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Traffic Control Devices and Rail-Highway Crossings

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Contents

The Effects of Wide Edge Lines on Lateral Placement and Speed on Two-Lane Rural Roads

BENJAMIN H. COTTRELL, Jr.

ABSTRACT

The results of an evaluation of effect of edge lines 4 in. and 8 in. wide on the lateral placement and speeds of vehicles on two-lane rural roads are presented. Data were collected at 12 locations on sections of roadway covering 55.2 mi. It was concluded from analyses of variance of lateral placement, lateral placement variance, encroachments by automobiles and trucks, mean speed, and speed variance that, overall, there were no statistically significant differences between the 4-in. and 8-in. wide edge lines. The mean lateral placement was significantly lower for the 8-in. line. However, changes in lateral placement and speed were not significant from a practical viewpoint.

There are a high number of run-off-the-road (ROR), drunken driving, and night accidents in rural areas. In 1980 there were 18,792 ROR accidents in rural areas in Virginia (1). Of this total, 269, or 1.4 percent, were fatal accidents; 8,367, or 44.6 percent, injury accidents; and 10,417, or 54.0 percent, property-damage accidents. ROR accidents accounted for 31.9 percent of all rural accidents, 38.5 percent of the fatalities in rural accidents (the largest percentage for any type of accident), and 35.1 percent of the persons injured in rural accidents. Drinking drivers--persons driving under the influence of alcohol (DUI)--were involved in 12,025, or 20.4 percent, of all rural accidents. Accidents involving DUI accounted for 31.7 percent of fatal accidents, 27.1 percent of personal injury accidents, and 16.3 percent of property-damage accidents in rural areas. There were 25,621 accidents during nighttime, which constituted 43.5 percent of all accidents in rural areas.

To provide guidance to motorists on two-lane rural roads, edge lines are used to delineate the right edge of the roadway. The edge line is one element in a pavement marking system that provides warning and guidance information to the driver without diverting his attention from the roadway (2). Reflectorized pavement markings are the most common form of delineation at night when the reduced visibility creates a greater need for guidance information.

Two research studies conducted on controlled test sections have concluded that 6-in. and 8-in. wide edge lines have an impact on the lateral placement of vehicles, especially those driven by alcohol-affected persons (3,4). Edge lines 8 in. wide have the potential to reduce the probability that a driver will run off the road and increase the probability that he will position his vehicle close to the center line. However, because wide edge lines have the potential to influence the lateral position of the vehicle in this manner, the probability of center-line encroachment may increase. No information was available on the impact of wide edge lines on lateral placement and speed under road conditions.

Virginia Highway and Transportation Research Council, Box 3817 University Station, Charlottesville, Va. 22903-0817.

OBJECTIVE AND SCOPE

The objective of this research was to evaluate the effect of edge lines on the lateral placement and speed of vehicles.

The scope was limited to two-lane rural roads. Primary routes were selected because accident data are more detailed and more readily available for them than for secondary routes.

The second phase of this research will address accidents.

STUDY DESIGN

The experimental plan for evaluating wide edge lines was a before-and-after study. Field data were collected for a before period with standard-width (4-in.) edge lines and for an after period following the installation of 8-in. wide edge lines. It was assumed that any differences in the measures of performance, lateral placement, and speed between the before and the after periods would be attributable to the wide edge lines. The primary measure of performance was lateral placement. The before-and-after data were collected during the fall of 1983 and 1984, respectively, for 8 of the 12 sites. The before-and-after data were collected during the spring and fall of 1984, respectively, for the remaining four sites. However, the traffic volumes at the sites are not dependent on the season. Data on lateral placement and speed were collected for a 24-hr period at the study sites by using a Leupold and Stevens traffic data recorder (TDR), described as follows.

Use of TDR

The configuration for collecting lateral placement and speed data with the Leupold and Stevens TDR is shown in Figure 1. The speed detector consisted of two sensor cables placed perpendicular to the edge line and 6 ft apart (Channel A). The position detector consisted of two sensor cables placed 6 ft apart at the edge of the pavement, but the trailing cable was laid at an angle other than 90 degrees to the edge of the pavement. A typical angle for the trailing detector was 45 degrees (Channel B). Vinyl tape was used to secure the sensor cables to the pavement.

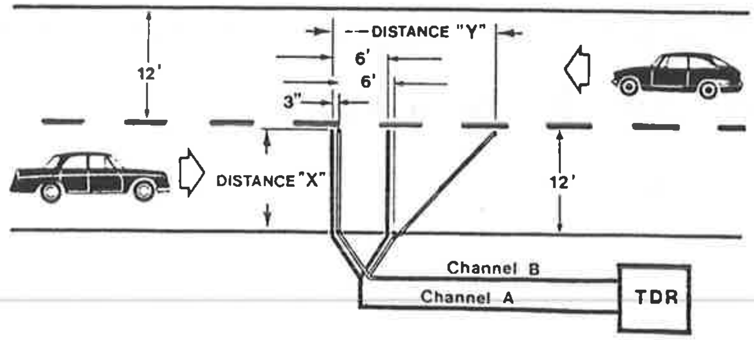

FIGURE 1 Configuration for collection of lateral placement data.

Traffic data were recorded on a magnetic cassette tape that was brought in from the field, read, and filed on a computer. The raw data were printed and screened for recording errors. Summary data on lateral placement and speed such as the mean, standard deviation, and frequency distribution were printed by using TDR report generator programs and programs developed at the Virginia Highway and Transportation Research Council.

Study Sections

Two sections of roadway, 36.3 mi and 18.9 mi long, were selected for the study. In the selection of those sections, the accident data on 11 road sections with high accident experiences were reviewed and these 2 were ranked first and second for the percentage of ROR accidents and alcohol- or drug-related accidents.

Study Sites for Field Data Collection

A sampling method based on the following criteria was developed to select sites for field data collection along the study sections.

1. Ideally, study sites should be located at 5-mi intervals along the study section (intervals of 3 to 7 mi were acceptable).
2. The direction of travel of the traffic volume to be studied should alternate (e.g., northbound, southbound, northbound).
3. The posted speed limit should be 55 mph.
4. The sites should be representative of the overall geometrics of the roadway (e.g., for a road section with many horizontal curves, the sites should be at curves).
5. Interference from intersections and driveways should be avoided.
6. The total sample should include left and right horizontal curves and tangent sections.
7. For curves the study site should be located midway between the beginning and middle of the curve.
8. A convenient parking area should be available for the vehicle transporting the data collection equipment.

With these criteria, 12 study sites were selected. Descriptive data on these sites are shown in Table 1. It is noted that some of the edge lines intended to be 8 in. wide were not.

ANALYSIS OF LATERAL PLACEMENT AND SPEED

The analysis of the lateral placement and speed data was performed for individual sites and for all sites. The objective was to determine whether there were any significant differences in lateral placement or speed for the 4-in. line as compared with the 8-in. line. The measures of performance and statistical tests are discussed in the following paragraphs.

Comparison of the Variances of Lateral Placement

For each site, the variances of the lateral placement of 4- and 8-in. wide edge lines were compared by using an F-test under the hypothesis that the variances are equal (6). The underlying populations were

TABLE 1 Data on the Study Sites

Location and Site No.	Direction of Travel	Geometrics	Lane Width[a] (ft)	24-Hr Traffic Count	Width of Edge Line (in.)
Route 20, Albemarle County					
1	Southbound	Left curve 6 degrees	10.92	2,307	7.0
2	Northbound	Left curve 11 degrees	9.58	1,982	7.5
3	Southbound	Straight	11.00	1,420	8.0
4	Northbound	Left curve 10 degrees	8.92	1,379	7.0
Route 20, Buckingham County					
5	Southbound	Right curve 5 degrees	9.00	911	7.0
6	Northbound	Straight	8.71	631	8.0
7	Southbound	Straight	8.79	534	7.8
8	Northbound	Straight	9.00	1,028	7.5
Route 501, Bedford County					
9	Southbound	Left curve 10 degrees	9.75	688	10.0
10	Northbound	Straight	8.46	559	10.0
11	Southbound	Right curve 7 degrees	8.17	390	10.0
Route 501, Rockbridge County					
12	Northbound	Left curve 3 degrees	9.83	1,482	7.0

[a]Inside lane markings.

assumed to be normally distributed. The alternative hypothesis is that the variance of the 8-in. wide edge line is either greater or less than the variance of the 4-in. line. A significance level of 0.05 was used.

Research by Stimpson et al. concluded that longitudinal change in lateral placement variance is one of the two most sensitive indicators of hazard (7), and Taylor et al. noted a strong correlation between the lateral placement variance and accident experience (8). In other words, the higher the variance in the lateral placement, the higher the hazard potential and number of accidents. Consequently, it was concluded that the lower the variance in lateral placement, the better the edge line performs.

The results of the F-test are shown in Table 2. For the day and night periods, 7 (58.3 percent) and 9 (75.0 percent), respectively, of the 12 sites showed no significant difference in the lateral placement variance.

TABLE 2 Comparison of Lateral Placement Variances

	Preferred Lateral Placement					
	Day			Night		
Site No.	4 in.	8 in.	No Difference	4 in.	8 in.	No Difference
1			X	X		
2	X			X		
3	X					X
4	X					X
5			X			X
6		X				X
7			X	X		
8			X			X
9	X					X
10			X			X
11			X			X
12			X			X
Total	4	1	7	3	0	9
Percent	33.3	8.3	58.4	25.0	0	75.0

The Wilcoxon matched-pairs signed rank test was employed to determine whether the variance in lateral placement variance was significantly different for the 4- and 8-in. wide edge lines for all 12 sites combined. This is a two-sample, nonparametric test (no assumptions are made on the distribution of the variances) for comparing two populations (4- and 8-in. wide lines) on the basis of a paired sample (4- and 8-in. lines lateral placement variance measure at a site) (6). For the two time periods, it was concluded that there was no significant difference in the variance of the lateral placement for 4- and 8-in. lines at a 0.05 level of significance.

Comparison of Means of Lateral Placement

The means of the lateral placement for each site were compared with a t-test under the hypothesis that the mean lateral placements of the two edge lines are equal. A significance level of 0.05 was used.

It is noted that good or preferred lateral placement is controversial. Research by Johnson, as well as others, has concluded that a corner-cutting strategy is used on curves (4). Other researchers have recommended driving in the center of the lane (3,7,8).

In a telephone conversation, one of the three driver education supervisors for the Commonwealth of Virginia Department of Motor Vehicles stated that the Department's policy on driver position in the lane is as follows:

1. The center of the lane is the predominantly recommended driver position in Virginia;
2. On left curves, drivers should stay to the left when there is no opposing traffic, to avoid gravel near the shoulder, which may cause skidding; otherwise, they should drive in the center of the lane; and
3. On right curves, they should always drive in the center of the lane.

Gravel near the shoulder did not appear to be a problem at the study sites. Therefore, in general, good lateral placement was considered to be synonymous with driving in the center of the lane. The preferred edge-line width is the one that results in a mean lateral placement closest to the center of the lane. For all sites, the mean lateral placements of both edge-line widths indicate that motorists tend to drive to the left of the center of the lane. In other words, the mean lateral placement was greater than the preferred placement. Consequently, the lower mean lateral placement was preferred.

As can be seen in Table 3, for 11 (91.7 percent) and 7 (58.4 percent) of the 12 sites, the mean lateral placement for the 8-in. wide edge line was significantly less than the mean for the 4-in. line for the day and night periods, respectively.

Similarly, a one-tailed, paired t-test of the 12 sites revealed that the mean lateral placement for the 4-in. wide edge line was significantly greater at a level of significance of 0.005 for the day period and of 0.05 for the night period. Therefore, from a statistical standpoint, the 8-in. wide line results in significantly better lateral placement than does the 4-in. line.

TABLE 3 Comparison of the Mean Lateral Placements

	Preferred Mean Lateral Placement					
	Day			Night		
Site No.	4 in.	8 in.	No Difference	4 in.	8 in.	No Difference
1		X			X	
2		X			X	
3		X			X	
4		X				X
5		X				X
6		X				X
7		X			X	
8		X				X
9	X					X
10		X			X	
11		X			X	
12		X			X	
Total	1	11	0	0	7	5
Percent	8.3	91.7	0	0	58.4	41.6

Encroachments on Opposing Lane

Encroachments on the opposing lane were compared by using a chi-square test under the hypothesis of independence of edge-line width and encroachments. A significance level of 0.05 was used. The alternative hypothesis is that the edge-line width causing the lower percentage of encroachments is preferred.

