RELATING NO-PASSING ZONE CONFIGURATIONS ON RURAL TWO-LANE HIGHWAYS TO THROUGHPUT TRAFFIC

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Because the roadway designer has no satisfactory technique to evaluate the traffic flow consequences of making selective passing sight distance improvements at spot locations on two-lane, two-way rural highways, a study was undertaken to investigate the relationships between the two independent variables (percentage of total length of a section of highway marked with no-passing barriers and the traffic volume input to the section of highway) and the dependent throughput variables (mean speed, speed variance, volume, travel time, and completed passes). Additional dependent variables investigated, but not verified with field observations, were attempted passes, delay, and speed change cycles. A digital computer traffic model is used to simulate traffic flow over 5- to 6-mile sections of highway at nine field locations in North Carolina. Volumes ranged from 175 to 650 vph. The model is calibrated to generate simulation throughput data that statistically match the throughput data observed at the field sites. The calibrated model is then used to analyze simulated systematic changes in no-passing barriers, over volume ranges of 175 to 1,200 vph. The conclusions are that the model is sensitive to these changes, as reflected by the throughput statistics, and that the dependent variables can be correlated with the independent variables in a statistically reliable manner, using multiple linear regression.

•TWO-LANE rural roads in rolling or hilly topography pose a problem for the motorist. Extended sections of restricted horizontal and vertical sight distance, coupled with inadequate passing opportunities, can make the overtaking and passing of slowly moving vehicles difficult or impossible. This type of highway environment not only promotes unsafe passing attempts by the driver but also tends to decrease average vehicle speed for the traffic stream. From the roadway designer's viewpoint, extensive no-passing barriers and the consequential inability of motorists to pass slower vehicles can cause reductions in throughput and level of service, while at the same time increasing delay, traffic interference, and accident potential.

The problem of up-grading a two-lane rural highway is more often one of making selective improvements at spot locations because of fund limitations or because these highways are not expected to develop traffic volumes in the future large enough to warrant complete reconstruction. But where and how should the funds be spent on improvements at spot locations for maximum cost-effectiveness? To answer this question, one must know the manner in which passing zone configurations influence traffic volumes, speed, delay, and level of service. The highway design manuals indicate only general guidelines relative to the provision of passing zones on two-lane roadways (1). It is apparent that there is a need to establish more specific relationships between no-passing zone configurations and the resulting throughput traffic performance. For this reason, a study was undertaken to investigate the relationships between the two independent variables—percentage of the total length of a 5- to 6-mile section of two-lane highway marked with no-passing barriers and the traffic volume input to the section of highway—and the dependent throughput variables—mean speed, speed variance, travel time, traffic volume, and completed passes. (Throughput variables are defined as traffic statistics that have been calculated using only data for those vehicles, moving in both directions, that have traversed the entire length of a specified section of highway.) Additional dependent throughput variables related to the independent variables, but not verified through observation, were delay (calculated on the basis of the time consumed while a vehicle is prevented by other vehicles from traveling at its desired speed), speed change cycles, and attempted passes. A speed change is a measure of the change in operating speed for a vehicle. A speed change cycle is a measure of the change in operating speed from and back to an initial speed (e.g., from 50 mph to 30 mph and then back to 50 mph).

METHODOLOGY

Various methodologies for achieving the research objectives were reviewed, including empirical techniques, mathematical models, and computer models. The large number of highway geometric and traffic variables, along with many no-passing zone configurations, that have the potential to influence throughput traffic suggests that the cost of obtaining statistically reliable observations at field sites would be prohibitive. Technical literature indicates numerous mathematical models, but the majority describes only a particular aspect of traffic flow (2), and in none of the models is the passing maneuver of primary importance. Computer simulation models have been written to describe traffic behavior on a vehicle-by-vehicle basis. Moreover, input data for the variables used in the computer model can be systematically altered and the consequences noted in the output statistics. Implicit in the use of any computer model for analysis of traffic flow behavior relative to passing zone configurations are the requirements that the simulation roadway incorporated into the model be a reasonably accurate representation of the field site, that the movement and interactions of individual vehicles generally approximate actual driver behavior, and that the vehicles moving over the simulation roadway interact and respond to the simulated highway environment.

