific redesign alternatives were identified. These alternatives are attractive because they have low cost per unit of effectiveness, they have low construction costs, and their improvement is contained in all later extended improvement plans. Adding a lane to subsections 20 and 21 is estimated to cost $200,000 and will result in a cost-effectiveness of 15 cents per passenger-hour saved. Adding a lane to both subsections 20 and 21 and 5 and 6 is estimated to cost $430,000 and will result in total cost-effectiveness of 28 cents per passenger-hour saved. Both alternatives are now under active consideration by the California highway department.

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SOME CASE STUDIES OF HIGHWAY BRIDGES INVOLVED IN ACCIDENTS

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Accident reports, field evaluations, state police and highway engineer questionnaire replies, and other data sources were used to conduct a general study of accidents involving highway bridges in Virginia. Several geometric characteristics were found to predominate at many of the arterial and primary system bridges investigated. Some of the more salient characteristics were pavement transitions on bridge approaches, approach roadway curvature to the left, narrow bridge roadway widths, intersections adjacent to bridges, and combinations of these and other geometric factors. On Interstate highway bridges, poor surface conditions were found to prevail during a significantly high proportion of accidents. Several case studies are presented that illustrate some of the characteristics of bridge sites that have been involved in highway accidents.

•BASED ON an average during the period 1966 to 1969 inclusive, 25.1 percent of the accidents on Virginia's Interstate, arterial, and primary highway systems were of the fixed-object type, whereas 30.9 percent of the deaths were associated with this accident type (1). As indicated from the data given in Table 1, one of the most formidable of the various types of fixed objects is the highway bridge. These data can be illustrated more vividly by expressing accident severity for any given year and type of highway system (or systems) in the form of a severity index. For any general accident category, we can define

$$SI = \frac{D_p}{A_p}$$

where

SI = severity index,
D_p = proportion of persons killed, percent, and
A_p = proportion of all accidents, percent.

Thus, the relative severity of accidents involving highway bridges becomes more apparent, as is shown in Figure 1. In this figure the average severity of all accidents of all types on any given highway system would have an SI of unity. Comparatively, then, general fixed-object accidents are more severe than average; and accidents involving bridges are roughly twice as severe as the average accident occurring over the 4-year period illustrated.

To combat the severity of accidents involving structures, recent Virginia bridge designs have incorporated the General Motors type of safety parapet wall (2), wherein the approach roadway guardrail is anchored to the face of the wall at each end of the structure, and the full roadway shoulder width is carried across new bridges wherever possible. In addition, electronically controlled ice warning devices (3) have been installed at a number of hazardous bridge locations. In concert with this progress, a study was undertaken to identify some of the design and geometric features and other conditions, as noted in the Highway Safety Action Program (4, 5), that could possibly be related to the frequency or severity of accidents or both at bridge sites.

DATA SOURCES AND PROCEDURES

The following data sources were used in the study:

1. Standard form SR300 for Virginia State Police accident reports,
2. Questionnaire replies submitted by the six Virginia State Police divisions and the eight highway district offices,
3. Engineering and geometric data obtained from the original roadway plans for a select group of Interstate highway bridge sites involved in accidents during 1966, and
4. General physical and geometric data obtained from field inspections of a number of arterial and primary system bridge sites.

From the accident report data, a number of bridge sites were detected that had been the scene of several accidents during 1966. For those sites that appeared to have experienced an unusually high number of accidents, accident reports for subsequent years through 1969 were reviewed.

To utilize the experiences of state police officers and the district highway field engineers, questionnaires were mailed to each of the six state police divisions and eight highway districts. The same questionnaires, which were limited to two general but broad requests, were mailed to each organization. The first request was that the respondent list those bridges in his area that, in his view, had been the scene of more than a normal number of accidents and that he provide any information possible regarding those sites listed. The second request solicited any general remarks or suggestions that the respondent wished to make regarding hazardous conditions at bridge sites.

From the information in the accident reports and the questionnaire replies, a list of bridge sites was compiled, and 30 arterial and primary system bridges were randomly selected for field inspection. In addition, a select group of Interstate bridges (those involved in more than two accidents during 1966) were studied separately by obtaining the engineering and geometric data from the original roadway plans.

EVALUATION OF QUESTIONNAIRE REPLIES

Table 2 gives the factors that the police officers and engineers mentioned most frequently as contributing to accidents at certain bridge locations. The three most frequently mentioned contributing factors were (a) narrow bridge roadway, (b) curved approach roadway alignment, and (c) curved bridge alignment. It is interesting to note that the order of these factors in Table 2 is the same for each reporting group. Nearly half the bridges commented on by each group were felt to have inadequate roadway width. Curved approach and curved bridge alignment were cited as factors contributing to hazardous conditions at approximately a quarter of the sites commented on. The combined effects of restricted bridge roadway width and curved approach roadway alignment or curved bridge alignment were cited in approximately half the cases where curvature was considered a contributing factor. Other factors of accord between the two groups were downhill approach and inadequate vertical clearance.

More subtle factors such as approach roadway lane drops and transitions, intersections adjacent to bridges, and snow and ice on bridge decks were cited much more frequently by police officers than by highway engineers. Approach roadway lane reductions and transitions at the entrances to some bridges were felt to contribute to the likelihood that fixed objects (e.g., bridge and guardrail) would be involved in accidents. Intersections and interchange ramp connections adjacent to bridges were also cited as constituting a hazard because the bridge railings obstruct vision, and entering and turning traffic increases the possibility of accidents involving collisions with the structure. Although the questionnaires that provided the information given in Table 2 were subjective in nature, substantial support from the work of others exists (7, 8, 9, 10, 11).

A general comparison of the two groups of questionnaire replies revealed several facts that might be expected but, nonetheless, are worthy of mention. First, on-the-scene accident investigation is one of the regular duties of police officers. Consequently, because of their experience, police officers would be more likely to recognize roadway factors that might contribute to accident frequency and/or severity than would most highway engineers. Second, the replying engineers recognized and reported many of the

Table 1. Percentage of accidents involving Interstate, arterial, and primary highway bridges in Virginia.

Year	Interstate Highways: Percentage of All Accidents	Interstate Highways: Percentage of All Persons Killed	Arterial and Primary Highways: Percentage of All Accidents	Arterial and Primary Highways: Percentage of All Persons Killed
1966	3.7	7.3	1.8	3.9
1967	3.2	6.8	1.5	2.9
1968	2.7	5.1	1.4	3.7
1969	3.1	9.0	1.5	3.0
Average	3.2	7.1	1.6	3.4

Note: Data developed from statistics of Virginia Department of Highways (6).

Figure 1. Severity of accidents involving bridge structures and fixed objects.

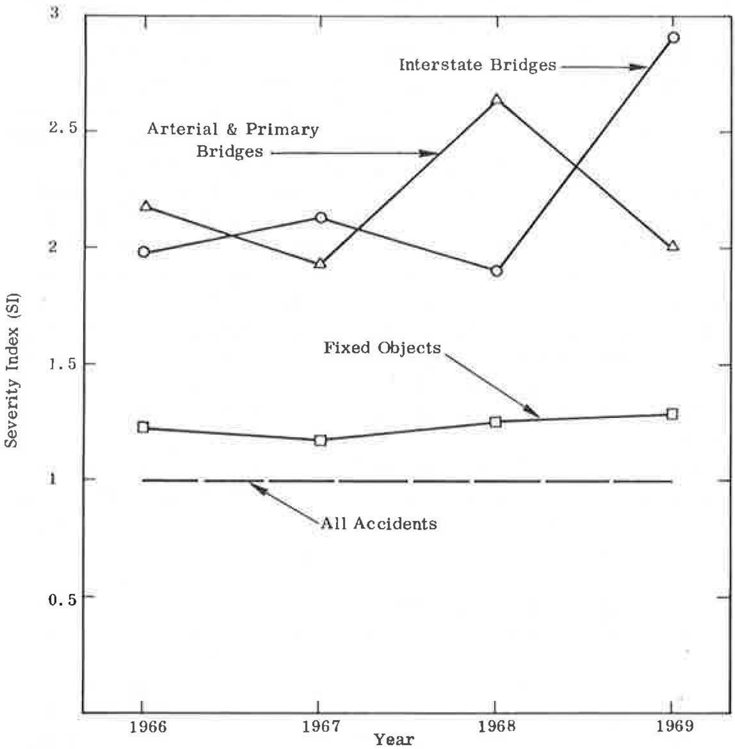

Table 2. Factors contributing to accidents at bridge sites.

Contributing Factor	State Police Officers: No. of Bridges	State Police Officers: Percentage of Total Bridges Cited	Highway Engineers: No. of Bridges	Highway Engineers: Percentage of Total Bridges Cited[a]
Bridge roadway too narrow[b]	32	46	24	48
Curved approach roadway[b]	19	28	11	22
Bridge curved[b]	16	23	10	20
Intersection adjacent to bridge	8	12	1	2
Approach lane drop and transitions at bridge	6	9	1	2
Downhill approach[b]	5	7	6	12
Snow and ice	5	7	-	-
Slippery when wet	4	6	-	-
Inadequate vertical clearance	3	4	3	6
Insufficient curve elevation	2	3	1	2
Rough approach and rough bridge	2	3	-	-
Pedestrian crossing on narrow bridge	2	3	-	-

[a]Percentage based on 50 sites commented on from a total of 79 sites listed by highway engineers.
[b]Combined effects of these factors were frequently cited.

bridge sites that have had abnormally high numbers of accidents, but some engineers appeared more inclined than the police officers to accept driver errors as the basic cause of most accidents.

ARTERIAL AND PRIMARY SYSTEM BRIDGES

General Geometrics

Along with the questionnaire replies, the accident reports for 1966 were used to compile a list of accident-prone locations. Field inspections were made of 30 bridge sites randomly selected from this list, and the alignment, grade, roadway widths, and so forth were noted for each bridge and its approaches. The four most prevalent geometric factors found at the locations were (a) downhill approaches, (b) narrow bridge roadway widths, (c) curved approach roadway, and (d) entrances or intersections adjacent to the bridge. The order of the dominant factors is much the same as that summarized from the state police questionnaire replies with the exception of downhill approach. Considering them as an individual element, Kihlberg and Tharp (7) found gradients to be less significant than factors such as curvature and intersections. Twenty-one percent of the accidents reviewed in the present study, however, occurred when snow and ice conditions existed, so it is probable that downhill gradients are often a contributing factor from this standpoint in addition to affecting vehicle speeds.

Fifteen (68 percent) of the structures with downhill approaches had approach roadway curvature, and 70 percent of those with approach roadway curvature had narrow bridge roadway widths. All three of these factors were present at 50 percent of the sites with downhill approaches. Thus, the high occurrence of combined geometrical factors at the sites surveyed appears to be significant because the likelihood of a bridge site having combined geometrical factors decreases with increased numbers of factors involved. Similarly, only a small percentage of all the arterial and primary highway bridges have intersections or pavement transitions immediately adjacent to them. Yet, intersections (or entrances) and pavement transitions were located at 43 and 13 percent respectively of the sites studied.