Encroachments were measured by using a lateral placement zone system consisting of 10 zones, in which each zone was 10 in. wide. The zones of encroachment are the zone in which the average vehicle would be crossing the center line and all zones to

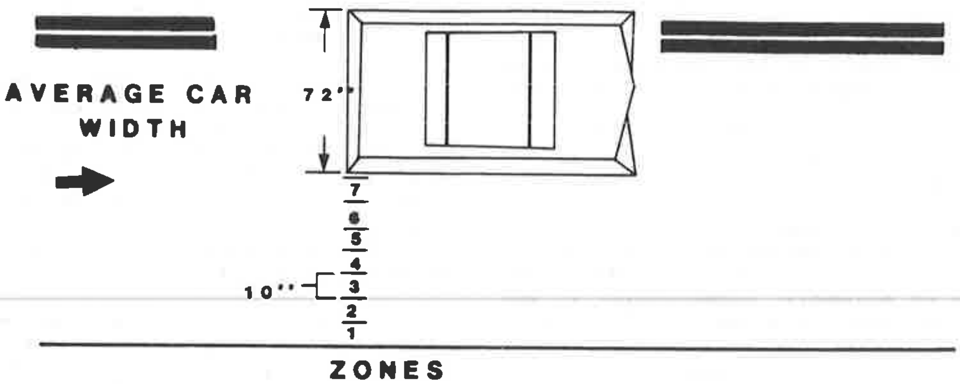

FIGURE 2 Zones of encroachment.

the left of this zone, as shown in Figure 2. The average widths of 6 and 8 ft were used for automobiles and trucks, respectively. Data from Consumer Reports show that the widths of 1984 model automobiles range from a 63.8-in. mean for small automobiles to a mean of 78.6 for large automobiles (9). The mean for medium automobiles is 70.8 in. Because data were not available on the distribution of automobile ownership by automobile size, the medium automobile was selected as the average vehicle and the average vehicle width of 72 in. was used. The American Association of State Highway and Transportation Officials design lengths for automobiles and trucks are 7.0 and 8.5 ft, respectively (10). An average truck length of 8 ft was selected, because the design vehicles are larger than the actual vehicles.

The zones of encroachment were determined as follows: (a) the average vehicle width was subtracted from the lane width to determine the minimum lateral placement for encroachment; (b) the associated zone was identified; and (c) if this position was in the half of the zone closest to the edge line, this zone and all higher zones represented the zones of encroachment; otherwise, all higher zones represented the zone of encroachment.

The encroachment results are shown in Table 4. For both time periods and for both automobiles and trucks, neither edge line appeared to perform consistently better than the other. This is supported by the Wilcoxon matched-pairs signed rank tests, which concluded that there were no significant differences in the encroachments for the two edge-line widths for both time periods between automobiles and trucks, with one exception. For trucks at night, the encroachments were significantly greater for 4-in. wide edge lines.

Distribution of the Lateral Placement of Vehicles by Zones

An example of the distribution of the lateral placement of automobiles by zones is shown in Figure 3 for Site 1 for the total period. In general, there were no noticeable changes in the position or range of lateral placements. These data are consistent with the earlier findings on the means and variance of lateral placement.

Comparison of the Variances in Speed

The variances in speed for the 4- and 8-in. wide edge lines were compared by using an F-test under the hypothesis that the variances are equal. The underlying populations were assumed normally distributed and a level of significance of 0.05 was used. The preferred speed variance was the lower one, because uniform driving tends to promote safety (4,9). In Table 5, for the day and night periods, the data show that the variance in speed showed no significant difference for 5 (41.7 percent) and 9 (75.0 percent) of the 12 sites, respectively.

TABLE 4 Comparison of Encroachments on Opposing Lane

	Preferred Encroachment											
	Day						Night					
	Automobiles			Trucks			Automobiles			Trucks		
Site No.	4 in.	8 in.	No Difference	4 in.	8 in.	No Difference	4 in.	8 in.	No Difference	4 in.	8 in.	No Difference
1		X		X					X			X
2		X			X				X			X
3		X				X		X				X
4	X					X	X					X
5	X				X		X					X
6		X				X			X			X
7		X				X			X			X
8	X					X			X			X
9	X				X				X		X	
10		X		X					X			X
11	X					X	X					X
12	X				X		X				X	
Total	6	6	0	2	4	6	4	1	7	0	2	10
Percent	50.0	50.0	0	16.7	33.3	50.0	33.3	8.3	58.4	0	16.7	83.5

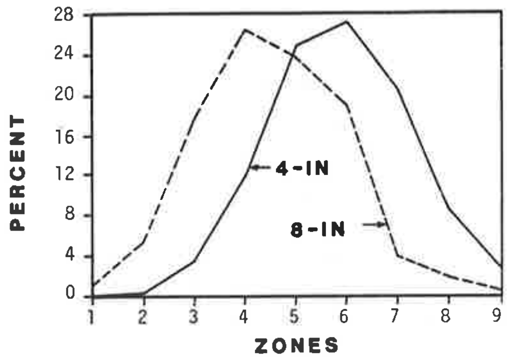

FIGURE 3 Lateral placement of cars by zones for the total period.

TABLE 5 Comparison of the Variances of Speed

	Statistically Lower Speed Variance					
	Day			Night		
Site No.	4 in.	8 in.	No Difference	4 in.	8 in.	No Difference
1		X		X		
2	X					X
3			X			X
4		X				X
5	X					X
6	X					X
7			X			X
8		X		X		
9	X					X
10			X			X
11			X			X
12			X	X		
Total	4	3	5	3	0	9
Percent	33.3	25.0	41.7	25.0	0.0	75.0

Use of the Wilcoxon matched-pairs signed rank test showed that there was no significant difference for the day and night periods.

Comparison of the Mean Speeds

The mean speeds were compared at each site by using the t-test under the hypothesis that the mean speeds are equal at a 0.05 significance level. The preferred speed was the lower one.

As shown in Table 6, for the day and night periods, 8 (66.7 percent) and 11 (91.7 percent), respectively, of the 12 sites showed no significant differences.

Similarly, the paired t-tests for all 12 sites combined concluded that at a 0.05 level of significance, the mean speeds of the 4- and 8-in. wide edge lines were not significantly different for either time period.

Summary of the Statistical Analyses

A summary of the findings from all the statistical tests is shown in Table 7. The lateral placement mean indicates a statistically better performance by the 8-in. wide edge line for both time periods. The superior performance in the Wilcoxon matched-pairs signed rank test for truck encroachments at night results from large differences between the 4- and 8-in. lines for two sites for this measure. In the Wilcoxon matched-pairs signed rank tests, large differences between the matched pairs are ranked higher,

TABLE 6 Comparison of the Mean Speeds

	Statistically Lower Mean Speed					
	Day			Night		
Site No.	4 in.	8 in.	No Difference	4 in.	8 in.	No Difference
1			X			X
2		X				X
3		X				X
4			X			X
5			X			X
6		X				X
7		X				X
8			X		X	
9			X			X
10			X			X
11			X			X
12			X			X
Total	0	4	8	0	1	11
Percent	0.0	33.3	66.7	0.0	8.3	91.7

and consequently one or two sites with large differences between the matched pairs may result in statistically significant differences whereas the remaining sites exhibit little or no difference. These 2 sites, when compared with the other 10 sites, are exceptions that favor the 8-in. wide edge line. The variance in lateral placement, automobile encroachments, and mean speed are not statistically different for 4- and 8-in. lines for either time period.

Therefore, the lateral placement mean is the only measure of performance that shows a statistically significant difference between the 4- and 8-in. wide lines. The difference suggests that 8-in. wide edge lines are preferred.

The study sites were grouped by road geometrics and lane width to examine performance trends related to these factors. However, no significant relationships were observed.

Practical Significance of Differences Between Edge Lines

The statistical significance of differences between performance measures for the 4- and 8-in. wide edge lines must be examined for practical significance, because statistical significance does not necessarily reflect a practical significance. In other words, given that there is a statistically measurable effect, is the change effective in improving traffic safety and operations? This question will be thoroughly addressed in the accident analysis in the second phase of this research project. Because only the mean lateral placement consistently showed a statistically significant difference, the practical significance of lateral placement differences based on engineering judgment is discussed in the following paragraphs.

In Table 8, mean lateral placement data are given for the 12 sites. A lateral placement shift of 6 in. is practical. With a tire width of about 6 in., the tire path will not overlap with a lateral placement shift of 6 in. or more. Also, it is believed that a shift is visibly noticeable at 6 in. On the basis of this information, only Site 3 displayed a practically significant lateral placement shift. This is probably because Site 3 had the widest travel lane and a greater variation in lateral placement was therefore possible. Consequently, it was concluded that, overall, there was no practically significant shift in lateral placement.

No other measure was closely examined for practi-

TABLE 7 Summary of Analysis for All Sites

Measure of Performance	Preferred Line Width: Day 4 in.	Day 8 in.	Day No Difference	Night 4 in.	Night 8 in.	Night No Difference
Lateral placement variance	4(33.3)	1(8.3)	7(58.4)[a]	3(25.0)	0.(0.0)	9(75.0)[a]
Lateral placement mean	1(8.3)	11(91.7)[b]	0(0.0)	0(0.0)	7(58.4)[a]	5(41.6)
Encroachment						
Automobiles	6(50.0)	6(50.0)	0(0.0)[a]	4(33.3)	1(8.3)	7(58.4)[a]
Trucks	2(16.7)	4(33.3)	6(50.0)[a]	0(0.0)	2(16.7)[a]	10(83.3)
Speed variance	4(33.3)	3(25.0)	5(41.7)[a]	3(25.0)	0(0.0)	9(75.0)[a]
Mean speed	0(0.0)	4(33.3)	8(66.7)[b]	0(0.0)	1(8.3)	11(91.7)[a]

Note: Percentages are shown in parentheses.

[a]Results of the Wilcoxon matched-pairs signed rank test.
[b]Results of the paired t-test.

TABLE 8 Mean Lateral Placement Data

Site No.	Day[a] Mean (ft) 4 in.	Day Mean (ft) 8 in.	Day Difference (ft)[c]	Night[b] Mean (ft) 4 in.	Night Mean (ft) 8 in.	Night Difference (ft)[c]
1	4.29	3.94	0.35	4.93	4.65	0.28
2	3.33	2.84	0.49	4.06	4.02	0.04
3	3.54	2.93	0.61	4.31	3.66	0.65
4	2.74	2.66	0.08	3.44	3.47	-0.03
5	2.17	2.04	0.13	2.94	2.98	-0.04
6	2.17	2.03	0.14	2.33	2.49	-0.16
7	2.76	2.32	0.44	3.60	3.22	0.42
8	2.05	1.92	0.13	2.26	2.26	0.00
9	2.80	3.02	-0.22	3.52	3.75	-0.23
10	2.16	1.81	0.35	2.86	2.57	0.29
11	2.21	1.99	0.22	2.93	2.58	0.35
12	2.85	2.34	0.51	3.47	3.04	0.43

[a]Mean difference = 0.27, standard deviation = 0.23, range of difference = 0.08-0.61.
[b]Mean difference = 0.17, standard deviation = 0.27, range of difference = 0.0-0.65.
[c]Difference: 4-in. values minus 8-in. values.

cal significance, because there were no overall statistically significant differences.

CONCLUSIONS

The following conclusions were drawn from the data presented in this paper:

1. Overall, there were no statistically significant differences between the 4- and 8-in. wide edge lines from the analysis of variance of lateral placement, lateral placement variance, encroachments by automobiles and trucks, mean speed, and speed variance.
2. The mean lateral placement was significantly lower for the 8-in. wide edge line. However, the difference was of a small magnitude and of no practical significance.
3. Lateral placement and speed were not practically affected by a change from a 4-in. to an 8-in. wide edge line.

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Rumble Strips and Paint Stripes at a Rural Intersection

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ABSTRACT

A common cause of traffic accidents at low-volume rural intersections is failure by drivers on the minor approaches to stop or slow down sufficiently, as warranted. The current experimental field study compared the effectiveness of transverse paint stripes, such as those developed by the U.K. Transport and Road Research Laboratory, and similarly placed rumble strips in inducing drivers to reduce speed and stop at intersections. The experiment was conducted on the two minor approaches to the same four-way rural low-volume intersection. A geometrically converging pattern of 38 paint stripes, each 60 cm (2 ft) wide, were laid out over a distance of 270 m (886 ft) of one leg, and a similar pattern of rumble strips, 12 to 15 mm (1/2 to 5/8 in.) high, was laid on the opposite leg. A before-and-after and a crossover (after a year) experimental design were used. Speeds were monitored at eight points on each leg along 420 m leading to the intersection for a total of over 2,500 lead vehicles. The main results and conclusions are as follows: (a) paint stripes have only minor influence on driver behavior; (b) rumble strips lowered speeds by an average of 40 percent; (c) both treatments had a small positive effect on compliance rate; (d) with no pavement treatment, deceleration began at 150 m (492 ft) and peaked within the last 60 m (197 ft); (e) with rumble strips, most of the deceleration took place before the vehicle passed the first strip, followed by an additional deceleration within the last 60 m (197 ft); deceleration become uniform and moderate; (f) rumble-strip effects remained stable after a year; and (g) a 150-m (492-ft) treatment of 12-mm strips is long enough to produce the positive effects of rumble strips.

A common cause of traffic accidents at low-volume rural intersections is, apparently, driver failure to stop or yield on the minor approach legs of the intersections. It is generally believed that insufficient speed reduction during the approach phase plays a direct causal role in the generation of such accidents and in increasing their severity. Therefore, some of the measures for preventing accidents at such intersections, as well as at other critical locations such as sharp curves and highway work zones, are specifically designed to bring about vehicle speed reduction during the approach phase.

Slowing down increases the likelihood of compliance with right-of-way rules (1); it improves the margin of safety in critical situations by allowing for more time and longer distances in case of an emergency stop, and it mitigates accident severity by reducing the energy levels in the case of a collision.