During the early phases of this investigation, the authors were notified of the availability of a recently developed computer simulation model for traffic flow on two-lane roadways; the model simulated actual passing maneuvers. This computer model had been developed by Janoff and Cassel at the Franklin Institute Research Laboratories (FIRL) (5). Because of the unique characteristics of the FIRL model, coupled with the overall advantages of computer simulation in the analysis of no-passing zone configurations, computer simulation was chosen as the fundamental analytical tool.

The overall strategy in the investigation was to utilize the basic FIRL model and to develop additions or changes to it as dictated by roadway design requirements. The testing and calibration of these revisions utilized data from nine locations. After calibration, the revised model was employed to analyze the consequences of changes in no-passing zone barriers at an actual field site. The experience gained in this exercise was then used to formalize a set of procedures for the application of the model to a typical roadway design or redesign alternative.

Development of Revised Computer Simulation Model

Although the immediate objective of this project was to analyze no-passing zone configurations relative to throughput traffic performance, a larger objective was to provide the roadway designer with a tool that would assist him in making decisions regarding optimal locations for passing zones. Because it was anticipated that any computer model developed would be employed to simulate and analyze traffic flow on North Carolina rural highways, it was essential that the model be capable of simulating a wide range of field conditions, including those normally found on these highways.

Collection of Field Data—The development of a set of computer programs that will model roadway conditions and simulate traffic behavior on North Carolina rural primary two-lane highways requires a set of field observations from these highways so that (a) the simulation model can be properly calibrated and (b) simulation throughput can be compared with actual highway throughput for the purpose of checking the realism and accuracy of the output data from the computer model. Field observations were made at nine sites on the rural primary system during the summers of 1969 and 1970. The field sites selected were 5 to 6 miles long and had no major intersecting highways or traffic signals. The data developed from these observations were used to calibrate input variables to the simulation model and to verify simulation throughput (Table 1).

Functional Specifications—When the range of traffic and geometric variables utilized by the roadway designer was reviewed, it appeared that a traffic simulation model should have the following functional capabilities, insofar as this investigation was concerned: (a) to simulate any specified two-way traffic volume from 150 to 1,200 vehicles per hour (vph); (b) to simulate any individual traffic lane volume from 75 to 1,000 vph; (c) to simulate any specified percentage distribution of passenger cars, medium trucks, and heavy trucks; (d) to simulate acceleration and deceleration characteristics of medium and heavy trucks on gradient sections ranging from -8 to +8 percent; (e) to utilize different speed distributions, if desired, for the three classes of simulation vehicles; (f) to generate, for any specified traffic lane volume per hour, an ordered list of headways that when added cumulatively will equal 3,600 sec at the point in the ordered list when the number of headways added cumulatively is equal to the specified hourly traffic volume; (g) to generate input queues of vehicles for the simulation roadway in which the individual speed and headway assigned to each vehicle are a part of a distribution of speeds and headways normally found on rural primary highways; and (h) to simulate two-way volumes as high as 1,200 vph for nominal real-time computer costs.

It should be noted that the FIRL traffic model permits the user to specify no-passing zone locations in almost any type of configuration.

Summary of Revised Computer Simulation Model—Because it is not the purpose of this paper to report the details of the development, testing, and calibration of the revised computer model, only an abbreviated flow chart is shown in Figure 1. The speed-headway program generates an ordered list of vehicles, which in turn is input to the NCSU modified model. The latter handles all vehicle simulation routines and the calculation of output statistics.

The FIRL traffic model consisted of a main routine and three subroutines. These programs have been incorporated into the NCSU modified model with only minor modifications. However, two additional subroutines have been written and added to the main program of the FIRL model. The speed-headway program and its integration into the total simulation procedure were developed at North Carolina State University as a part of this investigation.

The major input data for the NCSU modified model related to the roadway are length of highway section, no-passing zone barriers and coordinate locations for beginning and end, vertical gradients and coordinate location, and coordinate locations for restricted stopping sight distance. The input data related to traffic and vehicles are traffic lane volumes in vehicles per unit of time; percentages of medium and heavy trucks; mean and standard deviation for an input (i.e., desired) speed distribution for passenger cars and medium and heavy trucks; nominal rate of acceleration and maximum rate of deceleration for passenger cars (acceleration and deceleration characteristics for trucks are stored in the truck-on-grade subroutine); maximum attainable speed for all vehicles; maximum headway in simulation traffic stream; and minimum stopping distance. Input data related to simulation are percentile value from the throughput speed distribution to be used as the operating speed, length of real time to be simulated, and number of intermediate update reports desired between start and end of simulation.