Findings similar to those discussed have been reported by Kihlberg and Tharp (7), who found that the presence of structures, curvature, gradients, and intersections generally has an increasing effect on accident rates. More significantly, they found that combinations of any of these elements generate higher accident rates than do individual elements.

Eighty-five percent of the study sites having approach roadway curvature had left-curved alignment in at least one direction of approach, whereas only 45 percent were curved to the right. Brown and Foster (10), in a study of bridge accidents in New Zealand, found that the right-curved approach alignment contributed to 3 times more accidents at the left approach and bridge end post than did left-curved alignment. Because New Zealanders drive on the left side of the road, the analogous situation in the United States would be for more accidents to occur on left-curved approaches. Thus, the present study result is consistent with that of the New Zealand study.

Bridges with narrow roadway widths, particularly those with widths equal to or less than the approach pavements, have been shown to experience high accident rates (8, 9). Brown and Foster (10) found that 70 percent of the accidents occurred where the ratio of the bridge roadway width W_B to the approach roadway width W_R (including the shoulder width) was ≤ 0.79. A similar ratio could be determined on 19 of the sites surveyed in this study. Seventeen, or 90 percent, of these had W_B/W_R ratios of less than 0.79. Sixteen, or 84 percent, had ratios less than 0.69.

Case Studies

Discussion of some study examples serves the following purposes:

1. Indicates the general types of accidents that occur at some typical accident-prone bridge sites,
2. Explores possible safety improvements at some of these locations, and
3. Illustrates how on-the-site field inspections supplemented by accident report information can sometimes reveal roadway factors that could contribute to accidents.

Case Study 1—The first case study bridge has had a history of accidents, one fatal, and was recently involved in a sequence of collisions. When a bridge with a narrow roadway width is located within a passing opportunity section of a two-lane highway such as that shown in Figures 2 and 3, collisions involving the bridge railings appear to occur more frequently than when this situation does not exist. This 22-ft long, 23-ft clear roadway bridge was involved in a passing accident in August 1969 when a westbound vehicle met an eastbound vehicle passing another eastbound vehicle. The westbound vehicle went into a skid to avoid the eastbound vehicles, crossed to the opposite side of the road, knocked out the east end of the bridge railing, and went over the edge of the structure. The railing was rebuilt, but in March 1970 an eastbound vehicle, forced over by a passing vehicle, knocked out the west end of the same rail. Subsequently the rail was rebuilt, but in May 1970 an eastbound tractor-trailer, after being forced off the edge of the approach roadway, struck the same rail knocking it out entirely. The rail was again rebuilt, and in November 1970 the east end of the railing on the opposite side of the road was knocked out by an out-of-control eastbound vehicle. The last two accidents were single-vehicle property damage types in which the driver lost control after running off the edge of the pavement in the area of the intersection adjacent to the bridge. Note also that there is no pavement edge striping across the intersection. Under certain circumstances this could be a contributing factor and is discussed further in a later case study.

It is difficult to determine the total economic losses from the series of accidents described because property damages are only estimated by the reporter, some damages are not reported at all, and medical expenses are unknown. A reasonable estimate of the property damages, which occurred during a 15-month period, can be made as follows:

Item	Cost (dollars)
Personal property damages on two reported accidents	3,000
Personal property damages on two unreported accidents	1,000
Four repairs of handrail at average cost of $432 each	1,728
Total	5,728

The handrails were repaired by state forces. If medical costs, lost wages, etc. were included in this estimate, the total economic losses would, of course, have been higher.

Case Study 2—The second case study bridge was very similar to the first. It too was located on a two-lane highway in a passing opportunity area and had a narrow roadway width. Several accidents and one fatality have resulted from collisions at the site in recent years. This 32-ft long structure, however, was recently widened from a 23- to a 40-ft roadway width at a cost of $17,000 (cost of work performed by state forces).

Curves that can be used to forecast accident reductions and fatality-injury and property damage reductions through the widening of bridges have been developed by Jorgensen and Associates (12) and are shown in Figures 4 and 5. By extrapolating the curve D = 0 of Figure 4, we can estimate that an average reduction in accidents of approximately 95 percent can be expected from the 17-ft widening of the second case study structure. A similar reduction in property damages and injuries could be expected by extrapolation of the curves shown in Figure 5. Benefit and cost estimates can be calculated for the widening improvement by using the methodology presented by Jorgensen (12). Thus, for an annual cost of $985 (based on a 30-year service life), widening of the bridge will yield estimated average annual benefits of $11,350 for a benefit-cost ratio of 11.5 (1). Inasmuch as these two case study structures are quite similar, the first bridge could be widened for approximately the same cost as the second. The annual cost of such an improvement to the first structure would be less than one-fifth of the $5,728 property-damage estimate for the recent series of accidents.

Installation of guardrail in lieu of widening at either of these two bridges would probably not reduce the number of accidents. Also, maintenance costs for repairs would likely remain high if such an alternative were selected. Again using the same forecasts and methodology (12), we can estimate that the average annual benefits to be derived from a guardrail installation would be $2,520, whereas the annual cost would be $433. This yields a benefit-cost ratio of 5.8 (1). Thus, widening in each of these two cases would be the better alternative.

It should be emphasized that the benefits to be derived from guardrail installations at bridges are due solely to a reduction in accident severity. Therefore, the benefits derived from the widening of short-span bridges typical of those discussed should not be confused with the need to reduce the severity of collisions with structures typical of the one shown in Figures 6 and 7. In the latter type of situation, many older bridges that constitute potential fixed-object hazards should be upgraded to comply as nearly as possible with at least the following three of 10 bridge rail service requirements developed by Olson et al. (11):

1. A bridge rail system must laterally restrain a selected vehicle,
2. A bridge rail system must remain intact following a collision, and
3. A bridge rail system must have a compatible approach rail or other device to prevent collisions with the end of the bridge rail.

Progress toward meeting these requirements can be made. In Figure 8, for example, structural continuity between the approach rail and bridge rail has been obtained by a closer spacing of the approach rail posts adjacent to the bridge rail and by continuing the guardrail across the length of the bridge. In addition, the ability of the rail system to laterally restrain a vehicle and to remain intact after a collision is enhanced when the continuous guardrail is anchored to the existing bridge rail. Similar rail systems have been described by Tutt and Nixon (13).

Case Study 3—Slowing, stopping, or turning traffic at intersections, business entrances, and so forth increases accident potential. When bridges happen to be located adjacent to points of high accident potential, their potential for involvement also appears to be increased. A typical example is shown in Figure 9 in which a bridge with a narrow roadway is located adjacent to an intersection where traffic slows or stops for left turns. Collisions with the right bridge rail have resulted from situations in which a vehicle has maneuvered to avoid collision with other vehicles making turning or lane-change maneuvers. A business entrance adjacent to the right approach to the bridge probably adds to the traffic conflicts at this particular location.

Case Study 4—In the next case study, seven fatalities resulted from two single-vehicle collisions with the right end post of the bridge rail within a period of several weeks; six fatalities resulted from the first and one from the last. Both accidents occurred at night, and visibility was poor due to fog or rainy conditions. In these two accidents and another in the 1966-67 period, driver fatigue could have been a factor. As one approaches the bridge (Fig. 10), there is a transition from two to four lanes occurring simultaneously with a curve to the left. The approach pavement edge marking is discontinued on the right at an adjacent intersection, and there is no centerline lane marking in the pavement transition area. If we consider these factors and the environmental and visibility conditions existing at the time of the accidents, it is possible that each driver mistook the intersection to the right for the main roadway. Accordingly, they could have been misled to the extent that their recovery course headed into the bridge end post. Alternately, if the pavement edge marking was being used as a guide, one would be headed on a course beginning from the point where the pavement edge marking is discontinued and directed toward the bridge end post even though the road actually curves leftward. Thus, under the circumstances, the pavement transition, the curve to the left, the intersection to the right, and the discontinuation of the pavement edge marking all could have been contributing factors in these accidents.

Case studies 3 and 4 suggest that intersections should be located as far away from bridge sites as possible. Where intersections are located adjacent to structures, the main roadway pavement edge marking should be continued across the intersection. When advantage can be taken of main roadway gradients, intersections should be located to give maximum sight advantage over the bridge railings.

Case Study 5—Each approach to this case study bridge (Fig. 11) has a transition from four lanes to two lanes. It might be expected that transitions of this type would tend to have an effect similar to that of widening the roadway but not the bridge. This

Figure 2. A narrow bridge located within a passing opportunity section of two-lane highway with intersection to the right adjacent to the structure.

Figure 3. Same bridge shown in Figure 2 with east end of north rail knocked out.

Figure 4. Forecast chart of accident reduction through bridge widening (12).

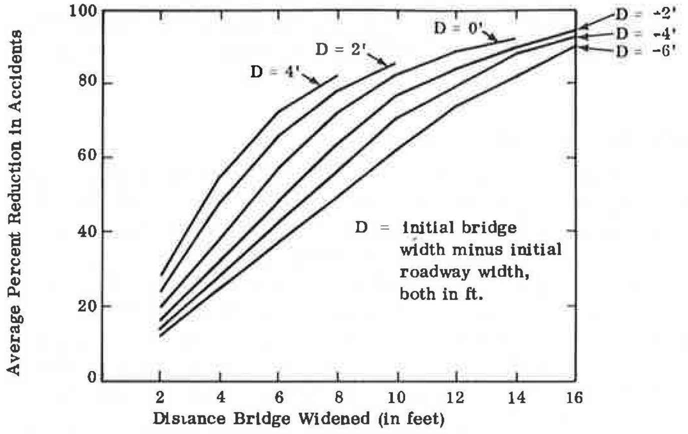

Figure 5. Forecast chart of fatality-injury and property damage reduction through bridge widening (12).

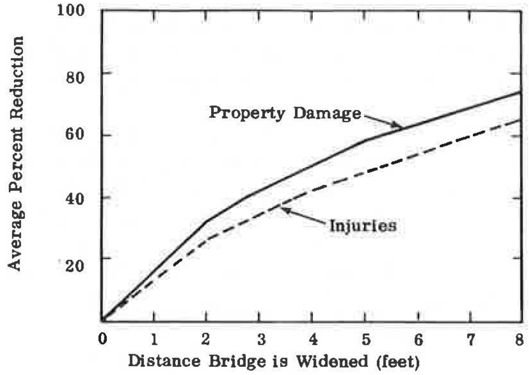

Figure 6. Restricted roadway width and exposed ends of rigid concrete railing.

type of practice, as prior studies have shown (9, 12, 14), results in increased accident rates. Many of the accidents at the structure in question have been related to passing maneuvers on the bridge or its approaches. In a recent accident of this type, a truck went through the steel railing and off the bridge; the driver was killed. Although the bridge is now marked as a no-passing zone, it appears that the four-lane highway on each side of the bridge creates a psychological "freedom to pass" attitude that prevails on the two-lane bridge as well. The rail penetration incident might also suggest that reinforced concrete parapet walls should always be used on the larger, higher, major structures such as the one illustrated.