During the last 25 years, several studies have reported the use of rumble strips and paint stripes to induce drivers to slow down or to exhibit otherwise appropriate behavior at intersections and other critical locations. Rumble strips or rumble areas were used as early as 1954 (2). They gained in popularity during the 1970s when hundreds of installations were implemented (3). Rumble strips come in so many forms and dimensions that it is doubtful whether they should all be grouped under the same heading. Nevertheless, most rumble-strip treatments share four basic features:

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1. They involve certain degrading of the roadway pavement surface smoothness.
2. The basic treatment element is either a groove in the pavement [about 12 mm (1/2 in.) deep x 100 mm (4 in.) wide] or a tacked-on strip of rough pavement material [10 to 20 mm (3/8 to 3/4 in.) high and 10 cm (4 in.) to many meters wide].
3. The basic elements are repeatedly placed as transverse strips across the roadway in a certain geometric pattern, starting some distance upstream and stopping some distance before the critical location.
4. The rumble treatment is assumed to provide drivers with visual, auditory, and tactile-vibratory stimulation, thus compelling them to be attentive to the demands of the situation.

If the physical properties of the strips are harsh enough, the noise, vibration, and bumpy ride associated with high speed may become too unpleasant to ignore, thus forcing drivers to slow down in order to reduce the stimulation.

Paint stripes for controlling driver speed at approaches to intersections or curves were also tried more than 20 years ago (4). Renewed interest in the idea can be attributed to studies by the U.K. Transport and Road Research Laboratory (TRRL) on the use of paint stripes at approaches to roundabouts (5-7). Paint stripes are usually narrow, 10 to 60 cm (4 in. to 2 ft), and are placed in a pattern of decreasing distance between stripes toward the critical location. Paint stripes, like rumble strips, have an obvious visual impact that may be informative to the driver and attract his attention. As paint stripes are considerably cheaper than rumble strips, it would be useful to know which device is more effective and under what conditions.

A review and analysis of U.S., European, and other experience with rumble strips and paint stripes (8) revealed large variations in physical and geometrical attributes of the treatments and in the criteria and methods of their application and evaluation. It was not possible to conclude that paint stripes, which appear to be a success in Britain, can effectively substitute for the more expensive and pavement-destructive rumble strips used elsewhere. Furthermore, other than the unsupported allusion to a "visual speed illusion" associated with a converging pattern of stripes, there is no evidence that any given pattern of stripes (or strips) is preferable over other, equally reasonable, patterns. Similarly, there is a lack of strong rationale or empirical justification for the distance of surface treatment, and intersection applications range anywhere from 90 to 450 m (295 to 1,476 ft).

It is clear that in order to derive engineering design criteria for paint stripes and rumble strips, a better understanding is required of their operating mechanism. This experimental field study evaluated paint stripes and similarly placed rumble strips under comparable conditions. Vehicle behavior was monitored and analyzed in order to determine vehicle speed, deceleration, and stopping behavior and, finally, to develop application criteria and design guidelines.

METHOD

Overall Experimental Design

The experiment was conducted on the two minor legs, controlled by a stop sign, to the same four-way low-volume rural intersection. A before-and-after and a crossover experimental design were used. The behavioral measures included stopping behavior, speed, and deceleration functions of each vehicle. In the "after" period, one approach was treated with paint stripes whereas the other was furnished with an identical geometric pattern of rumble strips. Data were collected 1 month and 1 year after treatment. Subsequently, the approach previously treated with paint stripes was treated with a shorter pattern of rumble strips and monitored again. A total of 2,500 vehicles were monitored by eight speed traps placed on each approach leg along 420 m (1,378 ft) leading to the intersection. Figure 1 shows the layout of the intersection, positions of the measuring traps, and the geometric pattern of the stripes and strips.

Site Description

The experimental site has the following features:

1. It is an intersection of a primary rural road with a secondary rural road controlled by stop signs.
2. The primary and secondary roads have similar roadway geometry standards, traffic volumes, and speeds: 7.0 m (23 ft) wide, 2.0-m (7-ft) soft shoulders, 2,100 to 2,000 annual average daily traffic (AADT), and an average speed of 75 km/hr (47 mph).
3. The two minor approaches appear similar from the driver's point of view.
4. Sight distance is limited on both approaches. Although warning signs and other cues to the approaching intersection are present at a distance of 400 to 500 m (1,312 to 1,640 ft), the crossing road is not in view until much closer to the intersection. Drivers could be misled into assuming that they are on the major road.
5. The intersection has had a consistent history of accidents attributable to nonyielding vehicles on the minor road. The characteristics of the site present the kind of problems that paint stripes and rumble strips are believed to help overcome. The similarity of the two approaches is essential, of

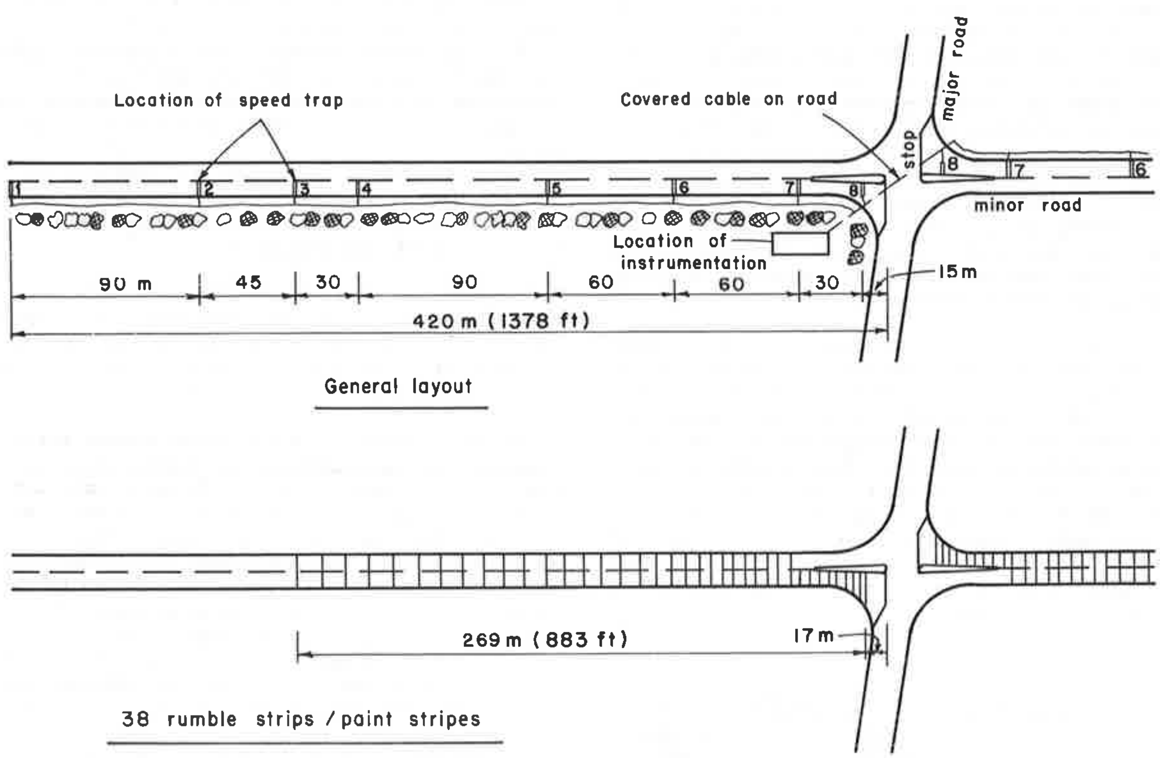

FIGURE 1 General view of intersection, layout of equipment, and arrangement of rumble strips or paint stripes.

course, for a valid comparison between the stripes and strips.

Description of Paint Stripes and Rumble Strips

A reasonable criterion for determining the distance of treatment application is to take the calculated stopping distance at a comfortable deceleration from a free approach speed to a full stop at the intersection. Preliminary speed measurements established that the 85th-percentile free-flowing speed at the site was about 80 km/hr (50 mph). Using the standard deceleration formula with a low deceleration value of 0.9 m/sec^2 (3 ft/sec^2), a treatment distance of $\approx$ 270 m (885 ft) was obtained ($s = v^2/2a$).

In order to facilitate comparisons with previous studies, a geometrically converging pattern of stripes was chosen, although there is no evidence of its hypothetical superiority. The converging pattern was designed so that a vehicle entering the treated zone at a speed of 80 km/hr and decelerating at a constant rate of 0.9 m/sec^2 would cross two stripes per second. In the case of rumble strips, this results in a 2-Hz rate of vibration. A total of 38 stripes (or strips) were installed, the first at 269 m (883 ft) upstream from the intersection, and the last one 17.4 m (55 ft) from the stop line. The last 17 m was left clear. In the third phase of the experiment, a modified, shorter pattern of strips was applied: 28 strips over 150 m (492 ft) up to 10 m (32.8 ft) from the stop line.

All stripes and strips were 60 cm (2 ft) wide. They were installed across the full width of the two-lane road except close to the intersection where the lanes are separated by a painted traffic island. The rumble strips were made of a premixed bituminous aggregate, size 3/8 to 1/2 in., hand rolled to a height of 12 mm (1/2 in.) and tapered at the edges. Both rumble strips and paint stripes were sprayed yellow with regular roadway marking paint mixed with reflective glass beads. Special advance warning signs were added to the normal sequence of signs on each approach.

Measurement System

Figure 1 shows the location of eight pairs of tapeswitches used to sense the passage of vehicles and calculate the speeds at these locations. Figure 2 shows the last measuring point and the other major components of the data collection system. Theoretical calculations of the different sources of errors and comparison with speed data obtained by radar showed that the accuracy of the measurement system is high. The maximal total error in measuring a 50-km/hr spot speed is less than 5 percent and decreases for lower speeds. The probability of a maximal error is less than 2 percent.

Data Collection Procedure

At all phases of the experiment, data were collected during four consecutive days, Monday through Thursday, from 8:30 a.m. to 6:00 p.m. Except for one day, weather was clear and dry. Traffic volumes were measured on the first day of each data collection period and were found stable: 2,000 ADT on the minor road and 2,100 on the major one. Right-turning vehicles made up about 10 percent of the volume. Data collection was carried out on both legs at the same time, with little intervention by the crew. The system's logic monitored a random sample of about equal numbers of free-flowing vehicles on each leg. Care was taken to conceal the measuring system and crew and to minimize the conspicuity of cables. The following variables were obtained for each observed vehicle:

1. Spot speeds at each of eight distances,
2. Decelerations between the speed traps,
3. Vehicle type,
4. Stopping behavior (not stopping, rolling, complete stop),
5. Direction through intersection, and
6. Presence of oncoming vehicles on major road.

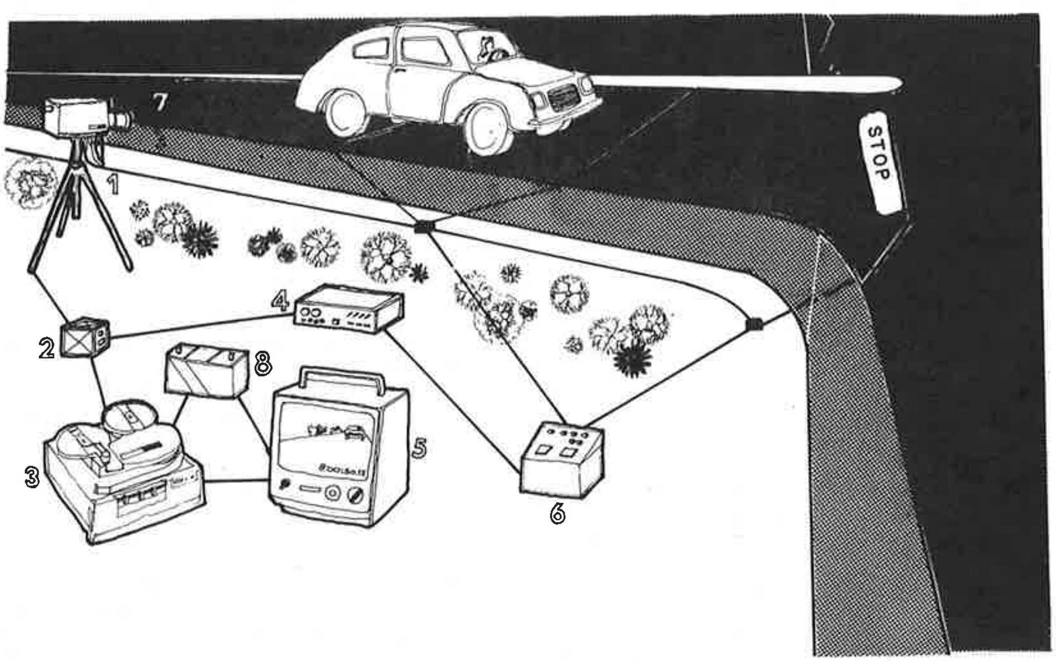

FIGURE 2 Arrangement of data collection system and its major components: 1, video camera; 2, synchronization box; 3, video tape recorder; 4, digital clock; 5, TV monitor; 6, control box; 7, tapeswitches; 8, power source.

TABLE 1 Average Speeds Before and After Painting Stripes and After a Modified Rumble-Strip Treatment

	Distance (m)							
Period	420	330	285	255	165	105	45	15
Paint stripes								
Before	77.8	76.4	74.5	73.0	66.2	57.0	43.7	25.9
After								
Speed	77.9	75.3	71.4	69.3	62.5	53.4	42.3	23.7
Percent change	1.0	1.5	5.0	5.0	5.6	6.3	3.2	8.4
Modified rumble strips								
Speed	77.7	71.4	68.3	66.2	53.9	38.7	27.4	17.0
Percent change	0	6.5	8.3	9.3	18.6	32.1	37.3	34.4

Note: 1 m = 3.3 ft; 1 km/hr = 0.6 mph. Speeds are given in kilometers per hour.

