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22 Highway Design

27 Bridge Design

53 Traffic Control and Operations

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Foreword

The nine papers in this Record will be of special interest to highway designers and traffic engineers who seek precision in evaluating factors and conditions that influence highway traffic, efficiency and safety.

The first two papers supplement each other as both explain the development and use of a modulus of geometric highway design aspects in terms of the ease with which traffic moves along a section of highway.

The next two papers present enlightening discussions and examples of engineering decisions made by automation. In one paper the author applies an automated theoretical method to a problem of highway interchange design. The other deals with the problem of selecting and evaluating trial grade line. It explains a system of electronic computation that achieves a balance between construction costs and the highway users' operating costs.

Local highway authorities who may be faced by problems of increasing the capacity or safety of existing routes in growing communities should welcome the abridged report, "Optimum Width for Widening Secondary Arterial Streets."

Another paper, written under the sponsorship of the Department of Traffic and Operations, discusses the effects of a barrier railing installed in a narrow median of an expressway in Philadelphia on traffic flow and driver performance.

Two papers in this Record discuss extensive full-scale impact tests of bridge parapets and designed barrier rails. One paper covers design and tests in California, while the other describes the features and results of tests on new General Motors Proving Ground structures at Milford, Michigan. The latter report offers evidence of increased safety through the use of a specially designed shape of concrete parapet.

The last paper, "Development of an Analytical Procedure for the Prediction of Highway Barrier Performance," covers two important phases of a continuing research project on highway barriers—a series of full-scale dynamic tests and the development of a mathematical analysis of the reaction of a vehicle during collision with a barrier.

Contents

Criteria for Balanced Geometric Design of Two-Lane Rural Highways

J. C. OPPENLANDER and R. F. DAWSON
Respectively, Assistant Professor and Instructor, Purdue University, West Lafayette, Indiana

This investigation was concerned with the development of numerical measures for the significant geometric-design elements of two-lane highways located in rural areas. A mathematical model for the modulus of geometric aspects was solved to evaluate the ease with which vehicular traffic traverses a highway section with a particular combination of geometric features. These geometric moduli are predicated on the approach speed of the traffic and on the speed reduction produced by the design elements.

Solutions to a multiple linear regression equation provided a reasonable estimation of the speed parameters. The geometric-design variables evaluated for two-lane highways were degree of curve, gradient, minimum stopping sight distance, and lane width. In addition, percent of out-of-state cars, percent of truck combinations, number of commercial roadside establishments per mile, and total traffic volume constituted the controls necessary for functional geometric design. The statistical model used for the generation of approach speeds and speed reductions was developed from the multivariate analysis of traffic-flow conditions observed on two-lane, rural highways.

Finally, criteria for different levels of balanced geometric design were developed for use by the design engineer. The moduli for geometric characteristics permit the engineer to select various combinations of geometric-design elements that produce the same influence on traffic flow. This technique can also be applied to the evaluation of redesign and to the conduct of various highway planning surveys, such as needs studies and sufficiency ratings.

•THE SAFE, expedient, and economic movement of traffic is probably most influenced by the geometric aspects of the highway. However, little attention has been devoted to the quantitative evaluation of geometric design. The knowledge of highway geometric design can be fully developed only when the effects of its elements on traffic flow are measured and expressed in numerical terms.

The purposes of this research investigation were to develop numerical ratings of the geometric elements that significantly influence the rate of traffic flow and to establish criteria for the balanced design of two-lane highways located in rural areas. The geometric moduli and the design criteria were predicated on two speed parameters. The average speed on the approach to a particular combination of geometric features indicates the level of the design in regard to efficiency. As average speed increases, the highway section carries the traffic in a more expedient manner. In addition, the speed reduction produced by the given geometric arrangement represents the relative safety

Paper sponsored by Committee on Geometric Highway Design and presented at the 43rd Annual Meeting.

of the highway location. Smaller changes in speed are associated with safer highway travel.

These geometric moduli for two-lane, rural highways can be used to:

1. Proportion the geometric-design elements of a highway to achieve a uniform level of traffic-flow conditions;
2. Compare the effects of various combinations of geometric features on the rate of traffic movement;
3. Analyze different geometric designs in regard to their operational characteristics;
4. Evaluate quantitatively the influence of redesign on the behavior of traffic flow;
5. Provide numerical evaluations of tolerable standards for the determination of deficiencies in highway needs studies; and
6. Determine sufficiency ratings for programming highway improvements.

Both practical and theoretical considerations were applied for a scientific approach to the quantitative evaluation of the geometric design of highways. These moduli of geometric aspects provide numerical ratings of various design combinations on an interval scale of measurement. Thus, the engineer can use these moduli for geometric characteristics to assist him in formulating the professional judgment necessary in achieving balanced geometric design.

PROCEDURE

To evaluate the moduli for geometric characteristics, it was necessary to solve a mathematical model for speed conditions that were generated by a statistical model for geometric-design elements and design-control variables. The expressions, "geometric modulus," "modulus of geometric aspects," and "modulus for geometric characteristics," are used interchangeably in this paper to denote the numerical rating of highway geometric design.

Recent theoretical studies of traffic flow have postulated that vehicular movement is produced by a motivating pressure potential. Differences in potential along the highway produce various rates of traffic movement. Therefore, this potential indicates the flow behavior of traffic traveling on a highway section with certain geometric-design characteristics. The formula for the modulus of geometric aspects was presented as:

$$F_O = \ln(4S_O - 2\Delta S) - \ln \Delta S \tag{1}$$

where

F_O = geometric modulus,
S_O = average speed on approach to a geometric element, and
ΔS = reduction in average speed produced by the corresponding geometric element.

The derivation of this expression is presented in the literature (2, 4). The modulus for geometric characteristics is a numerical measure of the ease with which traffic traverses a highway location with certain geometric features.

To solve the preceding equation for real values representing the actual conditions encountered on two-lane highways located in rural areas, a multiple linear regression equation was evaluated for estimating mean approach speeds and speed reductions. The following statistical model was developed from the multivariate analysis of traffic flow on two-lane, rural highways:

$$S = 39.34 + 0.0267X_1 + 0.1396X_2 - 0.8125X_3 - 0.1126X_4 + 0.0007X_5 + 0.6444X_6 - 0.5451X_7 - 0.0082X_8 \tag{2}$$

where

S = mean spot speed, mph;
X_1 = out-of-state passenger cars in traffic stream, percent;
X_2 = truck combinations (tractor with one or more trailers) in traffic stream, percent;
X_3 = degree of curve;
X_4 = gradient, percent;
X_5 = minimum stopping sight distance, ft;
X_6 = lane width, ft;
X_7 = number of commercial roadside establishments, such as restaurants, service stations, motels, and taverns per mile (counted on both sides of the roadway for $^1/_2$ mile in advance of and $^1/_2$ mile beyond the speed site), no. per mile; and
X_8 = total traffic volume, vph.

The coefficient of multiple correlation was 0.788 for this investigation and was significant at the 5 percent level. The precision of this multiple estimate was measured by a standard error of estimate equal to 4.47 mph. This regression model provides a reasonable and efficient evaluation of the functional relationship between mean spot speed and the eight variables that significantly influence the rate of traffic flow on two-lane, rural highways (3).

The multiple linear regression equation was solved to generate the average speed on the approach to a geometric element (S_O) and the average speed on the corresponding design feature (S_1). The difference between these two mean speeds (S_O - S_1) produced the reduction in average speed occasioned by the geometric-design element (ΔS). These two speed parameters (S_O and ΔS) were essential for the solution of the mathematical model representing the modulus for geometric characteristics. This computational technique was programmed in FORTRAN Language for the IBM 7090 computer to obtain the geometric moduli for various combinations of geometric elements and design controls.

Design criteria were developed in this investigation for various levels of highway design. Reasonable limiting values of degree of curve, gradient, minimum stopping sight distance, and lane width were formulated from accepted geometric-design policy for design speeds of 30, 40, 50, 60, and 70 mph (1). The evaluation of the expression for the modulus of geometric aspects permitted the establishment of minimum geometric moduli for the five design speeds.

RESULTS

Solutions to the statistical model were generated for average values of the design-control variables. The four design controls were established at the following levels:

1. Out-of-state passenger cars in traffic stream, 20 percent;
2. Truck combinations in traffic stream, 7.5 percent;
3. Number of commercial roadside establishments per mile, 1 per mile; and
4. Total traffic volume, 900 vph.

These average levels are representative of travel conditions on two-lane, rural highways for design purposes (1, 3).

Geometric Moduli

A multivariate analysis of traffic flow indicated that degree of curve, gradient, minimum stopping sight distance, and lane width were the four geometric-design elements that significantly influenced the rate of traffic movement (3). Therefore, only these four variables were considered over the following ranges in the development of numerical ratings of geometric design:

1. Degree of curve, 0 to 12 deg in 1-deg increments;
2. Gradient, 0 to 10 percent in 1 percent increments;

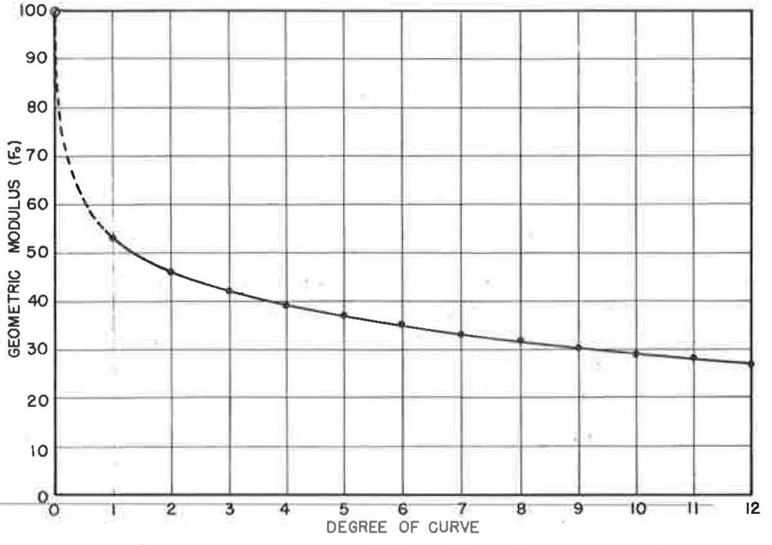

Figure 1. Geometric moduli for curvature restrictions.

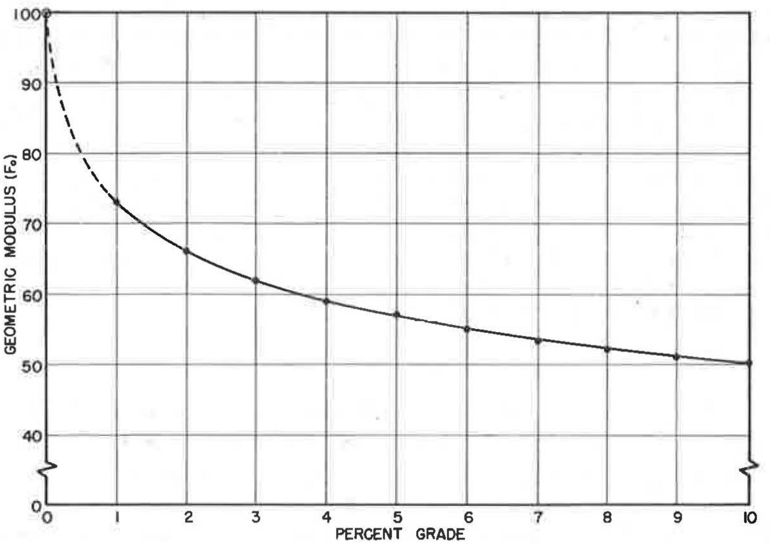

Figure 2. Geometric moduli for gradient restrictions.

3. Minimum stopping sight distance, 200, 300, 400, 500, 600, 800, 1,000, 1,500, 2,000, and 2,500 ft; and
4. Lane width, 9, 10, 11, 12, and 13 ft.

These values are fairly indicative of the geometric conditions of two-lane highways located in rural areas. In regard to sight distance, the limit of driver visibility is approximately 2,500 ft.

The moduli for geometric characteristics of two-lane, rural highways are presented in the Appendix, Tables 1.00 to 5.10. The geometric moduli calculated from the mathematical model were scaled by a factor of ten to produce the tabled values. Different

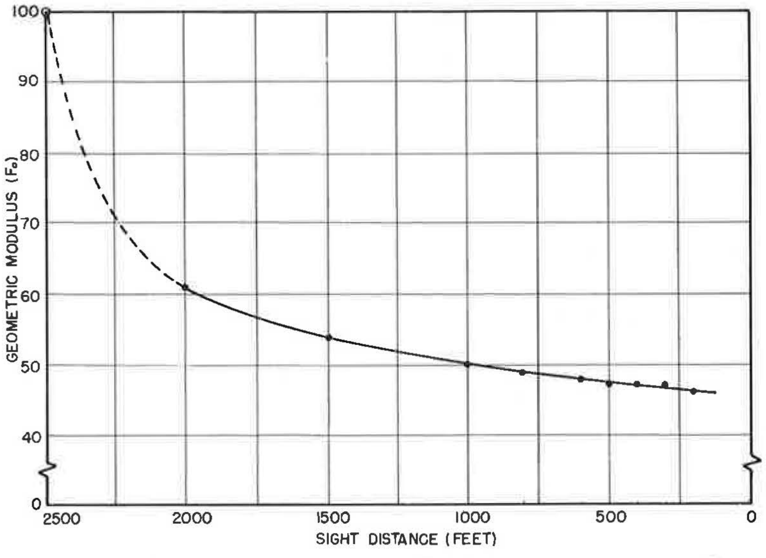

Figure 3. Geometric moduli for sight-distance restrictions.

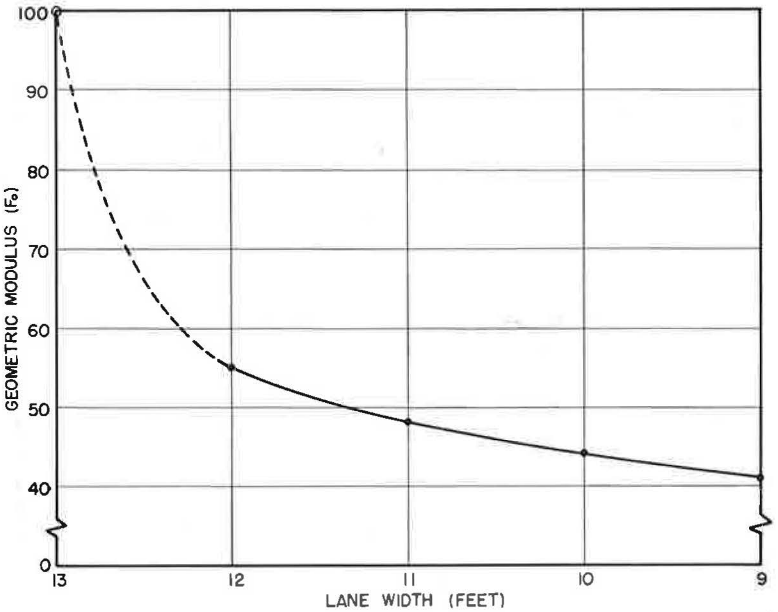

Figure 4. Geometric moduli for lane-width restrictions.

lane widths and various percent grades are represented, respectively, by the unit digit and the tenth and hundredth digits of the numerical designations of these tables. The selected values of degree of curve and minimum stopping sight distance are listed in each tabulation.

If there is no change in average speed, the calculated value of the geometric modulus becomes infinite. Therefore, in Table 1.00, Appendix, the ideal geometric-design condition of tangent and level alignment with a wide lane and an adequate minimum stopping sight distance was arbitrarily assigned the rating of 100. All remaining geometric moduli were computed in relation to this ideal situation.

The relationships between geometric moduli and each design element are shown in Figures 1 to 4. Similar effects on the ease of traffic movement are indicated for increasing values of all restrictive variables, although degree of curve has the most pronounced influence. The moduli of geometric aspects rapidly decrease for small restrictive values and continue to decrease at a decreasing rate as the restrictions of the geometric features increase.

Design Criteria

The development of design criteria for two-lane, rural highways is given in Table 1. The values of the four geometric elements represent reasonable upper limits for degree of curve and gradient and reasonable lower limits for minimum stopping sight distance and lane width at design speeds of 30, 40, 50, 60, and 70 mph. The minimum geometric moduli corresponding to these design speeds are, respectively, 19, 23, 27, 31, and 35. Therefore, in the balanced geometric design of a highway for a given level of

TABLE 1

DESIGN CRITERIA FOR TWO-LANE, RURAL HIGHWAYS

Geometric Elements	Design Speed, mph				
	30	40	50	60	70
Curve, deg	20	12	8	5	3
Gradient, %	7	6	5	4	3
Min. stop. sight dist., ft	200	300	400	500	600
Lane width, ft	11	11	11	12	12
Design level	19	23	27	31	35

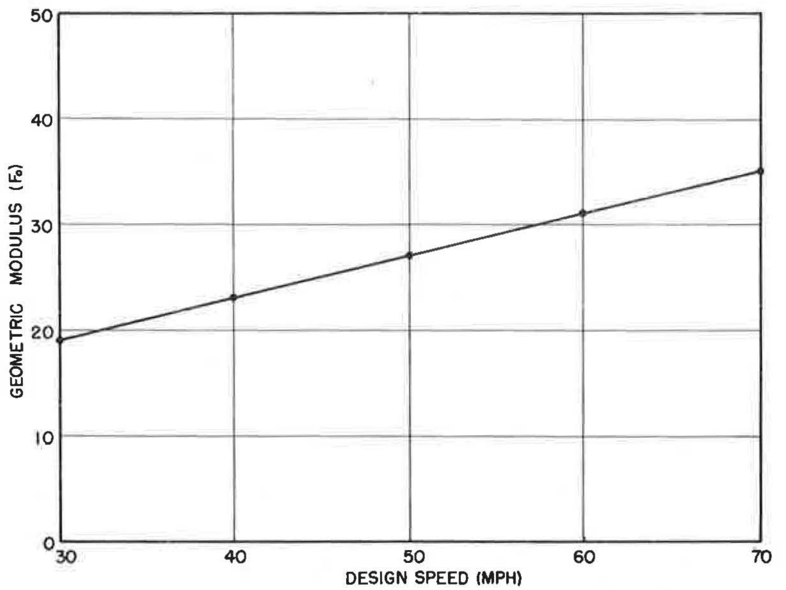

Figure 5. Geometric moduli for design speeds.

design speed, the geometric moduli of the various roadway sections must have ratings equal to or greater than the design-level value.

Design levels of the modulus for geometric characteristics are expressed as a function of design speed in Figure 5. The straight-line relationship is indicative of modern engineering principles, in that the engineer tends to analyze and synthesize in terms of linear associations.

Applications

Several examples are presented to illustrate the application of geometric moduli to problems in highway planning and design. Although reasonable values of degree of curve, gradient, minimum stopping sight distance, and lane width were selected in the development of minimum criteria for various levels of design and terrain conditions, the practicing engineer can apply design and planning standards that are more applicable to his particular situation.

The first illustration is concerned with the design level of the geometric modulus for the selection of various combinations of geometric features to produce a balanced design that is adequate for a specified design speed. If a design speed of 60 mph is warranted, then the moduli of geometric aspects must be equal to or greater than 31 for all sections of this roadway. The limiting conditions of the four geometric elements and the design levels are given in Table 1 as a function of design speed for two-lane highways located in rural areas.

The modulus for geometric characteristics can also be used to compare the desirability of various combinations of geometric features. To illustrate this second example, it is assumed that two different geometric designs are possible for a given highway section. The degree of curve, gradient, minimum stopping sight distance, and lane width are, respectively, 6 deg, 1 percent, 400 ft, and 12 ft for the first design. The corresponding geometric elements for the second case are 2 deg, 4 percent, 1,000 ft, and 12 ft. The respective geometric moduli are determined from Tables 2.01 and 2.04 (Appendix) as 31 and 37. Therefore, it is concluded that the second design affords a better combination of geometric features in regard to traffic-flow characteristics.

Another application of the moduli of geometric aspects is to obtain an average numerical rating for the entire length of a highway that is being designed or redesigned. This quantitative evaluation of the operational conditions is readily ascertained by calculating a weighted average of the geometric moduli for the individual sections that comprise the total highway. Each modulus is weighted by the length of highway for which it is the numerical measure. Weighted geometric moduli can also be used to compare the relative advantages of alternate highway locations.

The final example involves the development of tolerable standards for determining geometric deficiencies in highway needs studies. Estimates of degree of curve, gradi-

TABLE 2

TOLERABLE CRITERIA FOR TWO-LANE, RURAL HIGHWAYS IN THE PRIMARY-STATE CLASSIFICATION

Geometric Elements	Terrain		
	Level	Rolling	Hilly
Curve, deg	6	9	12
Gradient, %	5	6	7
Min. stop. sight dist., ft	500	400	300
Lane width, ft	11	10	9
Tolerable level	29	25	22

TABLE 3

TOLERABLE CRITERIA FOR TWO-LANE, RURAL HIGHWAYS IN THE SECONDARY-STATE CLASSIFICATION

Geometric Elements	Terrain		
	Level	Rolling	Hilly
Curve, deg	10	20	30
Gradient, %	8	10	12
Min. stop. sight dist., ft	400	300	200
Lane width, ft	10	9	9
Tolerable level	24	18	13

ent, minimum stopping sight distance, and lane width that permit reasonably safe, efficient, and comfortable travel on two-lane, rural highways are given in Table 2 for the primary-state classification and in Table 3 for the seceondary-state classification. These tolerable standards for level, rolling, and hilly terrain represent levels of geometric conditions that rank below design standards. The tolerable levels of geometric modulus were obtained from Tables 1.00 to 5.00 (Appendix) for the specified geometric-design conditions. If the road inventory of a primary state system shows that a given highway section located in level terrain has a degree of curve, gradient, minimum stopping sight distance, and lane width of 9 deg, 3 percent, 200 ft, and 10 ft, respectively, then the modulus of geometric aspects is determined from Table 4.03 (Appendix) as 25. In a comparison of the actual geometric modulus of 25 with the tolerable level of 29 (Table 2), it becomes evident that this road section has a geometric deficiency. Thus, numerical ratings of geometric design can be used to evaluate quantitatively the traffic services rendered by existing highway facilities.

CONCLUSIONS

Moduli for geometric characteristics were developed as numerical evaluations of the significant geometric-design elements applicable to two-lane highways located in rural areas. The geometric modulus quantitatively represents the ease with which traffic traverses a highway section with a particular combination of geometric features. Therefore, the effects of highway geometric design on traffic movement can be measured and expressed in terms of numerical ratings.

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Appendix

TABLE 1.00
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 0

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	100	61	54	50	49	48	47	47	47	46
1	53	49	47	45	44	43	43	42	42	42
2	46	44	42	41	40	40	40	39	39	39
3	42	40	39	38	38	37	37	37	37	37
4	39	38	37	36	36	35	35	35	35	35
5	37	36	35	34	34	34	33	33	33	33
6	35	34	33	32	32	32	32	32	32	32
7	33	32	32	31	31	31	31	30	30	30
8	32	31	30	30	30	29	29	29	29	29
9	30	30	29	29	29	28	28	28	28	28
10	29	29	28	28	28	27	27	27	27	27
11	28	28	27	27	27	26	26	26	26	26
12	27	27	26	26	26	26	26	25	25	25

TABLE 1.01
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 1

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	73	59	53	49	48	47	47	46	46	45
1	52	48	46	44	43	43	42	42	42	41
2	45	43	42	40	40	39	39	39	39	39
3	41	40	39	38	37	37	37	37	36	36
4	38	37	37	36	35	35	35	35	35	34
5	36	35	35	34	34	33	33	33	33	33
6	34	34	33	32	32	32	32	32	31	31
7	33	32	32	31	31	30	30	30	30	30
8	31	31	30	30	30	29	29	29	29	29
9	30	30	29	29	28	28	28	28	28	28
10	29	28	28	28	27	27	27	27	27	27
11	28	27	27	27	26	26	26	26	26	26
12	27	27	26	26	26	25	25	25	25	25

TABLE 1.02
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 2

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	66	56	52	48	47	46	46	46	45	45
1	51	48	45	43	43	42	42	42	41	41
2	45	43	41	40	40	39	39	39	38	38
3	41	40	38	37	37	37	37	36	36	36
4	38	37	36	35	35	35	35	34	34	34
5	36	35	34	34	33	33	33	33	33	33
6	34	33	33	32	32	32	32	31	31	31
7	33	32	31	31	31	30	30	30	30	30
8	31	31	30	30	29	29	29	29	29	29
9	30	29	29	28	28	28	28	28	28	28
10	29	28	28	27	27	27	27	27	27	27
11	28	27	27	27	26	26	26	26	26	26
12	27	26	26	26	25	25	25	25	25	25

TABLE 1.03
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 3

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	62	55	51	48	47	46	45	45	44	44
1	49	47	45	43	42	42	41	41	41	41
2	44	42	41	40	39	39	38	38	38	38
3	40	39	38	37	37	36	36	36	36	36
4	38	37	36	35	35	34	34	34	34	34
5	36	35	34	33	33	33	33	33	32	32
6	34	33	33	32	32	31	31	31	31	31
7	32	32	31	31	30	30	30	30	30	30
8	31	30	30	29	29	29	29	29	29	29
9	30	29	29	28	28	28	28	28	28	28
10	29	28	28	27	27	27	27	27	27	27
11	28	27	27	26	26	26	26	26	26	26
12	27	26	26	26	25	25	25	25	25	25

TABLE 1.04
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 4

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	59	53	49	47	46	45	45	44	44	44
1	49	46	44	42	42	41	41	41	40	40
2	43	42	40	39	39	38	38	38	38	38
3	40	39	38	37	36	36	36	36	36	35
4	37	37	36	35	35	34	34	34	34	34
5	35	35	34	33	33	33	33	32	32	32
6	34	33	32	32	31	31	31	31	31	31
7	32	32	31	30	30	30	30	30	30	30
8	31	30	30	29	29	29	29	29	29	28
9	30	29	29	28	28	28	28	28	28	27
10	28	28	28	27	27	27	27	27	27	27
11	27	27	27	26	26	26	26	26	26	26
12	27	26	26	25	25	25	25	25	25	25

TABLE 1.05
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 5

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	57	52	49	46	45	44	44	44	43	43
1	48	45	43	42	41	41	40	40	40	40
2	43	41	40	39	38	38	38	38	37	37
3	40	39	37	37	36	36	36	35	35	35
4	37	36	35	35	34	34	34	34	34	33
5	35	34	34	33	33	32	32	32	32	32
6	33	33	32	32	31	31	31	31	31	31
7	32	31	31	30	30	30	30	30	30	29
8	31	30	30	29	29	29	29	29	28	28
9	29	29	28	28	28	28	28	28	27	27
10	28	28	27	27	27	27	27	27	26	26
11	27	27	27	26	26	26	26	26	26	26
12	26	26	26	25	25	25	25	25	25	25

TABLE 1.06
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 6

DEG.OF	SIGHT DISTANCE -- FEET									
CURVE	2500	2000	1500	1000	800	600	500	400	300	200
0	55	51	48	45	45	44	43	43	43	42
1	47	45	43	41	41	40	40	40	40	39
2	42	41	40	39	38	38	37	37	37	37
3	39	38	37	36	36	36	35	35	35	35
4	37	36	35	34	34	34	34	34	33	33
5	35	34	33	33	32	32	32	32	32	32
6	33	33	32	31	31	31	31	31	31	30
7	32	31	31	30	30	30	30	29	29	29
8	30	30	29	29	29	29	28	28	28	28
9	29	29	28	28	28	28	27	27	27	27
10	28	28	27	27	27	27	27	26	26	26
11	27	27	26	26	26	26	26	26	26	25
12	26	26	26	25	25	25	25	25	25	25

TABLE 1.07
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 7

DEG.OF	SIGHT DISTANCE -- FEET									
CURVE	2500	2000	1500	1000	800	600	500	400	300	200
0	53	50	47	45	44	43	43	43	42	42
1	46	44	42	41	40	40	40	39	39	39
2	42	41	39	38	38	37	37	37	37	37
3	39	38	37	36	36	35	35	35	35	35
4	37	36	35	34	34	34	33	33	33	33
5	35	34	33	33	32	32	32	32	32	32
6	33	32	32	31	31	31	31	30	30	30
7	32	31	30	30	30	30	29	29	29	29
8	30	30	29	29	29	28	28	28	28	28
9	29	29	28	28	28	27	27	27	27	27
10	28	28	27	27	27	26	26	26	26	26
11	27	27	26	26	26	26	26	25	25	25
12	26	26	25	25	25	25	25	25	25	25

TABLE 1.08
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 8

DEG.OF	SIGHT DISTANCE -- FEET									
CURVE	2500	2000	1500	1000	800	600	500	400	300	200
0	52	49	46	44	43	43	42	42	42	42
1	45	44	42	41	40	40	39	39	39	39
2	41	40	39	38	37	37	37	37	36	36
3	39	38	37	36	35	35	35	35	35	34
4	36	35	35	34	34	33	33	33	33	33
5	34	34	33	32	32	32	32	32	31	31
6	33	32	32	31	31	31	30	30	30	30
7	31	31	30	30	30	29	29	29	29	29
8	30	30	29	29	28	28	28	28	28	28
9	29	28	28	28	27	27	27	27	27	27
10	28	27	27	27	27	26	26	26	26	26
11	27	27	26	26	26	25	25	25	25	25
12	26	26	25	25	25	25	25	25	24	24

TABLE 1.09

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 9

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	51	48	45	44	43	42	42	42	41	41
1	45	43	41	40	40	39	39	39	38	38
2	41	40	39	38	37	37	37	36	36	36
3	38	37	36	35	35	35	35	34	34	34
4	36	35	34	34	33	33	33	33	33	33
5	34	33	33	32	32	32	32	31	31	31
6	33	32	31	31	31	30	30	30	30	30
7	31	31	30	30	29	29	29	29	29	29
8	30	29	29	28	28	28	28	28	28	28
9	29	28	28	27	27	27	27	27	27	27
10	28	27	27	27	26	26	26	26	26	26
11	27	26	26	26	26	25	25	25	25	25
12	26	26	25	25	25	25	24	24	24	24

TABLE 1.10

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 13 FT. PERCENT GRADE = 10

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	50	47	45	43	42	42	41	41	41	41
1	44	42	41	40	39	39	39	38	38	38
2	41	39	38	37	37	36	36	36	36	36
3	38	37	36	35	35	35	34	34	34	34
4	36	35	34	33	33	33	33	33	33	32
5	34	33	33	32	32	31	31	31	31	31
6	32	32	31	31	30	30	30	30	30	30
7	31	30	30	29	29	29	29	29	29	29
8	30	29	29	28	28	28	28	28	28	28
9	29	28	28	27	27	27	27	27	27	27
10	28	27	27	26	26	26	26	26	26	26
11	27	26	26	26	25	25	25	25	25	25
12	26	25	25	25	25	24	24	24	24	24

TABLE 2.00

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 0

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	55	51	48	46	45	44	44	43	43	43
1	47	45	43	42	41	40	40	40	40	39
2	43	41	40	39	38	38	38	37	37	37
3	39	38	37	36	36	36	35	35	35	35
4	37	36	35	34	34	34	34	34	33	33
5	35	34	33	33	33	32	32	32	32	32
6	33	33	32	31	31	31	31	31	31	30
7	32	31	31	30	30	30	30	30	29	29
8	30	30	29	29	29	29	29	28	28	28
9	29	29	28	28	28	28	27	27	27	27
10	28	28	27	27	27	27	27	26	26	26
11	27	27	26	26	26	26	26	26	26	25
12	26	26	26	25	25	25	25	25	25	25

TABLE 2.01

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 1

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	54	50	47	45	44	43	43	43	42	42
1	46	44	43	41	41	40	40	40	39	39
2	42	41	39	38	38	37	37	37	37	37
3	39	38	37	36	36	35	35	35	35	35
4	37	36	35	34	34	34	33	33	33	33
5	35	34	33	33	32	32	32	32	32	32
6	33	32	32	31	31	31	31	31	30	30
7	32	31	30	30	30	30	29	29	29	29
8	30	30	29	29	29	28	28	28	28	28
9	29	29	28	28	28	27	27	27	27	27
10	28	28	27	27	27	27	26	26	26	26
11	27	27	26	26	26	26	26	25	25	25
12	26	26	25	25	25	25	25	25	25	25

TABLE 2.02

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 2

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	52	49	46	44	44	43	43	42	42	42
1	46	44	42	41	40	40	39	39	39	39
2	42	40	39	38	38	37	37	37	37	36
3	39	38	37	36	35	35	35	35	35	34
4	36	35	35	34	34	33	33	33	33	33
5	34	34	33	32	32	32	32	32	32	31
6	33	32	32	31	31	31	30	30	30	30
7	31	31	30	30	30	29	29	29	29	29
8	30	30	29	29	28	28	28	28	28	28
9	29	29	28	28	27	27	27	27	27	27
10	28	28	27	27	27	26	26	26	26	26
11	27	27	26	26	26	26	25	25	25	25
12	26	26	25	25	25	25	25	25	25	24

TABLE 2.03
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 3

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	51	48	46	44	43	42	42	42	41	41
1	45	43	42	40	40	39	39	39	39	38
2	41	40	39	38	37	37	37	36	36	36
3	38	37	36	35	35	35	35	35	34	34
4	36	35	34	34	33	33	33	33	33	33
5	34	34	33	32	32	32	32	31	31	31
6	33	32	31	31	31	30	30	30	30	30
7	31	31	30	30	29	29	29	29	29	29
8	30	29	29	29	28	28	28	28	28	28
9	29	28	28	28	27	27	27	27	27	27
10	28	27	27	27	26	26	26	26	26	26
11	27	26	26	26	26	25	25	25	25	25
12	26	26	25	25	25	25	25	24	24	24

TABLE 2.04
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 4

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	50	47	45	43	42	42	42	41	41	41
1	44	43	41	40	39	39	39	38	38	38
2	41	39	38	37	37	37	36	36	36	36
3	38	37	36	35	35	35	34	34	34	34
4	36	35	34	34	33	33	33	33	33	32
5	34	33	33	32	32	32	31	31	31	31
6	32	32	31	31	30	30	30	30	30	30
7	31	30	30	29	29	29	29	29	29	29
8	30	29	29	28	28	28	28	28	28	28
9	29	28	28	27	27	27	27	27	27	27
10	28	27	27	26	26	26	26	26	26	26
11	27	26	26	26	25	25	25	25	25	25
12	26	25	25	25	25	24	24	24	24	24

TABLE 2.05
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 5

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	49	46	44	43	42	41	41	41	41	40
1	44	42	41	39	39	39	38	38	38	38
2	40	39	38	37	37	36	36	36	36	36
3	38	37	36	35	35	34	34	34	34	34
4	36	35	34	33	33	33	33	32	32	32
5	34	33	32	32	32	31	31	31	31	31
6	32	32	31	30	30	30	30	30	30	30
7	31	30	30	29	29	29	29	29	29	29
8	30	29	29	28	28	28	28	28	28	28
9	29	28	28	27	27	27	27	27	27	27
10	28	27	27	26	26	26	26	26	26	26
11	27	26	26	25	25	25	25	25	25	25
12	26	25	25	25	25	24	24	24	24	24

TABLE 2.06

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 6

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	48	46	44	42	42	41	41	40	40	40
1	43	42	40	39	39	38	38	38	38	37
2	40	39	38	37	36	36	36	36	35	35
3	37	36	36	35	34	34	34	34	34	34
4	35	34	34	33	33	33	32	32	32	32
5	34	33	32	32	31	31	31	31	31	31
6	32	31	31	30	30	30	30	30	30	30
7	31	30	30	29	29	29	29	29	29	28
8	29	29	29	28	28	28	28	28	27	27
9	28	28	28	27	27	27	27	27	27	26
10	27	27	27	26	26	26	26	26	26	26
11	26	26	26	25	25	25	25	25	25	25
12	26	25	25	25	24	24	24	24	24	24

TABLE 2.07

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 7

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	47	45	43	42	41	41	40	40	40	40
1	43	41	40	39	38	38	38	37	37	37
2	39	38	37	36	36	36	36	35	35	35
3	37	36	35	34	34	34	34	34	33	33
4	35	34	34	33	33	32	32	32	32	32
5	33	33	32	31	31	31	31	31	31	31
6	32	31	31	30	30	30	30	30	29	29
7	31	30	29	29	29	29	29	28	28	28
8	29	29	28	28	28	28	28	27	27	27
9	28	28	27	27	27	27	27	27	26	26
10	27	27	26	26	26	26	26	26	26	25
11	26	26	26	25	25	25	25	25	25	25
12	25	25	25	24	24	24	24	24	24	24

TABLE 2.08

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 8

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	46	44	43	41	41	40	40	40	39	39
1	42	41	39	38	38	38	37	37	37	37
2	39	38	37	36	36	35	35	35	35	35
3	37	36	35	34	34	34	34	33	33	33
4	35	34	33	33	32	32	32	32	32	32
5	33	32	32	31	31	31	31	31	30	30
6	32	31	31	30	30	30	29	29	29	29
7	30	30	29	29	29	28	28	28	28	28
8	29	29	28	28	28	27	27	27	27	27
9	28	28	27	27	27	27	26	26	26	26
10	27	27	26	26	26	26	26	26	25	25
11	26	26	25	25	25	25	25	25	25	25
12	25	25	25	24	24	24	24	24	24	24

TABLE 2.09
MODULI FCR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 12 FT. PERCENT GRADE = 9

DEG.OF CURVE	SIGHT DISTANCE -- FEET 250C	2000	150C	1000	800	600	5CO	400	300	200
0	46	44	42	41	40	40	39	39	39	39
1	42	40	39	38	38	37	37	37	37	36
2	39	38	37	36	35	35	35	35	35	35
3	36	36	35	34	34	33	33	33	33	33
4	35	34	33	32	32	32	32	32	32	31
5	33	32	32	31	31	31	31	30	30	30
6	31	31	30	30	30	29	29	29	29	29
7	30	30	29	29	29	28	28	28	28	28
8	29	29	28	28	28	27	27	27	27	27
9	28	28	27	27	27	26	26	26	26	26
10	27	27	26	26	26	26	25	25	25	25
11	26	26	25	25	25	25	25	25	25	24
12	25	25	25	24	24	24	24	24	24	24

TABLE 2.10
MODULI FCR GEOMETRIC CHARACTERISTICS

LANE WICTH = 12 FT. PERCENT GRADE = 10

DEG.OF CURVE	SIGHT DISTANCE -- FEET 250C	2000	150C	1000	800	600	5CO	400	300	200
0	45	43	42	40	40	39	39	39	39	38
1	41	40	39	38	37	37	37	37	36	36
2	38	37	36	36	35	35	35	35	34	34
3	36	35	35	34	33	33	33	33	33	33
4	34	34	33	32	32	32	32	32	31	31
5	33	32	31	31	31	30	30	30	30	30
6	31	31	30	30	29	29	29	29	29	29
7	30	30	29	29	28	28	28	28	28	28
8	29	28	28	28	27	27	27	27	27	27
9	28	27	27	27	26	26	26	26	26	26
10	27	26	26	26	26	25	25	25	25	25
11	26	26	25	25	25	25	25	24	24	24
12	25	25	24	24	24	24	24	24	24	24

TABLE 3.00
MODULI FCR GEOMETRIC CHARACTERISTICS

LANE WICTH = 11 FT. PERCENT GRADE = 0

DEG.OF CURVE	SIGHT DISTANCE -- FEET 250C	200C	150C	1000	800	600	5C0	400	300	200
0	48	46	44	42	42	41	41	41	40	40
1	43	42	40	39	39	38	38	38	38	37
2	40	39	38	37	36	36	36	36	36	35
3	37	36	36	35	34	34	34	34	34	34
4	35	35	34	33	33	33	32	32	32	32
5	34	33	32	32	31	31	31	31	31	31
6	32	31	31	30	30	30	30	30	30	30
7	31	30	30	29	29	29	29	29	29	28
8	30	29	29	28	28	28	28	28	28	27
9	28	28	28	27	27	27	27	27	27	27
10	27	27	27	26	26	26	26	26	26	26
11	26	26	26	25	25	25	25	25	25	25
12	26	25	25	25	24	24	24	24	24	24

TABLE 3.C1
MODULI FCR GEOMETRIC CHARACTERISTICS

LANE WICTH = 11 FT. PERCENT GRADE = 1

DEG.OF CURVE	SIGHT DISTANCE -- FEET 250C	2000	150C	1000	80C	6C0	5CC	400	300	200
0	47	45	43	42	41	41	40	40	40	40
1	43	41	40	39	38	38	38	38	37	37
2	40	38	37	36	36	36	36	35	35	35
3	37	36	35	35	34	34	34	34	34	33
4	35	34	34	33	33	32	32	32	32	32
5	33	33	32	31	31	31	31	31	31	31
6	32	31	31	30	30	30	3C	30	29	29
7	31	30	30	29	29	29	29	28	28	28
8	29	29	28	28	28	28	28	27	27	27
9	28	28	27	27	27	27	27	27	26	26
10	27	27	26	26	26	26	26	26	26	26
11	26	26	26	25	25	25	25	25	25	25
12	26	25	25	24	24	24	24	24	24	24

TABLE 3.02
MODULI FCR GEOMETRIC CHARACTERISTICS

LANE WICTH = 11 FT. PERCENT GRADE = 2

DEG.OF CURVE	SIGHT DISTANCE -- FEET 25CC	2000	150C	1000	800	600	5C0	400	300	200
0	47	45	43	41	41	40	40	40	39	39
1	42	41	40	38	38	38	37	37	37	37
2	39	38	37	36	36	35	35	35	35	35
3	37	36	35	34	34	34	34	33	33	33
4	35	34	33	33	32	32	32	32	32	32
5	33	32	32	31	31	31	31	31	31	30
6	32	31	31	30	30	30	30	29	29	29
7	30	30	29	29	29	29	28	28	28	28
8	29	29	28	28	28	28	27	27	27	27
9	28	28	27	27	27	27	26	26	26	26
10	27	27	26	26	26	26	26	26	25	25
11	26	26	26	25	25	25	25	25	25	25
12	25	25	25	24	24	24	24	24	24	24

TABLE 3.03
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 3

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	46	44	42	41	40	40	40	39	39	39
1	42	40	39	38	38	37	37	37	37	37
2	39	38	37	36	36	35	35	35	35	35
3	36	36	35	34	34	34	33	33	33	33
4	35	34	33	32	32	32	32	32	32	32
5	33	32	32	31	31	31	31	30	30	30
6	31	31	30	30	30	29	29	29	29	29
7	30	30	29	29	29	28	28	28	28	28
8	29	29	28	28	28	27	27	27	27	27
9	28	28	27	27	27	26	26	26	26	26
10	27	27	26	26	26	26	26	25	25	25
11	26	26	25	25	25	25	25	25	25	24
12	25	25	25	24	24	24	24	24	24	24

TABLE 3.04
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 4

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	45	43	42	40	40	39	39	39	39	39
1	41	40	39	38	37	37	37	37	36	36
2	38	37	36	36	35	35	35	35	35	34
3	36	35	35	34	34	33	33	33	33	33
4	34	34	33	32	32	32	32	32	31	31
5	33	32	31	31	31	30	30	30	30	30
6	31	31	30	30	30	29	29	29	29	29
7	30	30	29	29	28	28	28	28	28	28
8	29	28	28	28	27	27	27	27	27	27
9	28	27	27	27	26	26	26	26	26	26
10	27	27	26	26	26	25	25	25	25	25
11	26	26	25	25	25	25	25	25	24	24
12	25	25	24	24	24	24	24	24	24	24

TABLE 3.05
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 5

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	45	43	41	40	40	39	39	39	38	38
1	41	40	38	37	37	37	36	36	36	36
2	38	37	36	35	35	35	35	34	34	34
3	36	35	34	34	33	33	33	33	33	33
4	34	33	33	32	32	32	31	31	31	31
5	33	32	31	31	31	30	30	30	30	30
6	31	31	30	30	29	29	29	29	29	29
7	30	29	29	28	28	28	28	28	28	28
8	29	28	28	27	27	27	27	27	27	27
9	28	27	27	27	26	26	26	26	26	26
10	27	26	26	26	25	25	25	25	25	25
11	26	26	25	25	25	25	24	24	24	24
12	25	25	24	24	24	24	24	24	24	24

TABLE 3.06
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 6

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	44	42	41	40	39	39	38	38	38	38
1	40	39	38	37	37	36	36	36	36	36
2	38	37	36	35	35	34	34	34	34	34
3	36	35	34	33	33	33	33	33	32	32
4	34	33	33	32	32	31	31	31	31	31
5	32	32	31	31	30	30	30	30	30	30
6	31	30	30	29	29	29	29	29	29	29
7	30	29	29	28	28	28	28	28	28	28
8	29	28	28	27	27	27	27	27	27	27
9	28	27	27	26	26	26	26	26	26	26
10	27	26	26	26	25	25	25	25	25	25
11	26	25	25	25	25	24	24	24	24	24
12	25	25	24	24	24	24	24	24	23	23

TABLE 3.07
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 7

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	43	42	40	39	39	38	38	38	38	38
1	40	39	38	37	36	36	36	36	36	35
2	37	37	36	35	35	34	34	34	34	34
3	35	35	34	33	33	33	33	32	32	32
4	34	33	32	32	31	31	31	31	31	31
5	32	32	31	30	30	30	30	30	30	30
6	31	30	30	29	29	29	29	29	29	28
7	30	29	29	28	28	28	28	28	28	27
8	28	28	28	27	27	27	27	27	27	27
9	27	27	27	26	26	26	26	26	26	26
10	27	26	26	25	25	25	25	25	25	25
11	26	25	25	25	24	24	24	24	24	24
12	25	24	24	24	24	24	24	23	23	23

TABLE 3.08
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 8

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	43	41	40	39	38	38	38	38	37	37
1	40	39	37	37	36	36	36	35	35	35
2	37	36	35	35	34	34	34	34	34	33
3	35	34	34	33	33	32	32	32	32	32
4	33	33	32	32	31	31	31	31	31	31
5	32	31	31	30	30	30	30	30	30	29
6	31	30	30	29	29	29	29	29	28	28
7	29	29	28	28	28	28	28	28	27	27
8	28	28	27	27	27	27	27	27	26	26
9	27	27	27	26	26	26	26	26	26	26
10	26	26	26	25	25	25	25	25	25	25
11	26	25	25	24	24	24	24	24	24	24
12	25	24	24	24	24	23	23	23	23	23

TABLE 3.09
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 9

DEG.OF	SIGHT DISTANCE -- FEET									
CURVE	2500	2000	1500	1000	800	600	500	400	300	200
0	42	41	40	39	38	38	37	37	37	37
1	39	38	37	36	36	36	35	35	35	35
2	37	36	35	34	34	34	34	33	33	33
3	35	34	33	33	32	32	32	32	32	32
4	33	33	32	31	31	31	31	31	31	30
5	32	31	31	30	30	30	30	29	29	29
6	30	30	29	29	29	29	28	28	28	28
7	29	29	28	28	28	28	27	27	27	27
8	28	28	27	27	27	27	27	26	26	26
9	27	27	26	26	26	26	26	26	25	25
10	26	26	26	25	25	25	25	25	25	25
11	25	25	25	24	24	24	24	24	24	24
12	25	24	24	24	24	23	23	23	23	23

TABLE 3.10
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 11 FT. PERCENT GRADE = 10

DEG.OF	SIGHT DISTANCE -- FEET									
CURVE	2500	2000	1500	1000	800	600	500	400	300	200
0	42	41	39	38	38	37	37	37	37	37
1	39	38	37	36	36	35	35	35	35	35
2	37	36	35	34	34	34	33	33	33	33
3	35	34	33	33	32	32	32	32	32	32
4	33	32	32	31	31	31	31	30	30	30
5	32	31	30	30	30	30	29	29	29	29
6	30	30	29	29	29	28	28	28	28	28
7	29	29	28	28	28	27	27	27	27	27
8	28	28	27	27	27	26	26	26	26	26
9	27	27	26	26	26	26	26	25	25	25
10	26	26	25	25	25	25	25	25	25	25
11	25	25	25	24	24	24	24	24	24	24
12	24	24	24	24	23	23	23	23	23	23

TABLE 4.00
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 0

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	44	42	41	40	39	39	39	38	38	38
1	41	39	38	37	37	36	36	36	36	36
2	38	37	36	35	35	35	34	34	34	34
3	36	35	34	33	33	33	33	33	33	32
4	34	33	33	32	32	31	31	31	31	31
5	32	32	31	31	30	30	30	30	30	30
6	31	30	30	29	29	29	29	29	29	29
7	30	29	29	28	28	28	28	28	28	28
8	29	28	28	27	27	27	27	27	27	27
9	28	27	27	26	26	26	26	26	26	26
10	27	26	26	26	25	25	25	25	25	25
11	26	25	25	25	25	24	24	24	24	24
12	25	25	24	24	24	24	24	24	24	23

TABLE 4.01
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 1

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	44	42	41	39	39	38	38	38	38	38
1	40	39	38	37	37	36	36	36	36	36
2	38	37	36	35	35	34	34	34	34	34
3	35	35	34	33	33	33	33	32	32	32
4	34	33	32	32	32	31	31	31	31	31
5	32	32	31	30	30	30	30	30	30	30
6	31	30	30	29	29	29	29	29	29	29
7	30	29	29	28	28	28	28	28	28	28
8	29	28	28	27	27	27	27	27	27	27
9	27	27	27	26	26	26	26	26	26	26
10	27	26	26	25	25	25	25	25	25	25
11	26	25	25	25	24	24	24	24	24	24
12	25	25	24	24	24	24	24	23	23	23

TABLE 4.02
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 2

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	43	42	40	39	39	38	38	38	37	37
1	40	39	38	37	36	36	36	36	35	35
2	37	36	35	35	34	34	34	34	34	34
3	35	34	34	33	33	32	32	32	32	32
4	33	33	32	32	31	31	31	31	31	31
5	32	31	31	30	30	30	30	30	30	29
6	31	30	30	29	29	29	29	29	28	28
7	29	29	29	28	28	28	28	28	27	27
8	28	28	27	27	27	27	27	27	27	26
9	27	27	27	26	26	26	26	26	26	26
10	26	26	26	25	25	25	25	25	25	25
11	26	25	25	25	24	24	24	24	24	24
12	25	24	24	24	24	24	23	23	23	23

TABLE 4.03

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 3

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	43	41	40	39	38	38	38	37	37	37
1	39	38	37	36	36	36	35	35	35	35
2	37	36	35	34	34	34	34	34	33	33
3	35	34	33	33	33	32	32	32	32	32
4	33	33	32	31	31	31	31	31	31	30
5	32	31	31	30	30	30	30	30	29	29
6	30	30	29	29	29	29	29	28	28	28
7	29	29	28	28	28	28	27	27	27	27
8	28	28	27	27	27	27	27	26	26	26
9	27	27	26	26	26	26	26	26	26	25
10	26	26	26	25	25	25	25	25	25	25
11	25	25	25	24	24	24	24	24	24	24
12	25	24	24	24	24	23	23	23	23	23

TABLE 4.04

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 4

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	42	41	39	38	38	37	37	37	37	37
1	39	38	37	36	36	35	35	35	35	35
2	37	36	35	34	34	34	33	33	33	33
3	35	34	33	33	32	32	32	32	32	32
4	33	32	32	31	31	31	31	31	30	30
5	32	31	30	30	30	30	29	29	29	29
6	30	30	29	29	29	28	28	28	28	28
7	29	29	28	28	28	27	27	27	27	27
8	28	28	27	27	27	27	26	26	26	26
9	27	27	26	26	26	26	26	25	25	25
10	26	26	25	25	25	25	25	25	25	25
11	25	25	25	24	24	24	24	24	24	24
12	25	24	24	24	23	23	23	23	23	23

TABLE 4.05

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 5

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	42	40	39	38	38	37	37	37	37	36
1	39	38	37	36	35	35	35	35	35	34
2	36	35	35	34	34	33	33	33	33	33
3	34	34	33	32	32	32	32	32	32	31
4	33	32	32	31	31	31	30	30	30	30
5	31	31	30	30	30	29	29	29	29	29
6	30	30	29	29	28	28	28	28	28	28
7	29	29	28	28	27	27	27	27	27	27
8	28	28	27	27	27	26	26	26	26	26
9	27	27	26	26	26	26	25	25	25	25
10	26	26	25	25	25	25	25	25	25	24
11	25	25	25	24	24	24	24	24	24	24
12	24	24	24	23	23	23	23	23	23	23

TABLE 4.06
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 6

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	41	40	39	38	37	37	37	36	36	36
1	38	37	36	35	35	35	35	35	34	34
2	36	35	34	34	33	33	33	33	33	33
3	34	33	33	32	32	32	32	31	31	31
4	33	32	31	31	31	30	30	30	30	30
5	31	31	30	30	29	29	29	29	29	29
6	30	29	29	29	28	28	28	28	28	28
7	29	28	28	28	27	27	27	27	27	27
8	28	27	27	27	26	26	26	26	26	26
9	27	26	26	26	26	25	25	25	25	25
10	26	26	25	25	25	25	25	24	24	24
11	25	25	24	24	24	24	24	24	24	24
12	24	24	24	23	23	23	23	23	23	23

TABLE 4.07
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 7

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	41	39	38	37	37	37	36	36	36	36
1	38	37	36	35	35	35	34	34	34	34
2	36	35	34	33	33	33	33	33	33	32
3	34	33	33	32	32	32	31	31	31	31
4	32	32	31	31	30	30	30	30	30	30
5	31	30	30	29	29	29	29	29	29	29
6	30	29	29	28	28	28	28	28	28	28
7	29	28	28	27	27	27	27	27	27	27
8	28	27	27	26	26	26	26	26	26	26
9	27	26	26	26	25	25	25	25	25	25
10	26	25	25	25	25	24	24	24	24	24
11	25	25	24	24	24	24	24	24	24	23
12	24	24	24	23	23	23	23	23	23	23

TABLE 4.08
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 10 FT. PERCENT GRADE = 8

DEG.OF CURVE	2500	2000	1500	1000	800	600	500	400	300	200
	SIGHT DISTANCE -- FEET									
0	40	39	38	37	37	36	36	36	36	36
1	38	37	36	35	35	34	34	34	34	34
2	36	35	34	33	33	33	33	32	32	32
3	34	33	32	32	32	31	31	31	31	31
4	32	32	31	30	30	30	30	30	30	30
5	31	30	30	29	29	29	29	29	29	29
6	30	29	29	28	28	28	28	28	28	28
7	29	28	28	27	27	27	27	27	27	27
8	28	27	27	26	26	26	26	26	26	26
9	27	26	26	25	25	25	25	25	25	25
10	26	25	25	25	25	24	24	24	24	24
11	25	25	24	24	24	24	24	24	23	23
12	24	24	23	23	23	23	23	23	23	23

TABLE 4.09

MODULI FCR GEOMETRIC CHARACTERISTICS

LANE WICTH = 10 FT. PERCENT GRADE = 9

DEG.OF CURVE	250C	2000	150C	1000	800	600	5CO	400	300	200
	SIGHT DISTANCE -- FEET									
0	40	39	38	37	36	36	36	36	35	35
1	37	36	36	35	34	34	34	34	34	34
2	35	34	34	33	33	33	32	32	32	32
3	34	33	32	32	31	31	31	31	31	31
4	32	31	31	30	30	30	30	30	30	30
5	31	30	30	29	29	29	29	29	29	28
6	29	29	29	28	28	28	28	28	27	27
7	28	28	28	27	27	27	27	27	27	26
8	27	27	27	26	26	26	26	26	26	26
9	26	26	26	25	25	25	25	25	25	25
10	26	25	25	25	24	24	24	24	24	24
11	25	24	24	24	24	24	23	23	23	23
12	24	24	23	23	23	23	23	23	23	23

TABLE 4.10

MODULI FCR GEOMETRIC CHARACTERISTICS

LANE WICTH = 10 FT. PERCENT GRADE = 10

DEG.OF CURVE	250C	2000	150C	1000	800	600	5CO	400	300	200
	SIGHT CISTANCE -- FEET									
0	39	38	37	36	36	36	36	35	35	35
1	37	36	35	34	34	34	34	34	33	33
2	35	34	34	33	33	32	32	32	32	32
3	33	33	32	31	31	31	31	31	31	31
4	32	31	31	30	30	30	30	30	29	29
5	31	30	29	29	29	29	29	28	28	28
6	29	29	28	28	28	28	28	27	27	27
7	28	28	27	27	27	27	27	27	26	26
8	27	27	26	26	26	26	26	26	26	25
9	26	26	26	25	25	25	25	25	25	25
10	25	25	25	24	24	24	24	24	24	24
11	25	24	24	24	24	23	23	23	23	23
12	24	24	23	23	23	23	23	23	23	22

TABLE 5.00
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 0

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	41	40	39	38	37	37	37	37	36	36
1	38	37	36	36	35	35	35	35	34	34
2	36	35	35	34	34	33	33	33	33	33
3	34	34	33	32	32	32	32	32	31	31
4	33	32	31	31	31	30	30	30	30	30
5	31	31	30	30	29	29	29	29	29	29
6	30	30	29	29	28	28	28	28	28	28
7	29	28	28	28	27	27	27	27	27	27
8	28	27	27	27	26	26	26	26	26	26
9	27	26	26	26	26	25	25	25	25	25
10	26	26	25	25	25	25	25	24	24	24
11	25	25	24	24	24	24	24	24	24	24
12	24	24	24	23	23	23	23	23	23	23

TABLE 5.01
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 1

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	41	40	38	37	37	37	36	36	36	36
1	38	37	36	35	35	35	35	34	34	34
2	36	35	34	34	33	33	33	33	33	32
3	34	33	33	32	32	32	31	31	31	31
4	32	32	31	31	30	30	30	30	30	30
5	31	31	30	30	29	29	29	29	29	29
6	30	29	29	28	28	28	28	28	28	28
7	29	28	28	27	27	27	27	27	27	27
8	28	27	27	26	26	26	26	26	26	26
9	27	26	26	26	25	25	25	25	25	25
10	26	25	25	25	25	25	24	24	24	24
11	25	25	24	24	24	24	24	24	24	24
12	24	24	24	23	23	23	23	23	23	23

TABLE 5.02
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 2

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	40	39	38	37	37	36	36	36	36	36
1	38	37	36	35	35	34	34	34	34	34
2	36	35	34	33	33	33	33	33	32	32
3	34	33	32	32	32	31	31	31	31	31
4	32	32	31	31	30	30	30	30	30	30
5	31	30	30	29	29	29	29	29	29	29
6	30	29	29	28	28	28	28	28	28	28
7	29	28	28	27	27	27	27	27	27	27
8	28	27	27	26	26	26	26	26	26	26
9	27	26	26	25	25	25	25	25	25	25
10	26	25	25	25	25	24	24	24	24	24
11	25	25	24	24	24	24	24	24	23	23
12	24	24	23	23	23	23	23	23	23	23

TABLE 5.03
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 3

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	40	39	38	37	36	36	36	36	36	35
1	37	36	36	35	34	34	34	34	34	34
2	35	35	34	33	33	33	32	32	32	32
3	34	33	32	32	31	31	31	31	31	31
4	32	31	31	30	30	30	30	30	30	30
5	31	30	30	29	29	29	29	29	29	28
6	30	29	29	28	28	28	28	28	28	27
7	28	28	28	27	27	27	27	27	27	27
8	27	27	27	26	26	26	26	26	26	26
9	26	26	26	25	25	25	25	25	25	25
10	26	25	25	25	24	24	24	24	24	24
11	25	24	24	24	24	24	23	23	23	23
12	24	24	23	23	23	23	23	23	23	23

TABLE 5.04
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 4

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	40	38	37	36	36	36	36	35	35	35
1	37	36	35	35	34	34	34	34	34	33
2	35	34	34	33	33	32	32	32	32	32
3	33	33	32	31	31	31	31	31	31	31
4	32	31	31	30	30	30	30	30	29	29
5	31	30	30	29	29	29	29	28	28	28
6	29	29	28	28	28	28	28	27	27	27
7	28	28	27	27	27	27	27	27	26	26
8	27	27	26	26	26	26	26	26	26	26
9	26	26	26	25	25	25	25	25	25	25
10	26	25	25	24	24	24	24	24	24	24
11	25	24	24	24	24	23	23	23	23	23
12	24	24	23	23	23	23	23	23	23	23

TABLE 5.05
MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 5

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	39	38	37	36	36	35	35	35	35	35
1	37	36	35	34	34	34	34	33	33	33
2	35	34	33	33	32	32	32	32	32	32
3	33	32	32	31	31	31	31	31	31	30
4	32	31	31	30	30	30	30	29	29	29
5	30	30	29	29	29	29	28	28	28	28
6	29	29	28	28	28	28	27	27	27	27
7	28	28	27	27	27	27	26	26	26	26
8	27	27	26	26	26	26	26	26	25	25
9	26	26	26	25	25	25	25	25	25	25
10	25	25	25	24	24	24	24	24	24	24
11	25	24	24	24	23	23	23	23	23	23
12	24	24	23	23	23	23	23	23	22	22

TABLE 5.06

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 6

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	39	38	37	36	36	35	35	35	35	35
1	36	36	35	34	34	34	33	33	33	33
2	35	34	33	32	32	32	32	32	32	32
3	33	32	32	31	31	31	31	30	30	30
4	31	31	30	30	30	29	29	29	29	29
5	30	30	29	29	29	28	28	28	28	28
6	29	29	28	28	28	27	27	27	27	27
7	28	28	27	27	27	26	26	26	26	26
8	27	27	26	26	26	26	26	25	25	25
9	26	26	25	25	25	25	25	25	25	24
10	25	25	25	24	24	24	24	24	24	24
11	24	24	24	24	23	23	23	23	23	23
12	24	23	23	23	23	23	23	22	22	22

TABLE 5.07

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 7

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	38	37	36	36	35	35	35	35	35	34
1	36	35	35	34	34	33	33	33	33	33
2	34	34	33	32	32	32	32	32	31	31
3	33	32	31	31	31	30	30	30	30	30
4	31	31	30	30	30	29	29	29	29	29
5	30	30	29	29	28	28	28	28	28	28
6	29	28	28	28	27	27	27	27	27	27
7	28	27	27	27	26	26	26	26	26	26
8	27	27	26	26	26	25	25	25	25	25
9	26	26	25	25	25	25	25	25	24	24
10	25	25	24	24	24	24	24	24	24	24
11	24	24	24	23	23	23	23	23	23	23
12	24	23	23	23	23	22	22	22	22	22

TABLE 5.08

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 8

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	38	37	36	35	35	35	35	34	34	34
1	36	35	34	34	33	33	33	33	33	33
2	34	33	33	32	32	32	31	31	31	31
3	33	32	31	31	31	30	30	30	30	30
4	31	31	30	30	29	29	29	29	29	29
5	30	29	29	28	28	28	28	28	28	28
6	29	28	28	27	27	27	27	27	27	27
7	28	27	27	27	26	26	26	26	26	26
8	27	26	26	26	25	25	25	25	25	25
9	26	26	25	25	25	25	24	24	24	24
10	25	25	24	24	24	24	24	24	24	24
11	24	24	24	23	23	23	23	23	23	23
12	24	23	23	23	22	22	22	22	22	22

TABLE 5.09

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 9

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	38	37	36	35	35	34	34	34	34	34
1	36	35	34	33	33	33	33	33	32	32
2	34	33	33	32	32	31	31	31	31	31
3	32	32	31	31	30	30	30	30	30	30
4	31	30	30	29	29	29	29	29	29	29
5	30	29	29	28	28	28	28	28	28	28
6	29	28	28	27	27	27	27	27	27	27
7	28	27	27	26	26	26	26	26	26	26
8	27	26	26	26	25	25	25	25	25	25
9	26	25	25	25	25	24	24	24	24	24
10	25	25	24	24	24	24	24	24	23	23
11	24	24	24	23	23	23	23	23	23	23
12	23	23	23	23	22	22	22	22	22	22

TABLE 5.10

MODULI FOR GEOMETRIC CHARACTERISTICS

LANE WIDTH = 9 FT. PERCENT GRADE = 10

DEG.OF CURVE	SIGHT DISTANCE -- FEET 2500	2000	1500	1000	800	600	500	400	300	200
0	37	37	36	35	35	34	34	34	34	34
1	35	35	34	33	33	33	33	32	32	32
2	34	33	32	32	31	31	31	31	31	31
3	32	32	31	30	30	30	30	30	30	30
4	31	30	30	29	29	29	29	29	29	28
5	30	29	29	28	28	28	28	28	28	27
6	28	28	28	27	27	27	27	27	27	27
7	27	27	27	26	26	26	26	26	26	26
8	27	26	26	25	25	25	25	25	25	25
9	26	25	25	25	24	24	24	24	24	24
10	25	24	24	24	24	24	24	23	23	23
11	24	24	23	23	23	23	23	23	23	23
12	23	23	23	22	22	22	22	22	22	22

A Quantitative Evaluation of the Geometric Aspects of Highways

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This study is an investigation of a quantitative measure of the resistance to the flow of traffic as offered by geometric highway features. Under consideration is a mechanistic model resulting from the postulate that traffic reacts to a motivating pressure potential which in turn reflects the behavior of the traffic traversing a particular section of highway. When solved, the governing differential equation yields a parameter called the modulus of geometric aspects. This parameter is a measure of the ease with which traffic traverses the given roadway section.

To evaluate the developed model and determine the reasonableness of the modulus of geometric aspects, a detailed study was undertaken of vehicle speeds on an actual highway curve. A procedure was developed whereby the spot speeds could be calculated from observations recorded by photographic means. Statistical methods were used to analyze the data and to determine the goodness of fit of the theoretical and observed speed distributions.

The success of the results obtained from the study of the first highway curve indicated the advisability of extending the study to additional geometric highway features (other curves, merging conditions, etc.). Additional field experiments were conducted to provide a more generalized basis of evaluating the reliability of the developed modulus.

The results of the study reveal that the mechanistic model as developed conforms closely to the observed speeds in the vicinity of geometric features except for special highway features requiring extreme speed changes. Subject to the same condition, the modulus of geometric aspects provides a reproducible quantitative rating of the geometric highway feature.

•THE PRIMARY objective of this study is to evaluate a proposed method of rating geometric features of highways. The rating, named the "modulus of geometric aspects" and shown symbolically as F_0, is a measure of the ease (as determined by speed changes) with which a vehicle may traverse a section of highway with a particular geometric feature, that is, F_0 is the reciprocal of flow resistance offered to the vehicle by the feature. The theoretical development of this modulus follows.

NOMENCLATURE

p = motivating pressure potential,
x = a specific location along the roadway,
L = a particular length of roadway (ft),

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v = vehicle speed (mph),
t = time (sec),
ρ = vehicle density (vehicles per unit of effective area),
w = effective width of roadway,
N = number of vehicles,
c_1, c_2, k = coefficients of proportionality,
a^2 = $(c_1 + c_2)(k)^{-1}$ = constant,
P = passenger car,
T = truck, and
P.C. = point of curvature, that is the location where the vehicle first encounters the actual change in direction in a horizontal highway curve. The station of the P.C. in all cases was taken as 0 + 00. The stations of all points was taken as positive or negative relative to the P.C. For example, a point 243 ft on the curve would be denoted as Sta. 2 + 43, whereas a point 243 ft prior to the P.C. would be Sta. - 2 + 43.

THEORY

The development of the "modulus of geometric aspects" (1) is based upon three assumptions:

1. The behavior of a vehicle upon a roadway is the result of the driver's reaction to a motivating "pressure potential" under the prevailing ambient conditions. The pressure potential is defined so that

$$\frac{\partial p}{\partial x} = -kv \tag{1}$$

2. The change in vehicular density with respect to the pressure potential is proportional to the density:

$$\frac{\partial \rho}{\partial p} = c_1 \rho \tag{2}$$

3. The change in effective width of the roadway with respect to the pressure potential is proportional to the effective width:

$$\frac{\partial w}{\partial p} = c_2 w \tag{3}$$

Traffic flow may be considered as a conserved flow; that is, the change in the number of vehicles within a section, during a specified time interval, must equal the difference in the number of vehicles entering the section and the number of vehicles leaving the section (during the time interval). When expressed mathematically and reduced by the assumptions listed above, the controlling equation is of the form

$$\frac{\partial^2 p}{\partial x^2} = a^2 \frac{\partial p}{\partial t} \tag{4}$$

The unique solution of Eq. 4 depends upon the imposed boundary and initial conditions. For this purpose consider the section of roadway shown in Figure 1. At $x = 0$, the

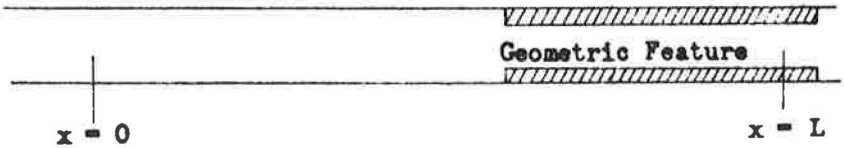


Figure 1. Schematic illustration of roadway section.

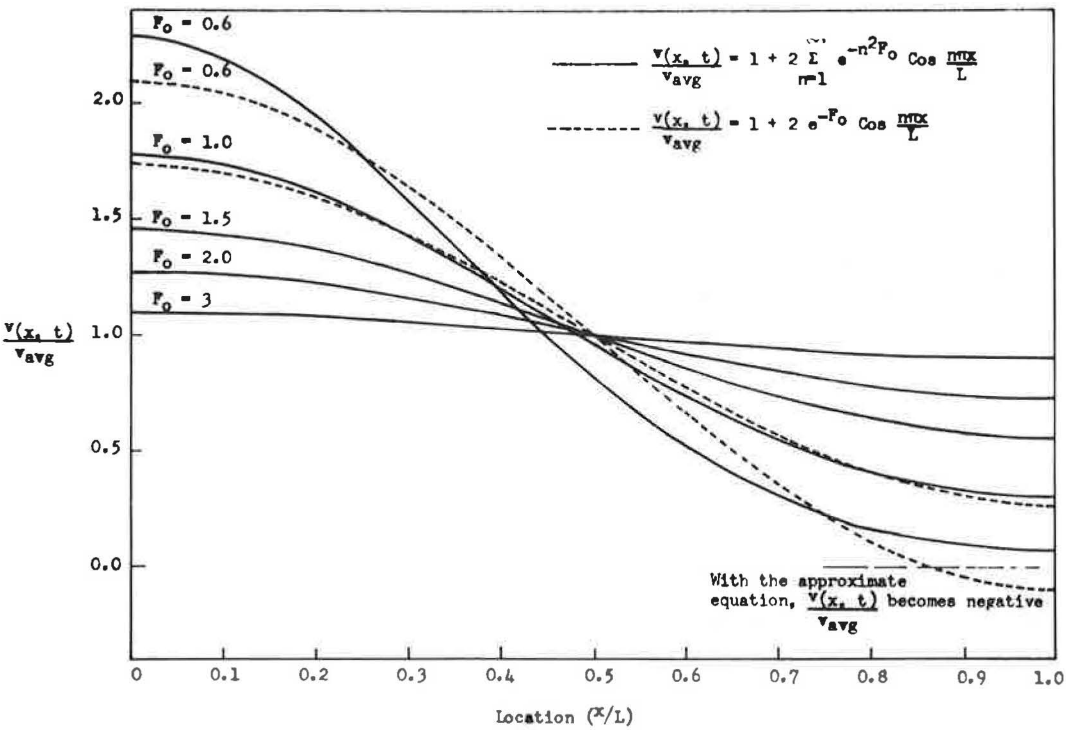


Figure 2. Theoretical speed ratio-distance relationships for selected values of F_0.

vehicle is unhampered by the geometric feature and is operating at a potential p_0. At $x = L$, a location influenced by the geometric feature, the potential has changed to p_1 where $p_1 = p_0 - \Delta p$. At time $t = 0$, the drivers of the vehicles are unaware of the necessity of a change in potential on the roadway, thus $p(x, 0) = p_0$.