Each of the last two examples demonstrates the general finding that pavement transitions on bridge approaches should be avoided. When transitions are necessary, they should be completed well in advance of the structure to allow drivers maximum opportunity to adjust to the change prior to entering the bridge.

Case Study 6—Inspection of the scene of an accident can sometimes reveal contributing roadway factors that are more related to maintenance or construction than to design and obsolescence. An example of such a case is shown in Figure 12, a bridge on which several skidding accidents occurred during wet surface conditions. Significant portions of the deck had been repaired with an epoxy surfacing material that had not been treated with a deslicking grit (sand) during the initial application. McKeel (15) has found that epoxy overlays lose their skid resistance rather rapidly as the initial grit application is lost due to wear. An epoxy surface with no initial deslicking treatment could thus be expected to polish rapidly under traffic wear and to become very slick.

INTERSTATE HIGHWAY BRIDGES

For 27 bridge sites that had two or more accidents during 1966, a summary of certain approach roadway geometrics and accident data was tabulated. Sixteen of the sites have curved approaches, 13 of these being 1 deg or less. Twenty-three of the sites have downhill approaches, and generally the higher the percentage of grade is and the higher the degree of curvature is, the greater will be the relative percentage of accidents during wet surface conditions. Approximately 50 percent of the accidents occurred when the bridge deck surface was either wet, snowy, or icy, whereas, for comparison, these conditions existed in 31 percent of all accidents on the total Interstate system during 1966 (6). Of 42 individual bridges involved in two or more accidents in 1966, 62 percent are approached by a downhill grade of 1,000 ft or more in length. An additional 24 percent have downhill approach lengths of between 500 and 1,000 ft. Thus, the most dominant factor in the Interstate highway bridge accidents appears to be adverse surface conditions, particularly when long, steep approach grades are present.

At one Interstate highway bridge site, six of 17 accidents reviewed for the period 1963 to 1967 involved icy conditions on the bridge deck. These two structures are approached on the northbound lane by a 1.4 percent downhill grade of approximately 1,600 ft in length and on the southbound lane by a 3.5 percent downhill grade of approximately 600 ft in length. Superposition of icy deck conditions on the long and relatively steep downhill approaches could explain part of the high accident rate at this location.

Of all Interstate highway bridge accidents in 1966 that were reviewed, 33 percent occurred under icy or snowy (excluding wet) surface conditions. The comparable figure on primary and arterial system bridges was 21 percent. The higher percentage on the Interstate highway bridges suggests that the freer traffic flow and higher speeds on Interstate highways contribute to higher accident rates during icy and snowy conditions. Either many drivers apparently are not aware of the fact that, when moisture is present during freezing temperatures, ice will form on bridge decks before it does on the roadway, or they are not making adequate speed adjustments for poor surface conditions.

It was difficult to evaluate the bridge-approach roadway relationships on all of the bridge sites investigated due to variations in ramp intersections at interchanges. At 19 of the sites, however, it was found that 63 percent of the most accident-prone Interstate highway bridges had clear roadway widths of 28 to 30 ft, whereas the remaining 37 percent were 40 to 42 ft. Seventy-four percent of the sites had a bridge-approach

Figure 7. Head-on collision with right end post of bridge shown in Figure 6.

Figure 8. Approach guardrail continued across a bridge.

Figure 9. Bridge with narrow roadway located adjacent to intersection (center) and business entrance (right foreground).

Figure 10. Bridge located at end of pavement transition from two to four lanes.

Figure 11. Transition from four to two lanes on approach to major bridge crossing.

Figure 12. Site of several skidding accidents on downhill, superelevated deck treated with epoxy surface treatment with no initial deslicking sand applied.

roadway width ratio of less than 0.8. Though these data are limited, the results are in line with those on the primary and arterial system; i.e., bridges with W_B/W_R ratios less than 0.8 are generally more accident prone than those with greater ratios.

SUMMARY

1. Probably because accident investigation is one of their regular duties, state police officers are more likely to recognize the more subtle roadway factors that might contribute to accident frequency and/or severity at bridge sites than are most highway engineers.

2. Some of the engineers replying to the study questionnaire appeared more inclined than did the state police to accept driver errors as the basic cause of most accidents. There was good general agreement between the two groups, however, regarding the most common roadway factors felt to contribute to accidents at bridge sites.

3. The results of the field inspections conducted in this study and the summary of the state police questionnaire comments were in general agreement regarding the most common roadway geometrics at arterial and primary system bridge sites with accident histories. These factors are (a) narrow bridge roadway width—accident potential appears to be high at bridge sites where the ratio of bridge roadway width to approach roadway width (including the approach shoulder) is less than 0.80; (b) approach roadway curvature—left-curved approach alignment appears to be a more dominant factor than curvature to the right; (c) pavement transitions on bridge approaches—transitions from four to two lanes and vice versa on bridge approaches appear to increase the potential for accidents involving components of the bridge; (d) intersections adjacent to bridges; (e) downhill approach gradients; (f) bridge curvature; and (g) combinations of any of these factors.

4. The severity of accidents at many of the relatively old bridges could probably be reduced by installing approach guardrails that either are effectively anchored to the existing bridge rail or continue across the full length of the bridge.

5. An analysis of a single-span bridge with a narrow roadway width that has been widened suggests that widening would yield favorable benefit-cost ratios for similar structures having accident histories.

6. On two-lane highways, narrow bridges that are located within passing opportunity sections appear to have a high potential for being involved in accidents.

7. Many bridge railings will not restrain a standard-sized vehicle, nor will they remain intact following a collision.

8. The discontinuation of main roadway pavement edge striping at intersections adjacent to bridges may be misleading or confusing to motorists approaching them under certain adverse environmental or physical conditions.

9. Intersections and entrances adjacent to bridge sites appear to increase the potential for collisions. Factors apparently involved include obstruction of view due to the bridge railings, increased traffic conflicts at the fixed-object location, and, under certain conditions, confusion on the part of motorists.

10. The most dominant factor in the 1966 Interstate highway bridge accidents studied was adverse surface conditions (wet, snowy, or icy), particularly when long, steep approach grades are present.

11. A larger proportion of accidents (33 percent of the accidents studied in 1966) occur on Interstate highway bridges when icy or snowy surface conditions exist than on primary system bridges (21 percent of the accidents studied). This suggests that many motorists are not making adequate speed adjustments for poor surface conditions on high-speed highway bridges.

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TRADE-OFF ANALYSES IN MAJOR INTERCHANGE DESIGN

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Recent expansion of limited-access, high-speed highways has emphasized the need for safely designed interchanges. Frequently, interchanges are designed as modifications to existing designs to increase capacity or as developments from experience of designers. A more analytic approach is needed. In the study reported, existing design criteria are reviewed and evaluated, and a decision-theory approach to design is developed. This approach assesses the relative operational and safety merits of alternative interchange configurations and suggests trade-offs between these merits and the cost of the alternative.

•RELATIONSHIPS among variations in geometric features (such as ramp curvatures, lengths of recovery zones at exit gore areas, and visibility distances to exit ramp noses) and operational efficiency and safety and cost are not well defined. On the other hand, a number of decisions involving trade-offs between cost and operational efficiency and safety have been and are being made in the major interchange design process. The views of practicing design engineers and traffic operations specialists on trade-off analyses, as derived from information gathered for a larger project, are presented with commentary by the author.

PROJECT OBJECTIVES

Since its introduction more than 4 decades ago, the interchange has established itself as an irreplaceable, though sometimes confusing, element of the world's roadways. In particular, the recent expansion of high-speed, limited-access highways has emphasized the need for major interchanges optimally designed to contribute to unambiguous, safe, high-capacity, predictable interhighway access. The key phrase in this assertion is "optimally designed."

Designs of interchanges are often based on evolutionary changes of past designs or on modifications of existing designs to increase capacity. Hence, the newer designs tend to develop from experience and engineering judgment rather than from a ranking of quantifiable alternatives based on performance. The changes seen in recent demands for highway systems suggest that a more analytic approach is necessary.

Consequently, the Federal Highway Administration sponsored the study on interchange design, the principal phases of which were as follows:

1. Preparation of a state-of-the-art document covering major interchange design and operations,
2. Review and evaluation of existing design criteria and constraints,
3. Development of a decision-theory approach to the design of major interchanges,
4. Development of recommendations to minimize operational problems associated with major interchanges, and
5. Determination of the viability of the various freeway-to-freeway interchange configurations for inclusion in adaptive freeway control schemes.

The project was completed in June 1973. The final report and three interim reports are available from the sponsor.

This paper deals with trade-off analyses (costs versus safety and operational efficiency) and is derived from broader information gathered for the overall project. Primarily, it is an exposition of the views (with contrasts) of design engineers, traffic operations specialists, and persons from the research and academic communities on this topic, including present practice and views on the probable and desirable directions of future practices. A presentation of a related "level-of-merit" design concept is also included.

WORKSHOPS

Two 3-day workshops were held at Pennsylvania State University to aggregate the experiences and personal views of practicing engineers and researchers on major interchange design and operations. In most instances, the attendees were intimately involved in or responsible for design policies within their respective organizations. This paper was derived from the workshop sessions on Trade-Offs; Level-of-Merit Concept, led by the author.

TRADE-OFF ANALYSES

Background Information (From Introduction to Session)

Economic evaluation of interchanges, as such, is not a standard problem inasmuch as the interchange is part of a larger freeway project and the project is generally evaluated as a whole. Thus, project economic analysis, for which there are a number of theories and methods (benefit-cost analyses, annual cost methods, rate-of-return methods, interest charges, depreciation, and so on), is not of interest here.

The problem of largest scope in this discussion is the evaluation of alternative configurations, involving road user benefits and road user cost, within the interchange itself. A problem of small scale is the level of investment in the individual components (higher design standards usually cost more) and is the primary subject of interest.

There are relatively few data available that clearly relate operational and safety benefits to costs of the various design features. Accident data are not sufficiently sensitive to the effects of variations in geometrics to be used as evaluative measures, except in a few cases where only gross geometrics are of interest and the data are corrected for exposure.

It appears from interviews with design engineers in a number of state highway departments that very little trade-off analysis is involved in the design process; rarely does the design engineer make a specific decision on whether to increase the design speed of a specific ramp at a cost of X dollars. In a number of states, considerable interest was expressed in defining the relationships between geometric features and operational efficiency and safety; interest in the relationship with cost was not so pronounced.

