TRANSPORTATION RESEARCH RECORD 1086

Roadway Markings and Traffic Control in Work Zones

TRB
TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1986

Transportation Research Record 1086
Price $7.20
Editor: Lisette Chou
Compositor: Harlow A. Bickford
Layout: Theresa L. Johnson

mode
1 highway transportation

subject areas
34 general materials
54 operations and traffic control

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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

Roadway markings and traffic control in work zones.

(Transportation research record, ISSN 0361-1981; 1086)
Reports of TRB's 65th Annual Meeting, held in Washington, D.C., 1986.
1. Road markings. 2. Roads—Maintenance and repair—Safety measures. 3. Traffic signs and signals.
I. National Research Council (U.S.). Transportation Research Board. Meeting (65th : 1986 : Washington, D.C.) II. Series.
TE7.H5 no. 1086 380.5 s 86-31240
[TE203] [625.7'94]
ISBN 0-309-04106-6

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Contents

Reflectivity and Durability of Epoxy Pavement Markings

JAMES E. BRYDEN, RONALD A. LORINI, AND PETER D. KELLY

Epoxy pavement markings on 16 projects were surveyed to determine durability and reflectivity. These markings were up to 6-years old and were installed on both portland cement and asphalt concrete pavements. Most projects were in good condition and providing acceptable daytime delineation. Although most markings also had fair or good reflectivity, some were not providing acceptable reflectivity. However, most of the poor reflectivity occurred on a few recent projects. It was not possible to relate differences in condition or reflectivity to roadway characteristics, traffic, striping contractor, or material supplier, and it appears that these differences are attributable to particular characteristics of each installation.

In 1979, the New York State Department of Transportation (NYSDOT) adopted a policy requiring inclusion of durable pavement markings on most capital construction projects. Over the next few years, interpretation of this policy was broadened to include contract application of durable markings on highways not otherwise involved in capital work. These projects generally included high-volume Interstates, expressways, and other arterials where it was difficult to maintain year-round markings using traffic paints, as well as remote areas where it was not efficient to schedule periodic repainting. New York's striping policies and practices are explained at length in NYSDOT'S Research Report 112 *(1)*.

Performance of the first few major striping projects using durable materials (e.g., thermoplastic, two-component epoxy, and preformed tape) was described in Research Report 114 *(2)*. Over the first few years of this policy, about 15,000 mi of durable pavement markings were let to contract, with thermoplastic comprising about two-thirds of the total. Performance surveys on the thermoplastic markings were completed in 1981 and 1982, and the results published in Research Report 120 *(3)*.

By mid-1984 about 3,500 mi of epoxy lines had been let to contract, and more were anticipated. Therefore it became desirable to inspect a larger sample of epoxy markings installed over the past few years to determine performance characteristics of the material. Results of a survey conducted by personnel of the Engineering Research and Development Bureau during the summer of 1984 are summarized in this paper.

PROJECT DESCRIPTIONS AND METHODS OF EVALUATION

NYSDOT construction records were searched to identify projects including epoxy pavement markings completed by 1983. A total of 15 projects were selected for the survey, all striped between 1978 and 1983, including one additional contract striped in early summer of 1984. The 16 projects, summarized in Table 1, included about 1,100 mi of epoxy striping, about one-third of the total that had been let to contract by mid-1984.

Engineering Research and Development Bureau, New York State Department of Transportation, Albany N.Y. 12232.

Projects selected were located throughout the state, and included a wide range of highway and pavement types, traffic volumes, and environments. Four different striping contractors were employed, and material from three different suppliers was used. Project sizes ranged from about 8 mi to nearly 200 mi of striping. Some were limited to a single route, others included a large number of routes over a wide area.

Each project was inspected by a research team experienced in rating pavement-marking performance. Markings were subdivided by type (e.g., edge line, solid lane line, skip line, centerline, and median line) as well as by color and route. On projects including no more than a few routes, each combination of marking type, color, and route was inspected as an individual sample. On projects including several routes, several locations were selected for the survey. The number of samples ranged from as few as 2 to as many as 28 per project, with a total of 145 samples on the 16 projects.

The same set of observations was made for each of the 145 samples. Durability was noted based on subjective evaluations, and reflectivity was measured. The percentage of material remaining was estimated for each sample, and a subjective condition rating of good, fair, or poor was assigned:

1. Good: marking essentially new, with no more than minor imperfections or discolorations noticeable, and small areas of missing line.
2. Fair: marking still visually effective, but imperfections, discoloration, and worn or missing areas readily apparent.
3. Poor: marking marginally effective or ineffective, widespread imperfections, badly discolored, large areas missing.

Because each sample included a large quantity of marking material—sometimes over a long length of pavement—the range of percent remaining and condition was recorded, as well as the estimated overall percent remaining for the entire sample. Examples of various levels of percent remaining are shown in Figure 1.

Reflectivity was measured at 10 locations for each sample using a retroreflectometer built by the Engineering Research and Development Bureau and patterned after one built by the Michigan Department of State Highways and Transportation *(4)*. The instrument includes an internal light source and photocell, and provides a digital readout representing the brightness of a few square inches of line. It has been used to measure a number of lines at various levels of brightness to relate them to subjective visual readings. Typical brightness readings for sample plates constructed using several materials follow:

- New white Stamark reflective tape, 350;
- New yellow Stamark reflective tape, 260;
- White unbeaded paint, 80; and
- Yellow unbeaded paint, 50.

A panel of new white tape is used as a calibration reference to keep the instrument adjusted in the field. Instrument measurement

TABLE 1 SUMMARY OF PROJECTS SURVEYED

Project No.	Contract	NYSDOT Region	Year Striped	Contractor Code	Material Supplier Code	Pavement Type[a]	Total Samples	Length, 1,000 linear ft
1	D095864	1	1978-1979	1	1	P/A	7	166
2	D096536	1	1980	1	1	P	3	130
3	D096902	1	1982	2	2	P	12	708
4	D250238	8	1982	2	2	A	2	203
5	D250239	8	1982	2	2	A	10	302
6	D250240	8	1982	2	2	A	10	512
7	D250402	8	1982	2	2	A	3	85
8	D250482	1	1983	3	2	P	12	890
9	D250656	1	1983	4	3	P/A	28	795
10	D250501	2	1983	2	2	P/A	17	1,000
11	D250474	3	1983	4	2	P/A	4	457
12	D250197	5	1983	2	2	P	3	44
13	D250557	7	1983	2	2	A	2	127
14	D250500	9	1983	2	2	A	26	74
15	D000000	9	1983	2	2	P	3	100
16	D250159	5	1984	2	2	P	3	232

[a]P = Portland cement concrete pavement; A = asphalt concrete pavement.

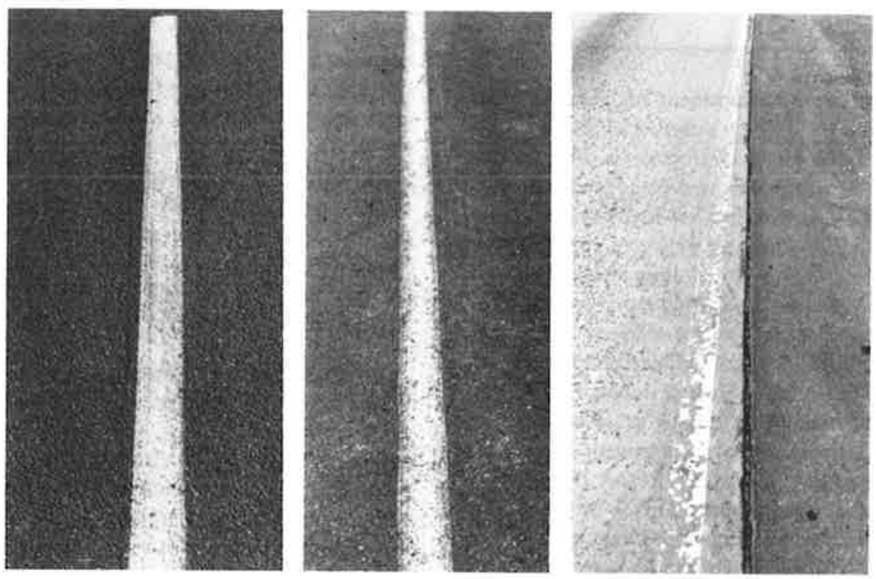

FIGURE 1 Epoxy markings rated 95 percent remaining (left), 65 percent (center), and 25 percent (right).

and subjective ratings have not been formally correlated, but based on several years of experience in subjectively rating marking materials, and 4 years of experience with the instrument, the following approximate relationships have been established:

	White	*Yellow*
Excellent	Over 300	Over 250
Good	225 to 300	175 to 250
Fair	140 to 225	110 to 175
Poor	Below 140	Below 110

More work is needed to define the relationship between measured brightness and a driver's perception of the pavement marking. However, during the interim period, these relationships provide a useful rule-of-thumb for assessing the adequacy of pavement markings, and for comparing alternative materials. Adequacy in terms of nighttime visibility also depends on pavement brightness (stripe-pavement contrast), roadway lighting, highway geometry, traffic speed and volume, and other factors. The measurements provided here are intended only as an assessment of the inherent visibility of the material. In some situations (such as lighted highways or on pavements providing dark background contrast), a white stripe with a brightness measurement of 125 may be adequate, and in others 175 may be required.

Survey data were computerized for subsequent tabulation and analysis. Summaries were generated to examine the overall condition of the markings, and various parameters (e.g., roadway, traffic, environment, etc.) were related to performance. Appropriate statistical tests were used in some cases to determine whether perceived differences in performance were statistically significant.

RESULTS

Observations for each of the 145 samples included in this survey are given in Table 2 and summarized in Table 3. Overall, most markings were in fair to good condition, and were providing good

TABLE 2 SUMMARY OF SURVEY RESULTS

Project No.	Route[a]	Lane Width and Pavement Type[a]	Shoulder Width[c]	AADT 1,000	Mark Type	Measured Brightness Average	Measured Brightness Low	Measured Brightness High	Percent Remaining Average	Percent Remaining Low	Percent Remaining High	Subject Rate	Failure Mode[d]
1	WA	4-12C	10	17.4	WEdge	213	125	358	20	2	50	11	CP
	WA	4-12C	10	17.4	YMed	247	191	309	85	70	90	33	CP
	WA	4-12C	10	17.4	WSkip	256	147	326	75	60	70	32	CA
	FR	4-10C	0	23	YCent	136	112	161	85	80	90	32	C
	20	4-12A	0	20.2	WEdge	262	222	288	65	60	80	22	CA
	20	4-12A	0	20.2	WSkip	163	136	190	75	50	70	32	CA
	20	4-12A	0	20.2	YCent	149	140	156	80	70	80	33	CA
2	188	4-12C	6	5.9	WEdge	345	238	435	80	70	90	32	CP
	188	4-12C	6	5.9	WSkip	378	235	462	75	70	80	32	CP
	188	4-12C	6	5.9	YMed	230	183	270	85	70	90	33	CP
3	I90	6-12C	10	49.7	WEdge	219	171	316	80	70	80	32	C
	I90	6-12C	10	49.7	WSkip	250	216	286	75	60	80	32	C
	I90	6-12C	10	49.7	YMed	222	202	299	85	70	90	33	C
	I787	6-12C	10	28.3	WEdge	428	373	465	80	70	80	32	C
	I787	6-12C	10	28.3	WSkip	266	252	282	70	50	80	32	C
	I787	6-12C	10	28.3	YMed	147	128	169	85	70	90	33	C
	9	4-12C	10	15.3	WEdge	358	293	423	80	70	90	33	C
	9	4-12C	10	15.8	WSkip	394	365	458	75	70	80	32	C
	9	4-12C	10	15.8	YMed	297	258	323	85	70	90	33	C
	85	4-12C	10	19.3	WEdge	289	156	484	80	70	80	32	C
	85	4-12C	10	19.3	WSkip	333	290	400	75	70	80	32	C
	85	4-12C	10	19.3	YMed	269	246	298	85	70	90	33	C
4	9H	2-12A	3	3.3	WEdge	318	232	436	85	70	90	33	AP
	9H	2-12A	3	3.3	YCent	142	93	231	75	60	80	32	AP
5	17A	2-12A	4	4.1	WEdge	209	129	290	80	60	80	32	AP
	17A	2-12A	4	4.1	YCent	172	160	193	80	70	80	32	AP
	17	4-12A	4	16.2	WEdge	205	132	245	75	60	80	32	A
	17	4-12A	4	16.2	WSkip	157	128	188	65	50	70	22	A
	17	4-12A	4	16.2	YCent	166	143	190	80	60	80	32	A
	59	2-12A	0	18.6	YCent	91	80	106	70	50	70	22	A
	94	2-12A	4	4.5	WEdge	199	147	255	80	60	80	32	A
	94	2-12A	4	4.5	YCent	127	90	155	75	60	80	32	A
	218	2-11A	3	3.7	WEdge	285	226	353	80	60	90	32	A
	218	2-11A	3	3.7	YCent	192	155	226	75	50	80	32	A
6	32	2-11A	2	5.6	WEdge	190	120	274	65	50	70	21	AP
	32	2-11A	2	5.6	YCent	91	62	108	65	50	70	21	AP
	32	2-11A	2	5.6	WEdge	161	140	219	70	50	70	22	AP
	32	2-11A	2	5.6	YCent	147	130	177	80	70	80	32	AP
	209	2-12A	6	6.1	WEdge	204	121	325	75	60	80	32	A
	209	2-12A	6	6.1	YCent	131	106	158	80	70	90	33	A
	17K	2-11A	4	7.9	WEdge	150	114	207	70	60	70	22	A
	17K	2-11A	4	7.9	YCent	181	156	210	80	70	80	33	A
	17K	2-11A	9	3.1	WEdge	204	143	302	80	70	80	33	A
	17K	2-11A	9	3.1	YCent	147	122	198	80	70	80	33	A
7	295	2-12A	2	2.6	WEdge	391	286	467	80	70	80	33	A
	295	2-12A	2	2.6	WSkip	248	228	268	75	70	80	32	A
	295	2-12A	2	2.6	YCent	228	184	260	80	70	80	33	A
8	I787	6-12C	10	28.3	WEdge	260	202	310	80	70	90	33	CA
	I787	6-12C	10	28.3	WSkip	301	244	352	75	60	80	32	CA
	I787	6-12C	10	28.3	YMed	203	164	244	85	80	90	33	CA
	I90	6-12C	10	26.2	WEdge	294	203	330	80	70	80	32	C
	I90	6-12C	10	26.2	WSkip	295	253	367	70	60	80	32	C
	I90	6-12C	10	26.2	YMed	175	155	202	85	70	90	33	C
	I87	6-12C	10	39.8	WEdge	211	189	264	75	70	80	32	CA
	I87	6-12C	10	39.8	WSkip	266	209	313	65	50	80	22	CA
	I87	6-12C	10	39.8	YMed	218	148	255	80	70	90	33	CA
	I890	4-12C	6	8.9	WEdge	322	245	376	85	70	90	33	C
	I890	4-12C	6	8.9	WSkip	267	163	414	75	60	90	32	C
	I890	4-12C	6	8.9	YMed	179	101	289	85	70	90	33	C
9	5	4-12A	0	28.9	WLane	173	136	253	75	60	80	32	A
	5	4-12A	0	28.9	YMed	126	98	158	85	70	90	33	A
	20	4-12A	0	16.9	WEdge	232	187	262	85	70	90	33	A
	20	4-12A	0	16.9	WSkip	175	140	202	75	60	80	32	A
	20	4-12A	0	16.9	YCent	160	103	225	85	70	90	33	A
	WR	4-14A	0	24.5	WLane	186	144	209	70	60	70	22	A
	WR	4-14A	0	24.5	WSkip	153	127	191	70	60	80	32	A
	WR	4-14A	0	24.5	YMed	155	97	248	80	70	80	33	A
	5	4-12A	4	6	WEdge	301	261	340	80	70	90	33	A
	5	4-12A	4	6	WSkip	282	190	352	70	60	80	32	A
	5	4-12A	4	6	YMed	240	174	295	80	70	90	33	A
	9	4-12A	0	18.3	WEdge	249	204	301	80	70	80	33	A
	9	4-12A	0	18.3	WSkip	215	163	260	70	50	70	32	A
	9	4-12A	0	18.3	YCent	169	141	205	80	70	80	33	A
	378	4-12C	4	11.1	WEdge	319	212	383	30	0	80	31	C
	378	4-12C	4	11.1	WSkip	247	171	350	20	0	70	31	C
	378	4-12C	4	11.1	YMed	227	186	280	60	0	80	31	C
	377	4-12A	8	6.7	WEdge	340	252	428	85	70	90	33	A
	377	4-12A	8	6.7	WSkip	266	242	296	85	70	80	33	A
	377	4-12A	8	6.7	YCent	259	201	294	85	70	80	33	A
	32	4-12A	0	10.6	WSkip	171	134	212	75	60	80	32	A
	32	4-12A	0	10.6	YCent	111	92	230	80	70	80	33	A
	GIBR	4-12C	0	–[e]	WEdge	280	218	324	85	80	90	33	A
	GIBR	4-12C	0	–[e]	WSkip	187	160	231	80	80	90	33	A

TABLE 2 (continued)

Project No.	Route[a]	Lane Width and Pavement Type[b]	Shoulder Width[c]	AADT 1,000	Mark Type	Measured Brightness			Percent Remaining			Subject Rate	Failure Mode[d]
						Average	Low	High	Average	Low	High		
	GIBR	4-12C	0	—[e]	YMed	170	151	191	85	80	90	33	A
	32	4-12C	10	6.5	WEdge	496	456	532	75	60	80	31	C
	32	4-12C	10	6.5	WSkip	284	262	363	70	60	80	21	C
	32	4-12C	10	6.5	YMed	168	122	211	75	70	80	31	C
10	173	2-10A	6	2	WEdge	209	127	350	75	60	80	32	CP
	173	2-10A	6	2	YCent	107	75	164	80	70	80	32	CP
	13	2-12A	5	4.3	WEdge	252	182	358	85	70	80	33	A
	13	2-12A	5	4.3	YCent	118	68	182	80	70	80	33	A
	5S	2-12C	6	4.6	WEdge	336	221	423	75	60	80	32	C
	5S	2-12C	6	4.6	YCent	177	103	211	80	70	80	32	C
	49	2-10A	3	1.8	WEdge	227	164	281	80	70	80	33	AP
	49	2-10A	3	1.8	YCent	115	85	166	80	70	80	33	AP
	12	2-11A	5	3.4	WEdge	204	114	398	80	70	80	33	A
	12	2-11A	5	3.4	YCent	80	65	105	80	70	80	33	A
	92	2-12A	10	5.6	WEdge	226	187	278	80	70	90	33	A
	12	2-12A	10	5.6	WSkip	145	138	155	75	60	80	32	A
	92	2-12A	10	5.6	YCent	133	100	167	85	70	90	33	A
	30	2-11A	5	1.2	WEdge	263	213	301	80	70	80	33	AP
	30	2-11A	5	1.2	YCent	121	96	176	75	60	80	32	AP
	8	2-10A	2	.4	WEdge	354	307	392	80	70	90	33	AP
	8	2-10A	2	.4	YCent	140	115	166	75	40	80	32	AP
11	3	2-12C	10	4.1	WEdge	289	175	414	80	70	90	33	C
	3	2-12C	10	4.1	YCent	115	100	140	85	80	90	33	C
	370	2-12A	6	3	WEdge	255	171	383	80	70	90	33	A
	370	2-12A	6	3	YCent	149	106	196	85	80	90	33	A
12	I990	6-12C	10	5	WEdge	379	221	545	80	60	90	33	CP
	I990	6-12C	10	5	WSkip	371	242	446	80	60	90	32	CP
	I990	6-12C	10	5	YMed	214	159	266	85	70	90	33	CP
13	12	2-12A	8	3.4	WEdge	340	314	377	85	80	90	33	A
	12	2-12A	8	3.4	YCent	109	97	117	85	70	90	33	A
14	17	2-12A	5	12.6	WEdge	156	126	179	80	70	90	33	A
	17	2-12A	5	12.6	YCent	99	80	117	80	70	90	33	A
	MCLO	2-12A	0	—[e]	WEdge	213	163	275	80	70	80	33	A
	MCLO	2-12A	0	—[e]	YCent	110	100	126	80	70	80	33	A
	MCLO	2-14A	0	—[e]	WEdge	220	185	269	80	70	80	32	A
	MCLO	2-14A	0	—[e]	YCent	80	74	89	75	60	80	32	A
	VSTL	2-14A	0	—[e]	WEdge	304	208	350	80	70	80	33	A
	VSTL	2-14A	0	—[e]	YCent	91	73	109	75	60	80	32	A
	VSTL	2-13A	0	—[e]	YCent	62	57	69	75	60	80	22	A
	UNON	2-12A	0	—[e]	WLane	202	181	235	70	60	80	22	A
	UNON	2-12A	0	—[e]	YCent	87	78	94	75	60	80	22	A
	UNON	2-12A	0	—[e]	WLane	111	106	117	75	60	80	22	A
	UNON	2-12A	0	—[e]	YCent	64	58	69	75	60	80	22	A
	JONC	4-12A	0	—[e]	WLane	159	139	194	75	60	80	22	A
	JONC	4-12A	0	—[e]	WSkip	192	173	207	70	50	80	22	A
	JONC	4-12A	0	—[e]	YMed	90	79	114	80	70	80	32	A
	JONC	4-12A	0	—[e]	YCent	97	87	107	80	60	80	32	A
	BING	2-12A	0	—[e]	WLane	130	118	137	80	70	80	32	A
	BING	2-12A	0	—[e]	YCent	77	57	108	75	50	80	22	A
	BING	2-12A	0	—[e]	YCent	89	79	101	80	60	90	32	A
	ONTA	2-12A	0	—[e]	WLane	101	91	112	70	50	70	22	A
	ONTA	2-12A	0	—[e]	WSkip	79	70	97	70	50	70	22	A
	ONTA	2-12A	0	—[e]	WLane	74	70	79	70	50	70	22	A
	ONTA	2-12A	0	—[e]	YCent	50	46	54	70	50	70	22	A
	NOWH	2-13A	0	—[e]	YCent	78	65	86	60	0	70	21	A
	NOWH	2-12A	0	—[e]	YCent	103	81	131	80	70	80	32	A
15	I88	4-12C	12	4.7	WEdge	391	277	464	80	70	80	33	AP
	I88	4-12C	12	4.7	WSkip	360	227	448	75	70	80	32	AP
	I88	4-12C	12	4.7	YMed	312	280	358	80	70	90	33	AP
16	I290	6-12C	10	50	WEdge	315	233	378	80	70	90	33	CP
	I290	6-12C	10	50	WSkip	220	185	260	80	60	90	32	CP
	I290	6-12C	10	50	YMed	238	184	274	85	70	90	33	CP

[a] Striping areas that are not numbered state routes were located on: Washington Avenue (WA), Fuller Road (FR), Wolf Road (WR), Green Island Bridge (GIBR), Vestal (VSTL), Monticello (MCLO), Union (UNON), Johnson City (JONC), Binghamton (BING), Oneonta (ONTA), and Norwich (NOWH).
[b] Shows number of lanes, width, and type of pavement (C = concrete, A = asphalt).
[c] 0 = curbed section without shoulder.
[d] C = chipping, A = abrasion, P = pavement deterioration.
[e] No data.

daytime delineation as shown in Figure 2. All 145 samples experienced some material loss, with most in the range of 70 to 90 percent intact. Only 26 samples were less than 70 percent intact. Most failure was in the form of small areas of missing line, with only occasional areas of more widespread failure. Abrasion failure caused by traffic and snowplow wear was the prevalent failure mode encountered, although chipping failures (loss of adhesion) were observed in a few cases. Some striping failure was also caused by failure of the pavement itself, either by deterioration of the pavement along joints and cracks, or by loss of peaks on rough-textured pavements (Figure 3) probably caused by snowplowing. Typical examples of marking failure are shown in Figure 1.

