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Roadside Hazards,
Traffic Barriers,
and Safety
Appurtenances

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Effect of Cargo Displacement on Vehicle Collision Behavior

JOHN J. LABRA

The potential for rollover after a vehicle has collided with a roadside barrier is greatest for vehicles that have a high center of gravity, such as school buses, intercity buses, and trucks. There are various computer programs that can simulate vehicle-barrier interaction, but they can be expensive to run and do not address the phenomenon of cargo displacement during vehicle impact with a roadside barrier. A computer program that can be used to estimate the effect of cargo displacement on secondary-impact vehicle roll response is described. It is applied to delineate the rollover potential of a school bus carrying unrestrained children during a collision with a highway barrier.

The rollover vaulting algorithm (RVA) (1) is a simple tool for evaluating the potential for vehicle rollover after collision with a highway barrier. In the RVA program, the single-unit vehicle is a 6-df model acted on by barrier-induced forces and tire-suspension reaction forces (Figure 1). Both vehicle and barrier are assumed to be nondeformable where the barrier is a vertical plane.

In RVA, the vehicle-barrier interaction takes place in three phases: (a) the initial impulsive impact with the barrier; (b) a continuous, nonimpulsive translational and rotational motion during redirection; and (c) a second impulsive impact when the rear of the vehicle swings around and strikes the barrier. Although the program provides insight into rollover potential, it, like other available vehicle-barrier models, precludes consideration of postimpact cargo movement during vehicle redirection. The RVA program was therefore modified in an effort to define the effect of cargo shift on vehicle performance.

Recently, an experimental test was conducted at Southwest Research Institute (SwRI) that involved the collision of a 9100-kg (20 000-lb) school bus with a flexible barrier at a speed of 97 km/h (60 miles/h) and an angle of 15°. The collision resulted in vehicle rollover. Sandbags positioned in the vehicle seats were used to simulate unrestrained children. The vehicle's center of gravity (including sandbags) was located 1.17 m (46

in) above the roadway, and the barrier height was 0.89 m (35 in). This relatively low center of gravity with respect to barrier height did not suggest a potential rollover problem. However, a film-analysis comparison with an earlier test performed by the Texas Transportation Institute showed that, after the bus redirected, significant sandbag displacement occurred as the rear section of the bus hit the barrier (at $t = 0.6$ s in Figure 2).

The decision to modify the original RVA computer program was based on the results of this full-scale test. The objective was to verify or refute the belief that the sandbag displacement during impact was a primary cause of vehicle rollover.

SPECIFIC PROGRAM MODIFICATIONS

Input Data

In the original RVA computer program (1), the user inputs the vehicle properties, including location of center of gravity; vehicle length, width, and height; and suspension properties with respect to a fixed reference frame such as that shown in Figure 1. These data remain the same in the modified version, RVA 2. The additional input requirements for the RVA 2 program are simply total cargo weight, initial position of the cargo center of gravity (x_2, y_2, z_2) with respect to the vehicle reference frame (x, y, z), and assumed displaced position of the cargo center of gravity (x_2', y_2', z_2') after secondary vehicle impact.

Computation of Center of Gravity

The additional input data are used in the program to estimate the change in the position of the vehicle's overall center of gravity and inertial properties because of cargo displacement. In RVA 2, the altered center of gravity is readily located by applying the theory of parallel forces. The vehicle center of gravity is first determined with the cargo excluded, i.e.,

$$x_1 = (W_v x_{v1} - W_c x_{c2}) / (W_v - W_c) \tag{1}$$

Figure 1. RVA vehicle model.

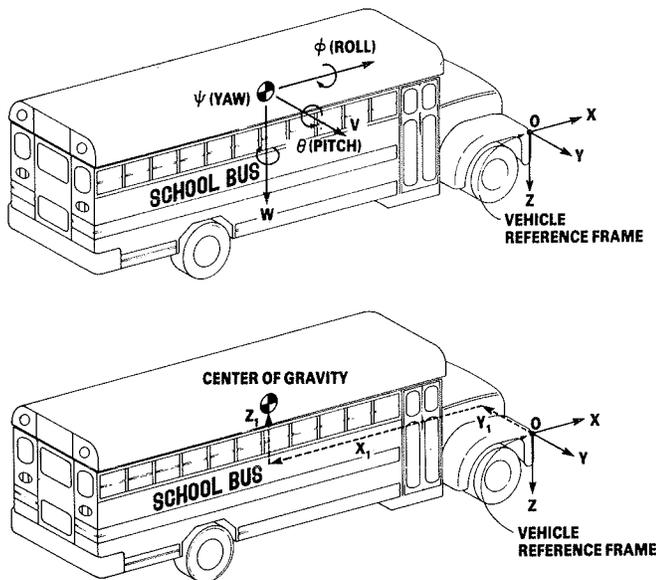
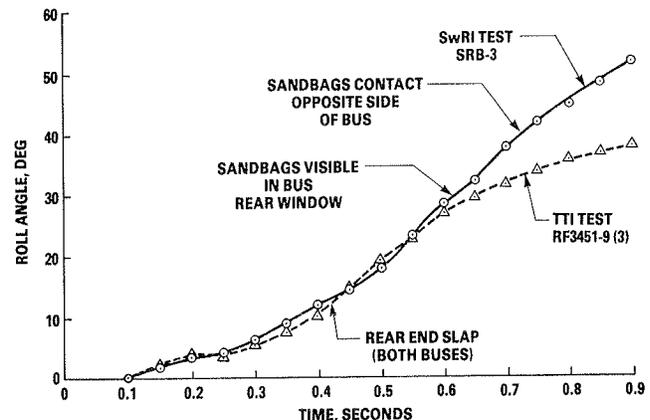


Figure 2. Effect of sandbag displacement on roll angle.



$$y_i = (W_v y_{i1} - W_C y_{i2}) / (W_v - W_C) \quad (2)$$

$$z_i = (W_v z_{i1} - W_C z_{i2}) / (W_v - W_C) \quad (3)$$

where

- x_1, y_1, z_1 = initial vehicle center-of-gravity coordinates including cargo,
 x_1', y_1', z_1' = initial vehicle center-of-gravity coordinates with cargo removed,
 W_v = total vehicle weight including cargo, and
 W_C = total cargo weight.

By using an assumed displaced-cargo position (x_2', y_2', z_2'), the location of the altered vehicle center of gravity (x_1, y_1, z_1) at the instant of secondary impact is evaluated as follows:

$$\bar{x}_1 = (1/W_v) [(W_v - W_C)x_1 + W_C x_2'] \quad (4)$$

$$\bar{y}_1 = (1/W_v) [(W_v - W_C)y_1 + W_C y_2'] \quad (5)$$

$$\bar{z}_1 = (1/W_v) [(W_v - W_C)z_1 + W_C z_2'] \quad (6)$$

where the RVA 2 program uses Equations 1-3 in conjunction with Equations 4-6.

Reevaluation of Vehicle Inertia Matrix

In the original RVA program (1), the moment-of-inertia matrix (e.g., I_{ij}) with respect to the vehicle center of gravity is evaluated in relation to the fixed-body coordinate system (I_{ij}^0) through application of the fundamental parallel-axis theorem. In RVA 2, the parallel-axis theorem is also applied to evaluate the effect of cargo displacement on the vehicle inertia matrix.

The RVA 2 program evaluates the expressions of cargo inertia in relation to the vehicle-body fixed reference frame (x, y, z) with the cargo in its original (x_2, y_2, z_2) and displaced (x_2', y_2', z_2') positions. Since the cargo is displaced within the confines of the vehicle, the analysis does not require an evaluation of the cargo-inertia matrix with respect to the cargo center of gravity.

In the program logic, the initial cargo-inertia matrix (with respect to the vehicle reference frame) is evaluated and subtracted from the overall

vehicle-inertia matrix. The inertia matrix of the displaced cargo in relation to the vehicle system is then added. The newly defined vehicle-inertia matrix is called in RVA 2 after the vehicle has redirected and secondary impact is imminent. At present, this computer flag assumes cargo translation when the yaw angle between the longitudinal centerline of the vehicle and the horizontal axis of the barrier is less than 3° (program logic precluded the use of a zero yaw angle to affect the cargo displacement). Although cargo displacement may occur to some degree after the initial impact, the SwRI full-scale test did not demonstrate significant displacement until secondary impact occurred. The updated center-of-gravity location and vehicle-inertia matrix are input into the original RVA energy equations (Equations 1-3) after the secondary impact to determine whether the resulting angular velocity is sufficient to cause vehicle rollover.

SIMULATION FINDINGS

The decision to modify the RVA program (1) was based, in part, on the recent SwRI full-scale test with a school bus. As a result, 12 initial simulations were performed in which a 9100-kg (20 000-lb) school bus was modeled interacting with 0.89-, 0.84-, and 0.76-m (35-, 33-, and 30-in) high rigid vertical barriers (see Table 1). An impact speed of 92 km/h (57 miles/h) and an approach angle of 17.5° were used in all 12 simulations. With the exception of cargo weight and position, all of the input vehicle properties (e.g., inertia matrix) were based on actual measurements obtained from the test vehicle. These corresponded to the actual test conditions.

For the first three simulations (cases 1, 2, and 3), the effect of cargo shift was excluded by inputting a zero cargo weight. According to the critical roll rate, which is defined in RVA and RVA 2 as the minimum roll angular velocity for rollover, these three simulations did not suggest a rollover problem. In the next series of simulations (cases 4, 5, and 6), a 2700-kg (6000-lb) cargo was used [total vehicle weight maintained at 9100 kg (20 000 lb)], and the cargo initial center-of-gravity position coincided with the center of gravity. Although this cargo position is an unlikely situation, these simulations were performed because

Table 1. Rollover potential of school bus.

Case	Barrier Height (m)	Cargo Position (m)									Post-Secondary-Impact Roll Rate	
		Vehicle Center of Gravity ^a (m)			Original			Displaced			Positive Toward Barrier (°/s)	Percentage of Critical Rate
		x_1	y_1	z_1	x_2	y_2	z_2	x_2'	y_2'	z_2'		
1 ^b	0.89	-5.39	-1.22	-0.28	-5.39	-1.22	-0.28	-5.39	0	-1.04	57.1	33
2 ^b	0.84	-5.39	-1.22	-0.33	-5.39	-1.22	-0.33	-5.39	0	-1.09	64.3	38
3 ^b	0.76	-5.39	-1.22	-0.41	-5.39	-1.22	-0.41	-5.39	0	-1.17	101.0	63
4	0.89	-5.39	-1.22	-0.28	-5.39	-1.22	-0.28	-5.39	0	-1.04	88.3	64
5	0.84	-5.39	-1.22	-0.33	-5.39	-1.22	-0.33	-5.39	0	-1.09	107.8	80
6	0.76	-5.39	-1.22	-0.41	-5.39	-1.22	-0.41	-5.39	0	-1.17	140.8	110
7	0.89	-5.39	-1.22	-0.28	-5.39	-1.22	-0.28	-6.15	0	-1.04	94.7	68
8	0.84	-5.39	-1.22	-0.33	-5.39	-1.22	-0.33	-6.15	0	-1.09	114.6	84
9	0.76	-5.39	-1.22	-0.41	-5.39	-1.22	-0.41	-6.15	0	-1.17	147.7	113
10	0.89	-5.39	-1.22	-0.28	-5.39	-1.22	-0.28	-5.39	0	-1.30	94.0	73
11	0.89	-5.39	-1.22	-0.28	-5.39	-1.22	-0.28	-5.39	0	-1.55	97.1	82
12	0.89	-5.39	-1.22	-0.28	-5.39	-1.22	-0.28	-5.39	0	-1.80	98.2	90

Notes: 1 m = 3.3 ft; 1 m·kg·s⁻² = 0.0115 in·lb·s⁻².