The interjudge reliability for the items that had to be detected visually from video was 97 percent and the errors appeared to be random.

RESULTS

Influence of Paint Stripes on Speed, Deceleration, and Stopping Behavior

The average speeds (of all vehicles) at the eight measuring points before and after painting stripes are given in Table 1. Figure 3 shows the 85th-percentile speeds.

It is evident that the free approach speeds just before the stripes are encountered, at 420 m and 330 m, have not changed. Afterwards the stripes produce a small, yet significant average speed reduction of about 3 km/hr and a smaller reduction close to the intersection. There were no significant changes in either average or maximum deceleration patterns. During both periods, average deceleration ranged from 0.20 m/sec^2 farthest from the intersection up to 1.6 m/sec^2 at the nearest distance measured [45 to 11 m (148 to 36 ft)]. Individual vehicles, however, differed considerably in their deceleration rates, and maximum deceleration values reached 5.8 m/sec^2 (19 ft/sec^2). About 7 percent of the drivers decelerated at rates that at some point exceeded 3.3 m/sec^2 (1/3 *g*). Compliance with the stopping requirement was quite high in both periods. Before treatment, 79 percent stopped, 11 percent made a rolling stop, and 10 percent did not stop. After treatment, 85 percent stopped completely, 7 percent rolled through, and 8 percent did not stop.

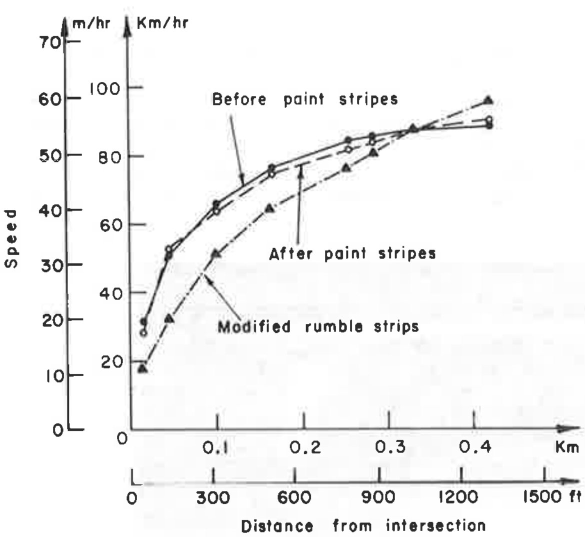

FIGURE 3 The effect of paint stripes and modified rumble strips on the 85th-percentile speeds.

Influence of Rumble Strips on Speeds, Deceleration, and Stopping Behavior

Table 2 shows the average speeds before and after rumble-strip treatment, and Figure 4 presents the 85th-percentile speeds. Before treatment, speeds were only 2 to 4 km/hr lower than on the opposite approach leg, thus allowing a direct comparison between the two treatments. Rumble strips have had a definite restraining effect on the approach speeds. The effect is apparent even at a distance of 420 m, where the strips could not yet be seen. At a distance of 285 m (934 ft), just before actual physical contact with the strips, the average speed is already 17 km/hr lower than it was in the "before" period. Speeds over the treated area are further reduced by an average of 40 percent. There is, however, some increase in the variance of vehicle speeds at each point. An examination of Figure 4 suggests that under no-treatment conditions, most of the deceleration occurred along 150 m up to the intersection. Rumble strips change that pattern so that much of the speed reduction takes place over the first 100 m from the point at which the strips are fully seen. A second, more moderate slowing down takes place during the final approach to the stop line. The foregoing pattern is also reflected in the values of average and maximum decelerations. These reached lower peaks, and only 1 percent of the vehicles decelerated at rates higher than 1/3 *g*.

Initial compliance rate at the stop sign was quite high, 82 percent, and rumble strips did not bring about any significant changes in stopping behavior.

The stability of treatment effects was evaluated after 1 year. There were no significant changes on the approach with paint stripes. The rumble strips maintained their speed-reducing effect, which became even more pronounced at the 85th-percentile speeds. Compliance rate also increased to a 90 percent level.

Crossover Manipulation

At this phase of the experiment, the approach previously treated with paint stripes was equipped with rumble strips. However, it was not deemed necessary to prove the ineffectiveness of paint stripes again on the alternate approach leg. Based on the speed and deceleration patterns previously observed, a modified rumble treatment was designed. Strips were laid along 150 to 10 m (492 to 33 ft) from the stop line.

Of the 28 strips, 25 were placed on the same

TABLE 2 Average Speeds and Standard Deviations Before and After Applying Rumble Strips

	Distance (m)							
Period	420	330	285	255	165	105	45	15
Before rumble strips								
Mean	73.2	72.1	70.8	70.1	64.6	57.5	41.6	24.7
SD	11.7	11.7	11.6	11.6	10.9	9.6	8.0	5.1
After rumble strips								
Mean	69.5	63.1	54.0	43.5	32.9	31.6	24.5	14.7
SD	11.5	12.8	15.4	18.5	14.0	11.6	8.3	6.2
Percent reduction in mean speeds	5	12.4	23.8	38.0	49.0	45.0	41.1	40.3

Note: 1 m = 3.3 ft; 1 km/hr = 0.6 mph. Speeds are given in kilometers per hour.

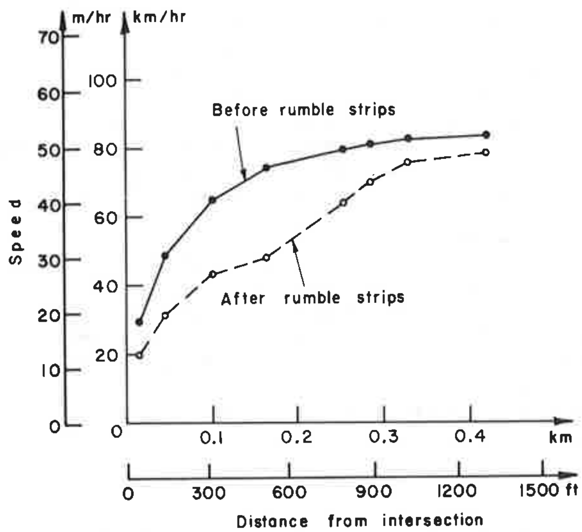

FIGURE 4 The effect of rumble strips on the 85th-percentile speeds.

spaces previously occupied by paint stripes. Table 1 presents the average speeds for the modified rumble treatment in comparison with the previous speeds measured on the same leg. Figure 3 presents the 85th-percentile speeds.

The modified rumble treatment caused a significant speed reduction, starting about 60 to 100 m before the first strip and continuing at a fairly uniform rate to the stop line. Compliance improved by another 4 percent--from 9 percent who did not stop with paint stripes to 5 percent with the modified rumble strips.

Additional Findings

Vehicle Type

Speed and deceleration functions for four vehicle classes--trucks, vans, private cars, and taxis--were essentially similar. So was stopping behavior.

Rain

No difference was detected between the data collected during the one rainy day and the rest of the data.

Traffic on Major Road

Presence of approaching vehicles on the major road had no effect on the deceleration pattern of vehicles on the minor approaches. However, all observed vehicles stopped (complete or rolling) when there was a vehicle on the major road at a range of 100 m (330 ft) from the intersection.

Accidents

The intersection was the scene of two to three right-angle collisions a year during the 3 years preceding the experiment. A total of 7 injury accidents occurred with 45 casualties (17 slightly injured, 15 seriously injured, and 3 fatalities). Small passenger-carrying pickup trucks were often involved. Although these are hardly the kind of accident data that can be considered experimental evidence, it was assuring to record that during the 4 years since the installation of rumble strips at the site, there have been only four accidents with seven casualties (five slightly injured and two seriously injured). Traffic volumes increased during this period.

Drivers' Opinion

Drivers understood the purpose of the rumble strips, rated them as quite tolerable, and endorsed their use. During an earlier test, they strongly objected to rumble strips 15 to 19 m (5/8 to 3/4 in.) high.

DISCUSSION OF RESULTS

Speed and Deceleration Effects

The small effect of paint stripes on vehicle speeds is in line with the relatively small, 3- to 10-km/hr speed reductions reported elsewhere (5,9,10,11). It should be noted that in Britain, the stripes (called "yellow bars") are made of thermoplastic paint stripes, and are not sprayed on with paint. Therefore, they have some thickness and a certain rumble effect. The rumble-strip effects in the current study are large in comparison with those in previous studies at similar locations (4,12-14). This is probably due to the particular design and placement employed. Many of the rumble-strip or rumble-area designs based on grooves or aggregates bonded to the road surface appear too gentle in their vibratory effect to achieve significant speed reduction. In addition, in many applications (3), the last 100 m or so close to the intersection was left untreated, thus leaving space for unchecked speed or even acceleration. Speed patterns in this study and others (11,13) show that at 80 km/hr, drivers do most of their deceleration during the last 150 m, and particularly the last 60 m. Therefore, this is the section of the vehicles' course that should be treated.

Influencing Mechanism of the Strips

Much of the speed reduction occurred before the strips were passed. Paint stripes had a similar, although much smaller, influence. This phenomenon, also observed by others (5,11,12,15), suggests that the visual pattern provides early information or warning about the approach to an intersection. This form of information may attract more attention than the regular warning signs by virtue of its conspicuity and rarity. Because of the latter, the strip pattern may be more convincing to the driver concerning its implied message: "slow down and prepare to stop." The preparation or planning is, perhaps, the factor mediating the influence of the stripes. The visual pattern, if appropriately laid out, may assist in planning a uniform and comfortable deceleration. The auditory and vibratory stimulation added by rumble strips delivers, however, a more compelling message that drivers cannot ignore. The expectation of vibration by itself induces drivers to plan their deceleration in advance. This is evident in the initial slowing before the strips are passed, which is not abrupt and is often accomplished by coasting without brake application.

Design Implications

It is recommended that treatment distance correspond to the declaration distance of the 85th speed percentile, empirically observed before treatment. About 10 to 15 m of pavement before the stop line should be left clear. The number and spacing of strips were not manipulated in this experiment. In view of the deceleration patterns observed, it should be possible to reduce the number of strips substantially. Also, the geometry of spacing is probably not that important. It is expected that at the study site, pairs or trios of strips placed 6 m (20 ft) apart and at about 50-m (165-ft) intervals over the 150 m (490 ft) would be sufficient to achieve similar effects. For a width of 50 to 60 cm (20 to 24 in.), the strip height should not exceed 12 to 15 mm (1/2 to 5/8 in.). Should more drastic decelerations be necessary at a given site, this can be achieved by spacing more strips, thus producing a harsher ride.

Where wide shoulders exist, treatment should extend onto the shoulder to deter evasive maneuvers. On asphalt roads, some sagging can be expected after 3 to 4 years. Repainting is also necessary. Snow and snow ploughing might eliminate the use of rumble strips.

Stopping Behavior

Previous experimental studies reported large, 30 percent improvements in stopping rates (4,16). A smaller improvement, 5 to 10 percent, was noted in a later survey (3). This study shows a small increase in compliance, perhaps because of a ceiling effect. At low-volume installations, it is not always clear to what extent stopping rate is correlated with safety. Often, at locations with sufficient visibility, stop signs could be safely changed to yield signs.

The failure to observe throughout the entire data collection effort even a single case of a "dangerous nonstopping" incident is rather typical of the conditions at low-volume intersections and of accident analysis in general. Counting those who do not stop at an intersection, therefore, may be a poor indicator of risk, as well as of countermeasure effectiveness. It is suggested that slowing down enough to be able to stop if necessary might be a more useful criterion for evaluating the effectiveness of rumble strips or similar treatments. Indeed, paint stripes in advance of roundabouts are credited with a substantial decrease in accidents, even though the reductions in speed are fairly small (7).

Criteria for Application

It may well be that the utility of stripes and other special measures depends in part on their being used sparingly. Their special attention-attracting characteristic can certainly be diminished if they are associated with the critical sites as often as the regular warning signs are. However, well-designed rumble strips may be unique in their resistance to familiarity. The vibration at high speeds is uncomfortable no matter how many times the driver encounters the strips. Their appearance would therefore arouse the expectation of vibration and, concurrently, induce a planned deceleration response. Nevertheless, there appears to be no need or justification for widespread use of rumble strips (in addition to cost and maintenance considerations). In the case of low-volume rural intersections, they should be considered only in instances in which an intersection is afflicted with poor sight distances, misleading cues about exact location, and ambiguity about which are the minor and which are the major approach legs, and where excessive speeds are diagnosed as causal factors in accidents. They should be installed only if these factors cannot be easily or cost-effectively improved or removed.

Summary

1. Paint stripes had a minor influence on driver behavior, whereas rumble strips lowered speeds by an average of 40 percent.
2. Both treatments had a small, though positive, effect on compliance with the stopping requirement.
3. The effects on driver behavior did not diminish after a period of 1 year.
4. Rumble strips changed the pattern of decelerations to a more moderate rate, with few vehicles exceeding the 1/3-*g* level.
5. A modified, shorter rumble-strip treatment was found to be just as effective as the previous, longer installation.