Only one sample was rated poor overall for appearance, with nine more rated poor to fair or poor to good. Therefore, nearly all the samples were in the fair and good ranges and provided adequate daytime delineation. Some graying of white markings was apparent on most projects, but the markings were still considered adequate for daytime delineation.

TABLE 3 MARKING CONDITION RELATED TO PAVEMENT TYPE

		Number of Samples for Each Marking Type				
Variable	Total Samples	Right Edge (White)	Skip (White)	Solid Lane (White)	Center-line (Yellow)	Median Edge (Yellow)
Percent remaining						
50 or less	3	2	1	0	0	0
51-70	23	4	10	4	4	1
71-90	119	41	17	4	38	19
Over 90	0	0	0	0	0	0
Total	145	47	28	8	42	20
Condition						
Poor	1	1	0	0	0	0
Poor-fair	4	1	1	0	2	0
Poor-good	5	2	1	0	0	2
Fair	19	3	4	6	6	0
Fair-good	53	14	20	2	16	1
Good	63	26	2	0	18	17
Total	145	47	28	8	42	20
Reflectivity						
Poor	24	0	1	4	18	1
Fair	55	16	10	4	19	6
Good	39	14	11	0	4	10
Excellent	27	17	6	0	1	3
Total	145	47	28	8	42	20

FIGURE 2 Typical epoxy markings in good condition.

Marking reflectivity was generally not as good as overall durability. Less than half the average brightness values were good or excellent, with 24 samples in the poor range and 55 more only fair. Considering the range of reflectivity values often observed within samples, even more had some unacceptable brightness values. In all, 55 of 145 samples had one or more poor reflectivity measurements. However, most samples with poor reflectivity were concentrated on a few projects, and most remaining projects had few or no samples with areas of poor reflectivity.

Table 3 also relates condition to marking type; skip lines and solid lane lines experienced substantially more material loss than edge lines and centerlines. Because these stripes are more exposed to traffic forces, it is expected that they would experience greater wear. The solid lane lines again were rated somewhat below the others in terms of subjective condition ratings, with 6 of 8 samples rated only fair. In terms of reflectivity, solid lane lines and centerlines performed much poorer than the others. Only 5 of 42 centerline samples and no solid lane-line samples had good or excellent reflectivity, but about two-thirds of the other types had good or excellent reflectivity. However, most of the poor and fair reflectivity ratings occurred on a few projects that consisted primarily of centerlines and solid lane lines, with no lines of the other three types. If these few projects were disregarded, it does not appear that any marking type performed very differently from others in terms of reflectivity.

Table 4 relates condition to marking color and pavement type. Because marking color and type are interdependent, trends observed for marking types would also be expected when the data are stratified by color. White markings experienced significantly more material loss than the yellow, but in terms of subjective condition, yellow markings were rated only slightly better than the white, and the difference is not significant. This same trend was seen when results were stratified by marking type, the skip lines and solid edge lines, both white, experienced the most material loss. Reflectivity of white lines was significantly better than

FIGURE 3 Loss of epoxy striping caused by chipping of peaks on tine-textured concrete pavement.

TABLE 4 MARKING CONDITION RELATED TO PAINT COLOR AND PAVEMENT TYPE

Variable	Marking Color		Pavement Type	
	White	Yellow	Asphalt	Concrete
Percent remaining				
50 or less	3	0	0	3
51-70	18	5	18	5
71-90	62	57	74	45
Over 90	0	0	0	0
Condition				
Poor	1	0	0	1
Poor-fair	2	2	3	1
Poor-good	3	2	0	5
Fair	13	6	18	1
Fair-good	36	17	32	21
Good	28	35	39	24
Reflectivity				
Poor	5	19	24	0
Fair	30	25	44	11
Good	25	14	16	11
Excellent	23	4	8	19
Total	83	62	92	41

yellow; this trend was apparent on most projects and not limited to a few worst cases.

No significant differences in percent remaining or subjective condition were found between pavement types; the minor differences apparent in Table 4 are not statistically significant. However, markings on concrete pavement had significantly better reflectivity, on the whole, than those on asphalt. About 80 percent of the markings on concrete had good or excellent reflectivity, compared to only about 25 percent on asphalt.

Epoxy is considered a long-life marking material, and these markings are expected to provide several years of satisfactory service before gradually failing through traffic wear. Accordingly, newer projects would be expected to be in better condition and to have better reflectivity than older ones. Table 5 relates marking condition to age. When samples striped in 1983 and 1984 are compared to older samples, no advantage is seen for the new markings either in terms of percent remaining or reflectivity. The older samples—up to 6-years old when inspected—are in as good condition as those 1-year old.

Individual project results reveal that highest line loss on the 1983-1984 projects is concentrated primarily on two of the nine projects (9 and 14). However, on the older projects, four of seven projects have some samples with high loss. Therefore, heavy wear is more widespread on older projects than on newer ones. Disregarding those two projects with high losses, the 1983 and 1984 projects actually have significantly less material loss than the older ones. The 1983 Project 14 is similarly responsible for nearly all of the poor average reflectivity ratings encountered on newer projects. However, some individual measurements in the poor range were encountered on five of nine 1983 and 1984 projects, even though average values for all but Project 14 were fair or better.

Material supplier and striping contractor were examined to determine whether differences in durability and reflectivity might be related to these variables. No significant differences were found among the three material suppliers in terms of either percent remaining or reflectivity. One contractor had significantly more low reflectivity values than the other three, but most of those poor and fair ratings were recorded on two 1983 projects, and several other projects striped by the same contractor had acceptable reflectivity. No significant differences in percent remaining were seen among the four contractors.

In addition to pavement age, traffic volume also has a direct effect on total wear experienced by a pavement marking. Traffic volumes were available for most of the 145 samples and were examined to determine whether they related to marking condition. However, no trends were apparent relating traffic volume to marking condition or reflectivity. Total annual average daily traffic (AADT), lane AADT, and total lane traffic over the life of the markings were all examined, but none showed a significant relationship to marking condition. In general, samples that had low total traffic exposure were in no better condition than markings exposed to high total traffic volumes.

Pavement and shoulder widths affect lateral vehicle placement, thereby affecting the number of vehicles actually crossing over the markings. These two parameters were examined to determine whether they related to marking durability. Unfortunately, only 21 of 145 samples had pavement widths less than 12 ft, and the total range was only 10 to 14 ft. Samples on pavement widths narrower than 12 ft performed almost exactly the same as those on wider pavements in terms of percent of material remaining. The narrower pavements had slightly more markings with poor and fair reflectivity values, but the small difference was not statistically significant.

The effects of shoulder width are summarized in Table 6. Samples with shoulder widths of 5 ft or less had significantly more low ratings—in terms of both percent remaining and reflectivity—than samples with wider shoulders. In terms of percent remaining, the narrow-shoulder group had about 25 percent rated poor or fair, compared to less than 10 percent for the wide-shoulder group. In terms of reflectivity, the difference was even greater; only about one-third of the narrow-shoulder group had good reflectivity, compared to about two-thirds for the wide-shoulder group. However, this cannot be interpreted as a cause-effect relationship. Even though there is a significant association between narrow shoulders and lower reflectivity, most low-reflectivity values were concentrated on a few projects that also had several samples with narrow shoulders.

TABLE 5 MARKING CONDITION RELATED TO AGE

Year Striped	Total Samples	Number of Samples							
		Percent Remaining				Reflectivity			
		≤50	51-70	71-90	>90	Poor	Fair	Good	Excellent
1978	2	1	0	1	0	0	1	1	0
1979	5	0	1	4	0	0	2	3	0
1980	3	0	0	3	0	0	0	1	2
1982	37	0	7	30	0	2	18	9	8
1983	95	2	15	78	0	22	32	25	16
1984	3	0	0	3	0	0	1	1	1

TABLE 6 MARKING CONDITION RELATED TO SHOULDER WIDTH

Shoulder Width, ft	Total Samples							
	Average Brightness				Percent Remaining			
	Poor	Fair	Good	Excellent	≤50	51-70	71-90	>90
0-2	2	18	5	3	0	7	21	0
3-5	2	14	10	3	2	4	23	0
6-9	2	8	6	7	0	1	22	0
10-12	0	10	17	18	1	4	36	0

DISCUSSION AND FINDINGS

Most of the 16 projects surveyed appear to be providing adequate service in terms of durability and reflectivity. However, considerable material loss and reduced reflectivity were experienced on several projects, and a few projects were performing poorly. Table 7 summarizes overall performance for each of the 16 projects. Twelve of 16 were rated in mostly good physical condition, but 10 are not providing good overall reflectivity.

TABLE 7 SUMMARY OF MARKING CONDITION

Project No.	Year Striped	Overall Rating[a]	
		Condition	Reflectivity
1	1978-1979	2	2
2	1980	3	3
3	1982	3	3
4	1982	3	2
5	1982	3	1
6	1982	1	1
7	1982	3	3
8	1983	3	3
9	1983	2	2
10	1983	3	1
11	1983	3	2
12	1983	3	3
13	1983	3	1
14	1983	2	0
15	1983	3	3
16	1984	3	2

[a]3 = mostly good ratings, 2 = less than half fair or poor ratings, 1 = mostly fair, poor ratings or both, and 0 = many poor ratings.

Several parameters were examined to determine whether possible causes of poor performance could be identified. A few trends were found that were statistically significant, but most appear to be related to poor performance on a small number of projects rather than being causal in nature. Thus, more projects must be surveyed to determine relationships that may be useful in predicting performance of epoxy pavement markings on a long-term basis. If the poor performance observed on a few projects in this survey can be related to construction practices or other parameters, it is important to identify those causes so that they can be remedied on future epoxy striping contracts.

Based on this survey of 16 epoxy striping projects, the findings that follow appear to be warranted:

1. Most projects inspected were in fair to good condition. While some striping material has been lost, most still provide an acceptable level of daytime delineation.
2. Most projects surveyed provide fair to good reflectivity, but about one-third provided marginal or unacceptable reflectivity. Most markings with marginal or unacceptable reflectivity were located on only a few projects, some only 1-year old.
3. Several parameters appeared to be associated with increased wear or lower reflectivity, but these apparent trends may be related to poor performance noted on a few projects, which introduced bias into the analysis.
4. A larger survey of epoxy markings in service over a longer period is needed to identify causes of the marginal performance noted on a few projects.

ACKNOWLEDGMENTS

This work was conducted in cooperation with the FHWA, U.S. Department of Transportation. Engineering Research and Development Bureau technicians assisting in the study included David J. Leininger and Richard D. Wright. The authors also wish to thank David R. Brewster and Harry P. Sloan of the New York State Department of Transportation's Materials Bureau for their help in providing project data.

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Publication of this paper sponsored by Committee on Coatings, Signing and Marking Materials.

Evaluation of Temporary Day-Night Visible Raised Pavement Marker Adequacy

THOMAS D. DAVIS

Compared to paint, day-night visible raised pavement markers improve construction-zone traffic performance significantly. However, there is a need to find temporary markers that can withstand construction-zone traffic. After initial screening, six marker types were tested further for visibility and durability. The features that met the criteria for an adequate day-night visible temporary marker installed with butyl on primer are a streamlined profile, a microscopic cube-corner, sealed prismatic air cell, cube-corner reflex, or multiple-glass lens reflector, and a balance between the reflector area and casing area exposed to the driver. Two marker systems met these requirements: (a) a hollow acrylic marker with a sealed prismatic air cell reflector such as the Stimsonite 66B by the Amerace Corporation; and (b) a combination of a domed-shaped polyester marker such as the Titan TM-40 by the Traffic Safety Supply Company for day visibility, and a filled ABS shell marker with a cube-corner reflex reflector such as the Ray-o-Lite by I.T.L. Industries, Inc., or equivalent for night visibility.

Compared to paint, day-night visible raised pavement markers improve construction-zone traffic performance significantly. However, until recently, the only day-night visible marker available was the ceramic marker. Ceramic markers are designed for permanent installation with epoxy adhesives. In construction zones, a butyl adhesive is used for pavement markers to permit easy removal. However, the combination of heavy, weaving traffic and butyl adhesives caused the markers to break up, especially on concrete pavements. Even when panel adhesives were used, the results were much the same: the ceramics broke up or came off.

BACKGROUND

A recent report by Davis *(1)* showed that raised reflective ceramic markers proved to be day visible and provided night and wet-pavement lane delineation superior to that provided by paint. The markers decreased lane changes and night lane encroachments. Unfortunately, 25 percent of the markers were lost or damaged within 6 days while under traffic. The markers were attached to the pavement with butyl adhesives. Losses were probably due to a combination of heavy, weaving traffic and the butyl adhesives. Another experiment in the same report showed that ceramic marker losses were 2 times higher on concrete than on asphalt pavements. Also, a commonly used plastic marker (not day visible) experienced a 29 percent failure rate, but the ceramic marker experienced a 79 percent failure rate, even with other one-step, panel-type adhesives.

Other reports related similar experiences with ceramic markers. Niessner reported: "On one project nearly 80 percent of the ceramic markers placed with the butyl pads were missing or damaged in 4.5 months. The contractor had used this detour to bring all of his heavy equipment onto and off of the construction site. Combined with the narrow 9-ft lanes on the detour, this may partly explain the poor performance of the ceramic markers. The damage and loss record for this location seems to indicate that a butyl pad does not afford the ceramic markers with the support and adhesive properties it requires" *(2)*.

Lanz reported on a permanent (nonconstruction) installation of ceramic markers. Although the 5 to 10 percent overall annual loss may seem relatively small, he noted that: "Many of these markers failed because of poor bond. Traffic is detrimental to ceramic markers in curves and in areas with much lane crossing. Replacement of ceramic markers is necessary in several locations where up to 50 percent are missing in 0.5-mi stretches" *(3)*. The markers were installed with epoxy adhesives.

This project described here was designed to select a durable temporary day-night visible marker system compatible with butyl adhesives that hold the marker in place during construction and permit easy removal once construction is complete.

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PROCEDURE

A review of the various procedures and criteria used to select adequate day-night markers follows:

1. Fourteen manufacturers were solicited for samples of day-night visible raised pavement markers. The markers had to be easy to install and present no hazard to drivers. A night reflective marker was assumed to also be wet-night reflective. The six marker types given in Table 1 and shown in Figures 1 through 6 were judged acceptable for further testing.
2. In April 1984, the six marker types were installed in a parking lot for approximately 1 month and run over at random. They did not crack, turn, or fall off, and they were removable with a shovel. Five marker types were attached with butyl, and the Swareflex type was installed with a thermoplastic.
3. The new candidate markers were tested for visibility on an unopened section of I-78, which was asphalt, and on I-295, which was concrete. Thirty-three of each type of marker were installed at 6-ft intervals side by side as shown in Figures 7 and 8. Eight observers independently judged the visibility of the markers from a car with the sun behind, the sun in front, and at night and recorded their opinions on a questionnaire as to the distance from which they could detect each marker.
4. Following the visibility study, the markers were installed at six sites with various pavement and traffic characteristics given in Table 2. In late May and early June, the six marker types were

TABLE 1 MARKER DESCRIPTION

Company	Model	Body	Reflector
Ferro Corporation	P-15	Ceramic	Microscopic cube-corner
Amerace Corporation	Stimsonite 66B	Hollow acrylic shell	Sealed prismatic air cell
I.T.L. Industries, Inc.	Ray-o-Lite	High-impact ABS	Cube-corner reflex
Traffic Safety Supply Company	Titan TM-40	Polyester	None
D. Swarovski and Company	Swareflex 3557	ABS shell	43 glass reflectors
Olympic Machines, Inc.	44C	Polymer	Single-glass lens

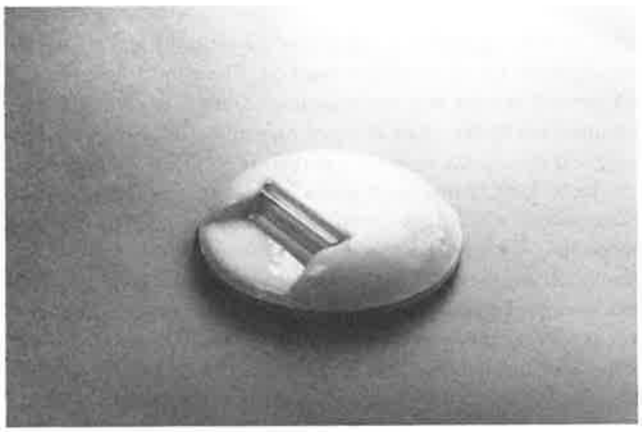

FIGURE 1 Ferro Corporation P-15.

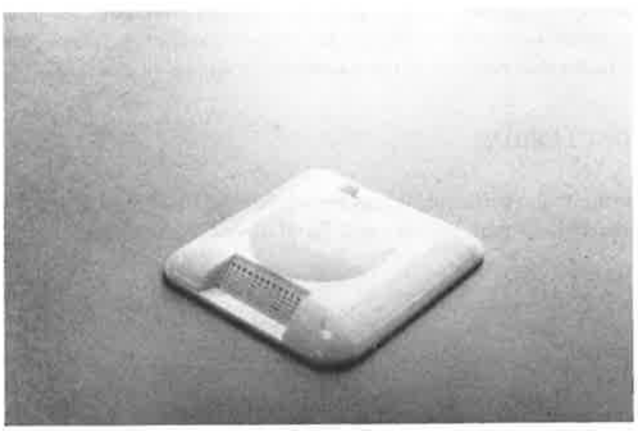

FIGURE 2 Amerace Corporation Stimsonite 66B.

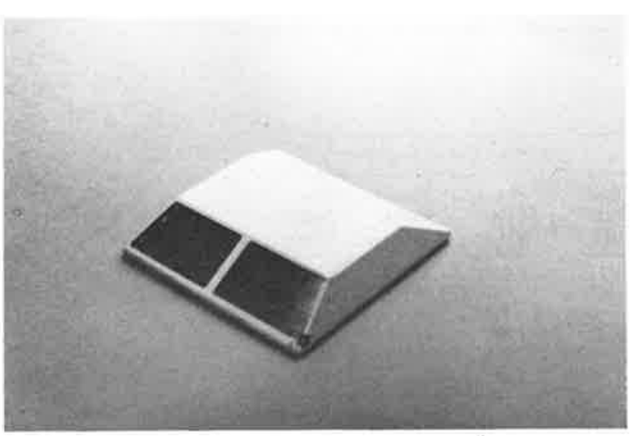

FIGURE 3 I.T.L. Industries, Inc., Ray-o-Lite.

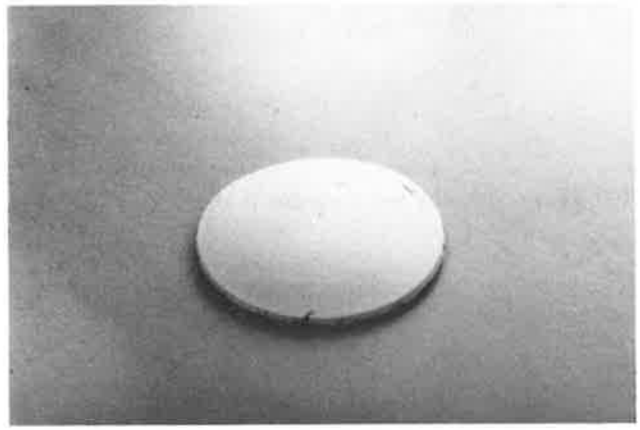

FIGURE 4 Traffic Safety Supply Company Titan TM-40.

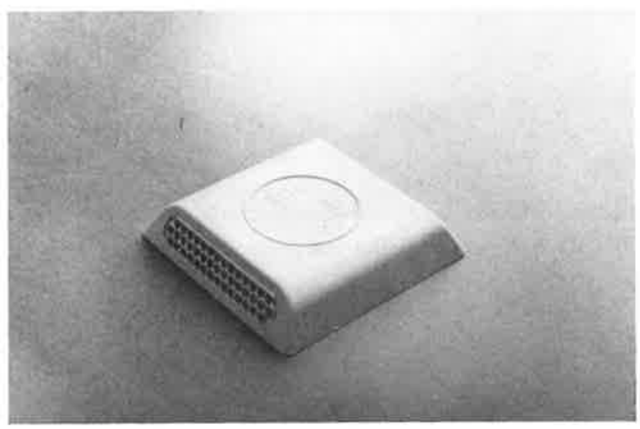

FIGURE 5 D. Swarovski and Company Swareflex 3557.

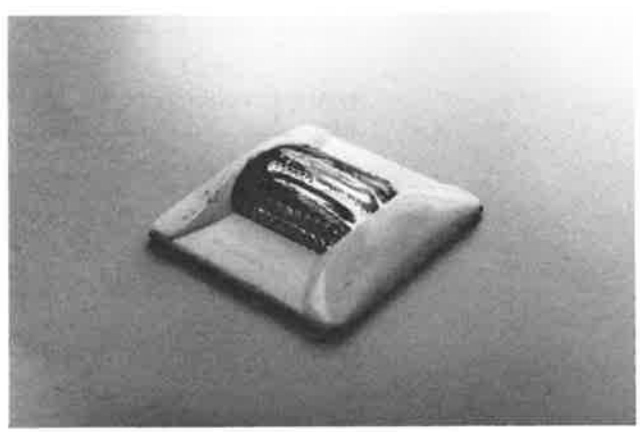

FIGURE 6 Olympic Machines, Inc., 44C.

FIGURE 7 Visibility test from 200 ft away.

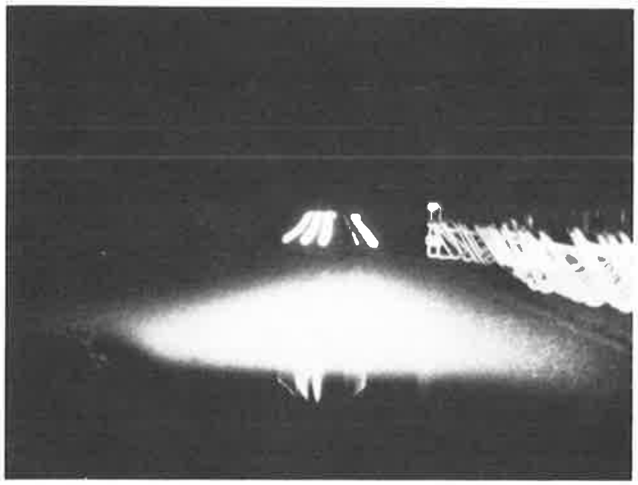

FIGURE 8 Night visibility test from 200 ft away.

randomly installed between skip lines at each site. For 6 months, the markers were monitored at 12 periodic intervals for cracks, losses, turns, slips, and dirt.

5. Following the 6-month durability test, the markers were completely removed with snowplows without noticeable damage to the pavement.

6. Finally, the worn markers were judged for visibility in the same manner as in Step 3.

RESULTS

Tables 3 and 4 summarize the results of the visibility and durability tests applied to the six candidate markers. The Ferro Corporation's P-15 ceramic marker was the control used in the experiment. The recommended system should equal or surpass the ceramic's visibility and exceed the ceramic's durability.