Vehicle was a 9100-kg school bus, for which speed = 92 km/h, angle = 17.5°, center of gravity = 1.17 m above the roadway, cargo weight = 2700 kg, roll = 495 m·kg·s⁻², pitch = 9165 m·kg·s⁻², and yaw = 9124 m·kg·s⁻².

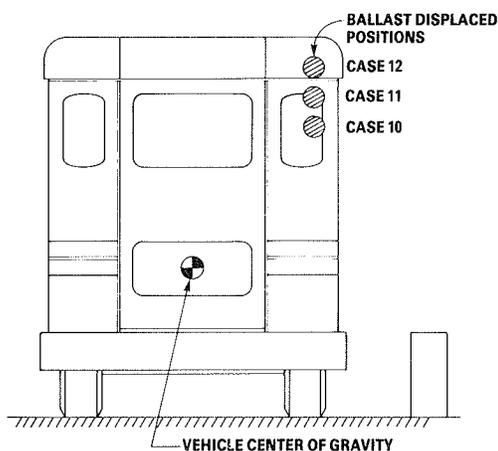
^a Locations in vehicle fixed reference frame (x, y, z) with origin at barrier-vehicle contact point (Figure 1).

^b Zero cargo used.

they are analogous to what is indirectly assumed with existing vehicle-barrier computer programs when a single center of gravity is delineated. At secondary impact, the cargo was assumed to be displaced 1.22 m (48 in) laterally (to the vehicle interior side wall) and 0.76 m (30 in) vertically (above the center of gravity). The results, given in Table 1, indicate a significant increase in post-secondary-impact roll rate, particularly in case 6, where the bus rolled over the barrier (i.e., 110 percent of critical roll rate).

The effect of a longitudinal cargo displacement on vehicle rollover potential was considered in cases 7, 8, and 9. For these simulations, the assumed displacement was 1.22 m (48 in) laterally, 0.76 m (30 in) vertically (upward), and 0.76 m (30 in) longitudinally (rearward). The findings demonstrated only minor effects on rollover potential.

Figure 3. Typical assumed positions of displaced cargo.



Simulations 10, 11, and 12 involved the 0.89-m (35-in) high barrier. In these three cases, the cargo was also displaced to the vehicle side wall. However, for each simulation, a different (displaced) cargo position above the vehicle center of gravity was imposed (see Figure 3). As Table 1 indicates, when the cargo was displaced 1.27 and 1.52 m (50 and 60 in) above the vehicle center of gravity, the resulting roll velocities reached 82 and 90 percent of the estimated critical roll values, respectively. These are substantially higher values than those that resulted from the simulation in which the cargo was not considered (case 1).

All 12 simulations assumed an initial cargo location at the vehicle center of gravity and either a 0- or 2700-kg (6000-lb) cargo. Test-vehicle measurements defined a cargo weight of 3200 kg (7100 lb) and a center-of-gravity location as shown in Figure 4. These data were used in three additional simulations. Results with the cargo weight and position included are given in Table 2. When the cargo was displaced 1.52 m (60 in) above the vehicle center of gravity (at the side wall), the hazardous rollover mode occurred (case 15). The significance of this finding is realized when roll-rate results

Figure 4. Cargo location in full-scale test.

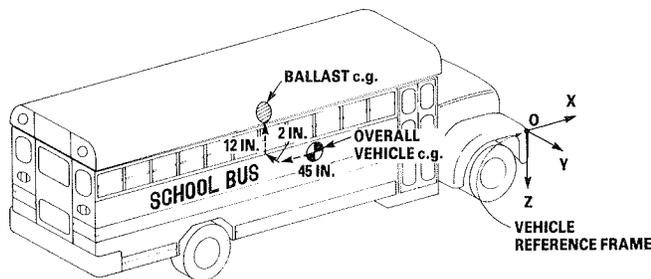


Table 2. School-bus simulations with 3200-kg cargo.

Case	Vehicle Center of Gravity ^a (m)			Cargo Position (m)						Post-Secondary-Impact Roll Rate	
	x ₁	y ₁	z ₁	Original			Displaced			Positive Toward Barrier (°/s)	Percentage of Critical Rate
13	-5.39	-1.22	-0.28	-6.53	-1.27	-0.58	-6.53	0	-1.30	96.5	76
14	-5.39	-1.22	-0.28	-6.53	-1.27	-0.58	-6.53	0	-1.55	103.6	89
15	-5.39	-1.22	-0.28	-6.53	-1.27	-0.58	-6.53	0	-1.80	107.7	102

Notes: 1 m = 3.3 ft. Barrier height = 0.89 m, speed = 92 km/h, and angle = 17.5°. Cargo weight and initial location correspond to measured values from actual SwRI vehicle that rolled over during full-scale test.

^aLocations in vehicle fixed reference frame (x, y, z) with origin at barrier-vehicle contact point (Figure 1).

Table 3. Effect of barrier height on school-bus collision behavior.

Case	Barrier Height (m)	Vehicle Center of Gravity ^a (m)			Cargo Position (m)						Post-Secondary-Impact Roll Rate	
		x ₁	y ₁	z ₁	Original			Displaced			Positive Toward Barrier (°/s)	Percentage of Critical Rate
16	0.91	-5.39	-1.22	-0.25	-6.53	-1.27	-0.56	-6.56	0	-1.78	106.4	101
17	0.94	-5.39	-1.22	-0.23	-6.53	-1.27	-0.53	-6.53	0	-1.75	105.4	100
18	0.97	-5.39	-1.22	-0.20	-6.53	-1.27	-0.51	-6.53	0	-1.73	103.9	99

Notes: 1 m = 3.3 ft. Speed = 92 km/h and angle = 17.5°.

^aLocations in vehicle fixed reference frame (x, y, z) with origin at barrier-vehicle contact point (Figure 1).

are compared with those from the case 1 simulation (Table 1), which excluded cargo displacement for the 0.89-m (35-in) barrier. Further, proper delineation of the cargo center of gravity (cases 13, 14, and 15) in the computer program in comparison with the assumption that it corresponded with vehicle center of gravity (cases 10, 11, and 12) also demonstrated a greater rollover potential.

A final series of simulations was performed to determine the barrier height that would have resulted in a safe redirecting of the school bus (see Table 3). For these simulations, an assumed cargo displacement (Figure 4), defined in case 15 (Table 2), was used. As the data show, even with the barrier raised to a 0.97-m (38-in) height, the resultant roll angular velocity was extremely high (99 percent of the critical roll rate). Note that this series assumed an extreme lateral and vertical cargo displacement. However, these displacement extremes cannot be overly conservative because of the many vehicle and barrier variables involved in an actual crash that are not addressed in this simple program (e.g., vehicle and barrier deformation).

CONCLUSIONS

The problem of cargo displacement is integral to

studies of the postimpact behavior of heavy vehicles in collision with barrier systems. This investigation substantiates the relationship. Moreover, the modified rollover vaulting algorithm (RVA 2) is an efficient tool with which to consider this complex phenomenon. Although the sample cases in this study were limited to those involving a school bus, RVA 2 can be used to simulate a wide range of single-unit vehicles where a potential cargo shift could contribute to hazardous vehicle behavior.

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REFERENCE

1. J.J. Labra, J. Rudd, and J. Ravenscroft. Roll-over-Vaulting Algorithm for Simulating Vehicle-Barrier Collision Behavior. TRB, Transportation Research Record 566, 1976, pp. 1-12.

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The Rural Mailbox: A Little-Known Roadside Hazard

HAYES E. ROSS, JR., KENNETH C. WALKER, AND W. J. LINDSAY

The results of seven full-scale crash tests conducted at the Texas Transportation Institute to evaluate the impact behavior of rural mailbox installations are reported. Three of the seven tests—two of single-box installations and one of a four-box installation—involved commonly used wood-post supports. Two tests of single-box installations involved promising new support concepts that use standard steel pipe with a breakaway mechanism. The final two tests involved a steel-pipe, multiple-box support in the shape of an inverted U. The results showed that installations with multiple boxes mounted on boards pose a serious hazard to motorists because the board can easily penetrate the windshield. The inverted-U design eliminated the penetration problem, but the change in vehicle momentum on impact exceeded the recommended limit. The results also showed that, for a single-box installation, a pipe post performs better than a wood post. Careful attention must be given to the box-to-post attachment to prevent separation during impact and thus minimize the potential for windshield penetration by the mailbox.

Little attention has been given to rural mailbox structures as potential hazards to motorists. Although the incidence of vehicle collisions with mailboxes may be small in comparison with collisions with more formidable hazards, mailbox accidents are not statistically insignificant. Table 1 gives a summary of 1972 data from four states on road accidents involving mailboxes (1). By using available accident data, the Federal Highway Administration (FHWA) has estimated that approximately 200 or more fatalities occur each year as a result of mailbox accidents.

The U.S. Postal Service issues design specifications for the rural mailbox itself and its vertical position (2), but it has no specifications for mailbox supports. In 1969, the American Association of State Highway Officials (AASHO) published guidelines for erecting mailboxes on

highways, including suggested structural supports (3). Some of these recommended supports, however, were found to be undesirable in the testing reported in this paper. A cursory review of several state and county agencies reveals an almost total absence of any standards for mailbox supports. Only one state is known to have such standards.

Figure 1 shows some of the "ingenious" devices used to support mailboxes. These include steel tractor wheels, old-time stoves, water pumps, plow-shares, milk cans filled with concrete, massive I-beams, and pipes. Such supports may be artistic to some, but most are serious roadside hazards.

Mailboxes are hazardous primarily because of the mounting height of the box. Most boxes are mounted approximately 42 in (106.7 cm) above ground to make it easier for the mail carrier to place mail in the box. As Figure 2 shows, this height places the box in direct line with the windshield of many vehicles. The support systems shown in Figure 2 were used for illustrative purposes only and are not considered crashworthy systems. The base of each of the three different-sized boxes is approximately 42 in above the ground.

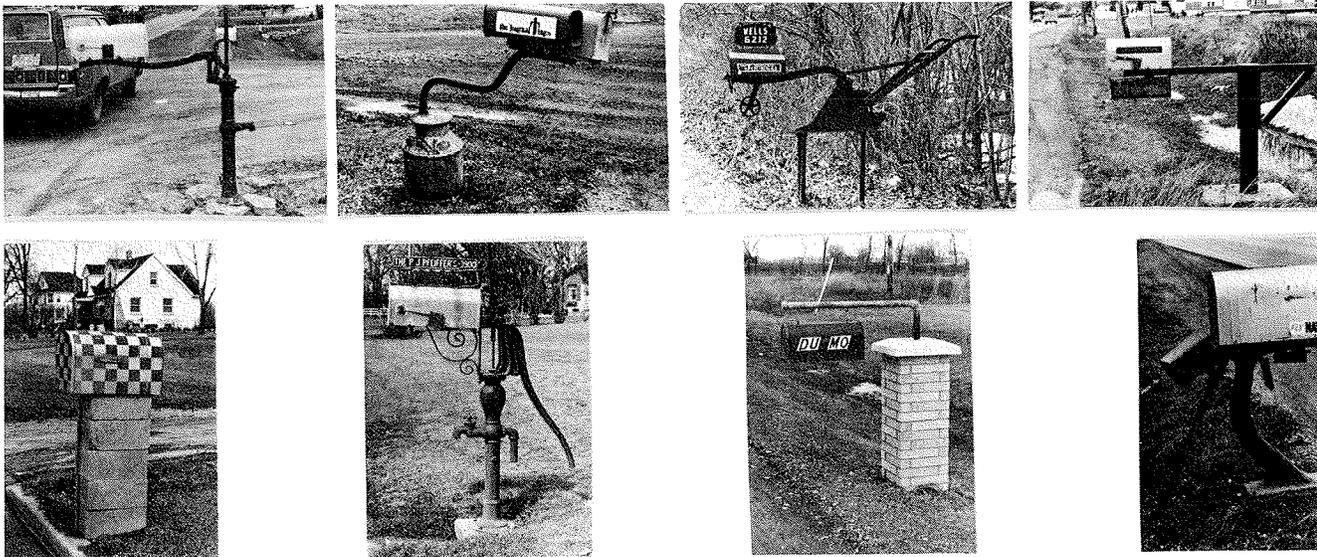
Mounting several boxes on a wood board compounds the problem, since the board can spear through the windshield. Field installations of multiple boxes are shown in Figure 3. Figure 4 shows what may be a record for the number of boxes mounted on one continuous support (more than 200).