A Fourier series solution to the controlling equation may be obtained with the listed conditions. The result is

$$\frac{v(x,t)}{v_{avg}} = 1 - 2\sum_{n=1}^{\infty}(-1)^{n-1} e^{-n^2F_0} \cos\frac{n\pi x}{L} \tag{5}$$

where

$F_0 = \pi^2 t/a^2L^2$.

The speed relationship is symmetrical about the location $x = L$ and repeats in intervals of $2L$. The ratio increases as x goes from 0 to L and decreases as x goes from L to $2L$. In the evaluation of specific geometric aspects vehicle speeds normally decrease

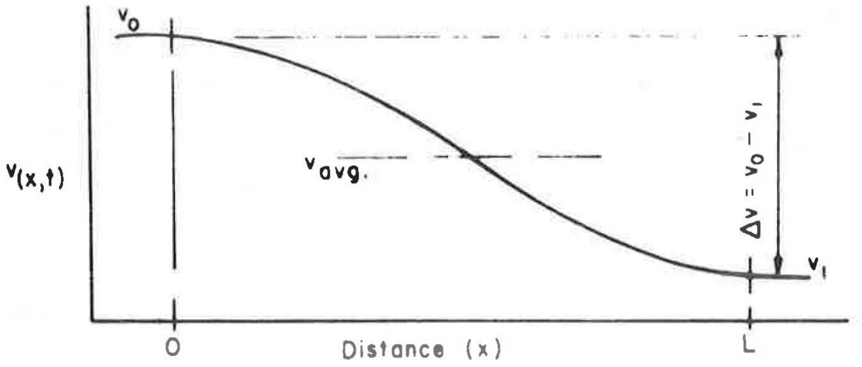


Figure 3. Theoretical speed-distance relationship.

as the feature is approached because traffic flow is impeded. For these features the theoretical curve from x = L to x = 2L (area where the ratio is decreasing) would be considered. Rewriting the speed ratio equation so that the origin is moved L units to the right (replacing x by x + L) and simplfying produces:

$$\frac{v(x,t)}{v_{avg}} = 1 + 2\sum_{n=1}^{\infty} e^{-n^2F_0} \cos\frac{n\pi x}{L} \tag{6}$$

Figure 2 shows the speed ratio-distance curves obtained by introducing various values of F_0 into the theoretical equation. The dashed line curves are secured by using only the first term of the Fourier series. When F_0 is above 1.0, the first term approximation and the complete series curves are indistinguishable. Thus the first term may be used without significant error when F_0 is equal to or greater than 1.0.

$$\frac{v(x,t)}{v_{avg}} = 1 + 2e^{-F_0} \cos\frac{\pi x}{L} \tag{7}$$

The general shape of speed ratio-distance curve is shown by Figure 3. At x = 0, the vehicle speed is v_0 and $v_0 = v_{avg}\left(1 + 2e^{-F_0}\right)$. At v = L, vehicle speed is v_1 ($v_1 = v_0 - \Delta v$) and $v_1 = v_{avg}\left(1 - 2e^{-F_0}\right)$. Solving for e^{-F_0} results in

$$e^{-F_0} = (v_0 - v_1)/2(v_0 + v_1) = \Delta v/(4v_0 - 2\Delta v) \tag{8}$$

If desired, F_0 may be obtained as a natural logarithm function of speed and speed change:

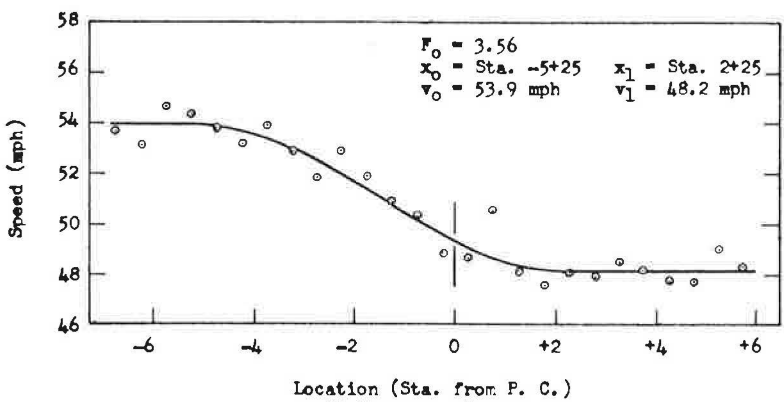
$$F_0 = \ln(4v_0 - 2\Delta v) - \ln\Delta v \tag{9}$$

DATA COLLECTION AND ANALYSIS

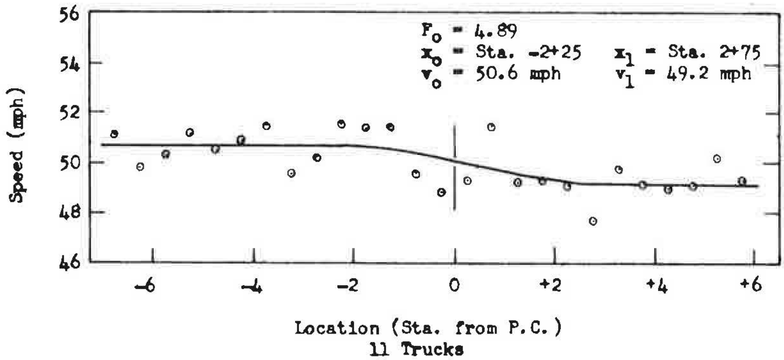
To obtain complete information of vehicle behavior on a section of roadway, a photographic method of data collection was used. A 16-mm motion picture camera, equipped with telephoto lens, was mounted at a vantage point some 1,500 to 2,000 ft from the geometric feature under consideration. From this distance, at an approximate right angle to the feature, a total roadway distance of 1,200 to 1,500 ft could be covered without difficulty. Fifty-foot intervals were measured along the approach and through the feature. These measurements were projected radially to the camera site and marked by white stakes located in the fence line along the edge of the right-of-way. From the camera location, a distance along the roadway could be determined by observing the vehicle passing the radial stakes. The motion picture camera was operated at a selected speed (generally 30 frames per second) and checked periodically by recording a stop watch on film for several seconds.

The developed film was viewed by a time-motion study projector. The viewer could estimate to the nearest one-tenth of a frame the arrival of a vehicle at any particular marker. Thence the time period required for the vehicle to pass from one marker to the next could be estimated to the nearest one three-hundredth of a second. With a known distance and a known elapsed time interval, "spot" speed in miles per hour could be calculated for the roadway section under study.

After the data had been tabulated they were separated into observations of passenger cars and observations of trucks because the two have different operating characteristics. Next the recorded data were reduced to vehicle speeds, summed and averaged for each section of roadway. The average speed was plotted versus location (Figs. 4-10).



29 Passenger Cars



11 Trucks

Figure 4. Speed-location graph for curve on US 24 near Reynolds, Ind.—Data Set No. 1.

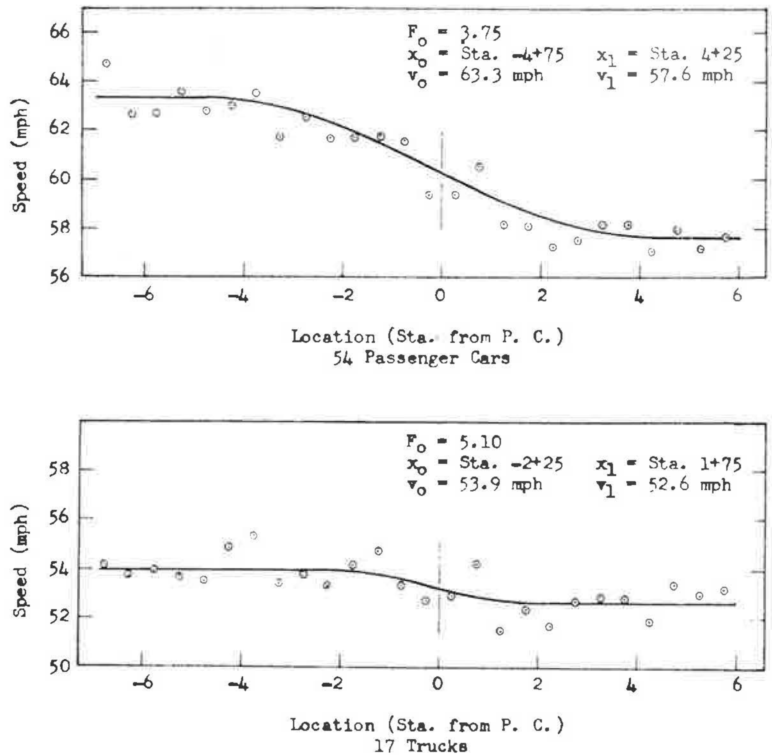


Figure 5. Speed-location graph for curve on US 24 near Reynolds, Ind.—Data Set No. 2.

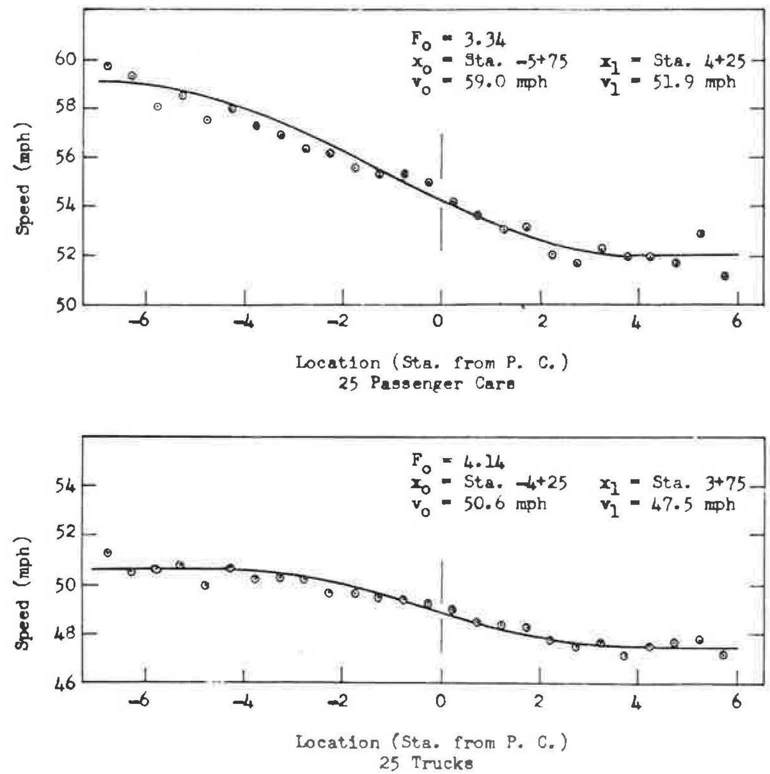


Figure 6. Speed-location graph for curve on US 24 near Reynolds, Ind.—Data Set No. 3.

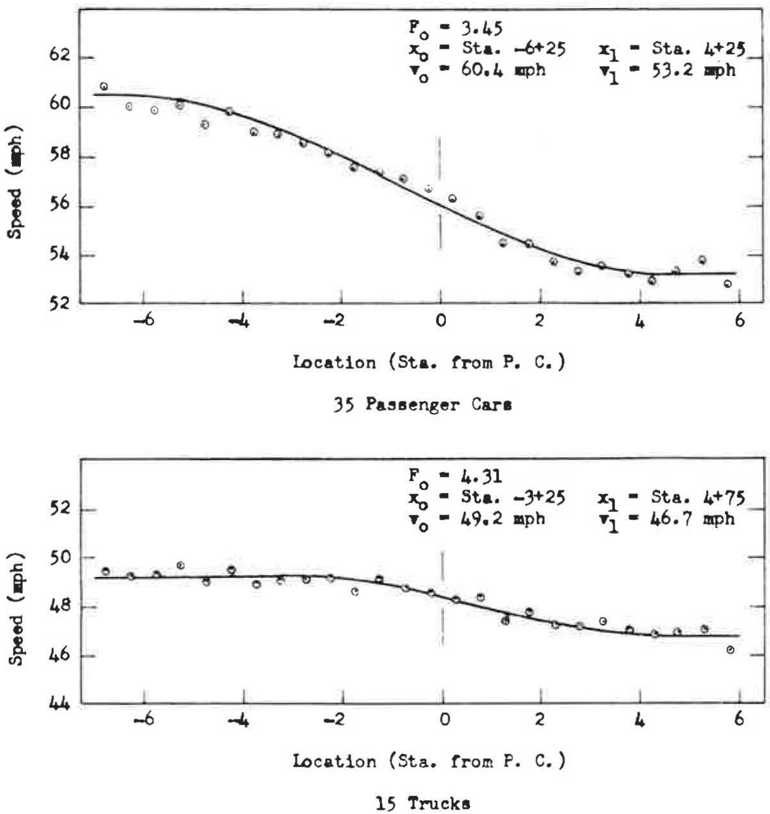


Figure 7. Speed-location graph for curve on US 24 near Reynolds, Ind.—Data Set No. 4.

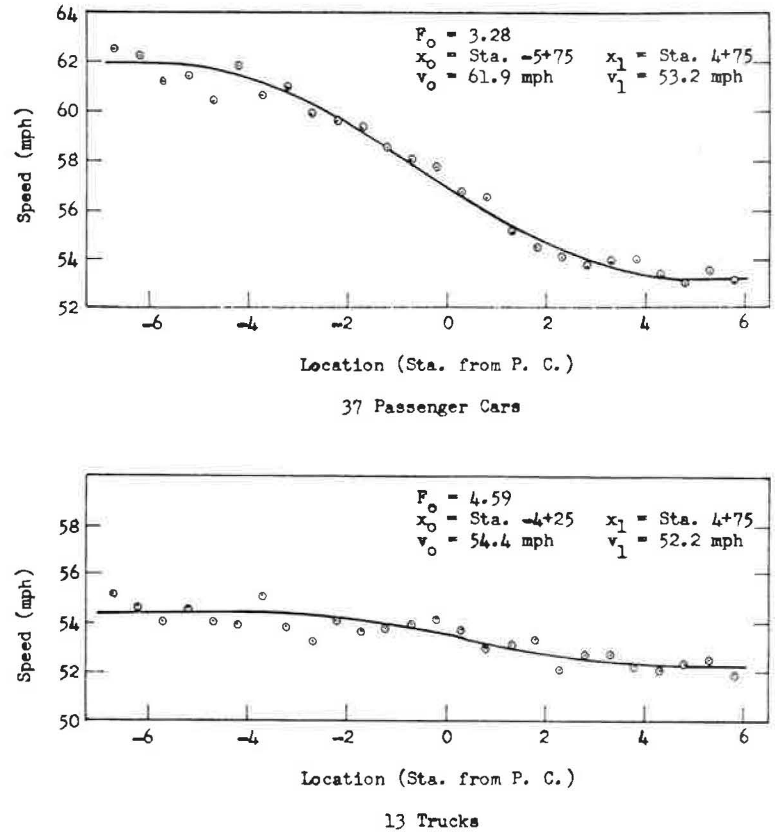


Figure 8. Speed-location graph for curve on US 24 near Reynolds, Ind.—Data Set No. 5.

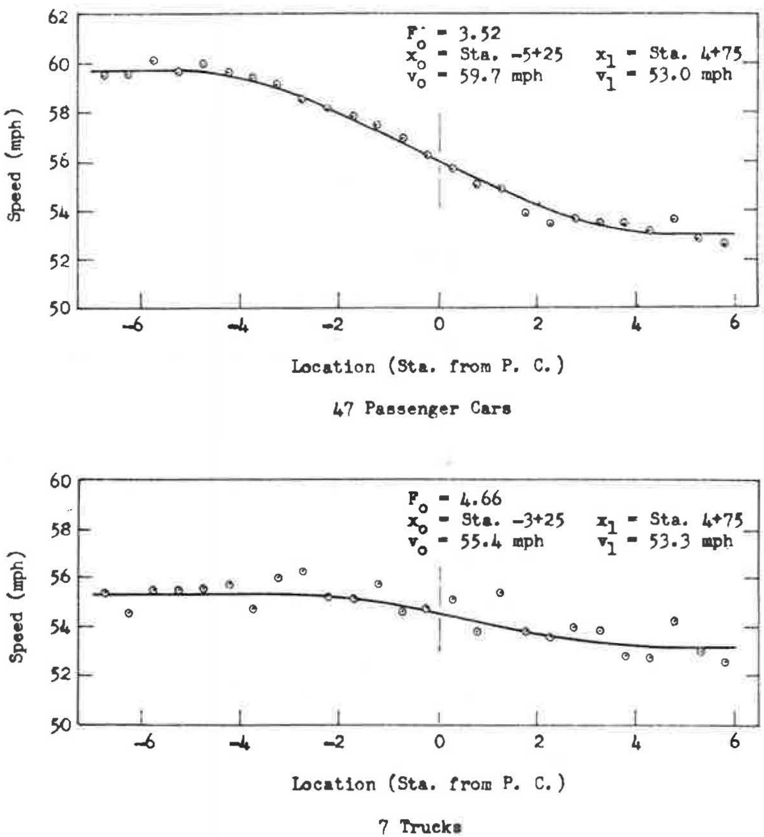


Figure 9. Speed-location graph for curve on US 24 near Reynolds, Ind.—Data Set No. 6.

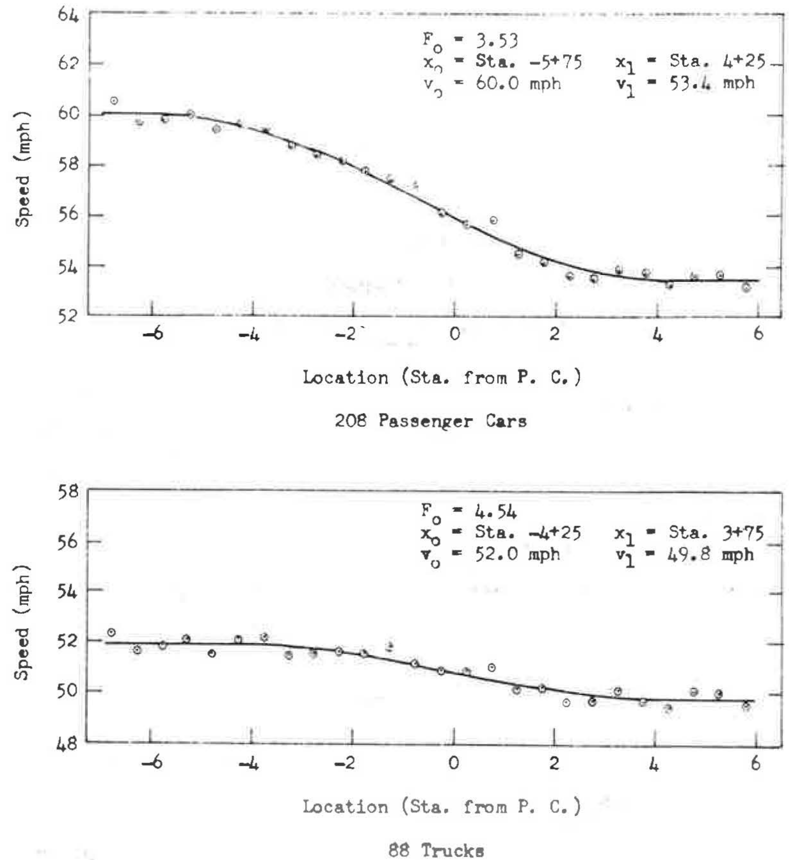


Figure 10. Speed-location graph for curve on US 24 near Reynolds, Ind.—all data sets.

PILOT STUDY

The first feature studied was a section of US 24 approximately one mile west of Reynolds, Indiana. The highway exhibits a $4\frac{1}{2}$-deg curve extending through a central angle of 45 deg. The curve is level throughout, exhibits no degree of superelevation and has no apparent change in road cross-section. The approach to the curve is a mile long straightaway with no perceivable grade; thus visibility is not restricted for the approaching driver. The section contains no intersecting roads or driveways except for farm entrances to the adjacent fields. No vehicle was seen entering, leaving, or stopping in or near the study section.

Data from this feature were collected at six different times to secure variations in vehicle performance under varying ambient conditions. The first set of data was taken when the roadway was partially covered with ice and snow; the second set, on a clear cold day with the road surface clear but snow on the adjoining fields; the third set, on a clear warm spring day at approximately noon; the fourth set, during late afternoon of the same day; the fifth set, during early morning hours of a weekday (drivers headed directly into the sun as they negotiated the curve); and the sixth set recorded the response of Sunday afternoon drivers. Figures 4 through 10 show the speed-location curves obtained from these data.

Visual inspection of the shape of the measured speed location curves indicated a striking similarity to the theoretical curve of Figure 3. Quantitative curve fitting of the observations to the theoretical curve (and hence an estimate of the modulus of geometric aspects, F_0) was obtained by an application of the principles of least squares. Various locations were assigned for $x = 0$ and $x = L$, the theoretical curve was calculated between these end points and the squares of the difference between the theoretical curve and the observed data were summed.

The theoretical curve which "bestfits" (Figs. 4-10) the observations was determined by selecting the minimum value of the sum of squared differences.

RESULTS

Fit of Theoretical Curve

The speed variability removed by the pressure potential theory is extremely high. The lowest statistical F-value (Table 1) for any of the data sets for the US 24 highway section is 8.9 (trucks of data set number 1) which is significant at the 0.005 level of probability; that is, only five times in a thousand would this amount of variability be removed by pure chance. The other data sets, for both passenger cars and trucks, indicate an even better fit of the theoretical curve to the observed data. Therefore, it must be concluded that all the data exhibit definite agreement with the advocated theory.

TABLE 1

STATISTICAL F VALUES FOR THE SIGNIFICANCE OF THE PRESSURE POTENTIAL THEORY

Data	Passenger Cars		Trucks	
	Stat. F	$(R^2)^*$	Stat. F	$(R^2)^*$
Set 1	68.5	(85.6)	8.9	(43.6)
Set 2	137.0	(92.2)	10.7	(48.2)
Set 3	245.0	(96.4)	214.4	(95.0)
Set 4	573.1	(98.0)	157.5	(93.0)
Set 5	622.0	(98.2)	41.8	(78.6)
Set 6	1016.0	(98.9)	28.6	(71.4)
All	949.1	(98.8)	131.6	(91.9)

*R^2 provides an estimate of the percentage of the variability removed by the pressure potential theory.

Speed Changes (v_0 - v_1)

The speed changes occurring on the approach to and within the feature produced a definite pattern. First, the variance within data sets by type of vehicle was statistically the same for all data sets. Second, the mean speed change for a vehicle type was statistically the same under all observed ambient conditions. Third, these changes followed a normal distribution. The interpretation of these observations indicates that a particular type of vehicle may tend to reduce speed the same amount at a specific geometric feature regardless of roadway and other conditions.

Modulus of Geometric Aspects (F_0)

Because F_0, as obtained from individual vehicles, is given as a function of the natural logarithm of the speed change and the speed change distribution was found to be normal, the distribution of F_0 for individual vehicles cannot be normal. Thus the study of speed changes reflects a study of the individual F_0 values under a required transformation to obtain a normal distribution. Therefore the individual F_0 distribution was not analyzed.

Although F_0 values as calculated from speeds and speed changes of individual vehicles do not represent the flow restriction, the F_0 as calculated from the average of several vehicle responses does provide a measure of flow restriction (as evidenced by the fit of the observed data to the theoretical curve). Based upon the fit of the theoretical curve to the average of all observations, the highway curve on US 24 has an F_0 of 3.53 for passenger cars and 4.54 for trucks.

To secure an estimate of the number of observations required to estimate F_0, it is necessary to consider the logarithmic equation, $F_0 = \ln(4v_0 - 2\Delta v) - \ln v$. With the conditions indicated for the geometric feature on US 24 (that is, average v_0 of 60 mph, average speed change of 6.36 mph, and a standard deviation for the speed change of 4.36 mph) 25 passenger cars will provide an estimate of F_0 within $\pm$ 0.6 at a 95 percent confidence level. For trucks (average v_0 of 52 mph, average speed change of 2.26 mph with a standard deviation of 3.04), 16 observations would produce an estimate within $\pm$ 0.65 at a 90 percent confidence level. (These limits for cars and trucks are not strictly symmetrical because the logarithm varies more with a unit change in Δv when Δv is small than when Δv is large.) It is recommended that a minimum of 25 passenger cars and 16 trucks be used to evaluate F_0 from the average observed values.

Summary and Recommendations

The modulus of geometric aspects provided a quantitative rating for the geometric feature of the pilot study when calculated from the average speeds of at least 25 passenger cars or at least 16 trucks. When working with individual vehicles, the speed change attributed to the geometric feature was a better representation of driver response to flow restriction.

On the basis of these findings, the study was extended to other geometric features in order to determine whether or not the proposed model—and its accompanying modulus of geometric aspects—may be applied in general to the geometric features of highways.

EXTENSION OF STUDY

Sites

On the basis of the findings of the pilot study, additional geometric aspects were selected for the extension of the study:

1. A $5\frac{1}{4}$-deg horizontal curve on US 41 (and 52) located approximately one mile north of Earl Park, Ind.
2. A right angle turn on Ind. 26 located approximately one mile east of Pine Village, Ind.
3. A transition section from 4-lanes to 2-lanes on US 52 north of Templeton, Ind.
4. A merging lane on the North River Road entrance to the William Henry Harrison Bridge in West Lafayette, Ind.
5. A narrow bridge on Ind. 43, located approximately two miles north of Chalmers, Ind.

Data Collection

From these sites data were recorded on days of comparatively good weather; that is, for all observations the road surface was clear and dry. Only one period of observation was used for each of the selected sites as it was believed that broader coverage should be obtained at the sacrifice of information for fewer sites under varying ambient conditions. In all cases, each vehicle for which data were recorded could choose its own speed—there was no vehicle immediately (10 seconds or less) preceding it in its particular traffic lane. However, where possible, the data were analyzed on the basis of encountering (or not encountering) vehicles traveling in the opposite lane. If an oncoming vehicle was met within the critical part of the feature, the data were classified as the opposing lane occupied. Otherwise the opposing lane was considered free of oncoming traffic.

Curve on US 41 (and 52)

Description.—The curve is on a heavily traveled section of US 41 and is the location of frequent accidents—some of which are severe. Both of the approaches are downgrade to the curve and visibility is not restricted. The pavement has been widened for the inside (northbound) lane. There is a minimum amount of superelevation through the facility. The approaches to the feature are marked by flashing amber caution lights and large signs proclaiming a dangerous curve ahead. Within the curve there is a minor county road intersecting the highway on the outside edge. During the period of observation no vehicle was seen using this minor facility. Data were recorded for vehicles traveling south (the outside lane).

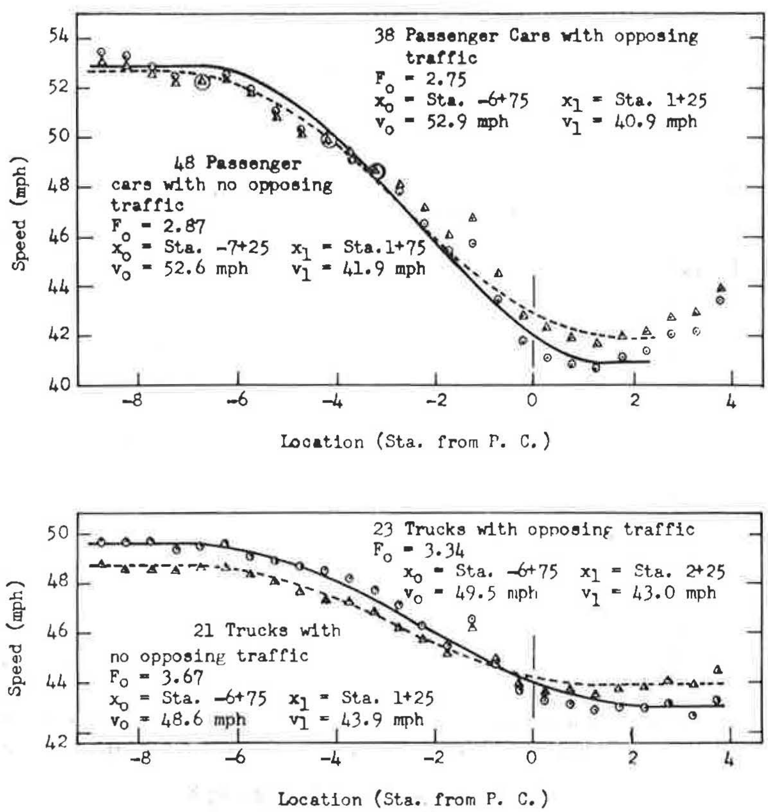


Figure 11. Speed-location graph for curve on US 41 (and 52) near Earl Park, Ind.

Results.—The difference between the two best-fit theoretical curves (Fig. 11) is not significant. Therefore, there is no evidence of a difference in driver response resulting from opposing traffic and all data may be considered as part of the population; that is, the pressure of oncoming cars produces no significant change in driver response to the highway feature.

The speed variability removed by the advocated theory is again extremely high, indicating an excellent fit between observation and theory. The distribution of speed changes plots as a normal distribution.

The best estimate of the modulus of geometric aspects for this highway curve from all observations is 2.78 for passenger cars and 3.42 for trucks.

Right Angle Corner on Ind. 26

Description.—This feature is on a lightly traveled section of Ind. 26; however, it is the location of frequent accidents, most of which are limited to minor damage. The

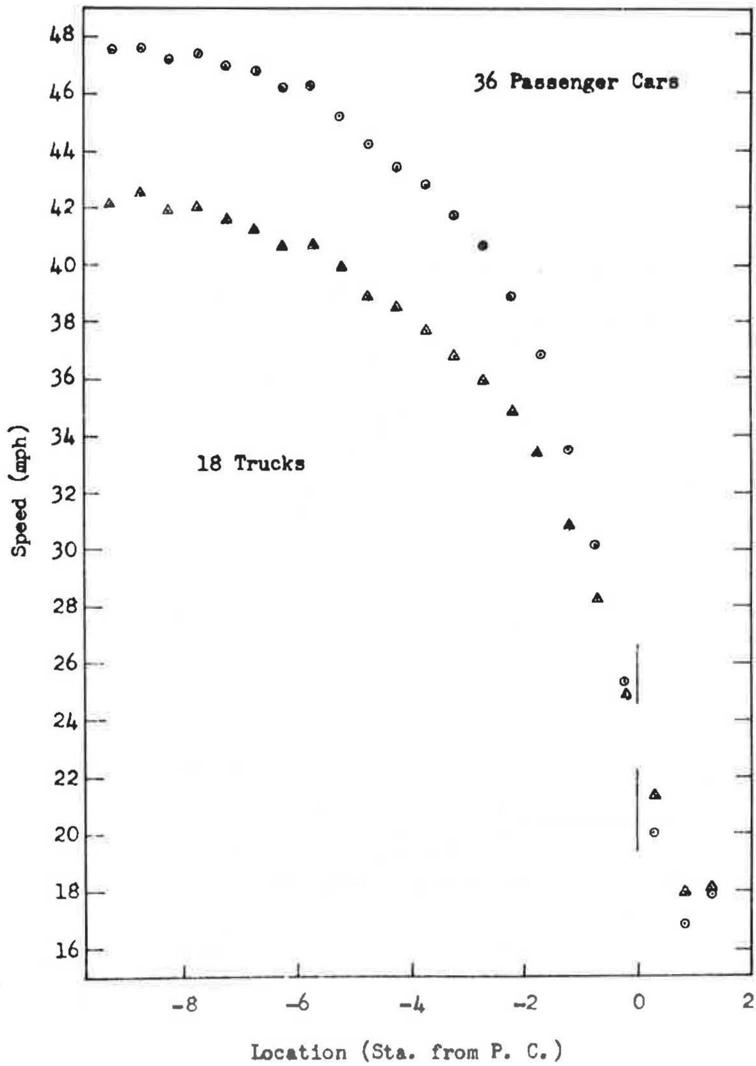


Figure 12. Speed-location graphs for corner on Ind. 26 near Pine Village, Ind.

approach to the turn is slightly downgrade and visibility is not restricted. There is considerable banking of the asphalt surface in the turn. Approaching drivers are informed of the conditions by means of a sharp turn sign only.

The roadway makes a complete 90-deg bend with an estimated radius of 42 ft for the westbound travel lane for which observations were recorded; thus, this condition is an example of an extreme rural highway curve.

Results.—The speed-location curve (Fig. 12) is not similar to the ones previously encountered in this study. Approaching vehicles begin to reduce speed approximately 1,000 to 1,200 ft prior to the turn—as in the other curves studied. However, in this case vehicles continue their deceleration at a more progressive rate as the feature is approached. When the minimum speed is reached (at approximately the center of the turn) the vehicles immediately undertake an acceleration. This type of response to the feature differs from the pressure potential theory previously discussed.

Matson et al. (2) show a comparable speed-location relationship which has been computed for the approaches to stop signs from a study by Beaky (3). The plotted observations for the subject curve on Ind. 26 resemble this condition more closely than the previous features in this study. Consequently this curve would be classified as a severe condition; that is, it requires an extremely large change of speed to permit vehicles to traverse it safely.

The shape of the speed-location curve may be attributed to this severity of operation. As the driver approaches the turn, he becomes aware of the increased resistance and subsequently allows his vehicle to decrease speed. As the turn becomes closer, the potential changes, and the driver must apply the vehicle's brakes to reduce speed more rapidly. Thus, at least two moduli of geometric aspects are required to be able to describe the response. An indication of this may be viewed in the similarity of the

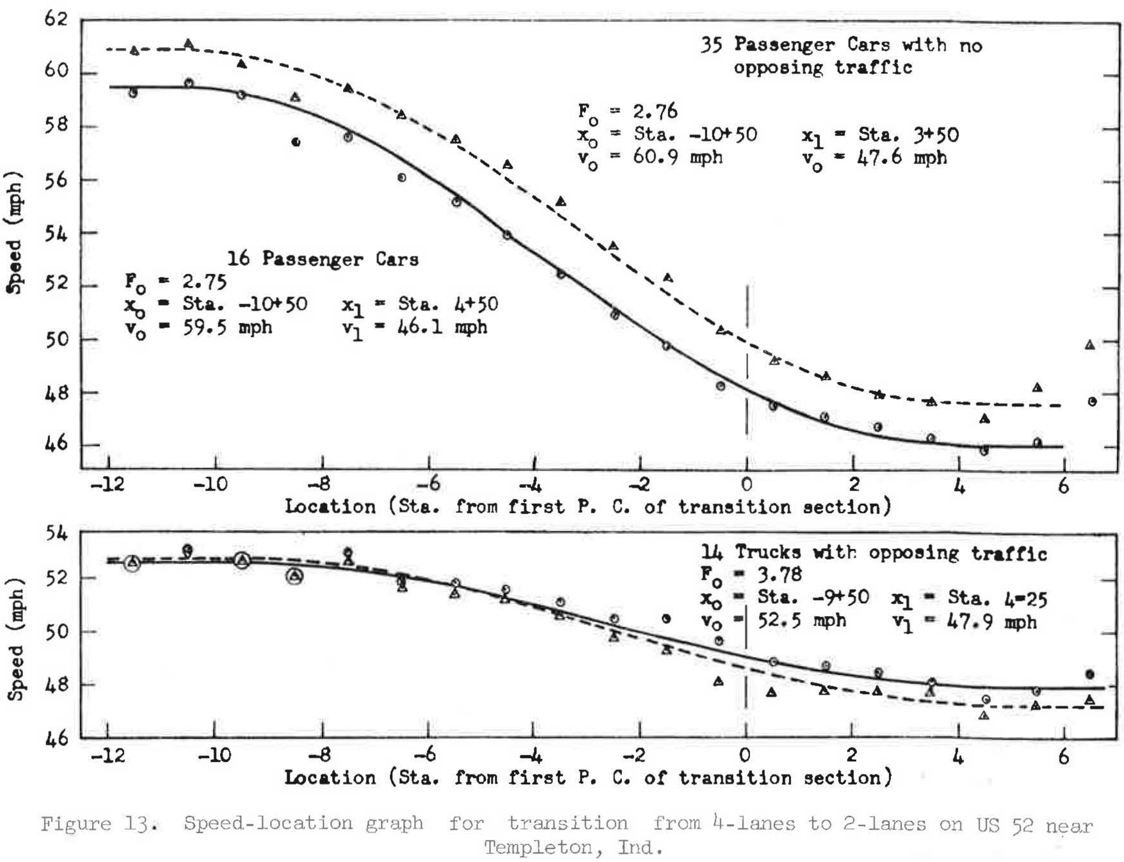
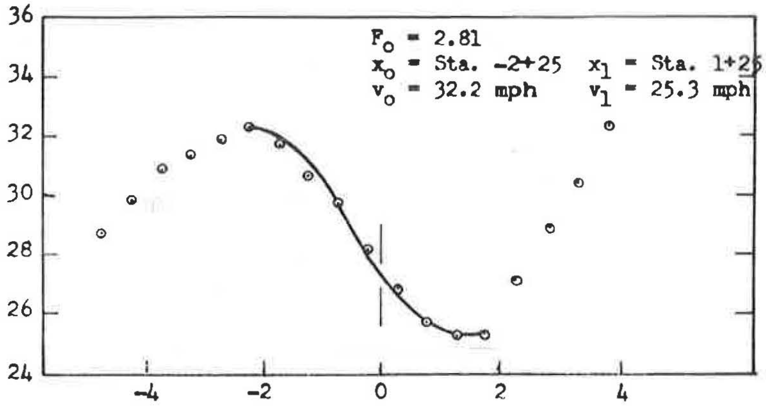
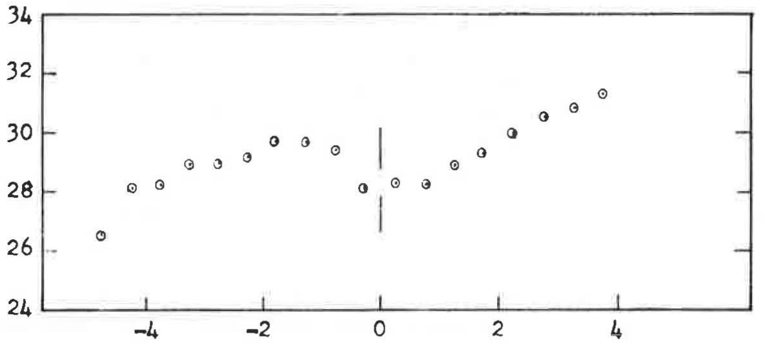


Figure 13. Speed-location graph for transition from 4-lanes to 2-lanes on US 52 near Templeton, Ind.



Location (Sta. from start of merging lane)

42 Passenger Cars



Location (Sta. from start of merging lane)

5 Trucks

Figure 14. Speed-location graph for merging section of approach to Wm. H. Harrison Bridge in West Lafayette, Ind.

first 500 ft of deceleration shown by the observed data for all features. Although the initial portion of the observed relationship is comparatively uniform, the remaining section exhibits extreme differences.

The detailed study of how to handle this particular response was beyond the scope of this study and was left for future investigations into the pressure potential theory.

US 52, 4-Lane to 2-Lane

Description.—The feature under consideration is a moderately heavily traveled section of US 52. For the vehicles observed the feature consists of a straight, level approach on the divided 4-lane pavement, a curve to the left, a short straight section, and a curve to the right which brings traffic onto the 2-lane roadway. Yellow paint has been applied to the pavement on the approach to encourage traffic to merge into a single lane prior to the transition. Warning and directional signs (for the transition) are located approximately 1,000 ft before the actual feature. There is only a minor degree of superelevation evident on the curved portion.

Results.—Although the speed-distance charts (Fig. 13) indicated some difference between the speeds of vehicles encountering oncoming traffic and those not encountering such traffic, this difference was found to be statistically insignificant.

The speed variability removed by the theoretical curve is extremely high and an excellent fit is evident both visually and by statistical F tests. The speed changes for passenger cars were normal; however, the speed changes for trucks were found to be skewed towards zero.

The best estimate of F_0 for this feature at the time of test was 2.76 for passenger cars and 3.69 for trucks.

Approach to Wm. H. Harrison Bridge

Description.—An investigation of the suitability of the modulus of geometric aspects was undertaken for the merging section of the North River Road approach to the Wm. H. Harrison Bridge. Observations were recorded of vehicles on the straight ramp approach, through the merging area, and onto the bridge itself.

Results.—Although the entire speed-location graph (Fig. 14) is rather complex, it was easily broken into individual parts for study. The speed-location graph for trucks is shown, but no conclusions were drawn from these data because only 5 trucks were observed.

The theoretical curve again closely approximates the observed speeds for the merging section, thus indicating an excellent fit of the theoretical curve and the observed data. Here also the speed change distribution plots as a straight line on probability paper. The modulus of geometric aspects for the merging area was found to be 2.81 from the observed data.

Ind. 43 near Chalmers

Description.—The bridge is an open truss bridge with an interior width of 22 ft 4 in. which is the same as the width of pavement on the approach to the facility. The approach is level and straight. Visibility is not restricted. A sign warning of a narrow bridge is located about 600 ft before the bridge.

Results.—There was only a minor speed change observed on the approach to the narrow bridge. This small speed reduction was statistically insignificant and thus the other calculations would have little basis for meaning. The only estimate of F_0 would be a high value indicating no resistance offered by this highway feature.

CONCLUSIONS

The pressure potential theory provides an excellent theoretical model for explaining the variations observed in speeds on the approach to and through most geometric highway features. It does not apply to geometric features requiring very severe speed changes.

The modulus of geometric aspects provides a reproducible quantitative rating of the ease of traffic flow through all geometric features except those requiring very severe speed changes.

RECOMMENDATIONS

The success of the modulus of geometric aspects as a quantitative rating of the highway features considered in this study suggests that the following additional investigations be undertaken:

1. A systematic tabulation of the modulus of geometric aspects for all pertinent highway features should be initiated. It is hoped that a spectrum of these values would (a) allow design decisions to be made in a quantitative manner; and (b) provide a basis for simulation approaches to traffic flow.

2. The relationship between the modulus of geometric aspects for cars and that for trucks should be explored in an attempt to obtain an equivalency between these vehicle types.

3. The theory should be amplified to include the compound modulus of geometric aspects as revealed in highway features requiring severe speed changes.

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Optimum Width for Widening Secondary Arterial Streets

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(A Condensation)

Committee D-A2, Geometric Highway Design, considers that there is general need for research data on vehicle operations on connector and secondary streets of various widths. Reliable data in substantial quantity would be helpful to policy-making agencies in establishing design criteria or standards for these streets. The report, as here condensed, is sponsored by the committee and describes a method for obtaining data for this purpose through field research. A summary presentation was made at the January 1964 meeting of the committee, after which this condensed form of publication was arranged both to emphasize the subject for research and to point out an easily used method.

This report describes a method for easily determining speeds and placement on streets, with parking on one or both sides, and outlines criteria for determining effective street width. The sample of data was too limited to warrant general conclusions but the report serves a very useful purpose by suggesting the types of data that should be obtained and by describing simple procedures for collecting and analyzing the information. The committee urges additional research studies of this type.

THE PROBLEM

The street pattern of many of the residential areas of American cities was laid out 30 to 50 years ago, when traffic volumes were much lower than today. In many instances, streets laid out with 50- to 60-ft right-of-way are now being used as collectors or secondary arterial streets. Many of these streets have a pavement width of only 20 to 30 ft on right-of-way which has numerous shade trees. When improving these secondary arterial streets, questions have arisen as to policies on street widening so as to provide adequate traffic service and still preserve the quality of the residential neighborhood.

According to the AASHO Urban Policy, it is recommended that at least two moving lanes be provided in outlying or residential areas, with a continuous parking lane on each side for a total pavement width of 40 to 44 ft. Table E-1 of this policy states in principle that two moving lanes with parking on one side only should not be considered.

This study was made for the purpose of investigating the operating characteristics of secondary arterial streets 23 to 32 ft wide, when operated with parking on one side only. It was reasoned that if a secondary arterial street will operate satisfactorily with parking on one side, and if restricting parking to one side will adequately take care of the parking demand, then perhaps some secondary arterial streets might be improved by widening to some width less than 40 to 44 ft.

Field studies were made of operating characteristics of secondary arterial streets of several different widths in Evanston, Ill. Factors studied included the following:

1. Lateral placement of moving vehicles as they approached and passed vehicles parked on one side of the arterial streets;

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2. Speeds of these vehicles traveling in both directions when approaching and passing the parked cars; and
3. The change in speed resulting from two vehicles about to meet each other at a point abreast of the parked car. On the narrower pavements, one driver may wait for the other to proceed, especially when one of the vehicles is a bus, or a truck.

PROCEDURE

A time-lapse movie camera, operating at 300 frames per minute was used to collect data at each of seven locations. At each test location, target boards 8 ft long were placed temporarily on both sides of the street at three transverse stations to provide the scale for lateral placement. The three stations were the center of the parked car and 44 ft ahead and behind that station.

After the target boards were photographed, the boards were removed, and the camera was operated each time a vehicle approached the test location. An average of approximately 8 seconds of exposure was used for each car.

LOCATIONS

An attempt was made to find sections of the selected locations where all conditions were comparable except for street width. It was not possible to get completely comparable conditions—there was some variation in cross slope of the pavement, parking conditions varied somewhat in the parking lane, and some of the streets studied were not arterial streets. Data are reported here for two locations, both in a section where the street serves as a secondary arterial street with a bus line which has about eight buses per off-peak hour. Parking was on one side only, with an average of only one parked car per 100 ft.

DATA COLLECTION

Data were taken off the films for speed and for lateral position, as follows:

1. The vehicle placement, defined as the horizontal distance from the right curb of the pavement to the outside edge of the right wheel of the moving car was determined for each direction of travel.
2. The meeting clearance, or the horizontal distance between the bodies of meeting vehicles, was computed.

Comparative data were tabulated for vehicles in the three following groups:

1. "Free-Moving" vehicles—those that passed or met the parked car at least 6 sec after any preceding vehicle traveling in the same direction, and at least 5 sec after and 10 sec prior to any vehicle traveling in the opposite direction.
2. "Meeting" vehicles—those that were spaced more than 6 sec behind any other vehicle traveling in the same direction and either had met, or were to meet, a vehicle traveling in the opposite direction within 1.5 sec of the time the vehicle passed the parked car.
3. "Other" vehicles.

ANALYSIS

Consider that two vehicles, A and B, moving in opposite directions meet opposite parked vehicle C. With the latter at the right, X is the distance from the curb at left to vehicle A (the oncoming vehicle) and Y is the distance from the curb at the right to vehicle B (the vehicle adjacent to the parked vehicle). Average lateral placement values (X and Y) for moving vehicles passing a parked car at a 23-ft section street and at a 29-ft section are given in Table 1.

For the 23-ft section it is apparent that meeting vehicles pass the parked car at lateral placements quite different from the free-moving vehicles. In four of twelve meeting maneuvers, one vehicle waited for the other to pass the parked vehicle.

TABLE 1

AVERAGE LATERAL PLACEMENT

Lateral Dimension	Free Moving	Meeting
(a) 23-Ft Street		
X	4.3	1.9
Y	11.2	9.0
W - (X + Y)	7.5	12.1
Avg. speed, mph	26.7	22.8
(b) 29-Ft Street		
X	4.3	3.0
Y	11.9	10.8
W - (X + Y)	12.8	15.2
Avg. speed, mph	29.7	27.0

In contrast, average lateral placement values at a 29-ft section of the same street changed only 1.1 to 1.3 ft because of the meeting maneuver. No vehicles during the field studies refused to meet another vehicle opposite the parked vehicle.

Table 1 also shows values of W - (X + Y) for the two widths of streets. The average width of pavement used for both directions of free-moving vehicles at the 23-ft section was only 7.5 feet, whereas this path width was 12.8 ft for the 29-ft section. Lateral placement for vehicles on the 29-ft section was not affected nearly as much by a meeting maneuver as on the 23-ft section.

Table 1 also indicates that average speeds are affected somewhat less by a meeting maneuver on a 29-ft pavement than on a 23-ft pavement.

Possible criteria for selecting optimum width are as follows:

1. All drivers "accept" a meeting maneuver (or 95 percent accept).
2. Most drivers keep same lateral position, regardless of whether free-moving or meeting.
3. Travel speeds are not substantially lower, as compared with a wider street.
4. Other constraints: snow removal problems, standing of delivery vehicles, demand versus supply of curb space.

More data are needed for applying these criteria with positive assurance. However, these two cases do indicate that a width of 29 ft was adequate for all observed drivers to accept a meeting maneuver opposite a parked car, whereas some drivers would not accept a meeting maneuver in the 23-ft section.

The second criterion (drivers holding the same lateral placement regardless of whether free moving or meeting) was not entirely satisfied by a 29-ft width. Extrapolation indicates a 31-ft width as optimum.

The third criterion, dealing with changes in travel speeds, needs more data for application.

More cases should be analyzed to permit a more positive determination of optimum widths of secondary arterial streets with parking on one side. However, this preliminary study indicates that the optimum width is in the 28- to 32-ft range for passenger vehicles.

The Design of Highway Interchanges: An Example of a General Method for Analyzing Engineering Design Problems

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MARVIN L. MANHEIM, Department of Civil Engineering, Massachusetts Institute of Technology

This interim report describes the application of a set-theoretic decision method to the problem of highway interchange design. The paper is one of a continuing series reporting the development of this approach to the analysis of engineering design problems.

A design problem can be described by a list of misfits, or ways in which the design can be "wrong," and by a list of the pair-wise interactions among the misfits. Description in this form makes it possible to associate a linear graph with every problem. A set-theoretic criterion is used to obtain a hierarchical decomposition of the graph. The result is a program for developing a solution to the design problem.

This paper introduces the method, and describes the shortcomings of the AASHO manual as a design program. It also describes the preparation of the interchange problem for analysis and discusses the results of a preliminary analysis of the problem. A program for the design of a highway interchange is presented and some partial solution attempts are described.

•THERE is no systematic approach to problems of engineering design. Present methods of design are largely ad hoc, they are based on trial and error, and they are influenced more by the forms which have been found as solutions to old problems, than by the specific nature of the problem in hand.

These methods are inefficient and expensive. The physical forms they lead to seem comparatively economical as far as the cost of their design goes. But they are actually much more expensive than they need be because their imperfections lead to long-range diseconomies.

ANALYSIS OF A HIGHWAY INTERCHANGE DESIGN PROBLEM

If we look at the number of possible types of interchange available to a practicing engineer today, we see what looks at first like a rather rich variety (1). If we look a little closer though, and think about these designs, we realize that to a large extent they are really nothing but variations on one basic idea. In fact, when the engineer faces the task of designing interchanges he is under tremendous cognitive constraints, in the sense that it is very hard indeed to escape from these habitual patterns or stereotypes, without being deliberately and willfully "different."

However, once we realize just to what extent the engineer is constrained by nothing better than habit, it occurs to us that possibly these interchanges are not even particularly well suited to the task they perform. Even if they were suitable at the time of

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their invention, they may be the hung-over formalism of a past task, like the horseless carriage, whose shape changed only long after the horse dropped off.

To find out whether this is in fact so, we must invent some way of explicitly getting around our cognitive bias, something which allows explorations unfettered by the conventionally accepted forms; but at the same time, something not based, like "brainstorms," on wild association, but related in a sensible and significant way to the problem at hand. Present methods lead to imperfect designs, because they do not take the structure of the design problem into account.

When confronted by a complex design problem, we automatically try to reduce its complexity by organizing it according to certain invented categories. Thus for instance, in the case of a highway design problem we begin by saying that one must consider economic factors, safety factors, and aesthetic factors. This assumes, tacitly, that the structure of the problem is such as to permit this subdivision, and that it makes some sort of sense to think about economics, safety, and aesthetics separately. But note that this subdivision has been generated purely by the words which happen to be at our disposal, not by the specific structure of the problem which confronts us.

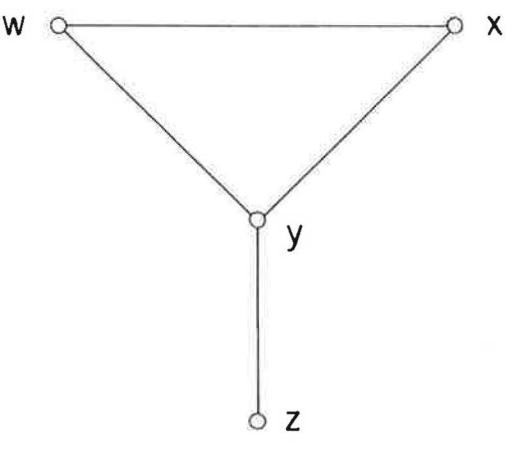
Designers do not deliberately disregard problem structure. But the way in which they naturally think out problems forces them to think in terms of categories, and whether such categories are derived from past experience, or self-consciously invented, they are still governed by the verbal labels now available in English. There is no reason to suppose that these are appropriate to the problem in hand.

Instead of superimposing irrelevant or misleading verbal labels on it, we have to find some way of letting the problem generate its own structure. To do this we must abstract from all the issues which crop up in a design problem some representation which we shall call the structure of the problem. Once we have this structure, it can be manipulated to gain insight into the problem itself, to improve the organization of the process of design itself.

Graph

The structure we shall use to represent problems is a special kind of topological complex known as a graph. It is uniquely defined by two sets, V and L. V is a set of points, or vertices; L is a set of unordered point-pairs, or undirected links. (Of course L must be a subset of the set of all possible links, $v \times V$.)

Thus, if the set V contains the points (w, x, y, z), L might contain (wx), (wy), (xy), (yz), and V and L would then define the graph:



Such a formal structure, simple though it is, allows us to represent the essential features of any design problem, no matter how complex.

Every problem of designing a physical form has these two fundamental characteristics:

1. There are certain requirements which the form must meet.
2. Many of these requirements conflict with one another.

The objective of any design process is to find a form which manages to meet all the requirements, in spite of the conflicts. We call the process of inventing any such form "solving the design problem."

Given any list of requirements and a list of the conflicts between pairs of requirements, we can associate with every requirement a vertex, and with each interaction between requirements a link between the corresponding pair of vertices. The graph so defined completely summarizes the structure and the difficulties of the problem. The whole of the analysis which follows is based on this correspondence between design problems and abstract graphs.

If the defining requirements were all independent, then any problem could always be solved, no matter how many requirements it contained, because we could specify the physical characteristics of the form which each separate requirement demanded, and be sure that these characteristics would not conflict. The solution to the problem could then be found by simply aggregating the relevant physical characteristics.

Of course, this usually cannot be done, because the desirable physical characteristics are to some extent mutually exclusive, so we have to compromise. It is just wherever this happens that we speak of a conflict between requirements.

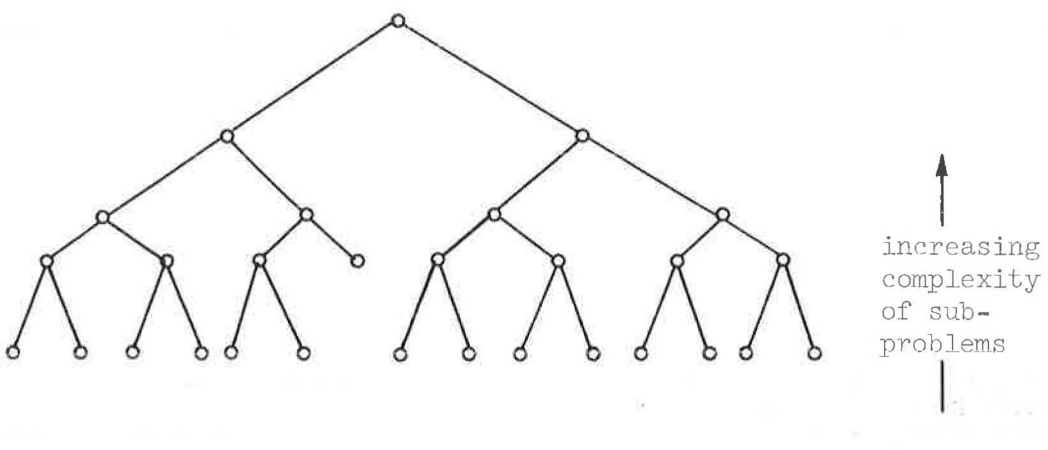
When there are such conflicts, the conflicting requirements must be juggled until we find a satisfactory solution. This is exactly what the designer does during the course of a day's work. But under circumstances where the requirements can no longer simply be solved one by one, the number of requirements and requirement interactions to be dealt with simultaneously begins to be critical.

In our opinion, it is just the cognitive limits on the number of interactions a designer can manipulate, which make the unaided designer obsolete in the face of the large complexes of requirements he meets today.

If this is so, the purpose of any analysis must be to bring the design task within cognitive limits generally available to all designers. This calls for careful control of the conflicts and requirements they try to deal with at any one time.

To achieve this kind of control, we have first to outline a systematic procedure which can take the place of the present confused process (called design), and we shall call such a systematic procedure a design program.

A kind of program which is known to be effective in solving complex problems is one which first solves simpler subsidiary problems (2, ch. 6; 3; 4). By solving simple subproblems, combining their solutions to form solutions to more complicated ones, and so on, hierarchically, we can buildup a solution to problems of great complexity:



Program

To construct such a tree or sequence we must first ask whether we actually know any simpler problems we can solve. The answer is clear. Any problem can be made easier by leaving out some of the requirements it contains. The more requirements left out, the easier the problem is. Any subset of the set of requirements is therefore a simpler problem.

Ideally, we would like to break the problem up into a number of smaller sets of requirements which constitute simpler problems, find a solution for each of these small sets of requirements independently, and then somehow "add" the solutions to one another.

The objection to this procedure is clear. While the small sets of requirements are themselves indeed simpler to solve, the problem of integrating the solutions to them is so great that it defeats the whole purpose of the procedure. Small sets of requirements are not in general independent of one another, but interact just as single requirements do. If there is interaction between the sets of requirements to begin with, the various subsidiary solutions will also interfere with one another when we try to aggregate them.

This difficulty is brought out very clearly in the graph chosen to represent the problem. If the points of G are divided into two mutually exclusive sets, the requirements represented by these sets will not in general be independent: there will be many links between the two sets, and the interactions these links stand for make the aggregation of two independently found solutions very difficult. In fact, the only way of escaping this difficulty altogether would be to find some way of dividing the points into two sets such that no point of either set is linked to any point of the other. In this case there would be no interaction between the two sets of requirements, we could solve each set independently, and we should then be free to combine these independently found solutions as we wished.

A solution to a set of requirements is a specification of certain aspects of form. It may specify quantifiable aspects of the form (such as road width and concrete thickness) or nonquantifiable geometrical aspects which can be represented diagrammatically. For every solution to the full set of requirements, there is a large class of forms which satisfy any subset of that set. In solving such a subset we specify just those aspects of the forms which uniquely identify this class.

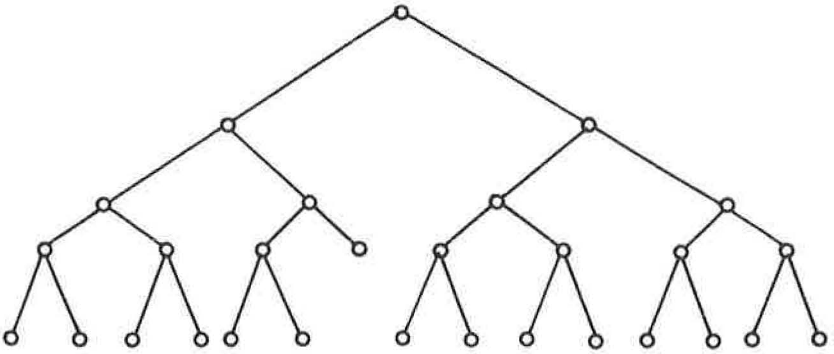
Thus, in finding a solution to any subset of requirements, we determine only certain aspects of the physical form—we do not determine it completely. When we find solutions to two such subsets, we have two different sets of specifications for aspects of the form—one set of specifications per set of requirements. If the two sets of requirements are completely independent, then the two corresponding solutions do not conflict; i.e., if two sets of specifications for the aspects of a form are derived from nonconflicting requirements, the specifications (the solutions) are themselves nonconflicting.

This means that these two sets of specifications may be combined without contradictions; or put another way, there exist forms which satisfy both sets of specifications, and these forms naturally satisfy both sets of requirements. Thus, if the solution to one set of requirements specifies the road width, for example, the solution to the other set, if it is independent, either does not specify any road width, or if it does, it specifies some road width compatible with the specification of the first solution. Under these circumstances, the two solutions can always be combined.

In practice it turns out that almost no problem can be divided into two such completely independent parts. That is, every practical problem must be represented by a connected graph, whose set of points cannot be divided into two subsets which are not linked to one another. In this case the best we can do is to find some way of dividing the set into subsets which are as independent as we can make them.

To do this we use a measure of independence, to evaluate each possible way of subdividing the problem (2, Appendix 2). Then, where there are n requirements, and so 2^{n-1} possible ways of dividing the set of requirements in two, we pick that way for which this measure of independence is greatest. The two sets which result from this subdivision are both simpler than the original problem, and because they are defined according to their independence, it will be relatively easy to combine their solutions.

Once we are able to divide a problem into its most independent parts, we can repeat the procedure, always operating on each of the two subsets derived from the previous operation. This process of subdivision, if used iteratively, allows us to break the original set into a number of distinct sets as small as we like. We can indicate this by a tree, in which each subset stands above the most independent subsets it in turn contains.



This tree is our design program. If we start at the lowest level of the tree, solving subproblems which contain only a few requirements, we can proceed to solve problems higher and higher in the tree. Because the pairs of subproblems we take together are always as independent as they can be, we can solve all the subproblems in turn, and then finally the original problem itself, provided we do so in the order prescribed by the tree.

Solution

Of course, in a sense, this is what the designer does already. He looks first at the component parts of a problem, then at larger and larger components, until finally he manages to achieve a synthesis which encompasses everything the problem demands. But such hierarchical programs can only work successfully if all the component problems at any level are independent of one another, and can therefore be put together without interference. It is only under these conditions that the designer can proceed systematically from one level of the hierarchy to the next.

The difference between what we propose and what usually happens is that we derive our component problems from the problem graph's structure in such a way that those components at any one level of the program are independent of one another and therefore capable of integration, while as a rule the designer selects the problem's components according to rather arbitrary conceptual schemas.

This can be made clear by referring to the specific case of the highway interchange problem which we have chosen to analyze. In this case, the schemas most often used by design engineers are those found in the AASHO manuals (5, 6). Of course, the AASHO manual (5) makes no claims to be a program. However, it does offer a conceptual framework within which to solve the problem of highway interchange design, since it is an organized statement of the issues which have to be considered, grouped under various headings and subheadings (Fig. 1).

We find that, in spite of the non-programmatic intention of the AASHO manual (5), it in fact does contain a hierarchy (Fig. 2) that is superficially very like the kind of hierarchical program we have in mind. The major design issues are laid out for consideration according to a conceptual framework which is intended, in virtue of its organization, to help the engineer overcome the cognitive limits which otherwise restrict his design ability.

Although we doubt the adequacy of this schema as a program, it is perfectly possible that it might be useful in designing highway interchanges. The chief trouble is that as it stands we can have no confidence in it, because it is too confused conceptually. We wish to know whether its organization is appropriate to the problems it is intended to solve. To do this we at least have to be able to state explicitly just what organization the schema has; that is, we have to be able to describe the arrangement of its elements and the relations between those elements. Although the AASHO schema has a hierarchical structure, it contains elements of many different logical types. Because its elements are not clearly classified we have no way of knowing just what the relations between them are. In other words, we have no way of estimating just what organization the manual has, and therefore no way of deciding whether it is appropriate to the problems in hand.

In brief, to decide whether a program is a valuable one, we must first see it in terms of elements which are classified so that their interrelationships are clear. Then we can determine what the organization of the program is, and then whether or not it is relevant to the structure of the design problem.

We have already pointed to two kinds of element which play an important part in engineering decisions: requirements, and the interactions between requirements. Of course, these two elements alone will not describe all the ramifications of design problems. But we suggest that everything about an engineering design process can be expressed in terms of five elements: the two already mentioned, and performance standards, data, and solution.

Contents

Chapter 1

INTRODUCTION AND SUMMARY OF PRINCIPAL DESIGN CONTROLS AND GUIDE VALUES

- Introduction
- Summary

Chapter 2

DESIGN CONTROLS AND CRITERIA

- Topography and Physical Features
- Traffic
  - Volume
    - Average Daily Traffic
    - Peak-Hour Traffic
  - Directional Distribution
  - Composition
  - Projection of Traffic
    - Current Traffic-Existing and Attracted
    - Traffic Increase
  - Design Traffic Data
- Vehicle Characteristics and Design Vehicles
  - Passenger Cars
  - Single-Unit Trucks and Buses
  - Truck Combinations
  - Height of Motor Vehicles
  - Trends in Motor Vehicle Size
  - Summary: Design Vehicles
- Speed
  - Design Speed
  - Running Speed
- Capacity
  - Uninterrupted Flow
    - Design and Possible Capacity
    - Two-Lane Highways
    - Three-Lane Highways
    - Multilane Highways
    - Capacity in Terms of ADT
- Capacity at Intersections
  - At-Grade-Unsignalized
  - At-Grade-Signalized
  - At-Grade-Channelized
  - Interchanges-Ramps and Ramp Terminals
  - Weaving Sections
- Design Capacity Versus Design Volume
- Safety
- Road User Benefit Analyses
- System and Classifications
- Design Designation
- References

Chapter 3

ELEMENTS OF DESIGN

etc.

Figure 1. Table of contents of an AASHO manual, A Policy on Geometric Design of Rural Highways.

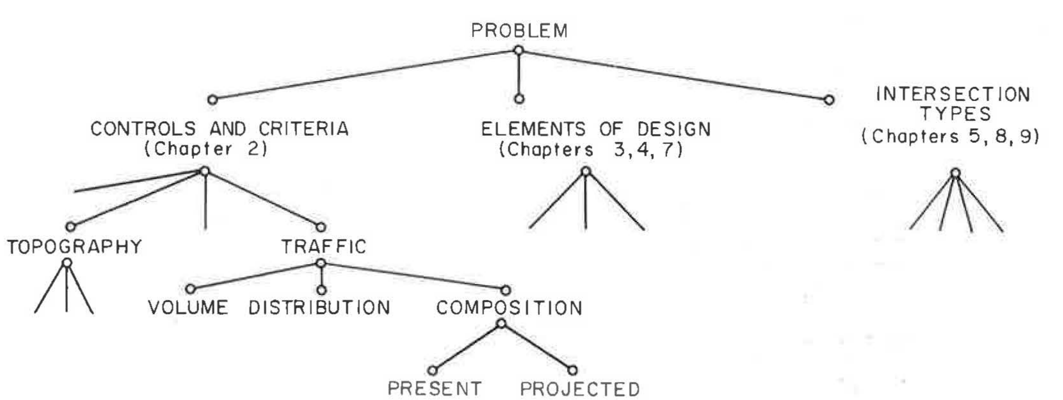


Figure 2. Hierarchical structure of an AASHO manual.

These elements are all discernible in everyday engineering practice, and in particular in the pages of the AASHO manual. But as things are at present, these five kinds of element are not clearly distinguished, there is no clear pattern to their arrangement, and the relations between them are not consistently expressed.

We have shown a part of the table of contents diagrammatically (Fig. 2) to bring out its hierarchical structure. But the elements of the hierarchy are very confused. Some of them, like SAFETY and UNINTERRUPTED FLOW, express requirements which the interchange must satisfy. Some elements, like VEHICLE CHARACTERISTICS, give information about the interaction, or possibility of conflict, between certain requirements. (For instance, VEHICLE CHARACTERISTICS tells us about the interaction of the two requirements, "grade not excessive" and "adequate vertical sight distance.") Other elements, like TOPOGRAPHY AND PHYSICAL FEATURES, refer to data; others (e.g., SPEED) refer to performance standards, and others, like DESIGNS WITH THREE INTERSECTION LEGS, are solutions.

The schema of the AASHO manual contains all five types of element, but treats them all alike, as though they were of the same logical type. But they are not—requirements affect the design process in a manner different from the effect of data, etc. Each kind of element plays a different role in the process of design.

DRAINAGE CHANNELS AND SIDE SLOPES

Modern highway design recognizes the desirability of the "streamlined" cross section. Safety, good appearance, economy in maintenance - and frequently in construction - are the direct benefits of flat side slopes, broad drainage channels, and liberal warping and rounding. These features avoid obsolescence and invite favorable public reaction.

Drainage Channels

Where terrain permits, roadside drainage channels built in earth** should have side slopes not steeper than 4:1 (horizontal to vertical), and a rounded bottom at least 4 feet wide. Minimum depth should vary from about 1 foot in regions of low rainfall intensity to about 3 feet in regions with substantial rainfall intensity. Channel depths of 2 feet or more usually are needed to keep the roadway above ground level. Drainage channels of these dimensions have the advantages given in the following paragraphs. More critical dimensions may be necessary because of terrain or limited right-of-way.

There is reasonable safety from overturning to the motorist who accidentally drives into a flat sloped drainage channel. Slopes of 3:1 are less advantageous than 4:1 slopes in this regard but are satisfactory when economics or lack of width dictate their use. In open country of northern regions, slopes of 5:1 and 6:1 are preferable in that they do not cause snowdrifts as do the steeper slopes.

The broad flat drainage channel provides a sense of openness that is of material benefit in relieving driver tension. With a slope of 4:1 from a 10-foot shoulder (or possibly 3:1, depending upon the shoulder slope and superelevation), the whole of the drainage channel is visible to the driver from a normal position on the traveled way. This lessens the feeling of restriction and adds measurably to the driver's willingness to utilize the area in emergencies.....

**footnote omitted in quotation.

Figure 3. Excerpt from AASHO manual, A Policy of Geometric Design (pp. 209-210).

Not only are these different kinds of element confused at this broadest level of analysis, but the confusion also extends to the most detailed discussion in the text (Fig. 3). In a few sentences elements of all five kinds are discussed, without any explicit reference to the fact that they are being treated differently.

For instance, the first paragraph mentions requirements ("safety, good appearance, economy in maintenance") and what seems to be a solution: "'streamlined' cross section" consisting of "flat side slopes, broad drainage channels, and liberal warping and rounding." The solution is made more detailed in the next paragraph by specification of "side slopes not steeper than 4:1 . . . and a rounded bottom at least 4 feet wide," but confused by data: "where terrain permits . . . channels built in earth." In the third paragraph, we find that what we thought was a solution becomes a performance standard qualifying a requirement: in other words, "slopes of 3:1" is now treated as a standard for deciding whether the requirement "reasonable safety from overturning" is met. In the same paragraph, there is conflict between the two requirements, "economics" and "lack of width." Nowhere in the passage is there any clear distinction between the different parts these five elements are to play in the designer's decisions, nor are the single elements themselves treated from any consistent point of view.

Although our five-part classification of elements is based on engineering practice, it could be argued that it is an arbitrary invention. Undoubtedly the authors of the AASHO manual did not have these particular elements in mind when they wrote the text, and there is no reason why they should have had. We do not condemn them for failing to use just the kinds of elements we happen to have chosen. Our objection to the manual is that it looks as though its authors did not have any clearly distinguished kinds of elements in mind when they developed it, which makes its organization meaningless.

An example of the use of the most important two elements, "requirements" and "requirement interactions," in an analysis of the problem of highway interchange design follows.

PROCEDURE IN ANALYZING THE PROBLEM

Before discussing interchange design at all, we must be able to say how we would decide that a particular design is good or bad. A simple way of deciding is with a checklist of performance standards associated with the requirements. However, it is not always possible to see every aspect of a design problem in terms of quantitative performance standards; there is a danger that the checklist fails to account for many requirements which we know to be important, but cannot quantify.