On the other hand, a number of decisions involving trade-offs between costs and operational efficiency and safety have been and are being made somehow. These range from the decision not to use diamond configurations for major interchanges through the elimination of loop ramps for turning movements with large volumes to requirements for 50-mph turning roadways for 70-mph through roadways.

Generally, the higher design standards cost more, yet we keep upgrading the standards. (This assumes some sort of cost analysis, but it is not obvious how these decisions are made.)

Many of the less-than-optimal designs and features found on old interchanges resulted from compromises for cost reductions. Today, on the other hand, cost factors seem to be a lesser constraint in the selection and evaluation of alternative component configurations and in the development of design details either because the designers now see a stronger relationship between their design details and the resultant operation and safety or because they are willing to spend more than previously to obtain these desirable ends.

Interviews with engineers from the various state highway departments, conducted within this project, indicate that feedback from operations analysts is usually poor and frequently nonexistent, except for those interchanges that are almost hopelessly inadequate for their opening.

In addition, some highway engineers observe that in recent years final designs are based less on a combination of optimal features than on the necessity to choose among the least objectionable constraints. They are particularly troubled by the fact that local sociopolitical groups, who possess meager information about or experience with roadway design, can force changes (e.g., provision of local access within a major interchange) that seriously impair operations and safety.

Workshop Questionnaire

After the preceding introductory remarks, a set of discussion questions was posed to the workshop participants. These were followed by a period of open discussion and distribution of a questionnaire. The participants were asked to complete and return the questionnaire the next day, thereby giving them an opportunity to discuss the subject further among themselves and to consolidate their thinking. In general, the questions were nearly the same as those presented for discussion. The questions, with answers received, are given in the Appendix.

As can be seen, interest in and necessity for economic analyses decrease somewhat as the design decision becomes more and more specific. This is logical in that the alternative costs become relatively smaller and the overall project constraints are rather well set by the time the design details are selected. A number of respondents indicated that more economic analyses would be desirable but that appropriate methodology was not available. However, there is no clear mandate for the development of this methodology.

Also, some of the answers indicate that "engineering judgment" is the most used decision-making procedure on including "desirable features." It is perhaps surprising, and certainly encouraging, that only about a third of the respondents indicated that their organization had adopted the policy of simply meeting certain minimums.

It is apparent from some responses that experience is the prime input to the design decision process, although considerable attention is being paid to accident record analyses and pertinent research results.

LEVEL-OF-MERIT CONCEPT

As mentioned, cost and some measure of operations and safety are two major trade-off factors receiving consideration in the selection of alternative component configurations (such as left versus right ramps or single versus double exits) and in the specification of design dimensions (design speed for a given ramp, length of acceleration lane in a given situation, etc.). In development of a final interchange design, a number of these trade-off decisions are made, although, perhaps, not consciously.

Design engineers are asked, Is it more desirable, from an operations and safety viewpoint, to provide a single exit (with subsequent branching for left and right movements) or two individual exits? The answer is almost unanimously, Single. However, when then asked which configuration should be established as a design standard to be rigidly adhered to, the answer becomes somewhat less definite, and "hedging" will be noted. Obviously, the hedging comes about because designers feel there are situations in which the single exit should not be selected, and this is often because, in that situation, the double exit could be achieved at considerably less cost.

The same types of questions and answers can be applied to other design features, such as right versus left ramp or length of acceleration lane. In other words, there are known desirable features, but something less is often used because of some cost factor. Designers claim it is impossible to give a set answer to any of these types of questions, which will hold across all situations. A major reason for this is that they are trying to assess cost and merit measures at the same time and, as the combinations are nearly infinite, so are the "correct answers."

It appears, then, that, because no definite universal answers can be had when the two factors are considered together, it would be helpful to decision-makers if they could assess the two factors (cost and operations and safety merit) individually with some degree of certainty and then make their decision on the basis of relative costs and relative merits.

Assessing relative costs will usually be possible although sometimes with considerable difficulty if the alternatives are such that a major portion of the interchange design is involved (such as a decision on a right or left exit). In the case of designating the length of an acceleration lane, the cost analysis may be very simple (if only a little change in earthwork quantities and pavement length is required) or somewhat difficult if the longer lane will also interfere with downstream features, require a larger grade separation structure, etc.

The problem, then, will be to assess the relative level of merit provided by the alternative configurations, or the alternative design dimensions, and then to choose among the alternative levels of performance and the corresponding costs.

Assuming, for the moment, that the specification of alternative merits is possible, the designer is then in a much better position to select the final design. This will still be a highly subjective process, depending largely on the designer's engineering experience and judgment; a benefit-cost analysis is not being suggested.

An example will illustrate the concept. Assume the conditions given in the following:

Configuration	Merit Rating	Additional Cost (dollars)
Single exit (on right)	10	3,000,000
Double exit (both right)	8	2,000,000
Double exit (right and left)	3	0

If the total interchange cost (with double exit, right and left) is estimated at \$40,000,000, which configuration should be selected? If the total interchange cost (with double exit, right and left) is estimated at \$7,000,000, which configuration should be selected? Now assume the ratings are changed to 10, 8, 6; which configuration should be selected?

The fact that different configurations might be chosen under these differing conditions points up the problem of setting definitive configuration selection criteria. Even in this simple example (in practice, other considerations, such as maintenance costs, road user costs, and the like, would also enter the decision-making process), it is not possible to select a single, "always correct" answer.

The merit ratings give some insight to the question, How much better? It is agreed that a single exit is better than one incorporating right and left exits, and therefore using a design incorporating a single exit justifies a higher cost, but how much higher? First, one must determine how much "better" one configuration is than another. The merit ratings, if available, could provide some feel for these qualitative comparisons.

Each time a decision has been made in the past, the designer did go through some similar assessment of the relative merits and costs. The merit ratings, if they can be developed in a credible and acceptable manner, will provide some basis for a rational choice. They would provide a means by which the decisions could be made more consistently by each designer, and more consistent designs could be obtained from various designers.

As another example, assume that a speed change lane (acceleration) from a turning roadway with a design speed of 40 mph to a through roadway with a design speed of 70 mph must be designed. The "Blue Book" suggests that this acceleration lane be 1,000 ft long. Suppose, due to situational considerations, a speed change lane 800 ft long would be \$500,000 less expensive than one 1,000 ft long; which should be selected?

Obviously, a judgment on the importance of that missing 200 ft is required. This assessment is usually made on the judgment of the design engineer. Suppose, however, that credible merit ratings are available: 8 for 1,000 ft and 7.5 for 800 ft. Would this affect the decision in a different manner than if the two ratings were 8 and 4? Would not this degree of specificity help the designer in making this decision?

The next question, obviously, then, is, Can the merit ratings be developed in a manner such that they will be respected and accepted by the design community? It is proposed that the approach to the development of these ratings include a combination of physical analyses, experimental research, and the operational experience of design engineers and operations specialists.

Workshop participants were asked to rate various exit-ramp configurations (Fig. 1) to illustrate the feasibility, and problems, of deriving consensus expert judgmental evaluations. The procedure was as follows. A value of 10 was to be given to the most desirable alternative, and then the other alternatives were to be rated against that one on the basis of operations and safety. (Totally unacceptable designs were rated 0.) Costs were to be considered later and were not to be a factor here. For the configurations shown in Figure 1, the participants were to assume a single-lane turning roadway on four-lane freeways with a DHV for each turning movement of 1,000 vph. The participants were also to rate alternative lengths for an acceleration lane of a major interchange. Turning roadway design speed was 40 mph, through roadway was 70 mph. (Blue Book value is 1,000 ft.) DHV was 1,000 vph. A tabulation of the ratings is given in Table 1.

When the results were tabulated, the participants were categorized into three groups: design engineers, traffic operations specialists, and academic and research. This allowed us to note any differences of opinion among these three areas of expertise.

It is interesting to note that all three groups selected configuration E as the best and considered the left exit designs the least desirable. The traffic operations specialists gave slightly lower ratings to the loop ramp configuration (C) than did the design engineers. Although the sample is small, the results tend to indicate that those who work with the "product" on a day-to-day basis feel that even more effort (and money) should be expended to eliminate second-choice design features.

In general, the academic and research group was not so critical of the left exit designs as the other groups. A possible interpretation is that the academic and research group bases its opinions primarily on conceptual principles and that, in fact, actual operations and safety at left exit ramps are even poorer than might be anticipated.

The results of the ratings of the alternative lengths of acceleration lanes are given in Table 2. Again, it can be noted that the three groups are essentially in agreement, and the design engineers are slightly less critical of substandard design.

It is also interesting to note that the Blue Book has a median rating of 9, which indicates that the participants believe this value to be adequate. A slightly higher value is reported for 1,200 ft, but then it tends to drop off again as the length is extended further. From comments, it would seem this dropping off is due to concern for the excessively long merging area that might result or the possibility that drivers might temporarily believe the lane was not going to be dropped.

The use of group medians in Tables 1 and 2 masks the rather wide range of individual ratings, as the "outliers" are lost in this process. As examples, the ratings for configuration A in Figure 1 ranged from 0 to 7, configuration D from 3 to 10, and configuration E from 7 to 10. These large discrepancies may indicate an interpretation problem on the part of some of the respondents or differences in past experiences with the various designers. Hence, the use of the Delphi method (1) or some similar technique for arriving at consensus opinion is suggested for future studies of this type.

Further Introductory Remarks

Before beginning the open discussion, it was further pointed out that, if these merit ratings can be set for alternative configuration choices and for design dimensions, the possibility for specifying different levels of merit for entire interchanges exists. For example, for a major interchange, the designer could specify that all configurations and design dimensions have merit ratings of 9 or better, whereas for a less important interchange configuration and dimensions with ratings of 7 might be acceptable.

Hence, these merit ratings could be used to select individual design features through comparison of relative merits and relative costs or as a means to ensure design features consistent with the importance of the interchange and, if desirable, consistent within a given interchange.

This last statement leads to another question: Is it ever desirable to purposely degrade a design feature so that the level of design will appear to be consistent to the driver? In other words, is it better if the driver encounters marginal quality throughout the interchange than if he observes high quality in all places in the interchange except at one critical site? Will he be deceived into thinking he is on a better grade facility than he is?

Figure 1. Exit-ramp configurations.