To gain insight into the hazard of mailbox installations, FHWA and the Texas State Department of Highways and Public Transportation (TSDHPT) elected to conduct limited full-scale crash-test

Table 1. Mailbox accident data from four states for the year 1972.

State	Total Mailbox Accidents	Percentage of Total Fixed-Object Accidents	Fatal Mailbox Accidents	Percentage of Total Fatal Fixed-Object Accidents	Personal-Injury Accidents	Percentage of Total Fixed-Object, Personal Injury Accidents	Property-Damage Accidents	Percentage of Total Fixed-Object, Property-Damage Accidents
Michigan	3326	5.2	15	2.3	726	3.0	2585	6.6
Missouri	272	3.0	7	3.0	93	2.7	172	3.2
Texas	1053	1.9	12	1.4	225	1.3	816	2.1
Washington	254	3.5	0	0.0	77	2.5	177	4.4

Figure 1. Single-box installations.



programs. Tests were conducted on widely used wood-post supports, two promising new support concepts that make use of standard steel pipe and a breakaway mechanism, and a steel-pipe, multiple-box support in the shape of an inverted U. The third design was developed by TSDHPT to reduce the hazard of wood-post and wood-beam multiple-box designs.

This paper summarizes the tests conducted and their results. Complete details of the tests can be found elsewhere (4,5).

TEST CONDITIONS

Table 2 gives a summary of the seven test conditions. Further design details are given in subsequent sections.

It was clear from previous tests of sign posts (4) that high-speed impacts would be more hazardous than low-speed impacts for the mailbox installations to be evaluated. Therefore, the impact speed in each test was approximately 60 miles/h (26.8 m/s). Test data were recorded and analyzed in accordance with recommended guidelines for testing roadside appurtenances (6). Soil at the test site conformed to recommended standards (6).

Test Mailboxes and Supports

Wood-Post Support

Tests 22, 23, and 24 involved a single 4x4-in (10.2x10.2-cm) wood-post support. In each of these three tests, the post was set 2 ft (0.61 m) into a drilled hole 18 in (45.7 cm) in diameter, the hole

was backfilled, and then the post was tamped into position. The base of the mailbox was 42 in (106.7 cm) above ground level for all tests.

Figure 5 shows the installation details. Although the post was similar in these tests, the arrangement, size, and attachment of the mailbox differed.

Single Mailbox

Test 22 used a single No. 1-A mailbox (2) mounted on a 1.625x8x19.5-in (4.13x20.3x49.5-cm) yellow-pine timber cap. The mailbox was attached to the cap by six 1.5-in (3.81-cm) long composition roofing nails. Three sixteenpenny nails were used to attach the cap to the post. Two triangular braces, each with four eightpenny nails, secured the timber cap to the post. Figure 6 shows the installation from the direction of impact and the mailbox height with respect to the test vehicle.

Single Mailbox with Strap Attachment

Installation details in test 23 were identical to those in the previous test with the exception of the two straps that were added to increase the mailbox-to-post connection strength. Commercially available hanger straps were used. Static load tests of the strap showed it had a tensile strength of approximately 500 lbf (2224 N). The width of the 24-gauge strap was 0.75 in (1.91 cm); the holes were 0.25 in (0.64 cm) in diameter on 1-in (2.54-cm) centers. Each strap was attached to the post with ten 1.5-in (3.81-cm) long composition roofing nails,

Figure 2. Mailbox height in relation to vehicle windshield.

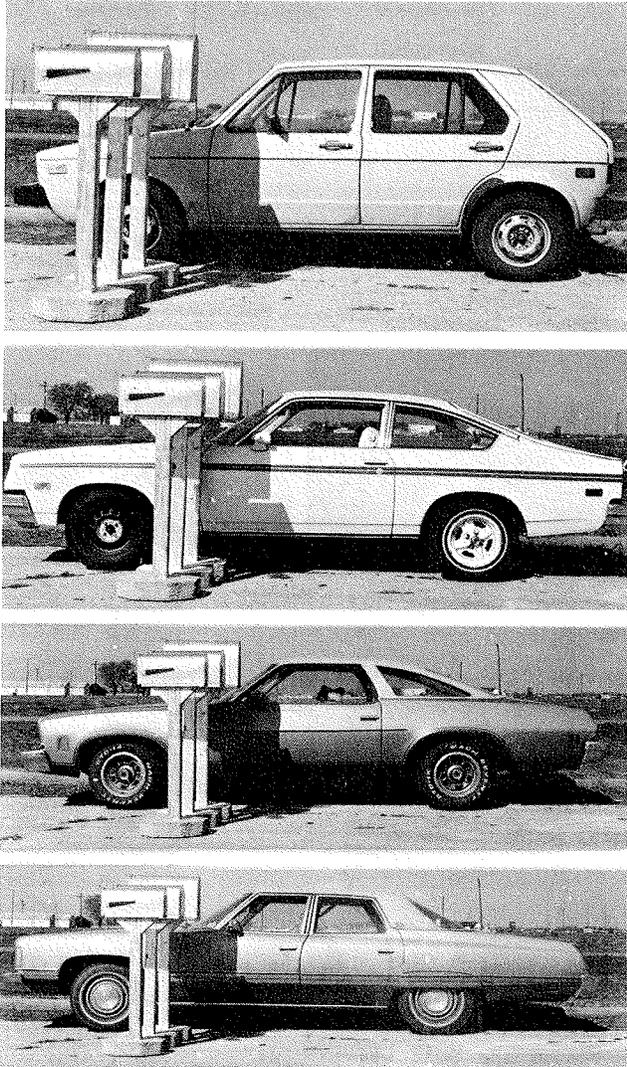


Figure 3. Multiple-box installation.

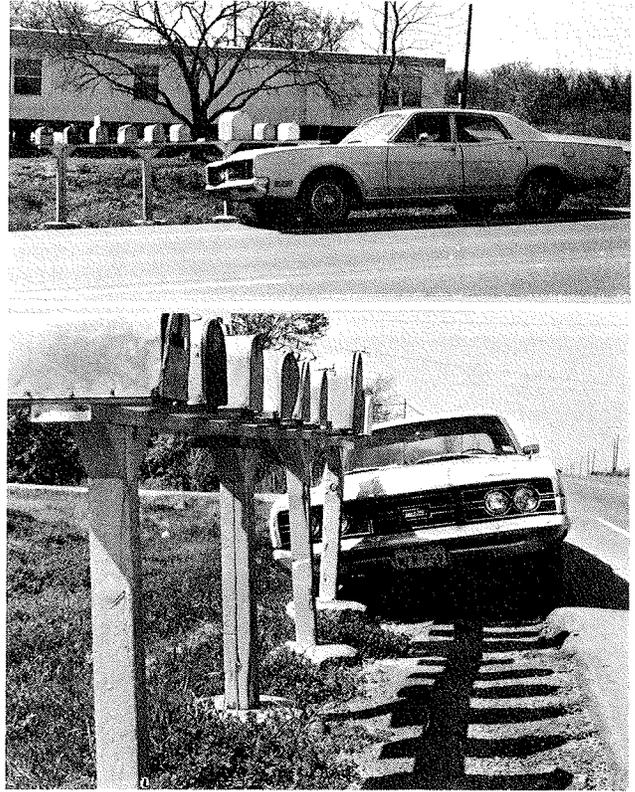


Figure 4. Field installation with numerous boxes.



five on each side, beginning 6 in (15.24 cm) from the bottom of the timber cap and continuing on 1-in (2.54-cm) centers. Connection to the timber cap was made with one 1.5-in (3.81-cm) long roofing nail on each side of the mailbox. Details of the installed strap and the direction of impact are shown in Figure 7.

Multiple Mailboxes on Wood Support

Test 24 involved four mailboxes: one No. 1, two No. 1-A, and one No. 2 (2). Each mailbox was attached to a yellow-pine timber cap with six 1.5-in (3.81-cm) long composition roofing nails. A 1.625x5.625x57-in (4.13x14.3x144.8-cm) support arm held the timber caps, each with three sixteenpenny nails. The support arm was connected to the 4x4-in (10.2x10.2-cm) post with four sixteenpenny nails and two triangular braces, each held by four eightpenny nails. The installation is shown in Figure 8.

Standard Steel Pipe Posts

In tests 25 and 26, 1.5-in (3.81-cm) inside-diameter (ID) standard steel pipe with two post-to-base combinations was evaluated. Both are commercially

available support systems. Installation details are shown in Figure 9. The base in both cases was a 1.5-in-ID standard steel pipe 30 in (76.2 cm) long with a 5-in (12.7-cm) auger on one end to facilitate installment. The base was installed by simply digging a small hole and then turning the augered pipe with a pipe wrench.

In test 25, a lap-spliced steel pipe with bolted shear connection, shown in Figure 9, was evaluated. The lower bolt was a 0.375x4.5-in (0.95x11.4-cm) retainer bolt, and the other was a 0.3125x4.5-in (0.79x11.4-cm) shear bolt with a short section machined down to the minor thread diameter of 0.2524 in (0.64 cm).

In test 26, the post-to-base assembly consisted of a frangible coupling. It incorporated two retainer straps attached with two 0.375x2.5-in (0.95x6.35-cm) bolts. Four 0.3125x1.5-in (0.79x3.81-cm) bolts connected the coupling to the base and post. All bolts were grade 2.

The mailbox bracket was attached to the threaded support post with lock nuts. One lock nut was set below the mailbox bracket. In test 25, one lock nut was used to hold the bracket down; in test 26, two

Table 2. Summary of test conditions.

Test No.	Vehicle Weight (lb)	Impact Speed (miles/h)	Type and Size of Support Post	Cap and Support Arm Details	Post Embedment Depth (ft)	Number of Mailboxes on Post	Type of Mailbox ^a
22 (4)	2270	58.2	Wood, 4 x 4 in (nominal)	Figures 5 and 6	2.0	1	No. 1-A
23 (4)	2270	62.6	Wood, 4 x 4 in (nominal)	Figures 5 and 7	2.0	1	No. 1-A
24 (4)	2440	58.8	Wood, 4 x 4 in (nominal)	Figures 5 and 8	2.0	4	One No. 1, two No. 1-A, one No. 2
25 (4)	2260	58.8	Standard steel pipe ^b , 1.5-in ID	Figures 9 and 10	2.3	1	No. 1-A
26 (4)	2260	60.9	Standard steel pipe ^c , 1.5-in ID	Figures 9 and 10	2.3	1	No. 1
5 (5)	2470	61.3	Standard steel pipe, 1.25-in ID	Figures 11 and 12	2.0	7	Three No. 1, three No. 1-A, one No. 2
6 (5)	2500	60.4	Standard steel pipe, 1-in ID	Figures 11 and 13	2.0	5	Two No. 1, two No. 1-A, one No. 2

Note: 1 lb = 0.45 kg; 1 mile = 1.6 km; 1 ft = 0.3 m; 1 in = 2.54 cm.

^aU.S. Postal Service type 1 standard rural mailboxes.

^bLap-splice design with bolted shear connection.