For this reason the checklist which follows is not simply a list of measurable standards, but a list of all aspects of an interchange which could be said to have "gone wrong." To make this list of requirements comprehensive, it contains such items as "lack of consistency in signs," for which there is no quantifiable standard, as well as items like "traffic lanes too narrow" which can very easily have quantities associated with them. In this fashion each item on the list describes some way in which a design can fail to meet our approval.

Each item refers to a way in which an actual design can fail to fit its specification: these "misfits" are the requirements which we must satisfy.

This list of requirements is our third. Briefly, the procedure was as follows: We made up a list of words which suggested problems in the design of interchanges (e.g., "safety," "construction costs," and "snow removal"). Then we refined this list, trying to make it as complete as possible, and trying to get as little overlap between requirements as we could. Accordingly, requirements were not listed as separate, if they referred to substantially the same physical aspects of the form. And then we recombined them to provide as parsimonious a description of the problem as possible. For example, at one state we had these three:

a. Erosion of the side slopes excessive,
b. Water and water-borne debris are carried on to roadway, and
c. Stones can roll off side slopes on to roadway.

These three issues refer essentially to two aspects of the interchange: the resistance of the side slopes to erosion, and the protection of the roadway from water, stones and debris. Therefore, the new list of misfits includes the two items 64 and 66 instead of the three a, b, c.

In addition to the list of requirements, we also need to identify the interactions between them. When we say that there is a link between misfits 46 and 89, we mean that decisions which we make about the form of the intersection when we try to satisfy only misfit 46 will probably affect those we make when we consider misfit 89 as well. We call the link "negative" if the attempt to satisfy one conflicts with the attempt to satisfy the other. We call the link "positive" if the two complement each other. (It is assumed that links are not directional, i.e., if 46 is linked to 89, then 89 is linked to 46.) In the analysis, it is not necessary to distinguish positive interactions from negative interactions—both are represented by unsigned links.

It was decided not to attempt a definitive enumeration of the links in this first preliminary analysis. Since there are 112 misfits in the final list, the number of possible links is 112 x 111 ÷ 2, or about 6,000. Allowing any significant amount of time (even five minutes) to decide whether or not a link exists between each pair of misfits would have taken too long. Therefore, we decided to make the link decisions very quickly.

We sat at an IBM card punch, reading down the list of misfits, and punched the data deck directly as each link decision was made. Each link decision was made twice, once for each side of the diagonal in the 112 x 112 matrix corresponding to the set of possible links. The computer program used (7) corrected the data to produce a symmetric matrix, by eliminating all those links which were not defined twice, i.e., from A to B and vice-versa.

The average time taken for each link decision was (20 hours)/(12,000 decisions), or about 5 seconds per decision. The solution, or tree, which was obtained is all the more remarkable evidence of the power of the method, therefore, because it is thoroughly reasonable, in spite of the very rapid definition of links.

The listing of links which follows is this corrected version.

Misfits—List No. 3

1. Lanes too narrow.
2. Lanes too wide.
3. Vertical sight distance not sufficient.
4. Horizontal sight distance not sufficient.
5. Inadequate acceleration distance.
6. Insufficient lateral clearance to the right.
7. Inadequate deceleration distances.
8. Superelevation, radius, and design speed and material are not consistent with safety.
9. Excessive downhill slope on roads entering the intersection or within it.
10. Downhill slope too long.
11. Curves too long.
12. No waste of land; every square foot used to the maximum.
13. Not enough length of roadway to merge.
14. Not enough length of roadway for weaving when preparing to diverge.
15. Visibility at point of entry is dangerously low.
16. Insufficient illumination.
17. Lack of consistency in signs.
18. Drivers do not receive adequate information as to how they should proceed to destination.
19. Too much information for the driver to take in (number of signs or information per sign).
20. Distance between successive signs too small for adequate reactions.
21. Too much stimulation (advertising, flickering, lights, visual noise).
22. Driver does not know what other drivers in his own traffic stream are doing (movement patterns are inadequately defined and controlled).
23. Driver does not know what course of action he should take (movement patterns inadequately defined and controlled).

24. Driver does not know what drivers in other traffic streams are doing (movement patterns inadequately defined and controlled).
25. Movement paths are counter-intuitive (because they are not oriented towards their ultimate destinations and are misleading in their functions as signals).
26. Median acts as lateral obstruction (psychological crowding).
27. Median provides insufficient separation of traffic.
28. Excessive headlight glare from comfort and safety.
29. Sun in your eyes.
30. Too many driver actions required.
31. Too few driver actions required—monotony dangerous.
32. Lack of consistency in actions demanded of the driver.
33. Access for police and emergency vehicles to all possible accident and breakdown points is not assured.
34. Power line, emergency telephone, etc., connections are not accommodated.
35. No access to service facilities.
36. Paths for pedestrian movement through interchange area are not defined.
37. Pedestrians and animals are not safely separated from vehicles.
38. Vehicular access to adjoining land is upset by the interchange.
39. Through flow on existing roads is upset by the intersection.
40. Process of construction interferes with traffic over existing facilities.
41. Construction periods too long financially—capital tied up, and prices liable to change.
42. Design of intersection is not standardized and so does not allow the use of standard or prefabricated components.
43. Traffic flow gets interrupted by regular events (pedestrians, traffic signals, policemen, swivel bridges, unlimited access).
44. Speed of interchange inconsistent with highway in speeds.
45. Number of lanes in interchange inconsistent with number of lanes on highways.
46. Location and arrangement of interchange disturbs linear continuity of intersecting highways.
47. Bridge structures disturb linear continuity of the roadways.
48. No provision for emergency breakdown—so that a driver does not interrupt traffic flow.
49. No provision of service facilities—gas, food.
50. Peak-hour congestion unacceptable.
51. No parking provided for service area.
52. Travel time too great.
53. Interchange interferes with watercourses on surrounding land.
54. Transitions between paths of different horizontal curvature and too abrupt.
55. Change of radius is so gradual as to deceive the driver.
56. Water falling on to roadway cannot drain.
57. User costs (wear and tear on the vehicles, and gas and oil) are too high.
58. Total cost of material too high.
59. Total construction cost (man and equipment hours) too high.
60. Cost of land too high (including the cost of litigation, etc.).
61. No rest area provided.
62. Cost of design too high (specialized work: skew bridge design, etc.).
63. Design does not permit maximum use of facility under variable traffic conditions.
64. Wind, water, and gravity-borne debris can get onto the roadway.
65. Internal stability of earthwork sideslopes is not assured.
66. Surface stability (erosion, etc.) of earthwork sideslopes is not assured.
67. Cost of replacement and renewal (pavement materials, etc.) is too high.
68. Cost (to maintenance department) of clearing snow, vegetation, garbage, etc., is too high.
69. Operation of snow clearance itself interferes with the operation of the highway.
70. Snow removal is too slow and leaves highway obstructed for long periods.
71. Maintenance of paint and pavement markings (channel markings), vegetation control, etc., sign changes and re-painting obstruct traffic.

72. Time for which road is closed for pavement renewal is excessive.
73. Process of pavement renewal itself interferes too much with operation of the highway.
74. Original load-carrying capacity too low.
75. Wear and tear on structure due to weathering (heat, water, ice, cold, wind, etc.) reduces original load-carrying capacity too quickly.
76. Loads carried themselves reduce original load-carrying capacity too quickly.
77. Surface too rough for comfort.
78. Surface such that it retains an excessive film of water.
79. Material of surface too highly reflectant, causes sun and headlight glare.
80. Possibility of improvements in terms of changed design standards, technological innovations, etc., is hampered.
81. Expected life is too long for its role in regional development.
82. Facility does not meet demands of expected future use.
83. Boundaries for administrative responsibilities (maintenance, policing, liability, etc.) are not clear.
84. Too few visual stimuli along roadside to avoid monotony.
85. Path of road pavement ahead of vehicle does not hold drivers' attention.
86. Road path unrelated to the topography.
87. Road path unrelated to buildings and distant objects.
88. Destruction of existing trees, vegetation, top-soil.
89. No coordination (simultaneous or sequential) of horizontal and vertical movements.
90. Physical presence of interchange (or of some part of it) is objectionable to social, political, or cultural institutions.
91. Some specific movement not provided for.
92. Intersection would be a greater benefit to regional economics in a different location.
93. Too many high embankments.
94. Too many deep cuts.
95. Number of lanes does not accommodate desired traffic volume and composition.
96. Interchange fails to provide for bus-bus and bus-auto transfer, if buses occur in the traffic composition, and for connection to rail transit.
97. Maneuver areas do not accommodate desire volume at design speed.
98. Conflicting movements are possible.
99. Rate of change of grade too great—causes oscillation (2nd derivative).
100. Rate of change of superelevation too great—causes oscillation (2nd derivative).
101. Vertical clearance in underpasses is too little.
102. Inadequate ventilation of exhaust from semi-closed spaces—underpasses, deep cuts, etc.
103. Uphill grades are too long for trucks.
104. Snow drifts dangerously on to roadway.
105. Fog can build up—dips in roadway, valleys, etc.
106. Windborne smoke can interfere with driver vision.
107. Sudden changes in wind pressure are dangerous, especially at high speed.
108. High cross winds a nuisance.
109. No opportunities for vehicles to change order (for instance, as in passing).
110. Actions demanded of driver are not well articulated—no rhythm, driving not a structured pattern of action.
111. No way in which a driver can rectify a wrong turn.
112. Interchange does not have unique identifying character.

RESULTS OF ANALYSIS

The list of 112 misfits and the list of links (see Appendix) representing pair-wise interactions among the misfits described the structure of the interchange problem as a linear graph. This information was punched on IBM cards and used as input to a computer program, HIDECS 2, which was run on the IBM 709 at MIT and the IBM 7090 at the Smithsonian Observatory in Cambridge.

The result of each computer run was a hierarchical decomposition of the graph, which yielded an arrangement of the requirements into clusters in a tree (Fig. 4). As previously described, this tree can be used as a program for the design of a highway interchange. The tree which is presented here was obtained as output from these computer analyses.

It is important to note that the results of each of the three runs were identical as far as the third level of the hierarchy. In view of the well-known difficulties associated with hill-climbing analyses (7), this indicates a remarkable degree of stability. We take it as evidence that the structure of the problem really does have the character described by the analysis, in some very deep sense. The slight differences from run to run, which appeared in the lowest levels of the tree, were reconciled by hand. (See 7.)

In the following pages, a number of clusters of requirements, as identified by the analysis, are discussed. Each cluster has specific implications for the design of an interchange; these implications are presented graphically and are discussed in text. Unfortunately, we have not had time to make these diagrams in detail, or to work them out for all the sets of requirements, or to demonstrate the process of combination developed in the higher levels of the program. (See 8.)

THE TREE

The "Tree" (Fig. 4) is presented on pages 60-66. To reconstruct this diagram use two copies or trace pages 61, 63, and 65 and follow diagram of complete chart shown on page 66.

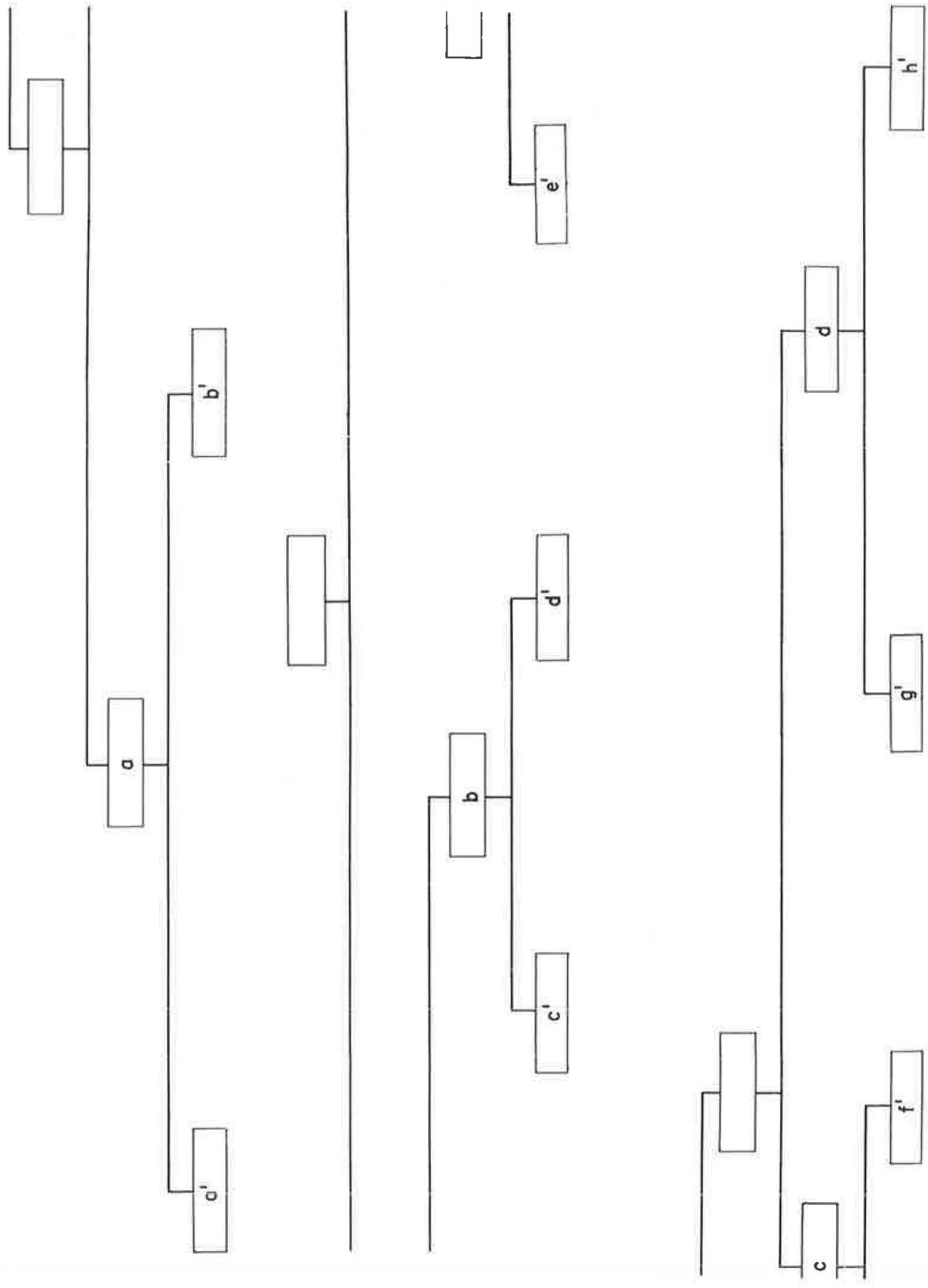
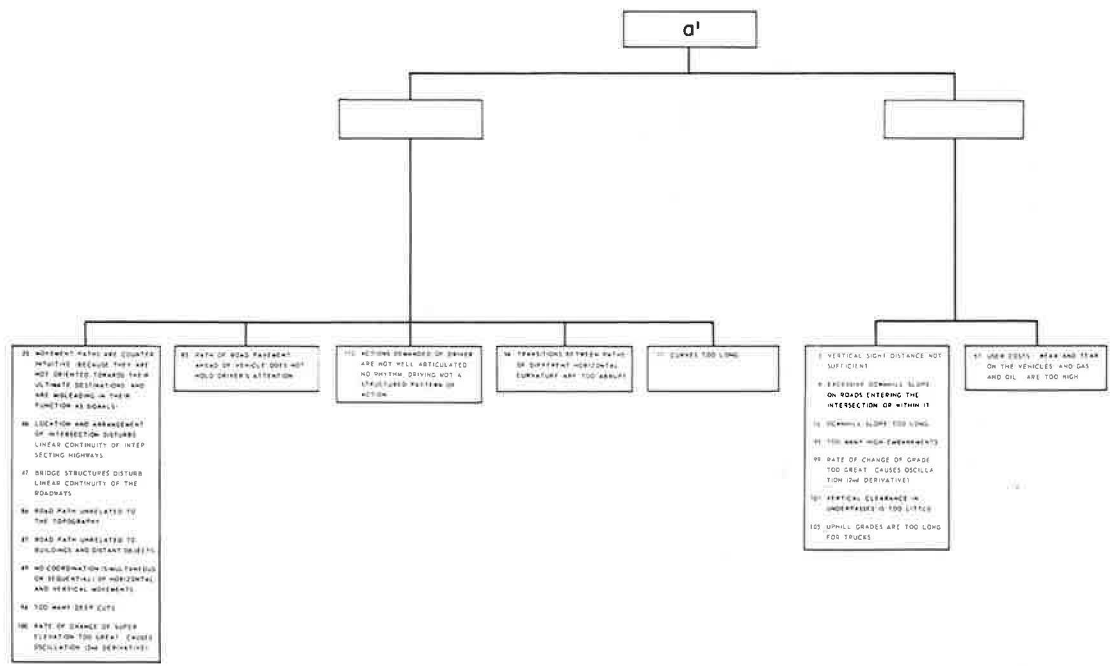
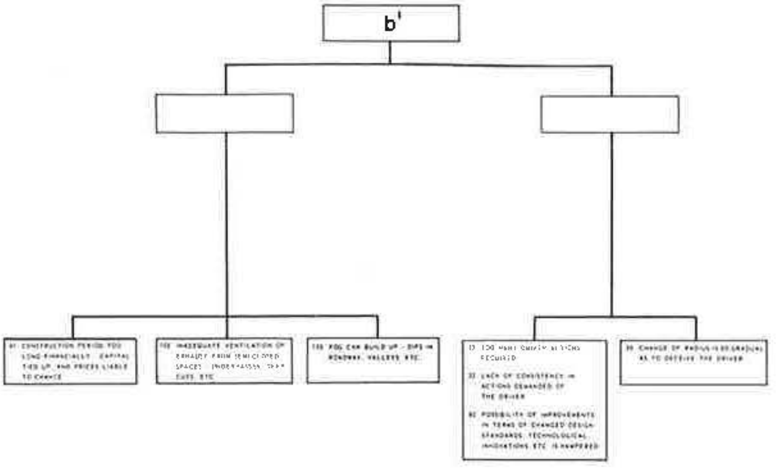
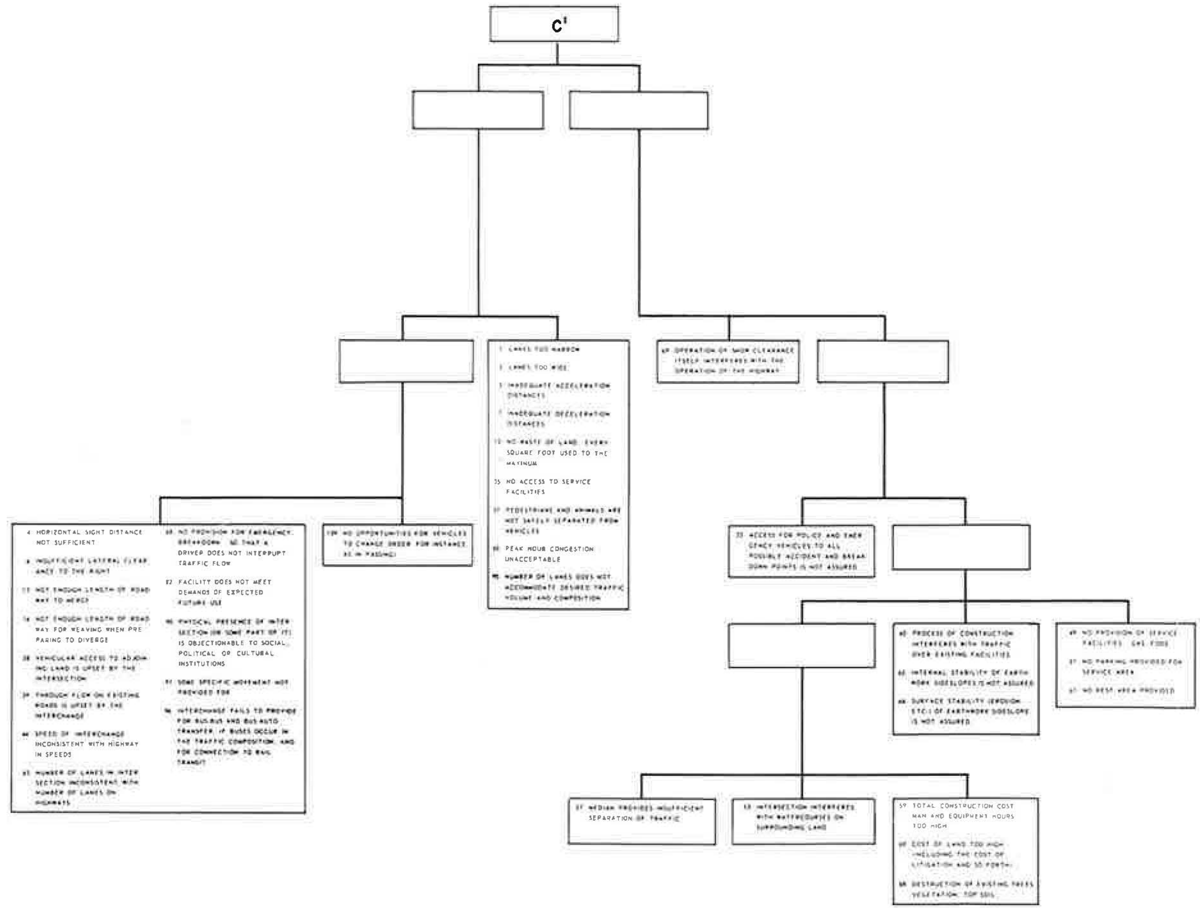
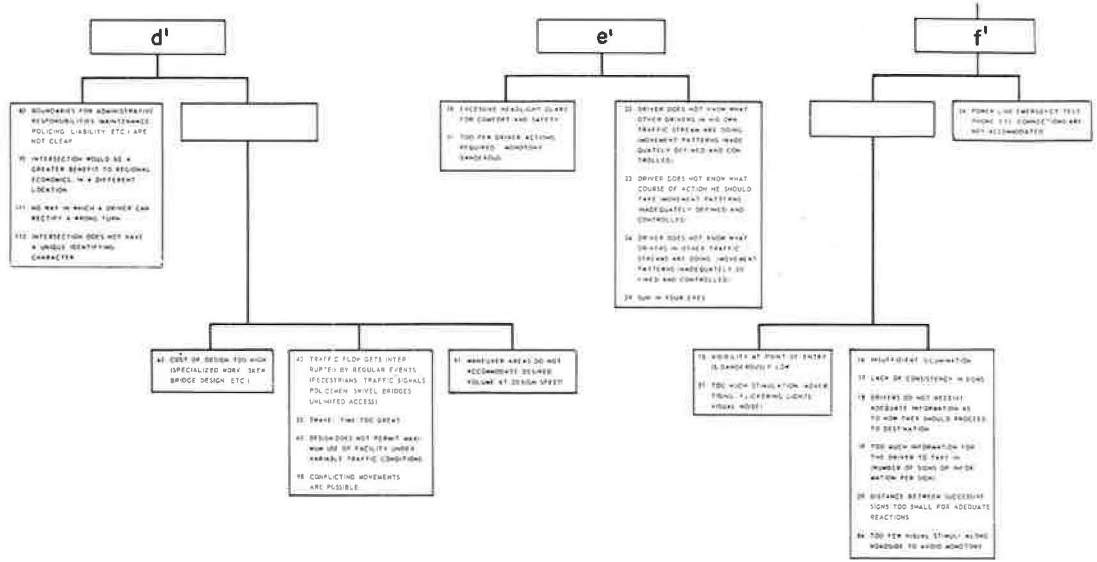


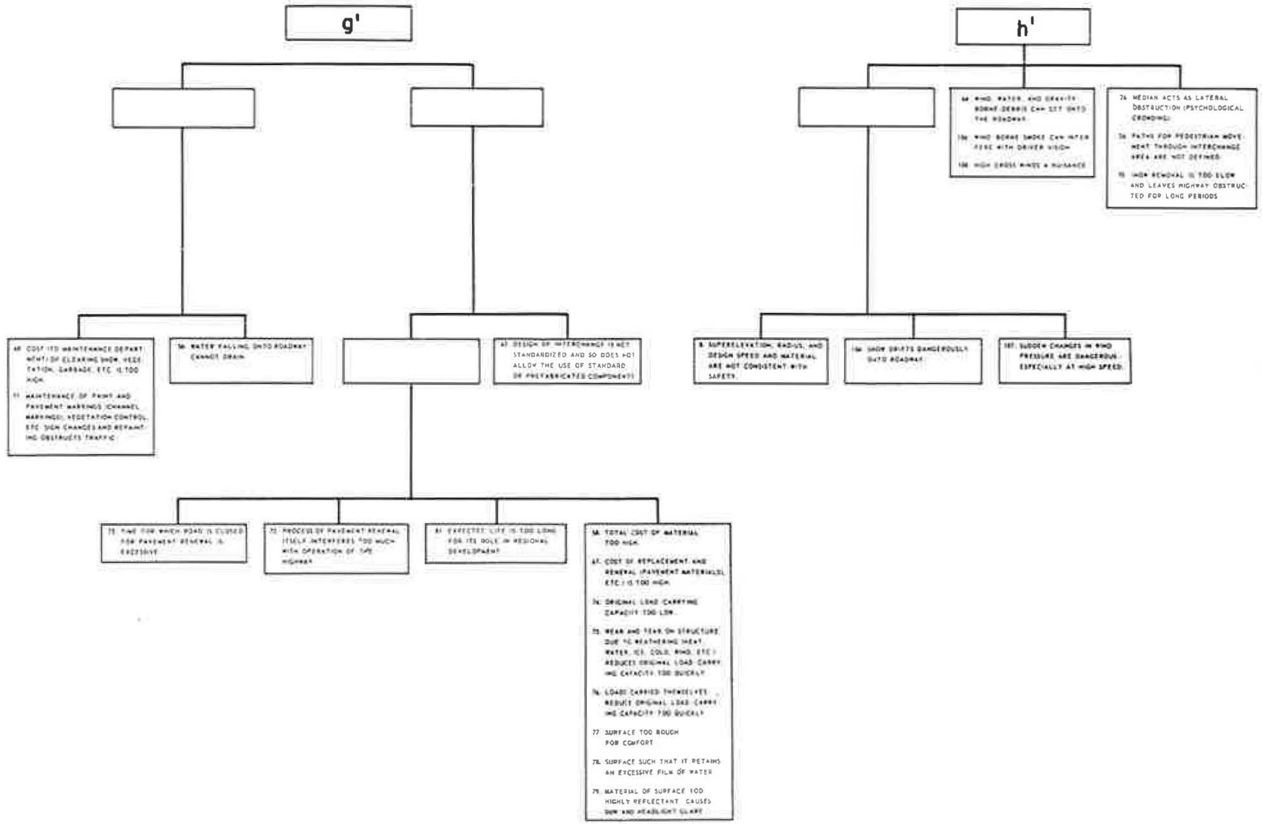
Figure 4. Structure of the highway interchange design problem.

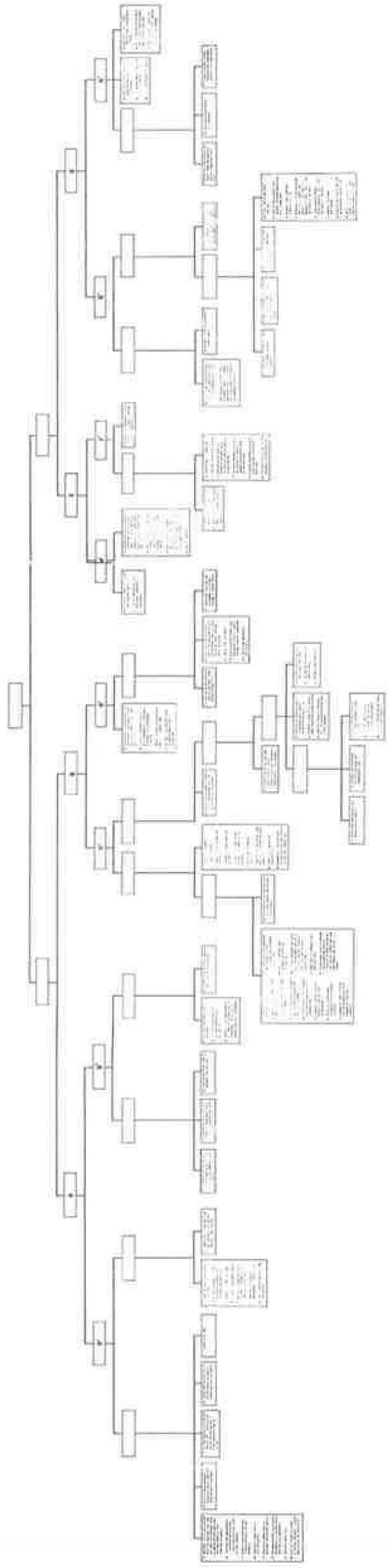

a'
85 PATH OF ROAD PAVEMENT AHEAD OF VEHICLE DOES NOT HOLD DRIVER'S ATTENTION
ARE NOT WELL ARTICULATED NO RHYTHM, DRIVING NOT A
46 LOCATION AND ARRANGEMENT OF INTERSECTION DISTURBS LINEAR CONTINUITY OF INTERSECTING HIGHWAYS
47 BRIDGE STRUCTURES DISTURB LINEAR CONTINUITY OF THE ROADWAYS
96 TOO MANY DEEP CUTS
CURVES TOO LONG
VERTICAL SIGHT DISTANCE NOT SUFFICIENT
ON ROADS ENTERING THE INTERSECTION OR WITHIN IT
99 RATE OF CHANGE OF GRADE TOO GREAT CAUSES OSCILLATION (2nd DERIVATIVE)
105 UPHILL GRADES ARE TOO LONG FOR TRUCKS
ON THE VEHICLES AND GAS AND OIL ARE TOO HIGH


b'
CONSTRUCTION PERIOD TOO LONG FINANCIALLY
TIED UP, AND PRICES LIABLE TO CHANGE
INADEQUATE VENTILATION OF
EXHAUST FROM SEMI-CLOSED SPACES
FOG CAN BUILD UP
VALLEYS, ETC.
TOO MANY
REQUIRED
LACK OF CONSISTENCY IN ACTIONS DEMANDED OF THE DRIVER
POSSIBILITY OF IMPROVEMENTS IN TERMS OF CHANGED DESIGN STANDARDS, TECHNOLOGICAL INNOVATIONS ETC. IS HAMPERED
CHANGE OF RADIUS
AS TO DECEIVE THE DRIVER


C'
4 HORIZONTAL SIGHT DISTANCE NOT SUFFICIENT
WAY FOR WEAVING WHEN PRE PARING TO DIVERGE
INCONSISTENT WITH HIGHWAY IN SPEEDS
DRIVER DOES NOT INTERRUPT TRAFFIC FLOW
FACILITY DOES NOT MEET DEMANDS OF EXPECTED
IS OBJECTIONABLE TO SOCIAL, POLITICAL OR CULTURAL INSTITUTIONS
SQUARE FOOT USED TO THE
35 NO ACCESS TO SERVICE FACILITIES
PEAK HOUR CONGESTION UNACCEPTABLE
SEPARATION OF TRAFFIC
59 TOTAL CONSTRUCTION COST MAN AND EQUIPMENT HOURS TOO HIGH

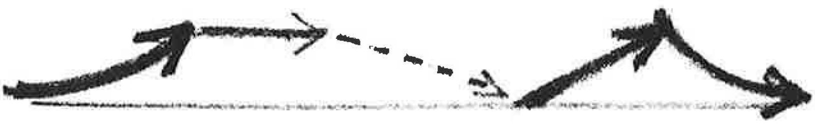

d'
e'
f'
POLICING LIABILITY ETC) ARE
NOT CLEAR
(SPECIALIZED WORK SKEW
BRIDGE DESIGN ETC)
43 TRAFFIC FLOW GETS INTER
RUPTED BY REGULAR EVENTS
(PEDESTRIANS TRAFFIC SIGNALS
POLICEMEN SWIVEL BRIDGES
UNLIMITED ACCESS)
98 CONFLICTING MOVEMENTS
ARE POSSIBLE
23 DRIVER DOES NOT KNOW WHAT
COURSE OF ACTION HE SHOULD
SIGNS TOO SMALL FOR ADEQUATE
REACTIONS


g'
h'
WATER FALLING ONTO ROADWAY CANNOT DRAIN
DESIGN OF INTERCHANGE IS NOT STANDARDIZED AND SO DOES NOT ALLOW THE USE OF STANDARD OR PREFABRICATED COMPONENTS
COST (TO MAINTENANCE DEPARTMENT) OF CLEARING SNOW, VEGETATION, GARBAGE, ETC. IS TOO HIGH.
MAINTENANCE OF PAINT AND PAVEMENT MARKINGS (CHANNEL MARKINGS), VEGETATION CONTROL, ETC. SIGN CHANGES AND REPAINTING OBSTRUCTS TRAFFIC
TIME FOR WHICH ROAD IS CLOSED FOR PAVEMENT RENEWAL IS EXCESSIVE
PROCESS OF PAVEMENT RENEWAL ITSELF INTERFERES TOO MUCH WITH OPERATION OF THE HIGHWAY
EXPECTED LIFE IS TOO LONG FOR ITS ROLE IN REGIONAL DEVELOPMENT
TOTAL COST OF MATERIAL TOO HIGH.
COST OF REPLACEMENT AND RENEWAL (PAVEMENT MATERIALS, ETC.) IS TOO HIGH.
ORIGINAL LOAD CARRYING CAPACITY TOO LOW.
WEAR AND TEAR ON STRUCTURE DUE TO WEATHERING (HEAT, WATER, ICE, COLD, WIND, ETC.) REDUCES ORIGINAL LOAD CARRYING CAPACITY TOO QUICKLY
LOADS CARRIED THEMSELVES REDUCE ORIGINAL LOAD CARRYING CAPACITY TOO QUICKLY
77 SURFACE TOO ROUGH FOR COMFORT
78 SURFACE SUCH THAT IT RETAINS AN EXCESSIVE FILM OF WATER
MATERIAL OF SURFACE TOO HIGHLY REFLECTANT, CAUSES SUN AND HEADLIGHT GLARE
WIND, WATER, AND GRAVITY BORNE DEBRIS CAN GET ONTO THE ROADWAY
WIND BORNE SMOKE CAN INTERFERE WITH DRIVER VISION
HIGH CROSS WINDS A NUISANCE
26 MEDIAN ACTS AS LATERAL OBSTRUCTION (PSYCHOLOGICAL CROWDING)
PATHS FOR PEDESTRIAN MOVEMENT THROUGH INTERCHANGE AREA ARE NOT DEFINED
SNOW REMOVAL IS TOO SLOW AND LEAVES HIGHWAY OBSTRUCTED FOR LONG PERIODS
8 SUPERELEVATION, RADIUS, AND DESIGN SPEED AND MATERIAL ARE NOT CONSISTENT WITH SAFETY.
SNOW DRIFTS DANGEROUSLY ONTO ROADWAY
SUDDEN CHANGES IN WIND PRESSURE ARE DANGEROUS, ESPECIALLY AT HIGH SPEED.



DIAGRAMS FOR SOME CLUSTERS OF REQUIREMENTS

Diagram A: Requirements 110, 136, 12.



Actions demanded of driver are not well articulated: no rhythm, driving not a structured path of action.

We identify five stages in changing roads in an interchange:

1. Preparation for exit from first major road,
2. Exit,
3. Transition—passage between major roads,
4. Preparation for entrance onto second major road, and
5. Entrance.

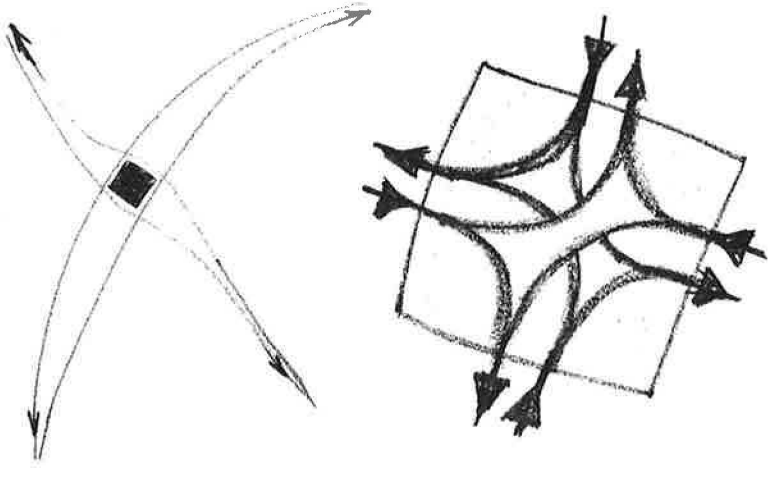
In each of these five stages, there are particular kinds of things which the driver should be doing:

1. Slowing down to prepare for turning off;
2. Controlled exit-slower speed than main road or transition road;
3. Transition roadway is fairly indeterminate as to speed and control;
4. Low relative speed, maximum control to allow driver adequate inspection of stream into which he is merging; and
5. Maximum buildup of speed to obtain zero relative speed for actual merge.

These stages have implications for the vertical profile of a path through the intersection which are expressed in Diagram A. The slowing-down required in the preparation stages 1 and 4 call for upgrades: the maximum buildup of speed required in stage 5 calls for a downgrade; the turning movement of stage 2 calls for a level roadway, and stage 5, the transition, is indeterminate insofar as grade is concerned.

In addition to grade, other physical devices might have been used to achieve this articulation: textures in the roadway (rumble strips), exaggerated superelevation, etc. However, manipulation of grade seems to be the most effective, and grade is a more basic feature of the interchange than the others.

Diagram B: Requirements 25, 46, 47, 86, 87, 89, 94, 100.



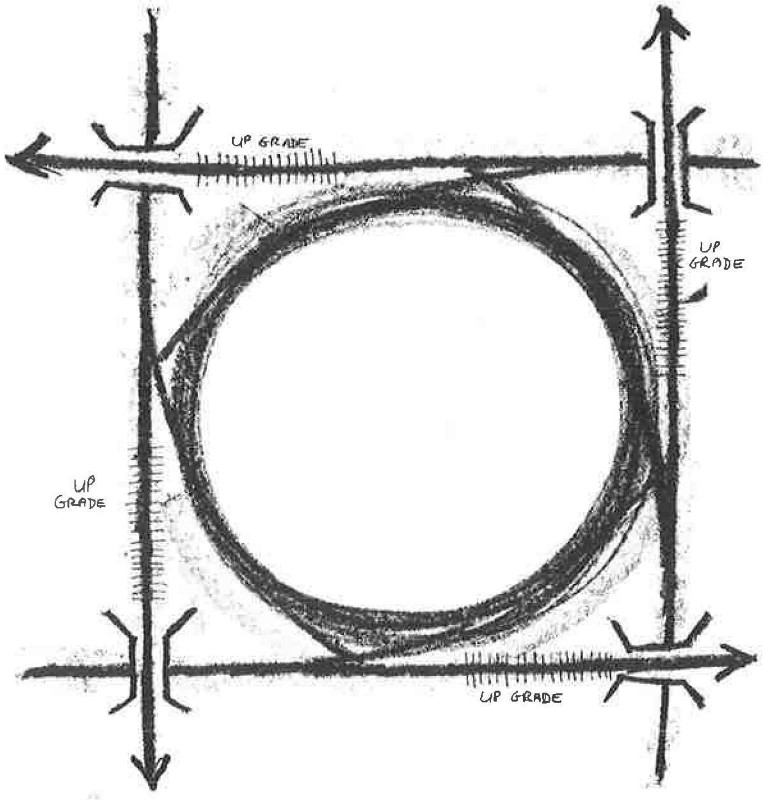
Requirements 46 and 89 suggest strong separation of that part of the interchange dealing with turning movements from the roadways which carry the main streams of traffic straight through the interchange (illustrated in the left-hand part of Diagram B).

Requirement 25 states that movement paths should not be counter-intuitive; this calls for designs in which movements to roadways heading left of the main roadway should be entered with left turns, straight-through movements should be made without major turns, etc. Together with 100, 25 also states that zig-zag movements should be avoided.

Requirement 46 requires that the occurrence of the intersection not cause the main roadways to deviate markedly from their general paths toward their ultimate destinations. This does not require dogmatic adherence to a straight path, only that the path have a general trend toward its ultimate destination, without zig-zags, as 25 suggests. It may even be a good idea for these paths to sweep clear of the mixing and turning functions, as illustrated in the left-hand diagram.

In the right-hand part of Diagram B, these mixing functions are shown in detail. Once the through paths are removed, this is made of two independent sections. If we interpret the diagram of the mixing functions as a bridge structure, we avoid extensive cuts (94). Although this may be expensive in its use of bridges, note that the requirement concerned with bridge costs (59) is not included in this set.

Diagram C: Requirements 3, 9, 10, 93, 99, 101, 103.

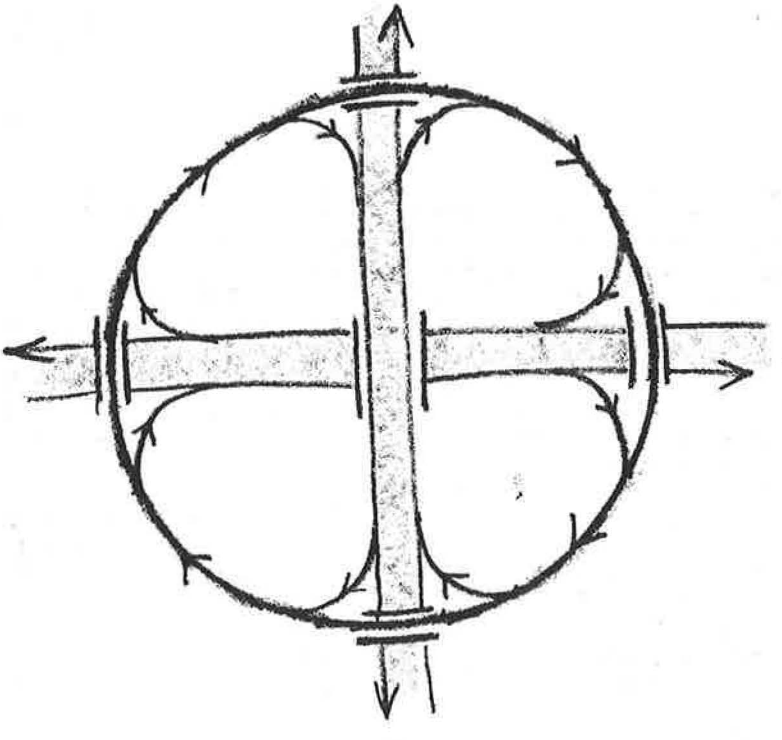


This group of requirements deals principally with grades. Ideally, it calls for a solution with no grades or bridges. Examples of such a form would be a rotary or an at-grade intersection. However, 57, user costs, calls for as little starting and stopping as possible, and makes these solutions unacceptable. Also, clearly, other requirements are going to call for bridges and grades, and make it important to study the less extreme implications of 3-103.

It is not imperative for the through roads to be level. It is the turning maneuvers which are especially dangerous if they contain grades; and 3-103, therefore, do call for an interchange in which all the turning takes place at one level. We solve this, in the Diagram C, by interlacing the four main roadways in "basketweave" fashion. This means that each roadway, as it comes out from under the first bridge, is "down." If the turning movements provided by the rotary all connect to this point, on the four roadways, respectively, the turning movements can all be at the same level.

The set of requirements has another interpretation: all these kinds of restrictions about grades, clearances, etc., call for a form which is not too "tightly packed." For instance, the same general form as the basketweave diagram shows might have this kind of flexibility, if the distances between the overpasses is of the order of 1,000 ft. Then, the grades and clearances of the main through roads can be adjusted independently over a fairly wide range, and the same form is adaptable to a variety of topographic conditions.

Diagram D: Requirements 4, 6, 13, 14, 38, 39, 44, 45, 48, 82, 90, 91, 96.



The number of requirements in this cluster is large (13). It is therefore interesting that a diagram was developed which satisfies almost all of these.

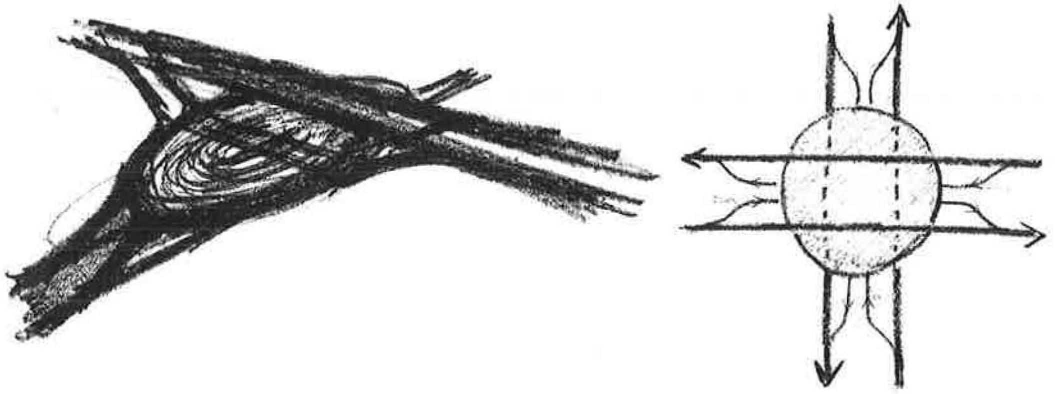
The single ring for all turning movements makes it easy to provide sufficient space on all roadways for emergency breakdowns (48), and for bus-auto transfer points (96). In this simple design, the fact that all turning movements use the same wide ring path makes less critical the possibility that the predicted volume for some particular movement may actually turn out to be very wrong (82). All movements can be provided for equally easily (91).

The design provides great flexibility with regard to grades. If the loop is elevated, instead of the through roads, accessibility to land uses within the loop can be preserved (38), and there can be a minimum disruption of through traffic on existing roads (39).

In this design, all exit movements are at opposite ends of the interchange from the point at which the roadway enters the interchange area. This allows the full crosswise length of the interchange to be used for acceleration and deceleration in merging and diverging movements. Similarly, although the ring volumes are high, the length of the ring allows adequate merging and diverging distances (13, 14).

The main through roadways have the same speed and number of lanes within the interchange as without, retaining their straight-through character (44, 45).

Diagram E: Requirements 1, 2, 5, 7, 12, 35, 37, 50, 95.

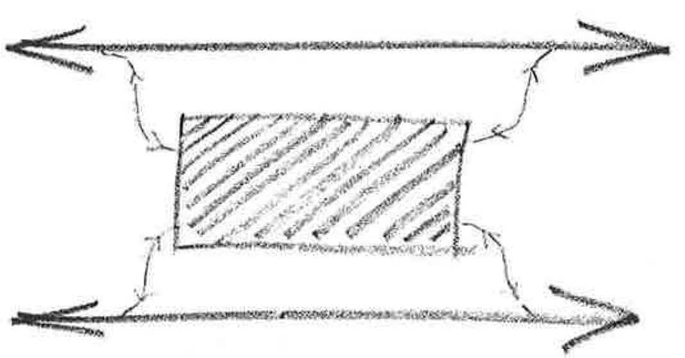


Requirement 12 calls for the maximum use of land—no waste. In making a diagram for this set we come to a point that has not, perhaps, been given sufficient attention in current highway design practice. If the speed of the individual highways is very great, we must expect to slow down in making the transition from one to another. We do not expect to transfer from a limousine to an airplane at high speed. Similarly, we feel that the transition from one highway to another should not necessarily be possible at high speed.

If we accept this premise, it becomes possible to reduce the amount of land used, drastically (Diagram E). A horizontal circular platform is built, above one roadway and below the other. All transition between the roadways comes straight to this platform, where flow stops almost dead. Circulation on the platform will probably be rotary; slow speeds allow a very tight radius.

This solution allows lanes of any desired width (1, 2); and provides ample acceleration and deceleration, since this can be tucked between the opposing paths of each roadway, where the median usually is (5, 7). Service facilities can be provided most simply in the middle of the transition zone; note that this is almost impossible on a cloverleaf (35). Congestion of this zone will only occur if opposing types of turn occur simultaneously (50). At peak hours this rarely happens; flow in one direction will predominate—and can be given priority by traffic-light control, etc., in the transition zone.

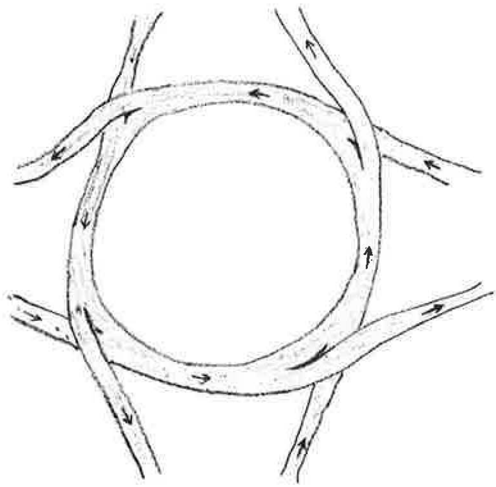
Diagram F: Requirements 83, 92, 111, 112.



Requirements 83, 111 and 112 call for a neutral zone, intermediate between the two main highways. This neutral zone will contain everything (including turning movements) which does not belong explicitly and functionally to one of the highways or the other. It will clarify administrative responsibility (83), and will give the interchange a substance capable of identifying it (111, 112).

The need for regional development (92) calls for an easy way of obtaining access to the interchange. Because of weaving distances, present designs such as cloverleafs make it very hard to provide access from the nearby land to the interchange itself. The neutral zone, however, allows it easily.

<u>Diagrams G and H:</u> Requirements 43, 52, 62, 63, 97, 98



Requirement 43 calls for steady flow without interruption. Requirements 63 and 97 call for a design which is homogeneous and continuous with the main roadways, and contains no narrow, tightly-curved connecting roads which are liable to bottleneck and slow maneuvering.

Diagram G is conceived on the basis of a single centrifugal flow, in which entering movements come in to the center, and all weaving is outward, away from the center. This satisfies 98, though not ideally, since it still makes for weaving which contains conflicts; 62 is not resolved. It is worth observing that it occurs just here, where skew bridges suggest themselves naturally.

A much better solution, derived from this one, is based on the need to avoid conflicting movements completely (98). Conflicting movements occur at places where merging precedes diverging. To avoid it, we must insure that diverging always precedes merging. The use of this principle leads to Diagram H. For the sake of clarity, it is shown without the right-hand turns, which are easily added.

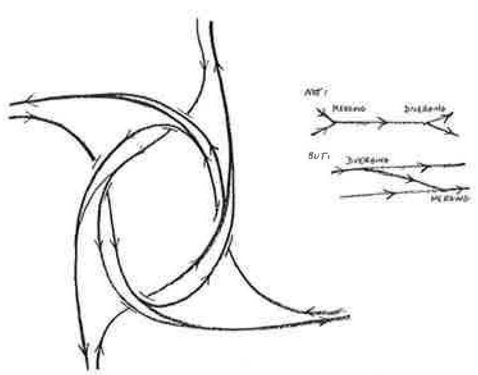
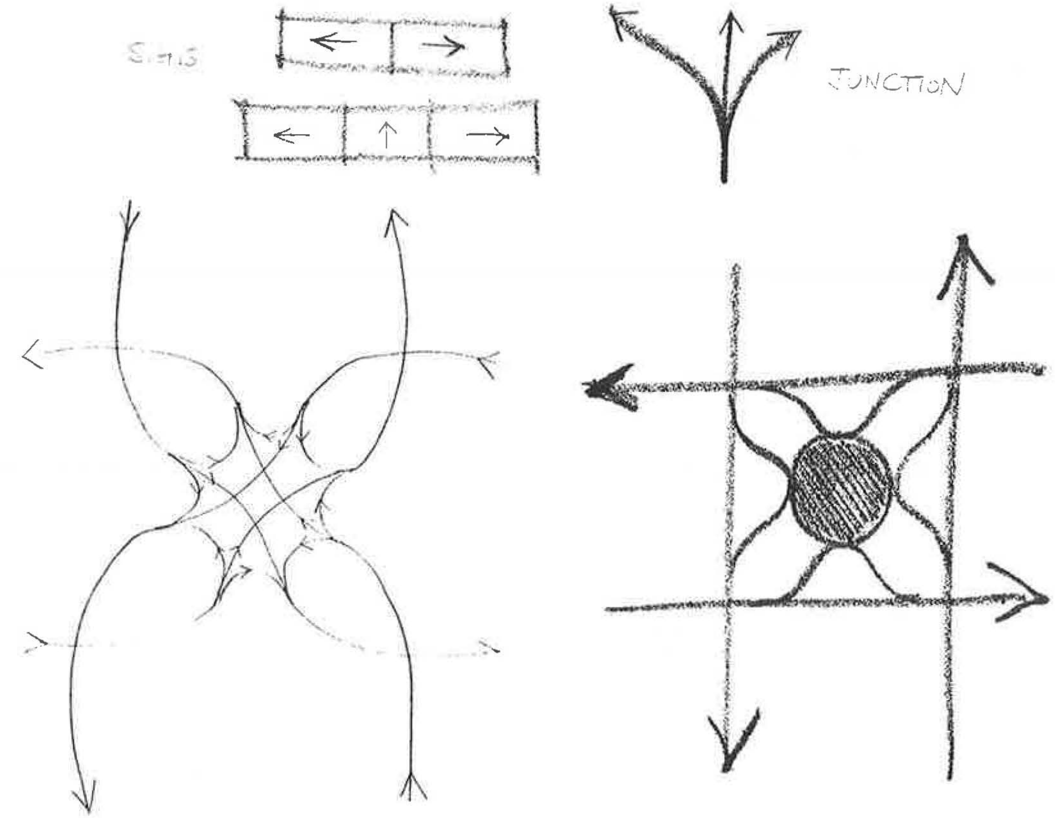


Diagram I: Requirements 16, 17, 18, 19, 20, 84.



This set of requirements deals principally with the fact that the driver has to find out where he is going as he drives through the interchange. The principal conflict which needs resolution is that between 18, 19, 20. On the one hand, 18 calls for as many signs and destination names as possible; on the other hand, 19 and 20 call for as little information as possible, so that it should not confuse the driver who can, after all, only digest a limited amount of information without slowing down. The resolution of this conflict suggests two things.

First of all, the information on the signs should be reduced to a minimum, and a parking place provided in which the driver may consult maps for further details.

Second, in view of this limited information, the signing for the left-hand turn and the right-hand turn should be presented to the driver simultaneously—if possible, also at the same time as the signs for the through path. Otherwise, when looking for an unfamiliar name, the driver may pass one exit, only to find, when he gets to the other, that it was the first he wanted.

These two matters, as shown in the bottom Diagram I, call for a form which contains a central parking space, from which any exit may be made, and for an interchange in which all possible turns are presented simultaneously, so that signs may be presented as shown at the top of the diagram.

CONCLUDING REMARKS

Some Comments on Design

These notes summarize the major ideas which we have been able to develop to date. Although we have by no means solved the interchange design problem completely, we can see emerging a possible solution in the diagrams described here.

The dominant theme seems to be the use of a central space or neutral zone, between the major through roads, for the mixing function, for transition between the through routes, and for information, emergency and service facilities. This represents a highly efficient use of the land usually wasted between turning roadways, and offers flexibility with regard to possible changes in traffic and in transportation technology (automatic control, rapid transit, ground-effect vehicles, etc.).

It is also important that this neutral mixing area can support regional development by providing access to surrounding land. Where two limited-access highways meet, weaving problems do not allow further interchanges for access to nonlimited-access roads. Therefore, for an area of several square miles about the interchange of the limited-access highways, potential growth is hampered by the distance traffic must go to get access to either of the roads: a pocket of dead land is created, and the great potential of the major highway interchange as a power in economic development is reduced.

We should perhaps mention the standard and widely used cloverleaf pattern. This type of interchange is cheap to build, does not consume too much land, and is easy to construct over an existing facility. Where cost and land are the major considerations, this type of interchange may still be a wise choice.

However, the cloverleaf has many noticeable defects. It provides very bad weaving, and offers only short acceleration and deceleration lanes. Its turns, which involve simultaneous change of grade and direction, are dangerous. It offers no opportunity to rectify a wrong turn, and yet presents the driver who wants to turn off with two sequential decisions, so that he does not know until he comes to the second one, which one was right; at that, the left-hand turn is counter-intuitive, and makes a 270° turn. It tends to waste the land inside its turning circles; and also tends to make access to neighboring land difficult, if both intersecting highways are of the limited-access type. Expansion is difficult (since inner turns cannot easily be widened) and opportunities for connection with other types of transportation, as part of a wider system, are non-existent.

SOME CONCLUSIONS FOR ENGINEERING PRACTICE

The computer analysis indicated that the highway interchange design problem had four major components. As a result of our attempts to diagram the implications of several of the subsystems of this problem, these four components can be described as follows:

1. The roadways in the interchange. Each of the roads in the interchange has characteristics as a path—both plane and vertical profile, in general and in relation to the topographical surface. Most of the kinds of things with which these requirements are concerned are characteristic of highways in general, and are not peculiar to interchanges alone.

2. The interchange as a system with particular characteristics and functions. These requirements describe the interaction of the interchange with its environment—its ecology: the relation to the system of vehicular flow—the properties of streams of vehicles; the demand for movement—various forces in the socio-economic system as expressed in terms of flow patterns; the ecology of the natural environment—trees, watercourses, weather; the interaction with the administrative structure of society—maintenance, control, emergencies, etc. These become explicit in the physical part of the interchange which carries on its unique functions—the mixing area.

3. The interchange as it concerns the driver. The driver's perception of where he is, where he is going, his place in the stream of vehicles—the provision of adequate information, comfort including avoidance of monotony, a clear view, provision for emergencies, etc.

4. The material structure of the interchange. These requirements concern the elements of the physical system which require local treatment. In a sense, they do not concern the basic functional organization (though they may have implications for it), but the gingerbread, the trimmings: decisions about materials, cross-sections and details of construction—the barriers to keep off snow, water, debris, wind, smoke;

the wearing surface and its base courses; etc. They are the kinds of things one would find dealt with in a table of specifications for standard designs.

By way of conclusion, we return again to the theme with which we introduced this discussion: the AASHO manual. Although this analysis is only preliminary, it has resulted in a way of looking at the interchange design problem which is distinctly different from the approach taken by the AASHO manual. The discussion of the diagrams and of the principal components of the problem have illustrated this. As we expected, even this preliminary analysis has suggested the way in which a manual for the design of interchanges should be constructed. For example, a division of the manual into four major sections, corresponding to the four major components of the problem outlined above, is not only theoretically justified (by our analysis) but also seems to make good sense on an intuitive basis, now that the analysis has pointed it out. Of course, the way in which each of these four sections is broken down into subsections should also reflect the structure of the problem as revealed by an analysis such as this; however, the analysis so far accomplished is not detailed enough to provide any well-founded basis for such further subdivision.

We do not plan to rewrite the AASHO manual; this is a task which is best handled by those who have had long experience with the detailed problems of designing real interchanges. Our analysis here has not been extensive, and should be considered only a first experiment. We have, however, demonstrated the kind of approach which we believe should be used to write a manual.

ACKNOWLEDGMENTS

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Appendix

LINKS AMONG THE REQUIREMENTS

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 37 38 39  0  0  0  0 44 45  0  0 48  0 50  0 52 53  0  0  0 57 58  0 60  0  0  0  0 65 66  0  0 69  0 71  0
 73  0  0 76  0  0  0 80  0 82  0  0 85  0 87 88  0  0  0  0  0 94 95 96 97  0  0  0  0  0  0  0105  0107108
109110  0  0

REQUIREMENT     002 IS CONNECTED TO--
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 37 38 39 40  0  0  0 44 45  0 47 48  0 50 51  0  0  0  0 56 57 58 59 60  0  0  0  0 65 66  0 68  0 70  0  0
  0  0  0  0  0  0  0  0  0 82  0  0 85  0 87  0  0 90  0  0  0 94 95 96 97  0  0100  0102  0  0105  0  0108
109110  0  0

REQUIREMENT     003 IS CONNECTED TO--
  0  0  0  0  5  0  0  0  9 10  0  0 13 14 15  0  0  0 19 20  0 22  0 24  0  0  0  0 29  0  0  0  0  0 35  0
  0 38 39 40 41  0 43 44  0 46 47 48  0  0  0  0 53  0  0  0  0  0 59  0  0  0  0  0 65  0  0  0  0  0  0  0
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109  0  0  0

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 37 38 39 40  0  0 43 44 45 46 47 48 49  0  0 52 53 54 55  0  0  0 59 60  0  0  0  0 65  0  0  0 69  0  0  0
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109  0111  0

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  0  0  0 76  0  0  0 80  0 82  0  0  0  0  0 83  0  0 91  0 93  0 95 96 97 98  0  0  0  0  0  0  0  0107  0
  0110  0  0

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 37 38 39 40 41  0  0 44 45 46 47 48 49 50 51  0  0  0 55  0  0  0 59 60  0 62 63 64 65 66  0  0 69 70 71  0
  0  0  0  0  0  0  0 80  0 82  0 84  0  0  0 88  0 90 91  0  0 94  0 96  0  0  0  0  0102  0104105  0107  0
109  0  0112

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 37 38  0 40  0  0  0 44 45  0 47 48 49 50 51 52  0 54 55 56  0 58 59 60 61  0 63  0 65 66  0 68  0 70 71  0
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  0110  0  0

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  0  0 39  0  0  0  0 44  0  0 47  0  0  0  0  0  0  0  0  0  0  0 59  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0 76 77 78 79  0  0  0  0  0  0 86 87  0 89  0 91  0  0  0  0  0  0  0  0100  0  0  0  0  0  0107108
109  0  0  0

REQUIREMENT     009 IS CONNECTED TO--
  0  0  3  0  0  0  7  0  0 10 11  0  0  0 15  0  0  0  0  0  0  0  0  0 25  0  0  0 29  0  0  0  0 34 35  0
  0 38 39 40  0  0 43 44  0  0 47  0  0  0  0  0 53  0  0  0 57 58 59 60  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0  0 85 86  0  0 89 90 91  0 93  0  0  0  0  0 99100101  0103  0  0  0  0  0
109  0111112
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```
REQUIREMENT    010 IS CONNECTED TO--
  0  0  3  0  0  0  7  0  9  0 11  0  0  0  0  0  0  0  0  0  0  0  0  0 25  0  0  0  0  0  0  0  0  0 35  0
  0 38 39  0  0  0  0  0  0  0 47  0  0  0  0 52  0  0  0  0  0 58 59  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0  0 85 86 87  0 89 90 91  0 93 94  0  0  0  0 99  0101102103  0  0  0  0  0
109  0111112

REQUIREMENT    011 IS CONNECTED TO--
  1  0  0  4  5  0  7  8  9 10  0  0  0  0  0  0  0  0  0  0  0  0  0  0 25  0  0  0  0  0 31  0  0  0  0  0
  0 38 39  0  0  0  0 44  0 46 47  0  0  0  0  0  0 54 55  0  0 58 59 60  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0  0 85 86 87  0 89 90  0  0  0 94  0  0  0  0  0100  0  0  0  0  0  0  0  0
109  0  0112

REQUIREMENT    012 IS CONNECTED TO--
  1  2  0  0  5  6  7  0  0  0  0  0 13 14  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0  0  0  0 33  0 35 36
 37 38  0  0  0  0 43 44 45 46  0 48 49 50 51  0  0  0 55 56  0 58 59 60 61 62  0  0  0  0 67 68 69  0  0 72
 73 74  0  0  0  0  0 80  0 82  0  0  0  0  0 88  0 90 91 92  0  0 95  0 97 98  0  0  0  0  0  0  0  0  0  0
109  0  0  0

REQUIREMENT    013 IS CONNECTED TO--
  1  0  3  4  5  6  7  0  0  0  0 12  0 14 15  0  0  0  0  0  0  0  0  0  0 27  0  0 30  0  0  0  0  0  0  0
 37 38 39 40  0  0 43 44 45 46 47 48  0 50  0  0 53 54  0  0 57 58 59  0  0  0 63  0  0  0  0 68  0  0  0  0
  0  0  0 76  0  0  0  0  0 82  0  0  0  0  0 88 89 90 91  0 93 94 95 96 97  0  0  0  0  0103  0  0  0  0  0
  0110  0  0

REQUIREMENT    014 IS CONNECTED TO--
  0  2  3  4  0  6  7  0  0  0  0 12 13  0  0 16  0  0  0 20  0  0  0  0  0  0  0  0 30  0  0  0  0 35  0
 37 38 39 40  0  0  0 44 45  0  0 48  0 50  0 52 53  0  0  0 57 58 59  0  0  0 63  0  0  0  0  0  0  0  0  0
  0  0  0 76  0  0  0  0  0 82  0  0  0  0  0 88 89 90 91  0  0 94 95 96 97  0  0  0  0  0  0  0  0  0  0  0
  0110  0  0

REQUIREMENT    015 IS CONNECTED TO--
  0  0  3  4  5  6  0  0  9  0  0  0 13  0  0 16 17 18  0  0 21 22  0 24  0  0  0  0 30  0  0 33  0  0  0
 37  0  0  0  0  0  0  0  0  0  0  0  0 50  0  0  0  0  0  0  0 58  0  0  0  0 63  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0 84  0  0  0 88  0 90  0  0  0  0  0  0 97  0  0  0101  0  0  0  0106  0  0
  0  0  0  0

REQUIREMENT    016 IS CONNECTED TO--
  0  0  0  4  5  0  7  0  0  0  0  0  0 14 15  0 17 18 19 20 21 22  0  0  0  0  0 28 29  0  0  0  0 34  0  0
 37  0  0  0  0 42  0 44  0  0  0  0  0  0  0  0  0  0  0  0  0 58  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0 84  0  0  0  0  0  0  0  0  0  0  0  0 97  0  0  0  0  0  0  0  0  0  0  0
  0  0  0112

REQUIREMENT    017 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0 15 16  0 18 19 20  0  0 23  0  0  0  0  0  0  0 32  0  0  0  0
  0  0  0  0  0 42  0  0  0  0  0 48  0  0  0  0  0  0  0  0  0 58  0  0  0  0 63  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0 82  0 84 85  0  0  0  0 90  0  0  0  0  0  0 97 98  0  0  0  0  0  0  0  0  0  0
  0  0  0112

REQUIREMENT    018 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0 15 16 17  0 19 20 21  0 23  0 25  0  0  0  0 30  0 32  0 34 35  0
  0  0  0  0  0  0  0  0  0  0  0  0 49  0  0  0  0  0  0  0  0  0 59  0  0 62 63  0  0  0  0  0  0  0 71  0
  0  0  0  0  0  0  0 80  0 82  0 84  0 86 87  0  0 90  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0111112
```

```
REQUIREMENT    019 IS CONNECTED TO--
   0   0   3   4   0   0   0   0   0   0   0   0   0   0   0  16  17  18   0  20   0   0   0   0   0   0   0   0   0   0   0  32   0   0   0   0
   0   0   0   0   0  42   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  58   0   0   0  62  63   0   0   0   0   0   0   0  71   0
   0   0   0   0   0   0   0   0   0  82   0  84  85   0   0   0   0  90   0  92   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
   0   0111112

REQUIREMENT    020 IS CONNECTED TO--
   0   0   3   0   0   6   7   0   0   0   0   0   0  14   0  16  17  18  19   0  21   0  23   0  25   0   0   0   0  30   0  32   0   0   0   0
  37   0   0   0   0  42   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  63   0   0   0   0   0   0   0   0   0
   0   0   0   0   0   0   0  80   0  82   0  84   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
0110   0112

REQUIREMENT    021 IS CONNECTED TO--
   0   0   0   0   0   6   0   0   0   0   0   0   0   0  15  16   0  18   0  20   0   0   0   0   0   0  27  28   0   0   0   0   0   0   0   0
   0   0   0   0   0   0  43   0   0   0   0   0  49   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
   0   0   0   0   0   0  79  80   0   0   0  84   0   0  87   0  89   0   0   0   0   0   0   0   0  99   0   0   0   0   0   0   0   0   0   0
   0   0 0113

REQUIREMENT    022 IS CONNECTED TO--
   0   2   3   4   0   0   0   0   0   0   0   0   0   0  15  16   0   0   0   0   0   0  23  24   0   0   0   0  29   0  31  32   0   0   0  36
   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  54   0   0   0   0   0   0   0  62   0   0   0   0   0   0   0   0   0   0
   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  89   0   0   0   0   0   0   0   0  98   0   0   0   0   0   0   0   0   0   0
109110   0   0

REQUIREMENT    023 IS CONNECTED TO--
   1   2   0   0   5   6   7   0   0   0   0   0   0   0   0   0  17  18   0  20   0  22   0  24  25   0   0   0  29  30   0  32   0   0   0   0
   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  63   0   0   0   0  68   0   0   0   0
   0   0   0   0   0   0   0   0   0  82   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0 0105   0   0   0
109110111112

REQUIREMENT    024 IS CONNECTED TO--
   0   2   3   4   0   0   0   0   0   0   0   0   0   0  15   0   0   0   0   0   0  22  23   0  25   0   0  28  29  30   0  32   0   0   0   0
   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  63   0   0   0   0   0   0   0   0   0
   0   0   0   0   0   0   0   0   0  82   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  98   0   0   0   0   0 0105   0   0   0
 109   0   0   0

REQUIREMENT    025 IS CONNECTED TO--
   0   0   0   0   0   0   0   0   9  10  11  12   0   0   0   0   0  18   0  20   0   0  23  24   0   0   0   0   0  30  31  32   0   0   0   0
  37  38  39  40  41   0   0   0  45  46  47  48  49  50   0   0  53  54  55   0   0  58  59  60   0  62   0   0   0   0   0   0   0   0   0   0
   0   0   0   0   0   0   0   0   0   0   0  84  85  86  87  88  89  90  91  92  93  94   0  96   0  98  99100101   0103   0105   0   0108
   0110111112

REQUIREMENT    026 IS CONNECTED TO--
   1   0   0   0   0   6   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  27  28   0   0   0   0  33  34   0  36
   0   0   0   0   0  42   0  44   0   0   0  48   0   0   0   0   0   0   0   0   0   0   0  60   0   0  63   0   0   0   0   0  69  70  71  72
  73   0   0   0   0   0   0  80  81   0   0  84  85   0   0   0   0   0  91   0   0   0  95  96   0   0   0   0   0   0   0   0 0106107   0
   0   0111112

REQUIREMENT    027 IS CONNECTED TO--
   1   2   0   0   0   6   7   0   0   0   0  12  13   0   0   0   0   0   0   0  21   0   0   0   0  26   0  28   0   0   0   0  33  34   0   0
   0   0  39  40   0   0   0  44  45   0  47  48   0   0  51   0  53   0   0   0   0   0  59  60  61   0  63   0  65  66   0  68   0   0  71  72
  73   0   0   0   0   0   0  80   0  82   0   0   0  86   0  88   0  90  91   0  93  94  95  96   0   0   0   0   0   0 0104   0   0 0108
   0   0   0   0
```

```
REQUIREMENT    028 IS CONNECTED TO--
  0  0  0  4  0  6  0  0  0  0  0  0  0  0  0 16  0  0  0 21  0  0 24  0 26 27  0  0 30 31  0  0  0  0  0  0
  0  0  0  0  0  0  0 44  0 46 47  0  0 50  0  0  0 54 55  0  0  0 59 60  0  0 63  0 65  0  0 68  0  0 71  0
  0  0  0  0  0  0 79  0  0 82  0 84  0  0  0 88  0 90 91  0  0  0  0 96  0  0  0  0  0  0  0  0  0  0107  0
  0  0  0  0