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A Further Note on Undulation as a Speed Control Device

KING K. MAK

ABSTRACT

Conventional speed bumps have sometimes been used as a passive means of controlling speed, but there are problems associated with them, such as damaging the suspension and front-end alignment of crossing vehicles and causing loss of control for drivers of two-wheeled vehicles under certain circumstances. The U.K. Transport and Road Research Laboratory (TRRL) developed a new speed control device known as an undulation (or speed hump) that eliminates many of the deficiencies associated with conventional speed bumps. This new design has been gaining acceptance in the United States; it has been installed in a number of cities and the results so far have been favorable. The results of a research study to evaluate the effectiveness of the undulation as a speed control device are reported. The study consisted of three parts: a speed study, an instrumented-vehicle study, and a questionnaire survey. The study results indicated that that the undulation design is an effective speed control device and is more desirable and acceptable than the conventional speed bump. The study results also suggested that the level of speed control can be varied by adjusting the height of the undulation for use with various speed limits.

Speed control on residential streets has long been a concern among traffic engineers. Some drivers tend to ignore the speed limit, thereby creating a hazardous condition to pedestrians and other motorists. It may be possible to alleviate the speeding problem by increased law enforcement or a safety campaign, but the effects are mostly short lived. It is sometimes necessary to resort to some form of passive speed control device.

One commonly used passive speed control device is the conventional speed bump. Typical dimensions of a speed bump are a height of up to 6 in. and a length (in the direction of vehicle travel) of up to 3 ft.

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Although widely used, conventional speed bumps present problems in both safety and operations. A vehicle crossing a speed bump at even low to moderate speed receives a severe jolt, which could result in damage to the suspension and front-end alignment of the vehicle. On the other hand, the jolt felt by the occupants of the vehicle varies fairly little and may even lessen with speed. This results in a hazardous situation in which most drivers will come to an almost complete stop before crossing the speed bump, whereas a few reckless drivers may actually speed over the bump. In addition, drivers of two-wheeled vehicles have been known to experience loss of control under certain circumstances.

The U.K. Transport and Road Research Laboratory (TRRL) developed a new speed control device known as an undulation (or speed hump) that eliminates many of the deficiencies associated with conventional speed bumps (1-4). The undulation design, as shown in Figure 1, differs from the conventional speed bump in that it has a length (in the direction of travel) of about 12 ft, approximately 1 1/2 times the average length of a subcompact car's wheelbase, and a parabolic profile with a maximum height of 4 in. Vehicle occupants crossing an undulation experience an uncomfortable rocking motion which intensifies with speed.

The TRRL reports that the undulation design appears to be effective in reducing vehicle speed and traffic volume as well as being beneficial to safety on a residential street and is favored by the majority of drivers. The undulation design has also been gaining acceptance in the United States and undulation humps have been installed in a number of cities. The results so far have been favorable (5,6), although there are still some who oppose any kind of speed control device (7).

Two series of undulations, one with five undulations and the other with two, were installed on two separate roadways on the grounds of Southwest Research Institute (SwRI) in San Antonio, Texas. These undulations were installed in response to expressed concern over speeding on these roadways and the accompanying hazards to pedestrians and other motorists. The average free-flow speed before the installation of these undulations was in excess of 30 mph on one roadway and above 35 mph on the other despite a posted speed limit of 25 mph.

The specifications for these series of undulations were a length of 13 ft, a maximum height of 4 in. with a parabolic profile, and a spacing of 300 ft between undulations. Note that the specified length of the undulation is 13 ft instead of the TRRL design length of 12 ft, because the average vehicle in the United States is larger and longer than that in the United Kingdom.

A research study, sponsored by SwRI, was conducted to evaluate the effectiveness of the undulation as a speed control device, the results of which are presented in this paper. The research study consisted of three parts:

1. Speed study,
2. Instrumented-vehicle study, and
3. Questionnaire survey.

For comparison purposes, a conventional speed bump on a third roadway, also with a speed limit of 25 mph, was included with the seven undulations. The conventional speed bump has a maximum height of 5 in. and a length of 12 in.

SPEED STUDY

With time-lapse video photography, vehicle speed data were collected at the seven undulations and the conventional speed bump. For this study, 216 hr of data and more than 8,000 vehicles were recorded. A sampling scheme, based on time of day and vehicle type, with oversampling during hours of darkness and for vehicle types other than passenger cars, was used. Also, only free-flowing vehicles were sampled, excluding following vehicles in a platoon and turning vehicles. Overall, 1,472 vehicles were sampled for study.

For each sampled vehicle, certain descriptive data were first recorded, including vehicle type, size, and direction of travel. Vehicle speed and acceleration-deceleration data were then obtained at nine different locations for each undulation and bump. The average speed profiles for the undulations and the conventional speed bump are provided in Figure 2. Speed and deceleration data are summarized in Table 1. Note that the 50-ft point is used as the reference for all speed-change and acceleration-deceleration data. Several trends are evident from the data:

1. The approach, crossing, and exit speeds are higher for the undulation than for the speed bump, although these speed differentials may partially reflect the differences between traffic characteristics of the roadways.
2. The speed changes are much higher for the conventional speed bump than for the undulation as are the acceleration-deceleration rates. This indicates much harder braking and faster speeding up at the conventional speed bump.
3. Most of the braking and acceleration occurs within 50 ft of the undulation or speed bump. The deceleration rate is fairly constant over the 50-ft

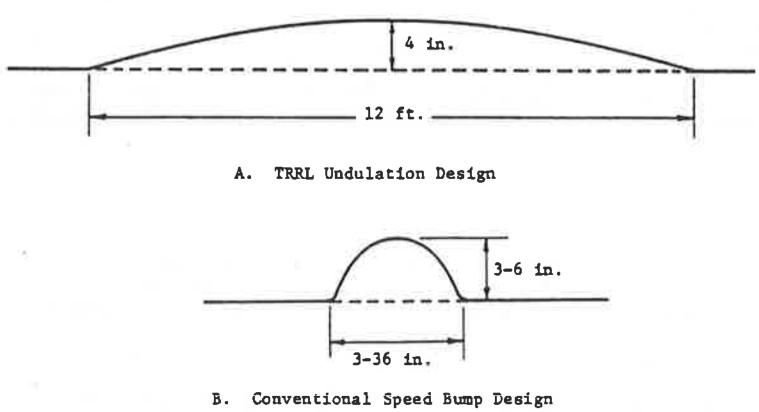

FIGURE 1 Comparison between undulation and speed-bump designs.

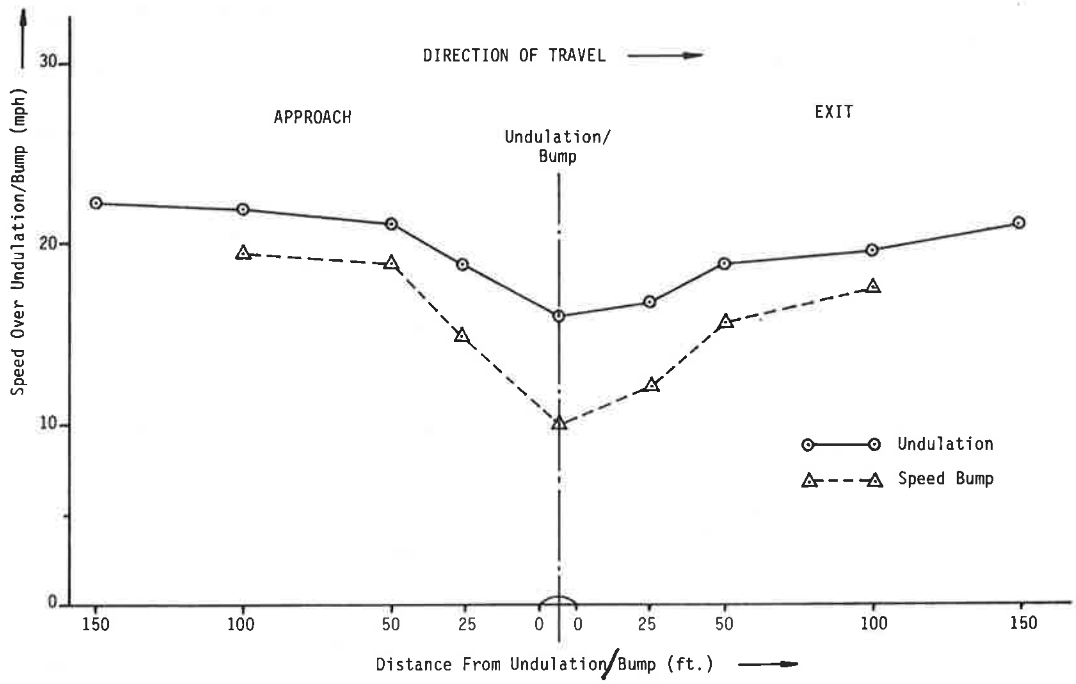

FIGURE 2 Average speed profiles for undulation and conventional speed bump.

TABLE 1 Speed and Acceleration-Deceleration Data

	Undulation	Conventional Speed Bump
Avg approach speed (mph) at 50 ft from undulation or bump	20.9	17.7
Avg crossing speed (mph)	15.9	9.8
Avg exit speed (mph) at 50 ft from undulation or bump	18.6	15.5
Speed change (mph)		
50 ft to undulation or bump	-5.0	-7.9
Undulation or bump to 50 ft	+2.7	+5.7
Acceleration-deceleration rate (ft/sec^2)		
50 ft to undulation or bump	-3.6	-5.0
Undulation or bump to 50 ft	+1.5	+2.6

Note: The minus sign denotes deceleration, whereas the plus sign denotes acceleration.

approach area. In comparison, the acceleration rate is lower in the first 25-ft exit area than in the 25- to 50-ft exit area, indicating that there is a short time lag after the vehicles cross over the undulation or speed bump before the drivers start to accelerate.

4. The highest average speed of 22.2 mph occurs in the mid-point between the undulations, as may be expected. The average speed is considerably lower than those before the installation of the undulations, indicating that the undulations are effective in controlling vehicle speeds. The 85th-percentile speed is around 28 mph, which is still higher than the posted speed limit of 25 mph.

There are marked differences in the speed profiles for the individual undulations. Some of the differences may be attributed to the location of the undulation and turning movements. However, much of the difference could be the result of variations in the physical dimensions of the undulations. The contractor did not construct the undulations according to specifications; the height ranged from 3.21 to 4.15 in. versus the specified 4 in. These variations are undesirable from the operational standpoint, but they provide an excellent opportunity to evaluate the effects of varying heights on the performance of undulations.

The undulation height, the crossing speed, and the speed change and deceleration rate at the 50-ft point are summarized in Table 2 for the individual undulations. It is evident from the data that the crossing speed decreases with increasing undulation height. Correspondingly, the speed change and the accompanying deceleration rate increase with increasing undulation height. In other words, as the height of the undulation increases, the speed of vehicles crossing it will be lower and there will be more and harder braking while approaching the undulation.

TABLE 2 Speed and Deceleration Data for Individual Undulations

Undulation No.	Height (in.)	Crossing Speed (mph)	Speed Change (mph)	Deceleration Rate (ft/sec^2)
1	3.35	20.4	-2.3	-1.96
2	4.15	14.0	-8.0	-7.87
3	3.93	17.1	-5.6	-2.40
4	3.58	15.1	-5.3	-4.79
5	3.43	15.3	-3.2	-2.70
6	3.86	14.6	-5.8	-4.42
7	3.21	18.2	-2.5	-2.51

Note: The minus sign denotes deceleration.

Linear regression equations relating the undulation height to the crossing speed, the speed change, and the deceleration rate were developed as follows and are shown in Figure 3:

Crossing speed = 35.03 - 5.13 (undulation height), $R^2 = 0.67$;

Speed change = 16.89 - 5.92 (undulation height), $R^2 = 0.92$;

Deceleration rate = 12.00 - 4.34 (undulation height), $R^2 = 0.51$.

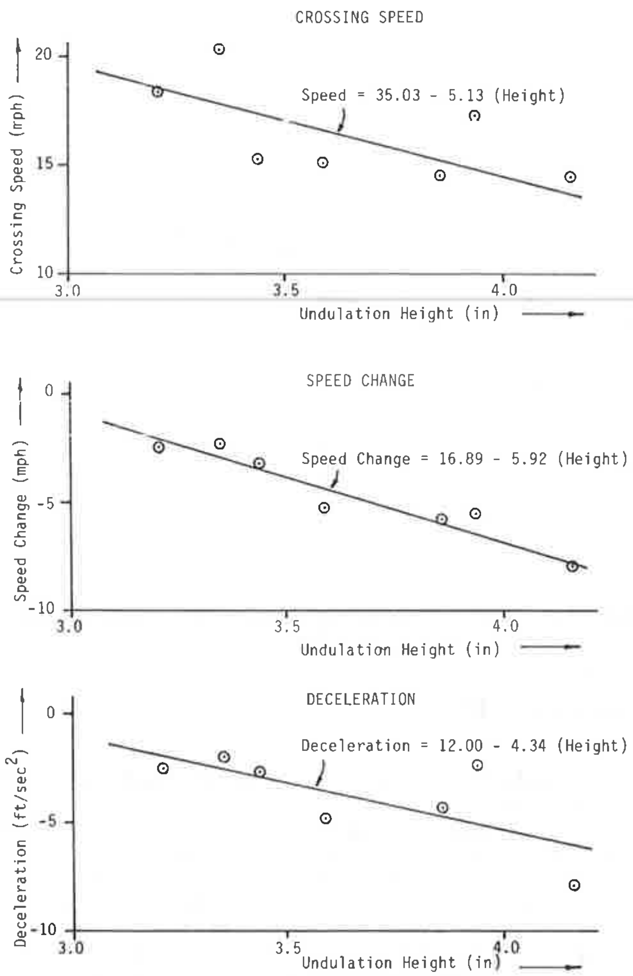

FIGURE 3 Relationships between undulation height and crossing speed, speed change, and deceleration rate.