Conclusions Regarding Revised Computer Simulation Model

The development, testing, and calibration of the revised computer model, consisting of the NCSU modified model and the speed-headway program, occupied a major portion of the time and effort expended on the project. This preliminary but essential work provided the necessary verification for the following facets of the revised computer model:

1. That data input to the model can be quantitatively related to model output;

Table 1. Roadway and traffic for nine North Carolina sites.

Site No.	Location	Road Length (ft)	No-Passing Zone (percent)	Largest Grade (percent)	Two-Way Volume (vph)	Directional Distribution of Traffic (percent)	Trucks in Traffic Stream (percent)	Average Speed (mph)	Range of Individual Travel Speeds (mph): Fastest	Range of Individual Travel Speeds (mph): Slowest	Posted Speed (mph)
1-69	US-1, south of NC-55	26,442	0	1	208	43-57	13	62.7	75.0	48.0	60
2-69	US-1, south of site 1	30,571	10.2	3	177	46-54	19	62.7	77.0	49.5	60
3-69	US-64, 13.7 miles west of US-1	27,361	46.3	7	227	49-51	15	51.7	70.0	43.0	60
1-70	US-15,501, 3 miles north of Creedmore	28,692	36.7	5	686	45-55	17	51.0	73.5	38.5	55
2-70	US-15,501, 3 miles north of Pittsboro	28,846	49.1	5	291	43-57	11	50.0	66.0	38.5	55
4a-70	NC-54, 1 mile west of Morrisville	16,329	51.0	4	268	53-47	8	48.5	67.5	37.0	55
4b-70	NC-54, 1 mile west of Morrisville	16,329	51.0	4	762	83-17	2	48.0	65.5	42.0	55
5-70	US-64, 1.43 miles west of I-40	29,416	61.0	6	506	54-46	21	50.0	67.0	37.0	55
6-70	US-301, 1.95 miles south of I-95	34,821	10	3	642	46-54	15	50.0	63.5	40.5	55

Figure 1. Flow chart for NCSU modified model and speed-headway program.

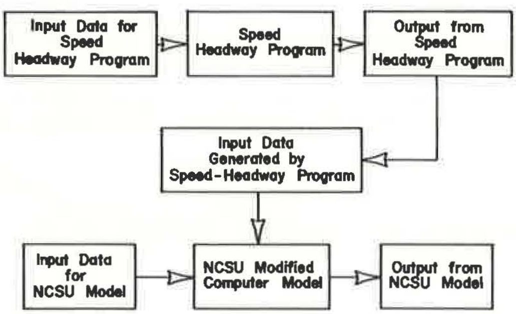
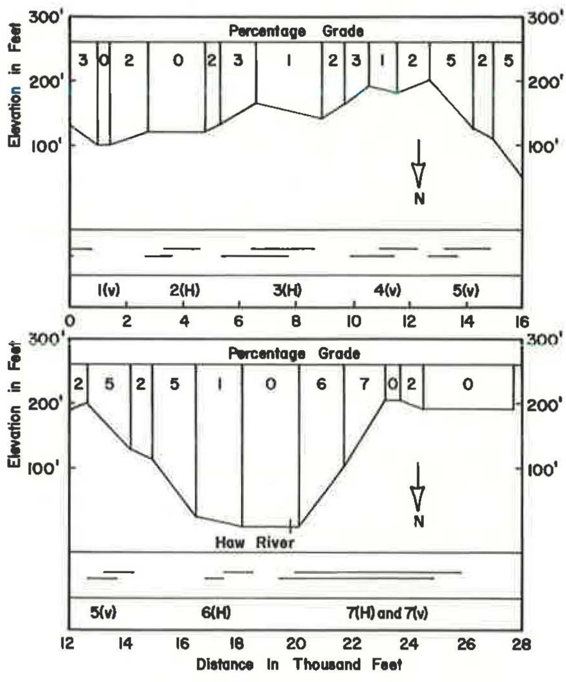


Figure 2. Vertical profile and no-passing barriers for a section of US-64.