REQUIREMENT    029 IS CONNECTED TO--
  0  0  3  4  0  0  0  0  9  0  0  0  0  0  0 16  0  0  0  0  0 22 23 24  0  0  0  0  0  0  0  0  0  0  0  0
 37  0  0  0  0  0  0 44  0 46  0  0  0  0  0  0  0 54 55  0  0  0  0 60  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0 79  0  0  0  0 84  0 86  0 88  0  0 91  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0

REQUIREMENT    030 IS CONNECTED TO--
  1  2  0  0  5  0  7  0  0  0  0  0 13 14 15  0  0 18  0 20  0  0 23 24 25  0  0 28  0  0  0 32  0  0  0 36
  0 38 39 40  0  0 43  0  0 46 47 48  0  0  0  0  0 54  0  0 57  0 59 60  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0 79 80  0  0  0  0 85  0  0  0 89  0  0  0  0  0  0 96  0 98  0  0  0  0  0  0  0  0  0  0
109110111  0

REQUIREMENT    031 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0 11  0  0  0  0  0  0  0  0  0  0 22  0  0 25  0  0 28  0  0  0 32  0  0  0  0
  0  0 39  0  0  0  0  0  0 46  0  0 49  0  0 52  0 54  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0 81  0  0 84  0 86 87  0 89  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
109110  0  0

REQUIREMENT    032 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 17 18 19 20  0 22 23 24 25  0  0  0  0 30 31  0  0  0  0  0
 37 38 39  0  0  0 43  0 45 46 47 48 49  0  0  0  0  0 55  0  0  0  0  0  0 62 63  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0 82  0  0  0 86 87  0  0  0 91  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
109110  0  0

REQUIREMENT    033 IS CONNECTED TO--
  1  0  0  0  5  6  0  0  0  0  0 12  0  0 15  0  0  0  0  0  0  0  0  0  0 26 27  0  0  0  0  0  0  0 35  0
 37 38 39 40  0  0 43  0 45  0  0 48 49 50 51  0  0  0  0  0  0 58 59 60 61 62 63 64 65 66  0 68 69 70 71 72
 73 74  0 76 77 78  0 80 81 82 83  0  0 86  0 88  0 90 91  0 93 94 95 96  0  0  0  0  0102  0  0  0  0  0  0
  0  0111  0

REQUIREMENT    034 IS CONNECTED TO--
  0  0  0  0  0  6  0  0  9  0  0  0  0  0  0 16  0 18  0  0  0  0  0  0  0 26 27  0  0  0  0  0  0  0  0  0
  0  0  0 40  0  0  0  0 45  0  0  0 49  0  0  0 53  0  0  0  0  0  0  0  0  0  0  0  0 66  0  0  0  0 71  0
  0  0  0  0  0  0  0 80  0  0  0  0  0  0  0 88  0 90  0  0 93  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0

REQUIREMENT    035 IS CONNECTED TO--
  1  2  3  4  5  0  7  0  9 10  0 12  0 14  0  0  0 18  0  0  0  0  0  0  0  0  0  0  0  0  0  0 33  0  0  0
 37 38 39 40 41  0 43  0 45  0 47 48 49 50 51  0 53  0  0  0  0  0 59 60 61  0 63  0  0  0  0 68  0  0  0  0
  0 74  0 76  0  0  0  0  0  0  0  0  0  0  0 88 89 90 91 92  0  0 95 96  0  0  0  0  0  0  0  0  0  0  0  0
  0  0111112

REQUIREMENT    036 IS CONNECTED TO--
  0  0  0  0  5  6  7  0  0  0  0 12  0  0  0  0  0  0  0  0  0 22  0  0  0 26  0  0  0 30  0  0  0  0  0  0
  0  0  0  0  0  0  0 44 45  0  0  0  0 50  0  0  0  0  0  0  0  0  0  0  0  0  0 64 65 66  0  0  0 70 71 72
  0  0  0  0  0  0  0  0  0 82 83  0  0  0  0  0  0  0  0  0  0  0  0 96 97  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0
```

```
REQUIREMENT    037 IS CONNECTED TO--
  1  2  0  4  5  6  7  0  0  0  0 12 13 14 15 16  0  0  0 20  0  0  0  0 25  0  0  0 29  0  0 32 33  0 35  0
  0 38  0 40 41  0 43 44 45 46 47 48  0 50  0  0 53  0  0 56  0  0 59  0  0  0 63 64 65 66  0 68 69 70 71 72
  0  0  0  0  0  0  0  0 81 82 83  0  0 86  0 88 89 90 91  0  0  0 95 96 97 98  0  0  0  0  0  0  0  0107  0
109  0111112

REQUIREMENT    038 IS CONNECTED TO--
  1  2  3  4  5  6  7  0  9 10 11 12 13 14  0  0  0  0  0  0  0  0  0  0 25  0  0  0  0 30  0 32 33  0 35  0
 37  0 39  0  0  0 43 44 45 46 47 48 49 50 51  0 53 54  0  0  0 58  0 60 61  0  0 64 65 66  0  0 69  0  0 72
  0 74  0  0  0  0  0  0 81 82 83  0  0 86 87 88 89 90 91 92 93 94 95 96 97 98 99  0101  0103104  0  0107  0
109  0111112

REQUIREMENT    039 IS CONNECTED TO--
  1  2  3  4  5  6  0  8  9 10 11  0 13 14  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0 30 31 32 33  0 35  0
  0 38  0 40 41  0 43 44 45 46 47 48 49 50 51 52 53 54 55  0 57 58 59  0  0 62  0  0 65 66  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0 82  0  0 85 86 87 88 89 90 91 92 93 94 95 96 97 98 99  0101  0103  0105  0107  0
109110  0  0

REQUIREMENT    040 IS CONNECTED TO--
  0  2  3  4  0  6  7  0  9  0  0  0 13 14  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0 30  0  0 33 34 35  0
 37  0 39  0 41  0  0  0 45  0 47 48 49  0  0 52 53 54  0  0 57  0 59 60  0 62 63  0 65 66  0  0  0 70  0  0
  0  0  0  0  0  0  0 80  0 82  0  0  0 86 87 88 89 90 91  0 93 94 95 96  0  0 99  0101  0  0  0105  0107  0
109  0111112

REQUIREMENT    041 IS CONNECTED TO--
  0  0  3  0  0  6  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 25  0  0  0  0  0  0  0  0  0 35  0
 37  0 39 40  0 42 43  0  0  0  0  0  0  0  0  0 53  0  0  0  0 58 59  0  0  0  0  0 65  0 67  0  0  0  0  0
  0 74  0 76 77 78  0 80 81  0  0  0  0 86 87  0 89  0 91 92 93 94  0 96  0  0  0  0  0102  0  0  0  0  0  0
  0  0  0  0

REQUIREMENT    042 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 16 17  0 19 20  0  0  0  0  0 26  0  0  0  0  0  0  0  0  0  0
  0  0  0  0 41  0  0  0  0 46  0  0  0  0  0  0  0  0  0 56  0 58 59  0  0 62  0 64  0 66 67  0  0  0 71  0
  0 74 75 76 77 78  0 80 81 82  0  0  0  0  0  0  0  0 91  0  0  0  0 96  0  0  0  0  0102  0  0  0  0  0  0
  0  0  0  0

REQUIREMENT    043 IS CONNECTED TO--
  0  0  3  4  0  0  0  0  9  0  0 12 13  0  0  0  0  0  0  0 21  0  0  0  0  0  0  0  0 30  0 32 33  0 35  0
 37 38 39  0 41  0  0 44  0  0  0 48  0 50  0 52 53  0  0  0 57  0 59  0  0 62 63  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0 82 83  0  0 86 87 88 89  0 91  0  0  0  0 96 97 98  0  0  0  0  0  0  0  0  0  0
109  0111112

REQUIREMENT    044 IS CONNECTED TO--
  1  2  3  4  5  6  7  8  9  0 11 12 13 14  0 16  0  0  0  0  0  0  0  0  0 26 27 28 29  0  0  0  0  0  0 36
 37 38 39  0  0  0 43  0 45  0 47 48  0 50  0 52 53 54 55 56  0  0 59  0  0  0 63  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0 82  0  0  0 86  0  0 89 90 91  0 93 94 95 96 97 98  0100101  0103  0  0  0107108
109110111112

REQUIREMENT    045 IS CONNECTED TO--
  1  2  0  4  5  6  7  0  0  0  0 12 13 14  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0  0  0 32 33 34 35 36
 37 38 39 40  0  0  0 44  0  0 47 48 49 50  0  0 53 54 55 56  0  0 59 60 61  0 63 64 65 66  0  0  0  0 71 72
 73  0  0  0  0  0  0 80  0 82  0  0  0 86 87 88 89 90 91  0 93 94 95 96 97 98  0  0  0  0103104  0  0107108
109  0  0112
```

```
REQUIREMENT    046 IS CONNECTED TO--
  0  0  3  4  5  6  0  0  0  0 11 12 13  0  0  0  0  0  0  0  0  0  0  0 25  0  0 28 29 30 31 32  0  0  0  0
 37 38 39 40  0 42  0  0  0  0 47 48 49  0  0  0 53 54  0  0  0 58 59 60 61  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0 82  0 84 85 86 87 88 89 90 91  0 93 94 95 96  0  0 99100101  0  0  0  0  0  0108
  0110  0112

REQUIREMENT    047 IS CONNECTED TO--
  0  2  3  4  5  6  7  8  9 10 11  0 13  0  0  0  0  0  0  0  0  0  0  0 25  0 27 28  0 30  0 32  0  0 35  0
 37 38 39 40  0  0  0 44 45 46  0  0  0  0 51  0 53 54 55  0  0  0 59  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0 82  0  0 85 86 87 88 89  0 91  0  0 94  0  0  0  0 99100101102  0  0  0  0  0108
  0110  0  0

REQUIREMENT    048 IS CONNECTED TO--
  1  2  3  4  0  6  7  0  0  0  0 12 13 14  0  0 17  0  0  0  0  0  0  0 25 26 27  0  0 30  0 32 33  0 35  0
 37 38 39 40  0  0 43 44 45 46  0  0 49 50 51 52 53  0  0  0  0  0 59 60 61  0 63  0 65 66  0 68 69  0  0 72
 73 74  0  0  0 78  0 80  0 82  0  0  0 86  0 88 89 90 91  0  0  0 95 96 97  0  0  0101  0  0  0  0  0  0  0
109  0  0112

REQUIREMENT    049 IS CONNECTED TO--
  0  0  0  4  0  6  7  0  0  0  0 12  0  0  0  0  0 18  0  0 21  0  0  0 25  0  0  0  0  0 31 32 33 34 35  0
  0 38 39 40  0  0  0  0 45 46  0 48  0  0 51  0 53  0  0  0 57  0 59 60 61 62 63  0  0 66 67  0  0  0  0  0
  0  0  0  0  0  0  0  0 81 82 83 84  0 86 87 88 89 90  0 92 93 94 95 96  0  0  0  0  0  0  0  0  0  0  0  0
  0110111112

REQUIREMENT    050 IS CONNECTED TO--
  1  2  0  0  5  6  7  0  0  0  0 12 13 14 15  0  0  0  0  0  0  0  0  0 25  0  0 28  0  0  0  0 33  0 35 36
 37 38 39  0  0  0 43 44 45  0  0 48  0  0  0 52  0 54  0  0  0  0  0  0 61  0 63  0  0  0  0  0  0 70  0  0
 73  0  0  0  0  0  0  0  0 82  0  0  0  0  0 88 89 90 91 92  0  0 95 96 97 98 99  0101102103  0105  0  0  0
109  0  0112

REQUIREMENT    051 IS CONNECTED TO--
  0  2  0  0  0  6  7  0  0  0  0 12  0  0  0  0  0  0  0  0  0  0  0  0  0  0 27  0  0  0  0  0 33  0 35  0
  0 38 39  0  0  0  0  0  0  0 47 48 49  0  0  0 53  0  0  0 57  0 59 60 61  0 63  0 65  0  0  0 69  0  0  0
  0 74  0  0  0  0  0  0 81  0  0  0  0  0  0 88  0 90 91  0  0  0  0 96  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0112

REQUIREMENT    052 IS CONNECTED TO--
  1  0  0  4  5  0  7  0  0 10  0  0  0 14  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 31  0  0  0  0  0
  0  0 39 40  0  0 43 44  0  0  0 48  0 50  0  0 53 54  0  0 57  0 59 60  0 62 63  0 65  0  0  0  0 70  0  0
  0  0  0  0  0  0  0  0  0 82  0  0  0 86 87 88 89 90  0  0 93 94  0 96 97 98  0  0101  0  0  0  0  0  0  0
109110111112

REQUIREMENT    053 IS CONNECTED TO--
  1  0  3  4  0  0  0  0  9  0  0  0 13 14  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0  0  0  0  0 34 35  0
 37 38 39 40 41  0 43 44 45 46 47 48 49  0 51 52  0 54  0 56 57 58 59 60 61 62  0  0 65 66  0 68 69  0  0  0
  0  0  0  0  0  0  0  0  0 82 83  0  0 86 87 88 89 90 91 92 93 94 95  0  0  0 99100101  0103  0  0  0  0  0
109  0111112

REQUIREMENT    054 IS CONNECTED TO--
  0  0  0  4  5  0  7  0  0  0 11  0 13  0  0  0  0  0  0  0  0 22  0  0 25  0  0 28 29 30 31  0  0  0  0  0
  0 38 39 40  0  0  0 44 45 46 47  0  0 50  0 52 53  0 55  0 57  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0  0  0  0  0 86 87 88 89  0 91  0 93 94  0  0  0  0  0100  0  0  0  0  0  0  0  0
  0110111112
```

```
REQUIREMENT    055 IS CONNECTED TO--
  0  0  0  4  5  6  7  0  0  0 11 12  0  0  0  0  0  0  0  0  0  0  0 25  0  0 28 29  0  0 32  0  0  0  0
  0  0 39  0  0  0  0 44 45  0 47  0  0  0  0  0  0 54  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0 82  0  0  0 86 87 88 89  0  0  0  0  0  0  0  0 99100  0  0  0  0  0  0  0  0
  0110  0112

REQUIREMENT    056 IS CONNECTED TO--
  0  2  0  0  5  0  7  0  0  0  0 12  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
 37  0  0  0  0 42  0 44 45  0  0  0  0  0  0  0 53  0  0  0  0 58 59  0  0  0  0 64 65 66  0 68 69  0  0 72
 73  0  0  0  0 78  0  0 81  0  0  0  0  0  0  0  0  0 91  0  0 94  0  0  0  0  0100  0  0  0  0  0  0  0  0
109  0  0  0

REQUIREMENT    057 IS CONNECTED TO--
  1  2  0  0  5  0  0  0  9  0  0  0 13 14  0  0  0  0  0  0  0  0  0  0  0  0  0 30  0  0  0  0  0  0
  0  0 39 40  0  0 43  0  0  0  0  0 49  0 51 52 53 54  0  0  0 58 59  0  0 62  0  0  0  0  0  0  0  0  0
  0  0  0  0 77  0  0  0  0 82  0  0  0 86 87 88  0 90  0 92 93 94  0 96 97  0 99100101  0103  0  0  0  0  0
  0110  0  0

REQUIREMENT    058 IS CONNECTED TO--
  1  2  0  0  0  0  7  0  9 10 11 12 13 14 15 16 17  0 19  0  0  0  0  0 25  0  0  0  0  0  0  0 33  0  0  0
  0 38 39  0 41 42  0  0  0 46  0  0  0  0  0  0 53  0  0 56 57  0  0  0  0  0  0 64  0 66 67  0 69  0  0 72
 73 74 75 76 77 78 79 80 81 82  0 84 85  0  0  0  0 90 91  0 93  0 95 96 97  0 99  0101  0  0104  0106107108
109  0  0112

REQUIREMENT    059 IS CONNECTED TO--
  0  2  3  4  5  6  7  8  9 10 11 12 13 14  0  0  0 18  0  0  0  0  0  0 25  0 27 28  0 30  0  0 33  0 35  0
 37  0 39 40 41 42 43 44 45 46 47 48 49  0 51 52 53  0  0 56 57  0  0 60 61  0  0 64 65 66  0 68 69  0 71 72
  0 74 75  0 77 78 79 80 81 82  0  0  0 86 87 88 89 90 91 92 93 94  0 96  0  0 99100101102103104105  0  0108
  0110111112

REQUIREMENT    060 IS CONNECTED TO--
  1  2  0  4  5  6  7  0  9  0 11 12  0  0  0  0  0  0  0  0  0  0  0 25 26 27 28 29 30  0  0 33  0 35  0
  0 38  0 40  0  0  0  0 45 46  0 48 49  0 51 52 53  0  0  0  0  0 59  0 61  0  0  0 65 66  0 68  0  0  0  0
  0  0  0  0  0  0  0  0  0 82  0  0  0 86 87 88 89 90 91 92  0 94 95 96 97  0 99  0101  0103  0  0  0  0  0
109  0111  0

REQUIREMENT    061 IS CONNECTED TO--
  C  0  0  0  5  0  7  0  0  0  0 12  0  0  0  0  0  0  0  0  0  0  0  0  0 27  0  0  0  0  0 33  0 35  0
  0 38  0  0  0  0  0  0 45 46  0 48 49 50 51  0 53  0  0  0  0  0 59 60  0  0  0  0  0  0  0 68  0 70  0  0
  0  0  0  0  0  0  0  0 81 82  0  0 85  0 87 88  0  0 91  0  0  0 95 96  0  0  0  0  0  0  0  0  0  0  0
109  0  0112

REQUIREMENT    062 IS CONNECTED TO--
  0  0  0  0  0  6  0  0  0  0  0 12  0  0  0  0  0 18 19  0  0 22  0  0 25  0  0  0  0  0  0 32 33  0  0  0
  0  0 39 40  0 42 43  0  0  0  0  0 49  0  0 52 53  0  0  0 57  0  0  0  0  0 63 64 65 66 67  0 69  0  0  0
  0 74 75 76  0  0  0 80  0 82  0  0 85  0  0  0 89 90 91  0  0  0  0 96  0 98  0  0  0  0  0  0  0  0  0
  0110  0  0

REQUIREMENT    063 IS CONNECTED TO--
  0  0  0  0  5  6  7  0  0  0  0  0 13 14 15  0 17 18 19 20  0  0 23 24  0 26 27 28  0  0  0 32 33  0 35  0
 37  0  0 40  0  0 43 44 45  0  0 48 49 50 51 52  0  0  0  0  0  0  0  0 62  0  0  0 66  0 68  0  0  0  0
  0  0  0  0 77  0  0 80  0 82  0  0  0  0  0  0  0  0 91 92  0  0 95 96 97 98  0  0  0  0  0  0105  0  0  0
109  0  0112
```

```
REQUIREMENT     064 IS CONNECTED TO--
  0   0   0   0   0   6   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  33   0   0  36
 37  38   0   0   0  42   0   0  45   0   0   0   0   0   0   0   0   0   0  56   0  58  59   0   0  62   0   0  65  66  67  68  69  70  71  72
 73   0   0   0   0   0   0   0   0   0  83   0  85   0   0   0   0   0  91   0   0  94  95   0   0   0   0   0   0   0   0104105106107108
  0   0   0   0

REQUIREMENT     065 IS CONNECTED TO--
  1   2   3   4   5   6   7   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  27  28   0   0   0   0  33   0   0  36
 37  38  39  40  41   0   0   0  45   0   0  48   0   0  51  52  53   0   0  56   0   0  59  60   0  62   0  64   0  66   0   0  69  70  71   0
  0   0   0   0   0   0   0   0   0   0  83   0   0  86   0  88   0   0   0   0  93  94  95   0   0   0   0   0   0   0   0104105   0   0   0
109   0   0112

REQUIREMENT     066 IS CONNECTED TO--
  1   2   0   0   5   6   7   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  27   0   0   0   0   0  33  34   0  36
 37  38  39  40   0  42   0   0  45   0   0  48  49   0   0   0  53   0   0  56   0  58  59  60   0  62  63  64  65   0   0  68  69   0  71   0
  0   0   0   0   0   0   0   0   0   0  83   0  85  86   0  88   0   0   0   0  93  94  95   0   0   0  99   0   0   0   0104   0   0   0   0
109   0   0112

REQUIREMENT     067 IS CONNECTED TO--
  0   0   0   0   0   0   0   0   0   0   0  12   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
  0   0   0   0  41  42   0   0   0   0   0   0  49   0   0   0   0   0   0   0  58   0   0   0  62   0  64   0   0   0   0   0   0   0  71  72
 73  74  75  76  77  78  79   0  81  82   0   0  85   0   0   0   0   0   0   0  93   0   0  96   0   0   0100101   0   0   0   0   0   0   0
  0   0   0   0

REQUIREMENT     068 IS CONNECTED TO--
  0   2   0   0   5   0   7   0   0   0   0  12  13   0   0   0   0   0   0   0   0   0  23   0   0   0  27  28   0   0   0   0  33   0  35   0
 37   0   0   0   0   0   0   0   0   0   0  48   0   0   0   0  53   0   0  56   0   0  59  60  61   0  63  64   0  66   0   0  69  70  71  72
  0  74  75   0  77  78   0   0   0  82   0   0   0   0   0   0   0   0   0   0  93   0   0   0   0   0   0   0101   0   0104   0   0   0   0
  0   0   0   0

REQUIREMENT     069 IS CONNECTED TO--
  1   0   0   4   5   6   0   0   0   0   0  12   0   0   0   0   0   0   0   0   0   0   0   0   0  26   0   0   0   0   0   0  33   0   0   0
 37  38   0   0   0   0   0   0   0   0   0  48   0   0  51   0  53   0   0  56   0  58  59   0   0  62   0  64  65  66   0  68   0   0   0   0
 73   0   0   0  77  78   0  80   0  82   0   0   0   0   0   0   0   0  91   0   0  94   0  96   0   0   0   0101   0   0104   0   0   0   0
  0   0   0   0

REQUIREMENT     070 IS CONNECTED TO--
  0   2   0   0   0   6   7   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0  26   0   0   0   0   0   0  33   0   0  36
 37   0   0  40   0   0   0   0   0   0   0   0   0  50   0  52   0   0   0   0   0   0   0   0  61   0   0  64  65   0   0  68   0   0   0   0
  0   0   0   0   0  78  79   0  81  82   0  84   0   0   0   0   0   0   0   0  93   0  95   0   0   0   0   0   0102   0   0   0   0   0108
109   0   0112

REQUIREMENT     071 IS CONNECTED TO--
  1   0   0   0   5   6   7   0   0   0   0   0   0   0   0   0   0  18  19   0   0   0   0   0   0  26  27  28   0   0   0   0  33  34   0  36
 37   0   0   0   0  42   0   0  45   0   0   0   0   0   0   0   0   0   0   0   0   0  59   0   0   0   0  64  65  66  67  68   0   0   0  72
 73   0  75  76   0   0   0  80   0  82   0   0  85   0   0  88   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
109   0   0   0

REQUIREMENT     072 IS CONNECTED TO--
  0   0   0   0   0   0   0   0   0   0   0  12   0   0   0   0   0   0   0   0   0   0   0   0   0  26  27   0   0   0   0   0  33   0   0  36
 37  38   0   0   0   0   0   0  45   0   0  48   0   0   0   0   0   0   0  56   0  58  59   0   0   0   0  64   0   0  67  68   0   0  71   0
 73  74  75  76  77  78  79  80   0  82   0   0   0   0   0   0   0   0   0   0   0   0  95   0   0   0   0   0   0   0   0   0   0   0   0   0
  0   0111   0
```

```
REQUIREMENT     073 IS CONNECTED TO--
  1  0  0  0  0  0  0  0  0  0  0 12  0  0  0  0  0  0  0  0  0  0  0  0  0 26 27  0  0  0  0  0 33  0  0  0
  0  0  0  0  0  0  0  0 45  0  0 48  0 50  0  0  0  0  0 56  0 58  0  0  0  0  0 64  0  0 67  0 69  0 71 72
  0 74 75 76 77 78  0  0 81 82  0  0  0  0  0  0  0  0  0  0  0  0 95  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0

REQUIREMENT     074 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0 12  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 33  0 35  0
  0 38  0  0 41 42  0  0  0  0  0 48  0  0 51  0  0  0  0  0  0 58 59  0  0 62  0  0  0  0 67 68  0  0  0 72
 73  0 75 76 77 78 79 80 81 82  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
109  0  0112

REQUIREMENT     075 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0 42  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 58 59  0  0 62  0  0  0  0 67 68  0  0 71 72
 73 74  0 76 77 78 79  0 81 82  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0104  0  0  0  0
  0  0  0  0

REQUIREMENT     076 IS CONNECTED TO--
  1  0  0  0  5  0  0  8  0  0  0  0 13 14  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 33  0 35  0
  0  0  0  0 41 42  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 58  0  0  0 62  0  0  0  0 67  0  0  0 71 72
 73 74 75  0 77 78 79  0 81 82  0  0  0  0  0  0  0  0  0  0  0  0 95  0 97  0 99100  0  0  0  0  0  0  0  0
109  0111  0

REQUIREMENT     077 IS CONNECTED TO--
  0  0  0  0  0  0  0  8  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 33  0  0  0
  0  0  0  0 41 42  0  0  0  0  0  0  0  0  0  0  0  0  0 57 58 59  0  0  0 63  0  0  0  0 67 68 69  0  0 72
 73 74 75 76  0 78 79  0 81 82  0  0 85  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0103  0  0  0107  0
  0  0  0112

REQUIREMENT     078 IS CONNECTED TO--
  0  0  0  0  0  0  0  8  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 33  0  0  0
  0  0  0  0 41 42  0  0  0  0  0 48  0  0  0  0  0  0  0 56  0 58 59  0  0  0  0  0  0  0 67 68 69 70  0 72
 73 74 75 76 77  0 79  0 81  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0

REQUIREMENT     079 IS CONNECTED TO--
  0  0  0  0  0  0  0  8  0  0  0  0  0  0  0  0  0  0  0  0 21  0  0  0  0  0  0 28 29 30  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 58 59  0  0  0  0  0  0  0 67  0  0 70  0 72
  0 74 75 76 77 78  0  0 81  0  0 84  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0

REQUIREMENT     080 IS CONNECTED TO--
  1  0  3  4  5  6  7  0  0  0  0 12  0  0  0  0  0 18  0 20 21  0  0  0  0 26 27  0  0 30  0 32 33 34  0  0
  0  0  0 40 41 42  0 44 45 46 47 48  0  0  0  0  0 54 55  0  0 58 59  0  0 62 63  0  0  0  0  0 69  0 71 72
  0 74  0  0  0  0  0  0 81 82 83  0  0  0 87 88 89  0 91  0 93 94  0 96 97  0 99100101102  0  0  0  0  0108
109110  0112

REQUIREMENT     081 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 26  0  0  0  0 31  0 33  0  0  0
 37 38  0  0 41 42  0  0  0  0  0  0 49  0 51  0  0  0  0 56  0 58 59  0 61  0  0  0  0  0 67  0  0 70  0  0
 73 74 75 76 77 78 79 80  0 82 83  0 85  0  0  0  0 90 91 92 93 94 95 96  0  0  0  0  0  0  0  0  0  0  0108
109  0111112
```

```
REQUIREMENT     082 IS CONNECTED TO--
  1  2  3  4  5  6  7  0  0  0  0 12 13 14  0  0 17 18 19 20  0  0 23 24  0  0 27 28  0  0  0 32 33  0  0 36
 37 38 39 40  0 42 43 44 45 46 47 48 49 50  0 52 53  0 55  0 57 58 59 60 61 62 63  0  0  0 67 68 69 70 71 72
 73 74 75 76 77  0  0 80 81  0 83  0 85 86  0  0 89 90 91 92 93 94 95 96 97 98  0  0101  0103  0  0  0  0  0
109110111112

REQUIREMENT     083 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 33  0  0 36
 37 38  0  0  0  0 43  0  0  0  0  0 49  0  0  0 53  0  0  0  0  0  0  0  0  0  0  0 64 65 66  0  0  0  0  0
  0  0  0  0  0  0  0 80 81 82  0  0  0  0 87  0  0 90 91 92  0  0  0 96  0  0  0  0  0  0  0  0  0  0  0  0
109  0111112

REQUIREMENT     084 IS CONNECTED TO--
  0  0  3  4  0  6  0  0  0  0  0  0  0  0 15 16 17 18 19 20 21  0  0  0 25 26  0 28 29  0 31  0  0  0  0  0
  0  0  0  0  0  0  0  0  0 46  0  0 49  0  0  0  0  0  0  0  0 58  0  0  0  0  0  0  0  0  0  0  0 70  0  0
  0  0  0  0  0  0 79  0  0  0  0  0  0 86 87 88 89  0  0  0  0 94  0 96  0  0  0  0  0  0  0  0  0  0107  0
  0110  0112

REQUIREMENT     085 IS CONNECTED TO--
  1  2  3  4  0  0  0  0  9 10 11  0  0  0  0  0 17  0 19  0  0  0  0  0 25 26  0  0  0 30  0  0  0  0  0  0
  0  0 39  0  0  0  0  0  0 46 47  0  0  0  0  0  0  0  0  0  0 58  0  0 61 62  0 64  0 66 67  0  0  0 71  0
  0  0  0  0 77  0  0  0 81 82  0  0  0 86 87 88 89 90  0  0 93 94  0  0  0  0 99100  0  0  0  0  0  0  0  0
109110  0112

REQUIREMENT     086 IS CONNECTED TO--
  0  0  3  4  0  0  7  8  9 10 11  0  0  0  0  0  0 18  0  0  0  0  0  0 25  0 27  0 29  0 31 32 33  0  0  0
 37 38 39 40 41  0 43 44 45 46 47 48 49  0  0 52 53 54 55  0 57  0 59 60  0  0  0  0 65 66  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0 82  0 84 85  0 87 88 89 90 91  0 93 94  0 96  0  0 99100101  0103104105106107108
109110111112

REQUIREMENT     087 IS CONNECTED TO--
  1  2  3  4  0  0  7  8  0 10 11  0  0  0  0  0  0 18  0  0 21  0  0  0 25  0  0  0  0  0 31 32  0  0  0  0
  0 38 39 40 41  0 43  0 45 46 47  0 49  0  0 52 53 54 55  0 57  0 59 60 61  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0  0 83 84 85 86  0 88 89 90 91  0 93 94 95 96  0  0  0100101  0103104105106107108
  0110111112

REQUIREMENT     088 IS CONNECTED TO--
  1  0  0  4  5  6  7  0  0  0  0 12 13 14 15  0  0  0  0  0  0  0  0  0 25  0 27 28 29  0  0  0 33 34 35  0
 37 38 39 40  0  0 43  0 45 46 47 48 49 50 51 52 53 54 55  0 57  0 59 60 61  0  0  0 65 66  0  0  0  0 71  0
  0  0  0  0  0  0  0 80  0  0  0 84 85 86 87  0 89 90 91  0  0 94 95  0 97 98 99100101  0103104  0  0  0  0
109  0111112

REQUIREMENT     089 IS CONNECTED TO--
  0  0  3  4  0  0  7  8  9 10 11  0 13 14  0  0  0  0  0  0 21 22  0  0 25  0  0  0  0 30 31  0  0  0 35  0
 37 38 39 40 41  0 43 44 45 46 47 48 49 50  0 52 53 54 55  0  0  0 59 60  0 62  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0 82  0 84 85 86 87 88  0 90 91  0 93 94 95  0  0 98 99100  0102103  0105  0  0  0
  0110  0  0

REQUIREMENT     090 IS CONNECTED TO--
  0  2  3  4  0  6  0  0  9 10 11 12 13 14 15  0 17 18 19  0  0  0  0  0 25  0 27 28  0  0  0  0 33 34 35  0
 37 38 39 40  0  0  0 44 45 46  0 48 49 50 51 52 53  0  0  0 57 58 59 60  0 62  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0 81 82 83  0 85 86 87 88 89  0 91 92 93 94  0 96  0  0  0  0101  0103  0  0  0  0  0
109110111112
```

```
REQUIREMENT    091 IS CONNECTED TO--
  0  0  3  4  5  6  7  8  9 10  0 12 13 14  0  0  0  0  0  0  0  0  0  0 25 26 27 28 29  0  0 32 33  0 35  0
 37 38 39 40 41 42 43 44 45 46 47 48  0 50 51  0 53 54  0 56  0 58 59 60 61 62 63 64  0  0  0  0 69  0  0  0
  0  0  0  0  0  0  0 80 81 82 83  0  0 86 87 88 89 90  0 92 93 94 95 96 97 98 99100101  0103  0  0  0  0  0
109110111  0

REQUIREMENT    092 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0 12  0  0  0  0  0  0 19  0  0  0  0  0 25  0  0  0  0  0  0  0  0  0 35  0
  0 38 39  0 41  0  0  0  0  0  0  0 49 50  0  0 53  0  0  0 57  0 59 60  0  0 63  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0 81 82 83  0  0  0  0  0  0 90 91  0 93  0  0 96  0  0  0  0101  0103  0  0  0  0  0
  0  0111112

REQUIREMENT    093 IS CONNECTED TO--
  0  0  3  0  5  0  7  0  9 10  0  0 13  0  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0  0  0  0 33 34  0  0
  0 38 39 40 41  0  0 44 45 46  0  0 49  0  0 52 53 54  0  0 57 58 59  0  0  0  0  0 65 66 67 68  0 70  0  0
  0  0  0  0  0  0  0 80 81 82  0  0 85 86 87  0 89 90 91 92  0 94 95  0  0  0 99100101  0103  0  0  0107108
109  0111112

REQUIREMENT    094 IS CONNECTED TO--
  1  2  3  4  0  6  7  0  0 10 11  0 13 14  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0  0  0  0 33  0  0  0
  0 38 39 40 41  0  0 44 45 46 47  0 49  0  0 52 53 54  0 56 57  0 59 60  0  0  0 64 65 66  0  0 69  0  0  0
  0  0  0  0  0  0  0 80 81 82  0 84 85 86 87 88 89 90 91  0 93  0 95  0  0  0 99100101102  0104105  0107  0
109110111112

REQUIREMENT    095 IS CONNECTED TO--
  1  2  0  4  5  0  7  0  0  0  0 12 13 14  0  0  0  0  0  0  0  0  0  0  0 26 27  0  0  0  0  0 33  0 35  0
 37 38 39 40  0  0  0 44 45 46  0 48 49 50  0  0 53  0  0  0  0 58  0 60 61  0 63 64 65 66  0  0  0 70  0 72
 73  0  0 76  0  0  0  0 81 82  0  0  0  0 87 88 89  0 91  0 93 94  0 96 97 98  0  0  0102103  0  0  0  0  0
109  0  0  0

REQUIREMENT    096 IS CONNECTED TO--
  1  2  3  4  5  6  7  0  0  0  0  0 13 14  0  0  0  0  0  0  0  0  0  0 25 26 27 28  0 30  0  0 33  0 35 36
 37 38 39 40 41 42 43 44 45 46  0 48 49 50 51 52  0  0  0  0 57 58 59 60 61 62 63  0  0  0 67  0 69  0  0  0
  0  0  0  0  0  0  0 80 81 82 83 84  0 86 87  0  0 90 91 92  0  0 95  0 97 98  0  0101  0  0  0  0  0  0  0
109  0  0112

REQUIREMENT    097 IS CONNECTED TO--
  1  2  0  4  5  0  7  0  0  0  0 12 13 14 15 16 17  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 36
 37 38 39  0  0  0 43 44 45  0  0 48  0 50  0 52  0  0  0  0 57 58  0 60  0  0 63  0  0  0  0  0  0  0  0  0
  0  0  0 76  0  0  0 80  0 82  0  0  0  0  0 88  0  0 91  0  0  0 95 96  0 98  0  0  0  0  0  0  0  0  0  0
  0110111112

REQUIREMENT    098 IS CONNECTED TO--
  0  0  0  0  5  0  0  0  0  0  0 12  0  0  0  0 17  0  0  0  0 22  0 24 25  0  0  0  0 30  0  0  0  0  0  0
 37 38 39  0  0  0 43 44 45  0  0  0  0 50  0 52  0  0  0  0  0  0  0  0  0 62 63  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0 82  0  0  0  0  0 88 89  0 91  0  0  0 95 96 97  0  0  0  0  0  0  0  0  0  0  0
109110111112

REQUIREMENT    099 IS CONNECTED TO--
  0  0  3  0  0  0  0  0  9 10  0  0  0  0  0  0  0  0  0  0 21  0  0  0 25  0  0  0  0  0  0  0  0  0  0  0
  0 38 39 40  0  0  0  0  0 46 47  0  0 50  0  0 53  0 55  0 57 58 59 60  0  0  0  0  0 66  0  0  0  0  0  0
  0  0  0 76  0  0  0 80  0  0  0  0 85 86  0 88 89  0 91  0 93 94  0  0  0  0  0100101102103  0105  0  0  0
  0110  0112
```

```
REQUIREMENT     100 IS CONNECTED TO--
  0  2  0  0  0  0  0  8  9  0 11  0  0  0  0  0  0  0  0  0  0  0  0 25  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 44  0 46 47  0  0  0  0  0 53 54 55 56 57  0 59  0  0  0  0  0  0  0 67  0  0  0  0  0
  0  0  0 76  0  0  0 80  0  0  0  0 85 86 87 86 89  0 91  0 93 94  0  0  0  0 99  0101102103  0105  0  0  0
  0110  0  0

REQUIREMENT     101 IS CONNECTED TO--
  0  0  3  0  0  0  0  0  9 10  0  0  0  0 15  0  0  0  0  0  0  0  0 25  0  0  0  0  0  0  0  0  0  0  0  0
  0 38 39 40  0  0  0 44  0 46 47 48  0 50  0 52 53  0  0  0 57 58 59 60  0  0  0  0  0  0 67 68 69  0  0  0
  0  0  0  0  0  0  0 80  0 82  0  0  0 86 87 88  0 90 91 92 93 94  0 96  0  0 99100  0102103104105  0  0  0
  0110111112

REQUIREMENT     102 IS CONNECTED TO--
  0  2  3  4  0  6  0  0  0 10  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 33  0  0  0
  0  0  0  0 41 42  0  0  0  0 47  0  0 50  0  0  0  0  0  0  0  0 59  0  0  0  0  0  0  0  0  0  0 70  0  0
  0  0  0  0  0  0  0 80  0  0  0  0  0  0  0  0 89  0  0  0  0 94 95  0  0  0 99100101  0103  0105  0  0  0
109  0  0  0

REQUIREMENT     103 IS CONNECTED TO--
  0  0  3  0  0  0  0  0  9 10  0  0 13  0  0  0  0  0  0  0  0  0  0 25  0  0  0  0  0  0  0  0  0  0  0  0
  0 38 39  0  0  0  0 44 45  0  0  0  0 50  0  0 53  0  0  0 57  0 59 60  0  0  0  0  0  0  0  0  0  0  0  0
  0  0  0  0 77  0  0  0  0 82  0  0  0 86 87 88 89 90 91 92 93  0 95  0  0  0 99100101102  0  0  0  0  0  0
109110  0  0

REQUIREMENT     104 IS CONNECTED TO--
  0  0  0  4  0  6  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 27  0  0  0  0  0  0  0  0  0  0
  0 38  0  0  0  0  0  0 45  0  0  0  0  0  0  0  0  0  0  0  0 58 59  0  0  0  0 64 65 66  0 68 69  0  0  0
  0  0 75  0  0  0  0  0  0  0  0  0  0 86 87 86  0  0  0  0  0 94  0  0  0  0  0  0101  0  0  0105  0107  0
109  0  0  0

REQUIREMENT     105 IS CONNECTED TO--
  1  2  0  4  0  6  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 23 24 25  0  0  0  0  0  0  0  0  0  0  0
  0  0 39 40  0  0  0  0  0  0  0  0  0 50  0  0  0  0  0  0  0  0 59  0  0  0 63 64 65  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0  0  0 86 87  0 89  0  0  0  0 94  0  0  0  0 99100101102  0104  0  0  0108
  0  0  0  0

REQUIREMENT     106 IS CONNECTED TO--
  0  0  0  0  0  0  0  0  0  0  0  0  0  0 15  0  0  0  0  0  0  0  0  0  0 26  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 58  0  0  0  0  0 64  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0  0  0  0  0  0  0 86 87  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0108
109  0  0112

REQUIREMENT     107 IS CONNECTED TO--
  1  0  0  0  5  6  7  8  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 26  0 28  0  0  0  0  0  0  0  0
 37 38 39 40  0  0  0 44 45  0  0  0  0  0  0  0  0  0  0  0  0 58  0  0  0  0  0 64  0  0  0  0  0  0  0  0
  0  0  0  0 77  0  0  0  0  0  0 84  0 86 87  0  0  0  0  0 93 94  0  0  0  0  0  0  0  0  0104  0  0  0108
109  0  0  0

REQUIREMENT     108 IS CONNECTED TO--
  1  2  0  0  0  0  0  8  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0 25  0 27  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 44 45 46 47  0  0  0  0  0  0  0  0  0  0 58 59  0  0  0  0 64  0  0  0  0  0 70  0  0
  0  0  0  0  0  0  0 80 81  0  0  0  0 86 87  0  0  0  0  0 93  0  0  0  0  0  0  0  0  0  0  0105106107  0
109  0111  0
```

```
REQUIREMENT    109 IS CONNECTED TO--
  1  2  3  4  0  6  0  8  9 10 11 12  0  0  0  0  0  0  0  0  0 22 23 24  0  0  0  0  0 30 31 32  0  0  0  0
 37 38 39 40  0  0 43 44 45  0  0 48  0 50  0 52 53  0  0 56  0 58  0 60 61  0 63  0 65 66  0  0  0 70 71  0
  0 74  0 76  0  0  0 80 81 82 83  0 85 86  0 88  0 90 91  0 93 94 95 96  0 98  0  0  0102103104  0106107108
  0110111112

REQUIREMENT    110 IS CONNECTED TO--
  1  2  0  0  5  0  7  0  0  0  0  0 13 14  0  0  0  0  0 20  0 22 23  0 25  0  0  0  0 30 31 32  0  0  0  0
  0  0 39  0  0  0  0 44  0 46 47  0 49  0  0 52  0 54 55  0 57  0 59  0  0 62  0  0  0  0  0  0  0  0  0  0
  0  0  0  0  0  0  0 80  0 82  0 84 85 86 87  0 89 90 91  0  0 94  0  0 97 98 99100101  0103  0  0  0  0  0
109  0111112

REQUIREMENT    111 IS CONNECTED TO--
  0  0  0  4  0  0  0  0  9 10  0  0  0  0  0  0  0 18 19  0  0  0 23  0 25 26  0  0  0 30  0  0 33  0 35  0
 37 38  0 40  0  0 43 44  0  0  0  0 49  0  0 52 53 54  0  0  0  0 59 60  0  0  0  0  0  0  0  0  0  0  0 72
  0  0  0 76  0  0  0  0 81 82 83  0  0 86 87 88  0 90 91 92 93 94  0  0 97 98  0  0101  0  0  0  0  0  0108
109110  0112