^cFrangible cast-iron coupling with retainer straps.

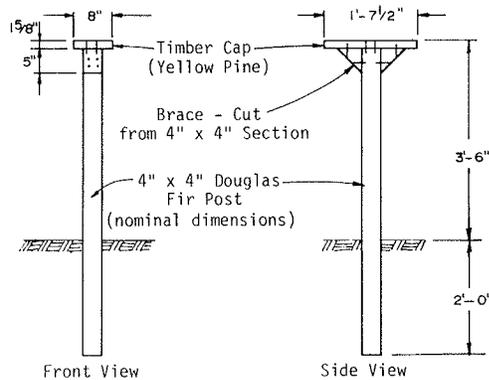
Figure 5. Installation details for wood-post mailbox supports.

Connections:

- 1) Cap-to-post: Three 16 penny nails
- 2) Brace-to-cap and post: Four ea. 8 penny nails
- 3) Mailbox-to-cap: Six 1½ in. composition roofing nails

Metric Conversions:

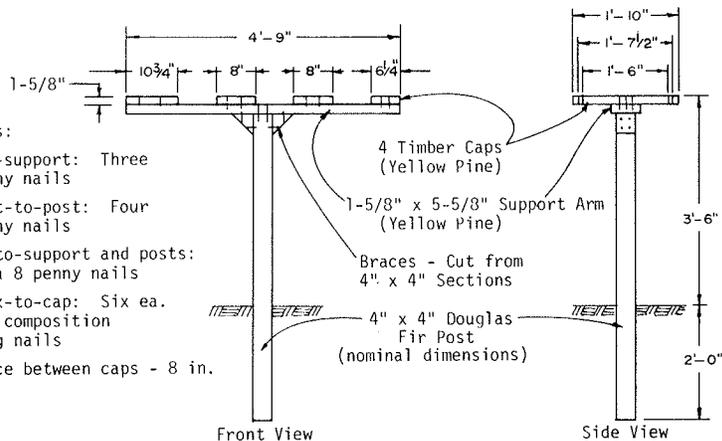
- 1 in. = 2.54 cm
- 1 ft = 0.305 m



SINGLE SUPPORT FOR #1-A MAILBOX (Tests 22 and 23)

Connections:

- 1) Cap-to-support: Three 16 penny nails
- 2) Support-to-post: Four 16 penny nails
- 3) Brace-to-support and posts: Four ea 8 penny nails
- 4) Mailbox-to-cap: Six ea. 1½ in. composition roofing nails
- 5) Distance between caps - 8 in.



SINGLE SUPPORT FOR MULTIPLE MAILBOXES (Test 24)

nuts were used. In addition, an ornamental S-brace was used in test 25 to connect the bracket and support post. For both pipe tests, six 0.1875x0.75-in (0.48x1.91-cm) bolts with lock washers were used to attach the mailbox to the bracket. A No. 1-A mailbox was used in test 25, and a No. 1 mailbox was used in test 26. The completed installations can be seen in Figure 10.

Multiple Mailboxes on Steel-Pipe Support

Details of the mailbox installations for tests 5 and 6 are shown in Figure 11. Details of the mailbox-to-support attachment are shown in Figures 12 and 13. The smaller boxes were attached to the brackets by four 0.1875x0.75-in (0.48x1.91-cm) bolts with lock washers. The middle-sized boxes and

larger boxes were attached to the brackets by six 0.1875x0.75-in bolts with lock washers. The completed installations are shown in Figure 14.

Test Vehicles

The vehicles for tests 22-26 were 1972 and 1973 Chevrolet Vegas that weighed approximately 2250 lb (1022 kg). The vehicles used in tests 5 and 6 were 1974 Chevrolet Vegas that weighed approximately 2500 lb (1135 kg). Data acquisition systems consisting of high-speed motion pictures and on-board instrumentation (4) were used.

PERFORMANCE SPECIFICATIONS

Although there are no formal performance

specifications for mailbox supports, it seems logical that such appurtenances should comply with recognized safety standards for sign posts, light poles, and traffic signals as published by the American Association of State Highway and Transportation Officials (AASHTO). According to AASHTO specifications (7), "Satisfactory dynamic performance is indicated when the maximum change in momentum for a standard 2250 lb (1020 kg) vehicle, or its equivalent, striking a breakaway support at speeds from 20 mph to 60 mph (32 km/h to 97 km/h), does not exceed 1100 pound-seconds (4893 N-sec), but

desirably does not exceed 750 pound-seconds (3336 N-sec)."

"Breakaway supports", as used in the AASHTO specification, is a generic term meant to include all types of sign supports whether the release mechanism is a slip plane, plastic hinges, fracture elements, or a combination of these. The specification states that "Breakaway structures should also be designed to prevent the structure or its parts from penetrating the vehicle occupant compartment." The specification also alludes to the unacceptability of vehicle rollover after impact with the sign post.

TEST RESULTS

Data derived from accelerometer readings include change in momentum, peak acceleration, highest 50-ms average acceleration, and duration of event. Peak acceleration is found directly from the accelerometer readings, and the highest 50-ms acceleration is derived by averaging the readings over all 50-ms intervals. Change in momentum is found by first integrating the acceleration-versus-time plot given by the accelerometer signals. This is the change in vehicle velocity, which, when multiplied by the vehicle mass, gives the momentum change. Since change in momentum is time dependent, the length of time over which the integration is performed must be specified. Because of this time dependency, guidelines have been established to determine "duration of event" (6) for computation.

Table 3 gives a summary of the five tests (impact conditions are given in Table 2). A discussion of each test follows.

Test 22

In test 22, almost immediately after impact, the mailbox, mailbox base, and base braces separated from the post. Shortly thereafter, the post broke away at ground level. Because of inertia, the mailbox and timber cap remained almost stationary as the post broke away. Then the box struck the top of the hood and rolled into the windshield, breaking and dishing the windshield and bending the molding above it. It did not, however, penetrate into the passenger compartment. Other than a broken windshield, the vehicle sustained only minor damage.

Test 23

Analysis of mailbox and support response in test 22 indicated that a stronger connection was needed to keep the mailbox attached to the post. The design was modified to include two galvanized hanger straps to secure the mailbox to the support. This design was developed to keep the breakaway features of the wood post and at the same time keep the post and mailbox from separating on impact.

In test 23, the post split on impact at the top where the connecting straps were attached to the post and then broke on the side away from impact. The mailbox and timber cap gained little velocity during the initial impact. After the support broke at ground level, the vehicle continued moving forward and the mailbox struck and broke the windshield. However, the box did not penetrate into the passenger compartment.

Test 24

In test 24, the support post broke on impact at ground level and at bumper height [20 in (50.8 cm)]. The support arm also separated from the post soon

Figure 6. Completed installation, test 22.

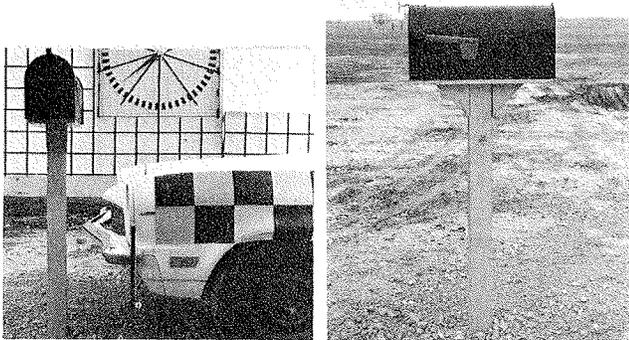


Figure 7. Completed installation, test 23.

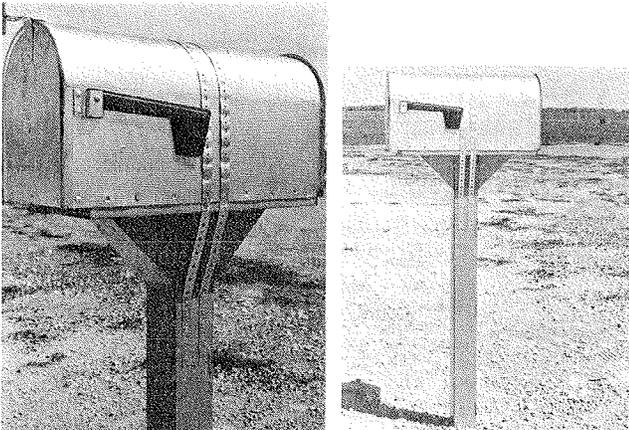


Figure 8. Completed installation, test 24.

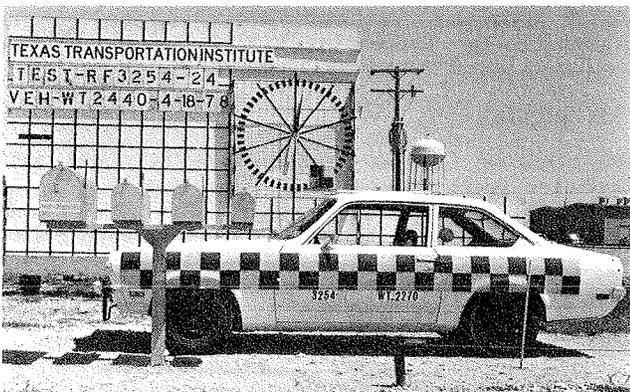


Figure 9. Installation details, tests 25 and 26.

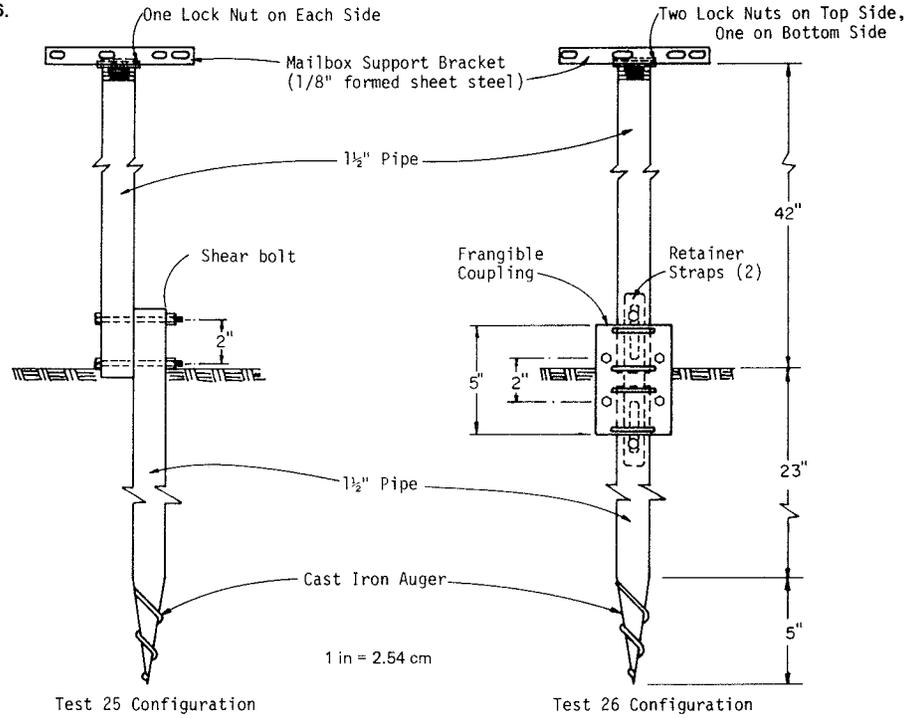
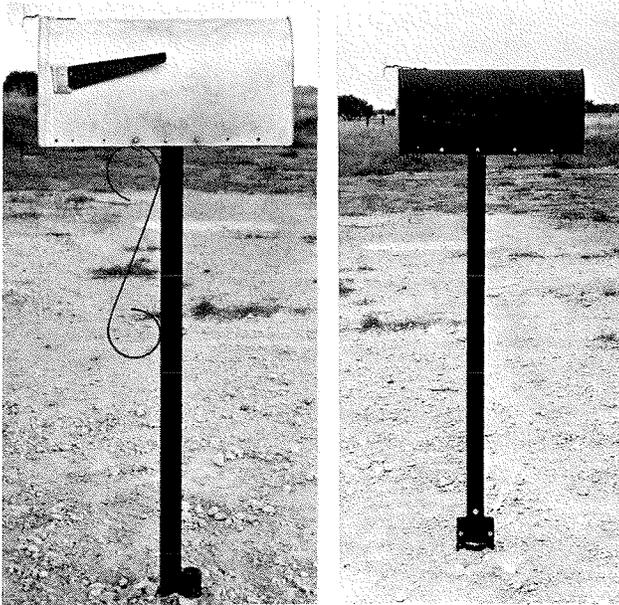


Figure 10. Test installations with standard steel pipe supports.