2. That selective changes in input will produce predictable changes in output;
3. That, given a set of field conditions for simulation, the necessary computer input data can be specified to produce simulation throughput data that will statistically match throughput data from the field site over volume ranges of 175 to 650 vph;
4. That the revised model appears to be realistically simulating two-lane, two-way traffic flow; and
5. That, on large-scale computer systems such as an IBM 370/165, the computer time for simulation appears quite reasonable (for a 5-mile section of roadway and a two-way traffic volume of 600 vph, approximately 3 min of computer time is required for simulation of 1.2 hours of real time).

APPLICATION OF THE REVISED COMPUTER MODEL TO A HIGHWAY REDESIGN PROBLEM

The problem outlined in the introduction suggested that there was a lack of satisfactory methodology to calculate the overall traffic flow consequences resulting from spot design improvements on two-lane, two-way rural highways. One of the nine field sites at which traffic lane input, output, and throughput data had been collected was utilized for the purpose of indicating the manner in which the computer model can be used to estimate the overall consequences of spot highway improvements, using computer simulation of traffic flow.

Description of Field Site

The site selected is a 5.3-mile section of US-64, southwest of Raleigh, that crosses the Haw River Valley. The maximum grade is 7 percent, and 46 percent of its length is zoned with no-passing restrictions. Figure 2 shows a schematic plan and profile for the sections with the location of the no-passing zone barriers. No-passing restrictions are identified by numbers marked with a suffix of (V) or (H). These labels designate whether the restriction is due to horizontal or vertical sight distance limitations. In the case of restriction 7, the limitation is due to both horizontal and vertical restrictions. Figure 3 shows photographs of the passing restrictions on this highway at locations 1(V), 3(H), the Haw River Bridge, and 7(H, V).

Spot Highway Improvements

The roadway designer, with access to detailed maps and records, can evaluate each of the no-passing locations shown in Figure 2. He can then prepare engineering estimates outlining the extent of spot improvements and their respective costs. Table 2 gives the design improvements used in this example problem. However, it should be noted that many other alternatives can be generated for each of these restricted passing locations.

The selection of specific spot improvements to be utilized in simulation is at the discretion of the roadway designer. For the purpose of this example problem, it is assumed that all of the redesign alternatives given in Table 3 are feasible, not only from an engineering point of view but also from the standpoint of cost.

Column 2 in Table 3 lists the redesign alternatives by noting the specific no-passing zone restrictions removed. As these restrictions are removed, the level of service and the construction costs both increase. Thus, simulation sequences 16 and 17 would be expected to provide the highest level of service and, at the same time, would cost the largest amount of money to implement. Column 8 in Table 3 indicates the mean travel time for the base condition with the travel time for each of the 17 alternatives. Column 9 shows the increase in overall mean speed as the magnitude of the spot improvements increases. In column 11, the value of t denotes whether there is any significant statistical difference between travel time for a particular simulation run and that for the base condition without any passing restriction removed. Table 4 gives additional throughput data for delay, passing, and speed change cycles. The data in both tables indicate a regular change in simulation output as the magnitude of the spot improvements increases. Although not shown here, the other five volume levels used for

Figure 3. Examples of horizontal and vertical passing sight distance restrictions on US-64.

view looking east—restriction 1(V)

view looking west—restriction 3(H)

view looking west—Haw River Bridge

view looking west—restrictions 7(H, V)

Table 2. Explanation of original and modified no-passing zone barriers for field site 3-69.

No-Passing Zone Barrier	Reason for Restriction	Engineering Basis for Modification
1(V)	Crest vertical curve	Barrier attributable to crest vertical curve, only a part of which is included in the actual field site selected for simulation; therefore, no modification or removal implemented.
2(H)	Limited visibility due to horizontal curvature	Horizontal curvature flattened to provide necessary passing sight distance for 70-mph design speed.
3(H)	Limited visibility due to horizontal curvature	Horizontal curvature flattened to provide necessary passing sight distance for 70-mph design speed.
4(V) 5(V)	Crest vertical curve and approach to an intersection	(These two vertical curves were treated together in removing the no-passing barriers because they are within 1,000 ft of each other.) Removal accomplished by replacing existing grades with flatter ones (1 and 2 percent) and placing a single crest vertical curve between the new tangents; the 500-ft no-passing barrier established in both directions because of the intersection left intact and not modified.
6(H)	Limited visibility due to horizontal curvature	Horizontal curvature flattened to provide necessary passing sight distance for 70-mph design speed.
7(H,V)	Limited visibility due to a horizontal curve, a crest vertical curve, and a narrow bridge	Existing steep 6 and 7 percent grades replaced with a uniform flatter grade (3 percent); a single crest vertical curve introduced to provide passing sight distance for 70-mph design speed.