REQUIREMENT    112 IS CONNECTED TO--
  0  0  0  0  0  6  0  0  9 10 11  0  0  0  0 16 17 18 19 20 21  0 23  0 25 26  0  0  0  0  0  0  0  0 35  0
 37 38  0 40  0  0 43 44 45 46  0 48 49 50 51 52 53 54 55  0  0 58 59  0 61  0 63  0 65 66  0  0  0 70  0  0
  0 74  0  0 77  0  0 80 81 82 83 84 85 86 87 88  0 90  0 92 93 94  0 96 97 98 99  0101  0  0  0  0106  0  0
109110111  0
```

Engineering of Location: The Selection and Evaluation of Trial Grade Lines by an Electronic Digital Computer

JOHN H. SUHRBIER and PAUL O. ROBERTS, respectively, Research Engineer, and Assistant Professor of Civil Engineering, Massachusetts Institute of Technology

One aspect of the highway route location problem, vertical profile selection and evaluation, has been chosen to show how digital computer hardware and software advances now under development can be used to improve the decision-making capabilities of the engineer.

The large number of variables involved complicates the total highway location and design problem. It is difficult to identify all pertinent variables for any particular problem; it is even more difficult to identify their interrelationships. However, these complications are somewhat fewer for the vertical profile phase of the location portion of this problem. In the usual problem, just three cost variables will primarily be a function of the choice of vertical profile. These are the earthwork portion of the initial capital investment and the user fuel and time portions of the continuing costs. The relationship between these variables is such that an increase in earthwork costs usually results in a decrease in user fuel and time costs. One possible criterion for profile selection is to select that profile for which the increase in earthwork cost just equals the decrease in user costs.

A system of computer programs has been formulated and tested which has as its basic input a digital model of the terrain, the horizontal alignment of the road, the typical roadway cross-section, the predicted traffic volumes, and a description of the types of vehicles that will use the road. A vertical profile suitable for project planning and preliminary engineering is automatically selected and evaluated.

The profile selection process is based on a method where a weighted average of the elevations of a range of terrain points, both in front of and behind a given point, is used to determine a trial elevation of the profile at that given point. This trial elevation is then adjusted to account for any control points within the range. The program marches forward from one end of the alignment continuously selecting profile elevations. At each point, the selected elevation is checked against grade, rate of change of grade, and control point restrictions. If these restrictions are not met, the program "backs up" and sequentially adjusts previously selected elevations until all restrictions are met and possible phase errors eliminated. A point elevation is accepted as final only when it is indexed out of the range of points then under consideration. Earthwork volume and user cost computations are performed as the profile elevation for each station is accepted as final.

The vertical alignment generated by the system can be controlled by adjusting either the "look ahead" and "look behind" distances or the shape of the weighting function. In this way, the profile can be varied until the difference in volumes cost just equals the difference in user costs.

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The automatic selection and evaluation of vertical alignments represents one phase of a problem at a particular level in the hierarchy of the complete highway location and design problem. Testing has indicated that not only is a system of this nature feasible and valuable, but also that computer models at the higher levels of analysis are a requirement if an engineer is to perform his work in a professional and responsible manner.

•ONE aspect of the highway route location problem, that of vertical profile selection and evaluation, has been chosen to show how mathematical techniques, when used in conjunction with electronic digital computers, can be used to improve the decision-making capabilities of an engineer. A system of computer programs has been formulated and tested which has as its basic input a digital model of the terrain, the horizontal alignment of the road, the typical roadway cross-section, the predicted traffic volumes, and a description of the types of vehicles that will use the road. With this information, along with certain cost and economic analysis measures, a vertical profile suitable for project planning and preliminary engineering is automatically selected and evaluated.

This paper presents a general discussion of the research which has been performed and of the resulting system of computer programs. Major emphasis is placed on a definition of the profile selection problem, a description of some of the problems which have been encountered in previous attempts to machine-select grade lines, a description of the basic system methodology, a summary of the various computer hardware configurations which might be used to implement the existing system of computer routines, and a brief description of the field tests which have been performed. The paper does not describe the detailed logic of each of the computer routines, nor does it give precise using and operating instructions. It is also not now possible to give a complete discussion of how the system might be used in the solution of a highway location and design problem. Obviously, any research concerned with "computer aided engineering design" has many deep and complicated effects on the way in which engineering will be performed in the future. The full implications of this work cannot be fully known until extensive field utilization of the system has been achieved. The possible uses of these programs will, therefore, be described only in sufficient enough detail to give an idea of the sort of analyses which might be performed.

STATEMENT OF THE PROBLEM

The Highway Engineering Process

In developing a definition of the profile selection problem, it is useful to examine briefly the total highway engineering process in order to place the problem into a real world context and to develop a motivation for its actual solution.

The process of highway engineering is essentially one of decision-making, involving operations upon information and culminating in a selected course of action. These operations include identifying goals and decision criteria, searching for a set of alternatives, predicting the physical consequences of each alternative, evaluating these consequences according to a value scheme, and deciding by means of some decision-making scheme to either accept an existing alternative or to continue the search process (Fig. 1).

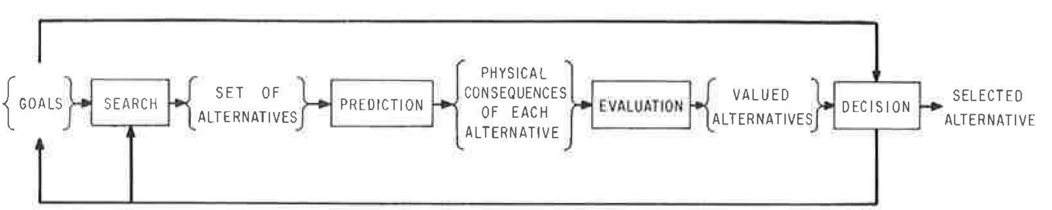


Figure 1. Schematic representation of the engineering process.

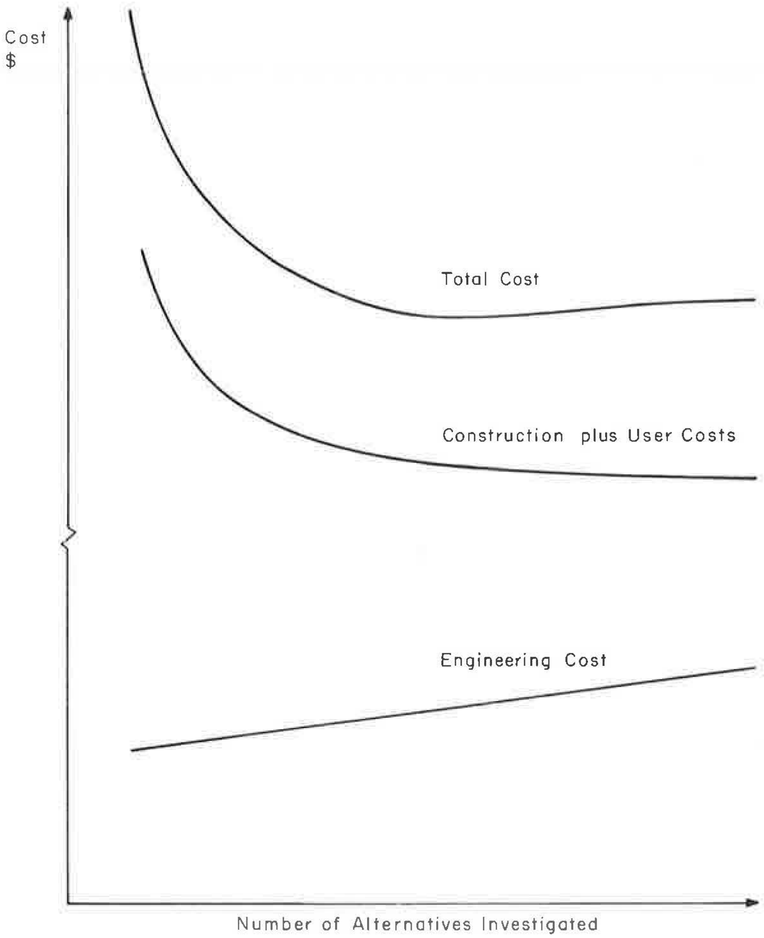


Figure 2. The cost and benefits of improved engineering.

Further examination of the highway location and design problem shows that it is hierarchical in structure. Typical of the higher levels of analysis are decisions such as choice of mode and geographical areas to serve. At somewhat lower levels, there are the questions of optimum network configuration involving the selection of bands of interest between individual nodes of the network. Still lower decision levels are concerned with the planning of specific projects and with the preliminary location of links within these bands of interest. The lower levels of this hierarchy are represented by preliminary engineering design and by the preparation of design plans and specifications. A characteristic of each level in this hierarchy is that it can be broken down into the five engineering phases. Each level is concerned with goals, search, prediction of consequences, evaluation, and decision.

An efficient and economical transport system is achievable only by good engineering at each level in this hierarchy. Highway engineers must employ engineering methods which produce designs which are consistent between the various levels and whose results at any one level can be directly and usefully presented to other levels in the design sequence. Suboptimization is meaningless unless there is feedback, involving both review and re-evaluation, at each level of the design process.

There are, at each stage in the hierarchy, an extremely large number of alternative courses of action open to an engineer. There is virtually no way to investigate them all, nor do they all necessarily need to be investigated. Figure 2 shows an idealized form of the relationship between engineering, construction, and user costs. Although the plot is hypothetical, it indicates what is felt, and hoped, to exist in the engineering world. As more alternative designs are investigated, higher engineering costs are incurred, but the payoff for these higher engineering costs is a lower "total cost" solution. One of the difficulties with current engineering methods is the impossibility of predicting with any certainty the quality of an alternative which is produced by additional search activity. This quality, or "cost," is a function of the particular problem under investigation, the amount of search activity already completed, the skill of the engineer, and the methods by which these additional alternatives are being generated. The development of new means for rapidly and selectively generating, evaluating, and improving alternative alignments, particularly at the higher analysis levels, would increase the productivity of an engineer and would increase his ability to examine systematically more of the literally millions of possibilities which do exist (11).

The highway design phases have received the majority of recent research efforts. Systems of digital computer programs now exist which permit the detailed analysis of earthwork quantities and which facilitate computation of right-of-way, interchange, and superelevation geometrics. On-line plotters are beginning to be used for the automatic preparation of plans. Vehicle operating costs and travel times can be examined by means of either tables or charts or by simulation programs. However, the analysis levels concerned with project location and planning and also with overall system design have received somewhat less attention. Simple cost per mile figures are often used to evaluate construction cost. Vehicle operating and travel time costs are still only rarely considered. Very few alternatives are typically investigated because of the limitations imposed by hand methods. Since the costs of a project are almost completely determined at these higher analysis levels, it is these levels that should receive future research interest and to which attention is turned in this paper.

The Profile Selection Problem

A highway can be described as a curve in three dimensional space. To describe this curve fully, it is necessary to specify, either explicitly or implicitly, three coordinates for every point on this curve. In usual highway engineering practice, this is done by identifying a series of horizontal lines and curves defining a vertical curved surface on which a set of vertical lines and curves can then be defined. These two sets of lines and curves are known as the horizontal and vertical alignments, respectively.

The selection of a spatial location for this three dimensional curve, along with the decision as to the volume level of traffic to serve, determines the costs associated with transportation along this highway. These costs can be separated into capital, or first, costs and into operating, or continuing, costs. Capital costs include the initial expenditures for land, structures, earthwork, pavement, drainage, and interchanges; continuing costs consist of the expenses to the highway user and the expenses required to operate the facility. In order to use a road, a user incurs time, fuel, tire, oil, as well as vehicle maintenance and depreciation costs. Continuing costs associated with the highway itself include maintenance, snow removal, police patrol, and administration.

The relationships of these cost variables to the horizontal and vertical locations and to traffic volume, while not being completely defined, are understood well enough to permit some general statements to be made. The decision as to the level of traffic to serve specifies pavement width and number of lanes. Right-of-way and pavement costs are primarily a function of horizontal location only. In addition, such cost variables as structures, drainage, and road maintenance, although affected by vertical alignment, are largely determined by the horizontal location. This leaves earthwork, vehicle fuel, and user time costs as the cost variables that have a large dependence upon vertical alignment. These same three cost variables generally constitute the largest percentage of the total "cost" of a highway project. A summary of these cost relationships is given in Table 1.

Vehicle fuel and user time costs can be combined into a single term called "user cost." The implication of the preceding paragraph is that if traffic volumes are as-

TABLE 1

COST VARIABLE RELATIONSHIPS

Cost Variable	Function of Horizontal Alignment	Function of Vertical Alignment
Earthwork	X	X
Structures	X	CP[1]
Pavement	X	
Drainage	X	?[2]
Interchange	X	CP
Relocation	X	
Right-of-way	X	
Fuel	X	X
Tire	X	
Oil	X	?
Vehicle maintenance	X	
Vehicle depreciation	X	
User time	X	X
Road maintenance	X	?
Accident	X	?
Political	X	
Social	X	
Aesthetic	X	

[1] Cost variable is usually accounted for through use of a control point.
[2] Effect of vertical alignment is considerably less than that of the horizontal alignment.

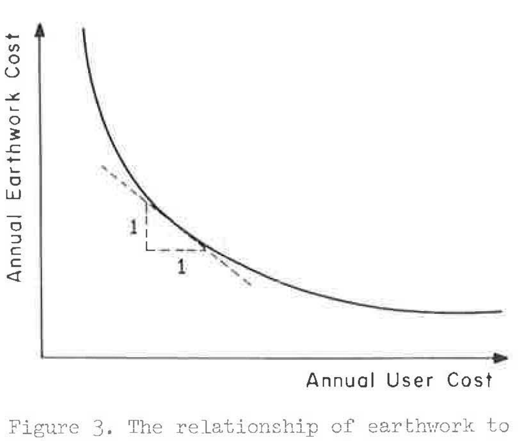


Figure 3. The relationship of earthwork to user time and fuel cost for a horizontal alignment alternative.

sumed fixed and once a single horizontal alignment is selected, the various vertical alignments which might be designed over this alignment may be ranked by considering only these two major variables, earthwork cost and user cost. The vertical alignment for which the sum of earthwork and user costs is minimum might be considered to be the "optimum" profile. At least the grade line is balanced in the sense that increased money spent on earthwork will not produce a corresponding savings in user cost.

This decision criterion can be demonstrated in a slightly different fashion. A theoretical plot of annual earthwork cost vs annual user cost for a range of various vertical alignment possibilities is shown in Figure 3. Points with high user costs and low earthwork costs correspond to highway profiles that follow the ground quite closely, while points having low user costs and high earthwork costs correspond to relatively smooth or flat grade lines that tend to deviate rather markedly from the existing ground profile. This plot indicates that for any one horizontal alignment, there is a point for which the decrease in user cost exactly equals the increase in earthwork cost. In theory, the vertical alignment selected by the engineer should correspond to this point, or at least be very close to it.

The highway profile must generally meet certain engineering restrictions. These restrictions are commonly associated with grades, rate of change of grade, and control points. Grade restrictions determine the maximum positive and negative grades that can exist on a road. The purpose of these restrictions is, ordinarily, to account for the relationships shown in Figure 3; that is, to prevent grades which result in excessively high operating costs caused by vehicle slow downs or hazardous operating conditions. Rate of change of grade restrictions provide adequate passing and stopping sight distances. Bridge elevations, railroad crossings, and major river crossings are typical examples of points where vertical control might be imposed. Control points then represent locations along the profile where the roadway should be at a specified elevation. The vertical alignment sought is that which results in the best balance of earthwork and user costs, yet meets the restrictions imposed by grade, sight distance, and control points.

The highway location and design problem has been described as being hierarchical in structure, and the earthwork and user costs have been shown to be critically related to the vertical profile. These two ideas can now be examined together to see what the role of a profile selection capability might be. Profile selection becomes important at that level where the actual link locations are being considered. At higher levels of analysis, decisions are more concerned with questions such as modal split and system planning. The vertical profile is ordinarily unimportant here. At the project planning or project location levels, link locations are considered in more detail. Within the band of interest of an individual link, there typically will be a large number of possible horizontal alignments, each having a large number of feasible vertical alignments. With present methods, an engineer either uses a simple cost per mile estimate which ignores the vertical profile altogether or he is forced to use basically the same method as at the final design level. A horizontal alignment is specified by hand and input to a computer program. The machine then calculates the horizontal geometry and plots the ground profile. After a vertical profile is chosen by hand, additional machine passes are required to complete the evaluation. Thus, almost the same amount of engineering effort is required to investigate an alternative alignment at the project planning and location phases as at the preliminary or final design phases.

The method with which an engineer selects a vertical profile is quite complicated and, at present, not completely understood. For this reason, it may be best to continue to hand-select the final vertical profile. However, at the higher levels of analysis where accuracy requirements are not so tight, this essentially two-pass selection-evaluation procedure seems inefficient. It would be desirable to have a method of analysis which would permit the engineer, by merely specifying the horizontal alignment, to obtain a rapid and reasonably accurate evaluation of this alternative.

Previous Work on the Profile Selection Problem

The desire to select automatically a highway profile is not new. It has existed, in fact, since the introduction of electronic digital computers to the highway engineering field. A number of the past efforts in this area were examined to gain an idea of the various approaches that had been taken towards this problem and to learn and understand the major problems that had been encountered.

Most of the early work was directed toward the area commonly referred to as reconnaissance or preliminary engineering. The majority of the approaches examined, in essence, made a least squares fit of a series of consecutive and continuous polynomial curves (first, second, or third degree and combinations thereof) to a set of terrain profile points subject to grade and control point restrictions.[1] This does not imply that these approaches are all similar in detail or even in basic method. They differ as to the objective function employed (if, indeed, one is used at all), the manner in which control points are handled, the method by which grade restrictions are satisfied, and the computer configurations required. Other studies have been performed in the area of automatic profile selection, the most notable of which is probably the application of the calculus of variations. However, these studies have not yet had completely acceptable results, nor have they proven to be economically feasible for use by public highway departments.

[1] Basically, these procedures start from an origin point which has specified initial elevation and slope conditions. A least squares fit, taking account of control points, of a curve is made to the terrain points for a specified range of fit. This range of fit can extend either in front of and behind the origin point, or just in front of the origin point. The resulting curve is used to compute the highway grade at the station of the next terrain profile point ahead. If the slope does not exceed the designated limits, the curve is accepted and the elevation of the highway at this next station is computed. If the slope restrictions are not satisfied, the appropriate limiting slope is used and the curve corrected before the highway elevation is computed. The highway elevation and slope at the new station are then taken as the new initial conditions and the curve fitting process repeated at the new point. The computations systematically step ahead, yielding a continuous highway grade line.

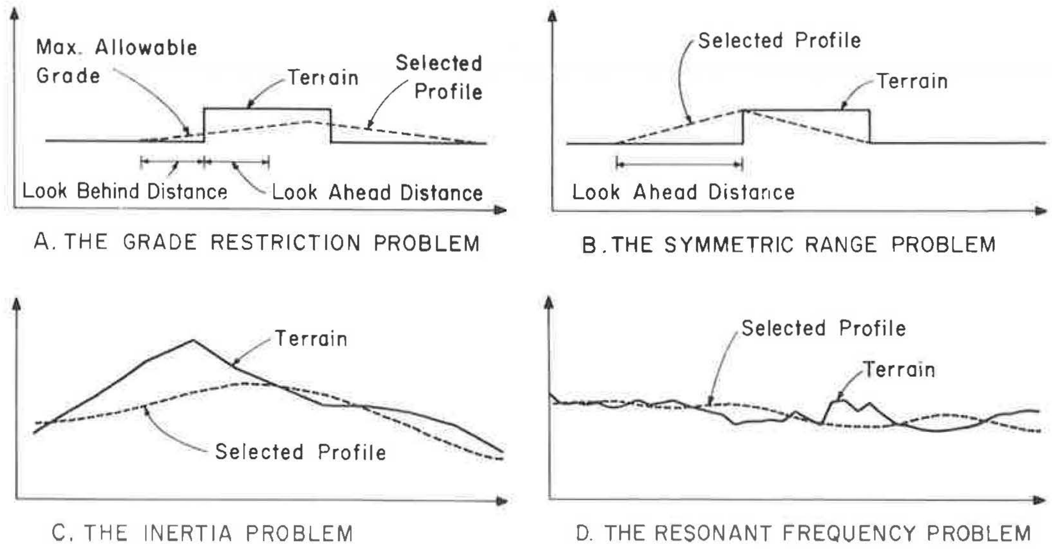


Figure 4. Summary of major problem areas in the automatic selection of highway profiles.

The following is a summary of the four major problems that have been encountered in the curve fitting type of profile selection techniques:

1. The grade restriction problem. Relatively good results can be obtained in gently rolling terrain. However, as the roughness of the terrain increases and grade restrictions are imposed, the resulting highway profile is skewed or biased to the right. This bias occurs for several reasons. The hills and valleys are often "seen" too late by the range of fit ahead to permit the highway to alter its course in time. This problem is compounded when grade restrictions set in, because the highway can only rise or fall at a specified maximum slope. If the terrain condition is seen too late and the grade restrictions are such that the terrain is falling or rising at a slope which the highway profile cannot match, the selected highway profile is bound to be skewed to the right (Fig. 4A).

2. The symmetric range problem. A second type of bias or skew is introduced when only terrain points ahead are taken into account in determining the highway grades and profiles. However, this will tend to skew the resulting profile to the left instead of to the right. This can be easily seen in the case of the "rectangular mountain" (Fig. 4B). By the time the roadway reaches the mountain, it will have already reached the highest elevation, since the only terrain points under consideration at this time will be those having elevations equal to the highest elevation. Similarly, the roadway elevation will have already returned to original ground level by the time the end of the mountain is reached.

3. The inertia problem. Whenever previous roadway elevations and grades are taken into account in determining the elevation and grade of other stations, there is a tendency for the selected profile to be skewed to the right (Figure 4C).

4. The resonant frequency problem. A "resonant frequency" type of problem can occur under special conditions. When the predominant wave length of the terrain profile is approximately equal in length to the total range of terrain points under consideration, an oscillation develops in the selected profile (Figure 4D).

METHODOLOGY OF AN AUTOMATIC PROFILE SELECTION-EVALUATION SYSTEM

The System

The logic of an integrated profile selection-evaluation system is shown in Figure 5. The system calculates the horizontal geometry of an alignment, generates and plots the

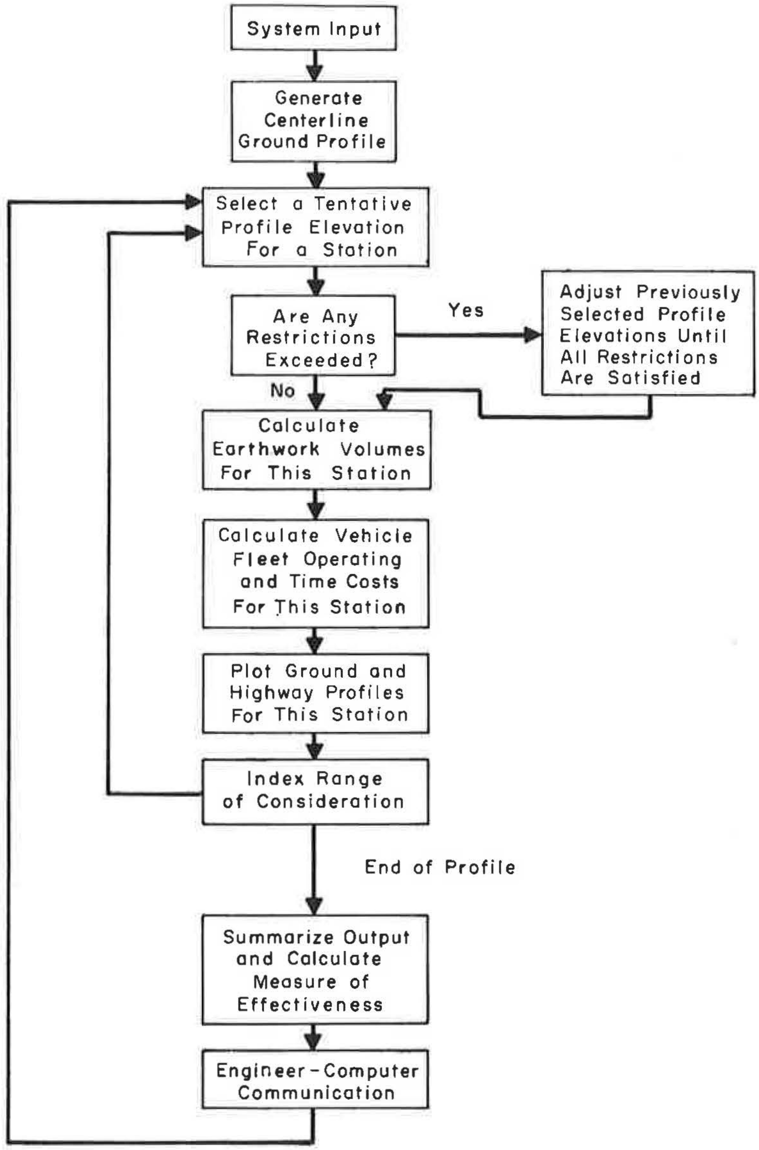


Figure 5. Logic of an integrated profile selection-evaluation system.

centerline ground profile, selects a tentative highway profile by means of a mathematical smoothing technique and plots it over the ground profile, calculates the earthwork volumes associated with this profile, simulates the operation of vehicles over this alignment to determine operating and time costs, summarizes the results in the form of total and annual costs, and finally asks the engineer whether he would like to try to improve the selected profile in order to obtain a better balance of earthwork and user costs. Calculations are based on the theory of the digital terrain model (DTM) and employ the routines and programs presently existing in the DTM location system.

The profile selection process is based on a method whereby a weighted average of the elevations of a range of terrain points, both in front of and behind a given point, is used to determine a trial elevation of the profile at that given point. This trial elevation is then adjusted to account for all control points within the range. The program

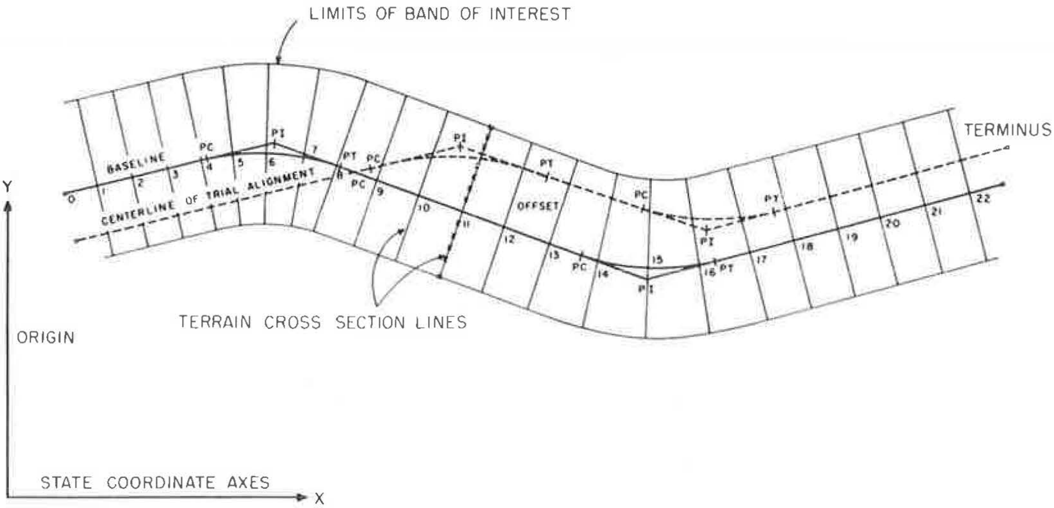


Figure 6. Digital terrain model showing baseline and terrain cross-section lines.

marches forward from one end of the alignment continuously selecting profile elevations. At each point, the selected elevation is checked against grade and control point restrictions. If these restrictions are not met, the program "backs up" and sequentially adjusts previously selected elevations until all restrictions are met and possible phase errors eliminated. A point elevation is accepted as final only when it is indexed out of the range of points then under consideration (see Fig. 10). Earthwork volume and user cost computations are performed as the profile elevation for each station is accepted as final.

The following sections describe each of the major computational blocks of Figure 5 and indicate how each is related to the overall goal of profile selection and evaluation.

Digital Terrain Model

Fundamental to an understanding of the complete profile selection-evaluation system is an understanding of how the terrain model is formed. The DTM is simply a sampling of the continuous surface of the ground by a number of selected terrain points with known X, Y, and Z coordinates in an arbitrary coordinate system. Terrain data are defined relative to a baseline (X-axis) and are taken along lines normal to this baseline called cross-section or scan lines (Y-axis) (Fig. 6). The reason for relating terrain data to a fixed baseline rather than to a centerline is that, for a new trial horizontal alignment, it is relatively easy to re-establish the relationship of the new centerline to the baseline. This enables the model to be used repeatedly for the fast evaluation of many different trial lines during planning and location studies without the necessity of retaking data.

Before the terrain data are taken, a band of interest is selected by the engineer. The width of this band varies with the amount of latitude that the location affords. This band of interest is then digitized for use by the computer.

The first step in the digitization process is the definition of a baseline within the band of interest. This serves as the X-axis of the data coordinate system and can be composed of either straight line segments or straight line segments connected by curves, depending upon the shape of the band of interest. Next, the surface of the ground is represented by a series of points (Fig. 7). Sample points are taken left to right across each cross-section and are referenced by giving the baseline station number (X-coordinate) of the cross-section line, their offset distance from the baseline (Y-coordinate), and their elevation (Z-coordinate). Points to the left of the baseline are recorded with a negative sign, while points to the right are considered to have a positive offset.

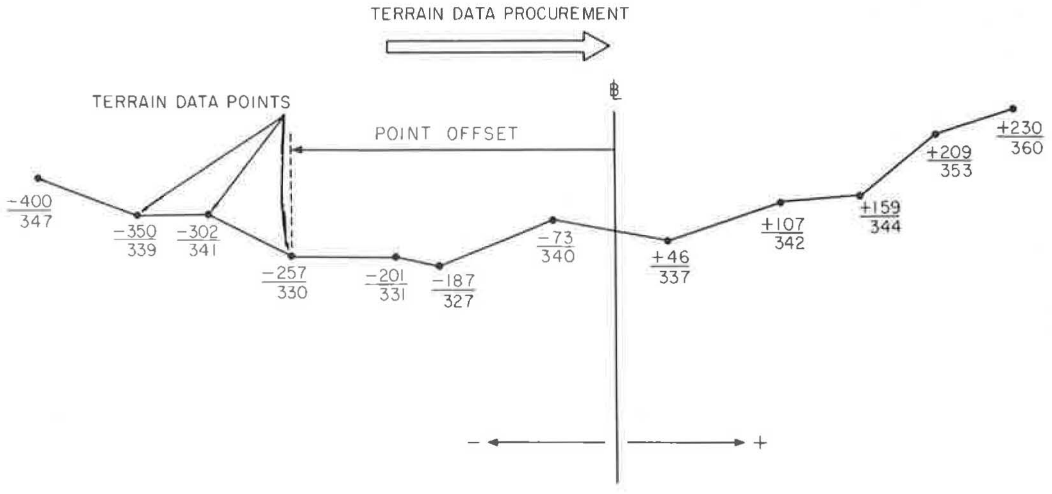
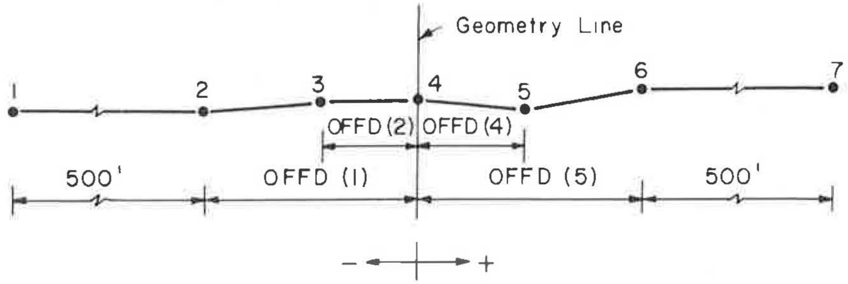


Figure 7. Typical cross-section showing definition of model by terrain points.



Note: OFFD (3) IS DEFINED AS HAVING AN OFFSET OF 0.0

Figure 8. Definition of approximated terrain section.

The spacing of the scan lines and the sample density of terrain points along a scan line depend upon the accuracy required, the maps available, and the nature of the terrain.

Terrain data for the DTM can be taken directly from field notes, from topographic maps either by hand or by the use of special instrumentation, or directly from the stereo model using automatic take-off equipment. Terrain data are taken only once and can be used repeatedly for the evaluation of many lines.

Location Alignment Program

The location alignment program calculates the geometry of the highway centerline alignment and determines its relationship with the baseline to which the terrain data are referenced. Once this relation has been established at each cross-section, an approximated terrain model for use in the actual profile selection-evaluation program can be interpolated from the real terrain model (Fig. 8).

Both the baseline and centerline are defined by the state plane coordinates of each P.I. and one curve defining parameter given at each P.I. The location alignment program computes the geometry of the defined baselines and of corresponding centerline alignments. This geometry information is then used in computing the offset distances from the baseline to the centerline. The approximated terrain model is generated by interpolating terrain data on each cross-section to find the ground elevation along the centerline of the roadway and along four parallel alignments whose offset distances from the centerline have been specified by the engineer.

A more detailed description of the operations of the various program phases is given in the program manual of the DTM location system (19).

Profile Selection

The selection of a trial highway profile is done by a heuristic which may be thought of as attempting to model or simulate the actions of an engineer by determining a weighted elevation of terrain points within a range of interest and then adjusting this elevation to account for the control points. The mechanics of the profile selection and grade adjustment procedure are indicated in Figure 9 and will be described as a series of iterative steps. The range of terrain points and associated terminology are shown in Figure 10. The range has been indexed so that point 0 is the middle point; it is referred to as the origin point.

1. The search point is initially set equal to the origin point and a trial profile elevation for the search point station is determined by calculating a weighted elevation of all terrain points within the current range of interest and then adjusting this elevation to account for each of the control points within the current range.
2. Both the grade and the rate of change of grade of the highway between the search point station and the station of the immediately preceding point are calculated. If both are acceptable and the current search point is point 0, go to step 3. If both are acceptable and the search point is somewhere between point -n and point -1, go to step 4. If either is unacceptable and the search point is any point except point -n, go to step 5. Finally, if either does not satisfy the appropriate restriction and the search point is point -n, go to step 6.
3. Since all grades and all rates of change of grade are acceptable, the profile elevation of point -n is accepted as final and the total range of terrain points is indexed so that point -n passes out of the range of terrain points under consideration and point 1 becomes the new point 0. Calculations are returned to step 1.
4. All grades and rates of change of grade on the selected profile from the current search point station to the origin point station are checked. If all are acceptable, go to step 3. If any grade or rate of change of grade does not satisfy the restrictions and the search point is not point -n, go to step 5. If some grade or rate of change of grade is determined to be unacceptable and the current search point is point -n, go to step 6.
5. If a particular trial grade or rate of change of grade does not satisfy the given restrictions, this is an indication that a terrain configuration in front of the search point was "seen" too late to permit the use of an acceptable design standard. To correct this, the search point is moved back one point (from point 0 to point -1 or from point -j to point -(j + 1) and step 1 repeated. In effect, this throws out the previously accepted profile elevation for this station and a new adjusted elevation is determined.
6. The search point has reached the back of the range and additional "backing up" is not possible. Therefore, any grade or any rate of change of grade between the search point station and the origin point station that exceeds the design restrictions must be set equal to the appropriate maximum positive or negative value. When all

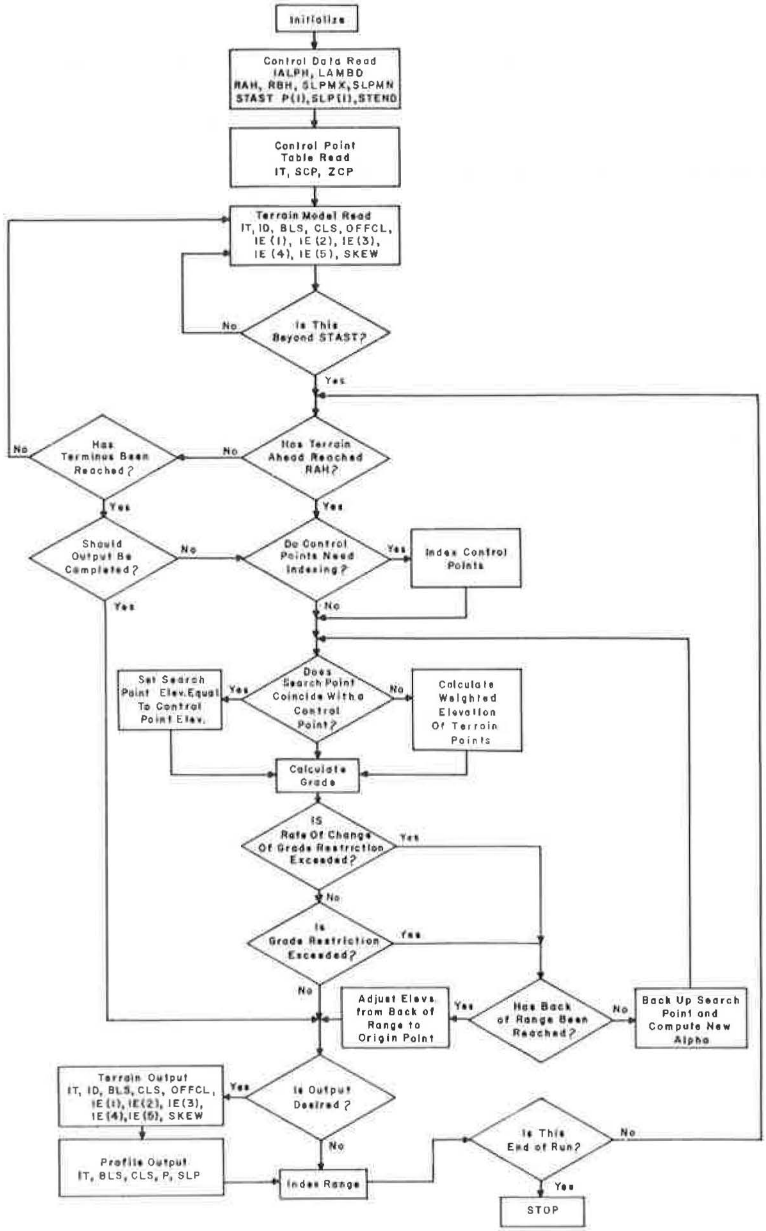


Figure 9. Macro flow diagram profile selection.

unacceptable conditions have been corrected, the range of terrain points is indexed so that point 1 becomes point 0. The search point is again set equal to point 0 and a new iteration is begun with step 1.

A profile selection procedure of the above nature must specify starting and stopping conditions. The manner in which these are handled is indicated in Figure 11.

Case 1: The information for the range of terrain points ahead of the initial origin point is read in and stored in memory before any profile selection calculations are made. The lengths of the range ahead and the range behind are specified by the engineer.

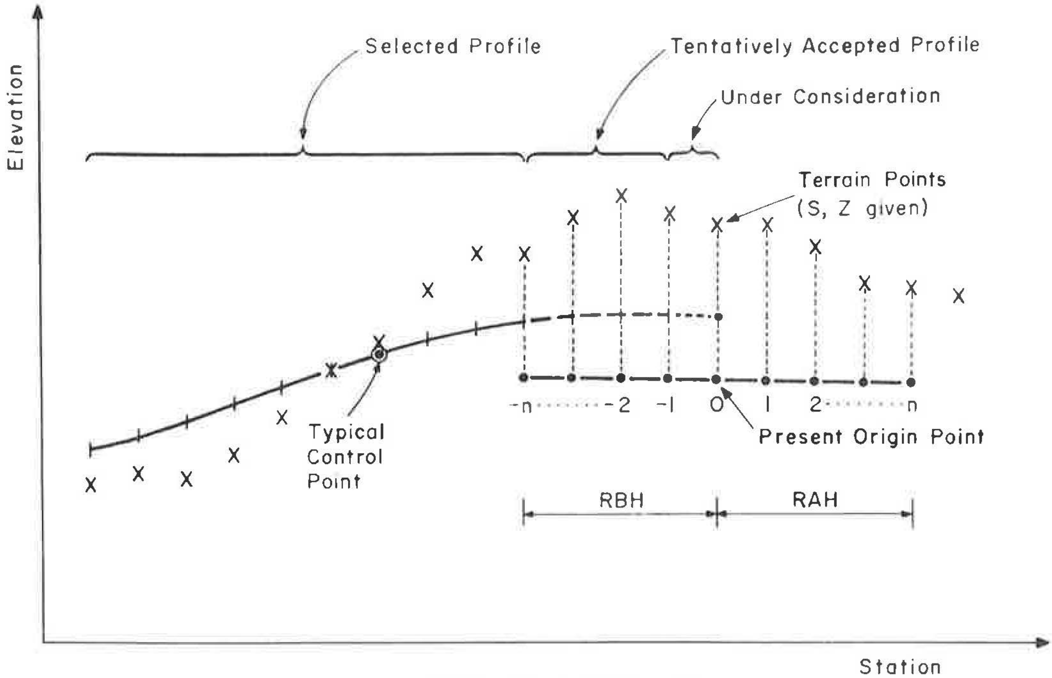


Figure 10. Range indexing procedure.

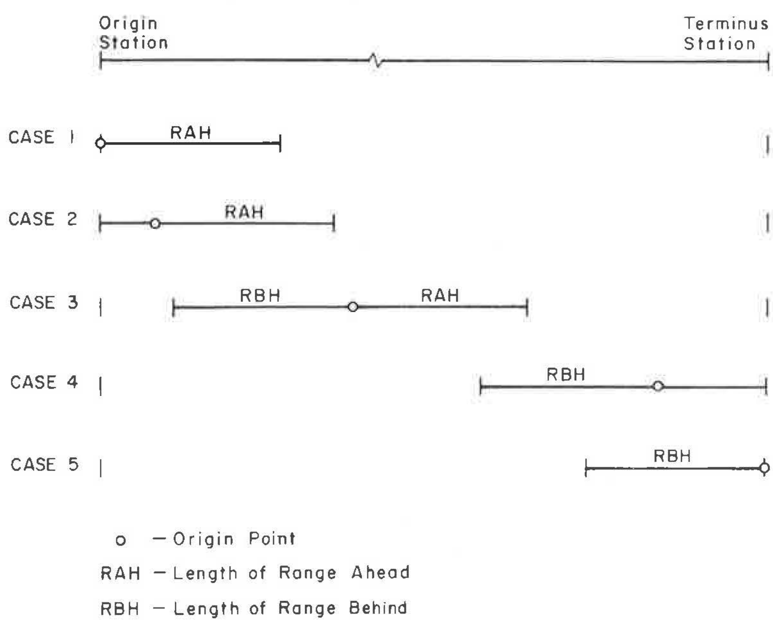


Figure 11. Starting and stopping procedures.

Case 2: As the origin point moves away from the initial or origin station, the range of terrain points behind the origin point is slowly built up until it reaches its full length.

Case 3: This is the normal operating procedure and is the case assumed in the description of the profile selection procedure.

Case 4: As the front of the range ahead reaches the terminus point, additional terrain information is not available to be read into storage. The origin point is indexed ahead in the normal manner, except the front of the range ahead stays fixed at the terminus point.

Case 5: When the origin point reaches the terminus point, the algorithm has reached an end. The elevations and stations for the selected profile between the back of the range behind and the origin point are output.

The initial selection of a tentative profile elevation is done by calculating a weighted elevation of the terrain points within the present range of interest. This initial elevation is then adjusted to satisfy the control point elevation constraints. A weight for each of the terrain point elevations is determined by the equation:

$$W = \frac{1}{\beta(\alpha+1, \lambda+1)} x^{\alpha} (1-x)^{\lambda} \qquad (\alpha, \lambda > -1; 0 \leq x \leq 1)$$

where

$$\beta(\alpha+1, \lambda+1) = \frac{\Gamma(\alpha+1)\,\Gamma(\lambda+1)}{\Gamma(\alpha+\lambda+2)}, \text{ and}$$

$\Gamma(n)$ denotes the generalized form of the gamma function

$$\Gamma(n) = \int_0^{\infty} x^{n-1} e^{-x} dx \qquad (n > 0)$$

The variable x varies from 0 to 1. The back of the range behind the origin point is denoted as 0 and the front of the range ahead is denoted as 1. Stations between these two points have a fractional x value. The weighted elevations for the terrain points are added and their sum divided by the sum of the terrain point weights to determine a trial profile elevation.

The equation used to determine the terrain point weights is identical in form to the probability density function of the beta distribution. The effects of the parameters on the shape of the terrain point weighting function curve are shown in Figure 12. The special case $\alpha, \lambda = 0$ yields the rectangular distribution, while if one parameter is 0 and the other 1, the triangular distribution is obtained. The curve is U-shaped if both parameters are negative, J-shaped if only one is negative, and unimodal with the mode at $x = \alpha/(\alpha+\lambda)$ if both are positive. For the latter case, the curve is symmetrical if $\alpha = \lambda$, skewed to the left if $\alpha < \lambda$, and skewed to the right if $\alpha > \lambda$. One of the major reasons for selecting this form of the terrain point weighting function was the large flexibility available in the shape of the curve. The effects of different shapes could be determined and the skew of the curve could be controlled so that the search point elevation always receives the largest weight.

The beta function cannot be integrated formally from 0 to x unless α and λ are both integral multiples of $\frac{1}{2}$. The profile selection computational procedure assumes that the values input to the program meet this restriction. All adjustments of α, λ within the program are made so that each will continue to be a multiple of $\frac{1}{2}$.

Control point elevations are introduced by adjusting the weighted terrain point elevation once for each control point within the current range of interest. The equation used for this adjustment is:

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ADJUSTED ELEVATION = (WEIGHTED TERRAIN POINT ELEVATION) X FACTOR
                   + (CONTROL POINT ELEVATION) X (1.0 - FACTOR)
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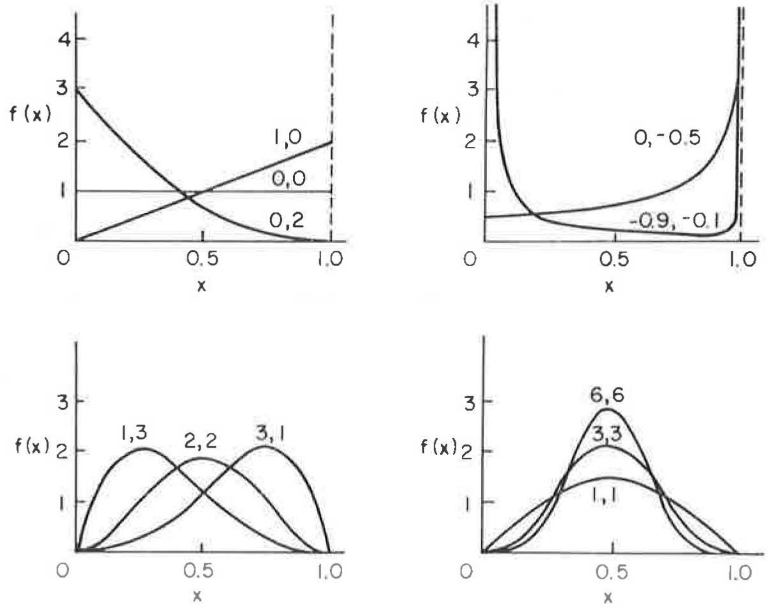

Figure 12. Beta distributions with various selections of parameters α, λ; the symbol a, b means α = a, λ = b (From "Introduction to Probability and Random Variables" by Wadsworth & Bryan, Copyright 1960, McGraw-Hill Book Company, Inc.).

where FACTOR equals the absolute value of the difference between the control point and the search point stations divided by the difference between the search point station and the front of the range of interest. A control point is thus given linearly increasing importance as it nears the search point. As a control point enters the range of interest, its weight will be 0; as the search point gets progressively closer to a control point, the control point weight increases to 1.0; and when the stations of the control and search point are equal, the weighted terrain elevation will be ignored completely since the FACTOR value in this case will be 0. If more than one control point is within the range of interest, they are accounted for in inverse order as their distance from the search point station increases. Thus, the farthest control point is accounted for first and the closest is accounted for last in adjusting the weighted terrain point elevation.

This approach to the automatic selection of highway profiles eliminates three of the four major problems that have been encountered in previous work. The immediate history of the roadway is ignored in computing new profile elevations, thereby eliminating inertia effects and its associated right-hand skew. Grade and rate of change of grade restrictions are not satisfied by simply setting excesses to the appropriate maximum positive or negative value. When a restricted area is encountered, the selection algorithm backs up until all grades and rates of change of grade have been adjusted to acceptable levels. Restricted values are used only if this backing up continues until the back of the range of terrain points presently under consideration is reached. This back up procedure will tend to eliminate the right-hand bias resulting from seeing a terrain configuration too late. Terrain points ahead and behind the search point are used in determining the profile elevations. Problems created by using an unsymmetrical influence zone are, therefore, eliminated. Problems associated with resonant frequency can still occur. However, if the length of the range of terrain points being considered is greater than the wave length of all objectionable oscillations in the profile, resonant frequency should not be a problem. Oscillations may still occur, but since their wave lengths will be greater than or equal to the length of the influence zone, these oscillations have been implicitly approved of by the engineer.

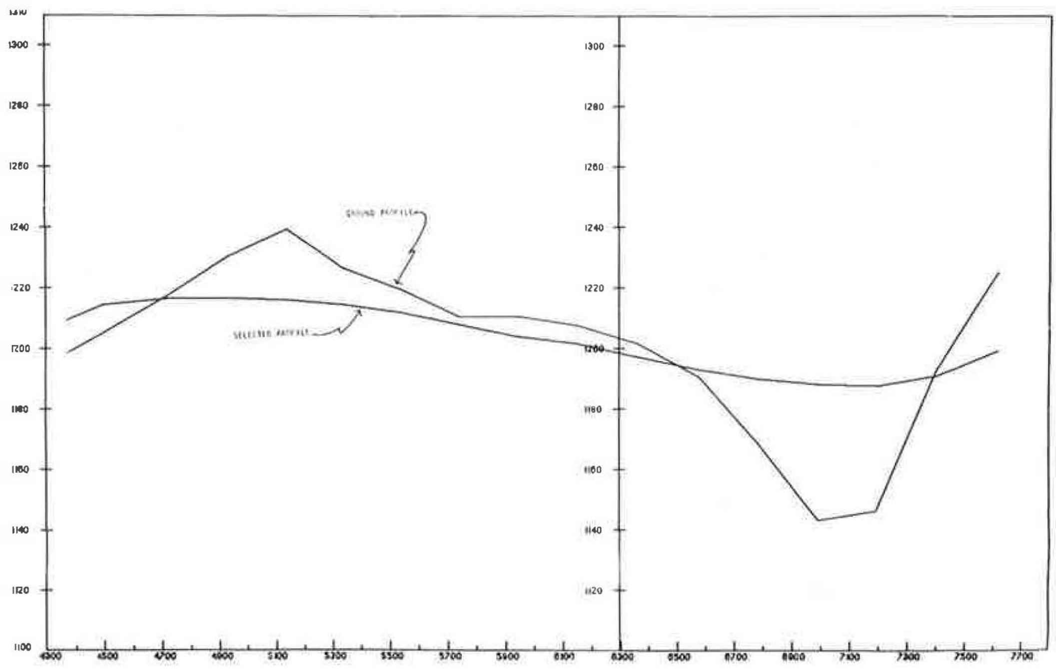

Figure 13. Example of an automatically generated highway profile.

System criteria are satisfied by this selection procedure. If the selected profile is to be iteratively improved upon, it is necessary to have a way in which the smoothness or quality of the profile can be controlled. It is possible in this procedure to adjust both the lengths of the ranges ahead and behind the origin point and the parameters α and λ.

Examples of profiles generated by this routine are shown in Figures 13 and 14.

Roadway Design and Earthwork Volumes

As the selected profile elevation for each station is indexed out of the range of terrain points under consideration, roadway design and earthwork volume computations are made for this cross-section. Using the approximated terrain model generated in the location alignment program and the profile grade chosen by the profile selection procedure, the roadway template is constructed from the defined template links and parameters, the slopes are selected according to specified design criteria, and the slope intercept and earthwork volume calculations made. Output consists of the total accumulated cut and fill volumes for the entire job with the slope stake and volume information for any particular station available at the option of the engineer.

The template is composed of a series of links, each defined by dy and dz distances, and a series of low and high cut and fill slopes (Fig. 15). The five-point approximate terrain model from the location alignment program is used as the terrain description in the template construction and earthwork computations. This model contains five offsets and elevations for each cross-section. Normally, two of these are to the right of the centerline, two are to the left, and one is the centerline. An additional two points, which actually make this a seven-point terrain model, are defined internally by the routine. These are 500 ft outside each of the two exterior offset points and are at the same elevation as the corresponding exterior offset point. This approximate terrain model, rather than the actual DTM, is used for two reasons: first, it results in a large increase in running speed with only a slight reduction in accuracy over the more exact terrain model. This is important since this is only one routine of a much larger pro-

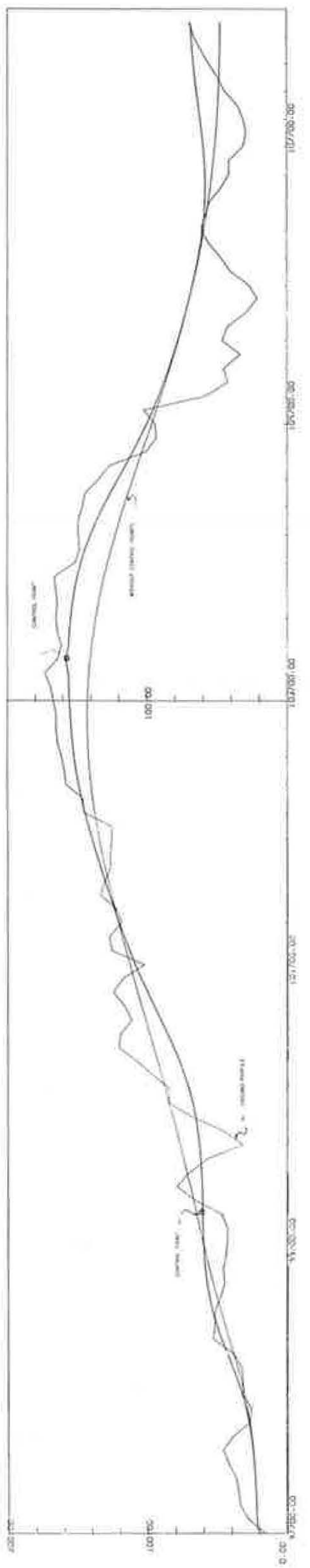

Figure 14. Examples of automatically generated highway profiles.

gram system. In addition, it is felt that this lower accuracy is acceptable during preliminary location work. Second, by internally defining the two extreme points, the engineer can select arbitrary offsets from the profile grade line without having to worry about whether or not the template slopes will intercept within the defined terrain.

Vehicle Fleet Operating and Time Costs

An engineer, in performing an analysis of a particular highway alternative, is interested in the operating and time costs incurred by the vehicle fleet for several reasons. In order to compare this alternative with others, the costs to the road user must be measured and compared with those of other alternatives. In the selection of a vertical alignment an engineer almost always, either implicitly or explicitly, makes an effort to balance construction and user costs. Lastly, if a highway is to serve as an efficient transport link, it must permit the individual vehicles using the facility to operate efficiently, i.e., vehicles, especially trucks, must be able to perform satisfactorily. While the output necessary for the profile selection-evaluation feedback loop is only the total annual user costs, it is also desirable to be able to obtain, at the option of the engineer, information concerning the operation and performance of various types of vehicles.

A considerable amount of work has been done in the field of vehicle performance prediction. These efforts fall into essentially three categories: (a) the tabulation of experimental data into tables and graphs, (b) the statistical analysis of data to construct regression equations, and (c) the development of systems of performance equations which can be solved by either hand or computer to give vehicle performance and operation.

In the usual table, cost per mile figures for fuel, oil, tires, maintenance, and depreciation are given as functions of profile grade, pavement condition, and vehicle class. These are generally empirical studies and give cost information only. Since these figures will not necessarily apply to vehicles manufactured outside the country in which the study was made and since an actual speed profile is not obtainable, this method of user cost pre-

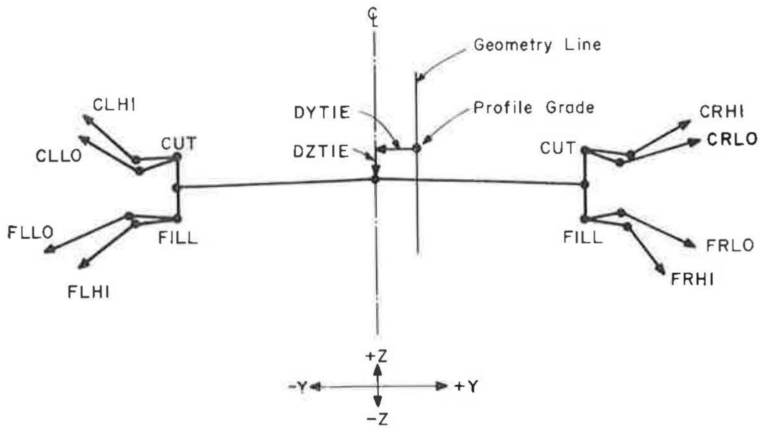

Figure 15. Definition of template.

diction is not considered to be completely satisfactory for a profile selection-evaluation system.

Because of the complexity of vehicle performance, statistical studies have necessarily been limited to rather special cases. For example, the cost per mile is given as a function of velocity for various vehicle classifications or fuel consumption for the acceleration of gasoline or diesel trucks is given as a function of magnitude and length of grade. To date these studies have not been general enough in nature to permit them to be used for the prediction of vehicle fleet performance over a specific alignment.

Simulation techniques have typically been of two types: those employing a digital computer to solve the system equations, and those employing a graphical or hand method of solution. Of necessity, the hand solutions have been significantly more limited in flexibility than the computer-oriented approaches.

The basis of any simulation is a determination and calculation of all forces acting on the vehicle (Fig. 16). The forces resisting the movement of the vehicle are air resistance, rolling resistance, and grade resistance. Rolling resistances are those forces inherent in the vehicle itself that tend to retard its motion. The grade resistance

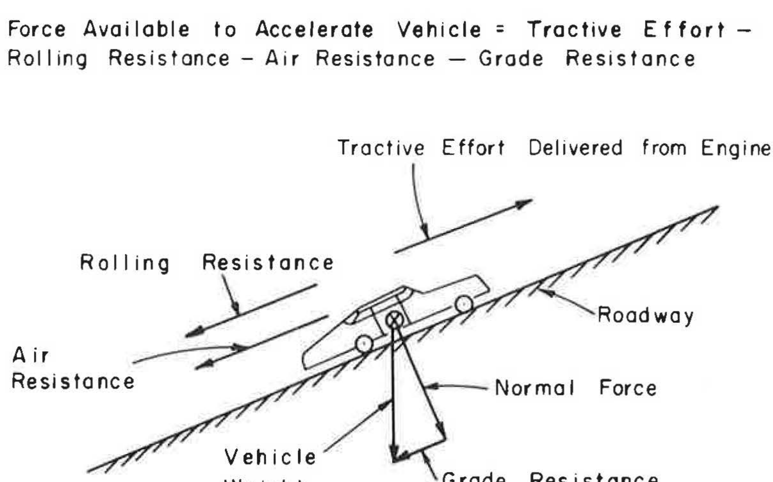

Figure 16. Simplified free body diagram of vehicle and roadway.

TABLE 2

FEATURES OF VEHICLE OPERATING COST ROUTINE

Cars and trucks	Yes
Torque converter	No
Shifting deceleration	Yes
Gear shift oscillations	Yes
Engine power curve	Torque vs rpm
Fuel prediction	Gen'l fuel map
Speed changes and stops	No
Chassis friction	% Efficiency
Turn around	Yes
Independent variable	Velocity
Effect of temperature	No
Horizontal curvature	No
Vertical alignment	Tangents only
Station equation	No
Summary information	Yes
Output comments	Yes
Data control cards	No
Program language	FORTRAN II

is equal to that component of the vehicle's weight which is acting in a direction parallel to the surface of the road. The force available to overcome these resistance forces and to move the vehicle forward is called the tractive effort force, which is equal to the force supplied by the engine minus certain internal friction losses. The difference between this motivating force and the total resisting force is the force available to accelerate the vehicle. All simulation techniques are similar in that they are concerned with the manipulation of these forces. The logic of the technique, whether computer oriented or not, drives any vehicle over any selected alignment. The vehicle is accelerated, decelerated, stopped, upshifted, downshifted, and, in general, operated in the same manner as if it were being driven by a normal driver. In this manner, the effect of such highway characteristics as vertical profile, superelevation, pavement type, and speed limits on vehicle road speed, engine rpm, vehicle tractive effort, and percent of full engine load can be determined. Since these vehicle performance characteristics are in turn directly related to the user costs, it is possible to study the sensitivity of these costs to changes in the highway design.

In comparing existing methods of vehicle performance prediction to the criteria of the profile selection-evaluation system, no satisfactory solution is found to be currently available. The computer simulation techniques are the best; yet, even these are not completely satisfactory in their existing form. The vehicle simulation and operating cost system developed at M.I.T. is in effect too flexible and consequently too expensive to use. It was decided that a simulation method which combined some of the flexibility of the existing M.I.T. method with the logical advantages of some of the other vehicle simulation techniques presently available would produce an efficient predictor of user fuel and time costs suitable for all classes of vehicles in not only the United States, but also foreign countries. Such user cost items as tires, oil, depreciation, and vehicle maintenance are not sufficiently well understood to merit their inclusion into a simulation routine on anything more than a cost per mile as a function of pavement type basis.

The basic features of the proposed vehicle simulation routine are given in Table 2. The independent variable is velocity. Vehicle velocity is changed in 1-mph units during acceleration or deceleration. The model equations are recalculated only when the velocity changes or when the resisting forces change due to a change in profile grade.

Input data required for this routine, in addition to the fuel map, are essentially of three types:

1. Vehicle data include such things as vehicle weight, frontal area, tire size, number of cylinders, engine bore and stroke, gear shift speeds and ratios, and full-throttle torque curve;
2. Control data include the speeds at which the vehicle is to travel and the maximum acceleration and deceleration rates to be used by the vehicle; and
3. Highway profile data are required in the form of a series of straight line segments.

Output data consist of two types. The first is the cost information required for the feedback loop—the total annual user fuel and time costs for each of the vehicle types selected. The second type of output is optional and is under control of the engineer.

For any vehicle operating on a particular vertical profile, it would include such things as current vehicle speed, distance traveled en route, elapsed time, rate of fuel consumption, and total fuel consumed to date. Areas where the vehicle was not capable of maintaining the desired road speed would be so indicated.