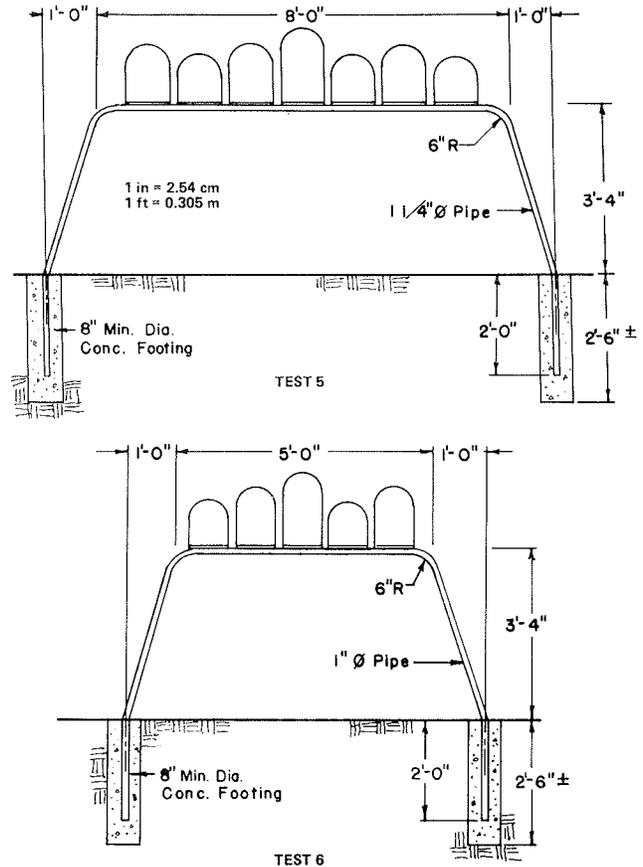


after impact without gaining any significant velocity. As a result, the support arm and four mailboxes hung in midair as the vehicle drove into it. When the support arm hit the windshield, three mailboxes entered the passenger compartment along with 75 percent of the support arm (see Figure 15). The dummy was struck on the chin by the support arm and was almost decapitated. The impact also forced the seat back. There is little doubt that a driver would have been killed under these circumstances.

Test 25

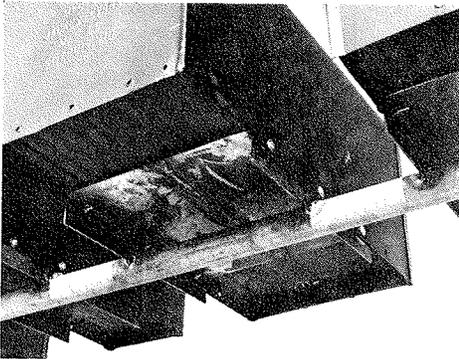
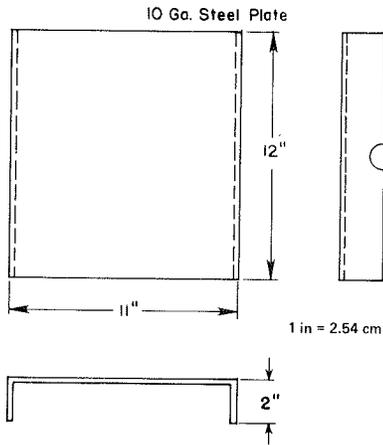
Test 25 was the first test in which the windshield

Figure 11. Mailbox support configurations, tests 5 and 6.



was not struck by some part of the mailbox or its support. Although the mailbox broke free of the support, it remained intact long enough to attain a

Figure 12. Mailbox-to-support attachment details, test 5.



velocity approximately equal to that of the vehicle. As a result, the mailbox rode along on the hood and harmlessly fell away. Investigation of the bracket-to-post connection showed that the lock-nut threads stripped, allowing the box to separate from the post. Two lock nuts on the top of the bracket would probably have prevented this separation (see the description of test 26). The support and the mailbox were too badly damaged to be reused. The anchor post, however, was not damaged and could be reused. The shear bolt fractured as it was designed to do, retaining the post but allowing it to rotate downward. Vehicle damage was minimal.

Test 26

In test 26, the frangible coupling broke, and the retainer straps held the post after fracture as desired. The mailbox remained with the support until the vehicle rode over the box and the support. Note that two lock nuts were used on the top side of the bracket to attach the bracket to the post (Figure 9). The retainer straps functioned as designed, preventing translational movement of the support post.

As in test 25, the support and the mailbox were damaged beyond repair and reuse. The anchor post was also damaged and would probably have to be replaced. Vehicle damage was minimal.

Test 5

In test 5, at impact the support formed a hook and the pipe wrapped around the hood of the vehicle. Continued forward vehicle movement broke the lead support post. The boxes and the remaining portion of the support were then ridden down, and the boxes

Figure 13. Mailbox-to-support attachment details, test 6.

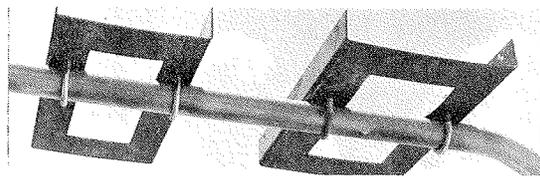
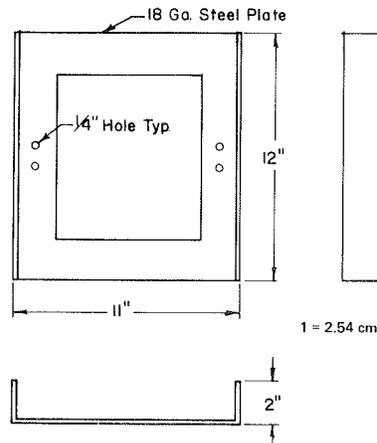
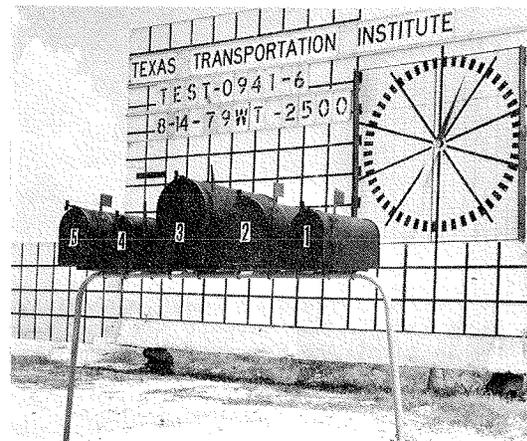
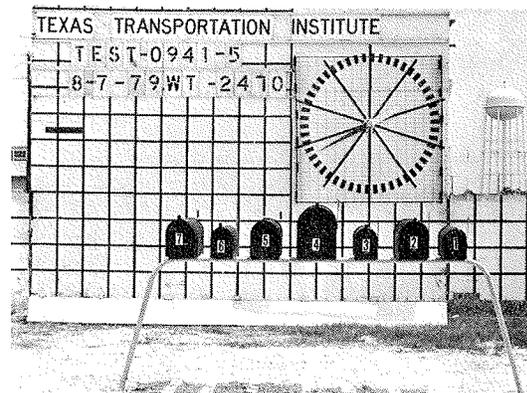


Figure 14. Completed installation, tests 5 and 6.



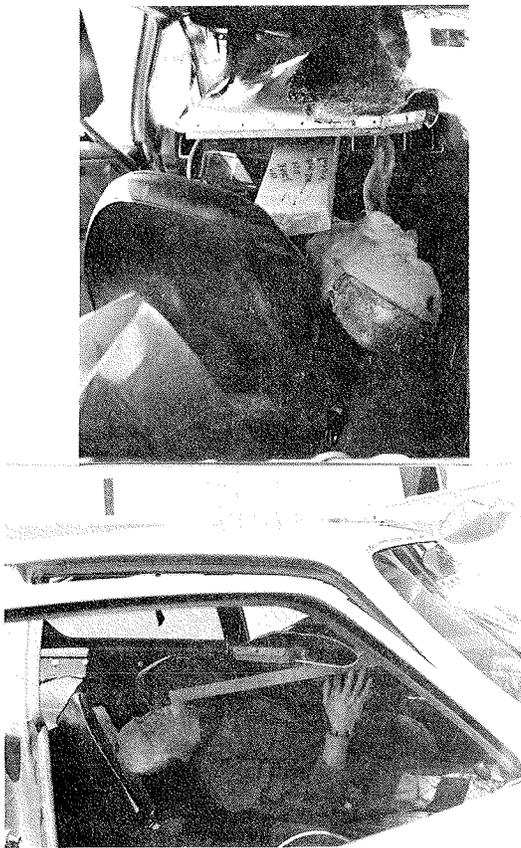
were stripped from the pipe. Impact occurred at the left quarter point of the front bumper. After impact, the vehicle spun or yawed approximately 120°. The vehicle sustained considerable front-end damage. There was no windshield contact by any part

Table 3. Summary of test results.

Test No.	Change in Momentum (lbf-s)	Comments
22	65	Box and cap separated from post and hit windshield
23	49	Box and cap separated from post and hit windshield
24	151	Support arm separated from post and penetrated windshield
25	217	Box and cap separated from post but did not hit windshield
26	114	Box and cap remained with post until box hit ground; no windshield contact
5	1925	Change in momentum exceeded recommended limit of 1100 lbf-s; boxes remained on support
6	1267	Change in momentum exceeded recommended limit of 1100 lbf-s; boxes remained on support

Note: 1 lbf-s = 4.45 N-s.

Figure 15. Interior vehicle damage, test 24.



of the installation. Change in vehicle momentum considerably exceeded the recommended limit.

Test 6

The impact behavior of the vehicle and the support in test 6 was similar to that in test 5. In this case, the lead support did not fracture and the boxes were not stripped from the pipe as the vehicle rode over the support. The vehicle yawed about 180° before coming to a stop. Although the change in vehicle momentum exceeded the recommended limit, this design represents a step in the right direction and is a vast improvement over the design evaluated in test 24.

CONCLUSIONS AND RECOMMENDATIONS

This study has delineated several noteworthy points:

1. Although the incidence of vehicle impacts with mailbox installations may be small in comparison with collisions with more formidable roadside hazards, mailbox accidents occur and the consequences are not always insignificant.
2. Although the U.S. Postal Service has design specifications for the mailbox itself, it has no such specifications for the mailbox support.
3. Only one state is known to have and to enforce a standard design for mailbox supports.
4. There is a bewildering variety of support designs, many of which are unnecessarily hazardous to the traveling public.

To gain insight into the hazard of mailbox installation, FHWA and TSDHPT elected to conduct limited full-scale crash-test programs. Tests were conducted on widely used wood-post supports, two promising new support concepts that use standard steel pipe, and a multiple-box support that uses an inverted-U steel pipe. All tests were conducted at approximately 60 miles/h (26.8 m/s).