In Table 3, eight observers made independent decisions on the distance from which they could detect each marker array under various light conditions and with asphalt and concrete pavements. The asphalt and concrete site data were combined. The 16 observations were averaged to produce the distances in the table. After 6 months of wear, the markers were removed from the durability test locations. Unfortunately, the Stimsonite markers came off the pavement in fragments. Therefore, nine of each marker type were used in the final visibility survey. The fact that the Stimsonite markers had to be fitted together again no doubt lowered the visibility results from what they would have been had the markers remained intact on removal. The other markers were removed intact from the pavement. Significant changes were detected with the paired t-test with a 95 percent level of confidence. There was no noticeable damage to the pavement from any of the markers.

CONCLUSIONS

During the experiment, it was noted that the ideal marker should incorporate certain features and avoid others. Although the plastic

TABLE 2 SITE DESCRIPTION

Name	Location	Pavement	Traffic Pattern	AADT (1,000)	Marker Type	Test Section	Total Markers
US-1 North	I-287	Asphalt	Weave	28	6	21	126
I-287 North	US-1	Concrete	Weave	39	6	8	48
I-287 North	Route 18	Asphalt	Exit	35	6	14	84
I-287 North	Mt. Airy Road	Concrete	Exit	24	6	5	30
I-78 East	I-287	Asphalt	Split	19	6	15	90
I-287 North	I-78	Concrete	Split	19	6	15	90

TABLE 3 VISIBILITY RESULTS

Type	New Markers Sun in Face (ft)	New Markers Face in Shade (ft)	New Markers Dry Night (ft)	After 6 Months of Wear Sun in Face (ft)	After 6 Months of Wear Face in Shade (ft)	After 6 Months of Wear Dry Night (ft)
Ferro P-15 (control)	1,075	1,063	1,110	1,045	816	419
Stimsonite 66B	1,175	1,050	1,025	1,048	–[a]	–[a]
Ray-o-Lite	737[b]	645[b]	1,100	849[b]	565[b]	385
Titan TM-40	1,175	1,063	NA[c]	1,048	781	NA
Swareflex 3557	1,088	800[b]	1,100	1,045	585[b]	1,041[b]
Olympic Machines 44C	1,172	1,063	457[b]	1,045	746	246[b]

[a]Figures were biased because the Stimsonite markers broke up on removal. Face in shade = 613 ft, and dry night = 271 ft.
[b]Significant change from ceramic at the 95 percent level of confidence.
[c]Not applicable.

TABLE 4 DURABILITY AFTER 6 MONTHS

Type	Marker on Asphalt No. Installed	No. Left	Percent Left	Marker on Concrete No. Installed	No. Left	Percent Left
Ferro P-15 (control)	51	29	57	28	15	54
Stimsonite 66B	50	47	94	28	23	82
Ray-o-Lite	50	43	86	28	24	86
Titan TM-40	50	50	100	28	28	100
Swareflex 3557	37	14	38	28	12	43
Olympic Machines 44C	49	43	88	28	12	43

cube-corner-type reflectors proved adequate, they lost over one-half of their visibility after 6 months of wear (Table 3). The Swareflex (Figure 5) multiple-glass lens reflector was the only one that retained 1,000 ft of dry-night visibility after 6 months.

Also, the more vertical and nonstreamlined the marker casing and reflector facing the driver, the less likely motorists would be able to see the marker in the day with the sun behind the visible face, and the more likely the marker would come off the pavement from tire impacts. The face of the Swareflex marker toward the driver sloped 60 degrees from the pavement causing the visible face to go into a shadow and lose visibility (Table 3). The Swareflex also experienced the highest losses, no doubt from tire impacts on its nonstreamlined surface (Table 4).

The ideal marker should also avoid dedicating too much visible surface to the reflector because the reflector has low visibility in the day. The Ray-o-Lite (Figure 3) visible surface is mostly reflector and is the least visible marker in the day (Table 3).

An adequate day-night, construction-zone marker should have the following features:

1. A streamlined profile;
2. A microscopic cube-corner, sealed prismatic air cell, cube-corner reflex, or multiple-glass lens reflector; and
3. A balance between the reflector and casing area exposed to the driver.

The systems that met the aforementioned criteria are (Table 5):

1. The hollow acrylic marker with a sealed prismatic air cell reflector (Amerace Corporation Stimsonite 66B), and
2. The combination of the dome-shaped polyester marker (Traffic Safety Supply Company Titan TM-40) for day visibility and the filled ABS shell marker with a cube-corner reflex reflector (I.T.L. Industries, Inc. Ray-o-Lite or equivalent) for night visibility.

Both systems used butyl adhesives on a primed surface, and both systems were removed from the pavement without any noticeable damage to the pavement.

The Titan TM-40/Ray-o-Lite system and the Stimsonite 66B were installed in an actual construction zone by contract forces during the 1985 construction season. The markers were used to delineate lane diversion through an I-78 bridge-deck restoration project in New Jersey. There were three lanes of traffic in each direction, the pavement was concrete, and the annual average daily traffic (AADT) was over 40. After the 1 month that the diversions were in effect, 100 percent of the Ray-o-Lite, 98 percent of the Titans, and 87 percent of the Stimsonites were in place.

FUTURE RESEARCH

Further research is needed to find an adhesive better than the butyl pad for temporary markers. The butyl requires a primer that must be allowed to dry, and a vehicle must then be driven over the marker. In practice, these steps may not be followed and the marker may fall off the pavement. There are also problems with cold temperatures, rough pavement, and incompatibility with some markers.

Hot-melt adhesives should be investigated because in comparison to butyl, they (a) require no primer, (b) can conform to pavement irregularities, and (c) can be used in a wide range of temperatures. This ensures fast, durable and economical installations without compromising removability.

There is also a need to enhance raised marker visibility by adjusting spacing and placement to account for special geometric situations such as reconstructed ramps.

ACKNOWLEDGMENTS

This research was done in cooperation with the FHWA, U.S. Department of Transportation.

Appreciation is expressed to William Mullowney for his direction and guidance in all phases of the project; Arthur W. Roberts and Eugene F. Reilly, for their administrative assistance; and the

TABLE 5 DAY-NIGHT VISIBLE MARKER ADEQUACY

Type	Visibility Sun in Face	Face in Shade	Night	Percent Remaining Bitumin	Concrete	Overall Adequacy
Ferro P-15	Control	Control	Control	Control	Control	Control
Stimsonite 66B	Equal[a]	Equal	Equal	Better[b]	Better	Pass
Ray-o-Lite	Worse[c]	Worse	Equal	Better	Better	Pass, night
Titan TM-40	Equal	Equal	NA[d]	Better	Better	Pass, day
Swareflex 3557	Equal	Worse	Equal	Worse	Worse	Fail
Olympic Machines 44C	Equai	Equal	Worse	Better	Worse	Fail

[a]Equal to ceramic.
[b]Better than ceramic.
[c]Worse than ceramic.
[d]Not applicable.

Division of Research staff, for the collection of data at inconvenient locations and times of day.

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Publication of this paper sponsored by Committee on Coatings, Signing and Marking Materials.

Evaluation of Temporary Pavement Marking Patterns in Work Zones: Proving-Ground Studies

CONRAD L. DUDEK, R. DALE HUCHINGSON, AND DONALD L. WOODS

Results of proving-ground studies for evaluation of temporary pavement markings for work zones are summarized. The objective was to investigate 10 candidate temporary marking treatments: one base treatment consisted of 4-ft stripes with 36-ft gaps, and nine other candidate marking treatments employed variations in stripe length, gap length, and reflective and nonreflective raised pavement markers (RPMs). The initial studies were conducted during dry-weather, daytime conditions. Based on the findings of the daytime studies, the base treatment and the best six of the nine other marking treatments were evaluated during nighttime, dry-weather conditions employing the same procedures and experimental design. The studies were conducted on the test track at the Texas A&M Research and Extension Center, with a demographically balanced sample of drivers individually driving an instrumented test vehicle. Measures of effectiveness included speed and distance data, erratic maneuver data, and subjective evaluations of treatment effectiveness. The nighttime studies aimed to determine whether the daytime findings were applicable to dry-weather, nighttime driving conditions. The approach was to essentially replicate the daytime study procedures with a matched, but different sample of drivers. The six markings selected were three with striping patterns and three RPM configurations. Daytime treatments deleted were those with 1- and 2-ft stripes, long (48- and 38-ft) gaps, or both.

In highway work zones traffic is often required to use different parts of the roadway for short periods of time, which necessitates changes in path delineation. For example, a significant portion of highway maintenance activities involves pavement overlay work. This type of work frequently requires more than one layer of pavement, and traffic is permitted to operate on the roadway between the times the first and last layers are laid. Therefore, there is a need to delineate pathways (lanes) through work zones for motorists, particularly for nighttime and adverse weather driving conditions.

There are basically two schools of thought: (a) to simply use the Manual on Uniform Traffic Control Devices (MUTCD) standard markings, resulting in the expenditure of considerable time and materials, which seems impractical for conditions where the marking would be in use for a short period of time, and (b) to use temporary and possibly an abbreviated marking pattern. Research is needed to develop a cost-effective temporary pavement-marking pattern for use in highway work zones.

Proving-ground studies were conducted to evaluate 10 candidate temporary pavement marking treatments: one base treatment consisting of 4-ft stripes with 36-ft gaps, and nine other candidate marking treatments. Studies were first conducted during daytime, dry weather conditions. The six best or most promising treatments, along with the base treatment, were then studied during nighttime, dry weather conditions.

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APPROACH FOR DAYTIME STUDIES

Objectives and Scope

The objective of this series of studies was to investigate 10 candidate temporary pavement marking treatments for use at work zones by determining the effects of each on various measures of driving effectiveness during daytime, dry weather conditions. The markings consisted of a set of low-profile markings (LPMs) and raised pavement markings (RPMs) applicable to work zones.

The optimal markings recommended were based on the best overall driving performance in negotiating a series of four horizontal curves on a test track located at the Texas A&M Research and Extension Center. The controlled field study consisted of drivers individually driving the test track course in both directions, thereby negotiating a total of eight test curves.

Experimental Plan

The daytime experiment consisted of three stages:

1. A pilot study to determine which of two entry speed strategies (cruise control and noncruise control) would likely result in the most useful test data.
2. A baseline study with the same control treatment (4-ft stripe with 36-ft gap) applied to all four curves of the test track. This study was conducted to determine if there were curve differences irrespective of treatment differences. Although the horizontal test curves were identical in degree of curvature, features near the curves could possibly affect the drivers' performance. Therefore, it was necessary to establish baseline data before application of treatments to the curves.
3. The main study consisted of a comparison of 10 marking pattern treatments: the baseline (control) treatment and nine other candidate treatments.

A description for each of the 10 treatments follows. Treatment 1 was the control condition. The other treatments consisted of both LPMs and RPMs with LPM variations in stripe length and gap length, and combinations of reflective and nonreflective RPMs.

- Treatment 1: 4-ft stripes (4-in. wide) with 36-ft gaps. (Control condition).
- Treatment 2: 2-ft stripes (4-in. wide) with 38-ft gaps.
- Treatment 3: 8-ft stripes (4-in. wide) with 32-ft gaps.
- Treatment 4: 2-ft stripes (4-in. wide) with 18-ft gaps.
- Treatment 5: four nonreflective RPMs at 3⅓-ft intervals with 30-ft gaps and reflective marker centered in alternate gaps at 80-ft intervals.
- Treatment 6: three nonreflective and one reflective RPM at 3⅓-ft intervals with 30-ft gaps.
- Treatment 7: 2-ft stripes (4 in.) with 48-ft gaps.
- Treatment 8: Treatment 2 and RPMs at 80-ft intervals.
- Treatment 9: two nonreflective RPMs at 4-ft intervals with 36-ft gaps and one reflective RPM centered in each 36-ft gap.
- Treatment 10: 1-ft stripes (4 in.) with 19-ft gaps.

Treatments 1-6 and 9 were day and night studies. All of the stripes and RPMs were yellow.

The experimental plan for the main study involved dividing the nine marking treatments into three sets (A, B, and C). Each set consisted of three randomly assigned treatments and the control condition from the baseline study. For example, the Set A study included three temporary treatments and the control. During Set B and Set C studies, three different treatments were tested along with the control treatment in each study.

Table 1 gives the curve treatment sets. The experiment used a matched group design, with matched but different driver subjects assigned to each set. The decision to use matched groups was based on the unusual length of administration time required to

TABLE 1 ASSIGNMENT OF TREATMENTS TO CURVES AND SETS FOR DAYTIME STUDIES

	Treatment by Curve No.			
	1	2	3	4
Base	1	1	1	1
Set A	2	5	3	1
Set B	4	6	7	1
Set C	10	9	8	1

have each driver exposed to each treatment. Also, possible learning or fatigue effects could have biased the results. Sixteen subjects were assigned to each set. They encountered three treatments and the control (base) condition in random order.

Test Subjects

Because the findings of the research were to be generalized to the U.S. driving population, a sample of driver subjects was selected based on demographic data on current drivers. Subjects were to be representative of the U.S. driving population in terms of age, sex, education, and driving experience. All were to hold a current driver's license and at least corrected 20/40 visual acuity.

In the matched group design, subjects in each set were assigned to ensure equivalency on the relevant subject characteristics. There were 48 subjects in the main study, 16 in the base study, and 5 in the pilot study. No subject was used more than one time.

Test Track

Figure 1 shows the test track at the Texas A&M Research and Extension Center located at an old air force base. The 6-mi test track included several horizontal curves. Four of the curves (1, 2, 3, and 4) were specifically designed for 50-mph speeds and were used as the test curves. Traffic control devices were installed adjacent to the test track at locations where speed reductions and stops were desirable. No devices were installed near the test curves.

The test track is a two-lane, two-way highway with 11-ft lanes. An 11-ft lane width was used in order to take advantage of part of an existing test track. Also, it reflects geometric standards common for construction and maintenance operations. A standard centerline was used and edgelines were placed on the outside of both lanes throughout the entire truck except near the four test curves. The temporary pavement marking treatment began and the edgelines were dropped on the tangent sections in both directions of travel 500 ft before the beginning of each test curve. The pavement marking treatment extended through the curve and was discontinued with the addition of the normal centerline and edgeline on the tangent section 500 ft beyond the point of tangency of the curve. The removal of the edgeline before the curve allowed for a transition between the tangent section and the test curve. If drivers reacted to the edgeline drop, it was surmised that the reaction would take place within the 500-ft tangent section before the curve. Therefore, the speed reductions in the curves could be attributed to the motorists' guiding ability on the temporary pavement marking treatment and not to the edgeline drop.

Curve	Curve Length (ft.)	Left Tangent (ft.)	Right Tangent (ft.)
1	1636	1200	1100
2	818	2600	1000
3	818	3600	1400
4	754	1100	900

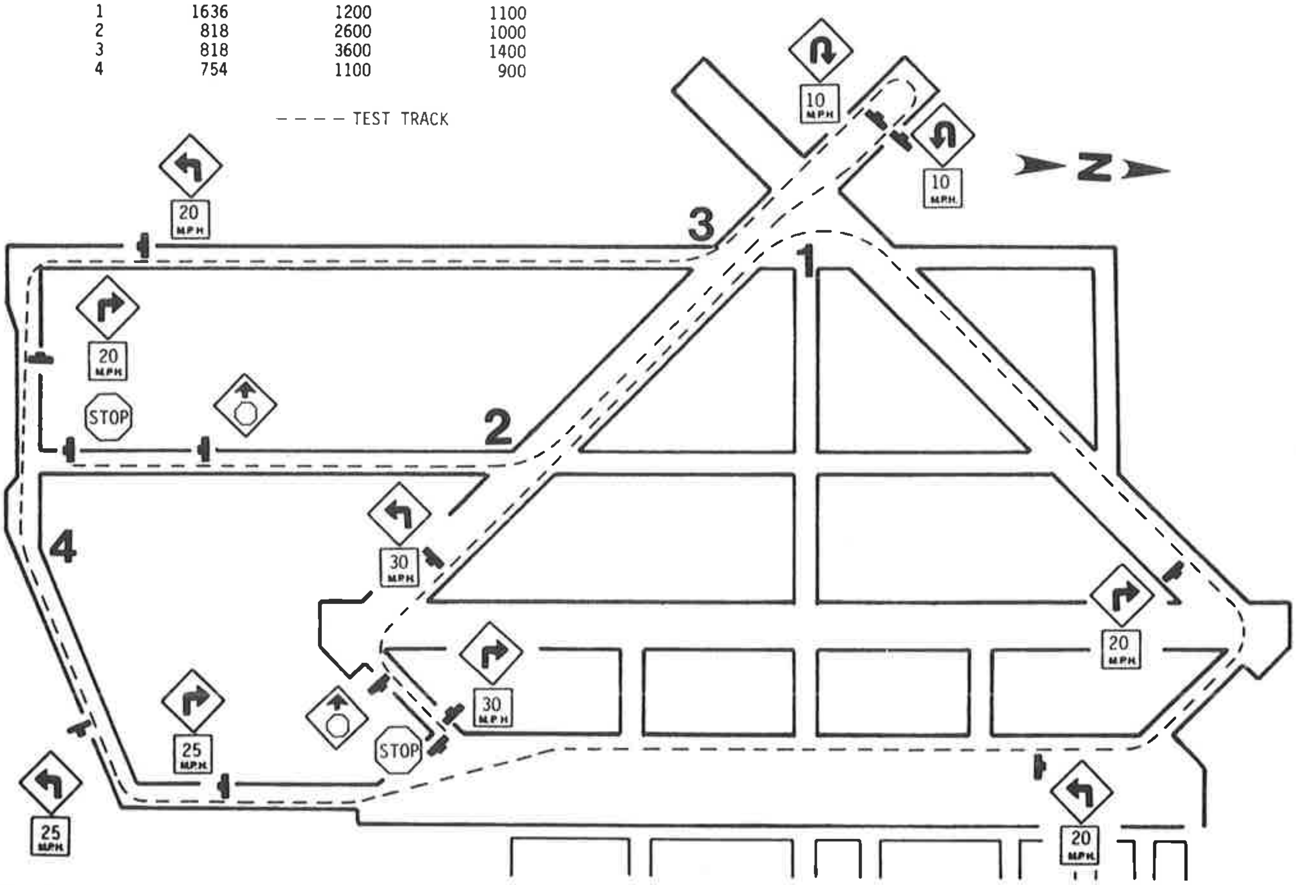

FIGURE 1 Test track at the Texas A&M Research and Extension Center.

Experimental Protocol

Measures of Effectiveness

The measures of effectiveness (MOEs) consisted of (a) seven measures derived from the speed and distance measurements, (b) frequency of erratic maneuvers (deviation from the centerline) and curve misses, and (c) drivers' subjective comments and ratings of treatment effectiveness.

The speed and distance MOEs were as follows: (a) maximum entry speed into the curve, (b) minimum speed while in the curve, (c) magnitude of the speed change (difference in maximum entry speed and minimum speed), (d) distance from the edgeline drop point to the location where the driver reached minimum speed, (e) minimum speed within the first 1,000 ft, (f) speed change within the first 1,000 ft, and (g) distance to the point of minimum speed within the first 1,000 ft.

In addition to speed and distance measures, frequencies of erratic maneuvers were obtained from videotape recordings of the left front wheel and the roadway ahead. The MOEs included total frequencies of erratic maneuvers, frequencies of lateral deviations of 11 to 16 ft to the right, frequencies of deviations of greater than 16 ft to the right, and frequencies of misses or driving past the curve completely, which necessitated a vehicle turnaround. Lateral deviations were measured from the left front wheel of the vehicle to the centerline. Erratic maneuvers were defined as lateral deviations equalling or exceeding 5 ft, or the left front wheel crossing the centerline.

At the end of the test run, drivers drove the course again and commented on what they liked or disliked about each treatment. They were then asked to select the treatments that were the best and least effective in terms of guiding a driver through the curves.

Data Recording Methods

The speed and distance data were recorded using a time-speed-delay and distance measuring device installed in the test vehicle. The printer was set to record data every 100 ft.

A video camera counted on the outside of the left rear door of the test vehicle (Figure 2) provided a permanent visual record of the path of the left front wheel with respect to the centerline on the test track. Therefore, erratic maneuvers could be evaluated by viewing the video recording on a large video monitor.

Instructions to the subjects, post-test interviews, and subject comments were recorded on audio cassette and videotapes. Detailed descriptions of the data collection and equipment calibration procedures are available elsewhere *(1)*.

Procedure

In the main study it was necessary to control for possible order effects that might bias the results. For example, if Curve 1 was always the first curve encountered and Curve 4 the last in succession, there might be uncontrolled effects that could influence the

FIGURE 2 Test vehicle recording equipment.

interpretation of treatment effects. Therefore, a series of eight coded routes (AX, AY, BX, BY, CX, CY, DX, DY) was developed (Table 2). Pairs of routes had one of four entry points: A, B, C, and D. The entry points varied in terms of which curve was encountered first.

Subjects were then assigned to the entry point in systematic fashion so that 4 of the 16 subjects started at each entry point. Two subjects drove the test track in the outbound direction and two subjects started in the inbound direction at each starting point. Each subject then retraced the course in the opposite direction. Therefore, each curve was negotiated as both a left and right curve (a total of eight treated curves observed by each subject).

The test began with the test administrator sitting in the passenger seat. Standardized instructions *(1)* were read to the drivers. Once on the test track, drivers were instructed to drive as fast as comfortable at a maximum speed of 50 mph, paying attention to safety in vehicle-control and traffic-control devices.

After driving the course (outbound and inbound), subjects were given a debriefing *(1)* during which they again drove the test course and evaluated the treatments.

THE PILOT TESTS

Strategy Study

A pilot study was conducted to determine which of two entry speed strategies would be likely to result in the most useful test data. Specifically, a decision was needed on whether to control for initial speed on entry of the curves by having drivers accelerate and press a cruise control button set for 50 mph (cruise-control mode), or whether to simply instruct drivers to accelerate to 50 mph but permit variation as they deemed acceptable (noncruise-control mode). A study with five subjects was administered with both sets of instructions. The same treatment (Treatment 1) was applied to all curves.

Although the project staff interpreted the results *(1)* as suggesting a need for speed control in the main study, it was decided by FHWA that it would be more realistic not to use the cruise control, thereby permitting drivers to assume whatever comfortable speed they wished on entering the curves. Therefore, in both the base and main studies, the noncruise control mode was used. It was also decided by the project staff that in the main study a reduction of speed of less than 4 mph would not be of practical significance, and that a sample size of 16 per set would be retained according to the originally proposed experimental design.

TABLE 2 TEST TRACK ENTRY POINTS AND ROUTES

Entry Point	Route	Curve Order 1	2	3	4
A	AX	1L	2L	3R	4L
	AY	4R	3L	2R	1R
B	BX	2L	3R	4L	1L
	BY	1R	4R	3L	2R
C	CX	3R	4L	1L	2L
	CY	2R	1R	4R	3L
D	DX	4L	1L	2L	3R
	DY	3L	2R	1R	4R

Note: L = left-turn curve (turn to the driver's left), and R = right-turn curve (turn to the driver's right).

Base Study

The base study was conducted initially to determine if there were differences among the curve means and variances and possible direction of travel effects, that is, whether the curve broke to the driver's left or right. If the variances on the MOEs were approximately the same, then the treatments could be applied randomly to any curve. Tests on equality of means are permissible only when the associated variances are homogeneous. This is a basic assumption of the statistical testing procedure.