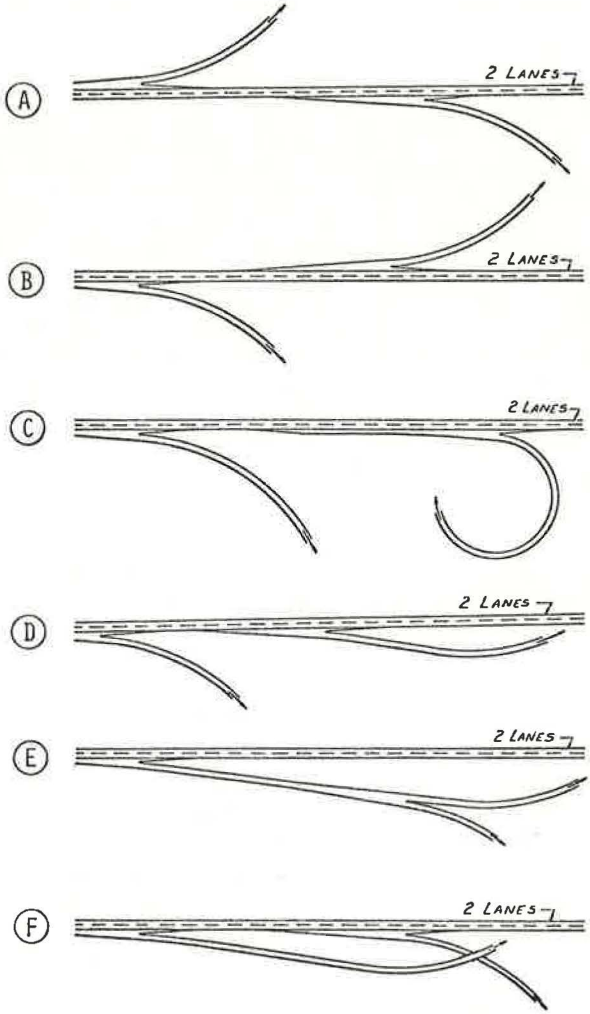

Table 1. Median ratings for exit-ramp configurations.

Figure	Response Group: Design Engineers	Traffic Operations Specialists	Academic and Research	All Groups
A	1	1	3	1
B	2	2	3	2
C	6	4	4	5
D	8	8	6	8
E	10	10	10	10
F	6	6	6	6
Number responding	18	6	7	31

Table 2. Median ratings for acceleration lane lengths.

Length (ft)	Response Group: Design Engineers	Traffic Operations Specialists	Academic and Research	All Groups
1,400	10	9	9	10
1,200	10	10	10	10
1,000	9	9	8	9
800	7	6	4	6
600	3	1	0	1
400	0	0	0	0
200	0	0	0	0
0	0	0	0	0
Number responding	18	6	7	31

Questionnaire Results

In addition to the illustrative rating questionnaire handed out during the introductory remarks, a session questionnaire was given to the participants at the end of the discussion, and they were asked to complete it and return it the following day. As in the case of the questionnaire on trade-off analyses, the questions generally paralleled those used to structure the discussion. The questions, with tabulations of the answers, are given in the Appendix.

The answers indicate that somewhat more than half of the participants believe it is possible to derive meaningful merit ratings. The design engineer group was about evenly split, whereas the other two groups were considerably more optimistic.

Assuming that merit ratings should be developed, almost everyone felt that all possible inputs should be used in developing these ratings. A number of the participants indicated that they were "not comfortable" making the ratings, but they provided little information on what would have been helpful. (Signing and lighting conditions were mentioned as other possible information inputs.)

The participants generally felt the level-of-merit design worthy of more investigation and trial but were not optimistic about obtaining a practical design tool.

No clear-cut conclusion can be drawn from the answers to the last question. This is perhaps due to the wording of the question; the comments accompanying the answers indicated that the participants were interpreting this question in a variety of ways.

CONCLUSIONS

Major conclusions in the areas of trade-off analyses and the level-of-design concept as applied to major interchange design and operation and traffic control, based on the literature survey, interviews with individual state highway departments, and the workshop discussions and questionnaires, are as follows:

1. The major interchange design process is very "soft," i.e., it is not possible to formulate a definitive flow chart because a number of considerations impinge on one another and all the data must be in before hard decisions can be made.
2. This state of flux in the design process means that trade-off analyses cannot be pulled out as individual decision-making processes but are interwoven with the total design process. This implies that specific definitive procedures will not be used by the design engineers (at least under the present design methodologies) and that aids or guidelines are more reasonable than rules or computation forms.
3. There is relatively little interest in and feeling of necessity for economic analyses in the process of selecting design details (e.g., the length of acceleration lanes, alternative design speeds for turning roadways, etc.). The author attributes this to the feeling among practicing engineers that there are so many factors to be considered in these decisions, in addition to the constraints they have already built in by selecting an overall general configuration, that economic analysis is just not practical or reasonable. However, it should be noted that at least one state indicated a strong interest in assistance in making these types of decisions.
4. Engineering judgment is the most used decision-making procedure in making trade-off analyses between desirable features and costs. There is an awareness that operational and safety characteristics of alternative configurations differ appreciably and that some cost is justified in providing the better features. The problem of how much should be expended is not solved, however. It appears that very few states follow a design policy of simply meeting certain minimums as published in design manuals.
5. Interest but not enthusiasm was expressed in the level-of-merit design concept; the majority of those questioned indicated that they felt the concept deserving of more investigation, but there were reservations regarding the practicality of the end results. At this time it is not clear whether they feel that it is not even practical to develop guidelines and aids for assessing the relative operational and safety merits of alternative interchange configurations or that it is simply not practical to set up computation procedures to select the most cost-effective alternative.

ACKNOWLEDGMENT

This project was sponsored by the Federal Highway Administration, U.S. Department of Transportation, with the Pennsylvania State University as the contracting agency. The Institute for Research, State College, Pennsylvania, and Gannett, Fleming, Corddry and Carpenter, Inc., Camp Hill, Pennsylvania, assisted in the study entitled Major Interchange Design, Operation, and Traffic Control.

REFERENCE

1. Dalkey and Helmer. An Experimental Application of the Delphi Method to the Use of Experts. Management Science, Vol. 9, 1963.

APPENDIX

Given are the results from two questionnaires, the first on trade-off analysis and the second on level-of-merit design concept. The numbers on the right are the number of participants who selected a given answer.

TRADE-OFF ANALYSIS QUESTIONNAIRE

1. Economic analyses (cost-benefit ratios, rate-of-return methods, etc.) as applied to major interchange design. Please circle the statement you feel most appropriate:
   a. Economic analyses in comparing alternative interchange configurations as a whole
      i. Common practice 5
      ii. Desirable and feasible but not usually carried out 5
      iii. Desirable but not feasible; appropriate methodology not available 3
      iv. Of little practical value; other considerations are determining factors 11
      v. Other 6
   b. Economic analyses in selection of alternative components (loop ramp versus direct connection, collector-distributor roadway versus double exit, etc.)
      i. Common practice 5
      ii. Desirable and feasible but not usually carried out 6
      iii. Desirable but not feasible; appropriate methodology not available 2
      iv. Of little practical value; other considerations are determining factors 12
      v. Other 5
   c. Economic analyses in specification of design dimensions (length of acceleration lane, radius of curvature of loop ramp, etc.)
      i. Common practice 2
      ii. Desirable and feasible but not usually carried out 5
      iii. Desirable but not feasible; appropriate methodology not available 4
      iv. Of little practical value; other considerations are determining factors 17
      v. Other 2
2. How do you reach decisions on "desirable features," such as exclusion of left-hand exits, good visibility of the exit area, uniformity of exiting maneuvers, etc.? (Circle one.)
   a. Decision to meet AASHO Blue Book minimums at all costs 7
   b. Decision not to incorporate (or exclude) certain features at all costs 2
   c. Attempt benefit-cost (or similar) analysis for individual situations 5
   d. Engineering judgment, i.e., no formal analysis of cost factors as such 15
   e. Other 1

3. Can meaningful cost data be obtained for individual components (ramp configurations, length of deceleration lane, etc.)?
   a. Yes; Comment 19
   b. No; Comment 9
4. How do you assess "benefits" to justify extra expenditures for improving on "minimum" design standards? (Circle any appropriate answers.)
   a. Accident record analyses of similar situations 15
   b. Experience in observing similar situations and relating this to extra costs involved 19
   c. Study of research results in these areas 12
   d. Consensus of personnel in your design department 12
   e. Usually use minimum values 0
   f. Other 7

LEVEL-OF-MERIT DESIGN CONCEPT QUESTIONNAIRE

1. Do you feel it is possible to derive meaningful ratings for alternative general configurations (as in the example of the various exit ramp configurations)?
   a. Yes; Comment 18
   b. No; Comment 12
2. Do you feel it is possible to derive meaningful ratings for alternative design dimensions (as in the example of the acceleration lane lengths)?
   a. Yes; Comment 19
   b. No; Comment 11
3. How should the merit ratings be developed, utilizing which inputs? (Circle all you feel apply.)
   a. Physical analyses (acceleration potentials, friction factors, reaction times, etc.) 20
   b. Accident data across alternatives 20
   c. Research studies on driver behavior and preferences 21
   d. Judgment of highway designers and operations specialists 17
   e. Others 8
4. Were you "comfortable" making the ratings requested in the earlier examples?
   a. Yes; Comment 17
   b. No. If no, what additional information would have been helpful? 12
5. Do you feel the concept of using level-of-merit ratings in interchange design is
   a. Feasible? Yes; No; Comment Yes - 14 No - 6
   b. Practical? Yes; No; Comment Yes - 5 No - 11
   c. Deserving of more investigation, better definition, more trial, etc.? Yes; No; Comment Yes - 17 No - 4
6. Is consistency in interchange "quality" important? Should some elements purposely be degraded to make them compatible with the lower standard design-controlling elements?
   a. Yes, usually. Comment 5
   b. Yes, sometimes. Comment 9
   c. No. Comment 13

LATERAL VEHICLE PLACEMENT AND STEERING WHEEL REVERSALS ON A SIMULATED BRIDGE OF VARIABLE WIDTH

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Many states are engaged in large-scale programs of highway construction and improvement, which include construction of new bridges as well as widening of older ones. At the present time there are no proven guidelines on the optimum shoulder width for bridges. The research reported here utilized a Greenshields Drivometer and an 8-mm time-lapse movie camera to record steering reversals and lateral placement in the vicinity of a simulated bridge. Eight male and two female subjects were tested for eight shoulder width conditions. Each subject drove the instrumented test vehicle across the simulated 50-ft bridge for a total of 30 runs for each of the eight test conditions. Statistical and graphical analyses of the data showed considerable variation among the individual subjects. However, certain trends were shown for all subjects. Steering reversals, both minor and major, were relatively constant for shoulder widths greater than 4 ft. The distance of vehicle from centerline of roadway also reached a maximum for a 4- to 6-ft shoulder width. The subjects tended to drive closer to the centerline for shoulder widths less than or greater than approximately 4 to 6 ft. These results indicate the need for a minimum shoulder width of 4 to 6 ft if traffic operations are not to be influenced.