The proposed vehicle fleet operating and time cost routine has not yet been programmed. The versions of the profile selection-evaluation system currently in use employ a table look-up procedure based on data published in the AASHO Red Book (1). Direct operating costs (fuel, tires, oil, maintenance and repairs, and depreciation) are obtained directly from the Red Book while time costs are calculated separately. Input data include the type or class of alignment, speed, user time cost, number of persons per car, truck factor, percentage trucks, present traffic volume, and projected traffic volume. Although no vehicle performance information can be obtained with this approach and although the basic cost data that are incorporated in the routine prevent it from being meaningful to areas outside the continental United States, this procedure is still thought to be acceptable for the purpose of testing the basic system and its operation.

Graphical Display of the Ground and Selected Highway Profiles

The manner in which information is displayed is critically important for any profile selection scheme. As a mass of numbers, a highway or ground profile is difficult to comprehend or to picture, but as a graphical plot it is clear and concise. Routines are incorporated in the system to simultaneously plot the ground and selected highway profiles. Depending on the machine configuration being used, these displays are in the form of either continuous line plots (Figures 13 and 14) or character plots and are generated via either on-line digital plotters (either a California Computer Products incremental Digital Plotter, Model 565, or a Gerber Scientific Instrument Company line plotter, Model VP 600) or printers.

Engineer-Computer Communication and Feedback Loop

The lower portion of Figure 5 indicates an engineer-computer feedback loop. This loop, by incorporating the concepts of a problem-oriented computer language, permits an engineer to use ordinary English and engineering terminology to change any of the initial input data and then to investigate the effects of these changes on the selected profile. He can modify such items as the roadway width, the grade restrictions, the sight distance or rate of change of grade restraints, the elevation of a particular control point, the predicted traffic volumes, or the expected percentage of trucks to determine how much a particular design standard or restriction is costing in terms of both construction and user costs.

The "smoothness" of the vertical alignment generated by the system can be controlled via this same engineer-computer feedback loop through the adjustment of the program parameters which control the length of the range of influence ahead and behind the origin point. In this way, the profile can be varied until the most economic balance of initial and continuing costs is determined. This iterative improvement of the highway profile is not presently being automatically done by the program because of the lack of specific knowledge concerning the exact relationship of user to construction costs. Past work in this area has been largely of a theoretical nature. Experimental data from a number of real projects will be necessary before this "learning type of behavior" can be confidently accomplished without engineer intervention.

COMPUTER HARDWARE CONFIGURATIONS

The engineering problems under discussion are basically ones in information acquisition, processing, storage, and display. The selected computer hardware should perform these information handling operations in as efficient a manner as is possible. In addition, the chosen computer configuration must result in an overall profile selection-evaluation system which is competitive with existing methods of profile selection and evaluation. In order for a method such as the one being proposed either to replace or augment present hand techniques, it must be approximately equal in cost. If not, it must provide enough additional information in a short enough time period to merit the additional expense. Thus, the proper choice of computer hardware is an extremely important part of the development work of this system.

Existing Equipment Requirements

Three separate versions of the system have been developed. These have been for a small-scale computer (either a 20,000 or a 60,000 digit memory IBM 1620 data processing system), a medium size computer (a 32,000 word memory IBM 7040 data processing system with 6 magnetic tape drives and on-line card reader, card punch, and printer); and a large-scale computer (a time-shared IBM 7094). The 1620 and 7040 versions have been used primarily for program development and field testing, while the time-shared 7094 version has been used for experiments in man-machine interaction and in the development of large and more sophisticated highway oriented programming systems. In the time-shared version, the engineer-user has essentially complete on-line control of a large-scale computer (currently an IBM 7094 under control of the M.I.T. compatible time-sharing system). The response time between input query and output result is such that entirely new and different approaches to engineering utilization of computers are possible.

Input-Output and Information Display Developments

A number of capabilities have recently been introduced which permit the performance of operations upon information in a more efficient manner than previously possible. One of these is the use of small-scale "satellite" computers to communicate with and to act as remote input/output consoles to a large computer. As a prototype development, the M.I.T. Department of Civil Engineering's IBM 1620 computer is now connected via a dataphone to an IBM 7094 computer. In this mode of operation, the 1620 can be used as a rather complete remote console in the compatible time-sharing system. The terrain vertical profile and the selected highway vertical profile data can be transmitted directly from the 7094 into the 1620 core or disc memory. A plotting program stored in the 1620 could then plot both the ground and highway profiles on a plotter attached on-line to the 1620. Because of the length and size of plots required, this means of plotting would most likely be preferred to either a scope attached directly to the 7094 or the teletype remote console produced character plots.

SYSTEM TESTING

Profile Selection

An important phase in the development of any new engineering technique is the testing to which this technique is subjected before it is actually placed into full production use. In addition to the tests being performed by the M.I.T. Department of Civil Engineering in cooperation with the projects sponsor, the Massachusetts Department of Public Works, a series of field experiments are being conducted by the Maine State Highway Commission. Both of these series are of a continuing nature and have been under active study by one or the other of these state highway departments. They differ principally in that the M.I.T.-Massachusetts series is being conducted primarily at M.I.T. while the Maine tests are being performed and evaluated by actual state highway department personnel. In both test series, the conclusion has been that the profiles generated by the program are generally satisfactory for the purpose of preliminary engineering location and they are at least equal to the first or second trial profiles chosen by engineers. Although it is not possible to describe all of the details and all of the results of the various tests which have been performed, the general conclusions can be summarized.

The three bias problems which existed in some of the earlier attempts at automatic profile selection appear to have been eliminated. Testing in both smooth and rough terrain has produced a profile that is "in phase" with the terrain; that is, the peaks and valleys of the profile occur at the same points as those for the terrain. Operation in grade restricted areas has been examined and the results indicate that the backing up procedure generally results in grades below the maximum allowable grade and that the problems associated with seeing a major terrain configuration too late have been eliminated or considerably reduced. If the engineer selects a look ahead and look behind range that is too short for the existing terrain, resonant frequency can still become a major problem. However, it is felt that this is more the result of poor input data than of the profile selection technique itself.

One of the criteria specified for the selection technique was an ability to change the quality of smoothness of the selected profile. If improvements are to be iteratively made in the profile selected for any one alignment, it must be possible to alter the profile in some manner. This is currently done either by changing the length of the look ahead and look behind ranges or by adjusting the α, λ parameters in the terrain point elevation weighting function. Testing has shown that as the length of the range is increased, the selected profile becomes significantly smoother. The majority of the testing has been conducted with both α and $\lambda = 3$. However, in tests where these parameters were decreased toward 0, the profile tended to become somewhat flatter. Although for α, $\lambda = 0$ (equivalent to a rectangular weighting function), the profile had many small dips and rises superimposed on the selected profile. A value of 3 for α and λ seems to give the best results.

A number of variations of the basic control point handling technique have been investigated. No single method has yet been found to be completely satisfactory for all cases. When the elevation of a control point is near the elevation of the desired profile, the

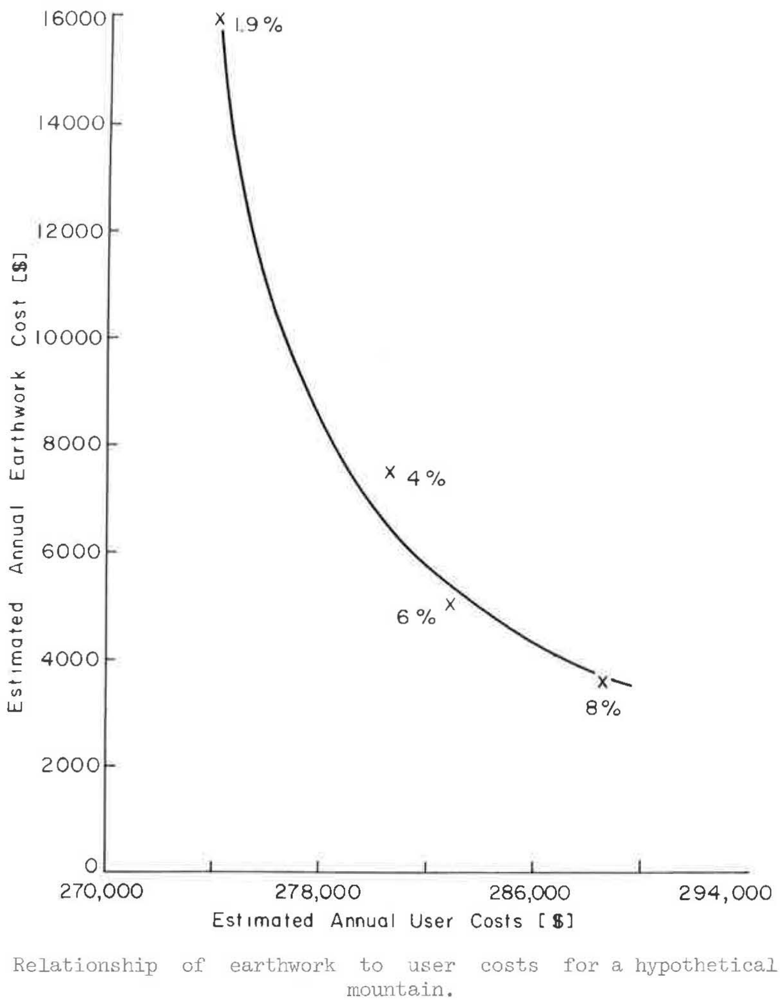

Figure 17. Relationship of earthwork to user costs for a hypothetical rectangular mountain.

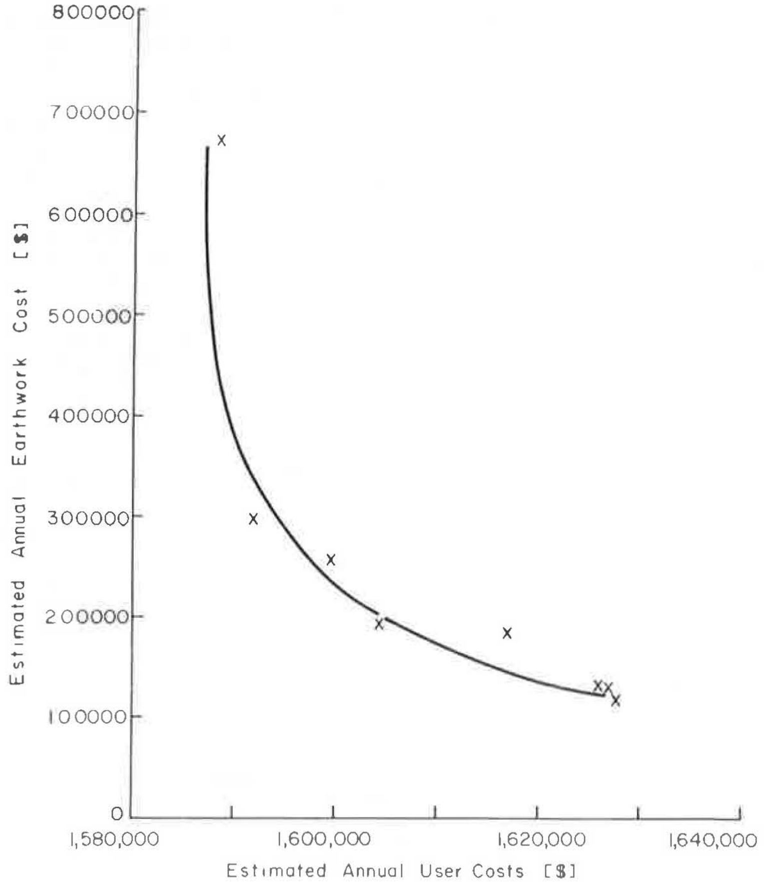

Figure 18. Relationship of earthwork and user costs for a 7-mile section of Interstate 495.

transition is both smooth and satisfactory. However, when the control point elevation is considerably different from the normal desired profile elevation, the selection technique does not account for the control point in a completely satisfactory manner. The conclusion of these tests is that the method of handling control points needs additional investigation.

The final and in the long run the only test of this technique is whether it produces a profile acceptable to the engineer. Present testing indicates that this goal can be reached.

Earthwork and User Cost Relationship

An extremely critical part of the selection-evaluation scheme is the curve (Fig. 3) showing the theoretical relationship of earthwork to user cost. Whether the proposed feedback loop is to be automatically included in the computer complex or is to be hand controlled by the engineer, real data must be collected to confirm the theorized shape of this cost relationship. If it turns out that these cost variables are not related or if their relationship is different than presently thought, it is doubtful that a profile selection-evaluation technique which attempts to balance earthwork and user costs will be either feasible or valuable.

A preliminary series of experiments that examined two separate test conditions has been performed to determine the relationship between these two cost variables. The first is the case of the so-called rectangular mountain. It is easy in this relatively simple example to define a series of progressively smoother vertical alignments and to evaluate both earthwork and user costs by standard hand methods. The results (Fig. 17) indicate that, at least for this very simple case, the theoretical relationship holds.

The second test series was performed on a seven-mile portion of highway near Franklin, Mass. The actual alignment chosen was one of those considered in the location of Interstate 495 and passed through two rather large hills. Eight separate vertical alignments were investigated; the "roughest" of which followed the terrain quite closely, while the smoothest was a constant 0.2 percent grade from beginning to end. Geometry, roadway design, and earthwork volume computations were made with the DTM location system, while user operating and time costs were estimated with the M.I.T. vehicle simulation and operating cost system. The traffic volumes predicted by the Massachusetts Department of Public Works were used as a means of obtaining equivalent annual user costs. The resulting plot of earthwork cost vs total user cost is shown in Figure 18. Although the hypothesized relationship appears to hold, at no point is the decrease in user costs large enough to merit the corresponding increase in construction cost. This is an indication that because of the low traffic volumes being used in this particular example, the chosen profiles represent points only at one extreme of the hypothetical curve.

These tests have been of a preliminary nature and their results do not constitute enough evidence to say that this relationship is either valid or not valid. A much more extensive test series is needed to definitely confirm this cost variable relationship.

CONCLUSIONS

Basic System Approach

In mathematical and operations research terminology, this approach to the solution of the profile selection problem would be classed as a heuristic and opposed to a mathematically rigorous optimization procedure. It is a strategy, a simplification, or a rule which attempts to produce a solution which is "good enough most of the time;" it is not necessarily producing an "optimal" solution. The procedure is trying to select a profile which would be classed as acceptable, satisfactory, or intelligent if the same profile had been chosen by a human engineer the first or second time he had examined a problem. In brief, it is simply a device "which drastically limits search for solutions in large problem spaces" (6).

An example of a more mathematically rigorous approach to this problem is the dynamic programming solution, which could produce a solution which would guarantee an optimum balance of earthwork and user costs. Although this approach has not yet been fully formulated by the authors, two general comments can still be made. First, the solution appears to break down to an exhaustive search of the decision space. This type of search with its corresponding high number of required evaluations of the objective function would probably require a class of computer which is not yet available to the typical state highway department. Second, the selection of a highway vertical profile is not a neat, tidy, and clean problem which directly lends itself to an optimum solution. Any profile selection technique can at best make suggestions to the location engineer.

Possible System Applications

An important question that should be investigated in the early stages of development of any system dealing with "computer-aided engineering design" is, "What is the demand by the engineering community for a system of this nature?" This is extremely difficult to determine; however, it is felt that a profile selection and evaluation system as described has a wide variety of uses.

To the student, it could serve as a valuable educational tool for studying the engineering of location. Certain real life subtleties, which are extremely difficult to teach in the classroom, would become immediately obvious after a few minutes of investigation into an actual problem.

To the researcher, such a system represents a powerful tool for studying and analyzing the engineering process. Certain insights may be gained into intricate cost variable relationships.

To the highway engineer of a developing or emerging nation, it could represent a means of greatly increasing engineering capabilities. In these areas, the flexibility of locating a transport system link is generally much greater than in the United States, and hence the decision-making problems associated with location are compounded.

To a highway engineer in the United States, a profile selection and evaluation system could have a number of direct applications. It could help to provide a uniformity of vertical design standards. Unbiased vertical design would allow different horizontal alignments to be more directly and fairly compared since the same set of restrictions and the same design procedure had been used on each. It could provide a means of speeding up the processing of horizontal alignments. One of the current restrictions on the number of horizontal alignments which can feasibly be investigated by an engineering team is caused by the number of computer passes required for even a preliminary evaluation. There is presently a human selection of the horizontal alignment which is followed by one or more machine passes to produce a centerline ground profile plot. After human review and human selection of a vertical alignment, additional computer passes are required to obtain earthwork computations, slope limit plots, and, if desired, vehicle performance and user cost data. The automatic selection of a vertical profile would allow the entire computer sequence of operations to occur without interruption. If the profile selection-evaluation system is iteratively solved for each of a number of horizontal alignments, with each iteration on a particular alignment using a different combination of the smoothing parameters and projected traffic volumes, it becomes possible to generate a complete set of transportation production curves. These curves could then be used to provide the necessary construction and user cost input data for a regional transportation network analysis.

Long-Range Research Objectives

The ultimate objective of this work is an integrated system of computer programs, computer and computer related hardware, and engineering procedures which can be used to solve highway location and design problems. The automatic selection and evaluation of vertical alignments represents one phase of this larger problem, at one particular level in the design hierarchy. Although much computer-oriented work has already been directed towards the final design aspects of this overall system, investigations are only beginning in such areas as regional network analysis, horizontal alignment selection, drainage, interchange design, and guidance or management of the total engineering process. Many more areas exist where mathematical selection, evaluation, and analysis models need to be built. The formulation, design, construction, and implementation of the individual models required to fill in these "gap" areas and of the ultimate man-machine system represents a fertile area for future research.

ACKNOWLEDGMENTS

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Effect of Guardrail in a Narrow Median Upon Pennsylvania Drivers

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A study has been performed to identify and evaluate changes in traffic behavior resulting from the erection of a barrier on a 4-ft median of a divided highway.

Using the Traffic Analyzer, vehicular speed, lateral placement and clearance were measured both before and after the erection of a median barrier on the Schuylkill Expressway.

Analysis of "before" and "after" data showed that median lane vehicular placements were shifted to the right by as much as 0.5 ft after barrier installation. As a result, clearance between adjacent vehicles was reduced by as much as 0.4 ft.

The presence of the median barrier did not cause any reduction in vehicular speeds or densities; therefore, the median barrier did not cause a reduction in capacity.

A hypothetical model was used to determine the effect of the observed clearance reduction on the safety or allowable tolerances afforded the driver in the passing or lane-changing maneuver. The results of this investigation coupled with other observations led to the conclusion that although the median barrier does have a measurable effect on traffic flow characteristics, it is not of an adverse nature with respect to the movement of vehicles.

To investigate the safety aspect in this work an accident study was conducted to determine further the implications of median barrier installation. The accident study results present a guide to possible outcomes of median barrier construction. Due to the approximations and assumptions inherent in the analysis, all conclusions should be carefully examined before applied to any particular expressway.

•MODERN HIGHWAY design standards emphasize the importance of providing adequate clearance to any fixed objects along the traveled way. One major exception to this practice is the installation of a raised barrier on narrow medians separating opposing flows of traffic. This study identifies and evaluates changes, if any, in traffic behavior resulting from the erection of a median barrier. Although the virtues and warrants for median barrier installation are not treated in this paper, the results herein presented may shed additional light on the subject.

FLOW CHARACTERISTICS

Description of Study

At the inception two basic study approaches were considered. The first examined facilities similar in geometric design and traffic load but different in that one had a median barrier and the other did not. The second approach studied the same facility both before and after median barrier construction. Since erection of a median barrier

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was already scheduled on the Schuylkill Expressway, a heavily traveled divided highway with a predominantly narrow median, connecting the Pennsylvania Turnpike and the western suburbs of Philadelphia with downtown Philadelphia, the latter approach was selected as the study technique.

After a preliminary survey, four sites were selected for study, three on four-lane divided sections, and the fourth on six-lane divided. All lanes were 12 ft wide, composed of portland cement concrete. In each case the median was 4 ft wide and of the mountable concrete type. Figure 1 is a typical median cross-section showing the steel median barrier with broken lines.

For the remainder of this study all reference will be made by location number, each site being composed of two locations, one in each direction of traffic. A description of each location is given in Table 1 and illustrated in Figures 2 through 5.

With the cooperation of the U. S. Bureau of Public Roads "before" data were successfully obtained at five locations during March 1962 using the Traffic Analyzer. In August 1962, several weeks after the median barrier installation was completed, "after" data were collected at the same locations. All measurements were performed during daylight hours and in fair weather. Several thousand vehicles were observed in each traffic lane incorporating either the AM or PM peak movements.

The Traffic Analyzer is an instrumented van capable of automatically recording in digital form on printed paper tapes by means of detectors placed across the roadway. It recorded the arrival time, speed and lateral placement for each vehicle in one to

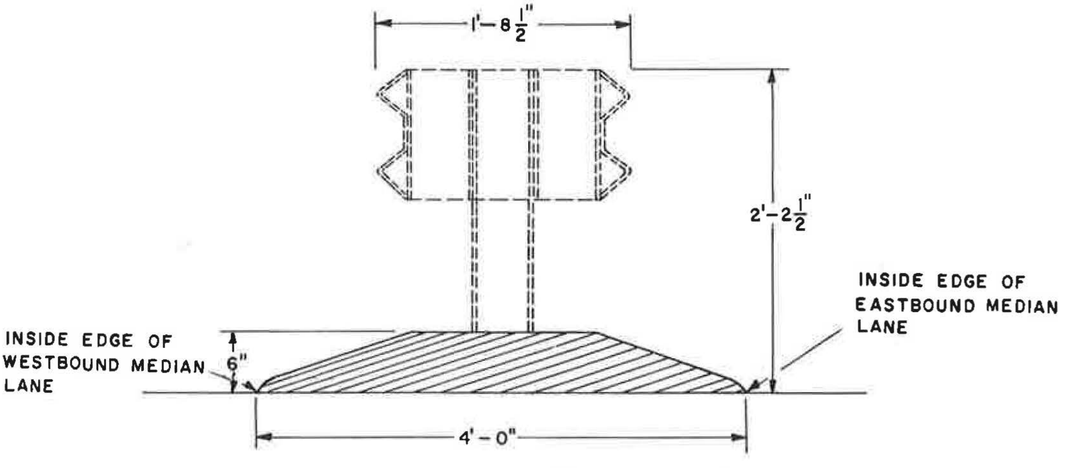

Figure 1. Typical median cross-section.

TABLE 1

Location No.	Station No.	Direction of Traffic	Median Width (ft)	No. of Lanes	Lanes Width (ft)	Alignment	Shoulder Width (ft)	Shoulder Type	Edge of Shoulder	Speed Limit (mph) Car	Speed Limit (mph) Truck
1	219+00	EB	4	2	12	Tangent	12	Gravel	3-cable guard-rail	60	50
2	467+00	WB	4	2	12	Tangent	12	Gravel	Slight embankment	60	50
3	467+00	EB	4	2	12	Tangent	10	Gravel	Steep embankment	60	50
4	110+00	EB	4	3	12	Curve, 2° 20′	8	Gravel	3-cable guard-rail	50	50
5	523+00	WB	4	2	12	Curve, 2° 20′	8	Gravel	3-cable guard-rail	60	50

Figure 2. Location 1, setting up the equipment.

Figure 3. Locations 2 and 3.

Figure 4. Location 4.

Figure 5. Location 5.

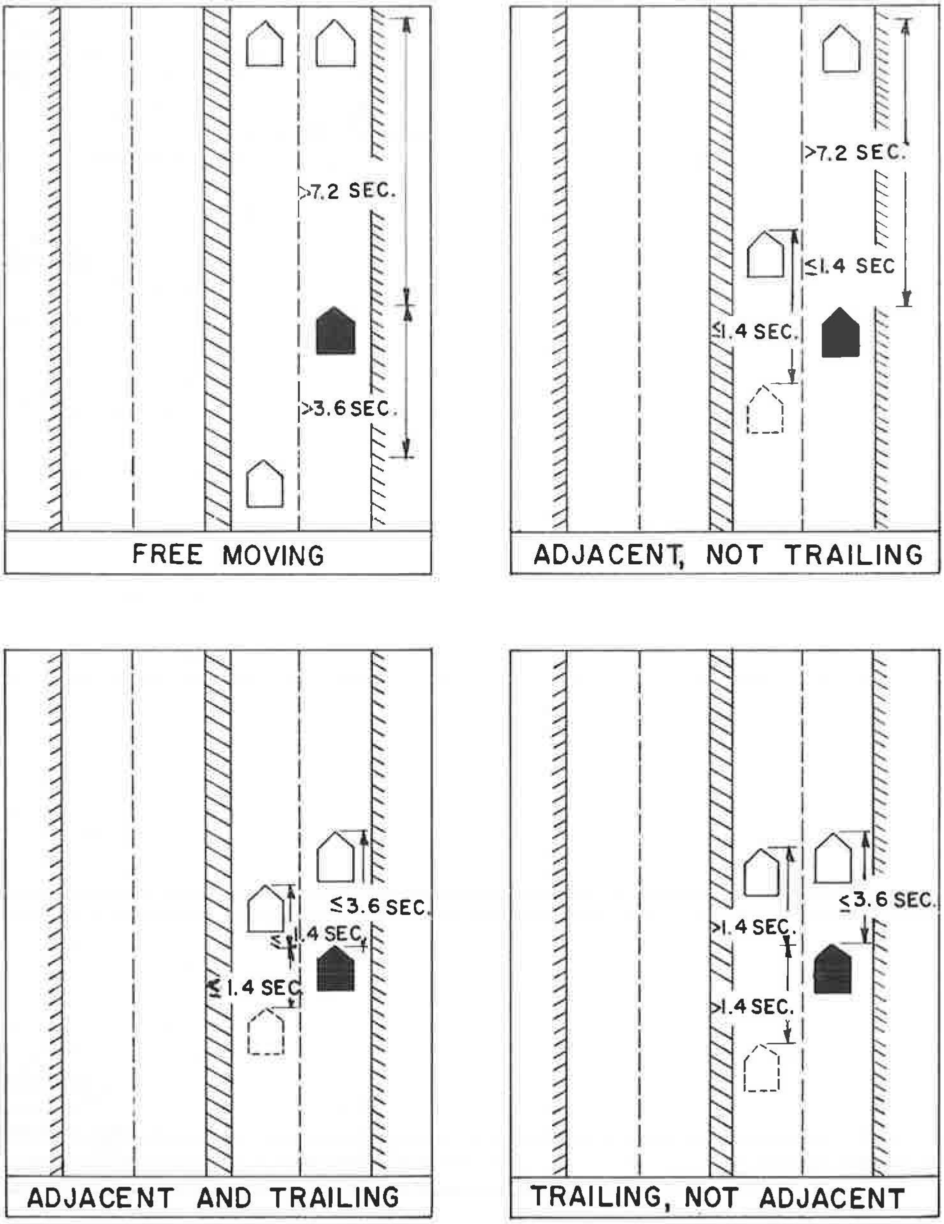

Figure 6. Different traffic maneuvers.

four separate lanes of traffic. In addition, vehicle classification is accomplished by manual recording and coding by means of push buttons. For vehicles other than passenger cars, recording is made on the same printed line as the other data for the vehicle as each vehicle crosses the detectors. The data for each lane of traffic are recorded on a separate tape, one line per vehicle (1).

After the field data were edited in the offices of the U. S. Bureau of Public Roads to confirm accuracy of the data recording, they were forwarded to the Pennsylvania Department of Highways along with two IBM 650 computer programs. Basic data cards were then punched from the data on each roll of tape. The first program used the basic data cards as input and produced as output a deck in which the speed and placement data, originally recorded in coded format, were presented in true form. The intermediate output decks for each lane at each location were merged in ascending arrival time sequence and then used as input for the second program. In this program, traffic in each lane was examined with respect to traffic in the other lane(s) at the location. The final output deck, one card for each vehicle in each lane at each location, contained the speed, placement, clearance between adjacent vehicles and traffic maneuver.

Each vehicle was classified (Fig. 6) in one of five types of traffic maneuvers with respect to nearby vehicles, as follows:

1. Free moving—vehicles more than 20 units of time (7.2 sec) behind a vehicle in any lane and more then 10 units of time (3.6 sec) ahead of a vehicle in the other lane(s).[1]
2. Adjacent, not trailing—vehicles more than 20 units of time (7.2 sec) behind a vehicle in the same lane but within 4 units of time (1.4 sec) either ahead of or behind a vehicle in the other lane(s).
3. Adjacent and trailing—vehicles within 10 units of time (3.6 sec) behind a vehicle in the same lane and also within 4 units of time (1.4 sec) either ahead or behind a vehicle in the other lane(s).
4. Trailing, not adjacent—vehicles 10 units of time (3.6 sec) or less behind a vehicle in the same lane but more than 4 units of time (1.4 sec) either ahead or behind a vehicle in the other lane(s).
5. All others—vehicles not classified in any of the previous categories.

For purpose of analysis, vehicle classification recorded in the field using nine separate categories was simplified, yielding two basic types of vehicles: all automobiles and two-axle, four-wheel, single-body trucks were classified as passenger cars (PC); all three-, four- and five-axle trucks, cars pulling trailers, buses, etc., were classified as commercial vehicles (CV).

Analysis of Data

In a first investigation an IBM 407 accounting machine was used to prepare frequency distributions separately for passenger cars and commercial vehicles in each lane studied for all vehicle placements and the average speed in each placement group. Similarly frequency distributions of vehicle speeds and average placement in each speed group were also constructed. In addition, average speeds and average placements were calculated for vehicles in each type of traffic maneuver. All sites are summarized in Table 2. All placement values tabulated are the distance from the center of the vehicle to the right-hand lane line measured to the nearest 0.01 ft.

As can be seen from Table 2, the presence of the median barrier did not have a uniform effect at all locations. Differences in the values of average speed and average placement, statistically significant at the 0.01 level except for commercial vehicles traveling in the median lane at two-lane locations, ranged from +5.0 to -0.5 mph and from +0.5 to -0.4 ft, respectively (a positive difference in speed indicating an increase in speed after the median barrier was installed and similarly a positive difference in placement representing a shift to the right). Statistical significance was ascertained by comparing the difference between means with the standard error of the difference.

In general, the "after" studies revealed higher average speeds and higher 85 percentile speeds than the "before" studies. A feeling of increased security induced by the presence of the median barrier may explain the speed increase. A lack of consistency between change in speed and change in placement discourages any general relationship to be formed although it would be possible to find the best straight-line fit.

At four locations the change in placement was more pronounced for vehicles traveling in the median lane than for that of vehicles traveling in the shoulder lane. The exception was Location 3 at which the average passenger car placement in the shoulder lane shifted 0.2 ft to the left after median barrier construction and the average passenger car placement in the median lane remained constant. At Location 3, an additional

[1] One unit of time equals one-ten-thousandth of an hour.

TABLE 2

SUMMARY OF SPEED-PLACEMENT DATA

Location	Lane	Vehicle Type	Before Barrier Avg. Speed (mph)	Before Barrier Avg. Placement (ft)	After Barrier Avg. Speed (mph)	After Barrier Avg. Placement (ft)
1	Shoulder	PC	52.8	6.7	57.0	6.7
1	Shoulder	CV	49.4	6.1	51.8	6.1
1	Median	PC	61.8	5.9	63.6	5.7
1	Median	CV	59.2[1]	5.3[1]	58.7[1]	4.9[1]
2	Shoulder	PC	49.2	6.6	54.2	6.7
2	Shoulder	CV	43.1	6.2	47.2	6.1
2	Median	PC	55.3	6.0	59.7	5.6
2	Median	CV	51.0[1]	5.8[1]	54.9	5.4
3	Shoulder	PC	53.2	6.3	53.9	6.5
3	Shoulder	CV	47.5	6.0	48.8	6.3
3	Median	PC	58.0	5.7	57.8	5.7
3	Median	CV	53.2[1]	5.5[1]	54.8	5.7
4	Shoulder	PC	43.5	7.4	45.7	7.4
4	Shoulder	CV	40.4	6.9	43.2	7.0
4	Middle	PC	45.2	7.0	48.4	7.4
4	Middle	CV	44.5	6.6	47.1	6.7
4	Median	PC	46.9	6.8	49.3	6.3
4	Median	CV	46.0	6.4	48.0	5.9
5	Shoulder	PC	51.4	5.8	54.2	5.7
5	Shoulder	CV	47.1	5.5	49.5	5.6
5	Median	PC	56.4	5.2	58.1	4.8
5	Median	CV	53.5	5.2	55.4	5.0

[1] Sample size less than 50.

1½ hr of data for the "after" study were used (4-hr "after" compared to 2½-hr "before"). Lighter volumes during much of this period might account for some of the shift to the left. The increased opportunity for passings to occur might also be a factor. Commercial vehicle placements in each lane also shifted to the left. A possible explanation for this initially unexpected behavior lies in the fact that the location is in cut with an embankment rising at approximately 60 degrees from the edge of the 10-ft shoulder. The security of the median barrier separating opposing traffic may have caused the leftward shift away from the more "constriction-inducing" embankment.

Both Locations 2 and 5 experienced a 0.4-ft average placement shift to the right for passenger cars in the median lane. This represented the maximum placement shift at two-lane locations. At Location 2 passenger car placements in the shoulder lane shifted 0.1 ft to the left and at Location 5, 0.1 ft to the right. Location 2 had 1½ hr more data in the "after" phase than in the "before," and also the time period 4:24 - 4:42 PM appears to show the effect on the shoulder lane of a police car and a truck parked on the shoulder. Removing these data probably would change the result noted.

The most unexpected change in placement occurred in the middle lane of Location 4. After median barrier construction, average placements for passenger cars and commercial vehicles shifted 0.4 and 0.3 ft to the left, respectively. At the same time both passenger car and commercial vehicle average placements in the median lane shifted 0.5 ft to the right. A careful examination of the placement distribution for this lane revealed that one of the placement detector sections was not functioning. If the distribution is smoothed, it is evident that the tabulated placement change represents a systematic error and should be discounted. Therefore, only speed data have been interpreted as valid at this location and lane.

The data for all locations support the conclusion that rightward shifts in the median lane vehicle placements have little effect in inducing similar displacements in the adjacent lane.

Separate frequency distributions of clearance between vehicle bodies and average speed in each clearance group were then prepared for passenger cars and commercial vehicles in each lane that were classified in an adjacent traffic maneuver with respect to vehicles in the contiguous lane. In the case of the middle lane of the three-lane location a vehicle was classified as adjacent to either a vehicle in the median or shoulder lane according to which was closer in time. For each lane the average speeds, placements and clearances were calculated for vehicles adjacent to other vehicles and then separately for each of the two types of adjacent traffic maneuvers. Sample clearance data are summarized for all sites in Table 3. Clearance values are measured to the nearest tenth of a foot. A positive change in placement indicates a shift to the right; a negative change in clearance indicates a decrease. Small sample sizes should be

TABLE 3

SUMMARY OF CLEARANCE DATA FOR ALL VEHICLES ADJACENT TO OTHER VEHICLES

Location	Lane	Description	Before Barrier			After Barrier			Change in	
			Avg. Speed (mph)	Avg. Placement (ft)	Avg. Clearance (ft)	Avg. Speed (mph)	Avg. Placement (ft)	Avg. Clearance (ft)	Placement (ft)	Clearance (ft)
1	Shoulder	PC adjacent to PC in median lane	51.6	6.5	5.5	55.1	6.4	5.6	+0.1	+0.1
1	Shoulder	PC adjacent to CV in median lane	49.7[a]	6.2[a]	4.2[a]	49.3[a]	5.8[a]	4.0[a]	+0.4[a]	-0.2[a]
1	Shoulder	CV adjacent to PC in median lane	49.3	6.1	5.3	51.3	6.0	5.0	+0.1	-0.3
1	Shoulder	CV adjacent to CV in median lane	49.9[a]	6.0[a]	4.5[a]	46.9[a]	5.7[a]	3.7[a]	+0.3[a]	-0.8[a]
1	Median	PC adjacent to PC in shoulder lane	61.6	6.0	5.6	63.6	5.8	5.5	+0.2	-0.1
1	Median	PC adjacent to CV in shoulder lane	61.4	6.2	5.3	63.0	6.0	4.8	+0.2	-0.5
1	Median	CV adjacent to PC in shoulder lane	59.6[a]	5.6[a]	4.3[a]	61.1[a]	4.3[a]	3.4[a]	+1.3[a]	-0.9[a]
1	Median	CV adjacent to CV in shoulder lane	61.8[a]	5.6[a]	3.9[a]	57.3[a]	5.5[a]	3.6[a]	+0.1[a]	-0.3[a]
2	Shoulder	PC adjacent to PC in median lane	47.2	6.3	5.9	52.7	6.5	5.4	-0.2	-0.5
2	Shoulder	PC adjacent to CV in median lane	41.8[a]	6.0[a]	5.3[a]	49.5[a]	6.0[a]	5.2[a]	0.0[a]	-0.1[a]
2	Shoulder	CV adjacent to PC in median lane	42.1	6.1	5.2	47.0	6.0	5.1	+0.1	-0.1
2	Shoulder	CV adjacent to CV in median lane	40.0[a]	6.6[a]	3.7[a]	45.1[a]	6.0[a]	5.2[a]	+0.6[a]	+1.5[a]
2	Median	PC adjacent to PC in shoulder lane	55.1	6.1	5.9	60.1	5.7	5.5	+0.4	-0.4
2	Median	PC adjacent to CV in shoulder lane	54.3	6.2	5.2	58.1	5.9	4.9	+0.3	-0.3
2	Median	CV adjacent to PC in shoulder lane	52.9[a]	5.9[a]	5.3[a]	56.8[a]	6.0[a]	4.8[a]	-0.1[a]	-0.5[a]
2	Median	CV adjacent to CV in shoulder lane	51.0[a]	5.9[a]	3.8[a]	53.1[a]	5.8[a]	3.9[a]	+0.1[a]	+0.1[a]
3	Shoulder	PC adjacent to PC in median lane	50.8	6.1	5.9	51.3	6.3	5.7	-0.2	-0.2
3	Shoulder	PC adjacent to CV in median lane	46.9[a]	5.9[a]	4.5[a]	47.6[a]	5.9[a]	5.1[a]	0.0[a]	-0.6[a]
3	Shoulder	CV adjacent to PC in median lane	47.1	6.1	4.9	47.3	6.2	4.9	-0.1	0.0
3	Shoulder	CV adjacent to CV in median lane	45.0[a]	5.7[a]	4.6[a]	43.4[a]	6.0[a]	4.2[a]	-0.3[a]	-0.4[a]
3	Median	PC adjacent to PC in shoulder lane	56.9	5.9	5.8	57.0	5.8	5.7	+0.1	-0.1
3	Median	PC adjacent to CV in shoulder lane	58.1	6.0	5.0	57.3	6.0	4.9	0.0	-0.1
3	Median	CV adjacent to PC in shoulder lane	53.4[a]	6.2[a]	5.7[a]	55.3[a]	5.9[a]	5.2[a]	+0.3	-0.5
3	Median	CV adjacent to CV in shoulder lane	52.1[a]	5.4[a]	3.7[a]	53.3[a]	5.6[a]	3.8[a]	-0.2[a]	+0.1[a]
4	Shoulder	PC adjacent to PC in middle lane	43.2	7.4	5.8	44.6	7.4	6.4	0.0	+0.6
4	Shoulder	PC adjacent to CV in middle lane	41.2	7.2	4.6	44.5	7.1	5.2	+0.1	+0.6
4	Shoulder	CV adjacent to PC in middle lane	40.1	6.9	5.6	42.6	6.9	6.1	0.0	+0.5
4	Shoulder	CV adjacent to CV in middle lane	38.8[b]	6.8[b]	4.4[b]	40.8[a]	7.1[a]	4.2[a]	-0.3[a]	-0.2[a]
4	Middle	PC adjacent to PC in shoulder lane	44.7	7.3	6.0	47.9	7.9	6.6	-0.6	+0.6
4	Middle	PC adjacent to CV in shoulder lane	45.6	7.7	5.9	47.8	8.2	6.2	-0.5	+0.3
4	Middle	CV adjacent to PC in shoulder lane	46.3[b]	7.0[b]	4.7[b]	47.9[a]	7.0[a]	5.3[a]	0.0[a]	+0.6[a]
4	Middle	CV adjacent to CV in shoulder lane	46.4[a]	7.2[a]	4.5[a]	45.7[a]	7.2[a]	4.5[a]	0.00[a]	-0.00[a]
4	Middle	PC adjacent to PC in median lane	45.0	6.9	6.0	48.0	7.2	5.3	-0.3	-0.7
4	Middle	PC adjacent to CV in median lane	44.5	6.7	4.7	46.9[a]	6.7[a]	4.3[a]	0.00[a]	-0.4[a]
4	Middle	CV adjacent to PC in median lane	44.0	6.5	5.6	46.6	6.8	4.6	-0.3	-1.0
4	Middle	CV adjacent to CV in median lane	42.8[a]	6.9[a]	3.5[a]	45.5[a]	5.5[a]	5.0[a]	+1.4[a]	+1.5[a]
4	Median	PC adjacent to PC in middle lane	46.5	6.8	5.9	49.0	6.3	5.0	+0.5	-0.9
4	Median	PC adjacent to CV in middle lane	47.4	7.0	5.4	50.6	6.3	4.5	+0.7	-0.9
4	Median	CV adjacent to PC in middle lane	45.7	6.4	4.6	48.5[a]	5.9[a]	4.1[a]	+0.5[a]	-0.5[a]
4	Median	CV adjacent to CV in middle lane	48.3[a]	6.6[a]	4.1[a]	47.0[a]	6.1[a]	4.2[a]	+0.5[a]	+0.1[a]
5	Shoulder	PC adjacent to PC in median lane	50.3	5.6	5.7	52.8	5.6	5.3	0.0	-0.4
5	Shoulder	PC adjacent to CV in median lane	48.9	5.6	4.8	51.5	4.9	5.2	+0.7	+0.4
5	Shoulder	CV adjacent to PC in median lane	47.2	5.3	5.2	48.6	5.4	4.7	-0.1	-0.5
5	Shoulder	CV adjacent to CV in median lane	41.9[a]	5.8[a]	3.1[a]	46.3[a]	5.8[a]	3.8[a]	0.0[a]	+0.7[a]
5	Median	PC adjacent to PC in shoulder lane	56.2	5.3	5.8	58.0	4.8	5.4	+0.5	-0.4
5	Median	PC adjacent to CV in shoulder lane	55.7	5.3	5.1	57.1	5.0	4.6	+0.3	-0.5
5	Median	CV adjacent to PC in shoulder lane	54.2[a]	5.2[a]	4.6[a]	56.3[a]	5.0[a]	4.9[a]	+0.2[a]	+0.3[a]
5	Median	CV adjacent to CV in shoulder lane	52.6[a]	5.2[a]	3.9[a]	54.2[a]	5.1[a]	3.4[a]	+0.1[a]	-0.5[a]

[a]Sample size less than 25. [b]Sample size less than 50.

interpreted with caution. They are found in conjunction with clearances involving commercial vehicles in the median lane, an uncommon driving practice.

At first examination the clearance measurements involving passenger cars in the median lane adjacent to passenger cars in the shoulder lane may appear to be a repetition of the measurements associated with passenger cars in the shoulder lane adjacent to passenger cars in the median lane. The two cases are not necessarily drawn from the same set of adjacent vehicles owing to the traffic maneuver classification system employed.

Changes in clearance, more so than placement, are indicative of the effects of the meidan barrier upon the unidirectional traffic stream. Both passenger cars and commercial vehicle placements for vehicles in the shoulder lane and classified in an adjacent traffic maneuver, while slightly less than (to the right of) the averages for all traffic maneuvers, remained fairly constant after the median barrier was installed. Changes ranged from -0.2 to +0.1 ft. In general, changes in clearance resulted from placement changes in the median lane.

Both Locations 2 and 5 exhibited significant clearance reductions of approximately 0.4 ft between passenger cars in the median lane adjacent to vehicles in the shoulder lane. Location 1 demonstrated no appreciable change in adjacent passenger car clearances; however, clearances between passenger cars in the median lane adjacent to commercial vehicles in the shoulder lane were reduced by approximately 0.4 ft. For reasons previously presented, differences in average placements and clearances at Location 3, where statistically significant, were very small. Clearance data for the "after" studies at Location 4 are biased by the placement error in Lane 2.

Interpretation of Data

Reduction in clearance between adjacent lanes of traffic may be studied with respect to safety and also with respect to roadway capacity and level of service. From the safety viewpoint a clearance reduction is manifested in lower tolerance limits for lateral drift as well as for lane-changing maneuvers. Little data, if any, are available on the subject of lateral drift thereby preventing further treatment. It is possible, however, by means of a simplified model, to evaluate a clearance reduction with respect to the passing or lane-changing maneuver.

If Driver B is traveling in the median lane and beginning to change lanes in front of Vehicle A in the shoulder lane, how far behind the front left corner of Vehicle A can the right rear corner of Vehicle B be if a collision is to be avoided?

First assume Vehicle B to be traveling 55 mph and Vehicle A 45 mph or a relative speed difference of 10 mph (14.7 ft/sec). Further assume that in changing lanes Vehicle B moves laterally at 4 ft/sec. Under these conditions if the initial lateral clearance between vehicles is 6.0 ft, the distance in question is 22.05 ft. If the initial clearance is 5.5 ft, the distance is 20.21 ft, or a difference of -1.84 ft. If the relative speed between vehicles is 5 mph (7.35 ft/sec), the difference in distance is -0.92 ft. With a lateral speed of 3 ft/sec, the difference at a relative speed of 10 mph is -2.50 ft; at a relative speed of 5 mph, -1.25 ft.

In these simplified cases the maximum tolerance difference with a 0.5-ft reduction in clearance is -2.5 ft. Although no allowance has been introduced for Vehicle A slowing down, the increased chance for collision remains at this point, a moot question.

In an attempt to discover any changes in roadway capacity or level of service introduced by the presence of the median barrier, summaries of speed-placement data by 6-min time periods were prepared for all vehicles in each lane of each location both before and after median barrier construction. The summaries contained, in addition to average speeds and placements, equivalent hourly volumes for each 6-min period, traffic densities and average absolute differences in speeds and placements between successive vehicles.

Although it is impossible to say whether maximum recorded volumes represent lane capacities, the traffic demand remaining unknown and there being no exhibited flow breakdowns, an appreciation of the level of service may be gained by examining maximum speeds attained throughout the array of recorded volumes both before and after median barrier construction.

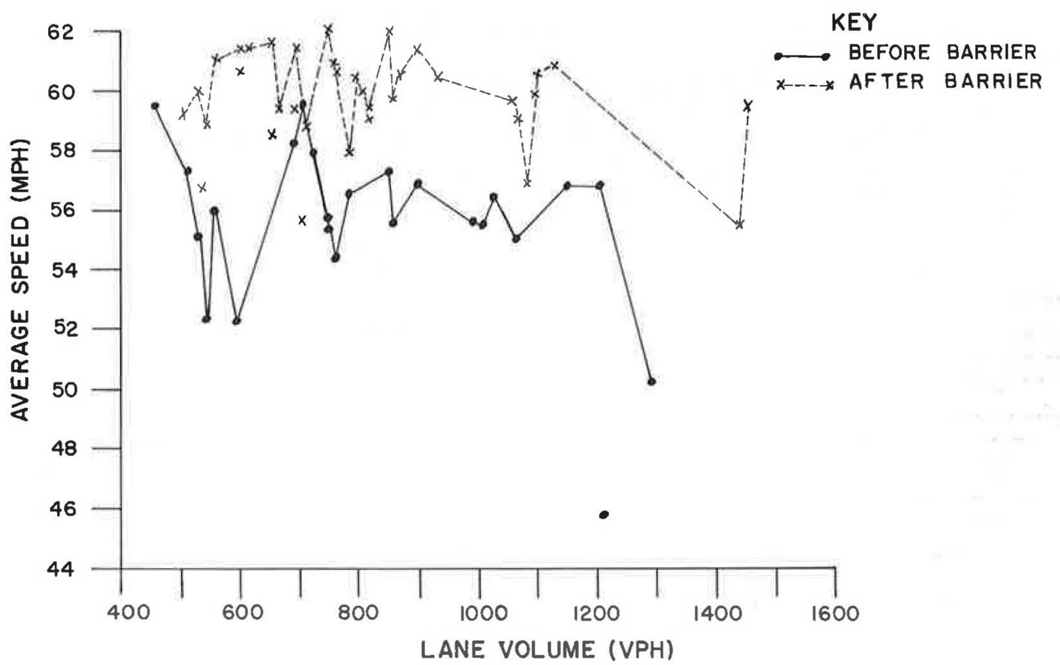

Figure 7. Average speeds and expanded 6-min volumes; maximum speeds at each observed volume, median lane, Location 2.

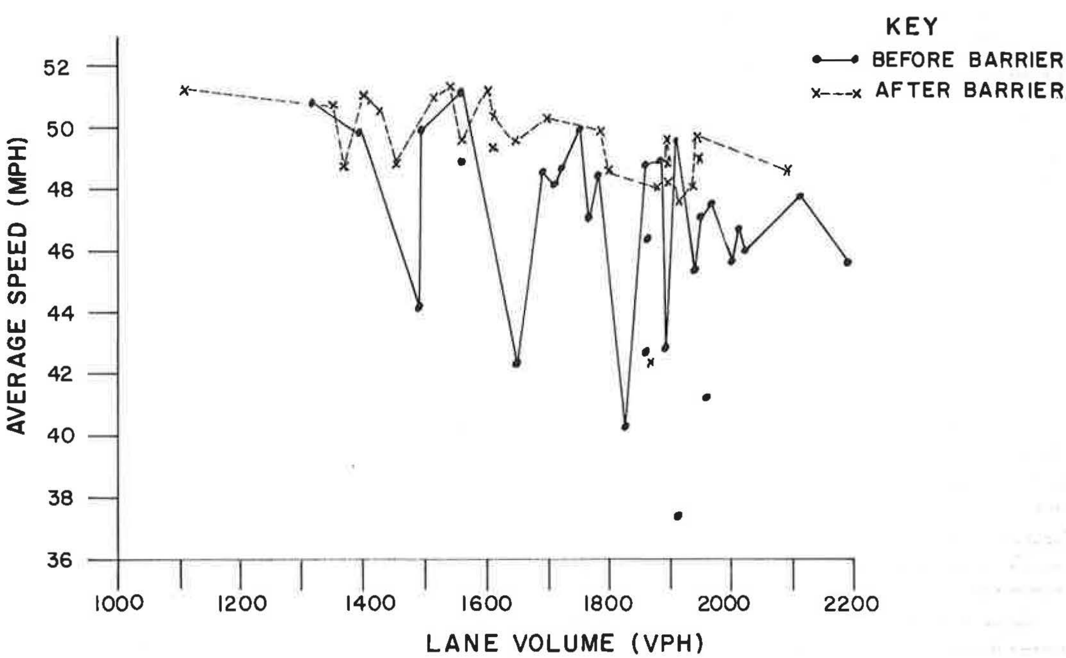

Figure 8. Average speeds and expanded 6-min volumes; maximum speeds at each observed volume, median lane, Location 4.

Figures 7 and 8 show the maximum speeds attained at observed volume levels for the median lanes of Locations 2 and 4, respectively. It was in these lanes that maximum clearance reductions occurred between adjacent vehicles. The graphs indicate that, in general, equal or greater speeds occurred at each volume level after the median barrier was constructed. Therefore, it can be concluded that the median barrier did not, in terms of volume and speed, lower the level of service.

Conclusion

It has been shown that the erection of a barrier on a 4-ft median does have a measurable effect on adjacent lanes of traffic. Whereas vehicle placement in the shoulder lane tends to remain unchanged, average median lane placements can be shifted to the right by as much as 0.5 ft. The clearance between adjacent vehicles at two-lane locations can be reduced by as much as 0.4 ft. Although the increase in average speeds after median barrier construction may be attributed to some variation of conditions, it is safe to say that the median barrier did not have a damping effect on travel speed. The data tend to support the conclusion that the median barrier causes no decrease in roadway capacity. The magnitude of the effects of the median barrier on traffic flow characteristics, smaller than some would expect, may be viewed from the aspect that the median barrier is a continuous rather than intermittent roadside obstacle.

Although changes in vehicle clearance and placement are measured in fractions of a foot, the elimination of the effects of a median barrier upon said may require increasing the median width by several feet.

ACCIDENT OCCURRENCE

Although the safety aspect was briefly treated in the first section, namely from a theoretical viewpoint, no conclusions were reached regarding the effect of median barrier upon accident occurrence. This section presents the findings of a study of traffic accident occurrence as related to the erection of the median barrier on the Schuylkill Expressway.

Description of Study

At present the Expressway has a back-to-back beam-type median barrier erected along its entire length. The median width, although 10 ft in some areas, is predominantly 4 ft. The barrier is erected atop a 6-in. mountable curb.

The barrier, although now continuous, was constructed in two different contracts. As it was decided to look at a 1-yr accident history both before and after median barrier construction, it was necessary to use different time periods for the two contracts. All further reference to the two study sections is made by contract number.

Table 4 gives the sections of roadway comprising each contract as well as the time periods associated with the "before" and "after" studies. Figure 9 is a map of the study area.

A 1-yr accident history for both the "before" and "after" studies was accepted for use for the following reasons.

1. The Schuylkill Expressway experiences daily volumes as high as 130,000 vehicles on certain roadway sections. Thus in one year there are many chances for the rare event, or accident, to occur.
2. The effects of volume growth between the "before" and "after" periods are minimized and any error induced by volume compensation is also reduced.

Generally the volume increase from the "before" study periods to the "after" study periods was 10 percent.

Only accidents occurring completely on the main-line portions of the road were included in the analysis. Those accidents occurring on the interchange crossroads, ramps or at the junction of ramps and the main line were deleted from the analysis. This practice was adopted with the idea that if the median barrier exerted any influence on accident occurrence it would be more pronounced in the proximity of the barrier.

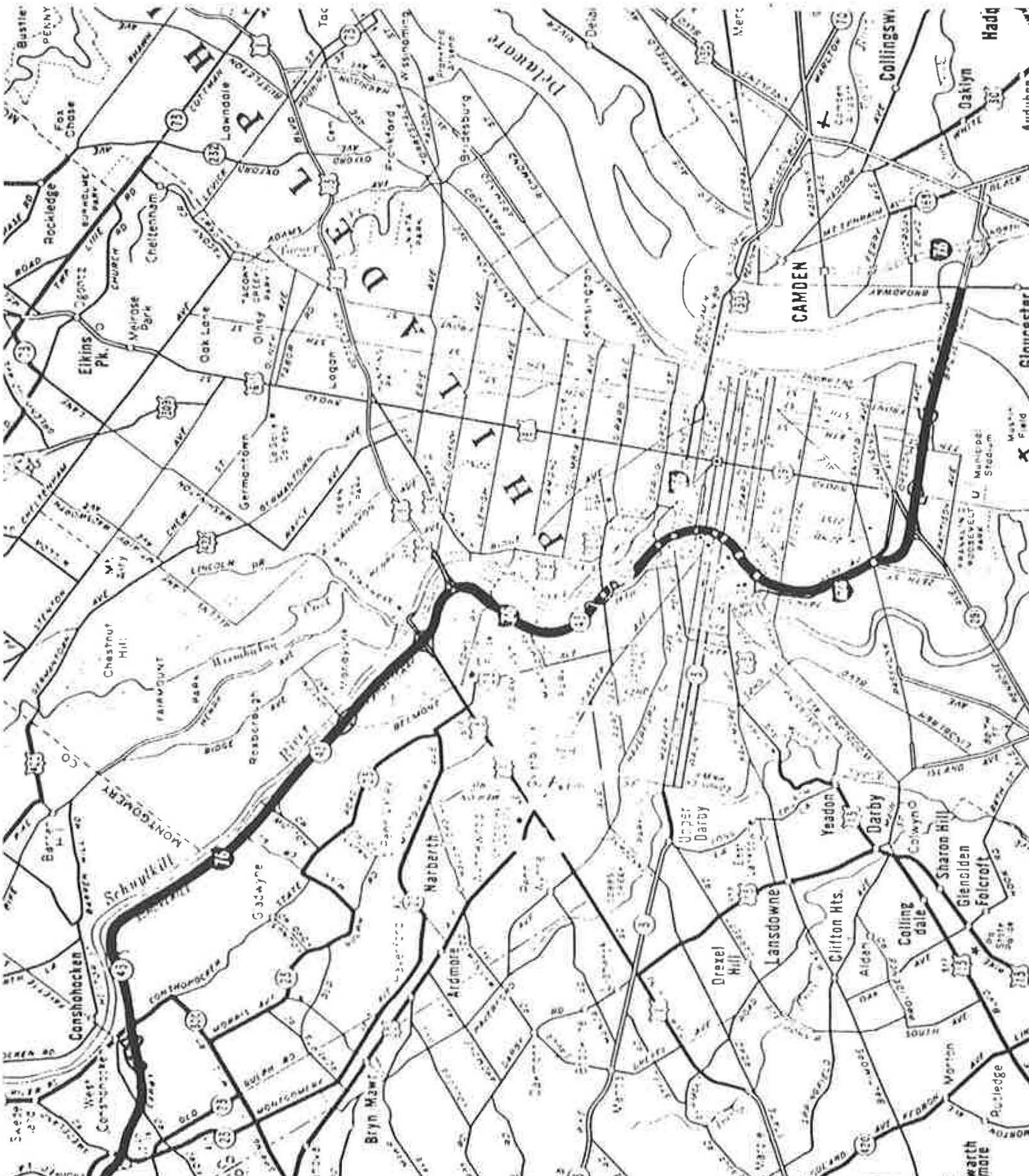

Figure 9. Study area.

TABLE 4

STUDY ROUTES AND TIME PERIODS

Legislative Route	Beginning Station	Ending Station	Length (ft)	Before Period	Construction Period	After Period
(a) Contract I						
769	0 + 65	153 + 87	15,322	7/60 - 6/61	7/61 - 9/61	10/61 - 9/62
	276 + 50	451 + 50	17,500	7/60 - 6/61	7/61 - 9/61	10/61 - 9/62
			37,822			
(b) Contract II						
769	153 + 87	276 + 50	12,263	3/61 - 2/62	3/62 - 9/62	10/62 - 9/63
	451 + 50	621 + 70	17,020	3/61 - 2/62	3/62 - 9/62	10/62 - 9/63
67057	48 + 42	160 + 00	11,158	3/61 - 2/62	2/62 - 9/62	10/62 - 9/63
	185 + 27	239 + 00	5,373	3/61 - 2/62	2/62 - 9/62	10/62 - 9/63
	267 + 67	302 + 00	3,433	3/61 - 2/62	2/62 - 9/62	10/62 - 9/63
	307 + 50	363 ± 62	5,612	3/61 - 2/62	2/62 - 9/62	10/62 - 9/63
67278	70 + 47	114 + 62	4,415	3/61 - 2/62	2/62 - 9/62	10/62 - 9/63
			59,274			

NOTES: 1. Median width is predominantly 10 ft in Contract I, whereas a predominant width of 4 ft prevails in Contract II.
2. Median post spacing is 12 ft 6 in. in Contract I; in Contract II the spacing is 6 ft 3 in.
3. Volumes in Contract II approach 130,000 veh/day. Volume in Contract I is considerably less.

Data were obtained from both the State Police and City Police Traffic Accident files. The State Police record is for Legislative Route 769. The remaining Lesiglative Route numbers comprising the study were patrolled by the Philadephia Police Department. All data were code classified and punched on cards for machine analysis.

Findings of Study

Overall accident resumes are presented in Tables 5 and 6. Conventional classification based upon severity suffered by individuals is used to define accident types.

The number of traffic accidents in Contract I increased from 50 before median barrier installation to 87 afterward. Based on the "before" period, this represents an increase of 74 percent. If it is assumed that accident frequency is linearly influenced by amount of travel (vehicle mileage), then, for a constant roadway length, it is also linearly affected by volume. Thus, for a 10 percent volume increase approximately 55 accidents should have occurred. Therefore, 32 accidents represent a certain deviation from the "expected norm," a 64 percent "abnormal" increase.[2]

The accident frequency increase in Contract II was 112, representing a total percentage increase of 38 percent over the "before" period. By similar reasoning to that presented above the "abnormal" increase was 82 accidents or 28 percent.

Time distributions of accident occurrence by month, day and hour (the latter separately for weekdays and weekends) have been prepared (Figs. 10, 11, and 12). The time distributions, while not subjected to rigorous analysis, do not appear to show any significant differences between the "before" and "after" periods. Thus it seems that the increase in traffic accidents experienced after median barrier construction is proportionately distributed throughout the hourly, daily and monthly time periods.

[2] It is acknowledged that accident frequency is probably more than linearly related to vehicle mileage; however, no mathematical relationship is known to allow a more exact calculation of "abnormal" accident frequency increase.

TABLE 5

OVERALL ACCIDENT RESUME

Accident Type	Contract I		Contract II	
	Before	After	Before	After
Fatal	1	0	6	1
Injury	17	29	82	100
Property damage	32	58	199	297
Total	50	87	287	398

TABLE 6

MEDIAN BARRIER ACCIDENT RESUME

Nature of Median Accident	Contract I		Contract II	
	Before	After	Before	After
Crossover	13	2	45	2
Non-crossover	3	18	11	61
Total	16	20	56	63

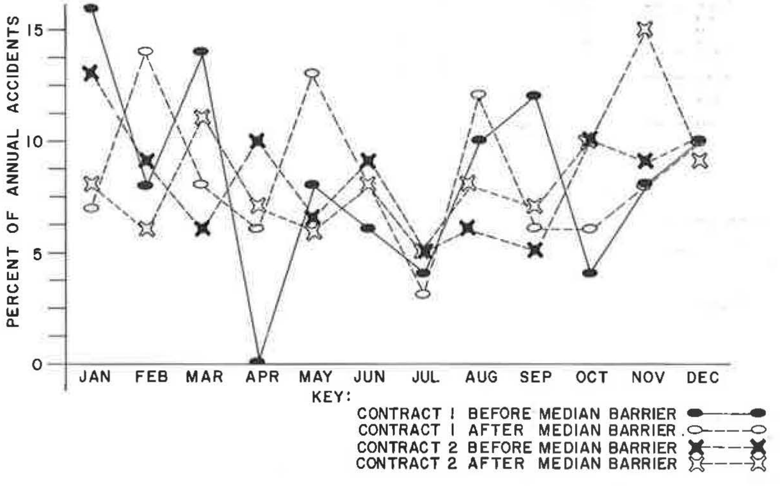

Figure 10. Distribution of monthly accident occurrence.

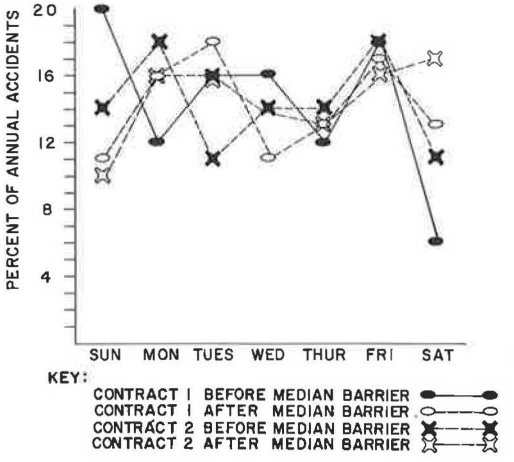

Figure 11. Distribution of daily accident occurrence.

Table 7 gives matrices of "before" accident occurrence with respect to collision type and illumination condition. Collision types are decided by the manner in which initial contact was made and what was first hit.