The conclusions drawn as a result of the tests are as follows:

1. Most mailboxes are mounted approximately 42 in (106.7 cm) above the ground, which places them in direct line with the windshield of most automobiles. It is therefore of primary importance that the strength of the box-to-post attachment be sufficient to prevent separation during impact. This will reduce the potential for the box to impact the windshield.
2. Multiple-mailbox installations usually have a beam or support member running parallel to the roadway. At impact, the beam can easily spear through the windshield, which obviously can cause and has caused serious injuries and fatalities. The inverted-U steel-pipe, multiple-box support eliminated the spearing problem, but change in vehicle momentum exceeded recommended limits. At the present time, there are no multiple-box support systems known to be acceptable.
3. Standard steel pipe that has a post-to-base design and incorporates some type of breakaway feature offers considerable promise as a single-mailbox support. Tests of a lap-spliced bolted-base design and a frangible coupling design proved to be satisfactory. The cost of such systems is nominal.
4. Wood posts may present special problems as mailbox supports when they are used on roadways where vehicle operating speeds exceed approximately 40 miles/h (64.4 km/h). The major difficulty concerns the box-to-post attachment and the brittle nature of wood under impact conditions. A wood post will typically fracture at bumper height and ground level. Depending on the size of the mailbox, the geometry of the vehicle, and the impact speed, the upper part of the fractured post and mailbox may remain together and impact the windshield. If the mailbox separates from the post, the probability of windshield impact increases significantly. In tests of two single-mailbox installations with wood posts, mailbox separation occurred and the windshield was impacted. Attempts to prevent separation by using hanger straps were unsuccessful.

As a result of the tests described in this paper, the following recommendations are made:

1. An effort should be undertaken on the national

level to provide standards and/or performance specifications for mailbox supports. Tests have shown that simple, safe, and economical support systems are attainable. The U.S. Postal Service, with assistance from FHWA, appears to be the logical agency to promulgate standards and specifications for mailbox supports. It seems reasonable to require that mailbox installations meet the same performance specifications now applied to structures such as signs and lightpoles.

2. Where possible, mailbox owners should be encouraged to place their installations on a side road, along a driveway, or a safe distance from the main roadway.

3. Multiple-box installations that include a beam or support member running parallel to the roadway are extremely hazardous and should be avoided. In our opinion, an acceptable alternative to multiple supports would be an individual, crashworthy support for each box. Impact should then cause a "domino effect". Tests are needed, however, to substantiate this hypothesis.

ACKNOWLEDGMENT

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The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the U.S. Department of Transportation. This paper does not constitute a standard, specification, or regulation.

REFERENCES

1. J. F. Fitzpatrick and others. *The Law and Roadside Hazards*. Michie Co., Charlottesville, VA, 1974, pp. 465-469.
2. *Rural Mailboxes*. U.S. Postal Service, USPS-STD-7, Feb. 1974.
3. *A Guide for Erecting Mailboxes on Highways*. AASHTO, Washington, DC, June 24, 1969.
4. H. E. Ross, Jr., and K. C. Walker. *Crash Tests of Rural Mailbox Installations*. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 3254-6, Feb. 1979.
5. H. E. Ross, Jr., and T. J. Dolf. *Crash Test Evaluation of Multiple-Mailbox Installations*. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 0941-2, Oct. 1979.
6. *Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances*. TRB, Transportation Research Circular 191, Feb. 1978.
7. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*. AASHTO, Washington, DC, 1975.

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Crash Tests of Construction-Zone Traffic Barriers

KENNETH C. HAHN AND JAMES E. BRYDEN

Tests conducted by the New York State Department of Transportation to determine the performance of various types of traffic barriers for construction zones are described. A 30.5-cm (12-in) timber curb with steel splice plates between sections and steel pins driven into the subbase was unable to redirect vehicles in minor impacts. A 40.6-cm (16-in) high timber curb with a W-beam steel rail bolted to the face was successfully tested at 76 km/h (47 miles/h) and 17° and at 61 km/h (38 miles/h) and 14°. Steel washers welded atop the anchor pins reduced barrier movement at impact. This barrier is suitable for use where moderate impacts may occur [64 km/h (40 miles/h) and 15°] and requires only a few inches of deflection distance. New York's standard portable concrete median barrier with pin-connected joints, which contained an impacting vehicle at 89 km/h (55 miles/h) and 25° without any connection to the pavement except the two terminal sections, appears to be suitable for use in high-speed work zones. Pulling the joints tight when it was installed and grouting the bottom corners reduced barrier deflection and damage. Deflection of as much as 38.1 cm (15 in) may be produced by 97-km/h (60-mile/h) impacts where anchorage to the pavement is not provided, but it would be less where conditions do not permit such severe impacts.

Timber curbs have been widely used as construction-zone barriers to provide delineation to guide traffic through work areas and redirect errant vehicles that leave the travel lanes. Several design variations have been used; most include timber sections from 25.4 to 30.5 cm (10-12 in) square and about 3.66 m (12 ft) long. In some states, vertical posts with horizontal rail elements are attached to the curb. Anchorage is generally

minimal--often only a metal clip to join adjacent sections but no anchorage to the pavement.

These barriers, which are relatively inexpensive and easy to install, are so narrow as to detract little from the narrow pavement widths frequently encountered in work zones, and they generally provide good delineation. Unfortunately, they provide little redirection to impacting vehicles. A recent Virginia study (1) reported that 73.5 percent of vehicles impacting a 25.4-cm (10-in) square timber curb with timber railing penetrated or straddled the barrier. What is possibly even more serious is that barrier sections were frequently dislodged on impact and became additional hazards to oncoming traffic and workers in the area. Tests conducted by Southwest Research Institute (2) confirmed these problems.

When the hazards associated with timber curbs were recognized, their use on federal-aid projects was restricted by Federal Highway Administration (FHWA) Notice N5160.27. Because this federal action left no simple, inexpensive barrier available for construction-zone use, several research efforts were initiated to develop suitable barriers. These efforts can be grouped in three categories:

1. Modifications of timber curbs,

2. Portable concrete median barriers (CMBs), and
3. Other concepts, particularly W-beam rail on 208-L (55-gal) drums.

Two variations of the timber-curb concept were tested by Southwest Research Institute (2). A type X concrete curb--basically the lower section of a safety-shape CMB--was not successful in redirecting a full-sized passenger car in a 56-km/h (35-mile/h), 8° impact. The second type, two 30.5-cm (12-in) square timber curbs stacked vertically and bolted through a bridge deck, was developed by the Structures Design and Construction Subdivision of the New York State Department of Transportation (NYSDOT) for use as a temporary bridge railing. It performed well on impact, successfully redirecting a 1986-kg (4378-lb) vehicle in an 83-km/h (51.5-mile/h), 13° impact, but it was not practical for highway use because it required rigid connection to the deck.

Good results were also obtained for W-beam rail mounted on 208-L drums. In both tests by Southwest Research Institute (2), this system achieved successful redirection in a 72-km/h (45-mile/h), 13° impact with a full-sized car. However, the 1.04-m (3.4-ft) deflection of the barrier, coupled with tipping of the barrels, might be a problem in narrow work zones.

Considerable effort was also expended on portable CMBs for use in construction zones. Several tests in California (3-5) underscored the difficulty of providing a simple, easily erected portable CMB system that is capable of resisting overturning or penetration on severe impact. One proprietary precast CMB system tested at Southwest Research Institute performed well in a 99-km/h (61.7-mile/h), 24° impact (6). That barrier included a very strong connection between 9.14-m (30-ft) sections and used steel dowels to pin some sections to the pavement.

In a major CMB study by Southwest Research Institute (7-8), the authors have cautioned that "portable concrete median barriers require either large permissible translations during standard strength test impacts, or considerable joint resistance to rotation unless the free-standing barriers are restrained by some foundation." The authors point out, however, that it was difficult to design barriers that would perform adequately because "barrier strength and stability theoretical investigations were complicated by interactions [that] made analysis very difficult."

In brief, few practical construction-zone barrier systems were available when the research reported here was initiated. W-beam steel rail on 208-L drums was available but required about 1.07 m (3.5 ft) of deflection space for 72-km/h (45-mile/h), 15° impacts. Stacked 30.5x30.5-cm (12x12-in) timbers worked well on bridges where anchorage could be developed but would be difficult to use on pavement. One proprietary precast CMB had been tested successfully as a free-standing barrier, although it was pinned to the pavement. Other tests on precast CMBs had not been successful in developing a free-standing, easily transported barrier capable of withstanding 97-km/h (60-mile/h), 25° impacts.

PURPOSE AND SCOPE OF THE STUDY

In 1977, New York initiated research to develop work-zone barriers that would provide adequate traffic protection and yet be easy to install and remove. This effort was directed toward two specific objectives:

1. It was desirable to develop a simple, low-cost barrier for moderate [72-km/h (45-mile/h), 15°] impact conditions that could be used in

lower-speed work zones. It was hoped that this system could be based on the previously used timber-curb barrier.

2. A portable CMB capable of containing vehicles in 97-km/h (60-mile/h), 25° impacts was desired for higher-speed work zones. NYSDOT had developed a standard design that included 6.10-m (20-ft) sections with pin-connected joints. Although the joint detail appeared sufficiently strong to withstand the design impact, testing was needed for confirmation. In addition, some barrier deflection on impact was expected. This could be determined from a full-scale test so that deflection limitations for the barrier could be established. Testing could also identify any deficiencies in the CMB joint and anchorage system.

Testing began in 1977 with six low-speed, low-angle impacts on a 30.5-cm (12-in) timber curb pinned to the pavement and joined with steel splice plates at the joints. That barrier was unsuccessful in redirecting the vehicle, so the design was modified and in 1978 two more tests were conducted. In addition, two tests on free-standing 6.10-m (20-ft) CMB sections were run in 1978. The three barrier systems tested are described here, and the tests results are summarized.

METHODOLOGY AND DESCRIPTION OF BARRIERS

This study consisted of 10 full-scale crash tests to determine the performance of three construction-zone barriers. Testing details were taken from NCHRP Report 153 (9) and its successor, Transportation Research Circular 191 (10).

The first barrier tested was a 30.5-cm (12-in) vertical-face timber curb donated for testing purposes by a local contractor (see Figure 1). The original intention was to test the performance of this curb with an X-shape on the traffic face, but tests at Southwest Research Institute completed in early summer of 1977 confirmed that the X-shape did not redirect impacting vehicles. By the time those results were received, the X-shape had already been cut on the timber. Rather than scrap this material, therefore, it was decided to test the 30.5-cm vertical face with the X-face rotated 180° away from traffic.

Before 1977, NYSDOT standards called for 3.66-m (12-ft) sections of 30.5-cm square timber joined by steel splice plates across each joint. Sections were not pinned to the pavement. Field experience showed, however, that this barrier was dislodged on impact, and individual sections then became hazards to other traffic and workers. Two modifications thus were made to the timber curb for the 1977 test series. A 0.91-m (3-ft) long steel channel, bent from 9.5-mm (0.375-in) steel plate, was added at each joint. Adjoining timber ends were set inside the channel, which was 14 cm (5.5 in) deep. Slotted holes were provided in the channel to allow steel pins to be driven through each timber. Steel pins 2.5 cm (1 in) in diameter and 61 cm (24 in) in length were driven 30.5 cm into the ground on each side of the joint and at a maximum 1.83-m (6.0-ft) spacing along each curb section, which varied with timber length. The splice plates and pins were intended to anchor the curb section firmly in place during impact.

The total length of timber curb installed for the series of six tests was 44.81 m (147 ft) and consisted of 11 sections that varied from 3.05 to 4.57 m (10-15 ft) each. For the first three tests, the curb was installed at an angle of 3° to the vehicle path. The barrier was then removed and reset at 7.5° for the next three tests. Damaged

Figure 1. Details of 30.5x30.5-cm timber curb.