Table 3. Summary of traffic data resulting from computer simulation of selected changes in no-passing barriers on rural two-lane highway with input volume of 226 vph.

		No-Passing Restrictions (percent)			Input Data		Throughput Data			
Simulation Sequence Number (1)	No-Passing Restrictions Removed (2)	Actual (3)	Less Than 1,500 Ft (4)	Sight Distance Greater Than 1,500 Ft (percent) (5)	Mean Speed[a] (ft/sec) (6)	Standard Deviation (ft/sec) (7)	Mean Travel Time (sec) (8)	Overall Mean Speed (ft/sec) (9)	Throughput Volume (vph) (10)	t-Value (11)
0	None-base condition	46.28	68.46	31.54	85.27	9.5	340.77	81.09	230	—
1	6(H)	43.50	53.62	46.38	85.86	9.7	333.27	82.91	230	2.30[b]
2	2(H)	42.51	62.30	37.70	86.01	9.7	337.70	81.82	230	0.89
3	7(V)	42.30	61.58	38.42	86.04	9.7	337.41	81.88	230	0.95
4	2(H), 6(H)	39.73	47.47	52.53	86.43	9.8	335.54	82.35	230	1.49
5	4(V), 5(V)	38.13	56.87	43.13	86.67	9.8	331.07	83.46	230	2.85[c]
6	3(H)	38.04	58.33	41.67	86.68	9.8	338.40	81.65	230	0.68
7	2(H), 3(H)	34.27	52.17	47.83	87.25	10.0	331.88	83.26	230	2.55
8	4(V), 5(V), 7(V)	34.15	50.00	50.00	87.26	10.0	330.81	83.53	230	2.86[c]
9	2(H), 3(H), 6(H)	31.49	37.33	72.67	87.66	10.1	329.62	83.83	230	3.36[c]
10	7(H)	30.24	53.25	46.75	87.85	10.1	331.77	83.28	230	2.71[c]
11	6(H), 7(H)	27.46	38.42	61.58	88.27	10.2	328.66	84.07	228	3.85[c]
12	7(H), 7(V)	26.26	46.38	53.62	88.44	10.2	328.58	84.09	228	3.60[c]
13	2(H), 3(H), 4(V), 5(V)	26.12	40.59	59.41	88.46	10.2	324.46	85.16	230	4.68[c]
14	2(H), 3(H), 6(H), 7(H)	15.45	22.13	77.87	90.06	10.6	320.49	86.22	231	5.67[c]
15	4(V), 5(V), 6(H), 7(H), 7(V)	15.32	22.67	77.33	90.08	10.6	321.58	85.92	232	5.64[c]
16	2(H), 3(H), 4(V), 5(V), 7(V), 7(H)	6.10	18.52	81.48	91.42	10.9	310.34	89.04	232	8.97[c]
17	2(H), 3(H), 4(V), 5(V), 6(H), 7(H), 7(V)	3.48	6.39	93.61	91.84	10.9	312.86	88.32	231	8.26[c]

[a]Mean input speed calculated from regression model: U = -0.65335 + 1.05718 x (posted speed) - 0.14964 x (percentage of no-passing zone).
[b]Denotes significant difference at 95 percent level of significance.
[c]Denotes significant difference at 99 percent level of significance.

Table 4. Summary of delay and passing data resulting from computer simulation of selected changes in no-passing barriers.