Tests of the equality of variances of the curve data revealed only random differences in curves and direction, and it was concluded that they were sufficiently homogeneous to permit the testing of the equality of means. A three-factor ANOVA model with curve, direction and subjects as main effects, and curve by direction interaction was used.

The new Duncan Multiple Range Test was also applied to the data. It was concluded that there were curve differences (notably Curve 4). Therefore, for the main study, it was recommended that the control condition (Treatment 1) always be applied to Curve 4. The other curves were sufficiently similar for different treatments to be applied with the assurance that differences in MOEs were due to the treatments rather than to the curves.

DATA SUMMARY OF SPEED AND DISTANCE MEASURES FOR DAYTIME STUDIES

Results for Speed and Distance Measures

The data collected included speed and lane position throughout the test track. Table 3 gives the means and variances across conditions for one of the MOEs (speed change within curve). Space limitations do not permit including tabulated results for all seven MOEs. Detailed results are available elsewhere *(1)*.

No differences among variances for the treatments in study Set A were found ($\alpha = 0.01$). For Sets B and C, the homogeneity of variance assumption was questionable for speed change, speed change in the first 1,000 ft, distance to minimum speed in the curve (Set B), and distance to minimum speed within the first 1,000 ft of the curve in the right direction of travel (Set C).

Statistical analyses of means revealed some individual differences in curves regardless of treatments. A point-by-point dis-

TABLE 3 SPEED CHANGE WITHIN CURVE: MEANS AND VARIANCES FOR DAYTIME STUDIES

Curve No.	Direction	Speed (mph) by Set Base	A	B	C
Means					
1	L	12.6	9.9	10.9	10.2
2	L	9.4	6.6	8.6	8.0
3	L	9.3	7.6	7.6	8.3
4	L	3.4	3.7	4.3	4.0
1	R	14.5	13.7	13.1	14.2
2	R	12.7[a]	9.5	8.3[a]	9.2[a]
3	R	9.2	6.7	9.9	8.5
4	R	8.3	8.2	7.9	8.6
Variances					
1	L	22.0	11.5	27.8[b]	23.1[b]
2	L	12.3	7.3	25.5	14.1
3	L	7.2	9.7	5.8	3.9
4	L	5.2	3.7	5.8	3.9
1	R	30.9	35.6	27.7	58.4[b]
2	R	30.5	18.3,	15.4	8.6
3	R	22.7	11.0	13.1	14.8
4	R	11.3	13.7	23.3	12.7

[a]Significantly different from base (0.01 level).
[b]Variances unequal. Means above cannot be compared.

cussion is not merited, but the consistency of the results did suggest the four subject groups responded to certain curves in similar fashion regardless of the treatment applied. This would be a disturbing finding (curve differences above and beyond treatment differences), however, the combined differences were very small. In other words, both curve and treatment effects were not sufficiently dramatic to reflect differences of a practical nature in the MOE. A practical difference in speed was defined as a change greater than 4 mph.

Although there are less powerful statistical tests (such as analysis of response change) that could be applied to the statistical data, the small differences in the MOEs would not seem to merit further statistical analysis.

Discussion of Speed and Distance Data

The ANOVA and Duncan's tests revealed a few significant differences in means. However, inspection of the data suggested the differences among treatment sets were too small to be of practical significance. No clear trends in any of the treatments were observed from the MOEs. In short, the speed and distance data did not provide a basis for selecting one or more treatments either better or worse, in a practical sense, than the base treatment.

SUMMARY OF ERRATIC MANEUVER DATA FOR DAYTIME STUDIES

Erratic maneuver data for the daytime studies were analyzed by a categorical (log linear regression) test to determine if there were differences across curves and treatment sets. The frequencies of erratic maneuvers are given in Table 4. The categorical data analysis revealed no significant curve or set differences. Drivers were equally likely to have an erratic maneuver regardless of curve; however, Curve 1 had slightly more erratic maneuvers.

Three additional MOEs on lane placement were: (a) frequency of driving more than 11 but less than 16 ft to the right of centerline, (b) frequency of driving greater than 16 ft to the right but essentially parallel to the centerline, and (c) frequency of missing curves completely (requiring redirection back to the course). A deviation of 11 ft would place the vehicle outside the marked lane. Sixteen feet or more would place the vehicle off the normal roadway crown. Data are given in Table 5. Assuming the maneuvers were random and discrete, the Poisson distribution assumption applies, with mean and variance essentially equal. Values that differ significantly are noted. Treatments 7 and 8 were found to have the greatest number of misses, course deviations, or both. Treatment 7 was the 2-ft stripe with 48-ft gap. The lengthy gaps on a curve may contribute to losing track of the centerline. Treatment 8 had 2-ft stripes with 38-ft gaps and widely separated (80-ft) RPMs.

TABLE 4 NUMBER OF DRIVERS COMMITTING AN ERRATIC MANEUVER FOR DAYTIME STUDIES

Curve No.	Direction	No. of Subjects	No. of Drivers by Set Base	A	B	C	Total
1	L	16	9	13	14	10	46
2	L	16	4	11	7	4	26
3	L	16	9	11	7	6	33
4	L	16	7	10	8	6	31
1	R	16	13	12	14	12	51
2	R	16	10	10	10	8	38
3	R	16	11	13	11	10	45
4	R	16	12	8	9	10	39

TABLE 5 FREQUENCIES OF DRIVING OUTSIDE NORMAL TRAFFIC LANE AS MEASURED FROM LEFT FRONT TIRE TO CENTERLINE FOR DAYTIME STUDIES

Treatment	Deviations to Right 11-16 ft[a]	Deviations to Right 16+ ft[b]	Misses[c]	Cumulative Misses and Deviations[d]	Passes
1	11 (1.5)	7 (1.0)	4 (0.5)	22 (3.0)	224 (32)
2	0	0	1	1	32
3	1	0	1	2	32
4	2	0	0	2	32
5	0	0	0	0	32
6	2	2	0	4	32
7	1	2	6[e]	9[e]	32
8	0	4	3	7[e]	32
9	1	1	2	4	32
10	0	0	0	0	32
Total	18 (8.5)	16 (10.0)	17 (13.5)	51 (32.0)	512 (320)
Average	0.9	1.0	1.4	3.2	

Note: Numbers in parentheses are normalized to 32 observations.
[a]Frequencies of maximum lateral displacement between 11 and 16 ft.
[b]Frequencies of maximum lateral displacement over 16 ft but not including misses.
[c]Frequency of missed curves only.
[d]Total of Columns 2, 3, and 4.
[e]Significantly different at the 0.05 level using the Poisson assumption.

SUMMARY OF SUBJECTIVE DATA FOR DAYTIME STUDIES

Results of Driver Evaluation of Treatments

At the end of the driving run each driver subject retracted the course and was asked to comment on the treatment on each curve and to select the treatments that were the most and least effective. Subjects were allowed to select two treatments as being equal; therefore, the number of observations in each set are not equal. A given subject saw only four treatments: the control condition (Treatment 1) and three other candidate treatments. All subjects saw the control condition (Treatment 1).

Table 6 summarizes the frequency of subjects judging treatments as most and least effective. Treatments 5, 6, and 9 were

TABLE 6 PERCENT OF VOTES FOR MOST AND LEAST EFFECTIVE TREATMENTS FOR DAYTIME STUDIES

	Set A Treatments				Set B Treatments				Set C Treatments			
Effectiveness	1	2	3	5	1	4	6	7	1	8	9	10
Most	0	0	38	62	31	13	56	0	28	13	59	0
Least	12	88	0	0	17	17	5	61	6	19	6	69

judged to be the most effective by the subjects. These treatments, judged best in each set, are all RPMs. Treatment 5 is four nonreflective RPMs at 3.33-ft intervals with 30-ft gaps and one reflective marker centered in alternate gaps at 80-ft intervals. Treatment 6 is three nonreflective RPMs and one reflective RPM at 3.33-ft intervals with 30-ft gaps. Treatment 9 is two nonreflective RPMs at 4-ft intervals with 36-ft gaps.

Treatments 2, 7, and 10 were judged least effective overall. No driver rated any of these treatments as most effective in comparison with the other three treatments in their set. Treatments 2 and 7 both have 2-ft stripes. Treatment 2 has 38-ft gaps and Treatment 7 has 48-ft gaps. Treatment 10 has 1-ft stripes and 19-ft gaps.

Although the best treatment appeared each time on Curve 2 there is no evidence that the subjects were rating the curve rather than the treatment. In the baseline study, best and worst ratings were distributed in equal proportion (3 to 5) on each curve.

Clearly, the drivers strongly preferred the RPMs. Of the striping systems without RPMs, Treatment 3 (8-ft stripes with 32-ft gaps) was preferred over Treatment 2 (2-ft stripes with 38-ft gaps).

Results of Driver Comments

The subjective ratings were largely supported by the drivers' verbal comments *(1)*. Positive ratings were supplemented by favorable comments and negative ratings by unfavorable comments.

The reasons given by drivers were that the RPMs clearly identified curves, were highly visible at a great distance, and provided noise and vibration when vehicles crossed them (a fact that drivers knew from previous experience). Comments on Treatments 5, 6, and 9 were almost equivalent. The markers stand out more than tape.

The comments on disliked markings (Treatments 2, 7, 10) were as follows:

1. In comparison with the RPMs and 4-ft stripes, the 1-ft stripes (Treatment 10) were judged to be short and required drivers to search for them even with the 19-ft gap; drivers could not see very far ahead to predict curves and plan actions.
2. The 48-ft gaps (Treatment 7) were deemed too far apart making the 2-ft stripes hard to follow and easy to lose sight of.
3. Two-foot stripes with 38-ft gaps (Treatment 2), called dots, were not long enough in comparison with the RPM pattern (Treatment 5), the 4-ft stripes and 36-ft gaps (Treatment 1), or the 8-ft stripes and 32-ft gaps (Treatment 3).
4. Two-foot stripes with 48-ft gaps (Treatment 7) were similarly judged in comparison with 2-ft stripes with 18-ft gaps (Treatment 4), 4-ft stripes with 36-ft gaps (Treatment 1), and RPMs (Treatment 6).

Treatments 3 (8-ft stripes) and 1 (4-ft stripes) received generally satisfactory comments on line length.

Comparison of Erratic Maneuvers and Driver Evaluation

Treatment 7, one of the three judged least effective, also had the most erratic maneuvers. Treatment 8, another with many misses and deviations, was judged least effective by 19 percent of the drivers.

Treatments 9 and 6, although highly rated, had four erratic maneuvers each, which was more than the average across all treatments. However, Treatment 5 with high ratings had no erratic maneuvers and was one of the most effective treatments. Treatment 1 (normalized for the number of observations) had an average number of erratic maneuvers and received favorable ratings.

Based on the erratic maneuvers, ratings, and comments collectively, RPM Treatment 5 was the best single treatment, but all RPM treatments were highly preferred. Of the striping systems, the 8-ft stripe pattern was preferred and the 1- and 2-ft stripe patterns were least acceptable. A maximum gap less than 38 ft was suggested by the results.

APPROACH FOR NIGHTTIME STUDIES

Objective and Scope

The objective of the nighttime studies was to investigate the base treatment and six other candidate temporary pavement markings for use at night in work zones. The six markings selected were based on the results of the daytime studies. It was decided that three striping patterns and three RPM configurations would be investigated further under nighttime conditions using an analogous test procedure.

Experimental Plan

The six treatments and the base or control condition (Treatment 1) were the same as those investigated under daytime conditions except for the deletion of three treatments (7, 8, and 10).

The experimental plan involved dividing the six marking treatments into two sets (A and B). Each set consisted of three randomly assigned treatments and the control condition: Set A was the base condition (Treatment 1) and three stripe conditions (Treatments 2, 3, 4); Set B was the base condition and three RPM conditions (Treatments 5, 6, 9).

DATA SUMMARY OF SPEED AND DISTANCE MEASURES FOR NIGHTTIME STUDIES

In order for valid statistical comparisons to be made to treatments across sets (1, 2, 6; 1, 4, 9; and 1, 3, 5), it was necessary first to establish that drivers responded identically on Curve 4, which always had the same treatment (base condition). It was concluded that this was true and that, therefore, each curve could be analyzed across sets. However, a two-way ANOVA revealed no significant differences on any MOE except distance of minimum speed within the first 1,000 ft on Curves 1 and 2. There were no treatment differences in the major MOEs (maximum and minimum speeds or speed changes). Therefore, it was concluded that for these comparisons, all of the candidate treatments tested were as effective as Treatment 1 (4-ft stripes with 36-ft gaps).

Another part of the analysis involved treatment comparisons within each set (2, 4, 3 and 6, 9, 5). The pattern noted in the daytime studies—the fact that all but two of the MOEs had the smallest mean value on Curve 4 from the left—was also evident in the nighttime studies. The new Duncan Multiple Range Test was applied to the data. If the base condition applied to each curve had resulted in no differences in MOE performance, then it could be assumed there were no curve effects except as related to the treatments and the above comparisons were valid. Unfortunately, curve differences for the base condition were found on all MOEs except two (minimum speed and distance in both directions, both absolute and at 1,000 ft). Therefore, only these MOEs were valid for making comparisons within sets. For Set A minimum speed and distance, it was found that Treatment 2 (2-ft stripes with 32-ft gaps) had a lower minimum speed than Treatment 4 (2-ft stripes with 18-ft gaps). This finding was plausible because the longer gaps should encourage maintaining a lower minimum speed. However, minimum speed was not less than the base condition. No other differences were found except for those confounded with curve differences.

In summary, the performance data offered no startling differences that would permit ranking one condition above the others. Of the limited comparisons that could be made statistically, no major findings can be reported.

SUMMARY OF ERRATIC MANEUVER DATA FOR NIGHTTIME STUDIES

Table 7 summarizes the frequencies of erratic maneuvers by treatment. Deviation frequencies were 0 to 2 per cell except for Treatment 9, an unexplained higher frequency of 5. Three of the nine misses were on Treatment 5. No particular significance can be attached to these higher frequencies and the overall distribution could be attributed to chance. Of the 13 misses with Treatment 1,

TABLE 7 FREQUENCIES OF DRIVING OUTSIDE NORMAL TRAFFIC LANE AS MEASURED FROM LEFT FRONT TIRE TO CENTERLINE FOR NIGHTTIME STUDIES

Treatment	Deviations to Right 11-16 ft[a]	Deviations to Right 16+ ft[b]	Misses[c]	Cumulative Misses and Deviations[d]	Passes
1	10 (1.7)	2 (0.3)	13 (2.2)	25 (4.2)	192 (32)
2	1	0	2	3	32
3	2	0	1	3	32
4	1	0	0	1	32
5	2	0	3	5	32
6	2	1	0	3	32
9	5	2	1	8	32
Total	23 (14.7)	5 (3.3)	20 (9.2)	48 (27.2)	384 (224)
Average	2.1	0.5	1.3	3.9	

Note: Numbers in parentheses are normalized to 32 observations.
[a]Frequencies of maximum lateral displacement between 11 and 16 ft.
[b]Frequencies of maximum lateral displacement over 16 ft but not including misses.
[c]Frequency of missed curves only.
[d]Total of Columns 2, 3, and 4.

four were on Curve 1, three on Curve 2, two on Curve 3, and four on Curve 4, again at essentially random expectation. The highest frequencies occurred on the base treatment on the first two curves (4 and 3 respectively) and Treatment 5 (three on Curve 3).

Table 8 summarizes total erratic maneuvers by curve, direction, and set. No curve or set differences are evident except those related to direction. Over twice as many erratic maneuvers occurred for right curves as for left curves (122 versus 56).

In short, the deviation and miss data revealed no significant trends relative to treatments. The only significant result was the unusually high frequency of erratic maneuvers for right turns compared to left turns.

TABLE 8 NUMBER OF DRIVERS COMMITTING AN ERRATIC MANEUVER FOR NIGHTTIME STUDIES

Curve No.	Direction	No. of Subjects	No. of Drivers by Set Base	A	B	Total
1	L	16	5	5	2	12
2	L	16	3	3	6	12
3	L	16	7	6	4	17
4	L	16	7	6	2	15
1	R	16	10	10	10	30
2	R	16	12	11	11	34
3	R	16	12	9	13	34
4	R	16	10	8	6	24

SUMMARY OF SUBJECTIVE DATA FOR NIGHTTIME STUDIES

Results of Driver Evaluation of Treatments

Table 9 summarizes the results of the driver preference study. Set A drivers compared only striping treatments (Treatments 2, 3, and 4 with the baseline Treatment 1). The Set A findings regarding most effective treatment clearly support Treatment 3 (8-ft stripes with 32-ft gaps) in preference to the other striping conditions with 2-ft and 4-ft stripes. The least effective treatment was judged to be Treatment 2, which had the 38-ft gaps in combination with 2-ft

TABLE 9 PERCENT OF VOTES FOR MOST AND LEAST EFFECTIVE TREATMENTS FOR NIGHTTIME STUDIES

	Set A Treatments				Set B Treatments			
Effectiveness	1	2	3	4	1	5	6	9
Most	6	6	75	13	13	31	37	25
Least	13	68	6	13	68	13	13	6

stripes. These findings essentially confirmed the daytime study results.

Set B drivers compared the three RPM treatments with the baseline. Although there was no single RPM treatment that was judged most effective, all RPM treatments were judged superior to the baseline treatment. This finding was further clarified in the drivers' judgments of least effective treatments, in that over two-thirds felt the baseline treatment (4-ft stripes with 36-ft gaps) was poorest in comparison to the RPM treatments.

Results of Driver Comments

The driver comments confirmed the ratings of the treatments. Regarding most effective, 12 of the 16 subjects (75 percent) in Set A had positive comments regarding Treatment 3. All stated that longer stripes were easier to follow in curves. Only two (13 percent) commented positively on Treatment 4, and one positive comment each (6 percent) was given on the other treatments.

Eleven (68 percent) of the Set A drivers commented negatively on Treatment 2 (2-ft stripes with 38-ft gaps). All stated that the short stripes with long gaps were more difficult to see at a distance. Two (13 percent) drivers had negative comments on Treatments 1 and 4, and one (6 percent) negative comment was given on Treatment 3.

In Set B, the positive comments were fairly evenly divided among the three RPM treatments with six for Treatment 6, five for Treatment 5, and four for Treatment 9. Comments were similar in each group, that is, closer reflectors made it easier to see ahead around the curve. Only two (13 percent) subjects commented positively on the baseline Treatment 1.

Eleven (68 percent) of the Set B drivers had negative comments on the baseline Treatment 1. Comments varied, but in essence the lack of buttons made it more difficult to see ahead. Only two (13 percent) subjects commented negatively on Treatments 6 and 5, and one (6 percent) commented negatively on Treatment 9.

Summary

The drivers' comments, unlike the speed data, provided very definite indications of a common hierarchy of preferences within sets. Of the striping systems, Treatment 3 (8-ft stripes with 32-ft gaps) was judged best, and Treatment 2 (2-ft stripes with 38-ft gaps) was judged poorest. All RPM systems were substantially preferable to Treatment 1 (4-ft stripes with 36-ft gaps), but no single RPM treatment was deemed best.

Unfortunately, the two sets of drivers did not see the treatments in the other set, therefore, an overall hierarchy could not be established; that is, Treatment 3 could not be compared with the RPM treatments. However, Treatment 2, in particular, and Treatments 1 and 4 were judged less desirable than the others. These data show that the 4-ft stripes with 36-ft gaps are judged to be inferior to the 8-ft stripes with 32-ft gaps and the three RPM treatments.

SUMMARY OF FINDINGS

Findings of the daytime studies are summarized by the following significant points:

1. The small differences observed in the speed and distance MOEs were judged not to be of practical significance as a basis for discriminating among the treatments investigated.
2. Drivers rated the 1- and 2-ft stripes with gaps of 38 ft or more as the least effective among the striping patterns tested within their respective groups.
3. Treatments 7 and 8 were found to have the greatest number of erratic maneuvers of any other treatments. These treatments each had 2-ft long stripes. Treatment 7 had 48-ft gaps and Treatment 8 had 38-ft gaps supplemented with RPMs at 80-ft intervals.
4. Treatments 5 and 10 had no observed erratic maneuvers. Treatment 5 consisted of four nonreflective RPM pavement markers at 3.33-ft centers and reflective RPMs at 80-ft centers. Treatment 10 was the 1-ft stripe with 19-ft gaps. However, drivers complained about the short stripe and rated this treatment very ineffective in comparison with Treatments 1, 8, and 9.
5. Treatments 2, 3, and 4 also had only one or two erratic maneuvers. Treatment 3 had 8-ft long stripes with 32-ft gaps and was very high in drivers' rankings of effectiveness. Treatments 2 and 4 each had 2-ft stripes. Treatment 4 had 18-ft gaps and Treatment 2 had 38-ft gaps. Treatment 2 was judged the least effective in comparison with Treatments 1, 3, and 5. Treatment 4 was of only average effectiveness in comparison with Treatments 1, 6, and 7.
6. In general, the subjective data suggested a strong preference for RPMs. Of the nonRPM treatments, Treatment 3 (8-ft stripes with 32-ft gaps) was the most preferred.
7. Drivers were largely indifferent to the 4-ft stripes with 36-ft gaps, rating them average in both most and least effective. However, the erratic maneuver data suggested it would lead to relatively few misses (four in 224 passes), but relative high frequency of deviations (5 ft or more) from centerline (18 in 224 passes).

The following points are significant for the nighttime studies:

1. The performance data, relative to speed and distance measures, yielded only small and insignificant differences across treatments. Therefore, no hierarchy of treatments is possible based on speed and distance data alone.
2. The erratic maneuver data also revealed no significant differences with respect to treatments. Only a high proportion of right-direction erratic maneuvers, as compared with those in the left-direction, was found.
3. The drivers' ratings of effectiveness during the nighttime studies revealed some definite biases in Set A (e.g., Treatment 3's 8-ft stripes with 32-ft gaps was best and Treatment 2's 2-ft stripes with 38-ft gaps was poorest). However, Treatments 5, 6, and 9 (RPM treatments), all of which were most preferred in the daytime studies when appearing in different sets, were approximately equal when compared against one another in Set B. This is taken to mean that they were all equally good, based on their previous daytime ratings.
4. In general, the nighttime studies support the findings of the daytime studies which also found Treatment 3 (8-ft stripes with 32-ft gaps) to be best (in terms of driver preference) of the nonRPM treatments, and the three RPM treatments to be highly effective as well.
5. Drivers did not like the use of Treatment 1 (4-ft stripes with 36-ft gaps) as much as some of the other treatments, but the performance data did not provide evidence that either speeds or erratic maneuvers were any different than with the more preferred treatments.

ACKNOWLEDGMENTS

This paper was prepared from data collected as part of a research project sponsored by FHWA entitled, "Improvements and New Concepts for Traffic Control in Work Zones." Justin True served as the Contracting Officer's Technical Representative for FHWA.

The authors gratefully acknowledge the assistance of Olga Pendleton for the statistical analysis, and Carlton Allen and Nada Huddleston who served as test administrators.

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Publication of this paper sponsored by Committee on User Information Systems.

Some Effects of Traffic Control on Four-Lane Divided Highways

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Nine field studies were conducted on four-lane divided highways in Texas and Oklahoma to evaluate two alternative traffic control approaches: single-lane closure in one direction versus a crossover with two-lane, two-way traffic operations (TLTWO). The variables studied were: worker productivity, job duration, construction costs, traffic control device costs, highway-user costs, accidents, conflicts, and capacity. Worker productivity was measured indirectly from job duration and construction costs. Because of limited data, it was not possible to identify the conditions under which one traffic control alternative offers costs savings over the other. Highway-user costs for each study site were calculated using a modified version of a work-zone queue and user-costs evaluation model. Graphs and tables show the relationships between hourly traffic volumes and road-user costs for the sites studied.