These relationships can be used to estimate the level of speed control when the height of the undulations is varied. For example, with a undulation height of 3 in., the average crossing speed is expected to be 19.6 mph with the vehicle slowing down by 0.87 mph at a deceleration rate of 1.02 ft/sec^2. When the undulation height is increased to 4 in., the average crossing speed drops to 14.5 mph and the vehicle slows down sharply by 6.8 mph at a deceleration rate of 5.4 ft/sec^2.

Although the crossing speed is affected by the height of the undulation, the approach speed is not. In other words, the vehicle speed between undulations is not a function of the undulation height. Spacing between undulations may be a more important factor for traffic speed between undulations. However, it is not possible to evaluate the effect of spacing between undulations because a uniform spacing of 300 ft was used.

In terms of vehicle type and size, the observed differences in crossing speed, speed change, and deceleration rate are surprisingly minor for the undulation, as shown in Table 3. There are more variations for the conventional speed bump, but these are still relatively small compared with the differences between undulation and speed bump. As stated previously, the average crossing speed is considerably lower for the conventional speed bump than for the undulation, and the speed change and deceleration rate are much higher. This is best illustrated by motorcycles, which have the highest crossing speed (21.2 mph) and the lowest speed change (0.8 mph) when crossing the undulations. In sharp contrast, the crossing speed of motorcycles over the conventional speed bump is only 9.1 mph with a speed change of 10.1 mph.

TABLE 3 Speed and Deceleration Data by Vehicle Type

Vehicle Type	Crossing Speed (mph)	Speed Change (mph)	Deceleration Rate (ft/sec^2)
Undulation			
Passenger car			
Subcompact	16.3	5.2	3.49
Compact	16.2	4.6	3.51
Intermediate	16.1	5.2	4.25
Full size	15.9	5.1	3.63
Avg	16.1	5.0	3.72
Motorcycle	21.2	0.8	2.42
Pickup			
Small	15.9	5.4	3.61
Large	15.7	3.2	3.75
Avg	15.8	4.3	3.68
Van	13.8	5.6	3.00
Truck			
Single unit	15.0	4.0	2.72
Tractor-trailer	12.2	3.0	1.44
Avg	13.6	3.5	2.08
Speed Bump			
Passenger car			
Subcompact	10.6	7.4	5.70
Compact	9.7	8.0	4.07
Intermediate	10.5	7.6	5.28
Full size	11.2	8.0	4.70
Avg	10.5	7.8	4.94
Motorcycle	9.1	10.1	5.82
Pickup			
Small	7.5	9.2	5.23
Large	9.1	7.7	4.27
Avg	8.3	8.5	4.75
Van	7.8	8.7	5.39
Truck			
Single unit	6.0	11.0	6.78
Tractor-trailer	6.6	2.2	0.40
Avg	6.3	6.6	3.39

INSTRUMENTED-VEHICLE STUDY

The speed study described earlier provides information on the speeds at which the vehicles cross the speed control devices. An instrumented-vehicle study was then conducted to examine the vertical acceleration, measured in g-forces, experienced by the vehicle and its occupants while crossing the undulation or speed bump. The hypothesis is that the speed at which a vehicle crosses over the speed control device is a function of the vertical acceleration sustained by the vehicle and its occupants, which in turn is a measure of the level of discomfort experienced by the occupants.

The experimental setup took into account three major factors:

1. Speed control device type and dimension: Three undulations of high, medium, and low profiles (undulations 6, 5, and 7 in Table 2, respectively) and the conventional speed bump were selected for the study.

2. Vehicle type: Three test vehicles were used: a 1979 Ford LTD full-size car with extra-heavy-duty suspension, a 1979 Ford Pinto subcompact car with original equipment suspension, and a 1974 Chevrolet C-10 long-bed pickup truck with heavy-duty suspension.

3. Vehicle speed: The test speeds generally ranged from 10 to 40 mph in 5-mph increments. However, the upper limit of the speed range was determined also by the speed at which the vehicle suspension bottomed out in order not to damage the test vehicles.

Each test vehicle was instrumented with a biaxial accelerometer attached to the floorboard near the vehicle center of gravity and a triaxial accelerometer mounted in the head of a 50th-percentile male dummy seated, but unrestrained, on the right front passenger seat. A fifth wheel was used to monitor the vehicle speed. The test runs were also documented on 16-mm movie films for analysis of the vehicle motion.

Figure 4 shows the vertical acceleration traces of the vehicle and the unrestrained occupant for the Ford Pinto at 20 mph over the high-profile undulation and the conventional speed bump. The maximum positive (G_{max}), the minimum negative (G_{min}), and the resultant (ΣG) vertical acceleration readings are identified.

For the undulation, the vehicle moves upward as the front tires contact the undulation, resulting in a positive vertical acceleration. The vehicle comes down after crossing the undulation as indicated by the negative vertical acceleration. The vehicle then bounces a couple of times before returning to its normal position. The unrestrained occupant moves in the opposite direction to the vehicle with a slight time lag. In other words, the occupant moves downward as the vehicle moves upward. The peak vertical acceleration usually occurs when the vehicle comes down after crossing the undulation.

The acceleration pattern for the speed bump is very different from that of the undulation. The pulses are very sharp, but of relatively short duration. Also, the pulses from the front and rear axles are distinct; that from the rear axle is more severe. In comparison, the acceleration pattern for the undulation is more gradual and smooth.

Linear regression equations were developed to relate the vehicle crossing speed to the resultant vertical acceleration (ΣG_{Veh} and ΣG_{Occ}) experienced by the vehicle and the occupant (averaged over the three vehicles) for both undulations (averaged over the three undulations) and the conventional speed bump. The regression equations are as follows and are shown in Figure 5:

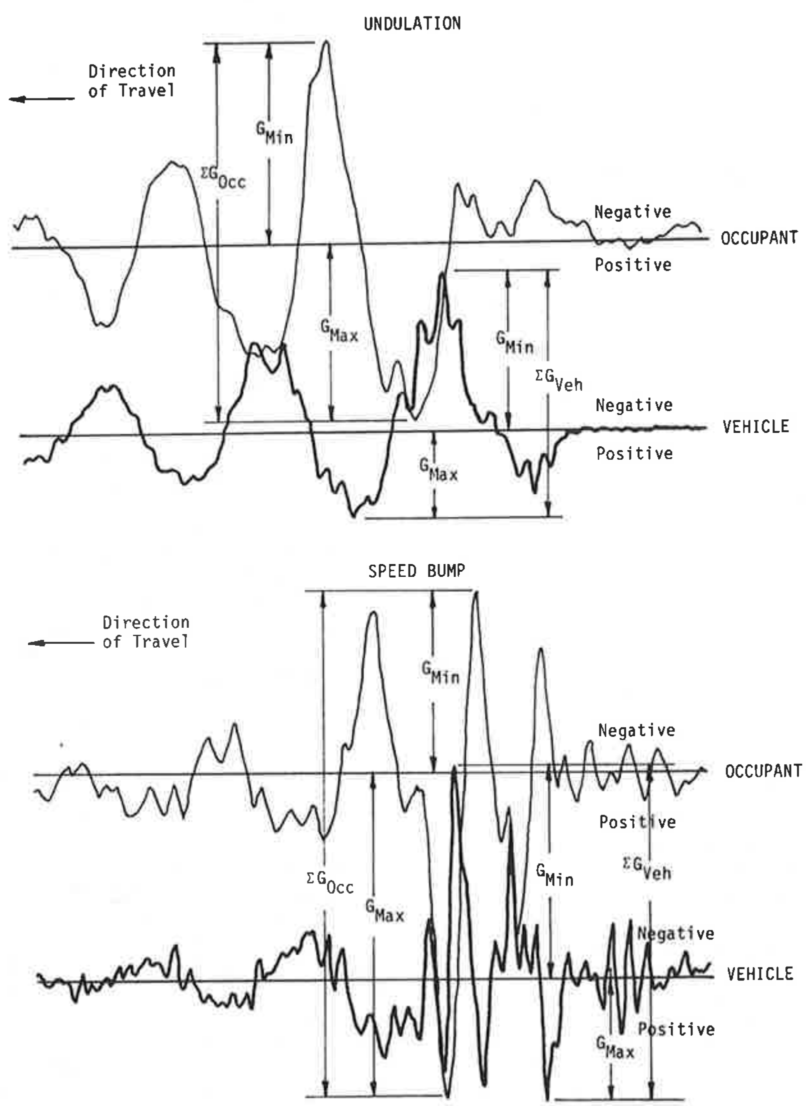

FIGURE 4 Acceleration traces of subcompact car at 20 mph.

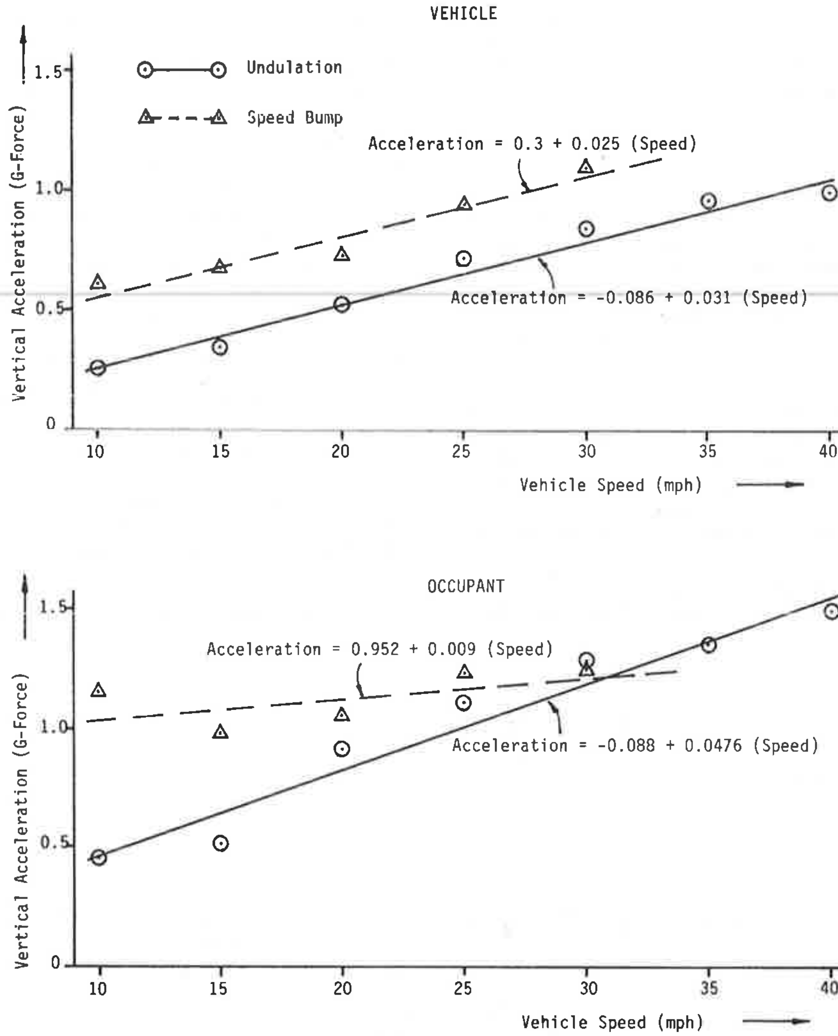

FIGURE 5 Average vertical acceleration as a function of speed.

Undulation:

Vehicle vertical acceleration = -0.086 + 0.031(speed), R^2 = 0.99;

Occupant vertical acceleration = 0.3 + 0.025(speed), R^2 = 0.94.

Conventional speed bump:

Vehicle vertical acceleration = -0.088 + 0.048(speed), R^2 = 0.94;

Occupant vertical acceleration = 0.952 + 0.009(speed), R^2 = 0.94.

The vertical acceleration experienced by the vehicle increases with higher crossing speed for both undulations and conventional speed bump. The rates of increase (as indicated by the slopes of the regression lines) are similar between the undulations and the speed bump, although the vehicle vertical acceleration for the speed bump is consistently more severe than that for the undulations.

The key difference between the undulations and the conventional speed bump is in the vertical acceleration experienced by the unrestrained occupant. Although the occupant vertical acceleration increases with higher crossing speed for the undulation, there is little change in that for the speed bump. At speeds of below 20 mph, the vertical acceleration experienced by the occupant is much higher for the speed bump than for the undulations. Then, as the crossing speed increases, the occupant vertical acceleration for the undulations catches up and eventually surpasses that of the speed bump.

This supports the contention that the undulation is a more desirable speed control device than the conventional speed bump. The undulation design provides a relatively smooth ride at low speeds and allows the drivers to maintain a somewhat constant speed. However, as the vehicle speed increases, the vertical acceleration experienced by the vehicle and its occupant(s) also increases proportionately, thus discouraging the driver from speeding.