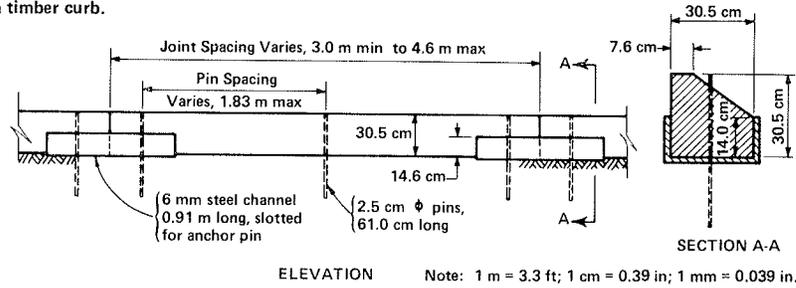
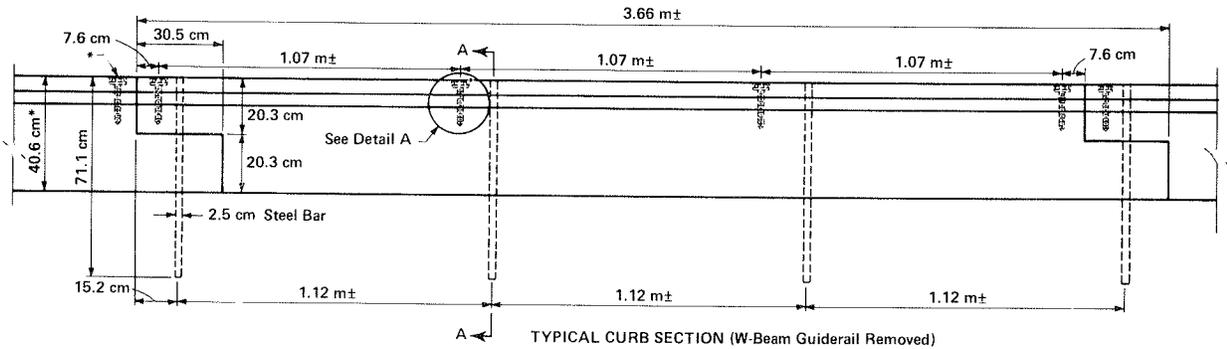
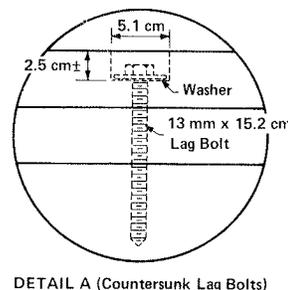
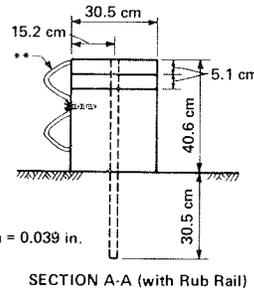


Figure 2. Details of W-beam timber curb.



* For testing purposes, this 30.5 x 40.6 cm curb is made of 30.5 x 30.5 cm timbers, with two 5.1 x 30.5 cm planks fastened to each with four 13 mm x 15.2 cm lag bolts with washers.

**W-beam guiderail section, 10 or 12 gage, 31.1 cm high, mounted 13 mm down from top of curb, bolted to curb with 9.5 mm x 8.4 cm lag bolts.



curb sections were replaced between individual tests. All six tests were run on a compacted gravel surface.

Because the 30.5-cm (12-in) vertical-face timber curb did not redirect vehicles during three of the first six tests, a second barrier was designed for testing in 1978. Early California research on curbs (11) pointed out the importance of providing an undercut and a smooth curb face to reduce tire climb, neither of which was provided by the 30.5-cm timber curb. In addition, the researchers believed additional curb height would reduce the possibility of vaulting by placing the top of the curb above the midheight of passenger-car tires (midheight is approximately the normal contact point of the tire on the curb).

Thus, two modifications were made to the timber curb: Height was increased to 40.6 cm (16 in), well above the midheight of passenger-car tires, and a standard W-beam steel rail element was bolted to the traffic face of the curb to provide a smooth curb face and an undercut to prevent tire climb. To allow simpler field installation and to eliminate tire damage on the steel joint-splice plate, joint details were also changed. Details of the barrier are shown in Figure 2.

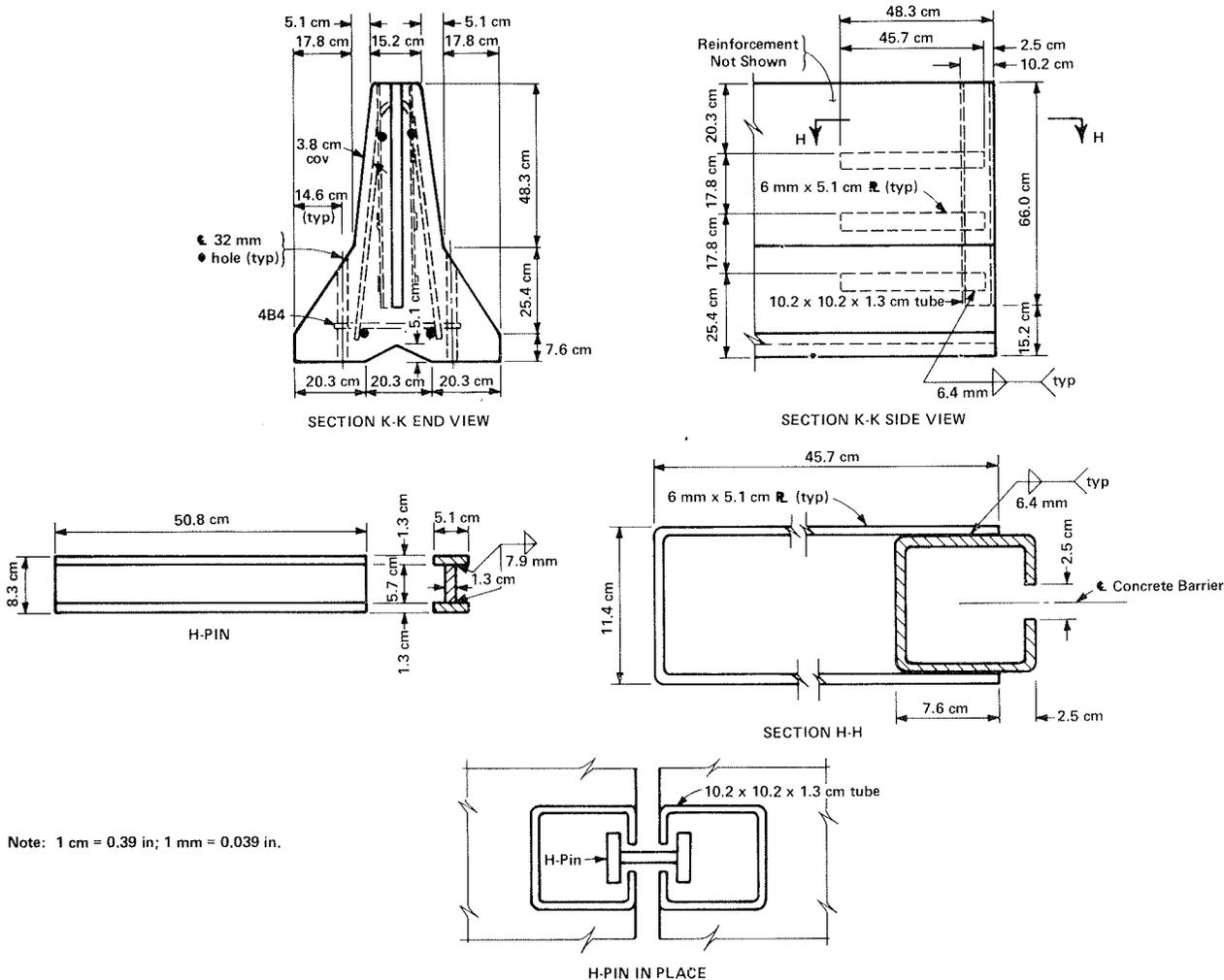
The timber curb was constructed of 30.5x30.5-cm (12x12-in) timbers with two 5.1x30.5-cm (2x12-in) planks spiked on the top. This would permit configuration changes if they were determined to be

necessary after the initial tests. Overlapping joints 30.5 cm (12 in) long and 20.3 cm (8 in) deep were cut with a chain saw, and steel anchor pins 2.5 cm (1 in) in diameter and 71.1 cm (28 in) in length were driven flush with the top of the timber. The pins were placed at the center of each overlapping joint and at the one-third points of each timber section. For the 3.66-m (12-ft) timbers used, the distance between joints was 3.35 m (11 ft), which resulted in a pin spacing of 1.12 m (3 ft 8 in). The W-beam was spliced in the usual manner with eight 16-mm (0.625-in) bolts and connected to the timber curb with 9.5x64-mm (0.375x2.5-in) lag bolts at the one-third points between the rail splices. Splices on the W-beam did not coincide with the curb joints. Ten sections of curb with a total length of 34.14 m (112 ft) were installed on the compacted gravel test pad. This system is referred to here as the W-beam timber curb.

Details of the portable CMB are shown in Figure 3. Basically, this barrier consisted of 6.10-m (20-ft) precast sections of New Jersey-shape concrete barrier. The longitudinal reinforcing consisted of four No. 6 bars along with stirrups and other steel near the joints. Joint detail consisted of 10.2x10.2x6.4-mm (4x4x0.25-in) slotted steel tubes cast into each end of each barrier section. A steel H-pin made from 12.7-mm (0.5-in) thick plate connected adjoining sections.

For these two tests, eight sections of barrier

Figure 3. Details of portable CMB.



Note: 1 cm = 0.39 in; 1 mm = 0.039 in.

were set on the asphalt pavement. In the first test, sections 1 and 7 were pinned to the pavement; in the second test, only section 1 was pinned. The sections used in the test differed from those shown in Figure 3 in that the anchor pin holes were placed in the center of the barrier rather than on the sides. Steel pins 2.5 cm (1 in) in diameter and 1.42 m (4 ft 8 in) in length were driven 0.61 m (2 ft) into the asphalt pavement and gravel subbase. Nine pins were used for each anchored section.

In the first test, the adjoining sections were simply set on the pavement and the H-pin was slipped into place as the second section was lowered. No special effort was made to pull the sections tight. In the second test, two specific steps were taken to reduce barrier deflection: First, each barrier section was pulled as tight as possible against the adjacent section as it was set in place, and then stiff concrete mortar was packed into the bottom 15.2 cm (6 in) of each joint on both sides of the barrier to provide continuous bearing across the joint.

TEST RESULTS

30.5x30.5-cm Timber Curb

The results of six full-scale crash tests of 30.5x30.5-cm (12x12-in) timber curb performed in

1977 are summarized in Table 1. These were the first tests performed under this research project, and many equipment problems were encountered. Malfunction of the oscillograph recorder caused a loss of data on acceleration and event duration for these tests. However, because of the failure of this curb design to redirect the vehicle, the loss of these data is not critical. For the first three tests, the impact angle was 3°, speeds were 16, 32, and 48 km/h (10, 20, and 30 miles/h), consecutively, and the instrumented vehicle--a 1969 Ford Fairlane 500 sedan--weighed 1515 kg (3340 lb). The first test (test 1A) resulted in vehicle redirection parallel to the curb and only minor vehicle damage. The right front (impacting) tire was blown out by a protruding splice plate corner and the front bumper, right front fender, door, and rocker panel sustained some scratches and shallow dents. The vehicle was in contact with the barrier for 16.46 m (54 ft). Although it did not mount the curb, tire marks on the curb face indicated that the vehicle started to climb. The curb was undamaged and deflected only 5.1 cm (2 in).