•MANY STATES are engaged in a large-scale program of new highway construction and also improving older highways. These programs include construction of new bridges and widening of older ones. There are no available proven guidelines on the optimum shoulder width for these structures. In the past, the design criteria of the various states have specified different widths for long- and short-span bridges. Because of the cost factor, long-span bridges were usually provided with narrower widths than were short-span bridges. AASHO published the following statement (9): "... the clear width on bridges should be as great as feasible, preferably as wide as the approach pavement and shoulders, in order to give drivers a sense of openness. On the other hand, bridges that are long are costly and on them some compromises from the desirable usually is necessary." In addition, Figure IX-8 on page 519 of that publication illustrates full shoulder widths being provided on short structures and no shoulder being provided on long structures.

Another AASHO report (10) recommends that "A full shoulder width should be carried across all structures." This recommendation is therefore a departure from previous practice. This significant decision, if implemented on all structures, would vastly increase the cost, particularly where longer structures are involved.

One of the latest AASHO publications concerning shoulder widths on bridges (13) has been adopted by all states. This report allows less than full shoulder width for low-speed (less than 50 mph) and low-volume (less than 750 ADT) roads. It also allows existing bridges on low-speed, low-volume highways to remain in place without shoulders if the clear roadway width meets certain minimum standards. Other recent publications (11, 12) recommend a constant width of shoulder and roadbed.

If all new bridges are constructed with a full-width shoulder and older structures widened to include a full shoulder, and if this full shoulder is not required from a safety or traffic operations standpoint, an unnecessary financial burden will have been placed on the funding agency. At the present time, little factual information is available concerning any operational benefits to be derived from a full shoulder width.

PREVIOUS RESEARCH

Although a number of studies have been carried out in the general area of roadway shoulders, there is no record of a controlled laboratory study such as this one. However, for comparison purposes the results of some of the reported field studies are presented here.

In 1947, a committee under the sponsorship of the Department of Traffic and Operations of the Highway Research Board was organized to evaluate traffic operations benefits as related to shoulder width (1). The committee reviewed past research projects and reported on a before-and-after study carried out in West Virginia during the period 1947 to 1949. The study revealed that the speed of a moving vehicle is not substantially affected by the width of the shoulder, providing the shoulder is more than 4 ft wide. The study also showed that the lateral position of a free-moving vehicle shows no significant relation to shoulder widths greater than 4 ft.

The first comprehensive analysis of accidents and their relationship to various roadway elements was reported by Raff (2). The study, involving only gravel shoulders, indicated that the most significant factors affecting accident rates are traffic volume, degree of curvature, percentage of cross traffic at intersections, and width of bridge roadways both absolutely and in relation to their approach pavement width. Any extra width in relation to the approach pavement definitely reduces the accident hazard on bridges. The actual width of the bridge pavement also contributed to the safety of the bridge.

Billion and Stohner (3) studied earth (grass) and gravel (macadam) shoulders. Their study was confined to accidents reported in New York State between October 1947 and July 1955. Only fatal and serious injury accidents and those accidents occurring on highways that used state-owned maintenance equipment were included in the study. The road sections studied were located on two-lane rural highways. The study indicated that medium-width shoulders had lower accident indexes than narrow shoulders under all conditions of horizontal and vertical alignment.

Head (4) studied gravel shoulders on various sections of rural highways. Considering curvature, terrain, sight distance, access and shoulder width, and other variables, he computed the relationship among total accidents, property damage, personal injury accidents, and the various roadway elements. Statistically, he concluded that total accidents and property damage accidents decreased as shoulder width increased in the 3,000 to 5,500 ADT range. No statistical relationship was found between accidents and shoulder width for those sections with an ADT of 5,000 to 7,000 vehicles per day nor between shoulder width and personal injury accidents.

Belmont (5) conducted a study of paved shoulders based on personal injury accidents reported in 1948 for two-lane rural highways on the California Interstate Highway System. The sample was limited to rural roads with a speed limit of 55 mph. Regression equations were computed by using the square root of the number of accidents as a dependent variable. The analysis was based on three groups of shoulder widths: less than 6 ft, 6 ft, and greater than 6 ft. The results showed that 6-ft shoulders were safer than narrower shoulders. They were also safer than wider shoulders for those sections with a traffic volume greater than 5,000 vpd.

In another study that used California accident data for the years 1951 and 1952, Belmont (6) confined his work to an analysis of personal injury accidents. For ungrouped accident data, regression equations were computed by using the square root of the number of accidents as the dependent variable, and, for grouped accident data, the number of accidents was used as the dependent variable. The results indicated a tendency for injury accidents to increase with increased shoulder width except for sections with traffic volumes less than 2,000 vpd for which no relationship was established.

Taragin (7) undertook a study on lateral placement of vehicles as related to shoulder type and width on two-lane highways. He reported that a relationship between vehicle speed and lateral positioning did exist on sections where the shoulders were paved to their full width and that the average positioning of slow-moving vehicles, regardless of type, was closer to the shoulder of the highway than that of fast-moving vehicles.

Jorol (8) observed lateral placement on bituminous-paved two-lane and four-lane rural highways having different shoulder designs in the state of Idaho. He recorded placement data for 7,777 free-moving passenger and commercial vehicles at eight locations during the period 1957 through 1959. The purpose of the study was to evaluate the influence of shoulder design on vehicle placement. Before-and-after data were recorded to measure the effect from other factors. Lateral placement was recorded from visual observations of the vehicle position relative to markings placed on the pavement at 1-ft intervals.

The study showed that the width of the shoulder influenced the lateral placement of vehicles. Both passenger and commercial vehicles traveled closer to the roadway centerline on sections with narrow shoulders than on sections with wide shoulders. In addition, more shoulder encroachment was observed for commercial than for passenger vehicles, and more encroachment was found on the sections with wide shoulders. The narrower the road was, the greater was the tendency for drivers of passenger vehicles to travel in the same wheel tracks.

In summary, the reported studies appear to give some contradictory results when accident rate and shoulder width are compared. For example, gravel shoulders showed a decreasing accident experience with an increase in shoulder width, whereas paved shoulders had an increasing accident experience with an increase in shoulder width. However, the majority of the studies indicated a shoulder width of 4 to 6 ft to be the safest width studied. With regard to lateral placement, the studies generally concluded that narrow shoulders encouraged drivers to drive closer to the pavement centerline.

METHOD

Simulated Bridge

The study utilized a simulated bridge, erected in a large parking lot. The guardrails of the bridge were represented by two 4- by 50-ft lengths of green canvas. Steel pipes, set in concrete bases, held the canvas in place. A broken centerline and solid edge lines were placed on the pavement to indicate two 12-ft traffic lanes. These pavement markings, of 6-in. white reflective tape, extended for 50 ft on both sides of the bridge. The bridge width was randomly varied during the study for a total of eight test conditions as given in Table 1.

Subjects

A total of 10 subjects, eight male and two female, participated in the study. The subjects, all volunteers, were students in an engineering class at West Virginia University. Ages ranged from 20 to 23 years. Nine of the subjects had at least 3 years' driving experience in various states. All subjects had a valid driver's license, and each subject was asked to wear corrective lenses if he or she normally did so while driving.

Data Recording

All subjects drove the same instrumented vehicle throughout the experiment. The vehicle, a 1969 four-door Ford sedan, was equipped with power steering, power brakes, and air conditioning. Driver and vehicle performance data were recorded by a Greenshields Drivometer. The following items were monitored continuously during each test run, and cumulative totals were printed out on paper tape at the command of the experimenter:

1. Macro steering wheel reversals ($8\frac{1}{2}$ deg),
2. Micro steering wheel reversals ($2\frac{1}{2}$ deg),
3. Speed change (2-mph intervals),
4. Accelerator pedal movement ($\frac{1}{8}$ in. up or down from any position),

5. Brake pedal applications,
6. Distance traveled (to $^1/_{100}$ mile),
7. Running time in seconds, and
8. Trip time in seconds.

The speed of the test vehicle, as it approached the bridge, was also recorded by a radar speed meter. The lateral placement of the vehicle along the entire length of the bridge was recorded by a super 8-mm time-lapse movie camera. All filming was done in color at a speed of 6 frames/sec. The camera was equipped with a remote control that allowed the operator to remove himself from the vicinity of the bridge.

Procedure

A single bridge test condition, previously chosen at random, was tested each day. Before a subject was brought to the test site, the time-lapse camera was set up and used to film "calibration tapes" placed at each end and in the center of the bridge. The tapes, made up of alternating black-and-white segments 1 ft long each, were placed perpendicular to the roadway. After several frames of film were exposed, the tapes were removed. The calibration film was later used to define a roadway grid system for the data analysis. The radar speed meter was also set up and tested at this time.

So that the true purpose of the experiment would be concealed, each subject was told that he or she was helping to calibrate a new piece of equipment, the Drivometer. It was felt necessary to take this precaution in order to avoid biasing the data. The subjects were instructed to drive in a normal and comfortable manner over a closed course that included the simulated test bridge. The subject was further instructed not to exceed a speed of 30 mph.

Actual data recording started only after the subject indicated that he had become thoroughly accustomed to both the test vehicle and the course. As the car approached the bridge, the time-lapse camera was remotely switched on and the radar speed meter reading recorded. The camera was switched off after the test car left the bridge. Drivometer readings were printed out as the test car entered and left the bridge. Thirty runs were recorded for each of the 10 subjects, for a given test condition, on a single day.

Data Reduction

After completion of the testing each day, the film was mailed to a commercial photographic laboratory for processing and the Drivometer data were keypunched into computer cards. As each roll of film was returned, it was immediately projected to verify that there had been no equipment failure during the filming. Actual film analysis began after all rolls had been returned.

The film was projected by a stop-motion projector on a 3- by 3-ft white screen from a distance of approximately 15 ft. The film was advanced at normal speed until the black-and-white calibration strips appeared on the screen and then was brought to a halt. The 1-ft interval strips in the picture were then marked on the screen. These marks were next joined by straight lines drawn parallel to the bridge abutment, dividing the bridge into 1-ft parallel strips 50 ft long. The film roll was then advanced until the calibration strips no longer appeared in the picture. While the film roll was advancing, care was taken to see that the position of the calibration strips remained in line with the grid markings drawn on the screen. After this initial preparation, recording of the lateral placement data started. The film was advanced frame by frame, and the position of the centers of the test car's right wheels was recorded for each frame as the vehicle crossed the bridge. This procedure was repeated for each roll of film.

RESULTS

Initial inspection of Drivometer data showed very few accelerator pedal movements or speed changes in the vicinity of the bridge. This was not surprising inasmuch as the drivers were instructed to maintain a steady, safe, comfortable speed during the test and not to exceed 30 mph. Therefore, the analysis was confined to steering wheel reversals for the Drivometer and lateral placement as recorded by the time-lapse camera.

Steering Reversals

The steering reversal data for minor and major movements while the subject was approaching the simulated bridge are given in Table 2. The values are averages for 30 runs per subject. Due to a malfunction in the Drivometer, data for the 4-ft shoulder width were not included in the analysis.