There is a growing concern among highway agencies and construction contractors about the effects of traffic control management requirements on construction work productivity, safety, and cost. For example, less restrictive traffic control management approaches are generally easier and cheaper to install. However, these approaches may adversely affect worker productivity and therefore increase the duration of work, with accompanying increases in overall cost. Safety may also be adversely affected. On the other hand, highly restrictive traffic control management approaches may improve work productivity, but may result in traffic congestion and delays and therefore increase road-user costs.

This concern is evident at work zones on four-lane divided highways where there are two basic work-zone traffic control practices available:

1. One lane in one direction is closed resulting in little or no disruption of traffic in the opposite direction (e.g., single-lane closure); and
2. One roadway is closed and the traffic that normally uses that roadway is crossed over the median, and two-lane, two-way traffic operation (TLTWO) is maintained on the other roadway.

Single-lane closures are generally less restrictive because they only affect traffic in one direction; however they may tend to increase the duration of construction, and consequently the construction cost. Conversely, a TLTWO traffic control plan may reduce the construction duration and cost, but it may also result in traffic congestion, and consequently it may increase road-user costs. There is a need to select the approach that balances work-zone productivity and safety with costs (e.g., construction, traffic control, and road-user costs). There may be levels of traffic volumes when one traffic control approach (single-lane closure versus TLTWO) becomes the better alternative. The need for highway agencies to objectively evaluate the single-lane closure and the TLTWO traffic control approaches to select the best of the two alternatives prompted FHWA to fund research to begin collecting the necessary data and developing cost relationships.

BACKGROUND

Objectives and Scope

The objectives of the research in this paper were to

1. Conduct field studies to evaluate the current traffic control requirements for work sites on four-lane divided highways to determine their effects in time, cost, and safety to perform the work; and

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2. Develop a procedure or methodology that quantifies productivity, worker and motorist safety, and user and construction costs.

Originally 16 case studies were to be conducted in Texas on four-lane divided highways involving pavement resurfacing or bridge reconstruction repair work where a single-lane closure (eight sites) or a TLTWO (eight sites) traffic control plan was used. As the studies progressed, the study design was modified by FHWA.

Case studies were initially conducted at five sites following a pilot study on the Central Expressway in Dallas. The types of work at the sites were concrete pavement repair, pavement resurfacing, bridge repair, bridge deck repair, and pavement reconstruction. After reviewing the results of the initial field studies, the decision was made to conduct all future studies at long-term work sites (24-hr duration or longer). The important consideration was that both the TLTWO traffic control approach and the single-lane closure approach had to be viable alternatives. Bridge reconstruction or repair projects were preferred. Activities such as pavement repair or pavement resurfacing no longer met the necessary site criteria.

The small number of TLTWO sites in Texas prompted the researchers to explore the surrounding states for other potential study sites. One additional single-lane closure and three TLTWO sites were located near Oklahoma City, Oklahoma. After a preliminary visit to the sites, arrangements were made with the Oklahoma Department of Transportation (DOT) to conduct studies at the four sites.

The added cost of the studies in Oklahoma prompted a decision by FHWA to reduce the total number of case study sites to nine, (four single-lane closures and five TLTWO sites, in addition to the pilot study).

Data were collected at each site to assess the following variables: worker productivity, job duration, construction costs, traffic control device costs, highway-user costs, accidents, conflicts, and capacity.

Field Study Approach

A study team consisting of a study coordinator, video cameraman, and two other technicians collected the following types of data at each study site: video, conflicts, road-user characteristics, volumes, and site characteristics. Video data were collected from the most advantageous location, generally from an overpass or a bucket truck furnished by the highway agency. The video data provided a record of work activities, adjacent traffic patterns, and work-zone traffic interactions.

One technician, positioned near the work area, observed and recorded the time and frequency of conflicts. Conflicts were classified as the following types:

1. Vehicle-traffic control device,
2. Vehicle-vehicle,
3. Equipment-traffic control device,
4. Vehicle-equipment, or
5. Vehicle-worker.

The data necessary to evaluate road-user costs included traffic volumes, speed changes, vehicle classifications, and vehicle occupancies. Speed changes were calculated from speed profiles recorded using a study vehicle equipped with a tachograph. The driver of the study vehicle made several runs through the work zone on each lane in both directions of flow during each study period. During each run, continuous speed data were gathered, and key locations (e.g., beginning of cone taper, end of cone taper, end of lane closure, etc.) were recorded on the tachograph using a remote push-button event recorder. Vehicle classification and occupancy counts were made by an observer who sampled the traffic during each study period and recorded the type of vehicle and number of occupants.

Tube-type traffic counters were installed upstream of the work zone in both directions of travel. Volumes were recorded every 15 min for a minimum of 24 hr so that the average daily traffic (ADT) and hourly demand volumes could be determined for both directions of travel. Additional counters were installed within the work zone to collect lane-distribution and service-volume data through the study site.

A minimum of 4 hr of data were collected at each site during a 1- or 2-day period. Data collection was accomplished using one of two approaches. The method employed depended on the work activities at the site. At some sites, data were collected continuously for a minimum of 4 hr. At other sites, data were collected for 1-hr intervals several times throughout the work day. Typical study-time periods were as follows: 9:00-10:00 a.m., 10:30-11:30 a.m., 1:30-2:30 p.m., and 3:00-4:00 p.m. The intervening periods allowed latitude for personnel rest periods, on-site coordination, and adjustment of data collection activities.

Site Characteristics

A summary of the characteristics of the nine study sites and the pilot study site is given in Table 1. Two of the five TLTWO sites had specific peculiarities. The site in Amarillo, Texas (Site 3) was on a six-lane freeway. The freeway was reduced to one lane in each direction before traffic was crossed over. One lane was maintained in each direction through the work area. The TLTWO site at Carthage, Texas (Site 5) was accomplished in a novel way. Instead of using a normal crossover, traffic was diverted from the main lanes through a signalized diamond interchange.

JOB DURATION, CONSTRUCTION COSTS, AND TRAFFIC CONTROL DEVICE COSTS

Work productivity was measured indirectly by considering project duration and construction and road-user cost relationships. Other approaches to evaluate work productivity proved to be unsuccessful.

An attempt was initially made to evaluate overall work productivity by analyzing individual worker productivity, including work delays or slowdowns encountered by workers because of traffic interaction. The work productivity estimates derived from this analysis, however, did not adequately reflect the complex nature of work-zone activities. Worker and equipment efficiency ratings were also explored as approaches to quantify worker productivity, but these also proved to be of questionable value.

The effects of the traffic control approach on worker productivity are indeed reflected in the duration of the construction project, and project duration affects both the cost of construction and road-user costs resulting from the degree and duration of congestion. Therefore, the most logical and meaningful approach to evaluating alternative traffic control strategies is to analyze the total construction and road-user costs and the safety impacts of the alternatives. This was the approach adopted, and the following relation-

TABLE 1 SITE CHARACTERISTICS

Site	Location	Type of Work	Traffic Control Plan[a]	Range of Hourly Volumes[b]	Left Side Traffic Control Device[c]	Right Side Traffic Control Device[c]	Taper/ Cross-over Traffic Control Device[c]	Available Travel Width (ft)	Length of Closure (ft)	Length of Bridge (ft)
1	Leon, Tex.	Concrete pavement repair	L/C	225-280	Cones	–[d]	Barrels	22.0	600	–[d]
2	New Braunfels, Tex.	Pavement resurfacing	L/C	705-875	Cones	–[d]	Cones	22.0	6,900	–[d]
3	Amarillo, Tex.	Bridge repair	TLTWO[e]	1,080-1,795	PCB	PCB	BA	18.0	3,400	400
4	Amarillo, Tex.	Bridge deck repair	L/C	175-240	BR	Cones	Barrels	19.0	2,400	225
5	Carthage, Tex.	Pavement reconstruction	TLTWO[f]	165-210	PM	–[d]	Barricades	12.0	12,000	–[d]
6	Oklahoma City, Okla.	Overhead structure repair	TLTWO	1,275-1,810	PCB	BR	Drums/PCB	15.0	3,100	–[d]
7	Oklahoma City, Okla.	Bridge repair	L/C	250-350 1,020-1,890	PCB	–[d]	Drums/PCB	15.0	2,500	990[g]
8	Edmond, Okla.	Base excavation and pavement resurfacing	TLTWO	600-960	Tubes	–[d]	Drums/PCB	20.0	22,700	–[d]
9	Oklahoma City, Okla.	Bridge repair and pavement resurfacing	TLTWO	550-680	Tubes	–[d]	Drums/PCB	20.0	25,500	–[d]
Test	Dallas, Tex.	Bridge repair	L/C	1,600+	Cones	–[d]	Barrels	15.0	2,135	200

[a] L/C = single-lane closure, and TLTWO = two-lane, two-way operation.
[b] In direction of lane-closure or crossover.
[c] PCB = portable concrete barrier, BA = barricades, BR = bridge rail, and PM = pavement markings.
[d] No data.
[e] Normal six-lane freeway.
[f] Crossover accomplished by exiting roadway, crossing an overpass, and reentering roadway using off-ramp on opposite side.
[g] Two bridges (270 and 160 ft long) 560 ft apart.

ship was applied to decide whether TLTWO (T/O) should be used rather than the single-lane closure (L/C):

$$\text{construction costs}_{T/O} + \text{user costs}_{T/O} < \text{construction cost}_{L/C} + \text{user costs}_{L/C}.$$

Discussions on worker productivity, worker and equipment efficiency, and work-zone efficiency, work space, and total cost are presented elsewhere *(1)*. Available data on job duration, construction costs, traffic control device costs, road-user cost, accidents, and conflicts are presented in the following sections.

Job Duration and Construction Costs

Table 2 gives the job duration and construction cost (as bid) for each site. Where data were available from the contractor or highway agency, the estimated job duration and construction cost for the alternative traffic control approach are also given for each site. For example, the contracted duration for Site 4 using the lane closure traffic control approach was 6 working days at a cost of $70,012. The contractor estimated that the same work would take 5 days and cost $78,012 using a TLTWO traffic control plan.

Alternative job duration and construction cost estimates were available for only three sites. Every effort was made to obtain data for the other sites, but the contractors and highway agencies did not have the resources to prepare confident estimates. Based on the limited data that were obtained, it is not possible to say that one approach (single-lane closure or TLTWO) definitely offers an advantage in terms of reduced construction cost or time. However, for the two large projects for which data were provided (Sites 3 and 5), a TLTWO apparently would result in significant construction cost and time savings, compared to a single-lane closure traffic control approach.

TABLE 2 COMPARISON OF JOB DURATION AND CONSTRUCTION COSTS FOR ALTERNATIVE TRAFFIC CONTROL APPROACHES

			Single-Lane Closure		TLTWO	
Site	Type of Work	Job Length (ft)	Job Duration (days)[a]	Total Cost ($)	Job Duration (days)[a]	Total Cost ($)
1	Concrete pavement repair	12	1	2,779		
2	Pavement resurfacing	21,120	60	416,712	–[b]	–[b]
3	Bridge repair	400	240[c]	1,162,683[c]	200	849,372
4	Bridge deck repair	225	6	70,012	5[c]	78,012[c,d]
5	Pavement reconstruction	12,000	300[c]	3,500,000[c]	225	2,925,660
6	Overhead structure repair	3,100			200	1,589,859
7	Bridge repair	430	130	996,708		
8	Base excavation and pavement resurfacing	22,700			120	1,708,201
9	Bridge repair and pavement resurfacing	25,500			270	5,195,980

[a] Contracted duration.
[b] No estimate given because job was dependent on the ability of the hot-mix plants to furnish materials. Hot-mix plants could not furnish materials as fast as the contractor could handle.
[c] Indicate alternative traffic control approaches.
[d] Contractor was working on bridges in both directions of travel. A TLTWO control plan would have prevented simultaneous work on both bridges, accounting for the higher cost for the TLTWO alternative.

Traffic Control Device Costs

During the gathering of traffic control cost data, it became apparent that the term traffic control has a variety of connotations among contractors and highway agency personnel. Traffic control may refer to (a) signs, barricades, and other work-zone traffic control devices; (b) the physical layout of traffic routes and detours through a work zone; or (c) both traffic control devices and traffic routing. These different meanings certainly have an impact on traffic control cost estimating and reporting, and were evident in the cost data obtained.

Traffic Control Bid Costs

Traffic control bid prices were available for three of the study sites in Texas. These bid price data are are given in Table 3, along with a comparison of the bid prices for traffic control with the bid prices for the entire construction project. As shown, the actual amounts bid for traffic control represented a small fraction of the total project bid (3 percent or less for all the projects). The Site 2 contractor actually bid less than $1 for traffic control on a $400,000 project.

Actual Traffic Control Costs

After determining that traffic control bid prices were misleading, the contractors and highway agencies for each job were requested to provide a realistic estimate of actual traffic control costs. These estimates are given in Table 4. Comparing the estimates in Table 4 with the bid prices in Table 3, it is apparent that work-zone traffic control generally costs more than is reflected in the bidding process. Traffic control cost estimates, as seen in Table 4, ranged from 4 to 39 percent of the total construction cost, and averaged 15 percent of the total construction cost.

Traffic control costs averaged 21 percent of the total cost for the single-lane closure work zones and only 9 percent for the TLTWO work zones. It is important to note that three of the four TLTWO sites were accomplished without barrier separation of traffic in the two-lane, two-way sections. The one TLTWO site where barriers were used was very short. If extensive concrete barriers had been used at all the sites, the cost of TLTWO traffic control would have been much higher. For example, prefabricated portable concrete barriers cost approximately $2.70 per foot to install and remove *(2)*. This cost does not include barrier construction. If barriers had been used for the full length of the two-lane, two-way section at Site 8, an additional traffic control cost of $122,500 would have been incurred for placement of barrier sections alone.

Project Size Versus Traffic Control Costs

The relationship between project size and traffic control costs was also investigated. Based on the limited data obtained, no significant trends were established. However, intuitively it would appear that traffic control costs relative to total project costs would decrease on larger projects. Also, as seen from the Site 1 data, traffic control can represent a major cost on small projects. Site 1 was a $4,500 project, and 39 percent of the total cost was incurred in handling traffic.

Alternative Traffic Control Approach Costs

Table 5 gives a comparison of the estimated cost for traffic control used at each project with the estimated cost for the alternative traffic control approach. Alternative cost data were available for

TABLE 3 COMPARISON OF BID PRICE ON TRAFFIC CONTROL WITH BID PRICE OF TOTAL PROJECTS

Site	Work Performer	Type of Work	Traffic Control Plan	Total Cost Bid ($)	Traffic Control Cost: Bid	Traffic Control Cost: Percentage
2	Contractor	Pavement resurfacing	Single-lane closure	416,712	1	1
3	Contractor	Bridge repair	TLTWO	849,372	12,000	1
4	Contractor	Bridge deck repair	Single-lane closure	70,012	1,990	3

TABLE 4 TRAFFIC CONTROL COST ESTIMATES

Site	Work Performer	Type of Work	Traffic Control Plan	Traffic Control Cost Estimate ($)	Percentage of Total Project Cost
1	State	Concrete pavement repair	Single-lane closure	1,798	39
2	Contractor	Pavement resurfacing	Single-lane closure	14,850	4
3	Contractor	Bridge repair	TLTWO	NA[a]	NA
4	Contractor	Bridge deck repair	Single-lane closure	10,500	15
5	Contractor	Pavement reconstruction	TLTWO	125,000	4
6	Contractor	Overhead structure repair	TLTWO	113,356	7
7	Contractor	Bridge repair	Single-lane closure	246,098	25
8	Contractor	Base excavation and pavement resurfacing	TLTWO	344,693	20
9	Contractor	Bridge repair and pavement resurfacing	TLTWO	287,595	6

[a]NA = not available.

TABLE 5 COMPARISON OF ESTIMATED TRAFFIC CONTROL COSTS FOR ALTERNATIVE TRAFFIC CONTROL APPROACHES

				Traffic Control Approach	
Site	Work Performer	Type of Work	Traffic Control Plan	Single-Lane Closure ($)	Crossover ($)
1	State	Concrete pavement repair	Single-lane closure	1,798	NA[a]
2	Contractor	Pavement resurfacing	Single-lane closure	14,850	NA
3	Contractor	Bridge repair	TLTWO	NA	12,000
4	Contractor	Bridge deck repair	Single-lane closure	10,500	38,500[b]
5	Contractor	Pavement reconstruction	TLTWO	225,000[b]	125,000
6	Contractor	Overhead structure repair	TLTWO	44,178[b]	113,356
7	Contractor	Bridge repair	Single-lane closure	246,098	288,142[b]
8	Contractor	Base excavation and pavement resurfacing	TLTWO	NA	344,693
9	Contractor	Bridge repair and pavement resurfacing	TLTWO	1,644,076[b]	287,595

[a]NA = not available.
[b]Indicates alternative traffic control approach.

five of the nine sites, and based on these data, it is difficult to make any generalizations. The lane closure approach would be (a) much more costly at two sites, (b) much less costly at two other sites, and (c) approximately the same as the TLTWO approach at the remaining site. Also interesting is the fact that the less costly traffic control approach was not used at Site 6. The highway agency indicated that it opted for the more costly approach in order to save construction time and thereby overall costs.

Summary

The cost analysis failed to indicate if either of the traffic control approaches offered cost savings under certain conditions. Instead, it supported the premise that the cost-effectiveness of a given traffic control approach depends on a number of individual site factors, and that these factors must be evaluated on a site-by-site basis. However, the data base is admittedly very limited. In order to fully address the cost issue, cost and alternative cost data would have to be reviewed for many more sites (20 to 30 more sites), along with detailed information on the nature of the work, job duration, special site conditions, and other pertinent project information.

The findings of the cost analyses, although based on very limited data, are generally consistent with a study conducted by Graham and Migletz *(3)*. Both studies imply that it may be difficult to generalize whether the single-lane closure or TLTWO approach offers inherent construction or traffic control cost savings. The best approach from a cost standpoint appears to depend on site characteristics and the details of the construction work.

HIGHWAY-USER COSTS

Data Analysis

The analysis was based on the assumption that the data sampled (e.g., traffic volumes, traffic speed changes, vehicle classifications, and vehicle occupancies) were in fact representative of the traffic streams at the individual work sites. Changes in highway-user costs were estimated by comparing the normal road-user costs without the work zone (before) to the costs with the work zone (during). Although before-work-zone data were not actually collected, it was assumed that in the absence of the work zone, drivers traveled through the area unimpeded. Therefore, the speeds upstream and downstream of the work zone with the work zone in place were representative of the unimpeded speeds on the entire length of the study area without the work zone.

Changes in user costs included changes in travel time or delay costs, vehicle running costs, and costs of additional speed-change cycles. Accident costs were not included in the analysis because of insufficient data to establish reliable accident rates before and during the work at each site. Costs were updated to 1982 prices.

Changes in user costs for each study site were calculated using a modified version of a work-zone queue and user costs evaluation model (QUEWZ) that relates vehicle volumes and speeds to user costs *(4)*.

As mentioned earlier, individual study vehicle runs were made in each lane of travel in both directions, and at least one run was made during each study hour. Therefore, the additional user costs for each study site were calculated for each lane and each direction of travel during each study hour. In cases where more than one instrumented vehicle run was made in the same lane, direction, and hour, the hourly user costs for each run were averaged.

Because the primary purpose of this analysis was to determine the differences in additional user costs resulting from the type of work-zone traffic control approach (single-lane closure and TLTWO), as many data points as possible were generated for the two approaches. All but one of the nine work zones studied involved closing one lane in the direction of the single-lane closure or the crossover. In the one exception (Site 3), two lanes were closed. However, all of the work zones had only one lane open in the closure or crossover direction. In the opposite direction, all the TLTWO sites and two of the single-lane closure sites had just one lane open.

User-Costs Results

Table 6 gives the user costs at the nine work sites by direction of travel during periods when no significant queues were present. User costs during periods when significant queues were present (Site 7 and the Dallas pilot test site) are given in Table 7.

From Table 6, the average additional user costs, when no significant queues were present, were found to be $0.11 per vehicle in the direction of the single-lane closure or crossover (TLTWO), and $0.08 in the opposite direction. Based on the volumes at the sites during the studies, this translates to $94 and $55 per hour of additional user costs. In contrast, the average additional user costs

TABLE 6 USER COSTS WITH NO SIGNIFICANT QUEUES PRESENT

Site	Direction of Travel[a]	Type of Traffic Control	Average Additional User Cost per Vehicle ($) Delay Cost	Operating Cost[b]	Total Cost	Average Hourly Vehicle Volume	Average Additional User Cost Per Hour ($)
1	C	L/C	0.03	−0.01	0.02	273	5
	O		<.01	<.01	<.01	286	<1
2	C	L/C	0.13	0.01	0.14	865	120
	O		NA[c]	NA	NA	NA	NA
3	C	TLTWO	0.16	0.03	0.20	1,139	228
	O		0.14	0.03	0.18	1,249	220
4	C	L/C	0.04	0.00	0.04	204	9
	O		0.01	0.01	0.02	175	3
6	C	TLTWO	0.15	0.02	0.17	1,625	276
	O		0.13	0.01	0.14	1,621	229
7	C	L/C	0.07	0.03	0.10	1,114	117
	O		0.04	0.01	0.05	260	14
8	C	TLTWO	0.21	−0.06	0.15	943	145
	O		0.20	−0.09	0.10	596	61
9	C	TLTWO	0.12	−0.02	0.10	662	64
	O		0.11	−0.03	0.08	601	46
Average							
	C		0.11	0.00	0.11	853	94
	O		0.09	−0.04	0.08	684	55

Note: Totals may not match separate values because of rounding errors and weighting of each run by the corresponding traffic volume.
[a]C = direction of single-lane closure or crossover, and O = opposite direction.
[b]Operating costs include vehicle running costs and speed change cycle costs.
[c]NA = not available.

when significant queues were present (Table 7) were found to be $0.64 per vehicle at Site 7 and $1.96 per vehicle at the Dallas test site. Evaluations with respect to queue length resulted in user costs of $0.96 per vehicle per mile of queue at Site 7 and $1.43 per vehicle per mile of queue at the Dallas test site.

Figure 1 shows regression curves that represent additional user costs per work-zone mile at different hourly demand volumes based on the data collected in this study. The three sets of data points shown on the graph are those for the single-lane closure direction (A1), crossover direction (A2), and crossover opposite direction (B2) when no significant queues were present. The curves A1, A2, and B2 explain a large portion of the variation in the data points, as evidenced by the high R^2 values 0.8383, 0.8592, and 0.9881, respectively. All three curves reflect similarly low hourly additional user costs per work-zone mile for demand volumes of less than 1,200 vehicles per hour (vph). As volumes increase beyond 1,200 vph, additional road-user costs per mile of work zone increase rapidly.

These results indicate that for the range of hourly vehicle demand volumes studied when no significant queues developed, additional hourly user costs per mile of work zone remain about the same for vehicles traveling in the direction of the single-lane closure (A1) or crossover (A2). Curve B2 suggests that vehicles traveling in the opposite direction to the crossover experience lower additional hourly user costs per mile of work zone than those traveling in the direction of the single-lane closure or crossover.