In both matrices a significant increase in both rear-end and hit-fixed object collision types is readily apparent. The increase in the latter is largely attributed to a new fixed object being present in the highway environment, the median barrier. The large increase in rear-end accidents after median barrier installation has been explained by some analysts as the result of the drivers' attention being diverted from what is in front of them to what is "passing" along side.

Table 8 gives accidents involving the

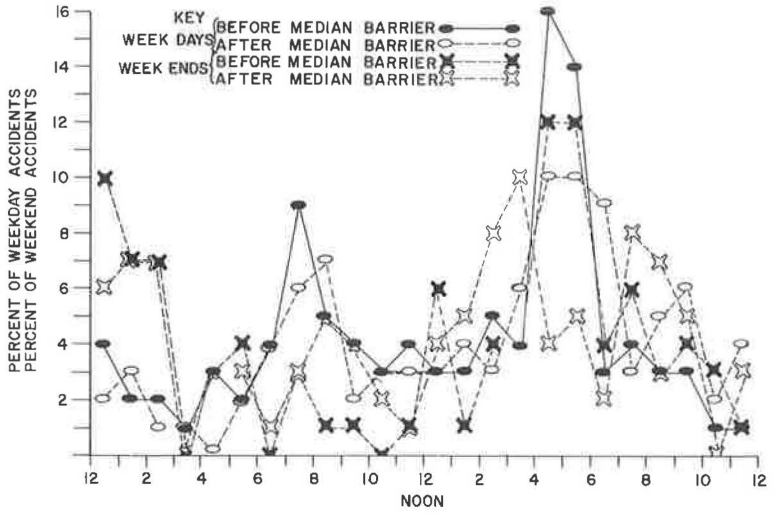

Figure 12. Distribution of hourly accident occurrence, Contract II.

TABLE 7

FREQUENCY OF ACCIDENT OCCURRENCE VS ILLUMINATION AND COLLISION TYPE

Illumination	Collision Type																			
	Head-On		Rear-End		Angle		Side-swipe		Backed In		Hit Ped.		Hit Fixed Object		Run Off		Others		Total	
	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A
(a) Contract I																				
Daylight	3	0	14	28	5	10	3	0	0	0	0	0	0	10	1	4	0	1	26	53
Dawn	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	3
Night, no lights	0	0	7	9	4	1	4	5	0	0	0	0	3	8	3	4	0	1	21	28
Night, lights	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	2
Dusk	0	0	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	1
Total	3	0	24	40	9	11	7	5	0	0	0	0	3	20	4	9	0	2	50	87
(b) Contract II																				
Daylight	8	0	97	135	17	10	21	26	0	0	2	0	6	35	6	9	1	0	158	215
Dawn	0	0	1	4	1	1	0	0	0	0	0	0	1	1	0	0	0	0	3	6
Night, no lights	3	2	10	18	6	5	1	2	0	1	0	0	6	21	7	3	0	0	33	52
Night, lights	2	0	31	61	3	1	7	10	0	0	1	2	15	33	8	0	0	0	67	107
Dusk	0	0	22	13	1	2	0	1	0	0	0	0	2	2	1	0	0	0	26	18
Total	13	2	161	231	28	19	29	39	0	1	3	2	30	92	22	12	1	0	287	398

NOTES: B = before median barrier construction.
A = after median barrier construction.

TABLE 8

FREQUENCY OF MEDIAN SUCCESSES AND FAILURES VS COLLISION TYPE

Median Accidents	Collision Type																			
	Head-On		Rear-End		Angle		Side-swipe		Backed In		Hit Ped.		Hit Fixed Object		Run Off		Others		Total	
	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A
(a) Contract I																				
Success	0	0	3	3	0	1	0	3	0	0	0	0	0	11	0	0	0	0	3	18
Failure	3	0	1	1	8	0	1	0	0	0	0	0	0	1	0	0	0	0	13	2
Total	3	0	4	4	8	1	1	3	0	0	0	0	0	12	0	0	0	0	16	20
(b) Contract II																				
Success	0	0	3	4	0	9	3	5	0	0	0	0	0	43	5	0	0	0	11	61
Failure	13	0	4	0	14	0	4	0	0	0	0	0	6	2	4	0	0	0	45	2
Total	13	0	7	4	14	9	7	5	0	0	0	0	6	45	9	0	0	0	56	63

NOTES: B = before median barrier construction.
A = after median barrier construction.

median by collision type. Median accidents have been classified in two types; a median success meaning that no part of the vehicle reached the pavement of the opposing lane of traffic, a failure meaning the opposite.

While head-on accidents were eliminated, fixed-object accidents involving the median increased greatly from 0 to 12 in Contract I and from 6 to 45 in Contract II. The median barrier was also hit in the other accident collision types recorded as median accidents in the "after" studies.

During the before period in Contract I, there was one cross-median accident in which three persons were killed. In Contract II there were four cross-median accidents in which five persons were killed. Neither contract suffered a cross-median fatality after

TABLE 9

DISTRIBUTION OF INJURY TYPES

Major Injury Inflicted	Accident Type							
	Non-Median Accidents		Median Failure Accidents		Median Success Accidents		All Accident Types	
	B	A	B	A	B	A	B	A
(a) Contract I								
Internal injuries	0	1	2	0	0	0	2	1
Contusions	0	0	1	0	0	1	1	1
Bruises	4	6	1	1	1	6	6	13
Fracture, head or back injury	7	6	3	0	0	2	10	8
Loss of eye	1	0	0	0	0	0	1	0
Lacerations	2	12	4	0	1	1	7	13
Abrasions	0	3	2	0	0	1	2	4
Total injuries	14	28	13	1	2	11	29	40
Total injury accidents							17	29
(b) Contract II[1]								
Internal injuries	2	0	0	0	0	0	2	0
Contusions	4	3	2	0	0	1	6	4
Bruises	14	12	2	2	1	4	17	18
Fracture, head or back injury	12	10	5	1	0	2	17	13
Loss of eye	0	0	0	0	0	0	0	0
Lacerations	11	10	9	1	1	6	21	17
Abrasions	2	6	2	0	0	5	4	11
Total injuries	45	41	20	4	2	18	67	63
Total injury accidents							37	45

NOTES: B = before median barrier.
A = after median barrier.

[1] Montgomery County portion only (i.e., investigated by the Pennsylvania State Police).

the erection of the barrier. While the accident severity associated with the loss of lives was eliminated, the effect of the median barrier upon the number and nature of injury accidents was much different. In Contract I the number of injury accidents inceased from 17 to 29. In Contract II the increase was from 82 to 100 injury accidents.

To understand better the effect of the median barrier upon the nature or severity of injuries incurred, a special study was undertaken of those injury accidents occurring in Contract I and those occurring on the Montgomery County, Legislative Route 769 portion of Contract II. This segment of the study roadway is patrolled by the State Police and the accident and injury data are more easily attainable. The findings of this study are given in Table 9. As a word of caution, State Police injury descriptions are not always most accurate. An examination of the distribution of injury types leads one to believe that no significant claim can be presented that the median barrier reduced either the number or nature of injuries. It is planned to expand the injury analysis to one that will incorporate data reflecting personal disability.

In another phase of this section an approximation was formed as to the probable nature of accident occurrence in the "after" periods had not the median barrier been installed. Probabilities were extrapolated from the "before" periods as to what would occur subsequent to the incidence of a vehicle upon the median. These probabilities were then applied to the "after" data.

Representative total property damage costs and number of injuries associated with different median barrier accident circumstances were computed and applied to the approximated accident occurrence in the "after" periods.

A cost difference was then computed between the actual and approximated "after" period accident occurrence. Incorporated in the economic analysis was the annual cost of the barrier and increased traffic delay costs owing to that portion of the "abnormal" accident frequency increase occurring during peak periods. Savings in time due to the increased operating speeds of vehicles after median barrier construction, approximately 2-3 mph, were deleted as insignificant. It is claimed that although total time savings to all road users may amount to thousands of hours, the incremental time saving to each vehicle occupant, approximately 30 sec/hr of trip, is too small to be recovered economically.

The accident analysis was somewhat conservative in that vehicles hitting the median barrier and continuing on were ignored. Additional study is now in progress to measure this deletion.

Nowhere in the analysis were monetary values labeled as "loss." Instead, the analysis was in terms of cost.

This study attempted to show the cost of saving a life through the use of median barriers. It did not attempt either to justify or condemn the use of median barriers for several reasons. First, is consumer sovereignty the governing factor; is something worth what the consumer will <u>knowingly</u> pay? Second, if consumer sovereignty is the proper economic approach, the amount the public is <u>willingly</u> and <u>knowingly</u> ready to pay for a life is undetermined.

The results of the economic analysis are being withheld at this time for several reasons:

1. The statistical base used in each segment of the economic analysis is not yet believed to be significant at any usable level.
2. Premature conclusions are too easily formed and cost figures tend often to be quoted out of context.

It is hoped that data gleaned from the next few years of experience will allow a definitive economic statement.

Conclusion

After median barrier construction, increases in accident frequencies in Contract I and Contract II of 74 percent and 38 percent, respectively, were observed with but an approximate 10 percent increase in travel.

Although the median barrier does eliminate, for all intensive purposes, the accident severity associated with the cross-median fatality, the frequency of injury accidents was found to increase.

"Abnormal" accident frequency increase attributed to the median barrier is found normally distributed throughout all time periods.

Total property damage costs suffered, as well as costs of congestion arising from accidents occurring during peak periods, increased after median barrier construction.

ACKNOWLEDGMENTS

Appreciation is expressed to William Whitby, Bureau of Public Roads, for directing the use of the Traffic Analyzer and reviewing the results of Part I; and to Dale Hunter and James McAllister, Bureau of Electronic Data Processing, Pennsylvania Department of Highways, for significant help in processing the raw data.

REFERENCE

1. Taragin, A., and Hopkins, R. C. A Traffic Analyzer; Its Development and Application. Public Roads, Vol. 31, No. 5, pp. 120-124, Dec. 1960.

Dynamic Full-Scale Impact Tests of Bridge Barrier Rails

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Five full-scale dynamic impact tests were made on two basic geometric designs and one trial design of bridge barrier rails.

The three geometric designs included a standard California Type 1, a standard California Type 2, and an experimental modified Type 1 bridge barrier rail (Exhibit 1). Each was tested utilizing extruded aluminum pipe rail and cast aluminum posts mounted on a concrete parapet. The concrete parapet used for the aforementioned Type 1 test was repaired and the rail design was modified for two additional tests, one with steel pipe railing on welded steel plate posts (Type 1-A) and one with extruded aluminum pipe railing on malleable iron posts (Type 1-B).

This report describes the test procedures, instrumentation and test results based on the data secured from five high-speed oblique angle impacts on the five bridge barrier rail designs.

The test results indicated that all of the bridge barrier rails tested could effectively withstand the impact of a 4,300-lb passenger vehicle at speeds in excess of 75 mph. Minor cracking and spalling of the concrete portion of the Type 1 barrier was successfully repaired with epoxy/aggregate patches for use in two additional tests.

Specific recommendations are made for a balanced design bridge barrier rail system. (Exhibits are given in the Appendix).

•THE FIRST dynamic impact full-scale testing of bridge barrier rail systems was conducted by the State of California in 1955 (1). These bridge barrier rail tests were preceded by a preliminary test project in which 47 impact tests were conducted at various speeds and angles on 4 barrier curb designs with heights of from 9 to 12 in. The purpose of this preliminary test series was to evaluate the efficiency of various combinations of barrier curb faces and heights in retaining and deflecting a 50 to 60 mph impacting vehicle (2). Minimum curb height and face contour for the first bridge barrier rail designs were determined from the data recorded during these preliminary curb tests. The basic test procedures established during the 1955 test series have remained relatively constant over the past eight years and have also been used by other states, research organizations, and other countries as a basis for conducting similar studies of guardrails, barrier rails, and bridge barrier rails. A recently completed full-scale barrier test project conducted in England (3) using these test procedures produced excellent correlation with the results of the State of California's recent flexible barrier study.

Paper sponsored by Committee on Bridges and presented at the 43rd Annual Meeting.

In 1958 California adopted two standard barrier-type railings (Exhibit 1A and B). The details of this railing evolved from data secured in the 1955 full-scale dynamic tests as well as actual operational experience with barrier prototypes (Exhibits 2 and 3A). The metal portion of the barrier prototypes consisted of posts and pipe welded together. This arrangement was found to be heavy and cumbersome and required special handling during shipping and erection. Prefabricated post castings with pipe rail attached by means of bolt fasteners alleviated these problems.

These modifications provided a more efficient and economical bridge rail system but did not reflect a radical departure from geometric features of the original prototype designs. These were considered more as a utilization of the various new materials, techniques and construction methods advanced by the Division of Highways and the industry during the ensuing years.

In 1959 and 1960 three additional designs submitted by the Bridge Department were dynamically tested (4). It was during this 1959-60 test series that the inadequacy of the existing baluster-type rail in retaining a moderately high-speed vehicle was confirmed (Exhibit 3B). By comparing the test results from this 1959 series with those of the 1955 series, it was apparent that a solid, non-yielding smooth-wall barrier is more efficient and effective than a barrier containing balusters or any other type of opening that would trap the solid portions of the impacting vehicle.

OBJECTIVES

The basic objectives of the latest (1963) dynamic impact series were as follows:

1. To test the overall effectiveness of the California standard bridge barrier rail (Tests B-1 and B-2);
2. To determine the difference in performance between two reinforced-concrete parapet heights (Tests B-2 and B-4);
3. To determine the effect a curb has on the point of impact at the parapet (Test B-1);
4. To rate the relative effectiveneness of various combinations of metals used in posts and rails (B-3 and B-5); and
5. Compile factual data that would be of use in updating the railing specifications contained in the 1961 edition of AASHO.

CONCLUSIONS

The operational efficiency of any bridge barrier rail system in effectively resisting a severe passenger vehicle impact can be summarized and evaluated on the basis of meeting the four structural conditions listed below:

1. The bridge barrier system should retain the vehicle on the structure. The impacting vehicle should not penetrate or climb over the barrier.
2. The impact should not dislodge any parts of the barrier system. Rails, posts, and concrete should remain intact and not break away and fall to the pavement or over the side of the structure.
3. While in contact with the bridge barrier, the vehicle should progress smoothly along the rail with a minimum of snagging on any part of the system or pocketing of the elements. The barrier system should be designed to resist any severe deflection that could contribute to a post-impact roll of the vehicle.
4. All elements of a barrier system should be so designed that if repairs are necessary to place a damaged section in operating condition, they can be effected quickly and with a minimum of special equipment.

Table 1 gives the results of the five bridge barrier tests in reference to these four conditions.

As a result of this test series, new design loading specifications have been adopted and a laboratory test method for a static load test of the barrier posts has been developed (Exhibits 4A and B). The malleable iron posts dynamically tested in this series are now being used on 1963 state contracts.

TABLE 1

RESULTS OF BRIDGE BARRIER TESTS

Item	Test No.				
	B-1	B-2	B-3	B-4	B-5
Barrier type	2	1	1-A	Modified 1	1-B
Railing material	Extruded aluminum pipe	Extruded aluminum pipe	Steel pipe	Extruded aluminum p pipe	Extruded aluminum pipe
Post material	Cast aluminum	Cast aluminum	Welded steel plate	Cast aluminum	Malleable iron
Retained by barrier	Yes	Yes	Yes	Yes	Yes
Elements dislodged	None	3 railing sections and 3 posts	None	None	None
Smooth progression	Good	Fair	Good	Good	Good
Dynamic horizontal deflection of pipe railing	3 in.	20 in. before failure	$3\frac{1}{2}$ in.	2 in.	$6\frac{1}{2}$ in.
Vehicle rise	$2\frac{1}{2}$ in.	12 in.	3 in.	3 in.	10 in.
Ease of repair	Good	Fair	Good	Good	Good

DISCUSSION

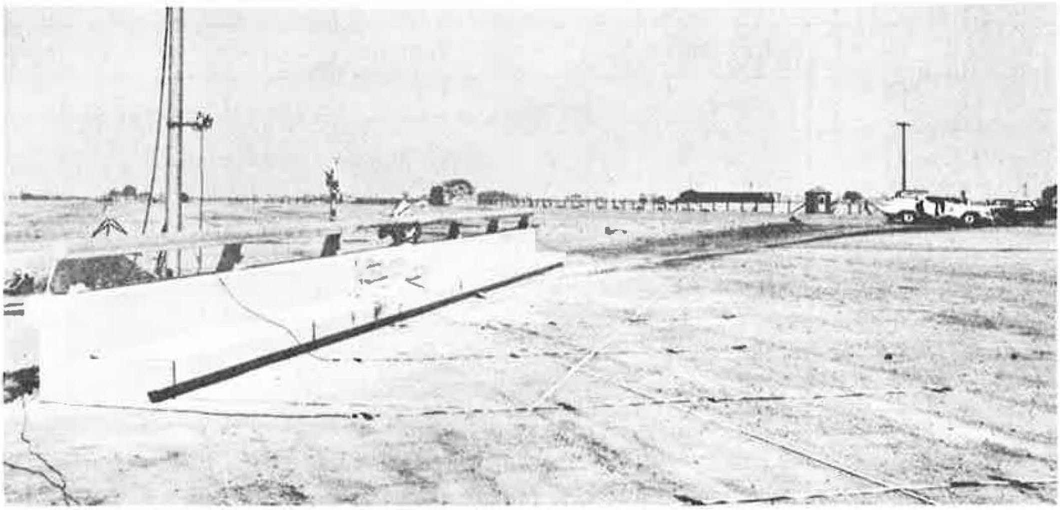
As indicated in the conclusions, the four basic conditions were met on all points by bridge barrier rail designs Type 2 and Type 1 modified. The Type 1-A design fulfilled all conditions to some degree and would be considered an effective design. The Type 1-B design was nearly as effective as the Type 1-A and utilized a more economical combination of post and railing materials.

The results of previous full-scale dynamic impact tests conducted at speeds below 60 mph proved that excessive deflection of a barrier system can result in post-impact roll-over (5). It is necessary to examine carefully the overall heights of the five barrier designs included in the 1963 study where the test vehicles impacted at speeds in excess of 75 mph. For example, it was noted that when the barrier parapet was of sufficient height to take a major portion of the impact load (barrier Type 2 and modified Type 1) there was no tendency for the impacting vehicle to climb. There was a tendency, however, for the vehicle to roll into contact with the modified Type 1 barrier due to severe deformation of the body and frame (Exhibit 12A).

At a parapet height of 28 in. these two barriers present smooth surfaces, well above the center of gravity of the impacting vehicle. However, even when the height of the concrete parapet was below the center of gravity of the vehicle, if the deflection of the posts and pipe railing was a minimum as in Test B-3 on the Type 1-A barrier, there was no tendency for the impacting vehicle to rise. When the parapet height was below the center of gravity of the vehicle and the pipe rail deflection was greater (as in Test B-5 on Type 1-B) or the posts failed (as in Test B-2 on Type 1), the vehicle had a tendency to climb. In the latter case where railing deflection was excessive, the vehicle almost vaulted the rail. However, other considerations such as sight distance, esthetics and cost also influence the overall design. It also should be realized that the impact forces applied during these tests represented the most severe to be reasonably encountered under normal operating conditions.

The following portion of the discussion is a detailed evaluation of the bridge barrier rail and vehicle performance in each of the five tests with reference to the four conditions outlined in the conclusions.

Test B-1 Type 2 Bridge Barrier Rail



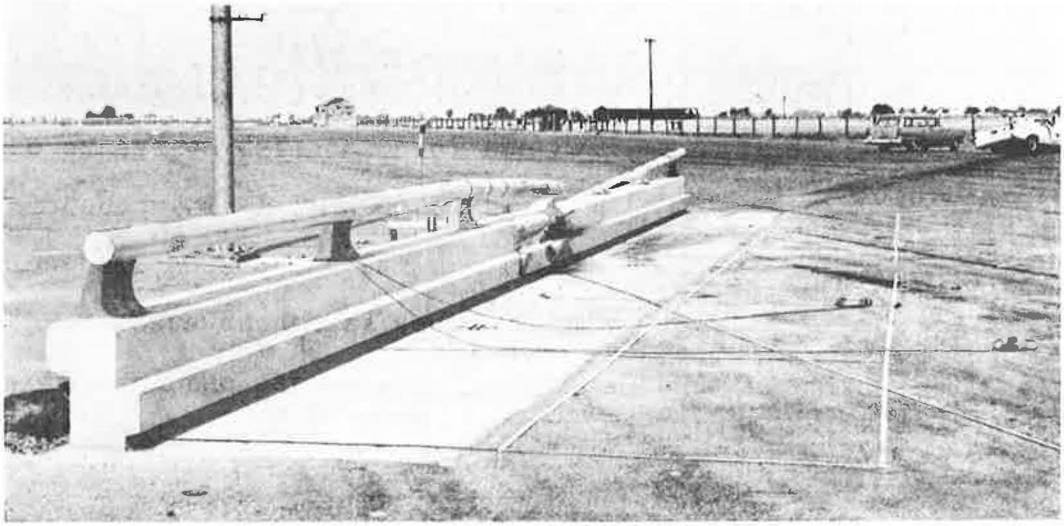
Bridge Rail: Standard Type 2 with aluminum posts and pipe railing. Height of concrete parapet: 28 in. Height of walkway: 10 in. Overall height of barrier: 43 in.

Purpose: To test the structural stability of the concrete parapet (as currently designed) in combination with aluminum posts and pipe railing, and to determine the jump that would be imparted to an impacting vehicle when crossing the two foot wide walkway at high speed and oblique angle.

Performance:

1. The vehicle did not penetrate the barrier rail. There was no evidence of rising even after crossing the walkway (Exhibit 5). However, it should be noted that previous tests (2) showed that there was a tendency to jump when the 10-in. high curb is struck at lower speeds and flatter angles.
2. All parts of the system remained intact. Damage to the concrete was minimal consisting of slight spalling and cracking.
3. There was a smooth progression of the vehicle through impact, even though the fender was torn from the body and lodged between the rail and parapet at the impact post (Exhibit 19).
4. Evidence of a slight spalling and scraping of the concrete was confined to the immediate impact area. Cracking occurred at the junction of the backwall and the walkway. The web of the first post contacted was ripped its entire length adjacent to the base. One section of the aluminum pipe railing was slightly gouged by the vehicle fender and was bent approximately 2 inches (Exhibit 6).

Test B-2 Type 1 Bridge Barrier Rail



Bridge Rail: Standard Type 1 with aluminum posts and pipe railing. Height of concrete: 21 in. Overall height of barrier: 36 in.

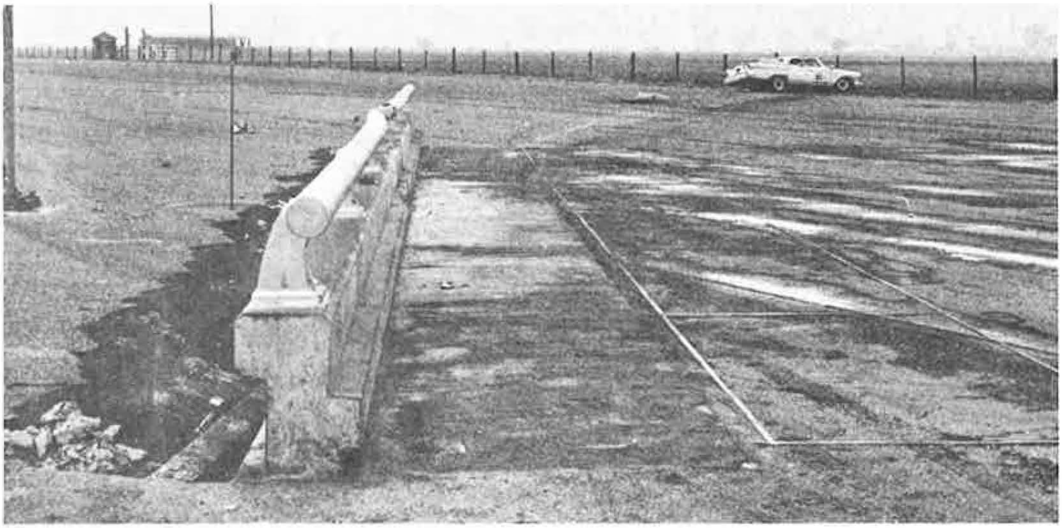
Purpose: To test the current design for structural stability of the concrete and aluminum railing element combination.

Performance:

1. Although the vehicle did not penetrate the barrier, it rose approximately 12 in. within 6 ft after initial contact due to excessive deflection of the railing (20 in.). This rise placed critical loading on the railing elements which resulted in their subsequent failure (Exhibit 7).
2. Three posts and three sections of the aluminum pipe railing were torn from the parapet and dropped to the ground behind the barrier. Major cracks developed in the concrete parapet.
3. The vehicle progressed fairly smoothly through impact even though the frame dragged on top of the parapet and sheared three posts before leaving the barrier (Exhibit 20). Excessive deflection of the railing elements and resultant vehicle jump imparted a rolling moment to the vehicle that almost caused a post-impact roll over.
4. Deep spalling and severe cracking of the concrete occurred in the impact area. Three posts were completely sheared from the barrier, and the web of the last post struck was bent. Three sections of the aluminum pipe rail were knocked to the ground behind the barrier; however, the sections remained intact and connected to the last post. Had there been adequate clearance behind the barrier for the pipe to drop, it is felt that the three pipe sections would have broken from the system and fallen to the ground (see Exhibit 8).

The cracking of the concrete parapet was not considered serious enough to necessitate complete replacement of the 5-ft cracked section adjacent to the expansion joint. Based on operational experience, it was felt that an epoxy resin bonding agent pumped into the cracks would have been sufficient to place the barrier in operating condition. However, as this parapet was to be used for two additional tests and a failure in this section would have influenced the test results, the concrete was jackhammered out, leaving the reinforcing bars intact. Plywood forms were clamped across the open section and a new concrete repair section was poured. The new concrete was bonded to the existing section with an epoxy adhesive applied to the exposed edges. An epoxy-sand mixture was used in resetting the damaged anchor bolts on the fourth post ahead of impact.

Test B-3 Type 1-A Bridge Barrier Rail



Bridge Rail: Type 1 parapet with welded steel plate posts and steel pipe railing. Height of concrete: 21 in. (repaired after Test B-2). Overall height of barrier: 36 in.

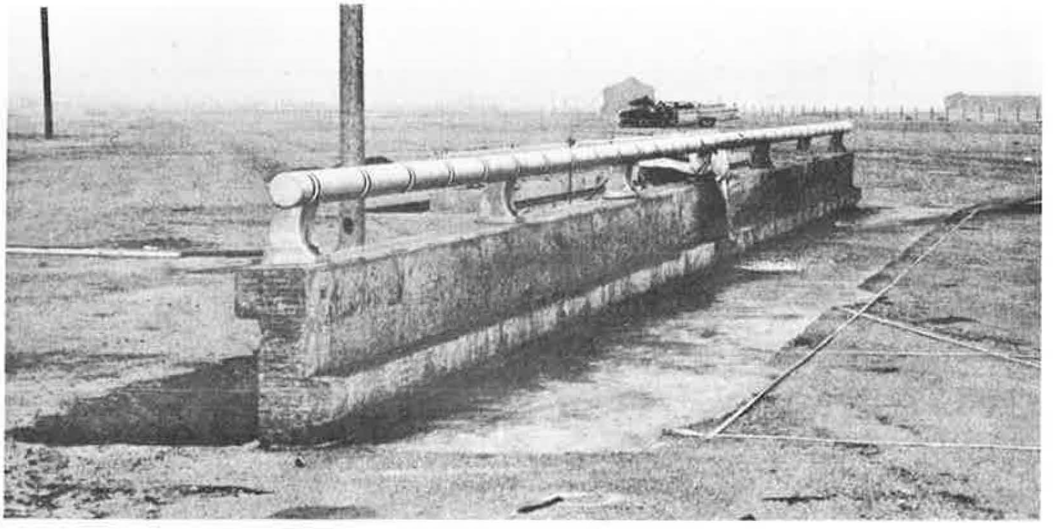
Purpose: To test the currently designed parapet with the steel posts and steel pipe rail. The posts and rail were purposely overdesigned so that they would not fail under impact loading. It was felt that this test would indicate the maximum performance of the concrete parapet and would assist in determining the relative loading on each part of the system.

Performance:

1. The vehicle did not penetrate the barrier. There was very little tendency for the vehicle to rise and no tendency to climb (Exhibit 9).
2. All parts of the barrier remained intact.
3. The vehicle progressed smoothly through impact with no tendency to roll (Exhibit 21).
4. The concrete developed deep cracks behind the first steel post contacted and required replacement of a small section of concrete around the anchor bolts. The front face of the concrete parapet was severely scraped and spalled in the impact area. One section of the steel railing was bent approximately 4 in. and deeply gouged. The welded steel plate posts sustained the impact with no evidence of damage (Exhibit 10).

The section repaired after the previous test withstood the impact with no evidence of cracking. From a design standpoint it is interesting to note that two anchor bolts effectively developed the strength of the relatively strong welded steel post.

Test B-4 Type 1 Modified Bridge Barrier Rail



Bridge Rail: Modified Type 1 concrete parapet with aluminum posts and aluminum pipe railing. Height of concrete: 28 in. Overall height of barrier: 43 in.

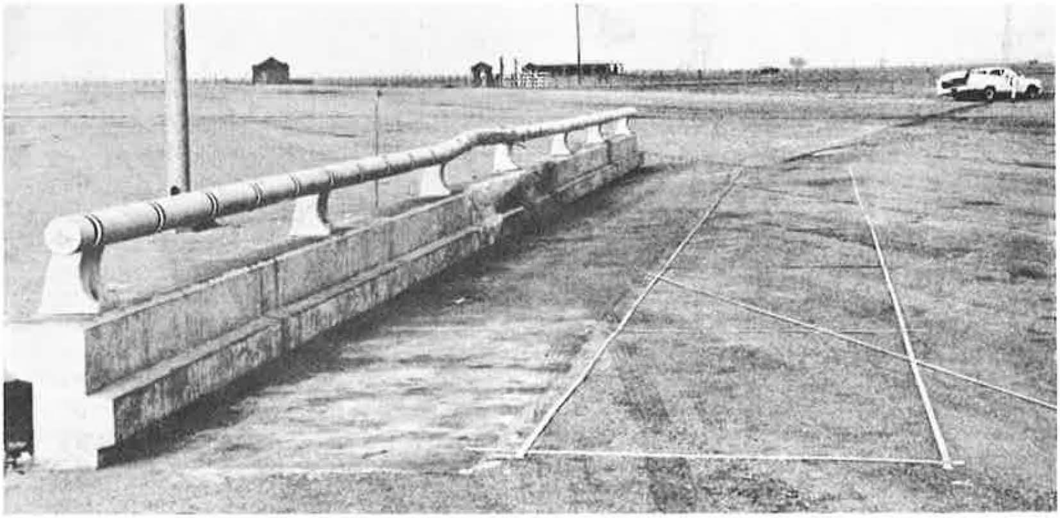
Purpose: To test and observe the effects of increasing the concrete parapet wall height of the Type 1 bridge rail from 21 in. to 28 in. while retaining the aluminum posts and pipe railing from the previous Type 1 test and to evaluate the effect of a 28-in. parapet height without safety walkway by comparison with Test B-1.

Performance:

1. The vehicle did not penetrate the barrier. There was very little tendency for the vehicle to rise and no tendency to climb (Exhibit 11).
2. There was no structural failure in any of the barrier elements.
3. The vehicle progressed reasonably smoothly through impact considering that the front left fender and door panel were torn from the vehicle and lodged between the pipe rail and parapet at the first post after impact (Exhibit 22).
4. Slight spalling and scraping of the concrete was confined to the immediate impact area. The flange of the first post contacted was bent and would have required replacement; however, there was no evidence of failure. One section of the aluminum pipe rail was bent 2 in. (Exhibit 12).

No repairs to the concrete parapet would have been required to place the barrier in operating condition.

Test B-5 Type 1 Bridge Barrier Rail



Bridge Rail: Type 1 parapet with malleable iron posts and aluminum pipe railing. Height of concrete: 21 in. (repaired after Test B-3). Overall height of barrier: 36 in.

Purpose: To determine the efficiency of the currently designed parapet with malleable iron posts under impact loading conditions similar to that of Test B-2.

Performance:

1. The vehicle did not penetrate the barrier. Deflection of the aluminum pipe rail (6½ in.) contributed to the 10 in. rise of the vehicle. This rise was considerably higher than that recorded in Test No. B-3, the same parapet with a steel pipe rail and steel plate posts (Exhibit 13).
2. All elements of the barrier system remained intact.
3. The vehicle progressed smoothly through impact with no tendency to roll (Exhibit 23).
4. A moderate amount of spalling of the concrete behind the anchor bolts on the first post after impact would have required an epoxy-sand patch and resetting of the anchor bolts to place the barrier in operating condition (Exhibit 14). The epoxy-sand anchor bolt repair from Test B-3 withstood direct impact with no evidence of failure.

TABLE 2

APPARENT EFFECTIVENESS

Barrier Design	Parapet Height (in.)	Post/Pipe Railing
Type 2	28	Cast alum., extruded alum.
Type 1 (mod.)	28	Cast alum., extruded alum.
Type 1-A	21	Welded steel plate, steel pipe
Type 1-B	21	Malleable iron, extruded alum. pipe
Type 1	21	Cast alum., extruded alum.

RECOMMENDATIONS

The five barrier designs are rated in Table 2 in order of their apparent effectiveness in meeting the forementioned structural conditions.

Specific recommendations for balanced design are as follows:

1. Barrier Height: The 21-in. parapet height (36-in. overall height) in the Type 1 bridge barrier rail and the 28-in. parapet height (43-in. overall height) in the Type 2 bridge barrier rail with safety walkway are adequate providing the pipe railing elements will sustain at least a 20,000-lb static loading, as indicated in Exhibit 4B and provided the metal posts are ductile enough to test to 10 percent minimum elongation (see Exhibit 4A and B for specifications and test methods).

2. Beam Strength of Parapet Wall: All three impacts on the Type 1 rail were made at the same concrete parapet wall expansion joint. There was only one structural failure of the wall and the vehicle did not penetrate. Therefore, it is felt the thickness and the size and distribution of the reinforcing steel is adequate. No changes are recommended in the basic structural design of the parapet wall.

3. Rubbing Curb: Based on the results of this test series on the Type 1 and Modified 1 bridge barrier rails, the rubbing curb is considered an unnecessary feature that complicates the forming for construction and adds to the cost. This rubbing curb does not function as a wheel deflector as originally intended. In all but the most narrow-angle low-speed contacts, the front and side overhang on the modern domestic passenger vehicle prevents the tire from contacting this curb before the body scrapes the parapet. Should the face be extended to more than the present 4 in., in an attempt to redirect the vehicle wheel in casual impacts, there is a strong possibility that a vehicle contacting the Type 1 at a narrow angle would mount the curb, climb the 21-in. high parapet, and vault the barrier. Therefore, if a wider rubbing curb is desired, the parapet wall should be 28 in. high as provided in the Type 2 design.

4. Repairs to Damaged Concrete Parapet: The epoxy-aggregate method of repairing damaged sections of concrete barriers has been used in the field for several years. The success of this repair method has been confirmed by the results of these controlled impact tests in which two successive impacts were concentrated on a repaired section with no evidence of cracking in the repaired joint.

CONSTRUCTION

Each of the three bridge barrier parapets was constructed on a reinforced-concrete deck section cantilevered from a 3-ft × 3-ft concrete anchor block (Exhibit 15). The earth was removed under and in back of this simulated bridge deck section so that there would be no restriction of the barrier/deck sections during impact. Bridge barrier rail Type 2 was constructed on the deck section after a bonding release agent had been sprayed over the entire area. The bonding release agent would be used to simplify curb removal for future widening. This test series indicated that use of this agent has no adverse effect on the structural strength of the construction joint. Reinforcing bar dowels, embedded in the slab during the anchor block construction, withstood the full impact loading, with the construction joint offering no other resistance to the load.

The Type 2 barrier rail resisted the full loading of the impact with no evidence of separation of the safety walk or parapet from the deck.

The Type 1 and Type 1 Modified barrier rail parapets were erected approximately

7 days after the deck section was poured and were provided with conventional construction joints. There were no evidences of failures in the construction joints of either the Type 1 Modified barrier after one impact or in the Type 1 barrier after 3 impacts.

Type A concrete with 1½-in. maximum aggregate and 6 sacks of cement per cubic yard was used on the anchor block, deck section and barrier parapet walls for all installations. Concrete test cylinders showed 28 day compressive strengths in excess of 3, 500 psi.

INSTRUMENTATION

Test Vehicles

The five test vehicles used in this 1963 research project were selected from a group of retired California Highway Patrol Dodge sedans, 1959 and 1960 models. The center of gravity of this special police pursuit model is approximately 22 in. above the pavement and at 4, 000-lb is slightly heavier than the standard Dodge sedan available to the public. These models were fitted with special sway bars for increased stability in making short-radius high-speed turns. Consideration was given to the effect of this increase in stability on correlation of the test results with previous bridge barrier tests which were conducted with standard models. However, since this test series was conducted primarily to determine the efficiency of the various barrier designs in effectively retaining vehicles at high-speed oblique impact, it was felt that this added stability would not affect the test results. This vehicle also offered other advantages over standard sedans that would have been selected from used car lots. The superior acceleration allowed the use of a very short impact course. Smoothness of the automatic shift permitted the cars to be started in gear with the engine running, rather than pushing to start as in previous test series. Since more than 100 vehicles were available, it was possible to select vehicles with similar steering, acceleration, and shifting responses.

The test vehicles were modified for remote radio control as follows:

1. A solenoid-valve actuated CO_2 system was connected directly to the brake line for fast remote brake application. With 700 psi in the accumulator tank, the brakes could be locked in less than 100 milliseconds. By pulsing the braking system, the car could be brought to a normal stop (if a run was aborted) with no tendency to slide or spin.
2. The throttle linkage was attached to a linear actuator energized by manually throwing a switch mounted on the trunk deck of the test car.
3. The ignition system was connected to the brake relay in a fail-safe interlock system. When the brakes were applied, the ignition was switched off. Any loss of radio signal or failure in the transmitting or receiving equipment would automatically energize the brake relay and switch off the ignition.
4. The gas tank was removed and replaced by a one gallon fuel tank equipped with a special cut-off valve to prevent fuel leakage in the case of a fire or roll-over.
5. Steering was controlled by a 2-HP gear-head motor (mounted on the front floorboard on the passenger side) through a V-belt connected to a pulley clamped on the steering wheel.
6. Two 12-volt storage batteries mounted on the floor of the rear seat supplied power to the remote control equipment.
7. The remote radio control receiver, tone actuated relays, steering pulse, and handi-talkie were mounted on a plywood panel in the trunk compartment. Whip antennae were mounted on the rear fender wings.

After five years of experience, the required time for remote control installation has been reduced to less than eight manhours per vehicle.

The three basic functions considered necessary for the safe, flexible operation of a crash car are: brakes on-off, ignition, on-off, steer right-left.

Control of the vehicle along the impact course was accomplished by a remote operator following 200 ft behind the test vehicle in a control car equipped with a tone transmission system. After sustaining more than 50 high-speed impacts over a period of 10 years, this remote control radio equipment continues to function efficiently with damage limited to an occasional shorted tube or broken solder joint.

Acceleration Instrumentation

Acceleration data representative of the forces a human driver would sustain under similar impact conditions were recorded by means of a triaxial mechanical-stylus accelerometer mounted in the chest cavity of a Sierra Engineering Co., Model 157, anthropometric dummy. The dummy was placed in the driver's seat and restrained by a lap belt and/or shoulder harness system. No attempt was made to relate deceleration information or dummy injuries to actual injuries that would have been sustained by a human counterpart. The primary function of the dummy was to evaluate the relative efficiency of various restraint systems in the prevention of partial ejection.

Photographic Instrumentation

The primary concern when considering the instrumentation for a research project of this scope is that an efficient method of gathering the pertinent data be provided. Experience has indicated that photographic records provide the most effective and dependable data coverage. In order to cover the event effectively with a minimum of cameras, it is essential to use cameras with a reliability approaching 100 percent. The six Photosonic Model 1-B 16-mm data cameras used for data coverage in this test series proved to be 100 percent reliable.

Equally important is the provision for recording significant data for documentary presentation. It has been found that curves and graphs based on data film records supplemented with documentary photographs provide an effective method of presentation. Documentary coverage for the past four test projects has been provided by a scaffold-mounted 70-mm sequence camera recording at 20 frames per second and at a shutter speed of 1/2,000 sec. During this 1963 test series, a cloud of concrete dust was produced by the vehicle impact and abrasion. Attempts to remedy this situation by the application of various penetrating oil dust palliatives proved unsuccessful. This dust obscured much of the action from the documentary sequence camera. Therefore, this camera will be located ahead of impact in future testing of concrete barriers.

In reducing data for past test series, it has been difficult to recover significant information, other than roll and jump data, from the ground-mounted data cameras. Since camera placement is not critical for gathering roll and jump data, ground positions for these cameras were not located by triangulation. To protect them from damage, the two ground-mounted data cameras equipped with 4-in. telephoto lenses and placed on line with the barrier face at locations 200 ft behind and 200 ft ahead of the point of impact. Although the 4-in. lens restricted angular coverage, it was felt that, for data reduction, the large image provided by this type of lens would be more useful than unrestricted coverage.

The three overhead data cameras, however, were carefully oriented and sighted-in for accurate recording of all data considered of any importance to this type of study. These cameras, mounted on a 35-ft tower, furnished coverage from 25 ft ahead of the point of impact to 25 ft beyond (Exhibits 19 through 23). For data reduction from the overhead cameras, a cloth tape grid was placed on the ground in the impact area. Preliminary static shots of the vehicle progressing in 5-ft increments through the impact area prior to each test were later projected and drawn on a screen for ground correlation of the vehicle through impact. Tape switches placed at 10-ft intervals leading into the point of impact were actuated by the approaching vehicle. Tire contact with the tape switches triggered a series of five flash bulbs located in view of all data cameras. These flash bulbs were also viewed by the crash-car mounted data camera to provide frame rate and event correlation with the ground-mounted and overhead data cameras. Table 3 gives information concerning the cameras used in this series.

All data cameras and the Hulcher documentary camera were motor driven and, with the exception of the crash car mounted data camera, were manually actuated from the central control console (Exhibit 16). The Bolex and Arriflex documentary cameras were motor driven and hand panned through impact. The crash-car mounted data camera was actuated along with the dummy accelerometer recorder by means of a release-pin-triggered switch on the bumper of the crash vehicle. The release pin was attached to a 50-ft length of nylon line anchored in the pavement directly behind the car.

TABLE 3

CAMERA INFORMATION

Camera	Type	FPS	Lens	Film	Location	Function
1	Photosonics 1-B	400	4 in.	16 mm	Front gnd.	Data
2	Photosonics 1-B	400	4 in.	16 mm	Rear gnd.	Data
3	Photosonics 1-B	400	½ in.	16 mm	Tower	Data
4	Photosonics 1-B	400	½ in.	16 mm	Tower	Data
5	Photosonics 1-B	400	½ in.	16 mm	Tower	Data
6	Photosonics 1-B	250	5.6 mm	16 mm	Crash vehicle	Data
7	Hulcher 70	20	6 in.	70 mm	Rear scaffold	Doc.
8	Bolex	24	Various	16 mm	Various	Doc.
9	Arriflex	24	Various	16 mm	Various	Doc.

After the crash vehicle had progressed 50 ft down the impact course, the pin was pulled from the switch and all data recording equipment including accelerometers were energized.

Data Correlation

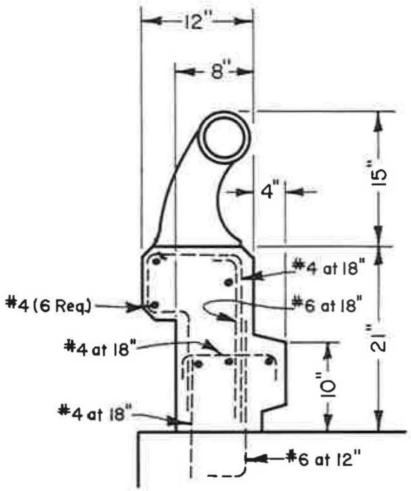
With the exception of the crash-car mounted camera, all data cameras were provided with a 1,000-cycle timing pulse projected on the data film records. The tape-switch actuated flash bulbs provided event correlation between all stationary cameras and were also used to establish frame rate and event correlation for the data camera located in the test vehicle. Flash bulbs mounted in the taillights of the test vehicle were used to establish vehicle location and the time at which the brakes were applied. The bulbs also served to alert the control car driver that the test car brakes had been applied. These flash bulbs were fired when the brake actuating relay was pulsed by the remote operator or when the remote radio equipment failed. This brake pulse was also connected to a solenoid-actuated stylus in the accelerometer recorder that provided an event marker on the recorder paper. The recorder chart drive in the accelerometer unit was governor-controlled to a chart speed of 1 in./sec. The oscillograph recordings from the strain gages attached to steel reinforcing bars in the concrete barrier installations were also correlated to the event by means of the flash bulb pulses from the tape switches. These pulses, recorded on the oscillograph chart, provided an accurate method of correlation with the data cameras and (as a convenience) an immediate check on the average vehicle velocity over the 50-ft section prior to impact. The tape switch/flash bulb method of event and timing correlation is considered sufficiently accurate for this type of information and is readily reducible from the various data film and oscillograph records.

Stress-Strain Instrumentation

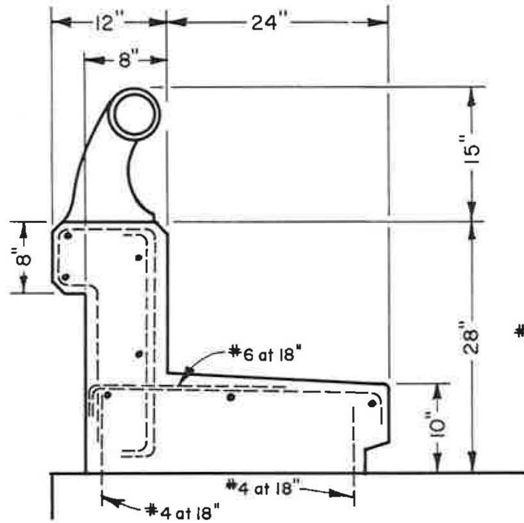
For each of the three basic types of concrete barrier parapet, reinforcing bar dowels in the impact area were instrumented with Baldwin bonded-type strain gages installed (Exhibit 17A). In order to determine the effect of dynamic loading on each bridge barrier rail and deck as a system, the barriers were constructed on a cantilevered anchor block (Exhibit 17B). These dynamic readings showed that the maximum stress occurred on the vertical reinforcing bars located approximately 4 ft beyond the point of impact. By using mechanical stylus gages, it was also possible to obtain direct measurement of the movement sustained by the entire system. These gages were cast in the concrete parapet and referenced to the ground behind the installation (Exhibit 17C). The stylus arms were constructed of ¼-in. brass rods with a short length of piano wire for the marking stylus. Deflection curves were recorded on a special waxed paper attached to stakes driven in the ground behind the barrier. Exhibit 18A shows a typical recording at a single gage point. Exhibit 18B shows the horizontal and vertical deflections recorded under impact at deck and parapet wall locations over the entire length of the Type 1-B barrier rail. These recordings are typical.

REFERENCES

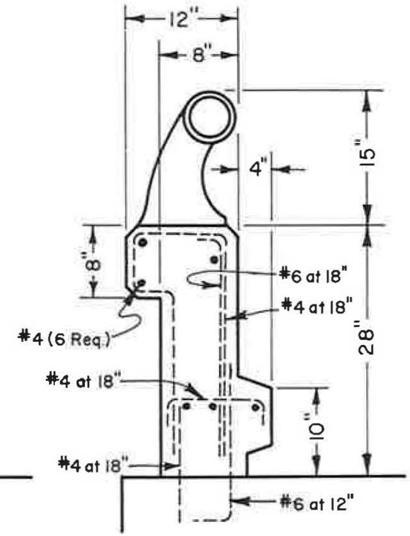
1. Beaton, John L. Full Scale Tests of Concrete Bridge Rails Subjected to Automobile Impacts. HRB Proc., 35:251-267, 1956.
2. Beaton, J. L., and Field, R. N. Final Report of Full Scale Dynamic Tests of Bridge Curbs and Rails. Calif. Division of Highways, Aug. 30, 1957.
3. Bender, H. M., and Mason T. Darfen Road Safety Barriers. British Ropes Limited report.
4. Beaton, J. L., and Field, R. N. Dynamic Full Scale Tests of Bridge Rails. Calif. Division of Highways, Dec. 1960.
5. Beaton, J. L., and Field, R. N. Dynamic Full-Scale Tests of Median Barriers. HRB Bull. 266, 78-91, 1960.



A. CALIFORNIA STANDARD BRIDGE BARRIER RAILING TYPE 1



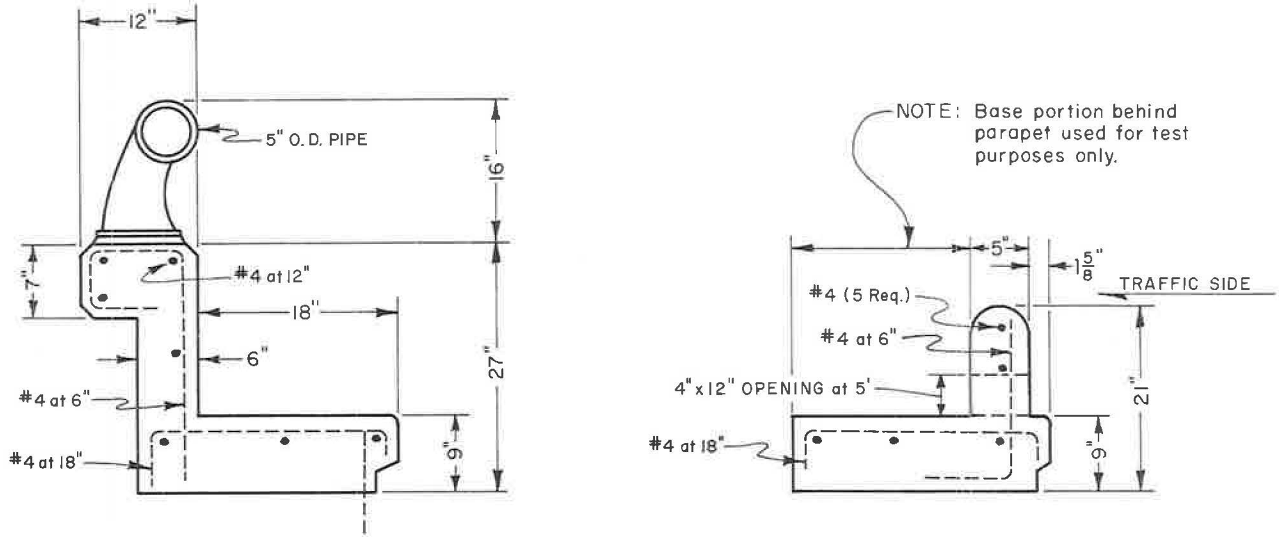
B. CALIFORNIA STANDARD BRIDGE BARRIER RAILING TYPE 2



C. EXPERIMENTAL BRIDGE BARRIER RAILING MODIFIED TYPE 1

1963 BRIDGE BARRIER RAIL TESTS

EXHIBIT 1



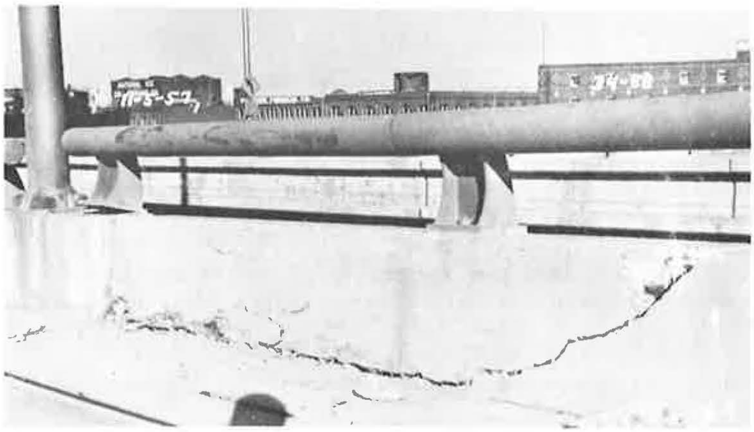
A

B

BRIDGE RAIL & CURB PROTOTYPES

EXHIBIT 2

Exhibit 3



A. Early Barrier Prototype showing insufficiency of reinforcing steel in parapet. (Also see Exhibit 2A).

B. Baluster type railtested during the 1959 test series. Impact speed 57 mph at 28 degrees approach angle.

EXHIBIT 4A

PERTINENT BRIDGE BARRIER RAIL SPECIFICATIONS

Metal railing shall be steel pipe with steel rail caps or aluminum pipe with aluminum rail caps, and metal posts. Steel rail caps may be either cast steel or malleable iron, or nodular iron.

Bolts and nuts for attaching the pipe to the posts and anchor bolt assemblies shall be steel.

Pipe and posts may be of the same or dissimilar metal, but on each bridge or retaining wall the metal railing shall be all of the same details and the same combination of metals.

Post material and the completed posts shall conform to the following:

1. Material shall be a ferrous or aluminous metal. The chemical and physical properties as required to conform to the provisions of this section shall be selected by the Contractor.
2. Metal cut from the side flanges of the post shall have an elongation of 10 percent minimum, when sampled and tested in accordance with Test Method No. Calif. 654-A.
3. Posts shall support a load of 20, 000 pounds when the load is applied and the test conducted in accordance with Test Method No. Calif. 654-A.
4. The dimensions and thicknesses of metal shown on the plans shall be the minimum permitted.
5. The sections of the post may be increased in thickness at the option of the Contractor as required to provide a post that will comply with the test requirements of Test Method No. Calif. 654-A.

The outside dimensions of the post shall not be increased. Increasing the thickness of vertical flanges and top member shall be done uniformly. Bulbs or ribs in addition to those shown on the plans will not be permitted.

The materials, except for posts, shall conform to the following requirements:

Material	ASTM Designation
Steel pipe	A 139
Steel structural tubing	A 53
Steel rail caps and block washers	A 27, Grade 65-35; or A 47, Grade 32510; or A 395
Steel bolts and Nuts	A 307
Aluminum pipe	B 235, 6063-T6
Aluminum rail caps and block washers	B 108, SG70B; or B26, SG70A

1. Steel pipe and tubing shall have a wall thickness not less than $\frac{3}{16}$ inch.
   Steel tubing conforming to American Petroleum Institute Specifications, 5L or 5LX will be accepted.
2. Aluminum pipe for single pipe railings shall have a wall thickness not less than $\frac{1}{4}$ inch.
   Aluminum pipe for multiple pipe railings shall have a wall thickness not less than $\frac{3}{16}$ inch.

EXHIBIT 4B

Test Method No. Calif. 654-A
September 1963

State of California
Department of Public Works
Division of Highways
MATERIALS AND RESEARCH DEPARTMENT

METHOD FOR TESTING BARRIER RAILING POSTS

Scope

This test method describes the procedures to be used in testing barrier railing posts. The tests include a strength test of the completed post and an elongation test on a specimen cut from the post.

Procedure

A. Apparatus

1. For the strength test of the post, use a static test jig which will provide for loading as shown in Figures A, B, and C. Apply the load by means of a compression testing machine or similar apparatus.

2. Refer to ASTM Designation: E8 for description of apparatus used to determine percent elongation.

B. Test Procedure

1. Bolt barrier railing post in test jig, apply test load to railing post as shown in Figure D and measure maximum load that the post will support without failure.

2. Take test sample from the railing post for determining the elongation, as shown in Figure E. Prepare standard test specimen and determine percent elongation as described in ASTM Designation: E8.

Reporting of Results

Report test results on Form T-616.

REFERENCE

ASTM Designation: E8
End of Text on Calif. 654-A

Exhibit 5

TEST B-1

I-.023 sec.

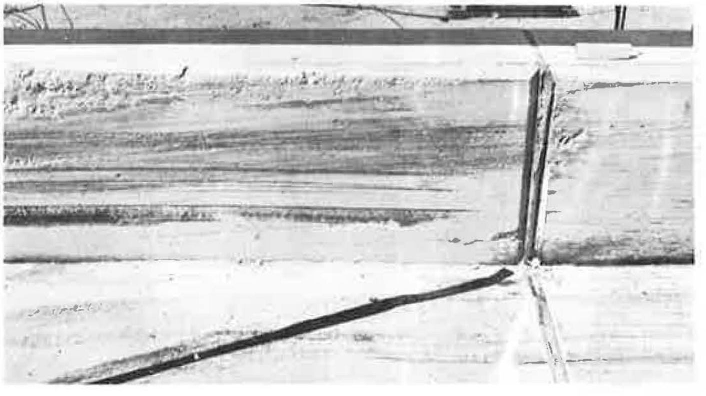
I+.011 sec.

I+.143 sec.

I+.320 sec.

I+.649 sec.

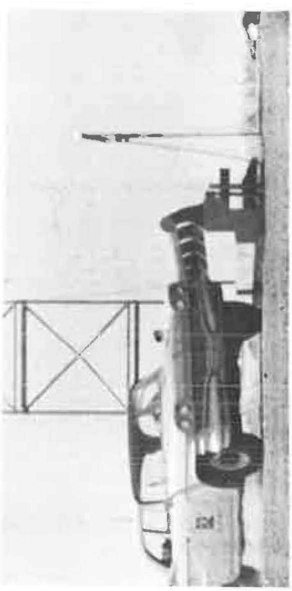
Exhibit 6



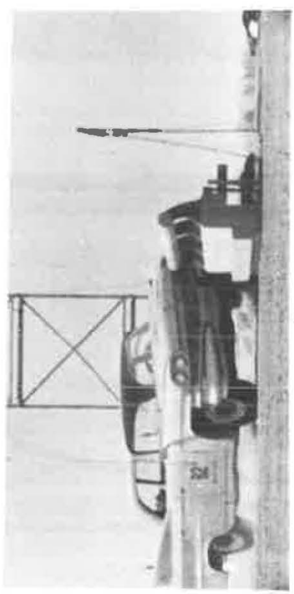
TEST B-1 VEHICLE & BARRIER DAMAGE

Exhibit 7

TEST B-2



I+.006 sec.



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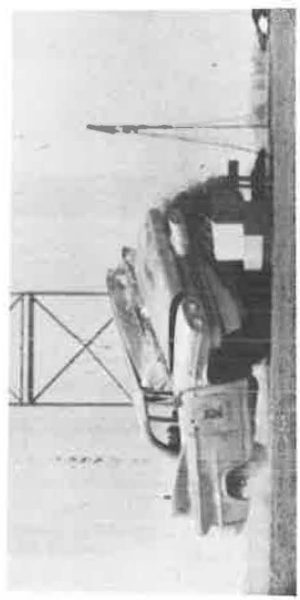
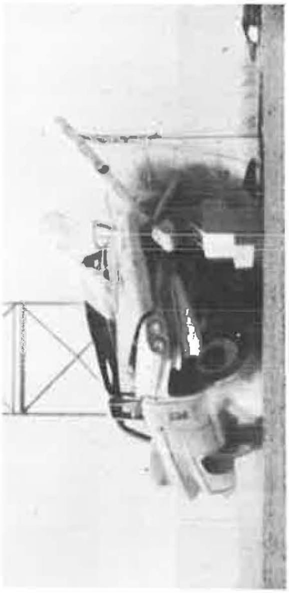


I+.094 sec.

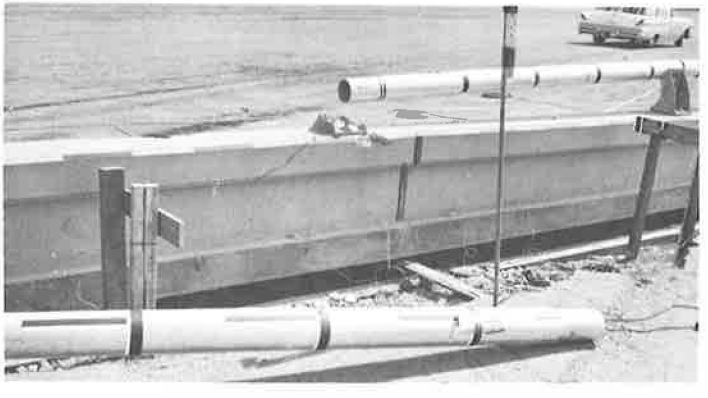


I+.194 sec.

I+.300 sec.

I+.556 sec.

Exhibit 8



TEST B-2 VEHICLE & BARRIER DAMAGE

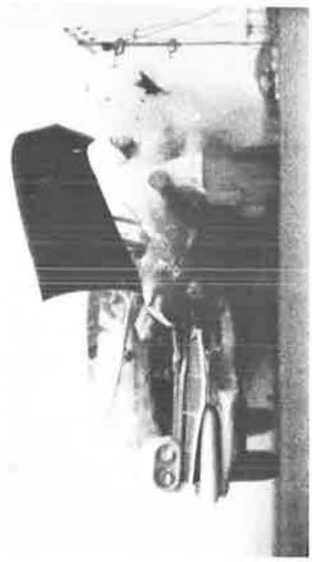
Exhibit 9

TEST B-3

I+.006 sec.

I+.065 sec.

I+.152 sec.

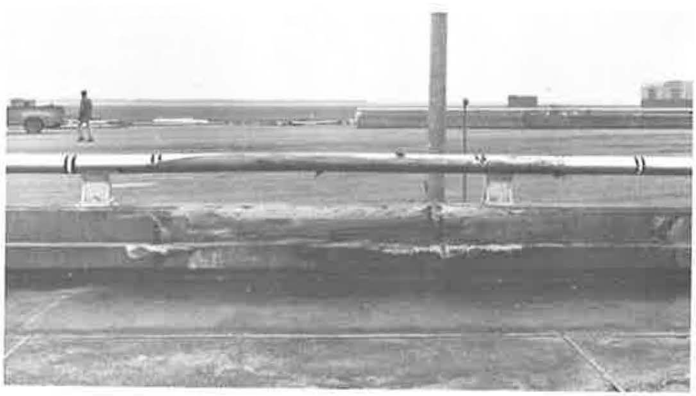


I+.256 sec.

I+.390 sec.

I+.575 sec.

Exhibit 10



TEST B-3 VEHICLE & BARRIER DAMAGE

Exhibit 11

TEST B-4

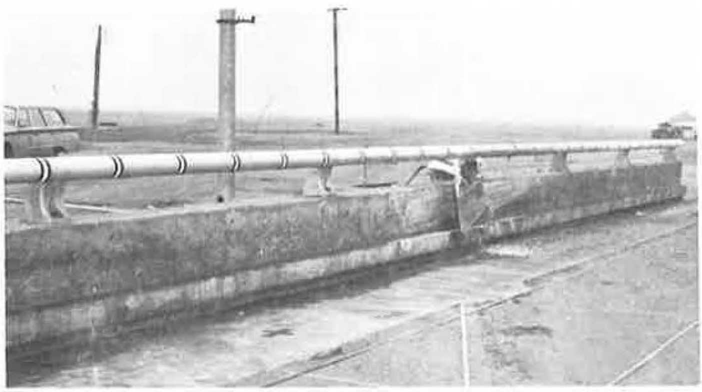
Impact

I+.211 sec.

I+.069 sec.

I+.422 sec.

Exhibit 12



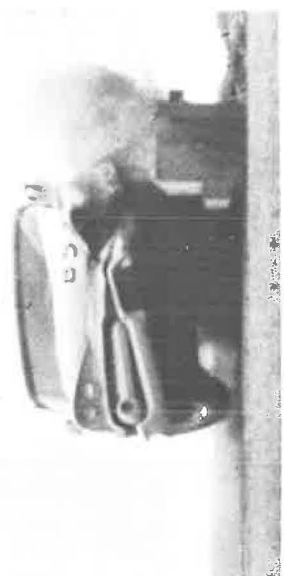
TEST B-4 VEHICLE & BARRIER DAMAGE

Exhibit 13

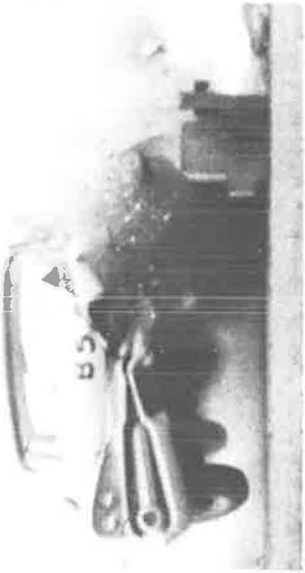
TEST B-5

I-.057 sec.

I+.063 sec.



I+.169 sec.



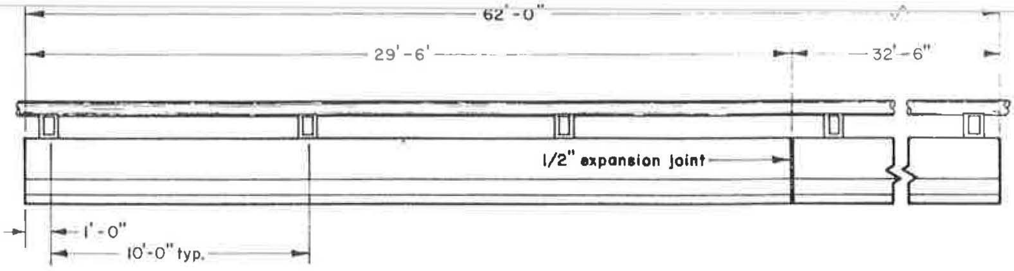
I+.329 sec.

I+.571 sec.

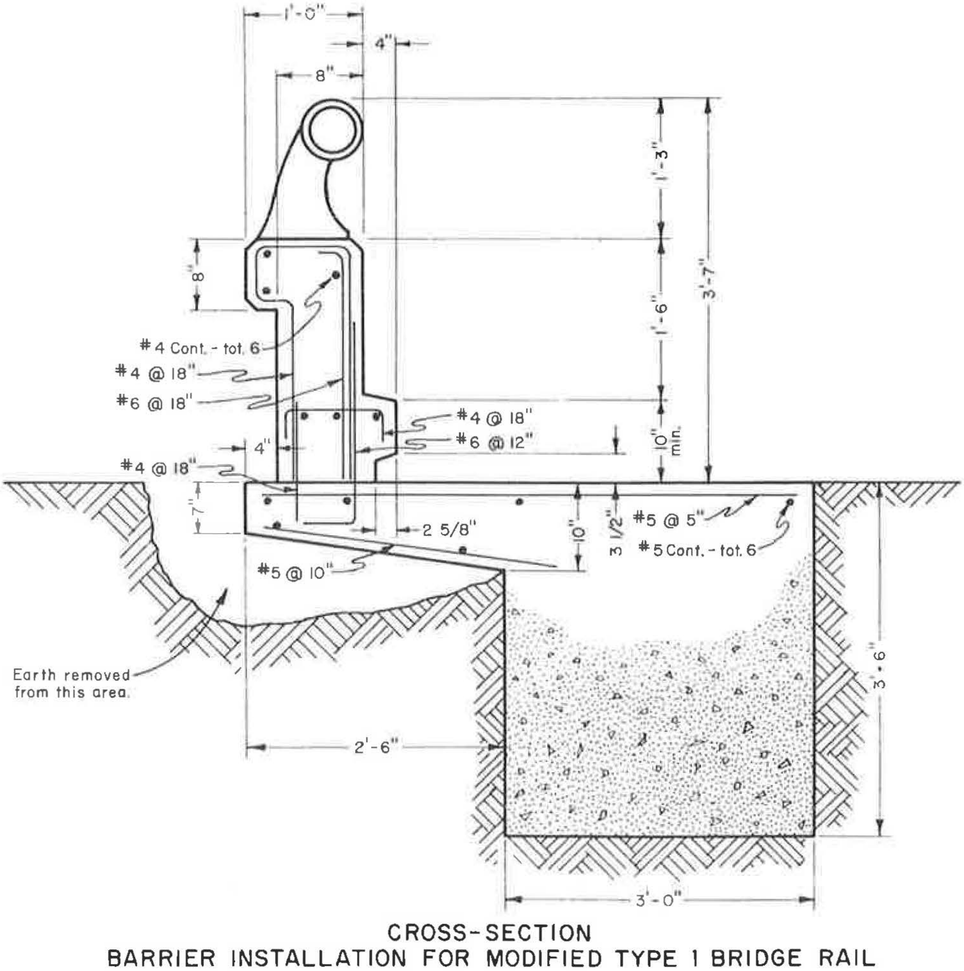
Exhibit 14

TEST B-5 VEHICLE & BARRIER DAMAGE

EXHIBIT 15



FRONT VIEW



CROSS-SECTION
BARRIER INSTALLATION FOR MODIFIED TYPE 1 BRIDGE RAIL

View of Test Site showing general layout. Control center is between camera scaffold and instrumentation trailer.

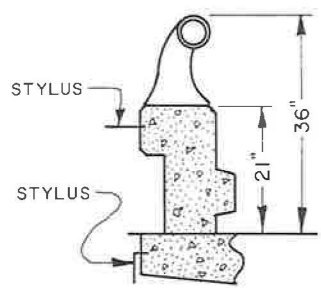
Exhibit 16

Exhibit 17

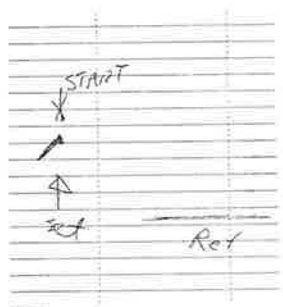
A, Installing strain gage instrumented re-bar dowels for barrier Type 2.

B. Barrier installation from rear showing cantelevered deck section and strain gage leads.

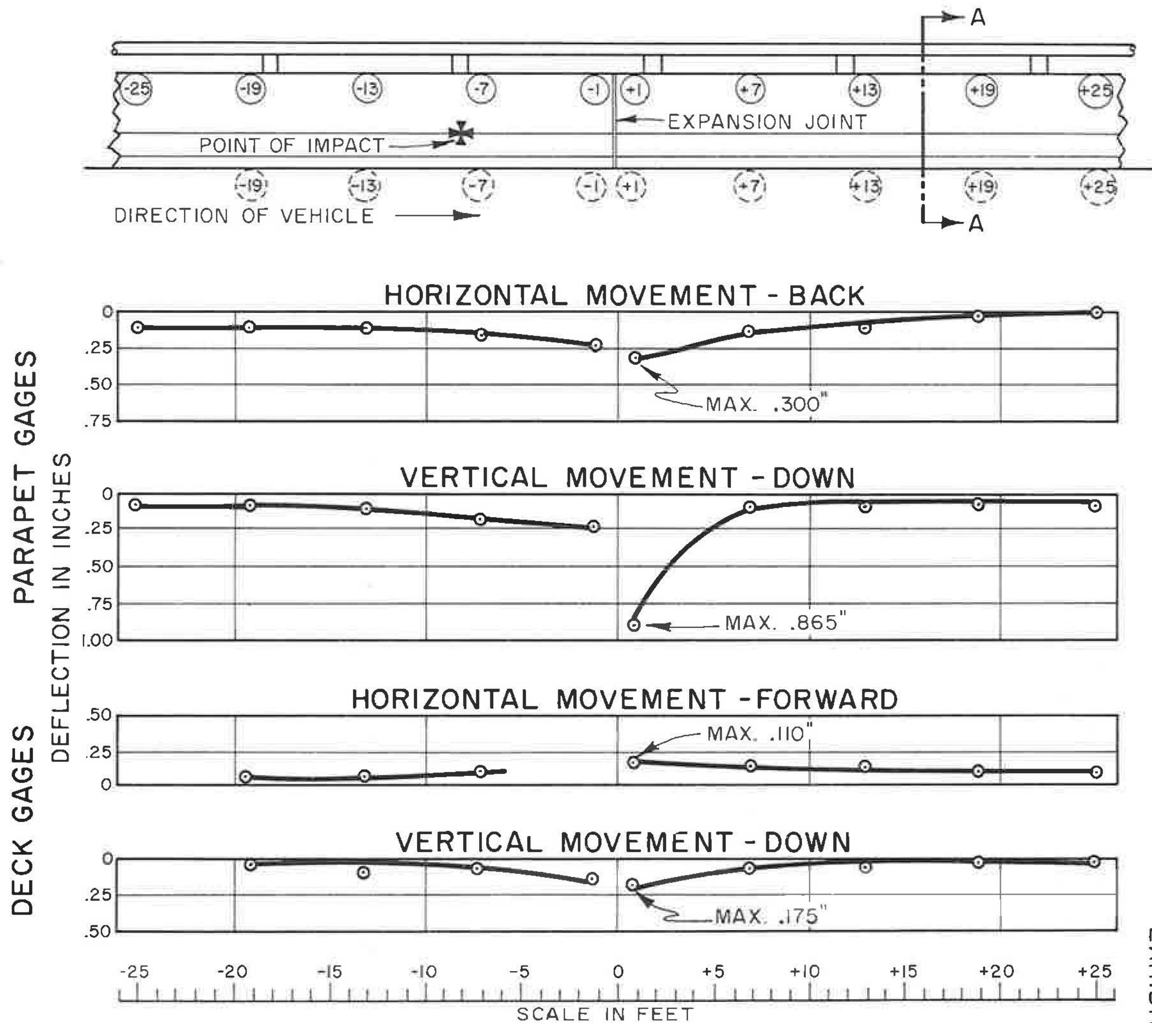
C. Deflection gage installation on Type 1 Modified Barrier Rail.



SECTION A-A

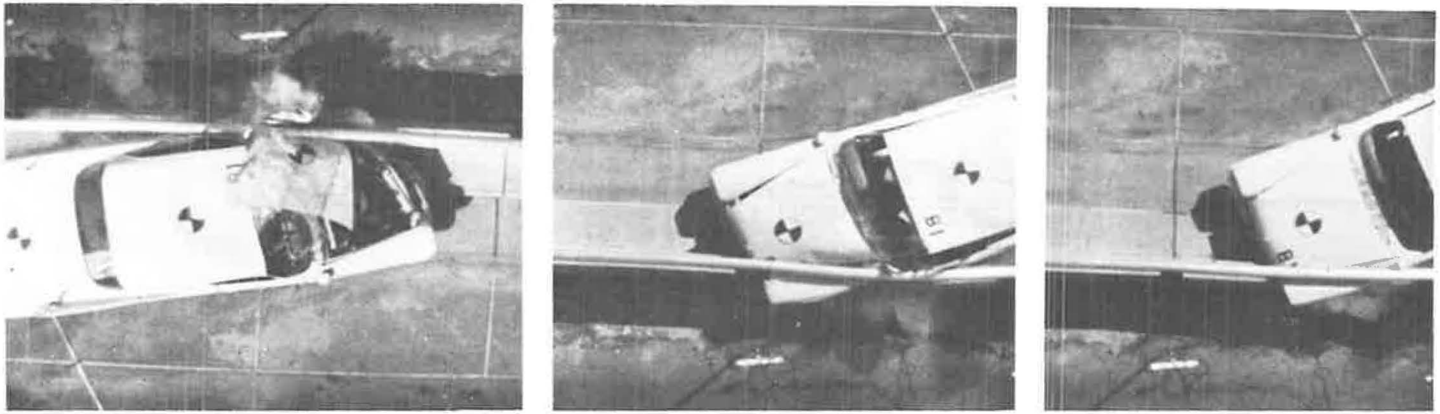


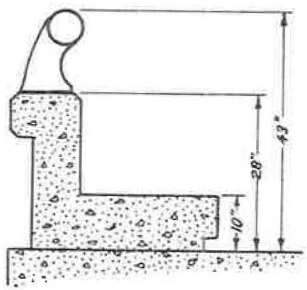
(A) PARAPET GAGE (-7)

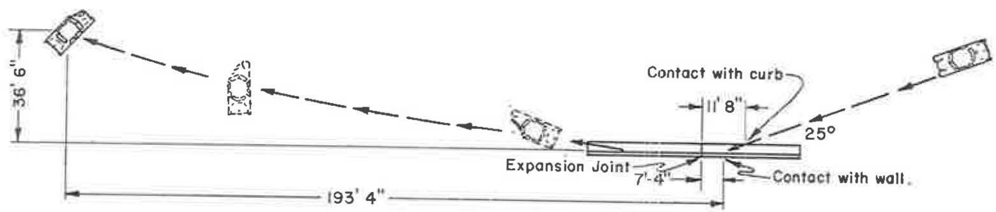


(B) DYNAMIC DEFLECTION OF PARAPET & DECK - RUN B-5

EXHIBIT 18





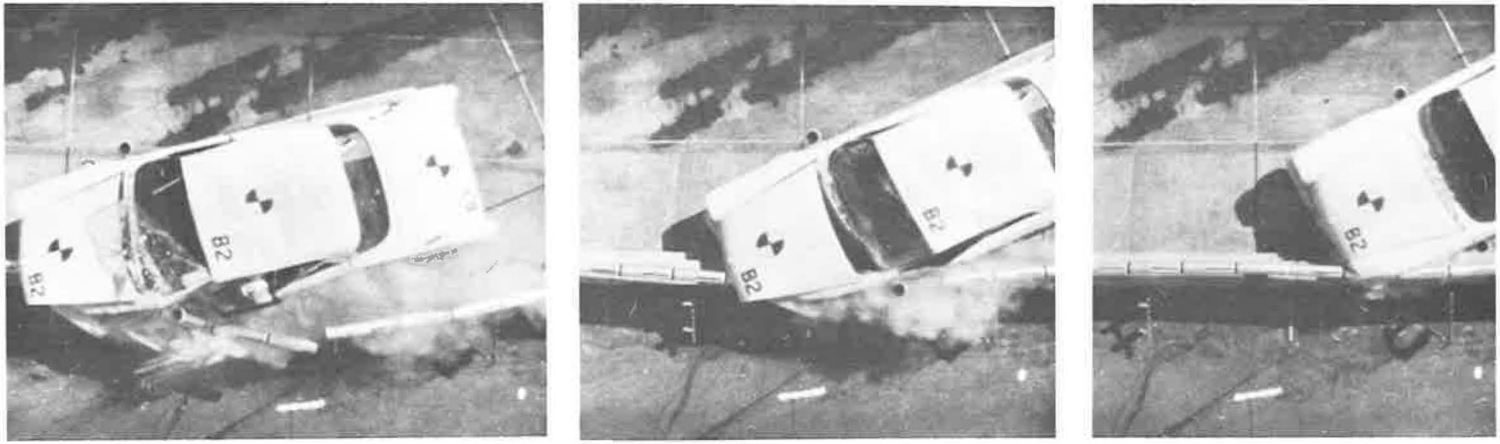


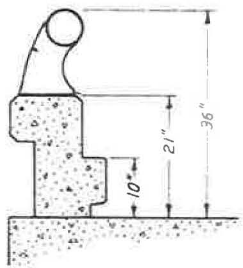
BRIDGERAIL.................. Type 2
ANCHOR BOLTS.............. Steel
POST.......................... Aluminum cast
FASTENING BOLTS.......... Aluminum
RAIL........................... Aluminum
POST SPACING.............. 10' O.C.
LENGTH OF INSTALLATION.. 62'

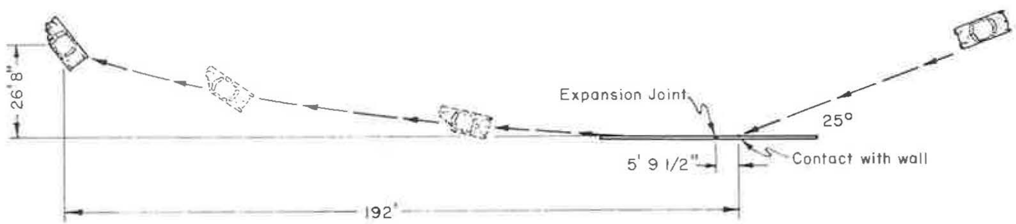
LENGTH OF CONTACT... 35'
WALL DAMAGE.......... Minor spalling and cracking where wall joins walkway.