The data in Table 2 were subjected to an analysis of variance using a standard ANOVA program. The results of the ANOVA are given in Table 3. The ANOVA shows a statistically significant difference for both subjects S and the various shoulder widths W. However, due to the significant interaction between the subjects and various widths, S x W, a test for individual means was not run. Instead the combined data for all subjects were plotted (Fig. 1). A general trend for both minor and major reversals is evident; more steering reversals were recorded for narrow shoulders than for wide shoulders. However, the number of reversals remains relatively constant for widths greater than 4 to 6 ft. Although not included in this report, similar graphs for each individual subject exhibit this same trend.

Lateral Placement

Lateral placement data were recorded for both directions of travel on the simulated bridge. However, due to positioning of the time-lapse camera, results for only one direction of travel were considered to be reliable and are presented here. Lateral placement data for eight of the 10 subjects are given in Table 4. Two subjects were not included in the analysis due to incomplete data. The values shown in Table 4 represent the distance from the center of the left wheel to the pavement centerline. As with the steering reversal data, the lateral placement data were subjected to an analysis of variance, and the results are given in Table 5.

Statistically significant differences are shown among subjects and for the various shoulder widths. A test for individual means is again inappropriate due to the significant interaction between S and W. The data for the eight subjects have been averaged and plotted as shown in Figures 2 and 3. Once again a general trend may be noted in that a shoulder of 4 to 6 ft in width appears to be optimum.

Summary of Results

The results of this study may be briefly summarized as follows:

1. A greater number of minor steering wheel reversals were recorded for narrow shoulders than for wide shoulders.
2. A greater number of major steering wheel reversals were recorded for narrow shoulders than for wide shoulders.
3. For both minor and major reversals, the number of reversals remained relatively constant for shoulder widths greater than 4 to 6 ft.
4. The subjects drove furthest from the marked centerline for a shoulder width of 4 to 6 ft.

DISCUSSION OF RESULTS

This report has concerned itself with steering reversals and lateral placement of vehicles driven by 10 subjects across a simulated bridge with variable shoulder width. Although the 10 subjects exhibited individual driving characteristics, certain general trends were common to all. The combined data for all subjects have been presented in the form of tables and graphs. Inspection of these tables and graphs shows that the number of steering reversals, both minor and major, decreases rapidly as the shoulder width increases from -4 to +6 ft and then increases slightly as the shoulder width increases from +6 to +12 ft. Similarly, the distance of left wheel from pavement centerline increases as shoulder width increases until it reaches an optimum at approximately

Table 1. Bridge test conditions.

Bridge Lane Width (ft)	Shoulder Width (ft)	Total Width (ft)
12	12	48
12	10	44
12	8	40
12	6	36
12	4	32
12	2	28
12	0	24
10[a]	—	20
8[a]	—	16

[a]Width of bridge lanes was less than combined 24-ft width of the two traffic lanes on either approach to bridge. In effect, the bridge acted as a bottleneck in the roadway.

Table 2. Average number of steering reversals.

	Shoulder Width (ft)							
Subject	12	10	8	6	2	0	-2	-4
Minor Reversals								
1	3.26	3.47	2.20	3.90	3.30	3.60	4.00	1.60
2	1.50	1.13	0.60	0.93	1.23	1.43	1.63	1.20
3	1.63	1.90	1.90	2.00	2.23	2.30	3.90	3.83
4	1.37	1.53	0.67	1.30	2.10	1.21	2.53	1.60
5	1.83	2.50	1.43	2.18	2.20	1.99	3.37	3.93
6	2.70	1.87	2.17	2.00	2.37	2.13	2.93	1.95
7	2.43	2.70	1.90	2.23	1.97	2.50	4.27	6.47
8	1.50	2.17	0.90	1.93	2.33	1.61	—	2.97
9	1.57	1.83	1.27	1.93	1.87	1.07	1.27	2.93
10	—	2.40	1.45	2.83	2.93	—	3.63	1.83
Mean	1.97	2.15	1.45	2.02	2.25	1.98	3.10	2.83
Major Reversals								
1	0.57	0.74	0.33	0.33	0.44	0.38	1.20	0.30
2	0.60	0.57	0.40	0.13	0.44	0.59	1.17	0.26
3	0.57	0.57	0.54	0.47	0.40	0.84	1.90	2.10
4	0.25	0.37	0.25	0.40	0.30	0.38	1.13	0.50
5	0.54	0.40	0.50	0.70	0.30	0.40	1.20	1.44
6	0.60	0.74	0.57	0.64	0.50	0.70	1.10	0.54
7	0.44	0.44	0.27	0.40	0.30	0.73	1.10	2.30
8	0.23	0.30	0.17	0.60	0.63	0.17	—	0.63
9	0.60	0.63	0.53	0.70	0.47	0.83	1.00	0.87
10	—	0.40	0.40	0.70	—	0.73	0.87	0.37
Mean	0.49	0.52	0.40	0.51	0.42	0.58	1.20	0.90

Table 3. Analysis of variance for steering wheel reversals.

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F-Ratio[a]
Minor Reversals				
Subject S	7	38.61	5.52	11.19
Shoulder width W	8	178.00	22.25	45.13
S × W	56	137.86	2.46	4.99
Experimental error	2,088	1,029.27	0.493	
Major Reversals				
Subject S	7	836.78	119.54	65.82
Shoulder width W	8	702.89	87.86	48.38
S × W	56	866.43	15.47	8.56
Experimental error	2,088	3,792.47	1.816	

[a]$p < 0.05$.

Figure 1. Steering wheel reversals for all subjects.

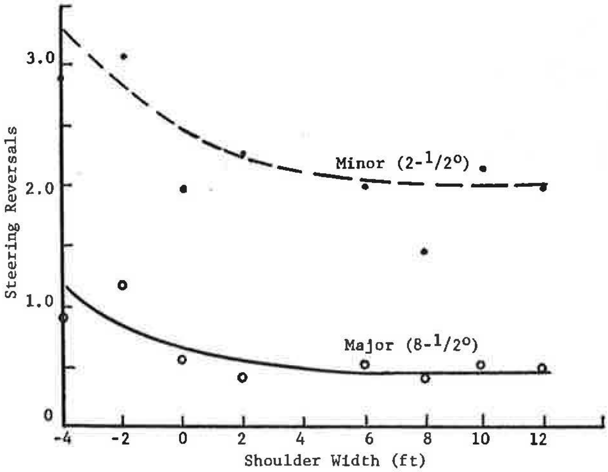


Table 4. Average lateral placement in ft.

	Shoulder Width (ft)								
Subject	12	10	8	6	4	2	0	-2	-4
1	1.87	2.60	2.27	3.47	3.00	2.13	3.00	1.80	-1.13
2	1.73	2.47	2.80	2.47	2.87	3.00	3.07	2.00	-0.80
3	3.00	2.20	2.80	3.13	2.93	2.93	3.80	3.00	0.00
4	1.33	2.07	2.40	1.87	2.33	2.47	2.33	2.00	-2.26
5	1.93	2.40	2.20	3.40	2.07	2.47	2.53	2.07	0.067
6	1.93	2.87	2.20	2.87	3.67	3.00	3.20	1.93	-0.933
7	2.73	2.06	2.73	3.06	2.67	2.73	2.93	2.13	0.067
8	2.00	3.07	2.33	3.73	2.93	3.13	3.67	3.00	-0.20
Mean	2.07	2.47	2.12	3.00	2.81	2.73	3.07	2.24	-0.65

Table 5. Analysis of variance for lateral placement.

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F-Ratio[a]
Subject S	7	101.97	14.57	62.53
Shoulder width W	8	1,237.45	154.68	663.86
S × W	56	144.18	2.57	11.03
Experimental error	1,008	235.87	0.233	

[a] $\underline{P} < 0.05$.

Figure 2. Percentage of vehicles with various lateral placements.

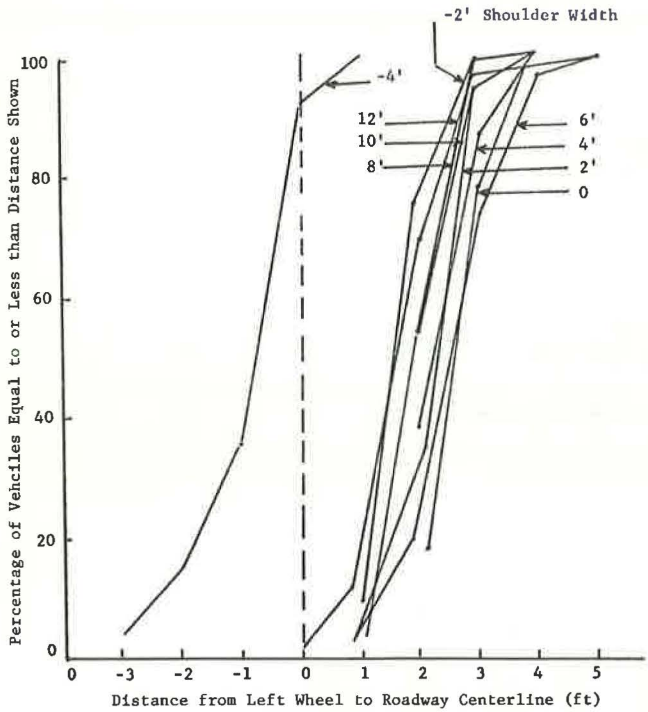
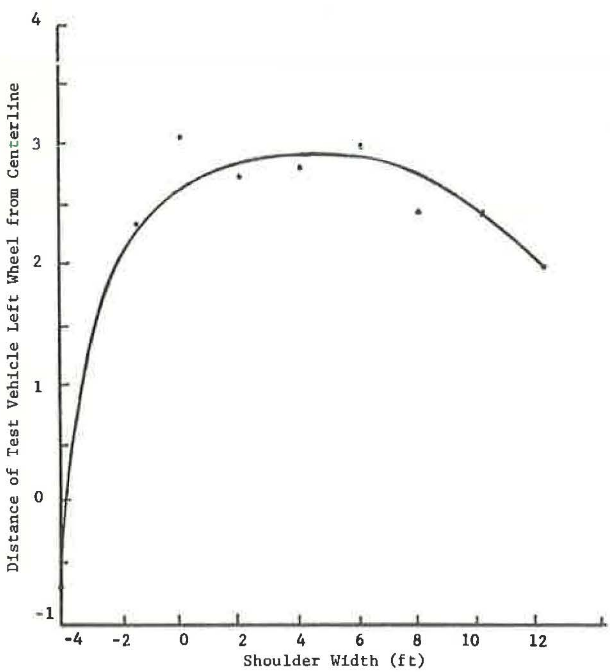


Figure 3. Lateral placement in relation to shoulder width.



+4 to +6 ft and then begins to decrease again. Based on these findings, it would appear that a minimum shoulder width of 4 to 6 ft would be required in order not to influence traffic operations. However, it should be noted that the results for this simulated bridge study have not as yet been verified by an actual field test.