				Throughput Data Per Hour Per Mile of Highway				
Simulation Sequence Number (1)	No-Passing Restrictions Removed (2)	No-Passing Zone (percent) (3)	Mean Speed (ft/sec) (4)	Delay (sec) (5)	Number of Attempted Passes (6)	Number of Completed Passes (7)	Number of Cars Passed in Multiple Passes (8)	Number of 1-mph Speed Change Cycles (9)
0	None-base condition	46.28	85.27	813.08	21.21	19.10	1.34	5,908
1	6(H)	43.50	85.86	867.35	24.84	23.31	3.65	5,081
2	2(H)	42.51	86.01	800.47	21.98	19.87	1.34	3,751
3	7(V)	42.30	86.04	789.77	19.68	18.34	1.91	3,967
4	2(H), 6(H)	39.73	86.43	781.17	26.18	22.93	2.68	5,803
5	4(V), 5(V)	38.13	86.67	616.46	21.78	19.68	1.15	3,925
6	3(H)	38.04	86.68	707.80	25.22	22.55	1.72	5,899
7	2(H), 3(H)	34.27	87.25	745.44	27.52	22.16	1.34	5,318
8	4(V), 5(V), 7(V)	34.15	87.26	693.46	26.56	23.12	1.72	6,747
9	2(H), 3(H), 6(H)	31.49	87.66	711.04	29.43	23.88	2.87	5,977
10	7(H)	30.24	87.85	816.14	27.90	25.41	2.68	2,910
11	6(H), 7(H)	27.46	88.27	715.44	32.29	29.81	2.48	5,526
12	7(H), 7(V)	26.26	88.44	750.98	25.03	23.12	1.91	3,871
13	2(H), 3(H), 4(V), 5(V)	26.12	88.46	571.54	31.72	29.42	2.29	5,455
14	2(H), 3(H), 6(H), 7(H)	15.45	90.06	638.81	36.69	31.91	3.44	5,524
15	4(V), 5(V), 6(H), 7(H), 7(V)	15.32	90.08	682.19	24.65	21.98	2.29	3,271
16	2(H), 3(H), 4(V), 5(V), 7(V), 7(H)	6.10	91.42	384.85	46.05	34.21	2.10	4,699
17	2(H), 3(H), 4(V), 5(V), 6(H), 7(H), 7(V)	3.48	91.84	566.19	31.53	24.08	2.10	3,975

simulation also resulted in regular changes in the output data relative to input changes in the percentage of no-passing zones.

The underlying relationships between the dependent simulation output variables—speed, travel time, delay, passing, and speed change cycles—and the independent input variables—percentage of no-passing zone and traffic volume—are difficult to discern by eye. Therefore, the next step is to develop regression equations relating dependent and independent variables. Finally, the regression relationships can be expressed in a graphical form that is easier to use.

Correlating No-Passing Restrictions and Vehicular Volume Levels With Throughput Traffic Data

In Table 4, a regression equation was developed for the independent variable in column 5 and the dependent variable in column 9. A second independent variable consisting of input traffic volume over the range of 200 to 1,200 vph was also included in the regression equation.

Five additional regression equations were developed by using the independent input variables of traffic volume and percentage of no-passing zone and the dependent variables of delay, number of completed passes, number of attempted passes, number of cars passed in multiple passes, and number of 1-mph speed change cycles. Table 4, columns 5 through 9, shows the dependent variables resulting from simulation for a volume level of 226 vph. These five regression equations were also developed by using simulation output data for input volume levels of 200 to 1,200 vph.

The regression equations are easier to interpret and use if they are expressed in graphical form. Figures 4 through 7 show the graphical equivalents for four of the six regression equations developed. Figure 4 shows speed-volume-sight distance relationships for the 5.3-mile highway section in the example problem. For the roadway designer, this is the most important graph of the set, for it represents the trade-off between speed and volume for any given level of sight distance greater than 1,500 ft. It also represents the trade-offs between sight distance and volume and sight distance and speed. Figure 5 shows a measure of delay as it is related to three input volume levels and the percentage of no-passing zone restriction. Figure 6 shows the number of completed passes per mile per hour, and Figure 7 shows the number of 1-mph speed change cycles per mile per hour, with both statistics also related to the same three input volume levels and percentage of no-passing zone.

Use of the Graphical Relationships

To illustrate the application of Figures 4 through 7 to the example problem, let it be assumed that the 30th highest hourly volume on this 5.3-mile section of highway is 600 vph. The base condition sight distance greater than 1,500 ft is 31.5 percent, as given in Table 4, column 4. Let it be further assumed that planning studies indicate that the 30th highest hourly volume will increase to approximately 800 vph over the next 5 years. Some construction funds may be available for spot improvements, but no money will be available for extensive reconstruction. It is also assumed that roadway designers would like to maintain the existing overall operating speed at its present level or better as the traffic volume increases over the 5-year period. Two questions arise: What spot improvements will satisfy these problem specifications, and what overall benefits accrue in relation to construction costs?