On the other hand, vehicles crossing the conventional speed bump receive a severe jolt even at low speeds, thus fostering the pattern of hard braking before crossing the speed bump followed by rapid acceleration after crossing the speed bump. Furthermore, the vertical acceleration experienced by the occupant increases very little with higher speed so that the speed bump is not particularly effective against excessive speeding.

As for comparisons between the three undulations with high, medium, and low profiles, the data clearly indicate that more severe vertical acceleration is associated with greater undulation height, particularly for the occupant. This is in total agreement with the previous finding that lower vehicle speed is associated with greater undulation height.

Of the different vehicle types, the subcompact car fares the worst, as may be expected with the suspension bottoming out above 20 mph. The rear suspension of the unladen pickup truck tends to bounce excessively. This results in high occupant vertical acceleration for the conventional speed bump, even at relatively low speeds.

Despite the substantial differences in vertical acceleration among the three vehicle types tested, results of the speed study presented earlier indicate that there is little difference among their crossing speeds. This suggests that the drivers of small cars and pickup trucks tolerate higher levels of vertical acceleration in order to keep up with the average traffic flow.

QUESTIONNAIRE SURVEY

A questionnaire survey was conducted to solicit the opinion of SwRI employees on the appropriateness and effectiveness of the speed control devices. The questionnaire was sent to 20 percent (398) of the SwRI employees selected at random and the response rate was a high 48.2 percent (198). The questionnaire had nine multiple-choice questions with space provided for written comments. Although the respondents may not necessarily be representative of the overall population, their responses do provide some indication of the expected level of acceptance of the undulation design by the driving public. Highlights of the questionnaire survey results are as follows.

The majority (66.3 percent) of the respondents prefers the undulation over the conventional speed bump (8.9 percent), although one-quarter (24.7 percent) of them do not like either of the speed control devices and some believe that stricter enforcement of the speed limit is a better solution.

Nearly half (49.4 percent) of the respondents indicate that they usually come to a complete or near stop before crossing the conventional speed bump, and one-third (32.0 percent) indicate hard braking before the speed bump. In contrast, almost all respondents (96.8 percent) reply that they either maintain constant speed or only brake lightly while crossing the undulation.

Some of the written comments and complaints are informative and helpful to future installations of undulations. Some respondents point out that the undulations are not uniform in their behavior and that they are poorly signed and delineated and difficult to recognize, especially under adverse lighting or weather conditions. Also, some respondents complain that they were not informed in advance of the undulation installations nor advised of the proper speed at which to cross.

These comments point to the importance of advance public information and education before the installation of the undulations because the undulation design is new to the driving public. The undulation should be properly signed and delineated to advise the motorists of the presence of the speed control device and the appropriate crossing speed. Finally, care should be taken in the construction of the undulations to ensure that they are constructed according to specifications.

Overall, the respondents favor the undulation over the conventional speed bump by an overwhelming margin. They recognize the more desirable characteristics of the undulation design and have adjusted their driving behavior accordingly. However, there is considerably objection to any form of passive speed control device, as may be expected. This suggests that undulations, though superior to the conventional speed bump, should only be used sparingly, confined to locations where speeding creates an unacceptably hazardous condition and other less drastic speed control measures have failed to achieve the desired results.

SUMMARY OF FINDINGS

• The undulation design is a very effective speed control device. The average speed was reduced from more than 30 mph to approximately 16 mph at the undulations, with a high of 22.2 mph between the undulations.

• The undulation design is more desirable because it overcomes many of the drawbacks associated with the conventional speed bump. It provides a smoother ride at low speeds and allows the drivers to maintain a somewhat constant speed without hard braking and rapid acceleration while discouraging excessive speeding. It is also safer for motorcycles because it eliminates the sharp jolts to the vehicle suspension. The undulation design is also more acceptable to the majority of the drivers than the conventional speed bump.

• The level of speed control can be varied by adjusting the height of the undulation. The recommended undulation heights for various speed limits are as follows:

	Undulation Height (in.)	
Speed Limit (mph)	Minimum	Maximum
25	3.75	4.0
30	3.0	3.25

• The driving public should be advised of the undulations and the appropriate crossing speed before their installation through public hearings or the news media. The undulations should be properly signed and delineated. Care should be taken to ensure that the undulations are constructed according to specifications. These measures would help with public acceptance and minimize future complaints and liability.

• The undulation design is not a panacea to the speeding problem, but simply one of the many means of speed control available to the highway engineer. It should be used judiciously and only after other less drastic measures, such as increased law enforcement or a community safety campaign, have failed to achieve the desired result, because any form of passive speed control device will penalize the good as well as the bad drivers.

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Abridgment

Analytical Warrant for Separate Left-Turn Phasing

NAGUI M. ROUPHAIL

ABSTRACT

The development of a new volume warrant for left-turn phasing at signalized intersections is presented. The concept is to maintain a fixed volume-to-capacity (V/C) ratio for all intersection movements. Thus left-turn phasing would be warranted when the unprotected left-turn V/C ratio exceeds that of through traffic. Left-turn capacity is derived from formulas in the Highway Capacity Manual and the Australian Road Research Board Capacity Manual. The warrant also combines both signal-timing and capacity-analysis procedures. The proposed warrant has been compared with other methods found in the literature and the results are, in general, favorable. This study is preliminary in nature; its scope is limited to four-legged intersections with one through lane and an exclusive left-turn lane of adequate length on all approaches. No adjustments for trucks, buses, or pedestrian interference are considered. Finally, it is understood that traffic signal parameters are selected according to Webster's optimum settings for fixed-time signals.

The provision of separate left-turn phasing at intersections has been the subject of considerable research. The need for developing guidelines for left-turn phasing stems from an absence of uniform criteria in the Manual on Uniform Traffic Control Devices (MUTCD) (1) and the need to formulate consistent policies regarding left-turn treatments at signalized intersections.

The majority of left-turn warrants follow these general criteria:

1. Left-turn delay exceeds a prespecified threshold (e.g., 30 sec),
2. Left-turn volume exceeds a threshold [e.g., 100 vehicles per hour (vph)],
3. The product of left-turn volume and opposing traffic exceeds a prespecified threshold (e.g., 50,000 or 100,000),
4. Left-turn volume-to-capacity (V/C) ratio exceeds a threshold (e.g., 0.70 to 0.90), and
5. Number of left-turn-related accidents reaches a prespecified threshold (e.g., four accidents per approach per year).

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Methodologies for developing these guidelines have included microscopic simulation modeling (2,3), analytical models calibrated with field observations (4), and comprehensive studies of accidents, conflicts, delay, and gap-acceptance parameters (5).

Several observations are noted from the literature:

1. Although it is evident that left-turn capacity is affected by the amount of volume and green time to the opposing flow (V_0 and g/c, respectively), many studies have dealt with these two parameters independently. Yet all signal-timing methods relate signal splits (g/c) to the critical-flow ratio (V/S). In fact, Messer has shown that improved unprotected left-turn operations can be expected by optimizing the signal settings alone (6).

2. None of the prescribed criteria relate left-turn operation to through-traffic operation.
3. Left-turn through passenger-car equivalents (PCEs) vary with the signal settings and opposing-flow rate (7). Thus, an exact solution would require iterating between the two values until an equilibrium is reached.

The proposed warrant avoids many of these pitfalls by ensuring that the unprotected left-turn movement does not become critical under two-phase operation. It should be realized that although the V/C ratio is maintained constant for all movements, left-turn delay is still higher, on the average, than through-traffic delay. This is because delay is a function of both V/C and the flow ratio V_1/S_1, where S_1 is the unprotected left-turn saturation flow rate. Because S_1 is in all cases less than its through-traffic counterpart, operation at the left-turn volume warrant values will always result in higher left-turn delays. The numeric delay values will vary according to the intersection V/C ratio. The proposed warrant is now stated as follows:

> For a given combination of critical lane volumes at isolated, signalized intersections with one through lane and an exclusive left-turn lane of adequate length on all approaches, a separate left-turn phase should be considered whenever the degree of saturation for any left-turn movement exceeds the critical intersection V/C ratio.

Subsequently, PCE values are calculated as the ratio of critical through-lane volume and the left-turn volume warrant.

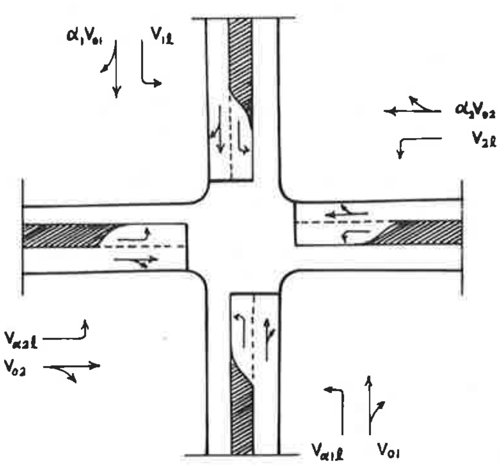

FIGURE 1 Intersection layout considered.

ANALYTICAL DEVELOPMENT

Consider the intersection layout in Figure 1. Critical lane volumes are denoted V_{01}, V_{02} and noncritical volumes $\alpha_1 V_{01}$ and $\alpha_2 V_{02}$, where α_1 and $\alpha_2 < 1$. Assume a fixed through saturation flow rate of S_T. The left-turn warrants are shown as $V_{\alpha 11}$, V_{11}, $V_{\alpha 21}$, V_{21}. On the basis of the warrant definition, a left-turn phase is considered when

$$V_{j1}/C_{j1} \geq V_{0j}/C_{jT}$$

where

V_{0j} = opposing volume in phase j, j = 1, 2;
C_{j1} = left-turn capacity in phase j, j = 1, 2; and
C_{jT} = through-lane capacity in phase j, j = 1, 2.

Note that for through movements,

$$C_{jT} = S_T * g_j/C_0$$

where

g_j = effective green time in phase j,
C_0 = Webster's optimum cycle length (8) = $(1.5 * L + 5)/[1 - (V_{01} + V_{02})/S_T]$, and
L = lost time per cycle (sec).

And for the left-turn movements, note that

$$C_{j1} = (S_{0j} * g_{j1}/C_0) + (3{,}600 * K/C_0)$$

where

S_{0j} = unprotected left-turn saturation flow rate in phase j = $S_{u1} - V_0/(g_j/C_0)$,
S_{u1} = unopposed left-turn saturation flow rate, and
g_{j1} = effective green time in phase j in which left turns may proceed in gaps of opposing traffic. This time can be estimated as (9)

$$(S_T * g_j - V_{0j} * C_0)/(S_T - V_{0j})$$

WARRANT FORMULATION

From the foregoing analysis, the following left-turn volume warrant is proposed for movement V_{11} in Figure 1:

$$V_{11} \geq V_{01}/(S_T * g_1/C_0) * \Big(\{S_{u1} - [V_{01}/(g_1/C_0)]\} * \{(S_T g_1 - V_{01}C_0)/[C_0 * (S_T - V_{01})]\} + 3{,}6000 * K/C_0\Big)$$

or, with some manipulation,

$$V_{11} \geq V_{01} * \Big(\{(S_{u1}/S_T) - [V_{01}/(S_T * g_1/C_0)]\} * \{[(S_T - V_{01}C_0)/g_1]/(S_T - V_{01})\} + [3{,}600 * K/(S_T * g_1)]\Big)$$

provided that $V_{01} \leq S_{u1} * g_1/C_0$ and

$$V_{11} \geq (3{,}600 * V_{01} * K)/(S_T * g_1) \text{ for } V_{01} > S_{u1} * g_1/C_0$$

Similar warrants can be derived for the remaining three movements.

COMPARISON WITH EXISTING WARRANTS

The proposed warrant is compared with four existing ones:

1. Volume product warrants of 50,000 and 100,000, respectively;
2. Critical left-turn volume warrants from the TEXAS model (2);
3. A left-turn peak-hour volume of 150 vph; and
4. Two or more left-turn arrivals per cycle.

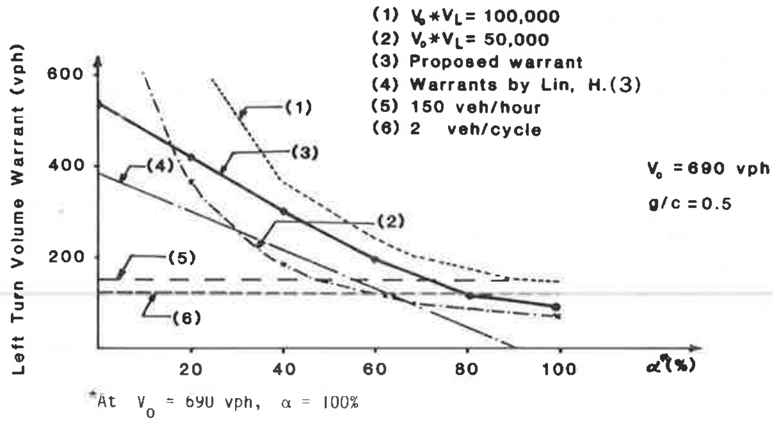

FIGURE 2 Left-turn volume warrants: a comparison at g/c = 0.50.

The results are shown in Figure 2 for various opposing flow rates and g/c = 0.50. In general, the proposed warrant appears to fit quite well within the existing literature. It lies between the two volume product warrants; thus a volume product warrant of 75,000 appears to be valid theoretically when g/c = 0.50 and V_0 is greater than 200 vph. On the other hand, the volume product warrants tend to overestimate the left-turn volume warrant at low values of g/c and V_0, as shown in Figure 3 (and vice versa for g/c < 0.4). The TEXAS estimates gave very conservative values of critical left-turn volumes, which could be explained from the simulation run parameters used in the model, such as minimum acceptable gaps and through-traffic saturation flow rates. Finally, fixed left-turn volume warrants were only valid under heavy opposing flows, that is, when the capacity is primarily developed in the clearance interval.