Figure 2 shows the same data points as in Figure 1 for the single-lane closure (A1), crossover (A2), and crossover opposite (B2) directions fitted with one composite cost curve. Again, an R^2 of 0.8226 indicates that a high amount of the variation in the data points is explained. Figure 2 indicates that hourly user costs per mile of work zone increase rapidly when vehicle demand volumes are nearing the capacity of the open lane in the work zone, as expected.

Total additional hourly user costs are plotted against hourly demand volume without regard to work-zone length in Figure 3. The curve fits the data points well, yielding an R^2 of 0.8773. The curve indicates that additional user costs at a single-lane closure or TLTWO work zone were less than $200 per hour for demand volumes of 1,200 vph or less. However, additional hourly user costs increased rapidly as the hourly volumes increased. For example they exceed $500 per hour at demand volumes of 1,600 vph.

Simulated User Costs

Although the data collected at the nine study sites were limited, it was of interest to explore the effects of work-zone length (dis-

TABLE 7 USER COSTS WITH SIGNIFICANT QUEUES PRESENT

Site	Direction of Travel[a]	Average Queue Length (miles)	Average Additional User Cost per Vehicle ($) Delay Cost	Operating Cost[b]	Total Cost	Vehicle Cost Per Mile of Queue	Average Hourly Vehicle Volume	Average Hourly User Cost ($)	Average Hourly User Cost Per Mile of Queue ($)
7	CQ	0.66	0.53	0.11	0.64	0.96	1,407	895	1,356
Test[c]	CQ	0.728	1.32	0.11	1.43	1.96	1,700	2,424	3,329

Note: Totals may not match separate values because of rounding errors and weighting of each run by the corresponding traffic volume.
[a]CQ = direction of crossover or lane closure while queue was present.
[b]Operating costs include vehicle running costs and speed-change cycle costs.
[c]Located on Central Expressway in Dallas, Texas.

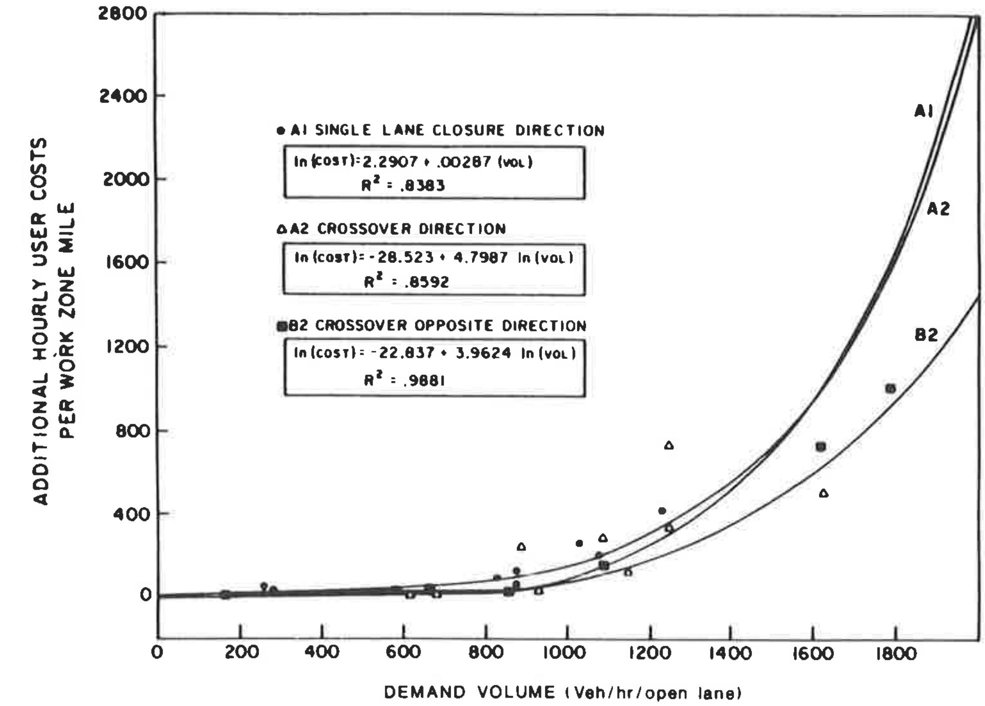

FIGURE 1 Additional hourly user costs per mile of work zone by direction of travel.

tance) on user costs. This was accomplished by using a queue and road-user cost evaluation computer model (QUEWZ) developed by Memmott and Dudek *(4)*. The simulation involved a single-lane closure approach on a four-lane divided highway, and considered traffic only in the closure direction. The simulation considered only the first hour of the closure.

The set of data assumptions for the simulated work zone example were as follows:

1. Four-lane divided facility before closure;
2. A single-lane closure through work zone;
3. Capacity restricted for 1 hr during day to calculate hourly user costs;
4. Normal capacity of 2,000 vehicles per hour per lane;
5. Restricted capacity through work zone of 1,332 vehicles per hour;
6. Eight percent of vehicles are trucks;
7. No queue present immediately before lane closure;
8. When demand exceeds capacity, a queue forms;
9. When a queue develops, the impacts of the queue on subsequent time periods are not taken into account; and
10. No diversion takes place, even if a queue is present.

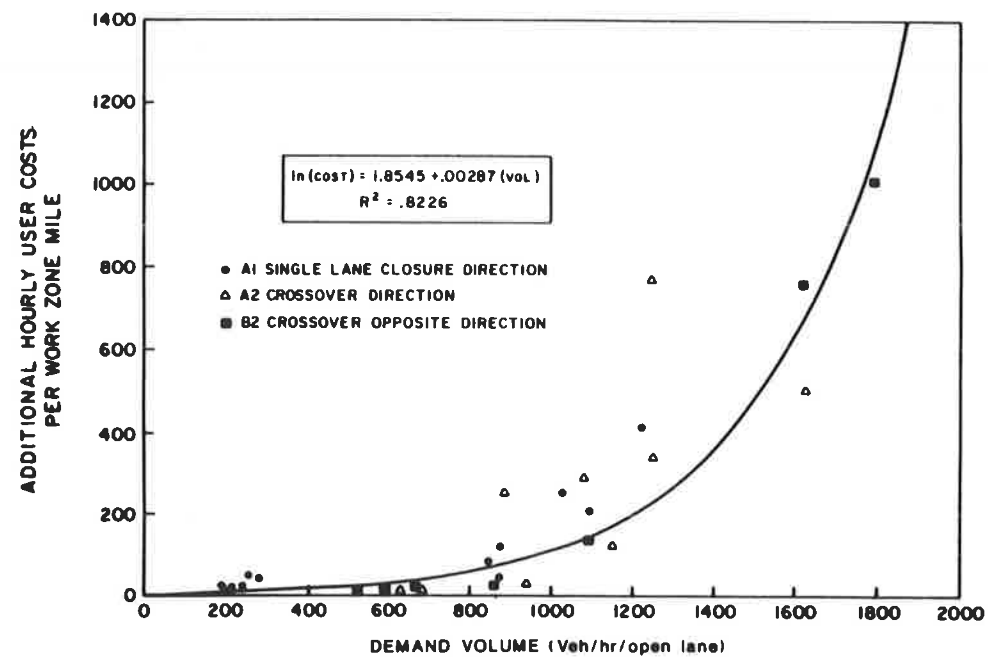

FIGURE 2 Additional hourly user costs for combined lane closures.

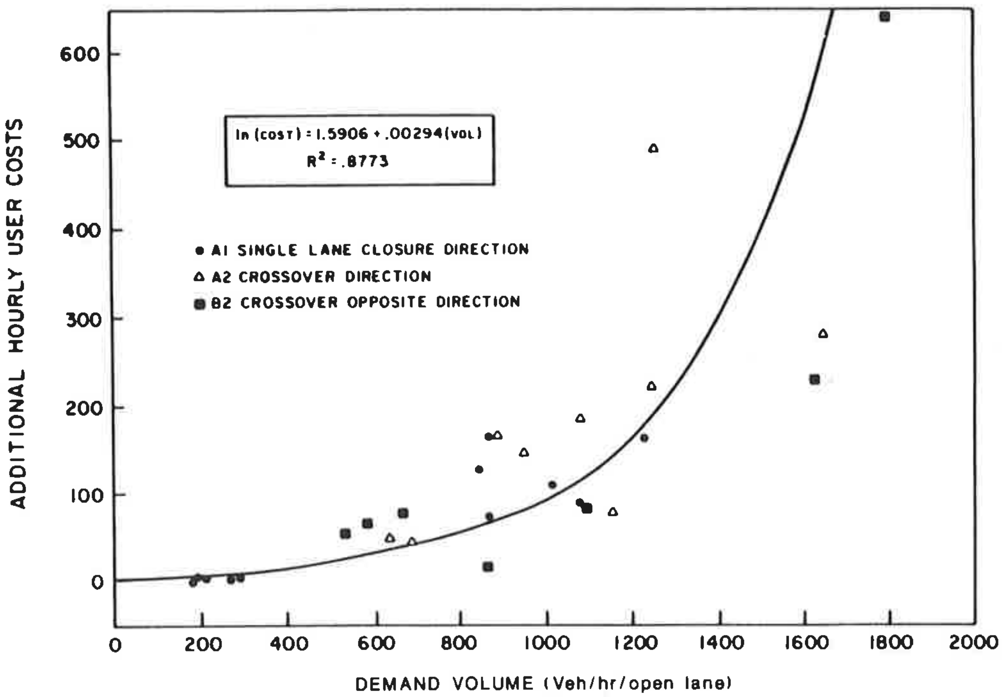

FIGURE 3 Additional hourly user costs for combined lane closure types.

Figure 4 shows the results of the simulation in the form of four curves, one for each work-zone length simulated (e.g., 0.1, 1, 2, and 5 mi). According to these results, work-zone length (at least for those studied) has only a minor effect on user costs at demand volumes below the work-zone capacity, as expected. When demands are greater than the capacity, work-zone length appears to have a significant effect on user costs.

SAFETY

Accidents

Accident experience, before and during construction, was evaluated using computerized accident data supplied by the highway agencies. A minimum of 1 year of before-construction data and all

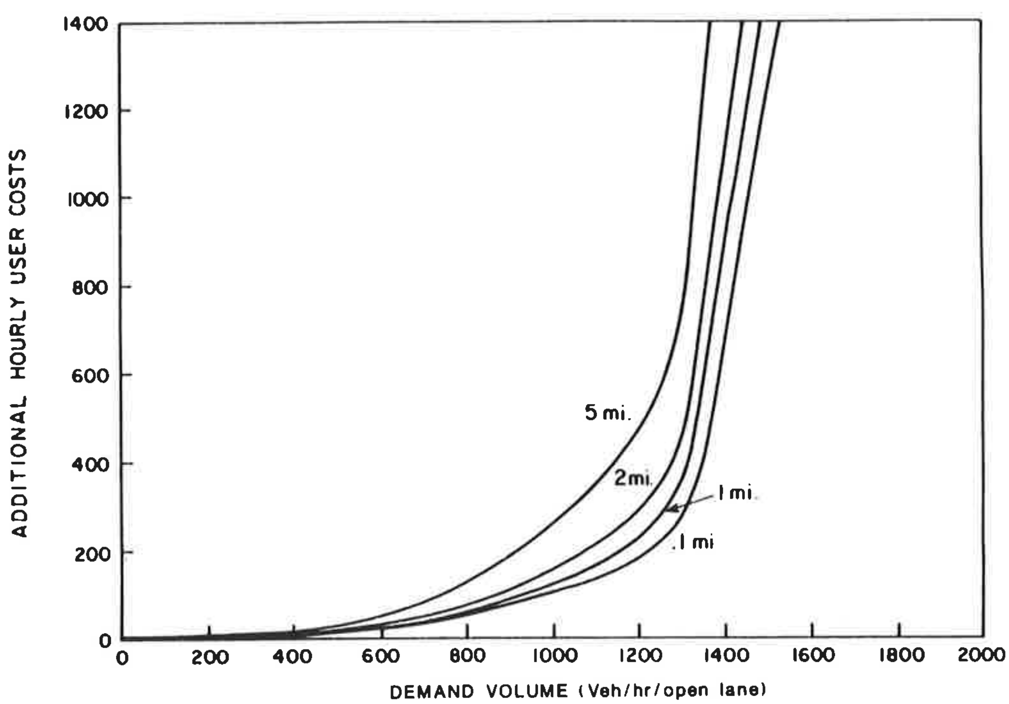

FIGURE 4 Simulated additional user costs by work-zone length for the first hour of a single lane closure.

available during-construction data were analyzed. No during-construction data were available for Site 9, because this work zone had been in place only a few weeks at the time of the study. Also, Site 1 was totally omitted from consideration because it involved a short duration work activity (i.e., pavement repair work) at various individual points along a rural freeway. No accurate accident records were kept. Therefore, during-construction accident data were only available for seven of the nine sites, and even these data were limited. The amount of during-construction data varied from only 1.2 months at Site 6 to 10.6 months at Site 3.

Table 8 gives the number of accidents reportedly occurring during construction at each site, as well as the number that occurred 1 year before construction. Accident frequencies were combined with project length and traffic-volume data to generate the accident rates given in Table 9. Note that the rates in the table are based on very small frequencies and time periods in most cases. Nevertheless, the accident rates reveal some preliminary findings. The three lane-closure work zones experienced relatively large accident rate increases during construction. Accident rates increased at Sites 2, 4, and 7 by 60, 55, and 53 percent.

The TLTWO sites generally had better safety performance based on rates alone. Two of the TLTWO sites (Sites 3 and 6) had modest rate increases of 28 and 12 percent, and the accident rate at Site 8 (TLTWO without barrier traffic separation) actually decreased by 34 percent. The remaining TLTWO site (Site 5) was unique in that traffic was detoured through diamond intersections into the two-lane, two-way section. The accident rate at Site 5 increased 64 percent during construction, and over one-half of the reported accidents occurred within the frontage road intersections.

Conflicts and Hazards

Conflicts were evaluated as an indirect measure of work-zone safety and productivity. The evaluation was based on the premise that, if a particular traffic control approach generates more frequent and severe conflicts, then it will be likely to result in more accidents and reduced productivity. Five categories of work-zone conflicts were identified and analyzed: (a) vehicle-traffic control device, (b) vehicle-vehicle, (c) equipment-traffic control device, (d) vehicle-equipment, and (e) vehicle-worker. These conflicts and their possible effects are summarized in Table 10.

Vehicle-Traffic Control Device Conflicts

Figure 5 shows composite data for the number of vehicle-traffic control device conflicts from eight of the study sites. It shows the general relationships between vehicle-traffic control device conflicts and traffic volume for left- and right-lane closures. As shown in Figure 5, conflicts occur more frequently at right-lane closures probably for two reasons: (a) right-lane volumes on a four-lane freeway are usually higher than left-lane volumes under light- to moderate-flow conditions, and (b) drivers are apparently more

TABLE 8 ACCIDENT FREQUENCIES BEFORE AND DURING CONSTRUCTION

Site	Type of Work	Separation	Accident Frequency: Before Construction[a]	Accident Frequency: During Construction	Time Period During Construction (months)
1	Concrete pavement repair	–[b]	–[b]	–[b]	–[b]
2	Pavement resurfacing	Cones	24	9	2.8
3	Bridge repair	PCB	101	111	10.6
4	Bridge deck repair	Cones	3	3	7.7
5	Pavement reconstruction	Markings	18	17	7.1
6	Overhead structure repair	PCB	14[c]	5[c]	1.2
7	Bridge repair	PCB	46	27	4.6
8	Base excavation and pavement resurfacing	Tubes	38	8	3.8
9	Bridge repair and pavement resurfacing	Tubes	46	NA[d]	NA[d]

[a] 1-year period before construction.
[b] No data.
[c] Includes frontage road intersection accidents.
[d] NA = not available.

TABLE 9 ACCIDENT RATES BEFORE AND DURING CONSTRUCTION

Site	Traffic Control	Separation	Project Length, (miles)	1982 ADT	Accident/100 Million Vehicle-Miles: Before[a]	Accident/100 Million Vehicle-Miles: During	Time Period During Construction (months)	Change (%)
1	Lane closure	–[b]	–[b]	–[b]	–[b]	–[b]	–[b]	–[b]
2	Lane closure	Cones	4.4	22,000	68	109	2.8	+60
3	TLTWO	PCB	2.0	38,000	364	466	10.6	+28
4	Lane closure	Cones	1.0	20,000	42	65	7.7	+55
5	TLTWO	Markings	2.2	6,800	324	531	7.1	+64
6	TLTWO	PCB	1.5	51,800	155	173	1.2	+12
7	Lane closure	PCB	0.9	41,500	337	517	4.6	+53
8	TLTWO	Tubes	6.3	22,600	73	49	3.8	–34
9	TLTWO	Tubes	7.8	20,800	78	NA[c]	NA	NA

[a] Based on 1-year old data.
[b] No data.
[c] NA = not available.

TABLE 10 CONFLICT DESCRIPTION AND EFFECTS

Conflict	Description	Effect
Vehicle-traffic control device	Vehicle in closed lane at taper	Forced merge to open lane
Vehicle-vehicle	Vehicle slows to merge into open lane, conflicts with vehicles in open lane, or both	Traffic slowdown, reduced traffic operation on open lane
Equipment-traffic control device	Work space restricted by traffic control device configuration	Operation slowdown, reduced productivity
Vehicle-equipment	Equipment enters or crosses traffic stream	Avoidance maneuver, accident, stopping for vehicle or equipment, reduced productivity
Vehicle-worker	Worker enters or crosses traffic stream	Avoidance maneuver, accident, stopping for vehicle or equipment, reduced productivity

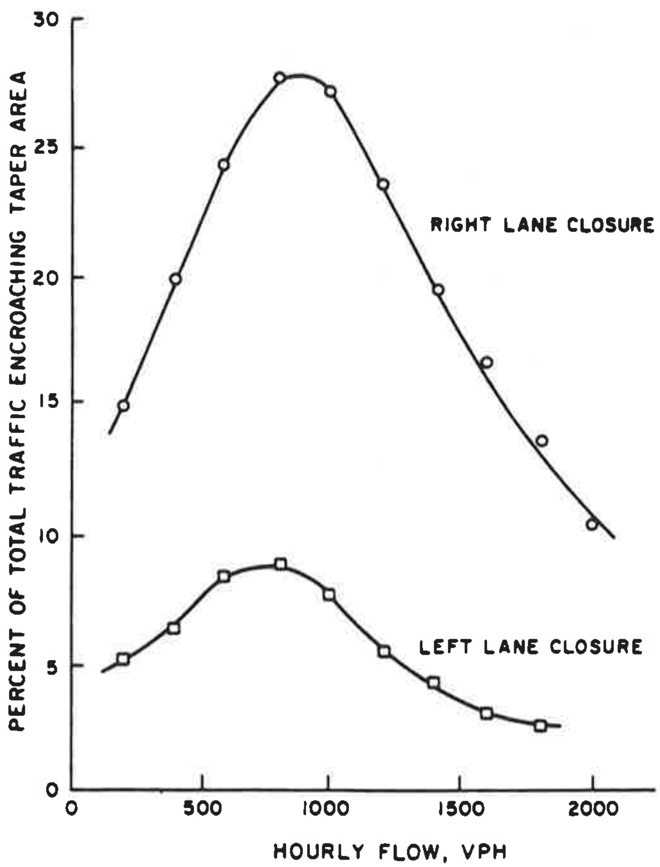

FIGURE 5 Relationship between traffic volume and vehicle-traffic control device conflicts at left- and right-lane closures.

hesitant to vacate a closed right lane, possibly because they fear missing their downstream exit ramp. No conclusions, however, could be reached from the data regarding the relative safety of right- or left-lane closures.

Figure 5 also shows that vehicle-traffic control device conflict rates (e.g., percentage of total traffic encroaching the taper area) are lower under low- and high-volume conditions, and highest under moderate-volume conditions. The conflict rate is low in light traffic probably because drivers have plenty of opportunity to change lanes without interference from other traffic. Also, visibility to the point of closure is not generally restricted by other vehicles. Under heavy-volume conditions, it is theorized that when drivers see the lane closure every effort is made to vacate the closed lane early to avoid being trapped at the taper. Because traffic is moving slower, drivers are able to merge into smaller gaps. Therefore, the vehicle-traffic control device conflict rate is relatively low.

The moderate-volume condition is most conducive to vehicle-traffic control device conflicts. Visibility to the closure is limited by other traffic. Also, some drivers may have a false feeling that they will have plenty of opportunity to change lanes at the last moment.

Vehicle-traffic control device conflict data do not indicate if the lane closure or TLTWO strategy is more desirable because both strategies require an equal number of right- and left-lane closures to complete the construction job. The data do, however, suggest that the greatest vehicle-traffic control device hazard exists during moderate-volume conditions. Also, the total number of vehicle-traffic control device conflicts can be reduced simply by minimizing the time that lanes are actually closed, particularly the right lanes.

Vehicle-Vehicle Conflicts

Vehicle-vehicle conflicts arise when drivers in the closed lane must merge into the open lane. When traffic volumes are light or moderate, the number of vehicle-vehicle conflicts will be related to the traffic volumes in each lane and the associated gaps in the open lane. When traffic demands are high and queues develop from the lane closure, drivers in the closed lane must generally force their entry into the open lane. Both the TLTWO and the single-lane closure traffic control approaches involve a lane closure; therefore, the number of vehicle-vehicle conflicts at a given traffic volume would be the same for both approaches. Because the primary interest was to compare the differences between the two approaches, a comprehensive analysis of vehicle-vehicle conflicts was not warranted.

Equipment-Traffic Control Device, Vehicle-Equipment and Vehicle-Worker Conflicts

The frequencies of equipment-traffic control device, vehicle-equipment, and vehicle-worker conflicts observed during the small sampling periods were very low at all the sites (less than five per hour). No trends were apparent in the data, because most of these conflicts resulted from actions peculiar to a site or work task. Unlike vehicle-traffic control device conflicts, these conflicts depend on a great number of factors (e.g., size of work force, equipment in use, available work space, traffic volumes, specific work task, traffic control approach, etc.). It should be noted again that the research team only observed activities for a short time period and not throughout the duration of the construction project.

CAPACITY

Table 11 gives the results of the work-zone capacity studies. As shown, the average capacities at Site 6 (a TLTWO work zone) were 1,450 vph in the crossover direction and 1,720 vph in the opposite direction. Therefore, the capacity in the crossover direction was nearly 300 vph lower compared to the opposite direction.

TABLE 11 WORK ZONE CAPACITIES

Study Site	Traffic Control Approach	Direction of Travel	Study Period	Raw Capacity Volume[a] (vph)	Average Axle per Vehicle	Average Capacity (vph)
3	TLTWO	Crossover	45 min	NA[b]	NA	1,550
		Opposite	1 hr	NA	NA	1,800
6	TLTWO	Crossover	3 hr	1,610	2.22	1,450[c]
		Opposite	3 hr	1,880	2.19	1,720[c]
7	Lane closure	Lane closure	1 hr	1,920	2.16	1,780[c]

[a] Raw volumes are based on total traffic counter actuations divided by 2.
[b] NA = not applicable because capacity counts were made directly from videotapes.
[c] Average adjusted capacity = (Raw capacity volume x 2)/Average axles per vehicle.

At Site 7, where the single-lane closure approach was used, the average capacity was 1,780 vph.

Observations of the videotapes from Site 3 indicated that at this TLTWO site, capacity flow was reached for approximately 45 min in the crossover direction and 1 hr in the opposing direction. Based on the observed data, the capacity in the opposite direction was estimated to be approximately 1,800 vph. The estimated capacity in the crossover direction was approximately 1,500 vph.