In Figure 4, point a on the graph is the existing situation at 600 vph, with a mean speed of 52 mph and a sight distance greater than 1,500 ft of 31.5 percent. The intersection at point c of a horizontal line through 52 mph and a vertical line through 800 vph yields the necessary sight distance percentage to satisfy the problem requirements. This value is approximately 52 percent. Point b indicates that, after the spot improvements have been completed, the mean speed will rise to approximately 54 mph, at a volume level of 600 vph. However, the mean speed will decrease over the 5-year period as the throughput volume increases from 600 to 800 vph, with the speed decrease following the 52 percent sight distance line b-c.

Column 5 in Table 4 indicates that simulation sequences 4 and 9 and 11 through 17

Figure 4. Relationship between average overall travel speed and input volume for various sight distances.

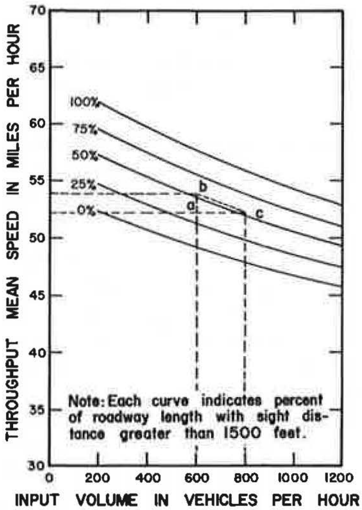


Figure 5. Least square curves relating vehicular volume and percentage of no-passing zone to vehicle delay.

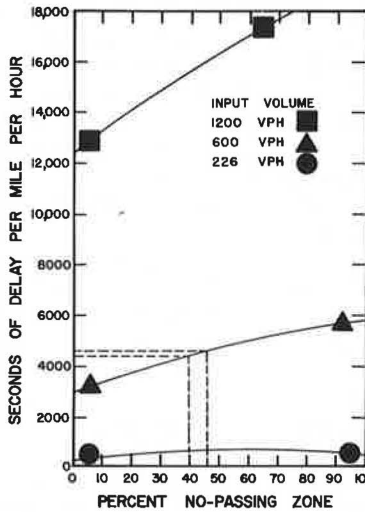


Figure 6. Least square curves relating vehicular volume and percentage of no-passing zone to number of completed passes.

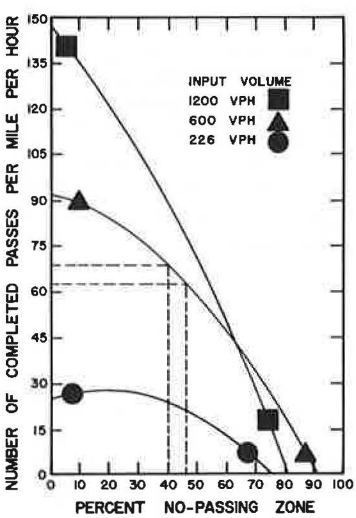


Figure 7. Least square curves relating vehicular volume and percentage of no-passing zone to number of 1-mph speed change cycles.

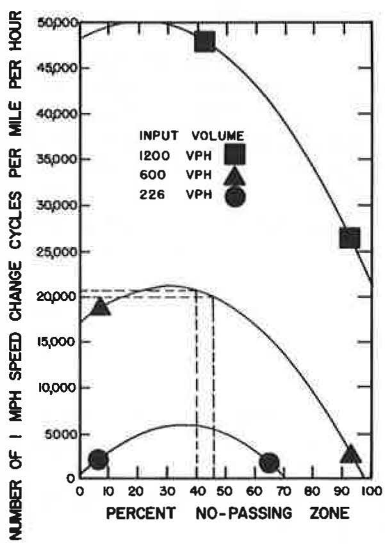


Table 5. Computer simulation time required for 1.2 hours of real time.

Volume (vph)	Computer Time (min)		
	IBM 370/165	IBM 370/145	IBM 360/40
200	1	12.5	50
600	3	37.5	150
1,200	7	87.5	350

will all satisfy the sight distance specification. Let it be assumed that sequence 4 is selected for additional investigation because it has the lowest construction cost. Let it be also assumed that the roadway designer desires to amortize the construction costs for sequence 4 over the 5-year period.