Limited sensitivity analyses performed on the model indicate that the proposed warrants are very sensitive to the unopposed left-turn saturation flow rate S_{ul}; an increase in S_{ul} by 200 vehicles per hour of green (vphg) results in a 100-vph increase in the proposed warrant. The effect of lost time per cycle (L) and clearance capacity per cycle (K) was not significant, however.

IMPLEMENTATION

On the basis of the literature and sensitivity analyses, it is recommended that the following parameter values be used for typical traffic conditions:

S_T = 1,750 vphg,
S_{ul} = 1,440 vphg,
L = 7 sec, and
K = 2 veh/cycles.

The recommended warrants are summarized in Table 1 along with the conditions for applying the formulas. In addition, an interactive microcomputer-based program was developed to perform these calculations and generate intersection performance measures. A sample screen of the program output is shown in Figure 4.

CONCLUSIONS AND RECOMMENDATIONS

This paper has presented an analytical signal-timing-based approach to determine left-turn volume warrants for two-phase pretimed intersections. The following conclusions are offered:

1. The proposed volume warrants were found to be consistent with existing models of left-turn capacity and warrants.
2. A simple volume-product warrant is not feasible when V_0 is less than 200 vph or when g/c is significantly different from 0.50.
3. The critical left-turn volumes derived from the TEXAS model appear to be unrealistically conservative.

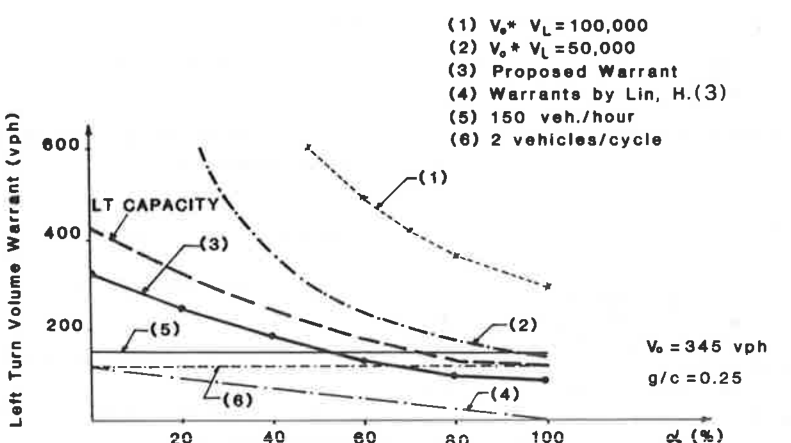

FIGURE 3 Left-turn volume warrants: a comparison at g/c = 0.25.

TABLE 1 Summary of Proposed Warrants

Phase	Movement	Condition	Left-Turn Volume Warrant (vph)
1	V_{01}, $V_{1\ell}$	$V_{01} < \frac{1440g_1}{C_o}$ (2)	$V_{1\ell} = V_{01}\left\{\left[0.82 - \frac{V_{01}/1750}{g_1/C_o}\right] * \left[\frac{1750 - V_{01}C_o/g_1}{1750 - V_{01}}\right] + \frac{4.11}{g_1}\right\}$ (3)
		$V_{01} \geq \frac{1440g_1}{C_o}$	$V_{1\ell} = \frac{4.11V_{01}}{g_1}$
1	$V_{\alpha 1\ell}$, (1) $\alpha_1 V_{01}$	$V_{01} < \frac{1440g_1}{\alpha_1 C_o}$	$V_{\alpha 1\ell} = V_{01}\left\{\left[0.82 - \frac{\alpha_1 V_{01}/1750}{g_1/C_o}\right] * \left[\frac{1750 - \alpha_1 V_{01}C_o/g_1}{1750 - \alpha_1 V_{01}}\right] + \frac{4.11}{g_1}\right\}$
		$V_{01} \geq \frac{1440g_1}{\alpha_1 C_o}$	$V_{\alpha 1\ell} = \frac{4.11V_{01}}{g_1}$
2	$V_{2\ell}$, V_{02}	$V_{02} < \frac{1440g_2}{C_o}$	$V_{2\ell} = V_{02}\left\{\left[0.82 - \frac{V_{02}/1750}{g_2/C_o}\right] * \left[\frac{1750 - V_{02}C_o/g_2}{1750 - V_{02}}\right] + \frac{4.11}{g_2}\right\}$
		$V_{02} \geq \frac{1440g_2}{C_o}$	$V_{2\ell} = \frac{4.11V_{02}}{g_2}$
2	$\alpha_2 V_{02}$, $V_{\alpha 2\ell}$	$V_{02} < \frac{1440g_2}{\alpha_2 C_o}$	$V_{\alpha 2\ell} = V_{02}\left\{\left[0.82 - \frac{\alpha_2 V_{02}/1750}{g_2/C_o}\right] * \left[\frac{1750 - \alpha_2 V_{02}C_o/g_2}{1750 - \alpha_2 V_{02}}\right] + \frac{4.11}{g_2}\right\}$
		$V_{02} \geq \frac{1440g_2}{\alpha_2 C_o}$	$V_{\alpha 2\ell} = \frac{4.11V_{02}}{g_2}$

(1) α_i = Ratio of lowest to highest through lane volume in phase (i).
(2) g_i = Effective green time in phase (i).
(3) $PCE_j = V_{oj}/V_{j\ell}$, $j = 1, 2, \alpha_1$ and α_2

```
                    R E S U L T S   F O R   P H A S E  1
                     CRITICAL LANE VOLUME = >> 500 VPH <<
C= 60   GREEN= 20  G/C= .3397436   VOPP= 500   DESIRED LT V/C RATIO = .7

  LEFT TURN CAPACITY--------> 175   OR APPROX   3   VEH PER CYCLE
  LEFT TURN WARRANT --------> 147   OR APPROX   3   VEH PER CYCLE
  WARRANT FROM DESIRED V/C--> 123   OR APPROX   3   VEH PER CYCLE
  P . C . E .    -----------> 3.381938
  EQUIVALENT THRU VOLUME =>> 499  VPH <<
  THRU LANE CAPACITY--------> 594
  DESIGN V/C RATIO  --------> .8409704

PRESS 1 TO CALCULATE DELAYS , 0 TO CONTINUE >
```

FIGURE 4 Sample screen output: signal timing and left-turn warrant program.

4. The proposed warrants are very sensitive to variations in the unopposed left-turn saturation flow rate. Other factors such as lost time and number of left turners per cycle were not significant.

5. Simple left-turn volume warrants are not sufficient indicators of the presence of left-turn problems. Supporting data regarding accidents, conflicts, delays, and sight distance restrictions must be considered before implementation.

6. The feasibility of applying the foregoing warrants to intersections with two or more opposing lanes needs to be addressed.

7. There is a need for developmental research on left-turn phasing warrants for coordinated signals.

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Accident Experience of Flashing Traffic Signal Operation in Portland, Oregon

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ABSTRACT

Traffic signals affect the safety and efficiency of traffic operations. Flashing-signal operation reduces delays during low-volume periods and may conserve energy. However, flashing operation has been found to affect the safety of the intersection adversely. The relative accident impacts of flashing-signal operation versus regular signal operation in the city of Portland are evaluated. Analyses were conducted to determine whether an increase in accidents occurred at the intersections when the control devices were operated in the flashing mode during low-volume nighttime hours. For the intersections studied, the accident levels, volume levels, intersection geometry, and speed and parking data were collected. A statistical analysis was made to determine the safety of flashing operation for intersections with various volume ratios, street classifications, types of approaches, approach speed limits, and parking conditions. Intersections at which the major-street volumes were more than twice the minor-street volumes experienced a significant increase in accidents when flashing operation was used. Significant increases in accidents were also found when flashing signals were installed at intersections with major-street approach speeds in excess of 30 mph. Accidents also increased with flashing operation when both streets were two-way and where parking was allowed on both streets. Accident severity increased for many situations, often because there was an increase in right-angle accidents.

Traffic signals affect the efficiency and safety of traffic operations. When traffic volumes are high, signals eliminate traffic conflicts by alternating the assignment of right-of-way. However, when traffic volumes drop substantially below the stated volume warrants for two or more consecutive hours, it may be desirable to replace the conventional signal for that period with a flashing signal (1). Flashing-signal operation reduces delays during low-volume periods and may conserve energy. The major argument for retaining 24-hr "full-color" operation may be that flashing operation may adversely affect the safety of the intersection.

This paper contains the summary of a research effort begun at Oregon State University in 1984 to analyze statistically the accidents experienced at 30 intersections in Portland, Oregon, at which the installation of flashing traffic signals was carried out in accordance with accepted guidelines. The specific objective of this study was to investigate the safety of use of flashing versus full-color

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operation at signalized intersections during low-volume hours.

Statistical tests were used to compare intersection safety performance with differing

- Volume ratios,
- Street classifications,
- Types of approaches,
- Approach speed limits, and
- Parking conditions.

The intersections were grouped on the basis of these variables. Before-and-after accident rates per million entering vehicles (MEV) for each group were calculated for the test of significance.

Experience has shown that the use of yellow or red flashing traffic signals at night has often resulted in an increase in accidents over that experienced under normal full-color operation. Both the Manual on Uniform Traffic Control Devices (MUTCD) (2) and the Traffic Control Devices Handbook (TCDH) (3) give criteria and guidelines for the safe operation of the flashing control signals. The MUTCD indicates that the major street should receive the yellow indication, and the flashing rate should be not less than 50 nor more than 60 times per minute. The TCDH gives criteria for flashing signal operation. These include

1. Unrestricted sight distance and low traffic volumes,
2. Monitored accident patterns and severity, and
3. Signal malfunctions, repairs, or maintenance.

In earlier editions of the MUTCD, a volume warrant of a 50 percent drop in volume for two or more consecutive hours justified the use of flashing operation (1).

In Oregon, the Department of Transportation has specified warrants for flashing-signal operation during low-volume periods based on volume, type of signal indication, and sight distance.

The most comprehensive review of the relative safety impacts of flashing versus regular signals is provided by an FHWA study (4) in which the results of several studies of the conversion of signal operations to flashing nighttime operation were evaluated. A primary result of the FHWA study was that the number of right-angle accidents was significantly higher at intersections when flashing signals were used than when regular signals were used. The two variables that had the most effect on accidents were two-way main-street volume in the first flashing hour and the ratio of main-street to side-street volume. This study recommended that flashing operation not be used if the main-street volume was more than 200 vehicles per hour (vph) unless the volume ratio was greater than 3.

Studies done by the Florida section of the Institute of Traffic Engineers (ITE) (5) and by Radelate (6) have also shown a significant increase in accidents with flashing-signal operation. However, operation in the flashing mode was found to conserve energy and reduce delay. Quan (7) found that if flashing operation were completely implemented in the city and county of San Francisco, there would be 514,000 vehicle-hr per year less delay, a saving of 450,000 gal of gasoline per year, and 10 percent conservation of electrical energy to power the signal operation.

ACCIDENT EXPERIENCE

Study Site

The city of Portland was selected for the field studies because of previous experience with night flashing traffic signals in 1981 and in early 1982. As a result of the increase in accidents, flashing operations were terminated in late 1983, and 30 intersections that had been changed from normal to nighttime flashing operation were returned to regular, full-color nighttime operation.

Data Collection

Before-and-after accident data were obtained from the computer records available from the Portland Bureau of Traffic Engineering. The before-and-after periods each encompassed either a 1- or 2-year interval, with an equal number of months in the study periods for all cases. The data were collected immediately before and immediately after the date of implementation of flashing-signal operation. In addition to accident data, information on traffic volumes, street classifications, types of approach, and speed limits was obtained.

Data Analysis

Maximum possible time frames were used in this study. All the intersections under study were grouped on the basis of the following variables:

1. Volume ratio: zero to twice as much volume on the major approach as on the minor (0.0 to 2.0), two to four times the volume on the major street (2.0 to 4.0), and more than four times greater major-street volume (greater than 4.0);
2. Street classification: an arterial intersecting with a collector, arterial with a local, collector with a local, collector with a collector, local with a local, arterial with a local or a collector;
3. Type of approach: two-way/two-way, two-way/one-way, one-way/one-way;
4. Speed limit: less than or equal to 30 mph or greater than 30 mph; and
5. Parking or no parking.

Accident data were split into the following categories for analysis:

1. Accident by type and
2. Accident by severity.

The before-and-after periods in the analysis were determined on the basis of actual date of installation and elimination of the flashing-signal system.

The volumes of the intersection approaches were used to obtain accident rates for both the before and the after periods for accident analyses. Intersection accident rates were calculated by using the following equation:

$$\text{Accident rate} = [(\text{no. of acc.})(1{,}000{,}000)] \div [(\text{volume})(\text{time})]$$

where

rate = accidents per million entering vehicles,
volume = entering volume (vehicles/day),
time = study period (days).

Before-and-after accident rates for each of the 30 intersections were calculated with this relationship. Each accident rate represents the average accidents per million vehicles passing through the intersection.

A measure of the relative severity of accidents was given by a severity index (SI), which is the