ACKNOWLEDGMENTS

This paper was prepared from data collected as part of a research project sponsored by the Federal Highway Administration, U.S. Department of Transportation, entitled, Improvements and New Concepts for Traffic Control in Work Zones. Justin True served as the contracting officers technical representative for FHWA. The authors gratefully acknowledge the assistance of Olga Pendleton in the statistical analysis.

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Publication of this paper sponsored by Committee on Traffic Safety in Maintenance and Construction Operations.

Benefits and Safety Impact of Night Work-Zone Activities

FRANK D. SHEPARD AND BENJAMIN H. COTTRELL, JR.

A literature review and discussions with highway and transportation officials in several states provided information on issues relating to the planning, safety, and traffic control aspects of night maintenance and construction activities, and their advantages and disadvantages. The information was used to develop general guidelines for nighttime maintenance and construction work. Although there are many potential disadvantages to working at night, it is believed that experience, proper planning, and attention to workmen and motorist safety make the night alternative feasible for selected work.

The rehabilitation and improvement of freeways, particularly in urban areas, entail considerable problems when closing traffic lanes for these activities creates heavy congestion on roads already loaded to capacity. The consequences are adverse effects on the safety of the traveling public and highway workers, and inconvenience and cost to the delayed motorists, which can lead to adverse public reaction. To minimize such effects, many agencies restrict roadway maintenance activities to hours of off-peak traffic, weekends, and nights. While conducting work during off-peak daylight hours (e.g., 9 a.m. to 3 p.m.) is feasible in some areas, there are many situations where lanes cannot be closed during the day because of high traffic volumes.

To avoid some of the problems encountered in daytime work, nighttime operations have been employed on numerous occasions around the country. In California, it was reported that a concrete paving operation conducted at night was completed in 16 working days, whereas it would have taken at least 35 working days to complete during daylight because of the fewer working hours and more interference from heavy traffic *(1)*. During the nighttime paving operation, traffic flow was at near-normal speeds.

Although night work is being conducted in numerous areas around the country, and is on the increase, important considerations concerning the safety of the motorist and the worker must still be met. Visibility is greatly reduced at night, some drivers are less attentive and travel faster than in daylight, and more impaired drivers are reported to be involved in work-site incidents and accidents. Also, night work creates problems concerning work forces, work scheduling, material acquisition, quality of work, and so on. The research to examine the problems inherent to conducting highway operations at night is described in this paper.

The research was conducted to (a) compile information on current practices in conducting highway maintenance and construction operations at night, and (b) to synthesize the information into guidelines for determining when nighttime work should be done and what traffic control devices should be employed.

The information developed for this paper was obtained from a survey of available literature on nighttime work-zone activities and discussions with personnel in those states and agencies that have developed expertise in night maintenance and construction operations.

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CONSIDERATIONS FOR CONDUCTING NIGHT MAINTENANCE AND CONSTRUCTION OPERATIONS

There are two main reasons for conducting night operations: (a) to allow work over a longer period of light traffic than is possible during the off-peak period between the morning and afternoon rushes, and (b) to decrease or eliminate the excessive traffic delays and congestion associated with lane closures during the daytime.

Certain types of road work require more time than is available between the morning and afternoon peaks. For example, a cast-in-place concrete patch in a pavement may require more time for setting than is available between the peak traffic periods.

Although the short length of time available for daytime operations dictates that some types of work be done at night, the interviewees for this research stated that the interference with traffic from daytime lane closures was their primary reason for scheduling night work. The basic factors considered in planning night operations and their interrelationships are discussed.

Agency Policy

Based on experience, and as a response to public and political pressure, some states have issued policy statements that dictate the levels of traffic and other criteria that should be considered when deciding if night operations are warranted.

Traffic Impact

The manner in which traffic volumes are factored into decisions on whether or not to conduct maintenance operations at night varies from state to state and largely depends on how much traffic can be allowed to back up, what the motoring public will tolerate, and the characteristics of the roadway. Some states will allow traffic to back up over long distances to avoid having to work at night, especially if the lane closure can be limited to one or two days, which indicates that night operations are viewed as a last alternative. A series of daily lane closures resulting in continuous congestion are usually followed by adverse public and media reactions leading to adoption of the night-work alternative.

Determination of traffic volumes is necessary to estimate the congestion created by lane or road closures. Also, knowing the daily variations in traffic volumes is helpful to pinpoint the low-volume periods for scheduling work. Although there are some who claim to know, either through experience or judgment, the conditions under which traffic volumes will cause undesirable daytime congestion, there are others who go through a detailed analysis in estimating congestion and must rely on reliable or recent traffic-volume counts.

Estimates of traffic volumes are usually available from data taken at permanent traffic-count locations; however, care should be taken to ensure that the data are reliable. The data should be current, and should be for time periods similar to those during which it is anticipated the work will be done and for the vicinity of the work zone, because ramps between the count station and work zone can significantly influence recorded volumes.

Analysis of Congestion

The ability of a lane closure strategy to accommodate traffic is the main determinant of whether operations will be conducted during the day or at night. Any strategy that does not adequately accommodate the traffic demand during the anticipated lane closure necessitates consideration of alternatives to daytime work, especially if the strategy imposes excessive congestion.

A procedure frequently used to investigate congestion is simply to plot the hourly volumes for the time period during which the work is to take place. For example, Figure 1 shows the volume distribution on a three-lane freeway during the probable construction period along with estimated capacities for the work area. It is apparent that two lanes will handle the demand during the midday period; however, this time period is too limited for the work to be accomplished. Also, the analysis indicates that there is a lengthy period of time each night when two lanes can be closed and only one lane will be needed to handle the traffic. The times at which the lanes can be closed and reopened to traffic can also be obtained by noting when the traffic demand and capacity are in the same range.

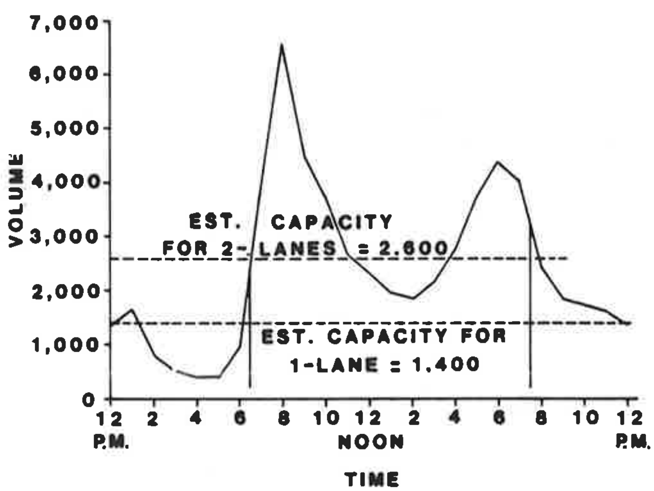

FIGURE 1 Volume distribution for three-lane freeway and estimated work-area capacities.

Clearly, an estimate of traffic capacity that can be accommodated through a work zone is an important consideration in the planning process.

Capacity

Some states and agencies have developed general guidelines for lane capacities based on their policies and experience, the work area, the type of work to be done, and so on. Table 1 summarizes these lane capacities in vehicles per hour per lane (vphpl). These capacities and comments provide a general guide to the capacity level at which the agencies would start considering night operations. Lane capacities for typical operations in California and Texas are given in Table 2, which notes that the type of work affects the capacity. Table 3 gives the average capacities for different lane-closure situations taken from 37 studies in Texas.

TABLE 1 GENERAL GUIDELINES FOR LANE CAPACITY

Area	Capacity (vphp)	Comments
Los Angeles	1,500–1,800	Usually no congestion unless more than one lane is closed for 1,500 vph Sometimes use 1,800 vph with some back ups to give contractor more time
Atlanta	1,200–1,500	No daytime closures. Depending on the area, no daytime closures if two or more lanes are closed
Chicago	1,300–1,500	Depends on location, number of ramps, and so on
Detroit	1,200–1,500	Volume before back ups start for 1,200 vph Expect serious back ups for 1,500 vph
Raleigh	1,300–1,600	Depends on area and experience
Long Island	1,500	If closure is two or more lanes, will detour traffic and work at night
Philadelphia	—[a]	Closures based on experience
Dallas	1,300–1,500	In many cases will accept daytime back ups rather than work at night
Houston	1,200–1,500	Start worrying for >1,200 vph Detailed analysis of the situation required for 1,500 vph <1,500 vph only with special traffic management
Norfolk	1,500	1,500 vph maximum at 35 mi/hr (56 km/hr) without back ups

Note: Capacity is given in vehicles per hour per lane (vphpl).
[a]Not applicable.

Delay

Work-zone closures often cause delays to motorists, with the magnitude of the delay depending on many factors such as the number of lanes closed, approach volume, time of closure, and length of any detours. An estimate of the impacts of vehicle delay is an important part of the analysis of the effects of work zone lane closures. Vehicle delay is divided into two categories:

1. Speed and distance change: delays due to speed changes, increased travel distance (detours), or both.
2. Capacity restriction: delays due to insufficient capacity causing vehicle queueing.

Other Considerations

Other factors that should be considered when contemplating night operations are listed. It is emphasized that each construction or maintenance job differs in some respect from others and each should be considered individually.

TABLE 2 SUMMARY OF CAPACITIES FOR TYPICAL OPERATIONS IN CALIFORNIA AND TEXAS

Type of Operation	Two Lanes in One Direction with One Open		Three Lanes in One Direction with One Open		Three or Four Lanes in One Direction with Two Open		Four Lanes in One Direction with Three Open	
	California	Texas	California	Texas	California	Texas	California	Texas
Median barrier or guardrail repair or installation	1,500	–[a]	–[a]	–[a]	3,200	2,940	4,800	4,570
Pavement repair	1,400	–[a]	–[a]	1,050	3,000	2,900	4,500	–[a]
Resurfacing, asphalt removal	1,200	1,300	–[a]	1,050	2,600	2,900	4,000	–[a]
Bridge repair	–[a]	1,350	–[a]	1,350	2,200	–[a]	3,600	–[a]

Note: Capacity is given in vphpl.
[a]No data available.

TABLE 3 AVERAGE CAPACITY FOR DIFFERENT WORK-ZONE CLOSURES

No. Lanes in One Direction	No. Lanes Open in One Direction	No. Studies	Average Capacity	
			Vph	Vphpl
3	1	5	1,130	1,130
2	1	8	1,340	1,340
5	2	8	2,740	1,370
4	2	4	2,960	1,480
3	2	8	3,000	1,500
4	3	4	4,560	1,520

- Supervision and communication,
- Work force morale,
- Material acquisition,
- Labor unions,
- Parts and utility service,
- Noise,
- Lighting,
- Quality of work,
- Efficiency of operations,
- Law enforcement, and
- Liability.

SCHEDULING LANE AND ROAD CLOSURES

Night operations should be scheduled to avoid peak travel periods that may lead to congestion. Also, the schedule should consider peak shopping periods, holidays, and special events.

The scheduling of night operations to avoid or minimize delay requires knowledge of the hourly traffic volumes. For example, in Figure 1, which shows the hourly volume distribution for the work area, it is observed that around 7:30 p.m. traffic volumes diminish to the point that little or no delay would result from a lane closure. Traffic volumes remain low enough throughout the night to allow closures without congestion until approximately 6:30 a.m., at which time the morning peak starts.

Adequate time should be given for opening the lanes to traffic in the morning because the traffic volumes increase rapidly during the morning peak, and a delay in reopening the lanes could result in sizeable back ups.

COST OF WORK-ZONE OPERATIONS

Because of policy decisions or unacceptable daytime traffic delays associated with lane closures, there are many situations where night work must be done regardless of cost; however, cost can be taken into consideration for whether work will be done during the day or at night. The delays, queuing, and so on, associated with day and night operations can be assigned costs, and can be combined with the operation and agency costs to obtain an estimate of the total cost of the project.

The costs associated with day or night maintenance and construction work-zone operations can be subdivided into the following general categories:

- Speed and distance change,
- Queuing,
- Vehicle operating cost,
- Accident cost, and
- Agency costs.

SAFETY

The concern for safety in nighttime maintenance and construction work zones is of utmost importance. The motorist is presented with unique and often unexpected situations where a lane, lanes, or an entire roadway is closed at night when visibility is limited and relatively more drivers are impaired from sleep, under the influence of drugs or alcohol, and so on. These conditions, coupled with the higher speeds generally prevailing at night, add a new dimension to safety considerations. Although night operations are potentially more hazardous, there are several factors that could offset the increased potential for accidents. For example, the reduced volume of traffic at night may be safer and easier to control than daytime traffic with its higher volumes and congestion. It is difficult to compare the safety aspects of night work as opposed to those for day work because of the lack of comparable data.

PUBLIC RELATIONS

Public relations are an important element in nighttime construction and maintenance activities and can facilitate the success of an operation in terms of reduced congestion, increased safety, and goodwill. With regard to a pavement resurfacing project, one official interviewed for this research noted that "on paper it was thought that traffic would be backed up for miles; however, after a media blitz, no back ups occurred." Various means of informing the public about night operations are discussed. It is difficult to state how extensive the coverage would be because the size of the project, location, traffic volume, experience of the agency, and availability of techniques, can all have an influence. It is believed that the payoff exceeds the cost in time and money for disseminating information, and that all available means of informing the public should be considered. One state official commented: "We go to extremes to get the word to the public that there will be pavement reconstruction upcoming."

It is interesting to note that although releases to newspapers, and radio and television stations constituted the most frequently used technique, the respondents to a survey *(2)* thought that special signs, door-to-door personal contact, and personalized letters were more effective.

Some states have used elaborate procedures to provide the public with information on upcoming maintenance and construction operations. The basic comprehensive plans prepared for major freeways in the Chicago area are a good example. The plans included descriptions of traffic routing, detailed maps, schedules for work, and an explanation of work to be undertaken. Agency officials met with the local radio, newspaper, and television companies for a special press conference. This special effort led to surprisingly little congestion and generally good public understanding and acceptance of the associated inconvenience.

Another example of a comprehensive plan for informing the public is that used for night closing of major freeway sections in the Detroit area. A color map detailing the detour route was distributed along with information on the closure area and time, and other pertinent information.

TRAFFIC CONTROL FOR NIGHT WORK-ZONE ACTIVITIES

The objectives of traffic control are (a) to ensure the smooth and safe movement of vehicles through the work zone, and (b) to provide safety for the workmen and the equipment in the work zone *(3)*. Drivers must be alerted to the hazards presented by the work zone and guided safely past them.

A hazardous condition is created by a lane closure on a roadway carrying a high volume of traffic. The importance and cost of controlling the traffic are emphasized by the following statement. "The development of traffic handling plans must be given as much comprehensive professional attention as is required for the physical repairs themselves. Agencies must be prepared in some instances to spend as many, or more, dollars on the traffic handling requirements of the project as on its basic construction features." *(4)*

Part 6 of the national Manual on Uniform Traffic Control Devices (MUTCD) states that night maintenance and construction activities increase the problems associated with delineating the work area and placing warning devices, mainly because of the reduced visibility at night *(3)*. Consequently, night activities necessitate increased use of warning lights and illumination or reflectivity for work areas and advance warning systems. The specific elements of traffic control are addressed in the section on general guidelines for night operations.

ADVANTAGES AND DISADVANTAGES OF NIGHT OPERATIONS

Note that because of the many variables associated with night work-zone operations, and because no two jobs are exactly alike, some of the advantages and disadvantages cited may not apply to every project, although they do reflect the overall attitude toward night operations.

Advantages

Major advantages of working at night as compared to working during daytime are listed for partial and complete roadway closures:

1. Avoidance of traffic congestion and motorist delay, which is the primary benefit,
2. Opportunity to enlarge work areas and to concurrently conduct multiple work functions,
3. Longer and more productive working hours,
4. Improved working conditions with less traffic interference and less heat,
5. Use of the full capacity of the production plant,
6. Better public relations and fewer motorist complaints, and
7. More efficient hauling because of less congestion.

Complete roadway closure allows for

1. Increased worker safety,
2. Higher efficiency in work performance,
3. Safer movement of vehicles, and
4. Shorter set-up time.

Disadvantages

The possible disadvantages of night work-zone activities as compared to daytime operations are listed for partial and complete roadway closures:

1. More prevalent driver drowsiness, inattentiveness, and intoxication (alcohol and drugs);
2. Greater potential for more severe accidents because of the higher accident rate at night coupled with higher speeds;
3. Unexpected conditions with restricted driver visibility;
4. Lessened visibility for the workmen, even with supplemental lighting, especially for tasks requiring accurate depth perception;
5. Adverse public reaction to noise in residential areas and restrictive noise ordinances;
6. Impaired communications between work-site personnel and main offices, media, police, and so on;
7. Low worker morale and difficulty in recruiting personnel in spite of pay incentives;
8. More employees working two jobs;
9. Difficulty with crew becoming accustomed to night work;
10. Problems in obtaining materials because some plants do not remain open;
11. Problems with quality control;
12. Difficulty in repairing equipment breakdowns;

13. Lower quality workmanship;
14. Difficulty in obtaining service from utilities;
15. Pressure to ensure completion of job or to have road open prior to morning rush period;
16. Higher cost for some operations because of pay differentials, increased traffic control, material acquisition, and so on; and
17. Less advance notice of impending poor weather.

Complete roadway closure results in

1. Problems with communication and coordination with local officials, for detours;
2. Additional traffic control, noise, environmental considerations;
3. Concern for capacity on detour routes;
4. Public resentment of detours and associated consequences;
5. Increased project costs if there is a need to improve the detour route; and
6. Degradation in safety.

GENERAL GUIDELINES FOR NIGHT OPERATIONS

The following general guidelines are offered as an aid in decisions concerning night construction and maintenance operations on high-traffic freeways. Included are factors and general criteria that should be used in deciding if night operations are feasible.

1. Evaluate proposed project.
2. Examine relevant traffic data.
3. Estimate roadway capacity for proposed project.
4. Determine potential daytime vehicle delay using aforementioned input.
5. Analyze feasibility of night work:
   a. Determine if delays associated with potential daytime closures (Step 4) will be excessive.
   b. Determine if cost is a factor including possible extra costs of day work, and possible cost savings.
   c. Determine if adequate time is available during night for work.
   d. Decide if possible secondary considerations are significant:
      (1) Safety including hazard potential, poor visibility, high speeds, impaired drivers;
      (2) Noise including noise ordinances and proximity to residential areas, hospitals; and
      (3) Quality of work, which may be lower quality.
6. Analyze feasibility of closing entire roadway:
   a. Determine if alternate detour routes are available;
   b. Determine if alternate routes have capacity for extra volume;
   c. Determine if traffic control (e.g., signs, signals, etc.) is adequate for traffic mix (e.g., cars, trucks, etc.); and
   d. Identify potential problems in coordinating and communicating with local officials.
7. Decide on night operations.
8. After deciding to conduct night operations:
   a. Perform advance planning,
   b. Ensure adequate advance public information,
   c. Emphasize safety through-traffic control,
   d. Schedule times for closing and opening roadway keeping in mind that work has to be completed and road has to be open to traffic in time to carry the morning peak traffic, and
   e. Continue to monitor project for possible improvements.

Guidelines for traffic control follow.

1. Develop a traffic control plan (TCP) using the standard procedure for the agency with emphasis on increased illumination and reflectivity to increase the visibility of traffic control devices.
2. Use the following elements of work-zone traffic control in the TCP (recommended):
   a. 48-in. x 48-in. or larger reflective warning signs with flashing yellow warning lights and orange flags;
   b. Drums, Type I or Type II barricades, with steady-burn yellow warning lights spaced in conformance with the MUTCD or agency standard (in most cases, this will be approximately 55 ft in the taper);
   c. Desirable and absolute minimum sight distances of 1,500 and 1,000 ft, respectively, to the lane closure;
   d. A flashing arrowboard located within the taper for each lane that is closed; and
   e. A physical deterrent, such as a truck or impact attenuator, immediately in advance of the work area.
3. Consider using the following elements in the TCP (optional):
   a. Advance warning by changeable message signs;
   b. Additional advance warning signs in advance of the end of the queue when lengthy delays causing back up of more than 1 mi from the closure is expected; and
   c. Emergency or enforcement controls, special controls, and flagging operations as needed and at the discretion of the person responsible for the TCP.
4. Use the typical traffic control layouts that conform with the state policy or the MUTCD as the basis for the TCP.
5. Monitor the effectiveness of the TCP to identify and then correct any problems. This requires that a person responsible for traffic control be on site for the duration of the work activities.

CONCLUSIONS

The question of whether to conduct construction or maintenance operations at night is difficult to answer because of the numerous considerations involved. The key consideration is the degree of congestion or vehicle delay caused by daytime lane closures. Although some agencies accept long delays as being a part of daytime road work, others do not and opt for night work, even though working at night is usually considered the least attractive alternative. Because of the increased emphasis on maintenance and reconstruction of existing facilities, coupled with the high traffic volumes in urban areas, there is reason to believe that more night operations will have to be scheduled.

Because night operations are conducted under reduced visibility, and there are more impaired drivers traveling at higher speeds than during the day, every effort has to be made to ensure the safety of workmen and motorists. In addition to increased attention to safety, consideration must be given to informing the public in advance of the work scheduled, and it must be recognized that cost, coordination of the work force, noise, quality of work, and the acquisition of materials are of more than usual concern.

Although there are many potential disadvantages of working at night, it is believed that through the experience that has been gained and proper planning, the night alternative is feasible for selected work.

ACKNOWLEDGMENTS

This research was financed by funds administered through FHWA. The opinions, findings, and conclusions are those of the authors and not necessarily those of the sponsoring agencies.

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Publication of this paper sponsored by Committee on Maintenance and Operations Management.

Implementation of Work-Zone Speed Control Measures

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Recommendations for implementing speed control at construction and maintenance work zones are presented. The following implementation steps are identified and discussed: (a) determining the need for speed reduction, (b) selecting a reasonable speed, (c) selecting a speed control treatment based on effectiveness, practicality and cost, and (d) selecting a location for the speed control treatment implementation. Four speed control approaches are studied: flagging, law enforcement, changeable message signs, and effective lane width reduction. The advantages and disadvantages of each of these approaches are discussed. Limited cost data for each of the approaches are also presented. The conclusions and recommendations are based on the results of field studies and observations at numerous street and highway work zones in Texas.

The issue of speed control through highway work zones has been a topic of concern for several years *(1,2)*. Excessive work-zone speeds can adversely affect the safety of the work crew and motorists. In an attempt to control work-zone speeds, highway agencies have followed standard signing practices, but drivers often do not slow down in response to posted speed limits.

Results of field studies conducted in Texas to evaluate selected methods of slowing traffic in work zones to acceptable speeds are presented elsewhere *(3)*. The methods included flagging, law enforcement, changeable message signs (CMSs), and effective lane width reduction. A detailed description of these methods and their effectiveness is presented elsewhere *(3,4)*. A procedure and several considerations for implementing work-zone speed control measures are presented in this paper.

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The implementation of work-zone speed control involves several steps: (a) determining the need for speed reduction; (b) selecting a reasonable speed; (c) selecting a treatment based on effectiveness, practicality and cost; and (d) selecting a location for treatment implementation. Also presented is a summary of treatment implementation considerations and limitations.

DETERMINATION OF THE NEED FOR SPEED REDUCTION

Although previous research did not specifically address the issue of when an agency should encourage reduced speeds at a particular work zone, after numerous visits to work zones, several important considerations became apparent.