Column 3 in Table 4 indicates that, if the spot improvements in sequence 4 are implemented, the percentage of sight distance restriction will drop from 46 percent to approximately 40 percent. If each of the 1-hour volumes that compose the 24-hour ADT is used, the reduction in delay per hour per mile can be obtained by using the regression equation plotted in Figure 5 but for the appropriate volume level. The sum of the 24-hour delay per mile multiplied by the 5.3-mile length of the section and then by the number of days in the year will yield the total yearly savings in delay. Assuming that the ADT can be estimated for each year in the 5-year period, the total savings in delay can be calculated and then converted to a dollar value. To illustrate the use of Figure 5, assume that one of the 24 hourly volumes in the ADT is 226 vph. The reduction in delay in moving from 46 percent sight distance restriction to 40 percent is approximately 100 sec/hour/mile. For an hourly volume of 600 vph, the savings in delay read from Figure 5 is approximately 300 sec/hour/mile.

Figure 6 can be used to estimate the number of additional completed passes associated with the reduction in percentage of no-passing zone from 46 to 40. For a volume of 600 vph, Figure 6 shows that the number of completed passes will increase from 64 to 69/hour/mile. Increases in passing can be estimated for other volumes in a similar manner.

The additional completed passes noted in Figure 6 for a volume level of 600 vph and a reduction in no-passing zone percentage from 46 to 40 will cause an increase in speed change cycles. The increase read in Figure 7 is approximately 850 one-mph speed change cycles. This increase can be converted to dollars of additional operating costs for the 5-year period in a manner similar to calculating the savings in delay and should be offset against the dollars of savings for delay.

Computer Processing Time

For a 5-mile section of two-lane, two-way roadway, the computer simulation time for 1.2 hours of real time is related to throughput volume as given in Table 5.

In reference to Figures 4 through 7, the data used to plot the curves were generated by using six input volume levels—200, 400, 600, 800, 1,000, and 1,200 vph. In Table 5, the redesign alternatives selected for simulation included the base condition, simulation sequence 0, sequence 17, and three intermediate alternatives. Thus, the data necessary to analyze the example problem can be generated in 30 computer runs. The total computer processing time will depend on the hardware available.

CONCLUSIONS

From the results and findings of the investigation, the following conclusions were drawn:

1. Based on statistical comparisons of throughput data from nine field sites with throughput data resulting from computer simulation of these nine sites, the revised computer simulation model can be calibrated to produce simulation throughput for traffic volumes, mean speed and its associated standard deviation, mean travel time, travel time distribution, and number of completed passes that match the same field throughput values, over volume ranges of 175 to 650 vph.
2. Based on computer simulation and input traffic volume levels of 175 to 1,200 vph, statistically reliable quantitative relationships exist between the two independent variables, percentage of highway marked with no-passing zone barriers and input traffic volume, and the dependent throughput variables, mean speed, delay, attempted passes, completed passes, number of passed cars in multiple passes, and number of 1-mph speed change cycles.
3. Over the input volume level of 175 to 1,200 vph, on the average, the input traffic volume is equal to the throughput volume.

4. The revised computer model, consisting of the NCSU modified model and the speed-headway program, can be employed to develop data useful to highway designers desiring to evaluate improvements in passing sight distance on two-lane, two-way rural highways.

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REFERENCES

1. A Policy on the Geometric Design of Rural Highways. American Association of State Highway Officials, 1966.
2. Andreassend, D. C. The Use of Mathematical Description of Vehicle Performance to Determine Overtaking Times and Distances. Proc., Third Conf., Australian Road Research Board, Sydney, Vol. 3, Pt. 1, 1966, pp. 518-538.
3. Firey, J. C., and Peterson, E. W. An Analysis of Speed Changes for Large Transport Trucks. HRB Bull. 334, 1962, pp. 1-26.
4. Grecco, W. G., and Sword, E. C. Prediction of Parameters for Schuhl's Headway Distribution. Traffic Engineering, Vol. 38, No. 5, Feb. 1968, pp. 36-49.
5. Janoff, M. S., and Cassel, A. Identification and Evaluation of Remedial Aid Systems for Passing Maneuvers on Two-Lane Rural Roads: Volume IV—Traffic Flow Model. Federal Highway Administration, Res. and Development Rept., 1970.
6. Schuhl, A. The Probability Theory Applied to Distribution of Vehicles on Two-Lane Highways. In Poisson and Traffic, Eno Foundation, Saugatuck, Conn., 1955.
7. Weaver, G. D., and Glennon, J. C. The Passing Maneuver as It Relates to Passing Sight Distance Standards. Texas Transportation Institute, Texas A&M Univ., 1969.