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AN ANALYSIS OF THE SHIRLEY HIGHWAY EXPRESS-BUS-ON-FREEWAY DEMONSTRATION PROJECT

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ABRIDGMENT

•FOR MANY YEARS one of the most critical problems in urban areas has been the supply of transportation facilities. As the automobile has become the predominant mode of transportation, planners and traffic engineers have attacked this problem by increasing the number of lanes of streets and highways in urban areas. Now it is realized that a single mode of transportation cannot fulfill the demands, and a balanced system is required. One concept for alleviating the transportation problem is the use of freeway lanes for express bus service. Several studies have revealed that this concept is feasible; however, governing bodies have been reluctant to implement it. An exception is the 9-mile exclusive bus lane on Shirley Highway, Interstate 95, in northern Virginia just outside Washington, D. C.

The Shirley Highway Express-Bus-on-Freeway Demonstration Project was sponsored by the U.S. Department of Transportation, the Metropolitan Washington Council of Governments, and the Virginia Department of Highways. The principal objective was to test the hypothesis that the provision of rapid and improved bus service over an exclusive bus lane would attract a significant number of passengers formerly commuting by automobiles. The diversions from automobiles to buses would increase the "people utilization" of the facility and should enable all commuters to travel more quickly and conveniently.

The project consisted of three elements: the busway, including the exclusive lane on Shirley Highway and the bus priority lanes in the Nation's Capital; bus transit operation, involving new buses and services; and residential fringe parking where existing shopping centers and new lots provide free parking for bus riders.

The exclusive lane was opened in three sections. The first segment, approximately 5 miles long, utilized the completed reversible lanes on Shirley Highway and was designated for exclusive bus service in September 1969. The second section, opened in September 1970, was a temporary facility that connected the permanent lanes and extended northward 1.6 miles. In April 1971, the temporary lane was extended to the newly constructed center-span bridge across the Potomac River between the two older 14th Street bridges.

The feasibility study report estimated that 90 additional buses would be required during the peak periods to handle the ridership by the time the demonstration project was concluded. This expansion in service would result in a major increase in the bus company's capital requirements, and, in September 1970, the Urban Mass Transportation Administration approved a grant to the Northern Virginia Transportation Commission that provided the funding for the required buses. These buses were purchased and placed into service as the demand increased.

The availability of free fringe parking was one of the keys to market expansion. Park-and-ride and kiss-and-ride facilities were provided near the residential areas of the Shirley Highway corridor. These included parking facilities in existing shopping centers as well as new parking lots that will later serve the rail rapid system.

STUDY OBJECTIVES

This investigation was conducted to determine the effectiveness of the express bus service in relieving the congestion on Shirley Highway, thus improving the level of service for the automobile commuter using the conventional lanes of the facility. The major objectives of this study were

1. To determine the effects of the busway on bus patronage and automobile travel trends,
2. To analyze sources of bus patrons and determine the reduction in automobiles, and
3. To use the automobile reduction to compare the level of service of the existing facility with that of the freeway without the exclusive busway.

The bus system provided similar services to the commuters during both daily peak periods; however, the scope of this study was limited to the inbound flow during the morning peak period to eliminate duplication of data collection and analysis.

STUDY LOCATION

Because the effectiveness of the busway varied over its entire length, a site that represented typical operations was chosen. This site is in the segment opening in September 1970. The geometrics of the inbound facility at this location consist of two 12-ft lanes with a 10-ft shoulder on the right and a wooden barrier guardrail, which separates the conventional lanes and the busway, located within 6 in. of the left edge of the pavement. It is a half-mile segment in level terrain, and the restricted average highway speed is approximately 50 mph.

ANALYSIS OF TRAVEL DATA

To achieve the objectives of the study, we identified and evaluated parameters such as travel times, passenger trends, commuter profiles, traffic volumes, automobile occupancy, and reductions in automobiles on Shirley Highway.

Bus travel times were based on actual scheduled runs, whereas automobile travel times were recorded by a license-plate survey. The results indicated that many of the buses saved 20 to 25 min over the automobile for the same trip.

To properly evaluate the passenger trends on Shirley Highway required that adjacent facilities in the Shirley Highway corridor be reviewed. The annual growth in travel in the 6 years prior to the initiation of the demonstration project was 4 percent, with only ½ percent being bus patronage. Since that time, travel in the corridor has decreased and for the past 2 years has experienced a decline of 1 percent each year.

The increase in bus patronage was determined from the travel counts made by the transit company. The counts included only the local buses and not special buses, many of which operated on a chartered or nonscheduled basis. At the initiation of the project, approximately 3,800 bus passengers traveled through the study site each day, but, at the end of the 31-month study period, patronage had increased to 9,200 persons or 142 percent. That is, an additional 5,400 passengers (gross increase) started riding the express bus during the study period. At the beginning of the project, approximately 25 percent of the commuters on Shirley Highway traveled by bus, whereas 54 percent of all commuters rode the bus during the peak periods in April 1972. In other words, more people were then commuting by bus than by automobile on Shirley Highway during the rush hours.

The gross increase in bus patronage was subjected to adjustments and assumptions in making the estimate of the number of commuters diverted from automobiles to buses on Shirley Highway. The adjustments included historical passenger trends in the Shirley Highway corridor, diversion from non-Shirley Highway bus routes, and transient population in the Shirley Highway corridor.

Of the adjustments evaluated, the only one that affected the number of riders significantly was the passenger diversion (6 percent) from non-Shirley buses. A 6 percent reduction of the busway's gross passenger growth (5,400) results in a net increase in patronage of 5,076 passengers. The net increase or growth represented the number of persons who formerly commuted on Shirley Highway and were attracted to the express bus system.

The demonstration project would not have been effective if only captive riders had been attracted, inasmuch as the project objective depended on a diversion from the automobile commuter population. An on-board bus survey provided data on the sources of bus patronage. The results indicated that two-thirds of the new bus riders had a choice between the express system and their automobiles, and they freely stated that the contributing factor in their taking the bus was the fast service provided by the exclusive lanes, which avoided the conventional traffic congestion.

The bus system attracted the young working male with a good income who lived and worked within walking distance of the bus route. The majority of the bus users had annual family incomes in excess of $15,000, whereas only 1 percent had incomes of less than $5,000. Only 6 percent reported that they did not own an automobile, and the average number of automobiles per household was 1.32. It was concluded that the bus users were choice riders and were diverted from automobiles.

In the method of computation used in the study, the number of automobiles removed from Shirley Highway depended on automobile occupancy rate. As one would expect, the number of automobiles using Shirley Highway decreased as bus patronage increased. In September 1970, automobile trips reached a peak of 9,300 vehicles; in March 1972, the trips had decreased to approximately 5,600. With the number of passengers and vehicles known, the automobile occupancy rate at the study site was calculated, and there was a variation during the early months of the project before the rate leveled off at approximately 1.4 passengers per vehicle. The average automobile occupancy rate for Shirley Highway during the study period was 1.43. A review of the other major facilities in the corridor, as well as other roads leading into Washington, indicated that the 1.43 occupancy rate was within the proper range, and therefore it was used in the analysis.

Dividing the occupancy rate into the increase in bus patronage revealed that 3,550 vehicles per peak period were removed from the conventional traffic lanes. A review of the automobile travel trends substantiated this estimate, and when the decrease in automobile travel along with the annual decrease of travel in the corridor was considered, the analysis revealed that the estimate was realistic.

Classification traffic counts secured during the peak period revealed that the "vehicle-moving" peak hour occurred between 6:30 and 7:30 a.m. and the "people-moving" peak hour between 7:15 and 8:15 a.m. During the peak period, 192 buses used the busway, and 131 trips were during the "people-moving" peak hour. Because the objective of the reserved-lane concept was to increase the "people-moving" capability of the facility, the "people-moving" peak hour was selected for the level-of-service analysis.

LEVEL-OF-SERVICE ANALYSIS

In evaluating the level of service, the Highway Capacity Manual technique (4) was used inasmuch as it is the most effective means available for this purpose. The major factors used in the technique are the operating speed and volume-capacity ratios.

Speed studies taken at the study site during the "people-moving" peak hour revealed an operating speed of approximately 30 mph. Computations revealed that the service, or demand volume, was 2,210 vehicles per hour, whereas the capacity of the conventional lanes was 3,420 vehicles per hour. The resultant volume-capacity ratio was 0.65. Utilizing this ratio, the 30-mph operating speed, and a peak-hour factor of 0.79, the Highway Capacity Manual technique revealed that the subject section of Shirley Highway was operating at level of service E. The characteristics of level of service E include unstable flow, stoppage of momentary duration, operating speeds in the vicinity of 30 mph, and volumes approaching the capacity of the roadway. On a field trip to the study site during the "people-moving" peak hour, all of these characteristics were observed.

To measure the effect of the busway on the level of service for the conventional traffic required that an analysis assuming that the busway was not in operation be made. In this analysis automobiles eliminated from Shirley Highway were placed back into the conventional traffic stream. The geometrics of the roadway and the peak-hour factor were assumed to remain unchanged. It was also assumed that the "people-moving" peak hour occurred at the same time as experienced on the existing facility, although it was surmised that this would change in a real world situation. Travel time on Shirley Highway by automobile is greater than by bus; therefore, theoretically, if the buses were removed, the commuters would be forced to travel by automobiles and thus would have to make their trips at an earlier time when the traffic flow was greater.

There was no way of predicting the operating speed on this hypothetical roadway. However, the anticipated volume-capacity ratio was 1.34. Under these conditions, the demand volumes would far exceed the capacity of the facility, and the facility would break down. The result would be low, if not zero, speeds; extremely high density; and very low volumes. A comparison with the standards established in the Highway Capacity Manual indicates a forced flow in traffic jams and an unacceptable level of service.

CONCLUSIONS

The foregoing analysis concluded that a significant number of commuters were diverted from automobiles onto the buses. This diversion reduced the number of automobiles on Shirley Highway, and consequently the level of service in the conventional lanes was improved. It is even conceivable that the busway has alleviated or at least not worsened the congestion on other facilities in the corridor. If the busway had not been provided, the number of automobiles using Shirley Highway would have increased until intolerable service levels were reached, and then motorists would have sought other routes. Thus, it is reasonable to assume that the volumes on other arterials would have been higher if the busway had not been implemented.

Apparently, the bus riders were receiving many benefits and were pleased with the service inasmuch as the patronage continued to increase even though a good majority of the riders never had a seat available for the trip. When the length of the trip is considered, much comfort must have been sacrificed for the other benefits received.

It must be concluded that the project is effective and successful inasmuch as a large number of automobile commuters have started using the attractive, rapid express bus system, and as a result the level of service for all commuters has improved. More importantly the project demonstrates the potential of the bus system as a method of public mass transportation. It appears to be feasible, practical, workable, and acceptable to the public.

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