POST DAMAGE.......... One post ripped thru entire web width adjacent to base.
PERMANENT DEFORMATION IN RAIL.. 1 1/2"

TEST NO.. .... B-1
DATE.............. 9-21-62
VEHICLE........... 1960 Dodge
SPEED............. 76 mph
IMPACT ANGLE..... 25°
VEHICLE WEIGHT.. 4300 lbs.
(W/ DUMMY & INSTRUMENTATION)

EXHIBIT 19







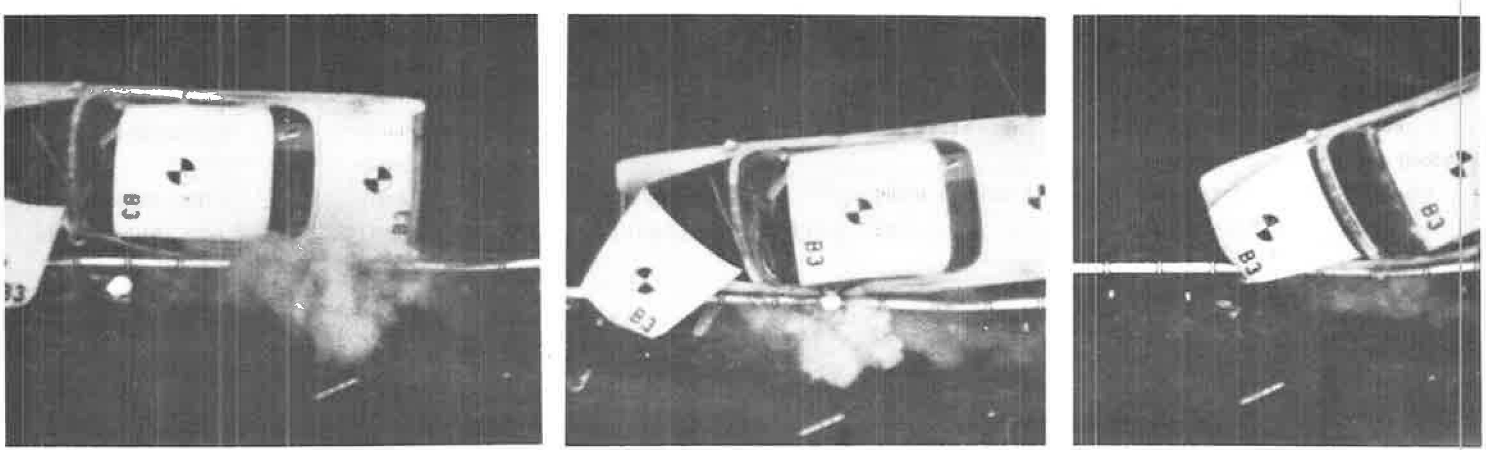
BRIDGERAIL	Type I
ANCHOR BOLTS	Steel
POST	Aluminum cast
FASTENING BOLTS	Aluminum
RAIL	Aluminum
POST SPACING	10' O.C.
LENGTH OF INSTALLATION	62'

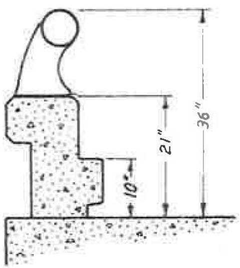
LENGTH OF CONTACT	Entire length of rail from point of impact.
WALL DAMAGE	Severe cracking adjacent to post #4. Req. replacement of 5' of concrete wall.
POST DAMAGE	Post nos. 4,5 & 6 sheared at base; 1 web slightly bent.
PERMANENT DEFORMATION IN RAIL	3 sections of rail pulled out of posts.

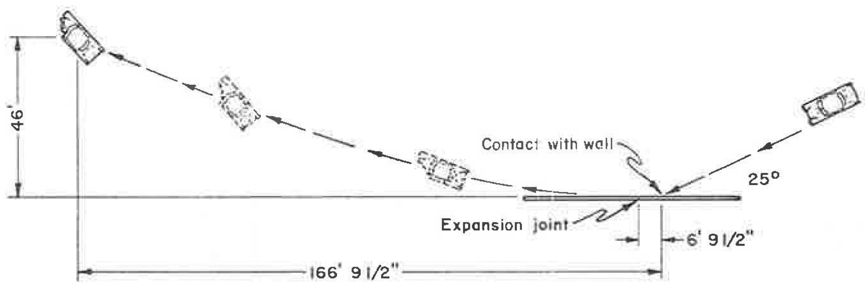
TEST NO.	B-2
DATE	9-27-62
VEHICLE	1960 Dodge
SPEED	76 mph
IMPACT ANGLE	25°
VEHICLE WEIGHT	4300 lbs.

(W/ DUMMY & INSTRUMENTATION)

EXHIBIT 20





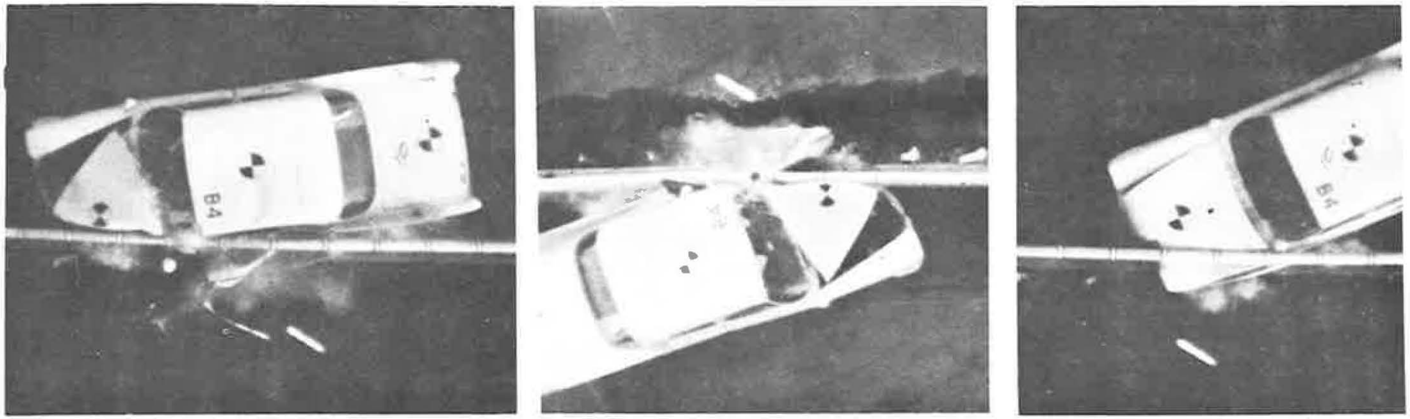


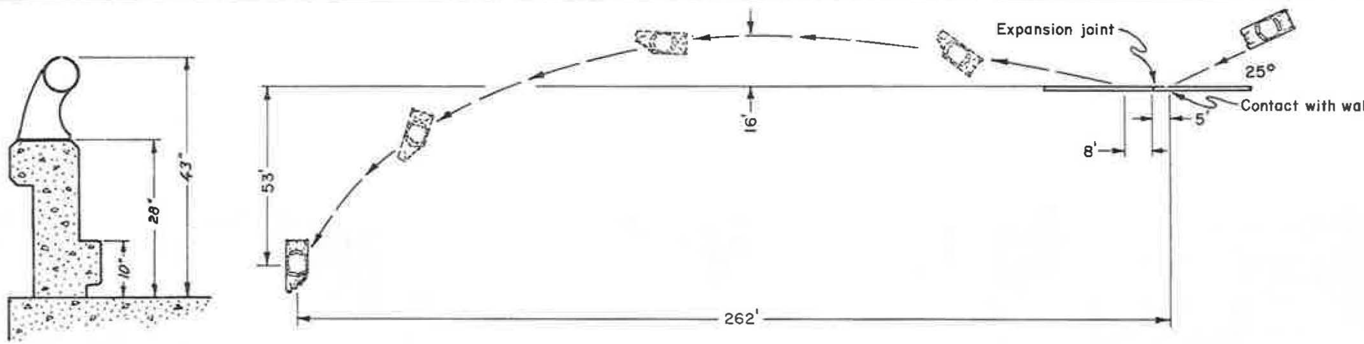
BRIDGERAIL..................Type I
ANCHOR BOLTS.............Steel
POST.......................Steel plate (Welded)
FASTENING BOLTS..........Steel
RAIL.........................Steel
POST SPACING.............10' O.C.
LENGTH OF INSTALLATION..62'

LENGTH OF CONTACT...26'
WALL DAMAGE..........Deep spalling & cracking at post #4. Reg. epoxy/agg. patch & replacement of post anchor bolts.
POST DAMAGE..........No visible damage.
PERMANENT DEFORMATION IN RAIL....3"

TEST NO...........B-3
DATE.............1-10-63
VEHICLE..........1960 Dodge
SPEED............73 mph
IMPACT ANGLE.....25°
VEHICLE WEIGHT..4300 lbs.
(W/ DUMMY & INSTRUMENTATION)

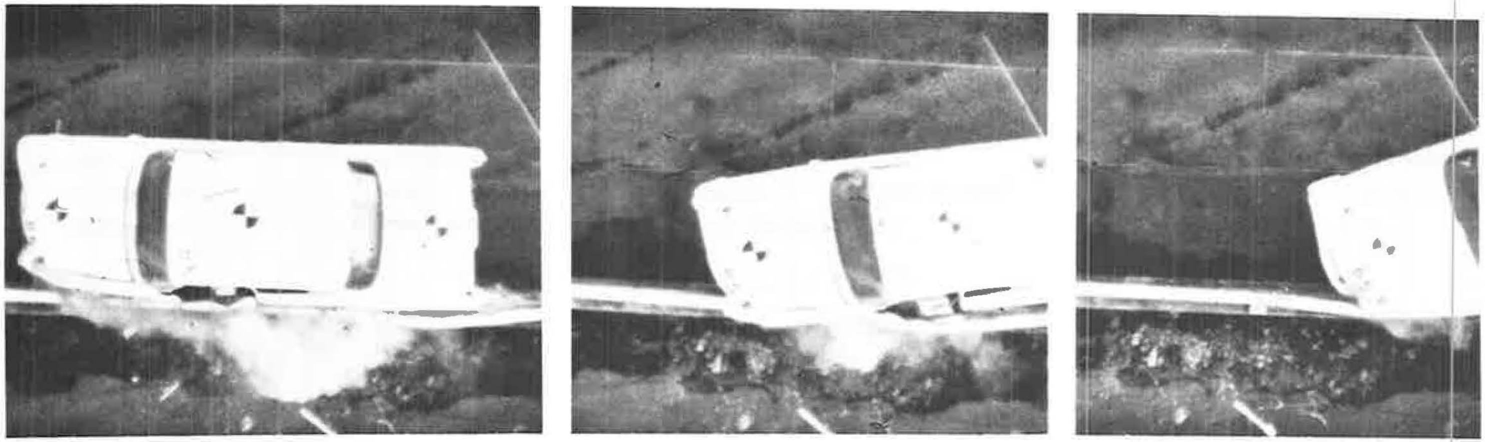
EXHIBIT 21

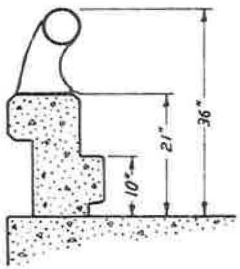


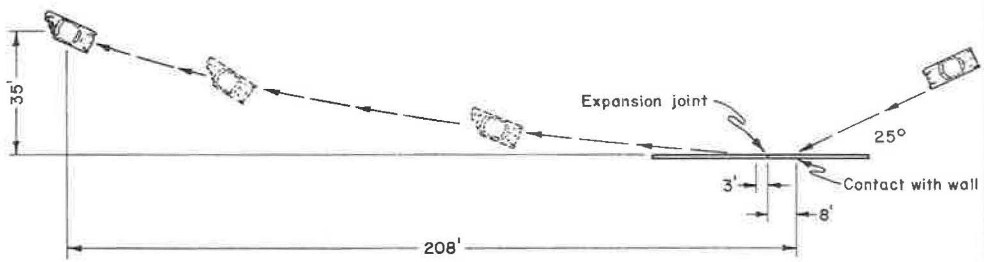


BRIDGERAIL	Type 1, modified	LENGTH OF CONTACT	20'	TEST NO.	B-4
ANCHOR BOLTS	Steel	WALL DAMAGE	Slight spalling.	DATE	1-24-63
POST	Aluminum cast			VEHICLE	1960 Dodge
FASTENING BOLTS	Aluminum	POST DAMAGE	One flange bent.	SPEED	77 mph
RAIL	Aluminum			IMPACT ANGLE	25°
POST SPACING	10' O.C.	PERMANENT DEFORMATION IN RAIL	1"	VEHICLE WEIGHT	4300 lbs. (W/ DUMMY & INSTRUMENTATION)
LENGTH OF INSTALLATION	62'				

EXHIBIT 22







BRIDGERAIL.................. Type 1
ANCHOR BOLTS.............. Steel
POST........................ Malleable cast iron
FASTENING BOLTS.......... Steel
RAIL......................... Aluminum
POST SPACING.............. 10' O.C.
LENGTH OF INSTALLATION.. 62'

LENGTH OF CONTACT... 19'
WALL DAMAGE......... Cracked around #4 post anchor bolts.
POST DAMAGE.......... One post web slightly dented.
PERMANENT DEFORMATION IN RAIL.. 5 1/2"

TEST NO........... B-5
DATE.............. 2-7-63
VEHICLE........... 1959 Dodge
SPEED............. 78 mph
IMPACT ANGLE..... 25°
VEHICLE WEIGHT.. 4300 lbs.
(W/ DUMMY & INSTRUMENTATION)

EXHIBIT 23

A Bridge Parapet Designed for Safety

General Motors Proving Ground

Circular Test Track Project

LOUIS C. LUNDSTROM, PAUL C. SKEELS, BLAINE R. ENGLUND, and ROBERT A. ROGERS

Respectively, Director, Engineer-In-Charge of Engineering Services, Assistant Plant Engineer, and Senior Project Engineer, General Motors Proving Ground, Milford, Michigan

This paper describes the design and testing of an improved bridge parapet subsequently incorporated in two bridge structures at the General Motors Proving Ground. These parapets are designed to provide maximum safety to the occupants of vehicles which might strike them. Testing involved running full-scale remotely controlled vehicles into test sections constructed especially for this purpose. These vehicles included passenger cars of various sizes and a loaded $2\frac{1}{2}$-ton truck. Performance of the final design is satisfactory in all respects and costs are not excessive.

•THE GENERAL Motors Proving Ground has recently constructed a new circular test track which includes two long-span bridges as part of the access road system. Every effort was put forth to make the entire facility as safe as possible and to incorporate every proven safety feature.

Our full-scale impact test program on highway guardrails (1) showed that most conventionally used bridge rails, or parapets, left much to be desired from a safety standpoint. There are many designs of bridge parapets currently in use but few were actually tested before construction. Some of them, even a bridge rail constructed at the Proving Ground in 1953, appear to be primarily decorative in nature (Fig. 1); some are designed with high strength but produce extensive vehicle damage (Fig. 2); and some are obviously too low to prevent a vehicle going over the top (Fig. 3). Frequent news items show that existing designs often do not retain the vehicle within the roadway (Figs. 4 and 5).

Because of these problems, a group of engineers were assigned to design and test an improved parapet for our new bridges. The following requirements were established for the bridge parapet:

1. It should be virtually impenetrable by any motor vehicle at the 50-mph speed at which the roadway was to be driven and at the relatively low angle of impact attainable within the 28-ft roadway width.
2. It should be designed to minimize longitudinal and lateral decelerations to vehicles impacting it and to their occupants.
3. It should minimize damage to a striking vehicle.
4. It should allow good visibility over the parapet, so passengers in cars of any size could see both above and below the horizon.
5. It should present a pleasing appearance.
6. It should be economical to construct and maintain.
7. The guardrail protecting the approaches should be "blended" and fastened to the ends of the bridge parapet in such a way that a vehicle would not be unreasonably endangered regardless of where it struck the guardrail-parapet system.

Paper sponsored by Committee on Guardrails and Guide Posts and Committee on Bridge Design, and presented at the 44th Annual Meeting.

Figure 1. Bridge rail design used on existing Proving Ground overpass constructed in 1953.

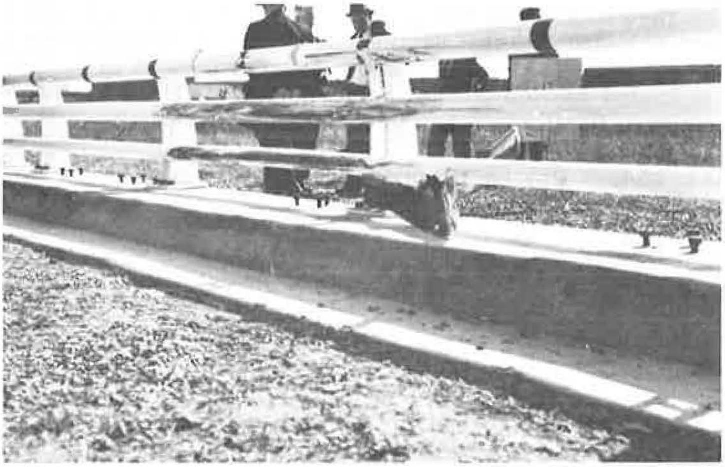


Figure 2. High-strength bridge rail which inflicted severe damage while redirecting impacting vehicle.

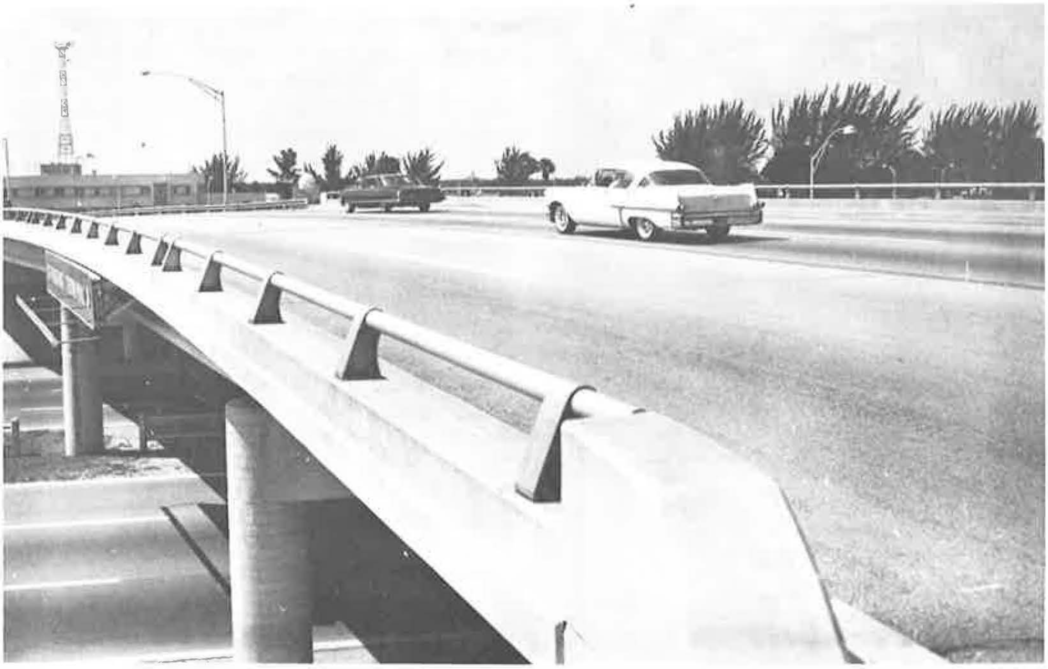


Figure 3. Bridge rail design low enough to permit some impacting vehicles to continue over top.

Figure 4. Bridge rail design lacking sufficient structural strength to retain impacting vehicle on roadway.

Figure 5. Low concrete bridge rail design with top rail lacking sufficient strength to retain impacting vehicle.

Since these bridges were not intended to handle pedestrian traffic, there was no need to consider a sidewalk. In many cases, the sidewalk is placed between the parapet and the road; the resulting curbing provides an unnecessary hazard for the out-of-control vehicle. If a crossing for pedestrians had been desired, the sidewalk would have been placed outboard of the parapet.

Many designs of existing bridge rails, curbs, and highway medians were considered, and several test sections were built and crash tested. The test sections consisted of a 5-in. diameter pipe curb and two sloped face concrete curbs. Figure 6 shows cross-sections of the designs tested. The 65° sloped face redirected the low-angle impacting vehicle more violently than did the 55° design. The pipe curb did not demonstrate any advantage. Some wheel and/or sheet metal damage was experienced with the 65° design. The design with the 55° angled surface was most effective in gently turning the vehicle under low-angle impacts with no sheet metal damage. However, at higher impact angles, the vehicle tended to climb the wall.

During the investigation of existing guardrails and highway median barriers, the designs used extensively in New Jersey had been brought to our attention. One of these is a concave cross-section concrete wall about 23 in. high. Another of a later design had a double sloped cross-section 32 in. high (Fig. 7). Reports from New Jersey indicated that this design performed well. It very successfully prevented cross-median accidents, and at the same time inflicted only minor damage to impacting vehicles (2). This general type of barrier appeared to have considerable merit in satisfying our ground rules; hence, the Proving Ground embarked on a program to develop this barrier into a bridge parapet. Twenty-one tests were run on various configurations before arriving at the final design.

Because of the requirement for good visibility, the height of the concrete portion had to be less than the eye height of a person driving a low vehicle. Drivers' eye heights on twenty-seven 1962 cars ranged from 44 to 48.8 in. above the road surface. All but three of the eye heights were between 45 and 48.8, with eight of them between 47 and 47.5 in. Based on these data, it appeared that a height of 32 in. for the concrete portion would provide adequate visibility (Figs. 8 and 9). Tests conducted during the development program indicated that this height was sufficient to prevent cars from climbing over the wall.

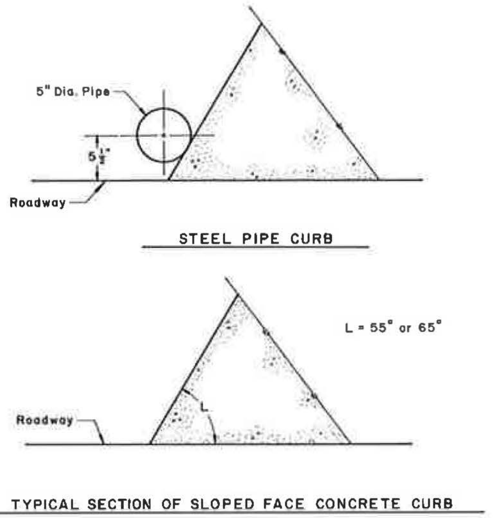


Figure 6. Cross-sectional views of curb designs tested for redirection of low speed and low angle impacts without sheet metal damage.

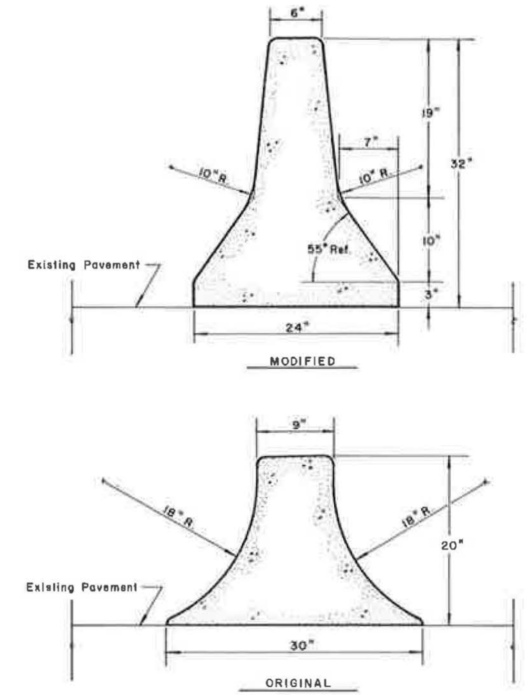


Figure 7. Typical section of New Jersey concrete barrier curb.

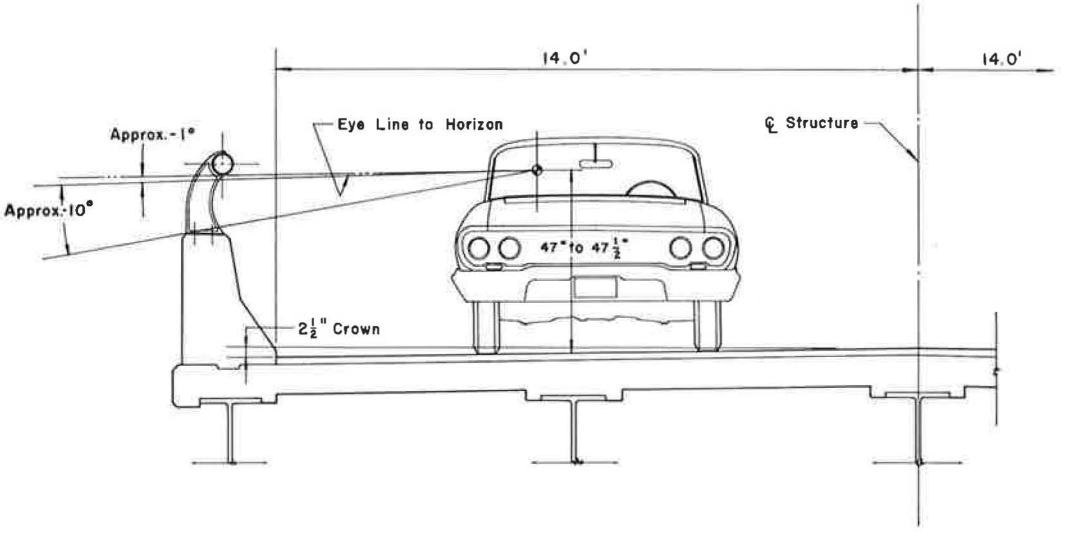


Figure 8. Typical cross-section of Proving Ground overpass.

Figure 9. Driver's eye view.

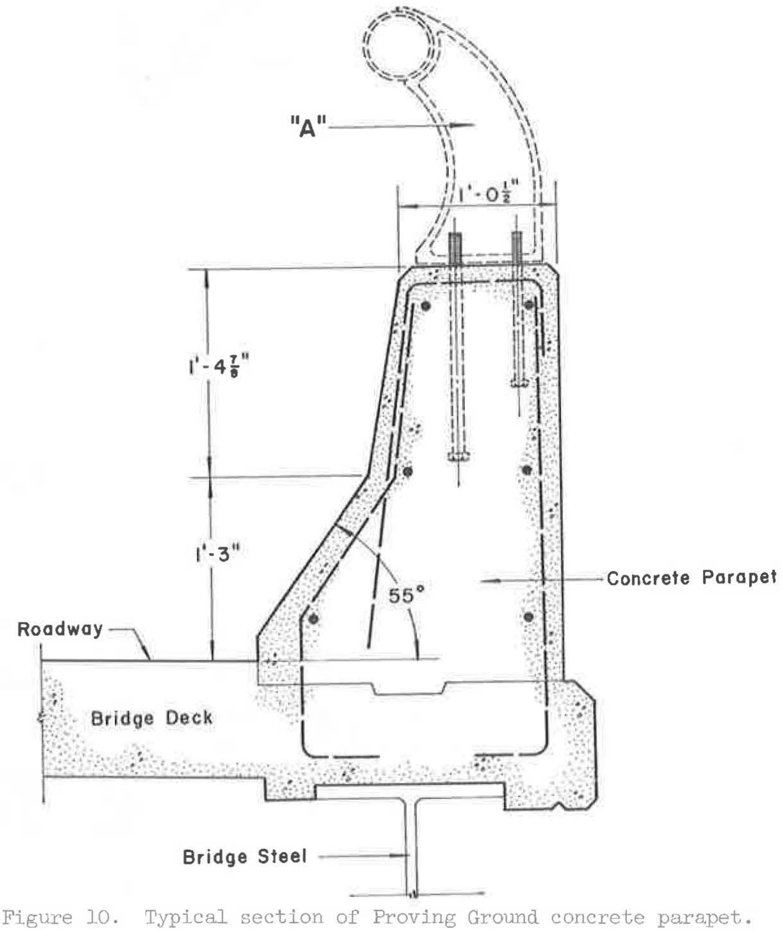


Figure 10. Typical section of Proving Ground concrete parapet.

Previous experience with guardrail testing had made us aware of the advantages to both car and passengers in minimizing lateral decelerations during impacts. Guardrails can be made to do this by utilizing the natural flexibility of the rail and posts, and adding additional flexibility by spring-mounting the rail to the posts. It did not seem practical to utilize this same principle on the bridge parapet because the requirement for impenetrability by all sizes of vehicles called for a lateral strength that could not be obtained with conventional guardrail materials.

Figure 11. Placement of junction of upper and lower angled faces chosen to permit low-speed, low-angle impacts without sheet metal damage.

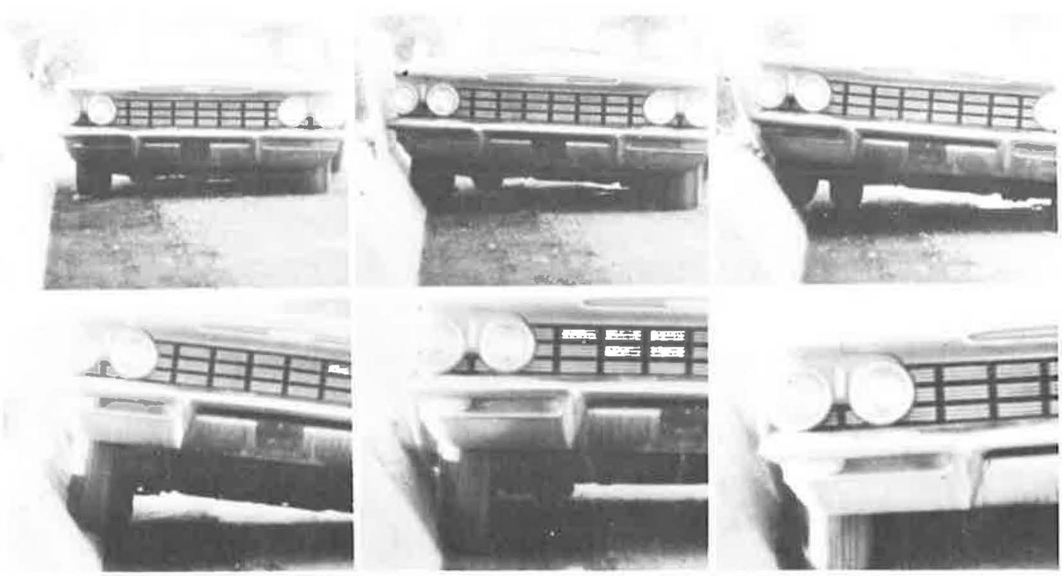


Figure 12. Hand driven evaluations of 55° sloped face. Vehicle "rides up" as it is redirected. Plywood mock-up above concrete was used during dynamic tests to evaluate other design dimensions.

As indicated earlier, the 55° angled face permitted low-angle impacts without sheet metal damage but involved a climbing tendency when impacted at higher speeds and angles. Careful measurements were made on vehicles to determine the proper elevation of the junction of the two angled faces of the parapet in order to eliminate sheet metal contact with the upper portion when the wheel contacted the base. After a series of plywood mock-ups were constructed and tested, an elevation of 15 in. was chosen.

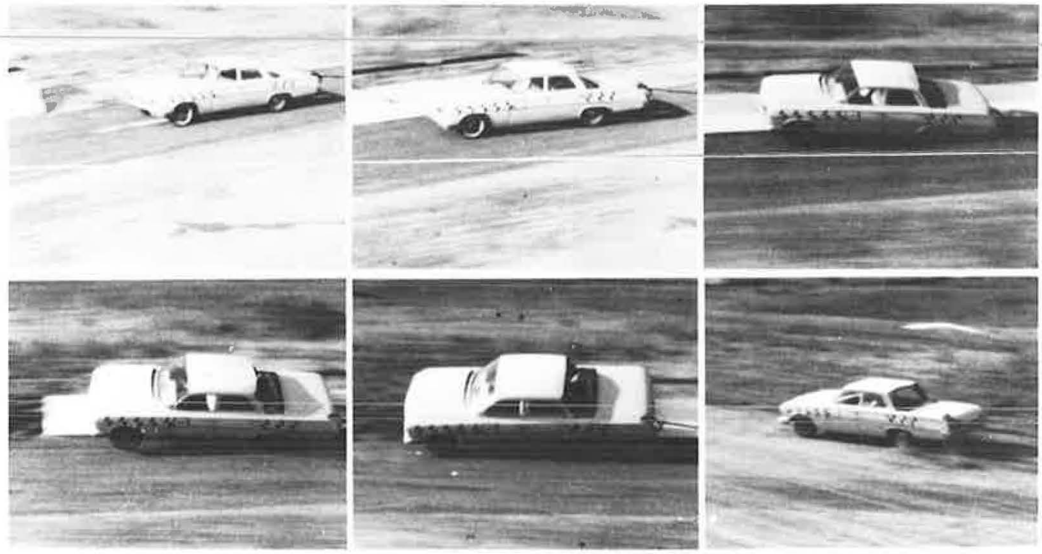


Figure 13. Dynamic test photo sequence where vehicle was remotely driven into finished design test section of bridge rail at 50 mph and 12°.

Figure 14. View of test section of bridge rail showing scrub marks from repeated testing; rail at top not design used in final system.

Figure 15. Remotely driven car redirected by rail; impact conditions, 50 mph at 12° angle.

Figure 16. Position of test car after impacting bridge rail test section at 50 mph and angle of 12°.

A short section of a proposed design was constructed using a near-vertical upper face similar to the New Jersey median (Fig. 10). The relationship between the final parapet design and a car is shown in Figure 11.

The Proving Ground staff tested this design with extreme thoroughness. Since this parapet was to be installed on a bridge located on access roads with a 50-mph speed limit, tests were confined to a maximum speed of 50 mph. The initial tests evaluated its performance under glancing low-angle and low-speed impacts up to 10° and 30 mph. Under most such conditions, the tire would ride up the lower portion of the barrier rather readily, but when the steeper section was encountered, the car was banked and the front wheel forced into a turn to deflect the vehicle back toward the roadway (Fig. 12). Several remotely controlled high-speed impacts were made at speeds up to 50 mph and angles to 12°, and in no case did the car climb to the top of the barrier or show any tendency to climb over it (Fig. 13). In most runs, the damage to the car consisted only of front bumper and fender rubbing and sometimes an upsetting of front wheel alignment. Figure 14 shows a test section (not the final design) with various scrub marks. These were not the final brackets. The test section has been repeatedly struck at 50 mph at an 8° angle by manually driven cars with no vehicle damage and no driver concern.

Figure 15 shows a remotely driven car being turned by the barrier after striking it at 50 mph at a 12° angle. This angle was chosen as representative of the maximum angle attainable within this 28-ft roadway width by a vehicle traveling at this speed. Figure 16 shows that the car was turned nearly parallel to the rail at the conclusion of this test. The car was guided by electrical remote control and stopped by remote application of its own brakes. Figure 17 shows the car after the test. This car had been used in previous tests and the headlight was missing before the impact. The right front fender was damaged when it hit the temporary rail support bracket, but the car was driven away from the test site under its own power. Lateral decelerations to the simulated human occupant during this run did not exceed 3 g. The fact that the car climbs the wall and tends to bank reduces lateral decelerations on the occupants much as going around any banked turn does.

Experiments were run to determine whether a reduction in coefficient of friction of the sloped surface would affect the performance in any way. The surface was first ground smooth, then greased, but performance was not changed. It was learned that

Figure 17. Damage to test car as result of 50 mph impact with bridge rail at angle of 12° (headlights removed before test).

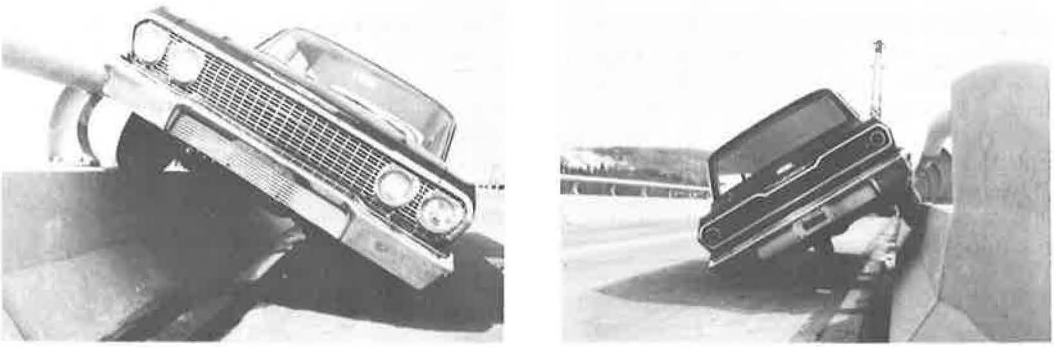


Figure 18. Placement of right front wheel of test vehicle on concrete parapet to show proximity of sheet metal to rail and wheel to post.

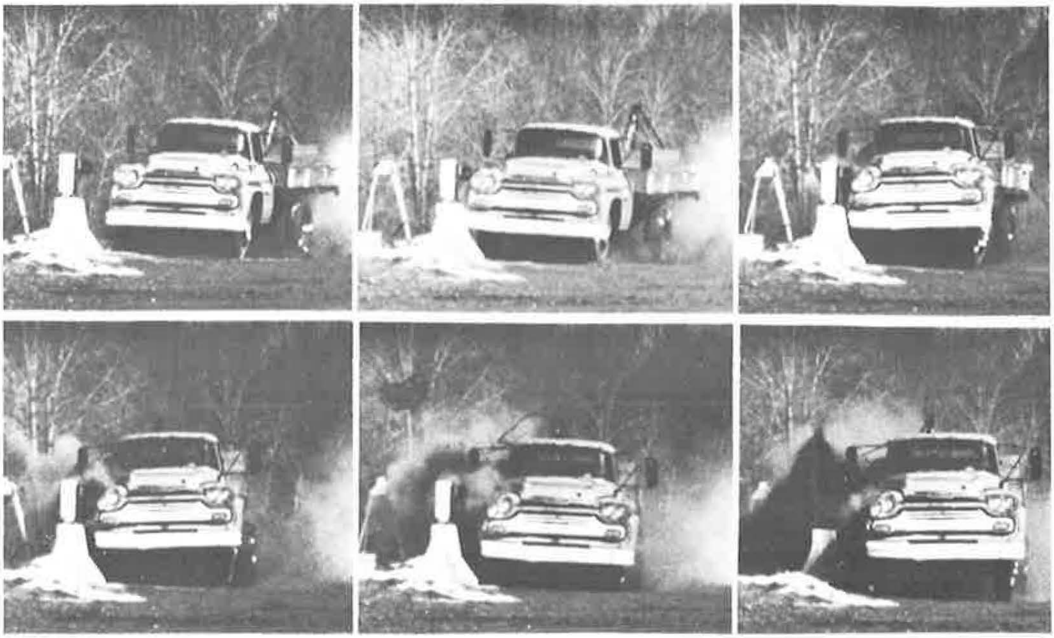


Figure 19. Photo sequence of $2\frac{1}{2}$ ton 16,000-lb GVW truck impacting bridge rail at 37 mph and 13°; top rail in this sequence in experimental position.

the height to which a car climbs the parapet is determined primarily by the lateral inertia force of the vehicle and the lateral friction force developed by the outside wheels. Tests indicated that the friction force of the wheel in contact with the parapet did not influence the action of the impacting vehicle as it was redirected by the rail.

It was fully realized that the 32-in. height of the concrete wall was not sufficient to guarantee that larger trucks would be safe. Accordingly, a pipe rail was installed on top to provide a higher barrier and to increase the height of the fulcrum over which a high-center-of-gravity vehicle would have to roll. For a rough approximation, the height of the rail should approach the height of the center of gravity of any vehicle using the bridge. Top rails are used on many bridges; however, most of these are made of relatively light-walled steel or aluminum tubing mounted on cast aluminum brackets.

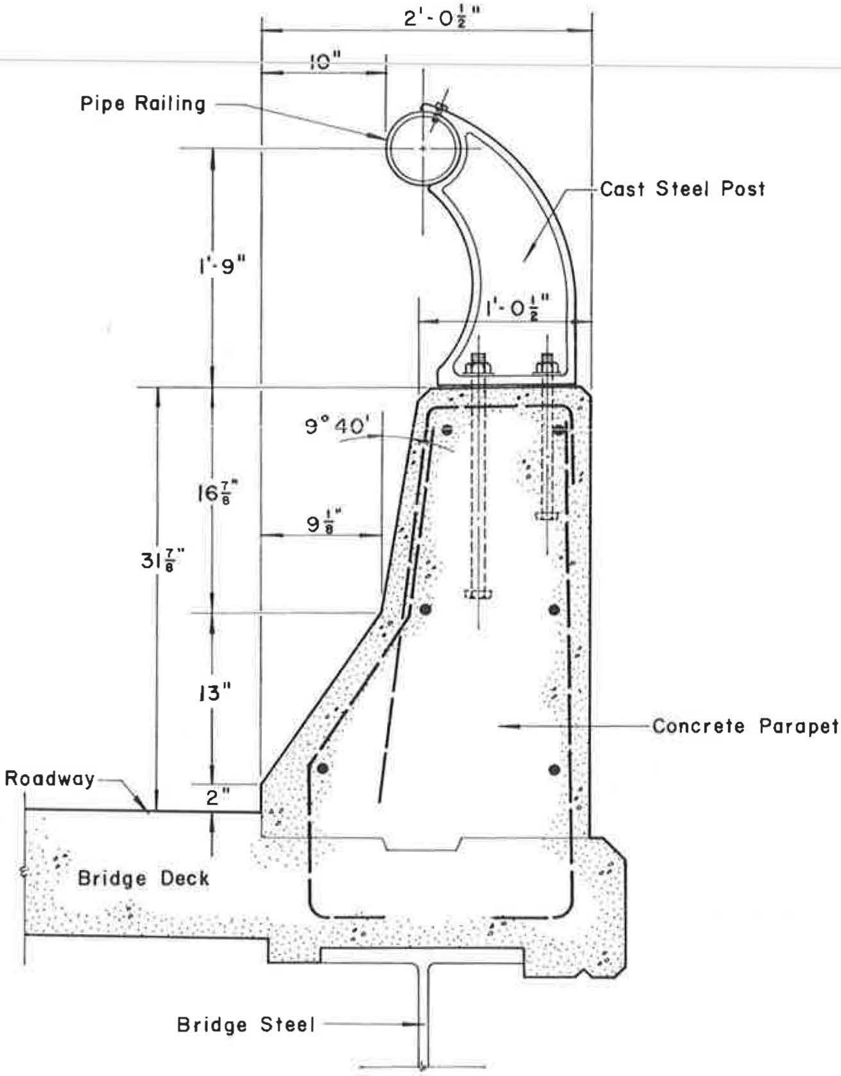


Figure 20. Typical section of Proving Ground bridge parapet.

The U. S. Bureau of Public Roads has issued a proposed recommendation for strength requirements for bridge rails (3). These indicate that the rail should be able to resist a lateral force of 15,000 pounds at any point along its length. Commercially available rails and brackets were tested, and none even came near meeting this requirement, so we designed a bracket that would give the required strength and still present a good appearance.

The same specifications require the rail on a parapet of this general configuration to withstand a transverse load of 15,000 lb (P/2, where P = 30,000 lb). It is further specified that the brackets be designed for a transverse load of 0.8 P/2 and a simultaneous longitudinal load of 0.4 P/2. This means that our brackets must withstand a transverse load of 12,000 lb and a simultaneous longitudinal load of 6,000 lb. The brackets as actually fabricated were tested at a transverse load of 22,000 lb without failure, indicating a substantial margin of safety over the BPR recommendations. Bracket spacing is determined primarily by the requirement that the pipe rail withstand

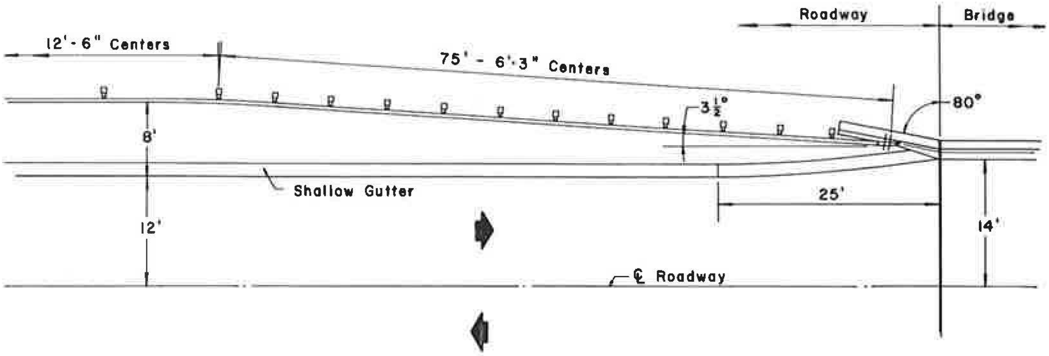


Figure 21. Typical plan of bridge approach.

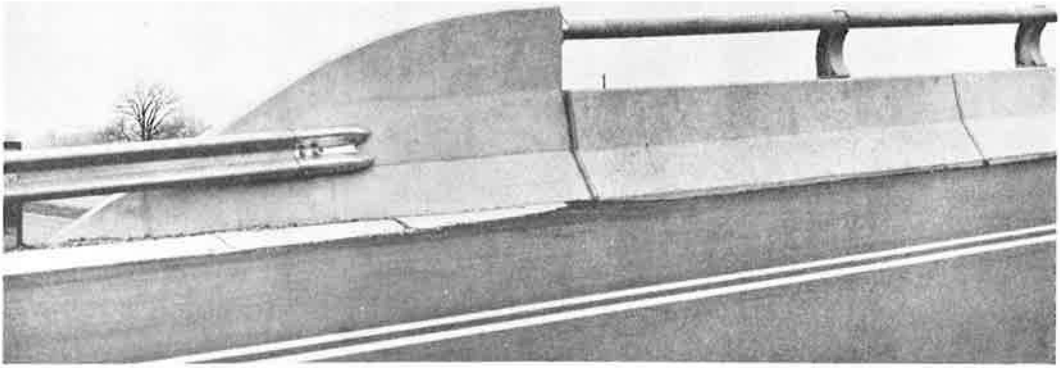


Figure 22. View of bridge rail end wall and blending of adjacent guardrail.

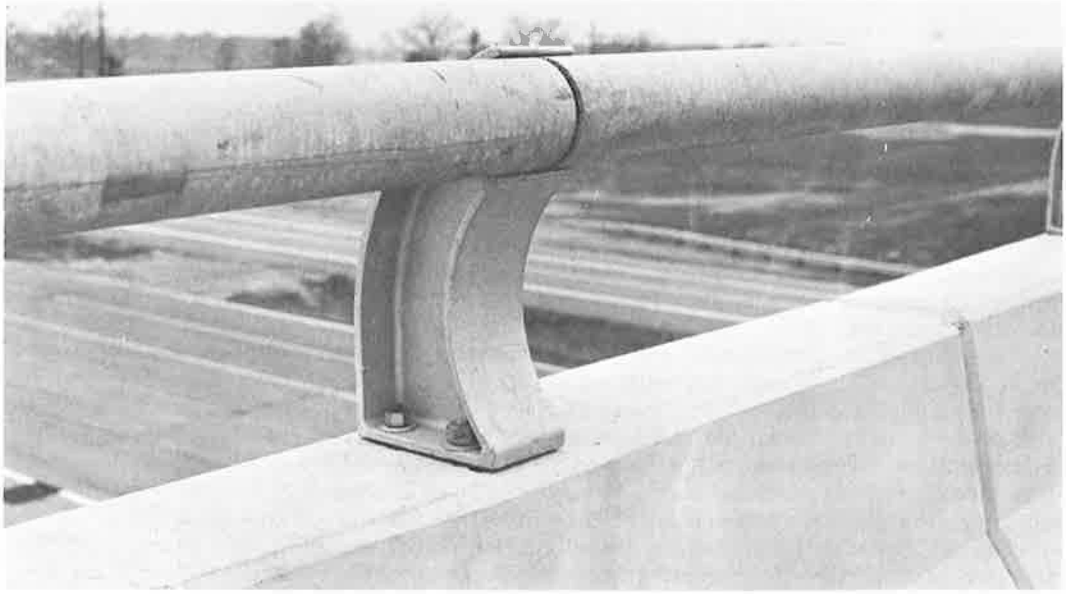


Figure 23. Detail of top rail construction on finished bridge.

Figure 24. Overall view of completed bridge structure.

Figure 25. Overall side view of completed bridge span.

15,000 lb applied at the midpoint between two brackets. Utilizing the design method in Section II, Article 1.2.11 (E) in the BPR specifications, we used an 8-ft nominal spacing between brackets and 5-in. extra heavy galvanized steel pipe. This design exceeds the BPR requirements. The brackets are designed of dip-galvanized cast steel, with an open clamp toward the road to provide a smooth "rub rail" for a vehicle striking the pipe. Provisions are made for rail expansion. Brackets are mounted to the parapet with two 1- by 18-in. and two $\frac{3}{4}$- by 12-in. high-tensile bolts.

The rail installed on the mock-up in Figure 14 is held in place with conventional aluminum brackets because the special brackets were not available during this phase of the testing period.

To establish the optimum lateral location of the rail, vehicles of all types were driven so that their wheels rubbed the parapet. The lateral extremities of the vehicles were measured and the rail positioned so that no vehicle would rub it until the body had rolled appreciably toward the rail. Tests determined that the rail did not contact sheet metal of cars. The inside of the cast-steel post (see A in Figure 10) was made concave to minimize the chance of contact with any portion of a wheel that might climb the concrete section. Under the most severe condition, a wheel might be rolling along the top of the concrete with the side of the wheel rubbing on the rail (Fig. 18); even in this extreme condition, the wheel would climb over the base of the steel posts without undue damage to either the post or the wheel.

Following the installation of the pipe rail at the established point, a full-scale test was run during which a $2\frac{1}{2}$-ton loaded stake truck was remotely driven into the test section at a speed of 37 mph and an angle of 13°. The performance of the section was entirely satisfactory in all respects (Fig. 19).

Figure 20 shows the complete cross-section of the bridge parapet finally used. The height of the parapet was dictated primarily by performance considerations. The height of the rail was dictated by performance considerations and visibility (Fig. 8). The lateral positioning of the rail was established by the need to have the sheet metal of a vehicle contact the rail at the same time the front wheel reached the top of the concrete parapet.

With the parapet itself designed and tested, we next devised an end wall that would protect the end of the parapet and rail from an end-on hit by an out-of-control vehicle approaching the bridge. This design is best shown in Figures 21 and 22. The wall slopes backward and downward and is overlapped by the approach guardrail, so that even if a car's wheels have climbed the guardrail and the vehicle is sliding along it, the wheels will encounter a sloping surface when they strike the parapet end section. The approach guardrail uses 6-ft 3-in. post spacing for the last 75 ft of rail before the parapet, and the rail is securely bolted to the parapet to develop maximum strength. This doubling of the number of posts supporting the approach guardrail greatly improved the performance of the conventional guardrail at this critical point. If the parapet was to be exposed to traffic speeds of 65 mph or higher, closer post spacing would be recommended, both adjacent to the parapet and in the main guardrail on the approaches. Figures 23, 24 and 25 show details of the completed bridges. Working drawings for the parapets are included in the Appendix.

SUMMARY

This bridge parapet satisfied our requirements in all respects. It costs approximately 20 percent more than conventional parapets but is considerably safer. Much of the increased cost is primarily due to the greater strength built into the pipe railing and supporting brackets. The concrete parapet itself should be no more expensive to construct than many conventional designs, and its superior performance has been proven by an adequate number of full-scale tests.

ACKNOWLEDGMENTS

We wish to thank the many individuals who participated in this project and contributed their work and ideas. This includes members of the Proving Ground Experimental Engineering, Plant Engineering, and Photographic Groups as well as several people from various state highway departments whose suggestions and encouragement were of great benefit.

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2. Brochure on Safety Construction 1954 to Aug. 31, 1960, New Jersey State Highway Dept., Bureau of Public Information.
3. Proposed Specifications for Bridge Railings. U. S. Bureau of Public Roads, Washington, D. C., April 1962.

Appendix

WORKING DRAWINGS FOR PARAPETS

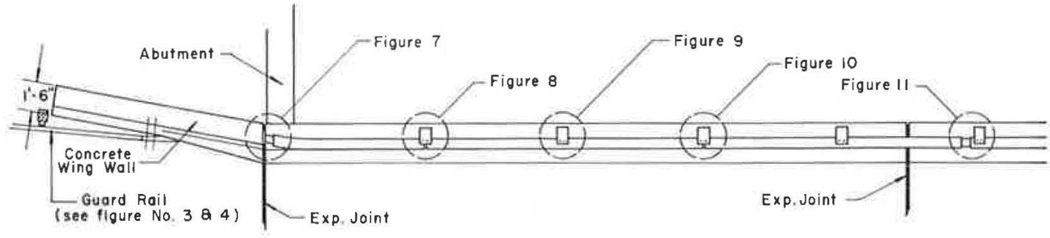

Figure 1A. Typical plan.

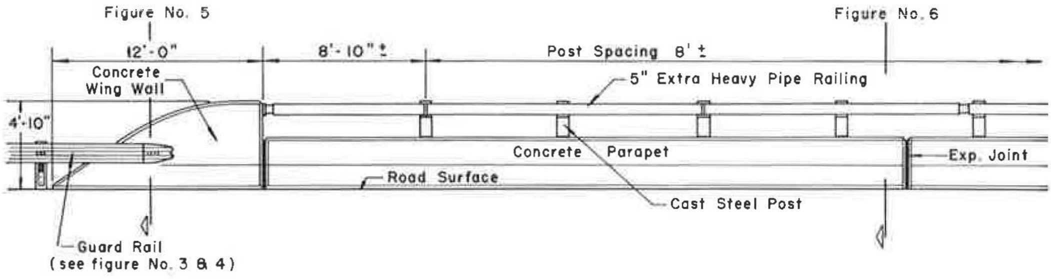

Figure 2A. Typical elevation.

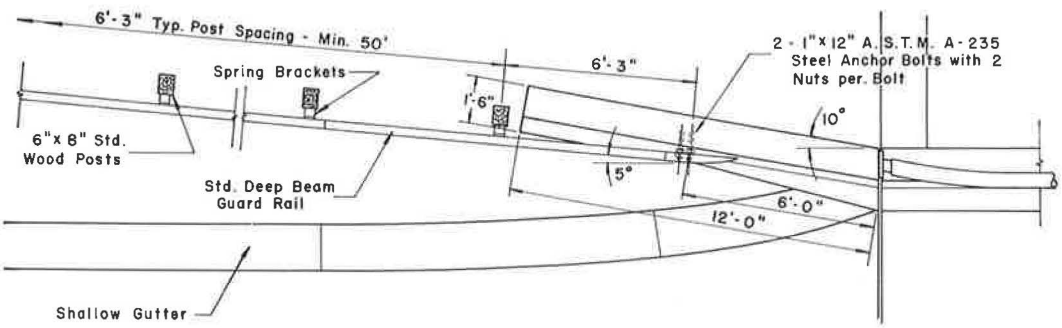

Figure 3A. Plan.

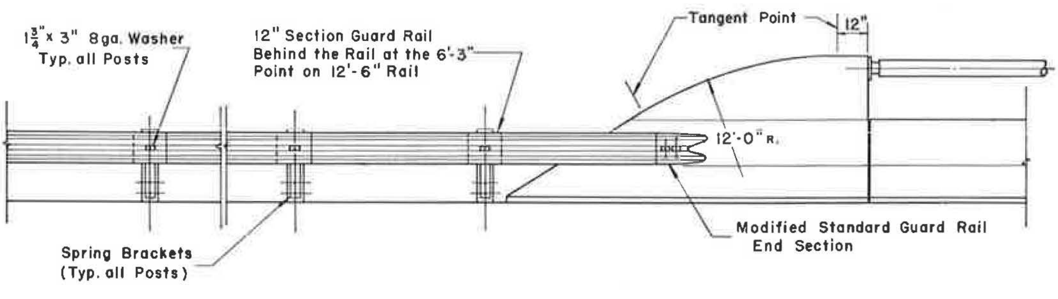

Figure 4A. Elevation.

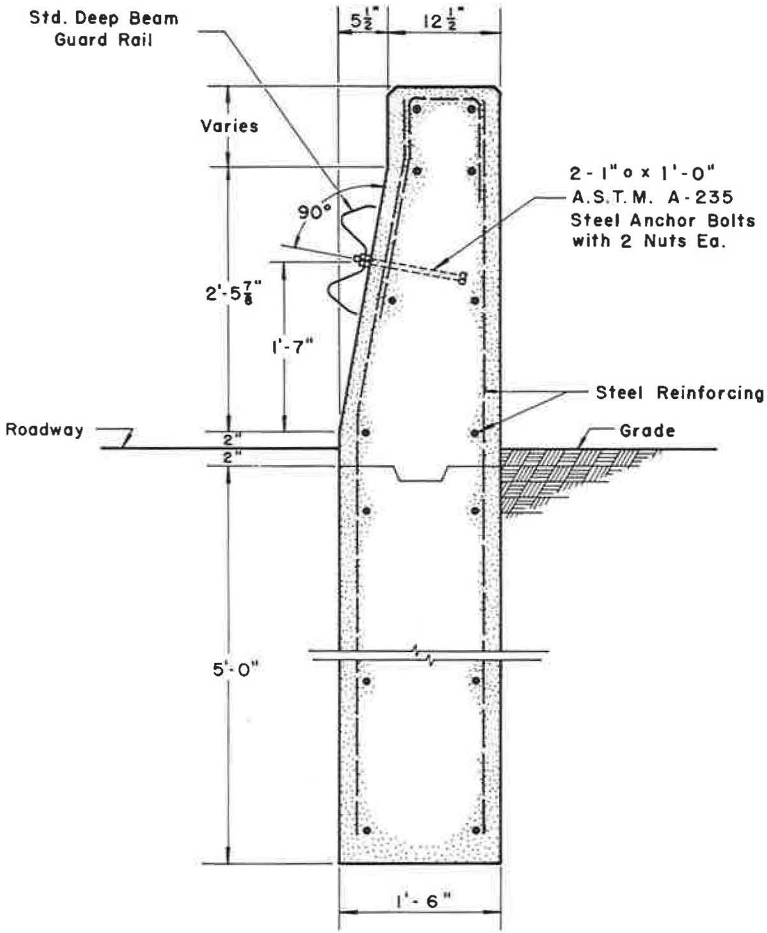

Figure 5A. Section at guardrail connection.

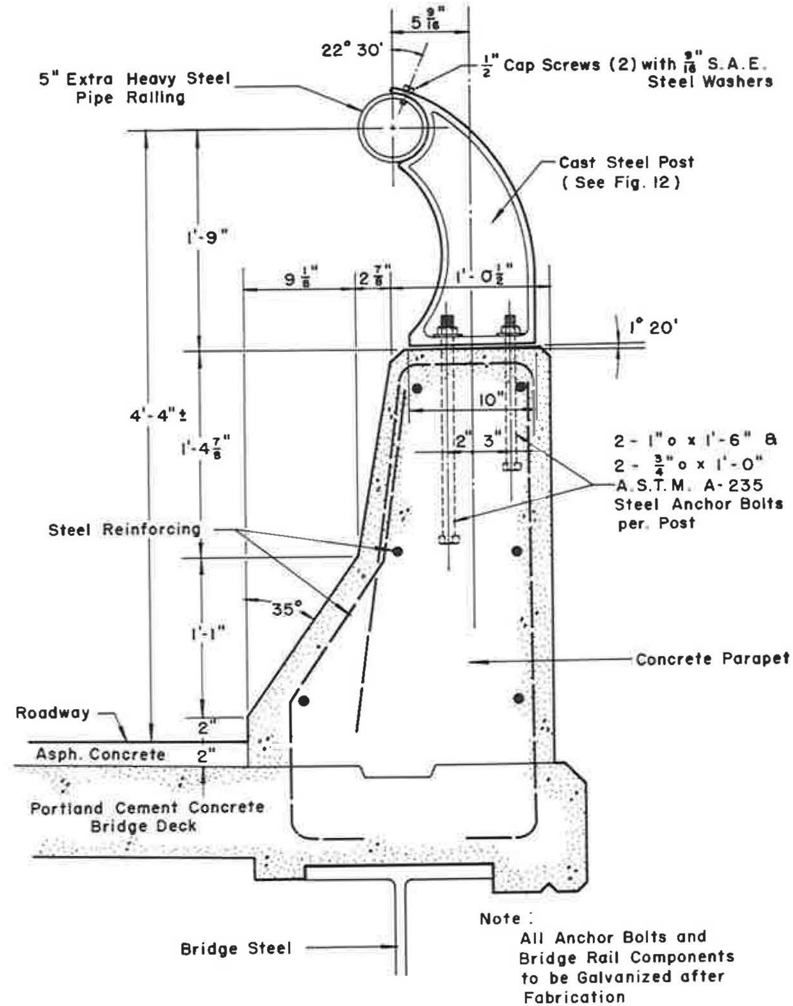

Figure 6A. Typical cross-section.

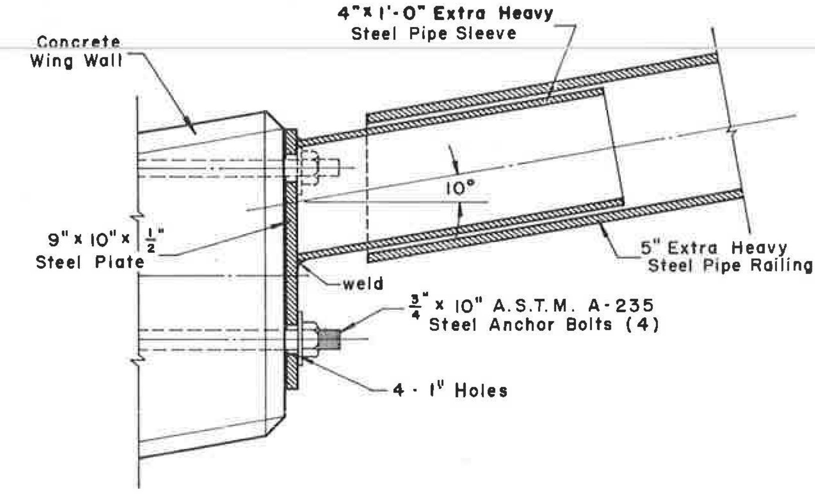

Figure 7A.

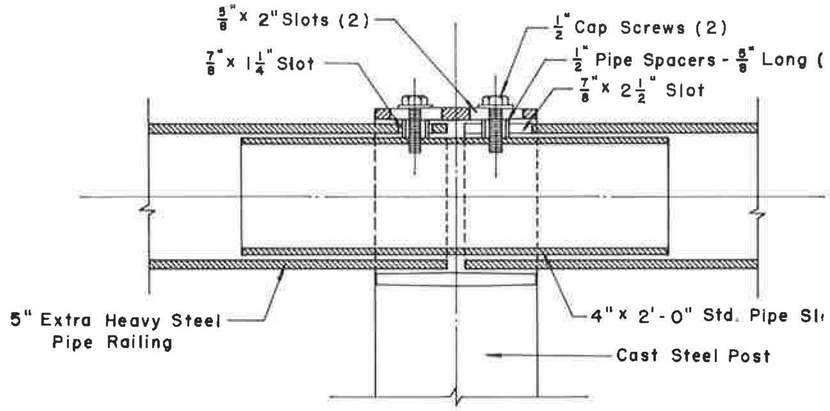

Figure 8A.

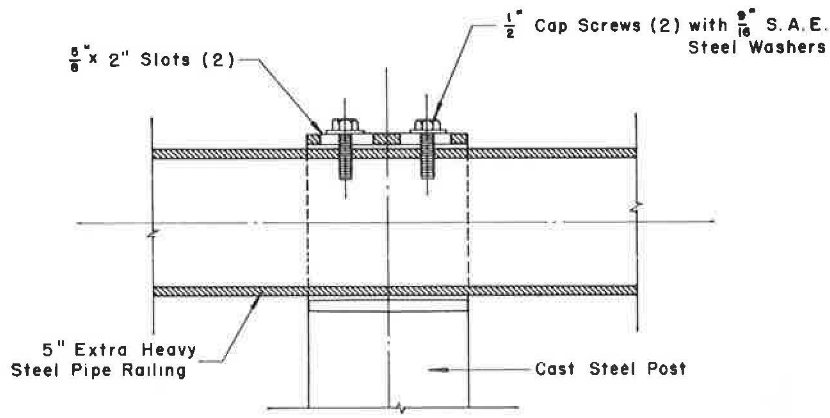

Figure 9A.

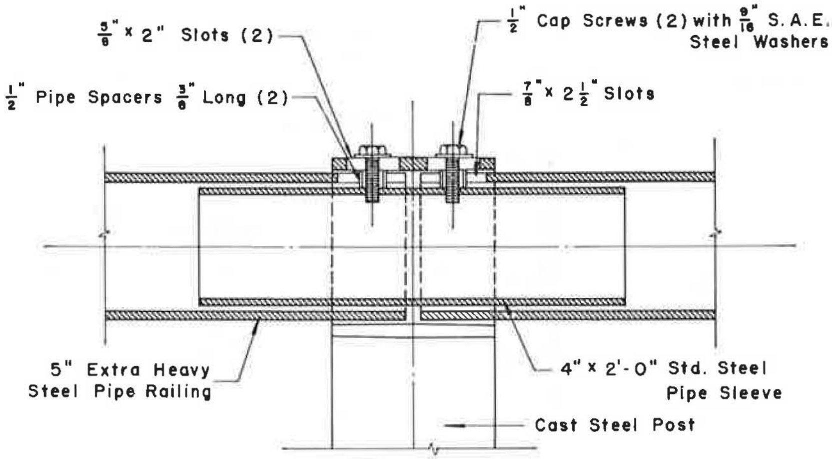

Figure 10A.

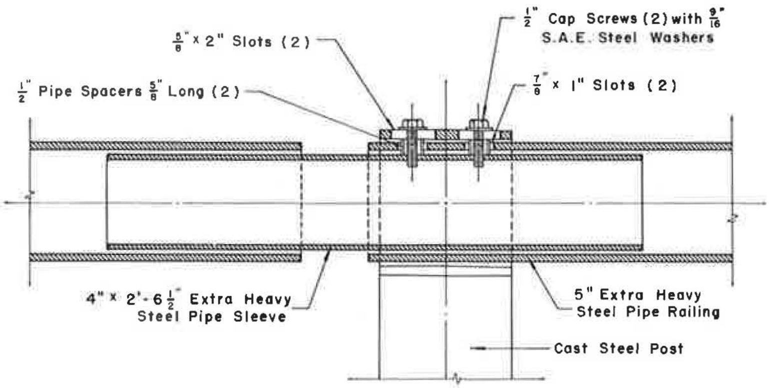

Figure 11A.

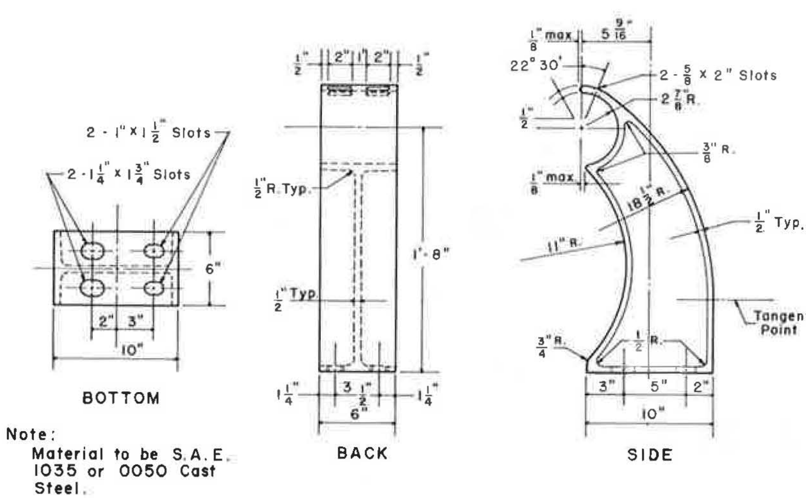

Figure 12A. Cast steel post details.

Development of an Analytical Procedure for Prediction of Highway Barrier Performance

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From 1960 to 1963, the New York State Department of Public Works, in cooperation with the U. S. Bureau of Public Roads, has sponsored and participated in a combined analysis and testing program on highway barriers at Cornell Aeronautical Laboratory. This program has included the full-scale dynamic testing of median barriers, guide rails and bridge rails to gain a fuller understanding of the forces involved between vehicle and barrier during collision. This work has resulted in the development of mathematical models for predicting the performance of current and proposed barrier designs. Barrier performance as predicted by computer solution of the mathematical model has been verified by subsequent full-scale crash tests.

Additional work is under way to evaluate more accurately some of the variables in the model and to refine the model accordingly. However, work to date has demonstrated the feasibility of predicting barrier performance by means of these equations and the development of a rational method of design of barrier systems based on dynamic principles.

•HIGHWAY BARRIERS are erected to delineate the roadway limits and physically restrain vehicles from entering areas they cannot safely traverse. Equally as important, impact with the barrier should not overturn the vehicle nor result in decelerations that would preclude human survival. The basic requirements of a highway barrier can be stated as follows:

1. Containment—The vehicle must not get beyond the barrier. If a barrier is required in an area, the consequences of penetration are presumably more serious than even a sudden stop on the barrier.
2. Minimum injury potential—To reduce the inherent hazards of a barrier collision, the vehicle must be decelerated at the lowest rate possible without exceeding the allowable barrier deflection.
3. Redirection—The barrier should not stop the vehicle abruptly or rebound it across the highway, thus presenting a hazard to following or adjacent traffic.

If highway barriers fulfill these requirements, the motoring public is assured the maximum possible protection. To provide this protection, the responsible highway department must determine the capabilities of existing, new, and modified barriers. The obvious approach has been to perform full-scale dynamic tests. A review of

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previous investigations revealed that a large number of tests had been performed. The resulting barrier designs were a significant improvement, but the tests appeared to be insufficient by themselves since at least one test was required for every factor evaluated. Research had also been conducted to analyze mathematically the reaction of a vehicle during a barrier collision. However, the mathematical analyses, without verification, were never demonstrated to be adequate because of the great number of simplifying assumptions required. Therefore, to achieve maximum benefit it was decided that this study would consist of two phases—a mathematical analysis of the reaction of a vehicle during collision with a barrier and a series of full-scale dynamic tests. This approach would permit the full-scale testing to serve as verification for the mathematical analysis and the mathematical analysis to prescribe the necessary changes in the barrier configuration.

The investigation was performed by Cornell Aeronautical Laboratory under contract with the New York State Department of Public Works, in cooperation with the U.S. Bureau of Public Roads. In all, 19 full-scale tests were performed—4 guide rail, 5 median barrier and 10 bridge rail tests. The theoretical analysis resulted in the development of four mathematical models programmed for solution by electronic computer. These models can successfully predict the performance of a highway barrier under a wide range of impact conditions.

GLOSSARY OF TERMS

"Snagging" of a vehicle wheel on a post is assumed to occur only when (a) the structural collapse of the vehicle has progressed to a point within the vehicle tread dimension, and (b) the corresponding deflection of the barrier rail (or cables) has progressed beyond the centerline of the undeflected posts. Since snagging occurs at a point on the post below the rail attachment, the yield force of a post can be considerably increased by the effect of rail tension if the individual posts are attached to the rail.

"Pocketing" of the vehicle is assumed to occur if the center of gravity of the vehicle passes the original centerline of the barrier while the vehicle is still headed into the barrier. The conditions for pocketing are considered to be similar to those for wheel snagging, except that the entire front of the vehicle, rather than only the impacting front wheel, is caught behind a post. If the vehicle were to pass through the barrier or be redirected out of the barrier, it would not be considered pocketed.

MATHEMATICAL MODELS

Mathematical models were constructed to compute the response of a vehicle during collision with a highway barrier. One type of model is used to calculate the barrier resistance during impact and another class of model is used to solve the vehicle responses.

The characteristics of a given barrier are first used to compute a series of force-deflection curves representing that barrier. The data required include post strength, post spacing, rail bending strength, and rail strength in tension. These force-deflection curves, along with the characteristics of the vehicle and additional post strength data, are then used to calculate vehicle responses for given impact conditions.

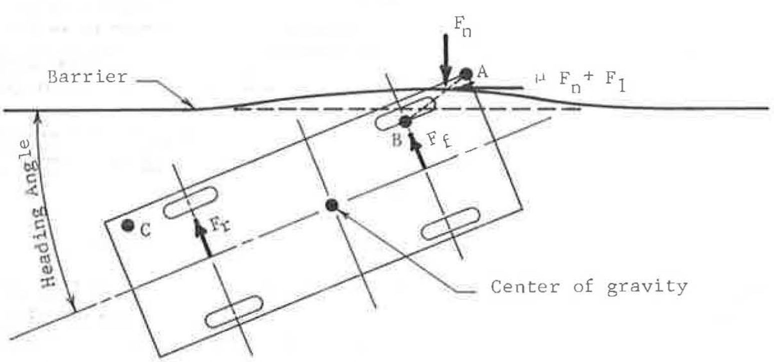
Vehicle responses are calculated by a repetitive process programmed for solution on an IBM 704 computer. During each millisecond of the collision, this process recalculates the vehicle position until the corresponding barrier deflections successively agree within specified limits (usually 0.01 inch). The computer than prints out the vehicle position, velocity, deceleration, and barrier deflection.

To account for the many important physical characteristics of vehicle and barrier and the forces which act during a collision, several simplifying assumptions must be made. The fact that good verification was obtained with full-scale tests appears to justify these simplifications. The relatively small portion of the total mass in the deformed part of the vehicle permits the vehicle to be regarded as a single rigid body. The horizontal force between vehicle and barrier is assumed to be a concentrated point load. All forces on the vehicle are assumed to act in the same horizontal plane and vehicle roll and pitch are neglected. In the primary (front-end) barrier impact, a given amount of energy is assumed to be dissipated within the vehicle as it is crushed. During crushing, the force between vehicle and barrier is assumed to increase linearly as it moves along a straight line (A-B, Fig. 1). A secondary (rear end) collision is assumed to occur and the barrier force is instantly moved from point B to point C when the line drawn from B to C becomes parallel to the original barrier centerline. The forces between tires and pavement (F_r and F_f, Fig. 1) are applied along the axles at the intersection of the car centerline. The coefficient of friction (μ) between vehicle and barrier is assumed to be constant during the collision. The longitudinal force on the vehicle caused by each post (F_l) is assumed to be constant and active for a given distance. The mass of the barrier is neglected.

Separate models were developed for three classes of barriers, depending on the way the rails resist impact. These classes correspond to the manner in which the rail acts as follows: (a) bending resistance only, (b) axial tension only, and (c) combined axial tension and bending resistance. Barriers not intended to deflect when impacted were not analyzed because they impart severe decelerations to the vehicle. The vehicle reaction during impact with a rigid barrier is also greatly influenced by the crushing characteristics of that particular vehicle.

A rail with bending resistance only is treated as a continuous beam which distributes the impact load over several posts. Progressing outward from the impact point, the posts provide increasing lateral support for the rail until they yield. Thereafter, each yielded post sustains a constant load until enough posts have yielded to resist the impact force completely. The rail is assumed to form a constant moment plastic hinge at the point of impact. It is this yielding of posts and rail that minimizes car deceleration and at the same time provides enough resistance to turn the car and redirect it.

A rail with axial tension only (e.g., cables) is represented by straight-line segments which intersect at post locations, starting at the applied load position and continuing to the original centerline of the barrier at a post location for which the lateral deflection is assumed negligible. Tension is calculated from the axial restraint at the end of the barrier and the total elongation of the cable. The following iterative method is applied: (a) an assumed tension is used to obtain a deflection profile for a given solution point; (b) the elongation of the cable is calculated from this deflection profile to determine the corresponding calculated tension; (c) the assumed and calculated tensions are then compared and an average is taken, if necessary, for a second assumed tension; and (d) this process is repeated at each solution point until agreement, within



A. Initial position of concentrated load point on vehicle (primary collision).
B. Position of concentrated load point on vehicle for maximum vehicle deformation (primary collision).
C. Position of concentrated load point on vehicle for secondary collision.

Figure 1. Vehicle model.

specified limits, is obtained between the assumed and the calculated values of axial tension. The axial restraint at the ends of the cable caused by end anchorages is treated as a linear tension spring.

If a barrier has W section or universal beam-type guide rails, the rails are assumed to act in a combination of lateral bending and axial tension. As the rail deflects laterally, the bending resistance increases until plastic yielding occurs, then decreases as axial tension increases. When the rail yields in tension, the lateral bending resistance is assumed to be zero.

The three barrier models and the vehicle response model have been completely described previously (1, 2).

FULL-SCALE TESTS

Crash tests were performed to verify the mathematical description of vehicle reaction during collision with the barrier. In addition, the tests yielded some information for refining the assumptions made in the mathematical models. The department's personnel also conducted dynamic tests near Albany to determine post strength in soil for use in the barrier models.

The crash test site was a wide concrete ramp on a privately owned portion of the Niagara Falls Municipal Airport. Since most vehicles on the highway are American-made medium-priced sedans, the vehicles obtained for crash tests were standard 1957 Ford and 1959 Plymouth sedans. During the first year of testing, the vehicles were remotely controlled with radio-activated equipment generously loaned by the California Department of Public Works. For the second and third years of testing, Cornell Aeronautical Laboratory developed an electrical servo-control mechanism that was found to be more satisfactory.

Particular attention was given to photographic coverage of the crash, since reduction of the movie film would provide vehicle position and acceleration while in contact with the barrier. Four data cameras, with shutter speeds of about 1,000 fps, and two or more documentary cameras were used to record the impact. To provide duplication in case of camera failure, the data cameras were placed at each end of the barrier, in a tower, and facing the barrier. Common time references were provided by pips on the film edges and two flash bulbs fired as the vehicle passed over switch tapes located 5 and 15 feet from the impact point. The time between flashes also provided a means of computing vehicle velocity at impact.

Before performing full-scale tests, it was necessary to select realistic impact conditions. It was realized that when a car, traveling parallel to a barrier, is suddenly turned sharply toward the barrier, there is a minimum radius of curvature which the vehicle is capable of negotiating. The vehicle is unable to make a sharper turn simply because the tires will not develop enough centripetal force to provide the necessary radial acceleration. Therefore, by assuming a reasonable maximum coefficient of friction (0.7), the maximum probable impact angles could be computed for various speeds and widths of highway. Such an analysis indicated that the maximum impact conditions for the field tests should be 60 mph and 25°. The majority of tests were performed under these conditions.

VERIFICATION OF MATHEMATICAL MODELS

The high-speed data films recorded the vehicle center of gravity location, heading angle and barrier deflection during the collision. The velocity and deceleration of the car were then measured by obtaining the first and second derivatives of the center of gravity locations. The reduction of vehicle motions from the film records may introduce a general error of about 10 percent and some peak decelerations may be as much as 20 percent in error.

Agreement between full-scale test results and computed results, using the actual speed and angle of impact, is best illustrated by comparing the vehicle position and, more importantly, vehicle deceleration during barrier impact. When comparing decelerations, durations were considered. A very high momentary vehicle deceleration would have little adverse effect on the occupants. The average deceleration over the highest 100-millisecond interval is considered more significant and was used in summarizing measured and computed values. For discussion, the barriers are classified by the way the rails resist vehicle impact.

Rails With Bending Resistance

Barriers which resist vehicle penetration primarily through the action of a strong rail were developed during the study as a result of a better understanding of the principles of barrier reaction to impact. In this type of barrier, posts are designed to allow lateral deflection of the rail and thus reduce vehicle deceleration. To keep deflection within acceptable limits, the rail must be stiff enough to distribute the impact load over a number of posts. The posts in turn must be able to absorb the lateral kinetic energy of the vehicle as it is redirected. This system is best applied to median barriers and bridge rails. During the program both types of barrier were developed and tested.

The median barrier is shown in Figure 2. Since the system consisted basically of a strong rectangular hollow rail, it was labeled a box-beam median barrier. The rail rests in saddles fastened to small posts which restrict lateral movement of the rail but allow the rail to remain at the original elevation when posts are struck down or deflected laterally.

Two tests were performed on this class of barrier. The first barrier had end anchors and the second did not. However, the results were not significantly different and only those of the second test are presented for verification of the mathematical model. The agreement obtained between measured and computed vehicle trajectories is illustrated in Figure 3 where the locations of vehicle center of gravity are shown

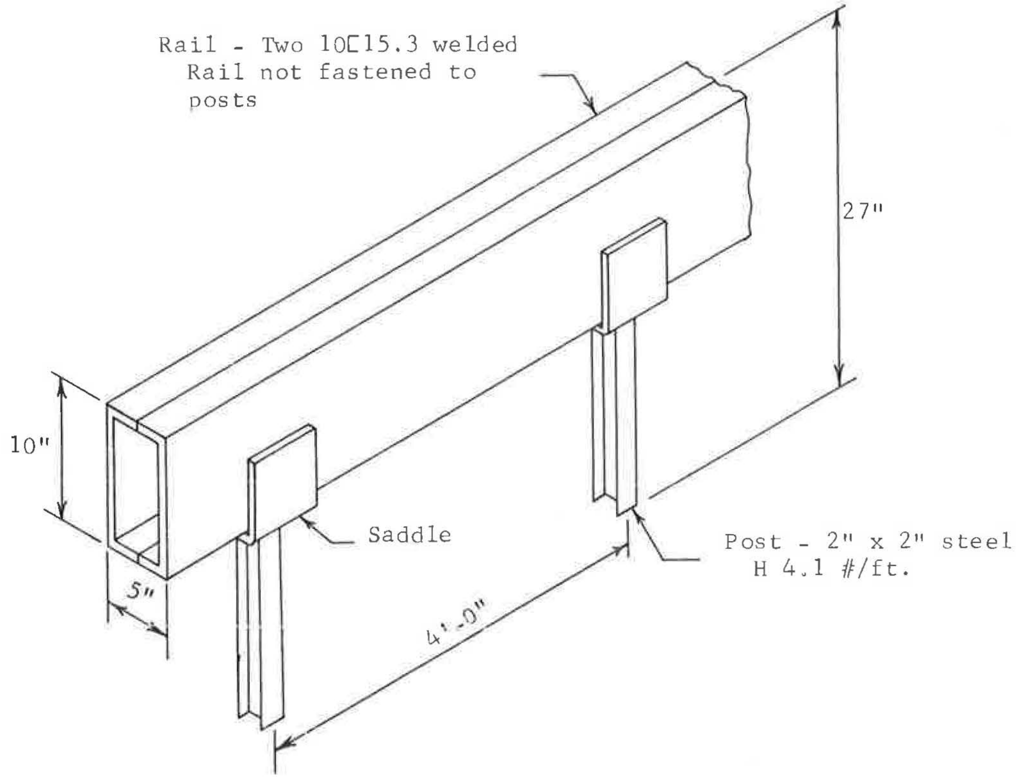


Figure 2. Box-beam median barrier.

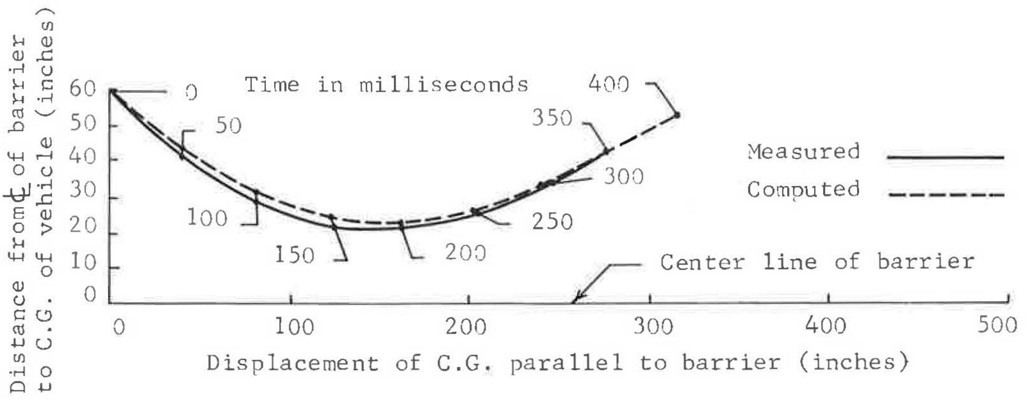


Figure 3. Vehicle trajectory, box-beam median barrier.

for each 50-millisecond interval. A very slight difference between the two curves is evident. The vehicle decelerations are plotted in Figure 4 and these also agree very well except for a high measured peak at about 125 milliseconds. The average measured total deceleration (7 g) over the highest 100-millisecond interval is not appreciably different from the computed value (6 g).

It is significant that the simplifications and assumptions used in calculating vehicle responses are verified by the close agreement with measured vehicle responses. With this agreement, it is possible to study the effects on vehicle responses caused by changing rail strength, post strength, post spacing, and impact conditions.

Following the success of the box-beam median barrier tests, a bridge rail was designed using the same strong beam-weak post principle. In this system, two rails

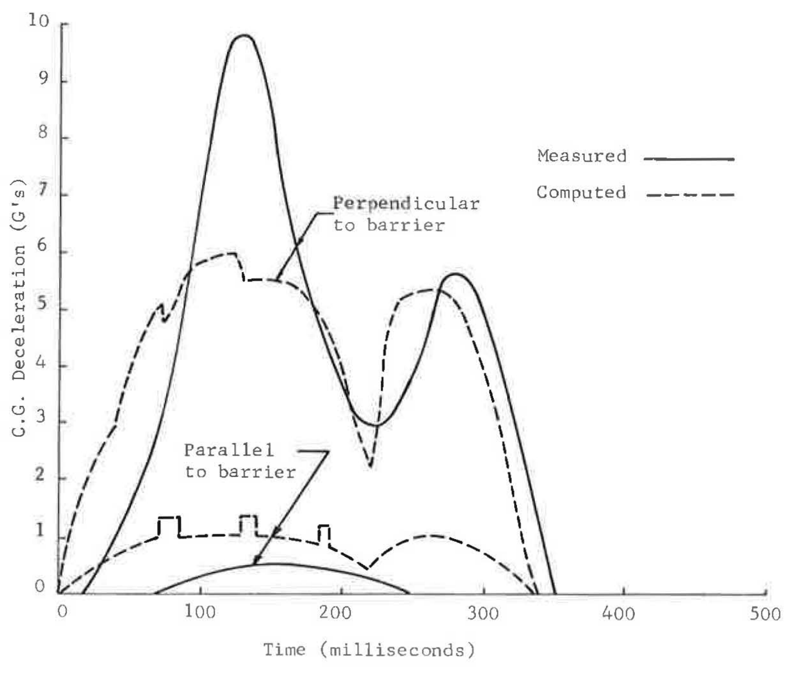


Figure 4. Vehicle deceleration, box-beam median barrier.

were fastened to the face of relatively weak posts. Although the posts were the same size as the median barrier posts, bolting them directly to the rails made the posts develop more resistance to lateral deflection and more resistance when struck by a vehicle. In addition, the location of the rails on the face of the posts meant that the vehicle contacted and knocked down fewer posts. This bridge rail, shown in Figure 5, was first impacted with a car and then tested with a school bus. In the calculation of the barrier force-deflection curve and vehicle responses, the rails were assumed to act as a single rail; this was verified by the test data films. In calculating the school bus response, the effects of crushing during first contact with the barrier had to be neglected.

Vehicle decelerations for these two tests are shown in Figures 6 and 7. The agreement between computed and measured responses is not quite so good as the agreement achieved with the median barrier. The computed vehicle responses depend on the assumption that the barrier absorbs nearly all of the lateral kinetic energy of the vehicle during impact. Since the barrier was relatively stiff, the car crushed and absorbed more energy than had been assumed. Therefore, the computed responses appear to be slightly in error. The computed deceleration curve for the car indicates that a secondary rear-end collision should occur (indicated by the second peak at about 250 milliseconds). However, the duration of this peak is so short that it is not significant. Both computed trajectory and deceleration curves indicate that the barrier is stiffer than the median barrier and the actual test results confirm this very clearly. The agreement between measured and computed responses for the school bus demonstrates the versatility of the model used for calculating vehicle responses.

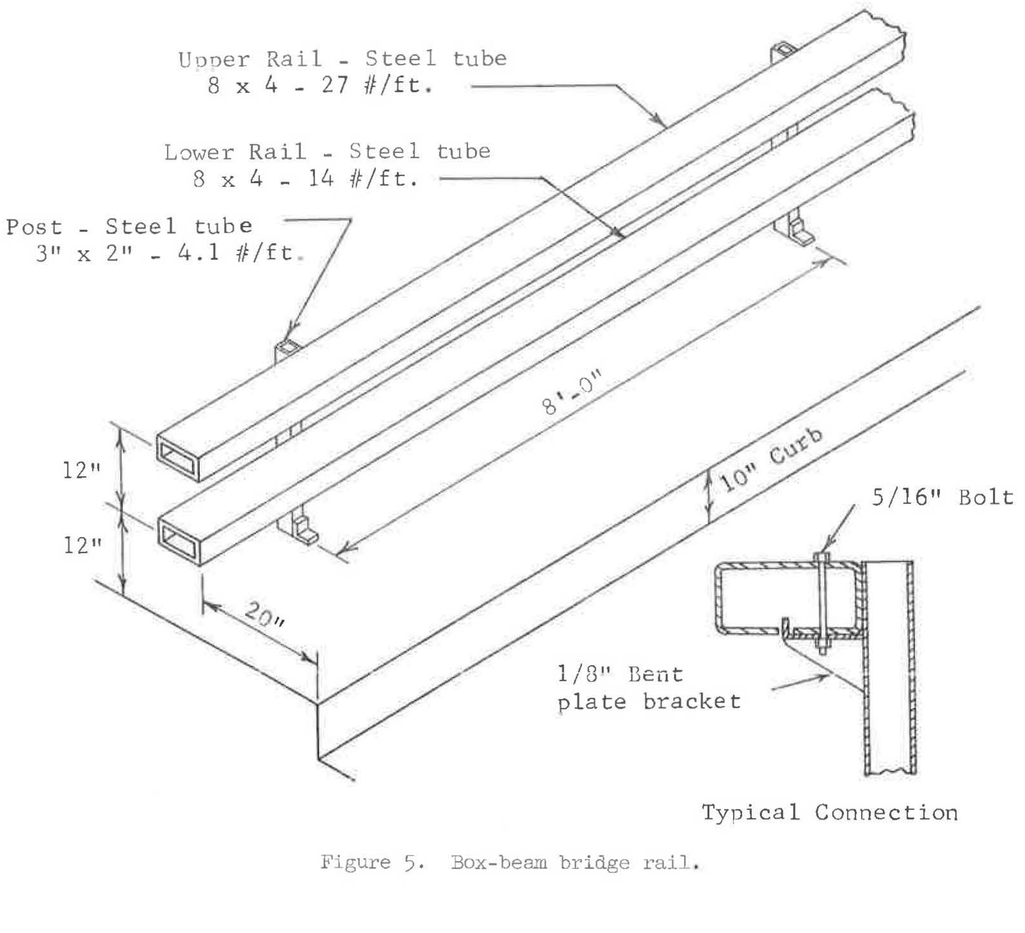


Figure 5. Box-beam bridge rail.

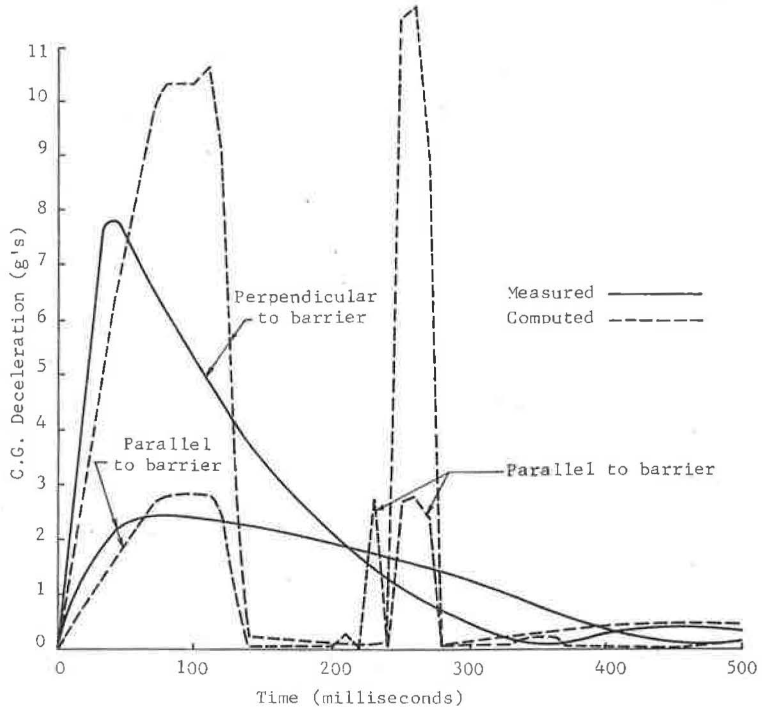


Figure 6. Car deceleration, box-beam bridge rail.

Rails With Axial Tension

The initial test was performed on a typical guide rail arrangement (Fig. 8) without first computing expected vehicle responses. The entire length of the vehicle penetrated the barrier with very little redirection; then the vehicle was severely pitched and rotated as the cable tightened around the strong posts. The computed vehicle responses could not show the effects of pitch and turning, but the occurrence of severe pocketing was indicated by the computed vehicle trajectory and heading angles. The center of gravity of the car passed the original face of the barrier at about 150 milliseconds after first contact. The computed heading angle at 150 milliseconds was nearly the same as at first contact, indicating that the vehicle was not being redirected and would remain in the barrier. Knowing the high strength of the steel posts, it could

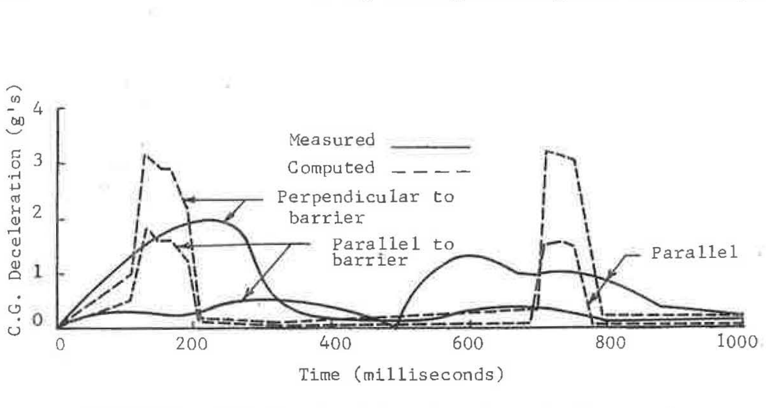


Figure 7. Bus deceleration, box-beam bridge rail.

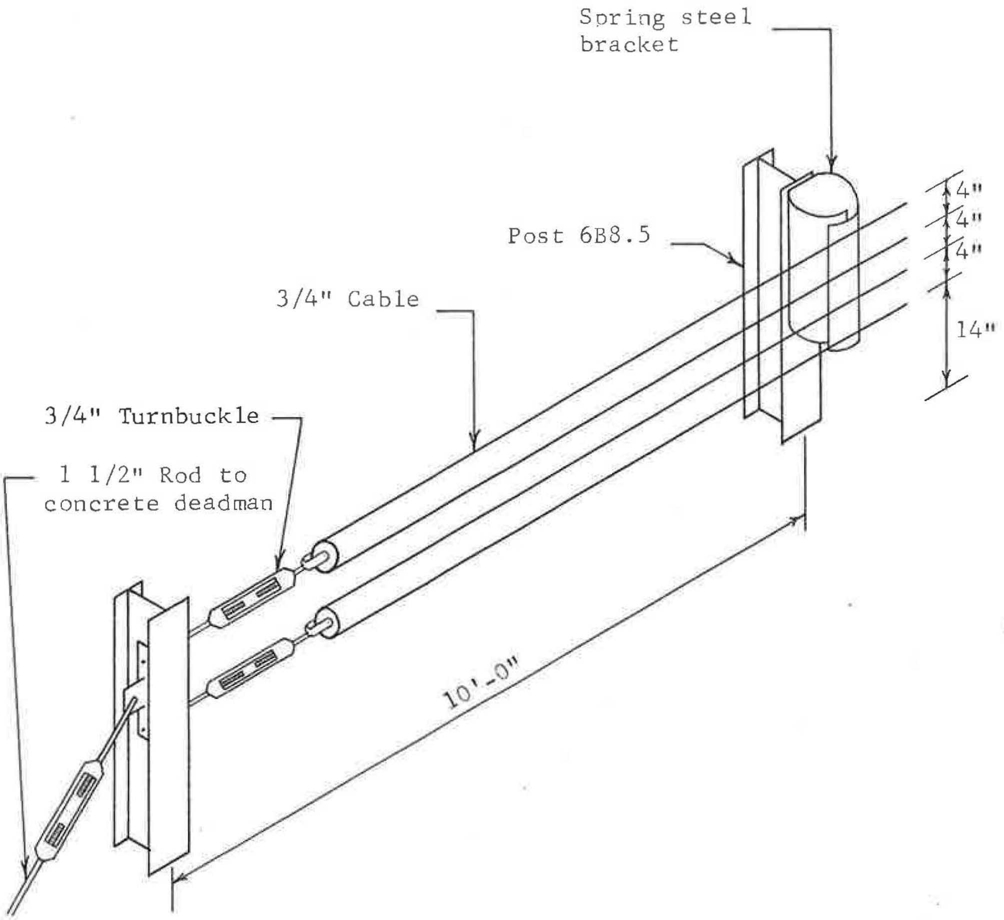


Figure 8. Typical cable guide rail.

only be concluded that the vehicle would be severely pocketed. The computed trajectory and heading angles were not available before the test; had they been, the barrier need not have been tested or at least not tested so severely.

A second cable guide rail arrangement (Fig. 9) was designed with the benefit of a better understanding of tension barriers and the influence of post strength on vehicle response. In this system the cables are supported by relatively weak posts which deflect easily when struck by a vehicle. The cables are fastened to the posts with J bolts which open when a post is struck directly or after the post has yielded and deflected too far to be effective. In addition, it was believed that when a vehicle struck the barrier at a shallow angle, the cables would be free to deflect without being pulled down by the posts. For testing, this barrier was placed in front of a ditch representing a highway fill on a 2 to 1 slope. This was done to study the effect the large anticipated deflections might have on vehicle roll. During the test of this barrier, the car actually passed behind several posts, became airborne, tipped slightly toward the road, settled onto the backslope and finally returned to the shoulder.

The computed and measured vehicle decelerations are shown in Figure 10. Agreement is extremely good perpendicular to the barrier up to about 400 milliseconds, when the computations assume a secondary (rear) collision which would increase the lateral deceleration. The cable was really in contact with the entire side of the car throughout most of the impact and no separate rear collision ever occurred. Deceleration parallel

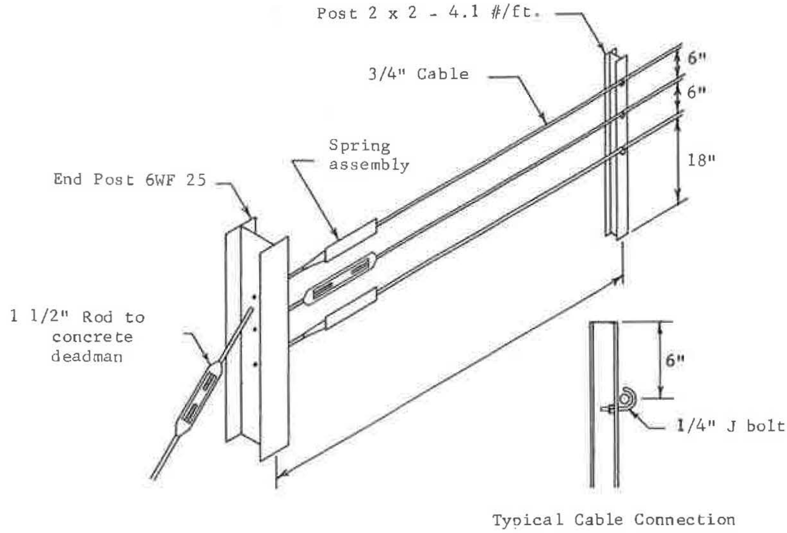


Figure 9. Modified cable guide rail.

to the barrier is influenced by the assumed coefficient of friction between vehicle and rail. A lower coefficient than used previously was assumed; however, the three cables pulled together and cut into the vehicle sheet metal. This action may have increased deceleration parallel to the barrier since the cables appeared to be wrapping and unwrapping as the vehicle passed. The fact that the vehicle was airborne for about 200 milliseconds and then came down on the backslope would also influence agreement. Considering all of these complications, agreement between measured and computed vehicle response is very good with this modified cable guide rail.

Rails With Combined Bending Resistance and Axial Tension

W section or universal beam-type guide rails usually act in a combination of bending resistance and axial tension. Two tests were performed on guide rail systems constructed with these rails, but in both cases pocketing occurred and was indicated by the calculated responses. A third test on the median barrier, illustrated in Figure 11, provided data better illustrating agreement between computed and measured vehicle response.

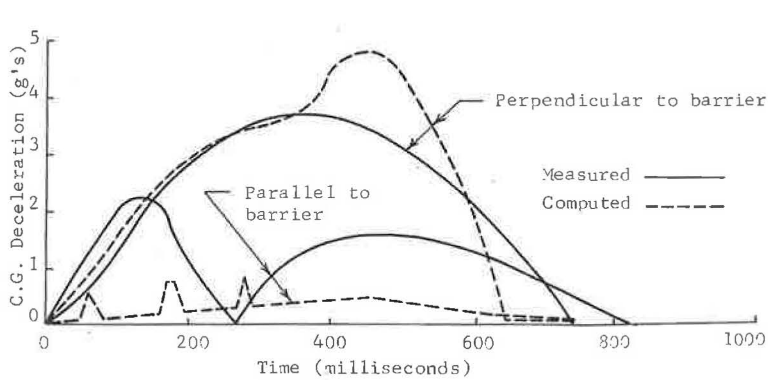


Figure 10. Vehicle deceleration, modified cable guide rail.

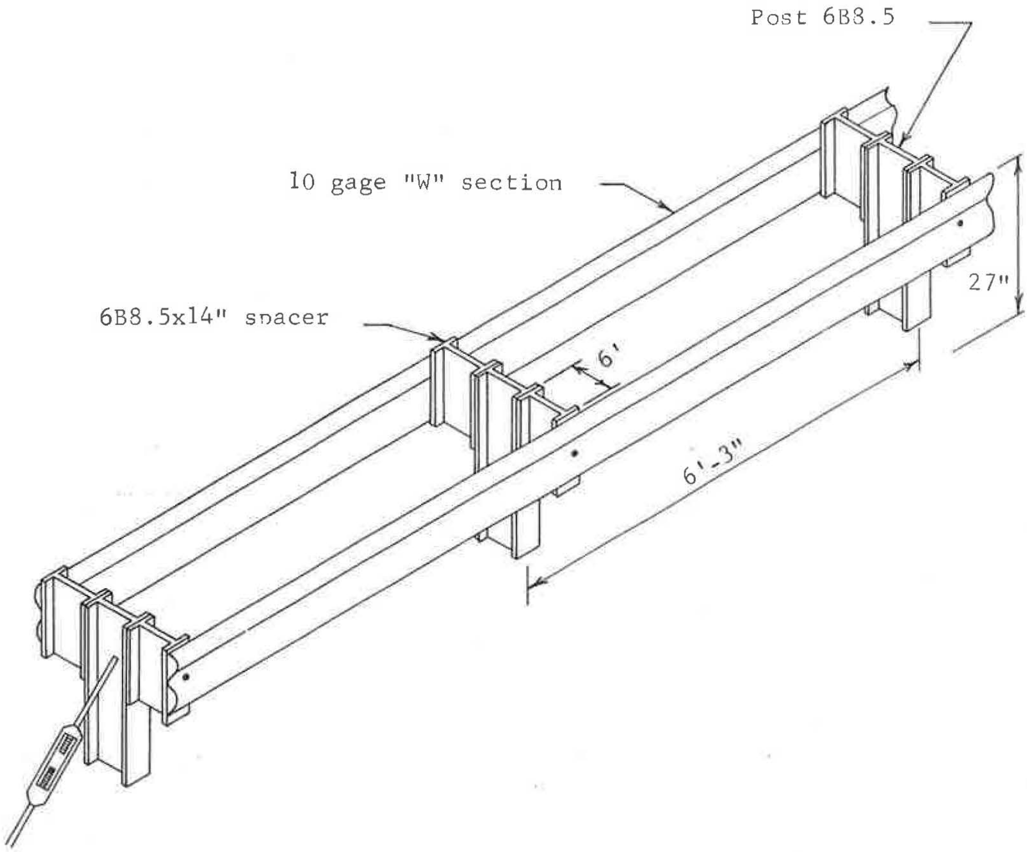


Figure 11. W section median barrier.

Considering that this barrier is "lumpy" (rigid at the posts and flexible midway between posts) and that secondary collisions are not computed precisely, the computed responses are remarkably close to the measured responses for the first 200 milliseconds of barrier contact (Fig. 12). The computed peak deceleration due to a rear

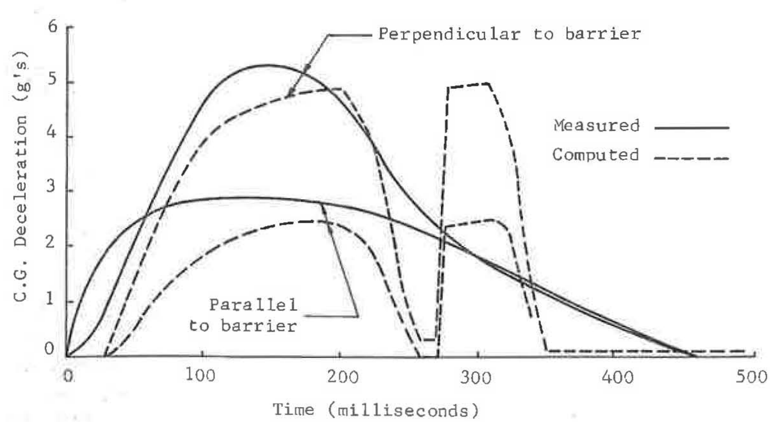


Figure 12. Vehicle deceleration, W section median barrier.

TABLE 1
MEASURED VS COMPUTED VEHICLE RESPONSE

Barrier Type	Impact Condition		Exit Speed (mph)		Exit Angle (deg.)		Max. Decel.[a] (g)		Max. Deflec. (in.)	
	Mph	Deg.	Meas.	Comp.	Meas.	Comp.	Meas.	Comp.	Meas.	Comp.
Box-beam median	52	24	47	44	7	6	7	6	10	13
Box-beam bridge rail:										
Car impact	55	25	37	44	5	3	7	9	5	5
Bus impact	30	20	25	23	6	2	2	3	4	5
Cable guide rail system:										
Modified	57	30	37	46	10	16	4	4	150	123
Typical			Vehicle pocketed as indicated							
W section median barrier	64	16	48	49	9	7	6	5	18	12

[a]Average over highest 100-millisecond interval.

collision at 300 milliseconds did not occur. However, the duration of this peak was very short and this discrepancy is not considered to be serious. The computed vehicle responses for this barrier are in good agreement with the measured responses; therefore, the models for this barrier and for the vehicle responses can be used with confidence to evaluate this type of barrier.

The verification of the mathematical analysis can be summarized as in Table 1. It is possible here to compare the measured and computed vehicle response for the three classes of barriers considered and to obtain a general idea of the ability of the barriers to fulfill the criteria of redirection, minimum injury potential, and containment.

SUMMARY

Full-scale dynamic tests of highway barriers provide factual information for barrier design. However, these tests are inefficient by themselves since at least one test is required for every factor to be evaluated. However, describing the vehicle reaction with mathematical equations is inadequate without verification by full-scale tests because of the great number of simplifying assumptions required. This investigation combined the theoretical approach with full-scale testing to produce mathematical models which will successfully describe the reaction of a vehicle during collison with a barrier.

In all, 19 full-scale dynamic barrier tests were run—4 guide rails, 5 median barrier and 10 bridge rail tests. The theoretical analysis resulted in the development of four mathematical models programmed for solution by electronic computer. These models were verified by the full-scale tests and found to be capable of predicting the performance of various barriers over a wide range of impact conditions. Characteristics of the vehicle and barrier, used as input for the mathematical models, were determined from structural analysis, direct measurement, published data, and dynamic post tests.

The complexity of the problem necessitated separate mathematical models for determining the force vs deflection characteristics of different types of barriers. The force vs deflection data obtained from the appropriate barrier model is used as input into another series of equations which yield printed tables of vehicle trajectory, vehicle deceleration and barrier deflection for each 10 milliseconds during the collision.

The use of the mathematical models has enabled New York State to revise its standard double beam median barrier and further refine the box-beam system developed during this investigation. In addition, the computer programs have permitted the evaluation of a new bridge rail design using principles developed during this project.

The mathematical models can be used either to extrapolate results from a limited number of full-scale tests or to provide a completely analytical evaluation of an untested barrier. New York State is continuing to obtain vehicle trajectories with the computer, thereby evaluating specific barrier designs over a wide range of impact conditions. In this way, information will be available on the level of protection offered by current barrier designs, and improvements that can be achieved by modification

of these barriers. Moreover, new designs can be formulated which will provide optimum performance in a specific situation.

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