Credibility

Speed control abuse and misuse at a work zone can render a speed reduction attempt ineffective and can damage the credibility of work-zone speed reduction efforts in general. Abusive practices include using unreasonably low speed limits, and leaving reduced speed limits in place after the work activity is removed.

Specific Goal

As with all traffic control efforts, any attempt to reduce work-zone speeds should be founded on an identifiable need. This need should be established based on engineering study, and not on intuition or a sound general policy. Speed reduction should be aimed at decreasing (a) the number, severity, or both, of work-zone accidents; or (b) the potential for accidents at sites where speed-related potential hazards exist.

Speed-Related Potential Hazards

Speed-related potential hazards are those that exist or worsen because traffic is traveling too fast for conditions. Typical examples of speed-related potential hazards are:

1. Hidden or unobvious work-zone features (e.g., subtle changes in alignment, edge drop-offs, etc.);
2. Reduced work-zone design speed (which is a real speed based on such factors as stopping sight distance, superelevation, degree of curvature, passing sight distance, etc.); and
3. Unprotected work space where an errant vehicle could result in catastrophic damage.

Passive Versus Active Control

Passive speed control refers to posting a reduced speed limit on a static sign (e.g., conventional regulatory and advisory signing). It is appropriate for all sites where reduced speeds are desired in the interest of safety. Passive control alone is generally sufficient at sites where the hazards are obvious, and drivers have plenty of time and information available to make reasonable and safe speed decisions without special encouragement.

Active control refers to techniques that restrict movement, display real-time dynamic information or enforce compliance to a passive control. Such techniques include: flagging, law enforcement, changeable message signs (CMSs), effective lane width reduction, rumble strips, Iowa weave sections, and so on. Active control would be needed in situations in which drivers are unable or unwilling to select the appropriate safe speed without active encouragement.

Duration of Potential Hazard

Another practical consideration is time. If a particular work activity will be in progress for an extended period of time (1 year) it would probably be impractical to use active speed control techniques for the life of the project. First, it would be too costly. Secondly, it would be unnecessary because the majority of drivers would eventually become familiar with work-zone conditions and drive at their own comfortable speed. A better approach might be to use active control only during the opening days of the project, and again following major changes in conditions. Passive speed control would be used during other times.

Adverse Impacts

Before attempting to slow traffic at a work zone, it should be recognized that speed reductions can have adverse effects. In particular, speed reduction measures can reduce roadway capacity and cause localized congestion if traffic volumes are moderate to heavy. The congestion, in turn, can increase the potential for rear-end accidents.

SELECTION OF A REASONABLE SPEED

After it has been determined that reduced speeds are desirable and practical, a safe and reasonable speed should be selected. A speed control strategy should be adopted that will reduce speeds to what is safe and reasonable for the conditions. The selected speed should not be unreasonably low but be the fastest speed that can still be considered safe.

Existing Speeds

Several factors influence what is a safe and reasonable speed for a given work zone. First, it should be recognized that drivers will only slow down to a certain level regardless of the presence of a speed control treatment. For example, previous studies *(3)* revealed that reductions in average work-zone speeds ranged from 5 to 20 mph, depending on the type of facility. Based on this finding, suggested maximum speed reductions for different types of roadways are given in Table 1.

TABLE 1 SUGGESTED MAXIMUM SPEED REDUCTIONS BY TYPE OF ROADWAY

Roadway Type	Speed Reduction (mph)
Rural two-lane, two-way highway	10–15
Rural freeway	5–15
Urban freeway	5–10
Urban arterial	10–15

Work-Zone Design Speed

The design speed of the various work zone features (e.g., horizontal curvature, sight distance, superelevation, etc.) also may dictate a safe and reasonable speed. It is very important that the design speed is not significantly lower than what drivers will reasonably expect or tolerate. If the work-zone design speed is too low, even active speed control may not be enough. Suggested maximum speed reductions in work zones by type of highway are given in Table 1.

Work-Zone Conditions

Work zones often involve workers and equipment very near the traffic stream, supply trucks entering and leaving the traffic stream, uneven pavement, shoulder drop-offs, fixed object hazards, rough pavement surfaces, distractions, and a number of other potential safety hazards. Selecting an appropriate speed for a particular set of conditions requires experience, objectivity, and good judgment.

It is extremely important that a reasonable speed for conditions be selected. If an unreasonably low speed is encouraged by the highway agency, drivers will quickly lose respect for the speed control effort. The loss of credibility and respect will result in reduced effectiveness of the speed control technique at the site and possibly other sites.

LOCATION OF SPEED REDUCTION

A speed control treatment should first be initiated 500 to 1,000 ft upstream of the hazardous location within the work zone. This ensures that drivers have adequate time to react, and the speed message will still be fresh in their minds when they reach the potential hazard. This applies especially to the flagging, law enforcement, and CMS speed control treatments that are applied at a point.

The effective lane width reduction treatment is unique because it is applied over a section. The lane width reduction treatment should be initiated approximately 500 to 1,000 ft upstream of the potentially hazardous location within the work zone and continued to a point just past the end of the potential hazard. It is critical to initiate the reduced lane width section before the potential hazard so that drivers have time to adjust their speeds and to focus their attention on the potentially hazardous condition rather than on the discomfort of driving in narrower lanes.

Location Relative to Other Work-Zone Features

The relative location of speed control treatments to other work-zone signing is also important. Ideally, speed control should be initiated after the first advanced sign and in a section that is relatively free of other work-zone signs. This practice lessens the possibility of overloading drivers with too much information and maximizes the amount of driver attention focused on the speed control effort.

Speed control treatments should not be placed in high driver work load areas such as near ramps, intersections, or lane-closure tapers.

Downstream Effects

The effective length of each particular speed control treatment was not evaluated in the studies on which this paper is based. However, it is reasonable to assume that all treatments will lose their impact eventually as drivers travel farther and farther through a long work zone. Therefore, it is likely that, if potentially severe hazards exist and drivers are not slowing down on their own, additional speed control applications (e.g., another flagger station, CMS, or law enforcement officer) may be needed downstream.

SELECTION OF SPEED CONTROL TREATMENT

Regulatory or advisory signing will not slow drivers down at work zones under normal circumstances. However, at the majority of long-duration work zones where drivers become conditioned to the work zone environment and select their own safe and reasonable speed, passive control can reinforce the existing speeds and provide a sound basis of speed enforcement. Also, if used prudently, advisory speeds will warn and advise unfamiliar drivers of common potential hazards experienced routinely in work zones.

With regard to active measures, four speed control methods were focused on in this research: flagging (including a police traffic controller), law enforcement (a stationary patrol car), CMSs, and effective lane width reduction. The selection of one or a combination of these methods for use at a particular work zone should include consideration of a number of interrelated factors including:

1. Duration of potential hazard requiring speed control;
2. Type of facility;
3. Desired speed reduction;
4. Overall cost of treatment; and
5. Institutional constraints (e.g., availability of CMSs, police officers, patrol cars, trained flaggers).

As a guide to speed control selection, the general advantages and disadvantages of the various speed control methods, with respect to the aforementioned factors, are summarized (Tables 2–5). Specific cost and implementation considerations of the various methods are discussed in the following sections.

TABLE 2 GENERAL ADVANTAGES AND DISADVANTAGES OF FLAGGING AND POLICE TRAFFIC CONTROL

Advantages	Disadvantages
Large speed reductions possible	Requires specially trained and conscientious personnel
Agency or contractor has direct control over performance[a]	Fatigue and boredom necessitate frequent relief
Relatively inexpensive for short duration applications	High labor costs for long-duration applications
Little or no disruption to traffic flow	Effectiveness may decrease with continuous use
Quick and easy to implement and remove	Two flaggers (one each side) may be needed on multilane roadways
Suitable for all types of highways and work zones	Additional flaggers may be needed for long sections
	Drivers may have a problem seeing flaggers or police traffic controllers at night; illumination of nighttime flagging stations is recommended
	Flagger safety considerations may preclude the use of flaggers at some work zones

Note: Only the use of a red flag, hand gestures, or both were considered. The effectiveness of the Stop/Slow paddle as a signaling device was not evaluated.

[a]The agency or contractor may not have as much control over a paid police traffic controller as it would over its own personnel. Also, availability of officers may be restricted by the police agency or officer interest. Some officers in urban areas are reluctant to attempt to manually control freeway traffic.

TABLE 3 GENERAL ADVANTAGES AND DISADVANTAGES OF LAW ENFORCEMENT

Advantages	Disadvantages
Large speed reductions possible	Constrained by availability of police officers and patrol cars
Relatively inexpensive for short-duration applications	Agency or contractor does not have direct control over performance
Quick and easy to implement and remove	High cost for long-duration applications
Can be effective at night, especially with lights flashing	Competes with other police functions
Sporadic use may encourage reduced speeds during nonuse periods	Long work zones may require additional patrol car units
Suitable for all types of highways and work zones	Success depends on good cooperation from enforcement agencies

Note: The statements apply to stationary patrol car treatments only, and not to use of a circulating patrol car. The circulating car approach was found to be ineffective *(3)*.

TABLE 4 GENERAL ADVANTAGES AND DISADVANTAGES OF CMSs

Advantages	Disadvantages
Relatively inexpensive for both short- and long-duration applications[a]	Only modest speed reductions possible
Agency or contractor has direct control over performance	Constrained by availability of signs
Little or no disruption to traffic flow	Effectiveness may decrease with continuous use
Quick and easy to implement and remove	Sign maintenance and repair may require technical expertise
Suitable for all types of highways and work zones	
Effective at night and in inclement weather	
May be used in combination with other techniques (e.g., flagger, law enforcement) for best results	

[a]If sign cost is extended over sign life (sign lease cost for a single, short-duration use may be high).

TABLE 5 GENERAL ADVANTAGES AND DISADVANTAGES OF EFFECTIVE LANE WIDTH REDUCTION

Advantages	Disadvantages
Moderate speed reductions possible	Expensive to implement and maintain, for short-duration applications, depending on devices used
Agency or contractor has direct control over performance	May disrupt traffic flow (reduce capacity)
Relatively inexpensive for long-duration applications, depending on devices used	May increase certain types of accidents
Retains effectiveness with continuous use and long-duration use	Device maintenance may be expensive
Speed reduction achieved throughout narrow lane section	May not be as effective on multilane highways
	Not easy to implement or remove

IMPLEMENTATION COSTS

As part of the studies, implementation costs for the various speed control approaches were assessed. The purpose of the assessment was not to attempt a detailed cost evaluation of specific treatments at individual sites, but rather to identify the major cost considerations of each approach. The scope of the research did not include developing relative cost comparisons between the various speed reduction measures.

Flagging

The cost of flagging includes the cost of labor; fringe benefits; equipment (e.g., flag, vest, and hard hat); and transportation to and from the site. It is important to budget for dead time (the time spent waiting for work to get started each day). Even more important is the requirement that flaggers be relieved every 1.5 to 2 hr. This recommendation is based on personal experience of the authors who served as flaggers during the speed control studies, and also on the observation of numerous flaggers' performance over time. Considering all costs, a highway official in Texas estimated that it costs his agency approximately $20 per flagger-hour (in 1983 dollars) *(5)*.

Law Enforcement

The results of a survey of city, county, and state police agencies in Texas regarding the cost of hiring off-duty officers for work-zone traffic control are given in Table 6. From the table, the hourly rates ranged from \$10.00 to \$22.50, with the average charge at about \$15.00 per hour.

Most of the police agencies surveyed do not normally allow officers the use of a patrol car for off-duty work. The agencies said that cars were too scarce. The Texas Department of Public Safety, by state statute, will not allow off-duty officers to use state vehicles or equipment, or even to wear their uniforms.

During the survey, the police agencies were asked about furnishing on-duty officers and patrol cars for work-zone speed control. Most of the agencies said they would provide assistance for no charge at selected sites. However, they do not have the resources to provide men and vehicles on a regular basis.

TABLE 6 COST OF HIRING OFF-DUTY LAW OFFICERS FOR TRAFFIC CONTROL IN 1983 DOLLARS

Agency	Off-Duty Wage Rate (\$/hr)
City of Austin	22.50[a]
City of Arlington	20.00
Brazos County Sheriff's Department	10–12
City of Dallas	15.00
City of Ft. Worth	15.00
Harris County Sheriff's Department	15–18
City of Houston	15.00
City of San Antonio	15.00[b]
Texas Department of Public Safety	12–15[c]

[a]Rate includes use of patrol car if approved by city.
[b]Rate drops to \$12/hr after 3 hr of continuous service.
[c]State statute prohibits off-duty officers from wearing their uniforms or using any state equipment.

Changeable Message Signs

In some locales, it is possible to rent or lease a CMS for short-term use at a work zone. However, in Texas where the studies were conducted, portable CMSs are not readily available for lease from traffic control suppliers. One supplier, however, offered to lease a three-line, bulb matrix sign for \$3,000 per month. This does not include operating costs such as fuel, oil, and routine servicing.

The Texas State Department of Highways and Public Transportation has acquired most of its CMSs by requiring contractors on major projects to buy signs for their projects. Once the projects are completed, the signs are turned over to the state for use on future maintenance and construction projects. The latest bid price received by the state for a three-line sign was just under \$50,000.

CMSs require routine maintenance and repair, and the cost of skilled labor and parts can be high. Also, it is common that inoperative signs must be shipped to the manufacturer for repair.

Effective Lane Width Reduction

As noted earlier, the cost of implementing reduced lane widths can vary greatly. The total cost includes the cost of the devices as well as installation, maintenance, replacement, and removal of the devices. The salvage or reuse value of the devices can be subtracted from total costs, however, to yield the net cost to the agency.

TREATMENT ANCHORING

The studies indicated that a speed reduction technique, to accomplish its desired effects, should be associated with (anchored to) an appropriate, reasonable speed. Anchoring refers to displaying a specific speed message along with the speed control technique so that drivers know at what speed they should travel through the work zone. The speed control technique may be anchored to a regulatory speed sign, an advisory speed plate, or a speed message displayed on a CMS. Advisory speed plates are intended for use to supplement warning signs. By anchoring a speed reduction treatment, drivers can better relate to the treatment as a speed reduction device, and the specific meaning or intent of the device is reinforced.

TREATMENT IMPLEMENTATION CONSIDERATIONS

During the course of the research, several observations were made concerning how best to implement the various speed control treatments. Some of the practical limitations of the treatments were also identified. These implementation considerations and limitations are enumerated and discussed in the following sections.

Flagging

Implementation considerations for flagging include the following:

1. Flaggers should be conscientious and dependable workers with good vision, hearing, and physical condition.
2. Flaggers should be properly attired in a fluorescent orange vest with reflective material. They should also wear a cap hard hat. The vest and headgear will enhance the conspicuity of the flagger and connote to drivers that he or she is an official member of the work force with authority to control traffic.
3. For the flagging approaches tested, the flagger should also be equipped with a standard red flag. The flag serves as an attention-getting device and increases the target value of the flagging operation. (Use of paddles was not included in the study.)
4. Flaggers should be well trained in the proper flagging procedures and techniques. The studies revealed that both the Manual on Uniform Traffic Control Devices (MUTCD) and innovative approaches produce relatively large speed reductions. The innovative approach has the advantage of indicating the desired speed to motorists. [As described elsewhere *(3,4)*, the innovative flagging approach involved the use of special hand gestures to supplement the MUTCD flagging procedure for alerting and slowing traffic.]
5. In the interest of personal safety, the flagger should not be in the travel lanes but on the shoulder if it is wide (8 to 10 ft) or just off the pavement.
6. Flagging is well suited for short-duration applications (less than 1 day) and for intermittent use at long-duration work zones. It is likely that flagging would diminish in effectiveness if it were used continuously over several days or weeks.
7. The flagging operation should be anchored to a speed sign. The research did not address whether a regulatory sign, advisory sign, or CMS was a better anchor, but did suggest that any of them would be adequate.
8. Flagging is a physically tiring and boring activity. To be effective, a flagger should be relieved at least every 1.5 to 2 hr.

9. Flagging appeared to be most effective on two-lane, two-way rural highways and urban arterials, where a flagger has the least competition for drivers' attention. On freeways, two flaggers may at times be needed, one on each side of the road, in order to achieve maximum effectiveness.

10. The studies did not evaluate the effective distance of flagging operations (how far speeds remained reduced downstream of a flagger station). However, it is reasonable to assume that in a long work zone (1 mi or more) speeds would eventually rise again. Thus, it may be necessary to establish additional flagging stations at work zones where speed hazards exist over long distances.

11. For nighttime operation, flagger stations should be illuminated.

12. It may be difficult or impossible to flag during inclement weather.

13. Flagger safety should always be of critical concern in the use of the flagging approach to reduce work-zone speeds. If speeds are too high, and sight distance is limited or there is no room for the flagger to stand off the road, then the flagging approach may not be appropriate.

Law Enforcement

Considerations for law enforcement include the following:

1. Where it was tested, manual police traffic control was the most effective law enforcement strategy. (However, a uniformed police officer was no more effective in slowing drivers than a well-trained, properly attired flagger using proper flagging procedures.)

2. A stationary patrol car, positioned next to a speed sign, was very effective in slowing drivers. By turning on the patrol car lights or radar unit, a stationary patrol car may improve its effectiveness marginally.

3. A circulating patrol car was the least effective law enforcement strategy evaluated in reducing overall speed.

4. Many officers apparently are reluctant to attempt to reduce speeds at freeway work zones by manual traffic control hand signals. During the studies, some officers refused to participate in the manual control treatment saying that their services were better utilized performing other traffic control functions. Some officers believed that they would not be effective, and some cited a concern over their personal safety. Officers were particularly hesitant to attempt manual traffic control at the urban freeway site.

5. To increase effectiveness during nighttime operation, a stationary patrol car probably would need to have its overhead emergency flashing lights on. This would ensure visibility of the patrol car to most drivers. The safety effects of a stationary patrol car with emergency lights on were not studied, although no problems were observed during the daylight tests. It is reasonable to assume, however, that there would be situations where the flashing lights would be too distracting and result in a safety hazard.

6. For maximum effectiveness, the patrol car should be highly visible to approaching traffic. The patrol car is only effective when in place, so attempts to pursue and ticket violators should be minimized. Also, it should be noted that issuing tickets in restricted width sections or lane or shoulder closure sections can have disastrous impacts on safety. Thus, if ticketing is desired to possibly further enhance the effectiveness of the stationary patrol car approach, tickets should be issued by a second patrol car unit located downstream of the work area, but in radio contact with the primary unit.

7. The various law enforcement treatments may increase in effectiveness over a period of time as more and more drivers anticipate police presence and the threat of speed enforcement. However, if drivers eventually perceive that they will not be ticketed for violations, the effectiveness may subside. Therefore, for long-term applications, it may be necessary to occasionally issue citations to violators.

8. It is likely that occasional use of the various law enforcement strategies will reduce speeds even when the law enforcement is not present. This was not addressed in the studies.

9. Additional stationary units may be needed to encourage reduced speeds through a very long work zone.

Changeable Message Signs

Considerations include the following:

1. CMSs resulted in only modest speed reductions at the sites where they were tested (urban arterial and freeway sites). It is unlikely that CMSs alone could produce very large speed reductions (greater than 10 mph). These findings are consistent with CMS studies conducted by Hanscom *(6)*.

2. The two types of messages tested (speed versus speed and informational) performed approximately the same.

3. CMSs are appropriate for day and night use.

4. CMSs retain most of their usefulness during inclement weather.

5. CMSs are versatile. The speed message may be changed as conditions change, and the CMSs may be used to display other types of information and warnings as needed. They are easy to install or relocate.

6. The appropriate type and size of CMSs should be used for the conditions. CMS selection and operation considerations are detailed elsewhere *(3)*.

7. CMSs must be properly serviced and repaired. Acquiring necessary parts and expert labor may require shipping the sign to a distant manufacturer or waiting for the manufacturer or his representative to service the sign locally.

8. CMSs, operated continuously for long periods with the same messages, may lose their effectiveness.

9. A survey of traffic control subcontractors conducted as part of this study revealed that CMSs are currently not readily available for lease on a short-term basis. In Texas where all the field studies were conducted, the highway agency requires that its contractors purchase CMSs for use on some major projects. When a project is completed, the sign is turned over to the agency for use at future construction and maintenance sites.

Effective Lane Width Reduction

Considerations include the following:

1. Slight effective lane width restrictions (e.g., 11.5- and 12.5-ft widths) will reduce speeds modestly. Although not tested, it is assumed that even narrower lanes (9 to 10 ft) may greatly lower speeds. However, the studies suggested that lane reduction, if effective, also increases speed variance and erratic maneuvers.

2. In order to implement a lane width reduction technique, it is usually necessary to interrupt traffic flow and expose workers to traffic (workers must get out into traffic and install the devices).

3. There are many devices and strategies available for implementing effective reduced lane widths (e.g., cones, drums, striping, barriers, barricades, etc.). The cost, maintainability, effectiveness, and safety of the various approaches probably varies widely. Only cones were evaluated in the studies.

4. Cones proved to be quick and easy to install and remove. However, they were frequently hit by large trucks and mobile homes when the 11.5-ft treatment was used.

5. Effective lane width reduction appears to be more practical for long-duration applications of several days or more. The time and initial cost to implement are relatively great; however, there is little labor or expense after installation.

6. On roadways with three or more lanes per direction, it may not be possible to accomplish the desired effective lane width reduction in the middle lanes without restriping the roadway.

7. Effective lane width reduction techniques may not suppress speeds long after the end of the narrow sections. Thus, the narrow lanes must be continued throughout the area where reduced speeds are desired.

CONCLUSION

For many years, work-zone speed control measures have been misused and, in some cases, abused. This is especially true for regulatory and advisory speed signing. As a result, driver respect for work-zone speed control efforts has suffered, and many agencies have lost confidence in attempting speed reductions. However, the research documented in this paper and elsewhere *(3)* indicates that traffic speeds can be significantly reduced at some construction and maintenance work zones in the interest of safety. For work-zone speed control to be effective, however, certain implementation considerations must be taken into account. These considerations have been identified and discussed along with the limitations of the various speed-control measures.

ACKNOWLEDGMENTS

The authors wish to thank Justin True, FHWA; and Blair Marsden and Lewis Rhodes, Texas State Department of Highways and Public Transportation, for their guidance and constructive comments during the course of the study. The research direction was guided by a Texas State Department of Highways and Public Transportation Technical Advisory Committee. The contributions of this Committee are gratefully acknowledged: Benjamin W. Bohuslav, supervising maintenance engineer, District 13; Walter Collier, district maintenance engineer, District 15; Billie E. Davis, district maintenance engineer, District 2; Milton Dietert, chief engineer of safety and maintenance operations, D-18; Herman Gadeke, district traffic engineer, District 15; Hunter Garrison, district maintenance engineer, District 12; Henry Grann, supervisory traffic engineer, District 18; Herman Haenel, supervisory traffic engineer, D-18T; Bobby Hodge, supervisory traffic engineer, District 2; Steve Levine, traffic management supervisor, District 12; Blair Marsden, senior traffic engineer, D-18T; Silas M. Prince, district maintenance engineer, District 11; Lewis Rhodes, senior traffic engineer, D-18T; Russell G. Taylor, engineering technician V, District 14; Milton Watkins, district maintenance engineer, District 18; and John Wilder, district maintenance engineer, District 14.

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Publication of this paper sponsored by Committee on Traffic Safety in Maintenance and Construction Operations.