The damage from the first test was painted over, and the same vehicle was used for the 32-km/h (20-mile/h) test (test 1B). Again, the vehicle was redirected parallel to the curb, and there was minor sheet-metal damage and a punctured right front tire. Tire marks on the curb indicated wheel climb

Table 1. Test results for 30.5x30.5-cm timber curb.

Item	Test					
	1A	1B	1C	2A	2B	2C
Impact condition						
Speed (km/h)	16	32	48	16	32	48
Angle (°)	-3	-3	-3	-7.5	-7.5	-7.5
Vehicle weight (kg)	1515	1515	1515	1533	1533	1533
Curb length (m)	44.81	44.81	44.81	44.81	44.81	44.81
Contact distance (m)	16.46	12.19	15.24	2.59	6.86	14.02
Curb deflection (cm)						
Dynamic	10.2	5.1	12.7	2.5	7.6	10.2
Permanent	5.1	2.5	5.1	0	2.5	5.1
Exit angle (°)	0	0	-12.6 ^a	+5.0	-29.8 ^a	-21.0 ^b

Note: 1 km = 0.62 mile; 1 kg = 2.2 lb; 1 m = 3.3 ft; 1 cm = 0.39 in.

^aNo exit; vehicle stopped on curb.

^bVehicle stopped behind curb.

to the top of the curb face at three locations during 12.19 m (40 ft) of contact, but it did not mount or vault the curb. There was no curb damage and only 2.5 cm (1 in) of deflection.

The same vehicle was used for the 48-km/h (30-mile/h) test (test 1C). At impact, the right front wheel climbed and mounted the curb. The vehicle slid along the curb top, dragging the right rear tire along the curb face. The anchor pins, which were left protruding 2.5 to 5.1 cm (1 to 2 in) above the curb to facilitate removal, severely dented the right front wheel and damaged the suspension. The vehicle came to rest straddling the curb at a 12.6° angle; only the right front wheel was over the curb after 15.24 m (50 ft) of contact.

For the next three tests, the impact angle was increased to 7.5°, and successive speeds again were 16, 32, and 48 km/h (10, 20, and 30 miles/h). The instrumented vehicle, a 1970 Ford Fairlane sedan, weighed 1533 kg (3380 lb). In the first 7.5° test, the vehicle was smoothly redirected and exited the curb at a 5° angle after only 2.59 m (8.5 ft) of contact. There was no measurable barrier deflection or damage, and vehicle damage was limited to minor sheet-metal scratches on the stone shield and a flattened right front tire caused, as before, by a protruding splice.

In the second 7.5° test, the right front wheel climbed and vaulted the curb on impact. The vehicle ran 6.86 m (22.5 ft) along the curb top and came to rest on the curb, in much the same attitude as in test 1C, at an angle of 29.8°. The vehicle gouged the curb top badly in two locations, and the splintered wood wedged between the tire and wheel, damaging both. However, the wood was somewhat deteriorated before these tests. Vehicle damage included badly bent tie rods, minor sheet-metal dents, and scratches on the bumper, stone shield, lower right front fender, and rocker panel. Curb damage was limited to the splintering and gouging just mentioned. Permanent deflection was only 2.5 cm (1 in).

In the final test of 30.5x30.5-cm timber curb, the vehicle's right front wheel climbed and vaulted the curb on impact. As it ran along the curb top for 14.02 m (46 ft), the rest of the vehicle also climbed over the curb, finally coming to rest perpendicular to the curb and 6.10 m (20 ft) behind it. Curb damage included some minor gouging along the top front edge and a 1.52-m (5-ft) long, 12.7-cm (5-in) wide splinter that remained in front of the curb, extending about 1.22 m (4 ft) into the traveled way. Vehicle damage was limited to the undercarriage--large splinters in the suspension, slightly bent tie rods, a badly dented gas tank, and a torn and bent left rear body panel from below the

rear bumper--and minor sheet-metal damage to the right front bumper, stone shield, and fender.

W-Beam Timber Curb

The results of two full-scale 1978 tests of W-beam timber curbs are summarized in Table 2. In the first test (test 11A), a 1746-kg (3850-lb) 1973 American Motors Corporation Matador sedan impacted the curb at 76 km/h (47.3 miles/h) and 17°. Impact occurred on the stone shield below the front bumper and on the right front wheel. The vehicle climbed about 25.4 cm (10 in) up the W-beam, deflecting the barrier back 27.9 cm (11 in). Both front wheels were airborne--about 15.2 cm (6 in) on the left and 25.4 cm on the right--for about 3.05 m (10 ft) of travel on the barrier and 3.05 m after leaving the barrier. Redirection parallel to the barrier occurred after only 6.10 m (20 ft) of contact. The vehicle then yawed 5° left as it exited the curb, but the flattened right front tire, after recontacting the ground, caused the car to swerve to the right, finally coming to rest some 76.2 m (250 ft) from impact.

Vehicle damage was minor and included a flat tire, a bent stone shield, and minor scratches and dents to the right side. Barrier damage was also minor. The impacting vehicle deflected the curb 27.9 cm (11 in) at the timber joint 1.22 m (4 ft) downstream from the impact point and tilted it back some 30°. No timbers were damaged, although several sections rode up the steel anchor pins, spreading but not separating several joints. Curb-section deflections at each joint are given below (1 cm = 0.39 in; impact between joints 3 and 4):

Joint	Deflection from Original Position (cm)	
	Test 11A	Test 11B
1	2.5	0
2	10.2	2.5
3	26.7	7.6
4	17.8	5.1
5	8.9	3.8
6	3.8	2.5
7	0	0
8	0	0
9	0	0

One section of W-beam was dented slightly and was replaced before the next test.

For the second test (test 11B), 38-mm (1.5-in) diameter steel washers were welded to the top of each anchor pin to reduce the tendency of the timbers to slide up the pins. Except for this difference, the barrier was unchanged. A 1751-kg (3860-lb) Matador sedan impacted the curb at 61 km/h (38.0 miles/h) and 14°.

Impact again occurred on the stone shield and right front tire. The front of the vehicle rose slightly--7.6-10.2 cm (3-4 in) on the right and 10.2-15.2 cm (4-6 in) on the left--after deflecting the barrier back 7.6 cm. Redirection to 6° away from the curb occurred within 4.88 m (16 ft) of contact. The vehicle ran some 73.2 m (240 ft) from impact to rest, sustaining only minor damage including a bent stone shield and some scratches to the right side doors and lower rear fender.

Maximum barrier deflection occurred at the joint 1.22 m (4 ft) downstream from impact, where it was tilted back 5°. None of the timbers or W-beams was damaged and the retaining washers appeared to help significantly to reduce barrier movement and in turn reduced rise on the pins and vehicle jump during impact.

Portable Concrete Median Barrier

The results of the two 1978 full-scale crash tests of portable CMBs are summarized in Table 3. Both test installations consisted of eight 6.1-m (20-ft) long sections of precast barrier set on an asphalt pavement. For the first test, the first and seventh sections of barrier were anchored with nine 2.5-cm (1-in) diameter steel pins, 1.42 m (4 ft 8 in) long. The pins were inserted through precast holes in the barrier top and driven about 0.61 m (2 ft) into the pavement and gravel subbase until flush with the top of the barrier.

In the first CMB test (test 17), a 1928-kg (4250-lb) instrumented vehicle, a 1975 Plymouth sedan, impacted the barrier at 85 km/h (52.8 miles/h) and 25°. On impact it climbed to the top of the barrier and within 4.57 m (15 ft) was redirected parallel to the barrier. During initial redirection, while the right-side tires were up on the barrier, the left side was airborne. When the vehicle left the barrier, the front end pitched forward, dragging on the pavement, while the rear was still airborne for some time. Impact occurred at the center of the fourth section, and the vehicle remained in contact with the barrier for 10.36 m (34 ft), leaving it just beyond the joint between the fifth and sixth sections. Because its right front suspension was severely damaged and that tire flattened, the vehicle turned back into the barrier, again striking the bottom 7.6-cm (3-in) high vertical barrier face about 1.83 m (6 ft) into the seventh section. The bumper rode about halfway up the sloped barrier face, and the vehicle's right side was in contact with the barrier for the entire length of the eighth section. The vehicle, which came to rest about 12.19 m (40 ft) beyond and perpendicular to the barrier, sustained heavy damage

to the front-end sheet metal, right front suspension, and along the entire right side.

Barrier damage was minor, and the maximum deflection was 40.6 cm (16 in). The third, fourth, and fifth sections were displaced laterally (Table 4), but examination of the test films revealed that there was no barrier rotation. The displacement was the result of the barrier simply sliding to the right on impact. Section 4, where impact occurred, sustained three vertical hairline cracks on the backside. Corners on the barrier base were spalled slightly at joints 2, 3, and 6 on the front, moderately at joint 4 on the front, and extensively [more than 30.5 cm (1 ft) long] on the back at joints 2 and 5.

For the second test (test 18), only the first section was pinned to approximate an upstream terminus. During installation, care was taken to ensure that each joint was pulled tight against the connecting H-pins. To further stiffen the barrier, each joint was packed with a stiff portland cement mortar to a height of 15.2 cm (6 in) and about 15.2 cm into the joint.

The 1919-kg (4230-lb) instrumented vehicle, a 1973 Matador station wagon, impacted the CMB at 88 km/h (54.8 miles/h) and 25°. The right-side tires quickly climbed to the barrier top and, on redirection, both tires were well above the top of the barrier. At 7.92 m (26 ft) after impact, the vehicle had rolled 36° to the left and was airborne--the left side some 20.3 cm (8 in) off the ground and the right side about 1.52 m (5 ft) off the ground. The vehicle's rear yawed right so that the right rear wheel was about 0.91 m (3 ft) above and 0.61 m (2 ft) behind the barrier. The vehicle recontacted the barrier 19.20 m (63 ft) downstream of impact with the right rear wheel atop the CMB, the right front and left rear wheels on the CMB

Table 2. Test results for W-beam timber curb.

Item	Test		Item	Test	
	11A	11B		11A	11B
Impact condition			Acceleration (g)		
Speed (km/h)	76	61	50-ms average		
Angle (°)	-17	-14	Longitudinal	1.81	1.81
Vehicle weight (kg)	1746	1751	Lateral	3.35	1.95
Curb length (m)	34.14	34.14	Maximum peak		
Contact			Longitudinal	-	9.97
Distance (m)	6.10	4.88	Lateral	14.76	14.16
Time (ms)	304	288	Average continuous		
Curb deflection (cm)			Longitudinal	1.19 ^a	0.36
Dynamic	33.0	15.2	Lateral	2.70	1.33
Permanent	27.9	7.6	Exit angle (°)	+5	+6

Note: 1 km = 0.62 mile; 1 kg = 2.2 lb; 1 m = 3.3 ft; 1 cm = 0.39 in.

^aFor 75 ms only; recorder failed at this point.

Table 3. Test results for portable CMB.

Item	Test		Item	Test	
	17	18		17	18
Impact condition			Acceleration (g)		
Speed (km/h)	85	88	50-ms average		
Angle (°)	-25	-25	Longitudinal	-	4.89
Vehicle weight (kg)	1928	1919	Lateral	5.99	11.52
Barrier length (m)	48.77	48.77	Maximum peak		
Contact			Longitudinal	-	15.14
Distance (m)	10.36	21.64 ^a	Lateral	14.30	26.35
Time (ms)	528	968 ^b	Average continuous		
Deflection (cm)			Longitudinal	-	0.69 ^c
Dynamic	40.6	27.9	Lateral	1.37	1.03 ^c
Permanent	40.6	27.9	Exit angle (°)	+5	+15

Note: 1 km = 0.62 mile; 1 kg = 2.2 lb; 1 m = 3.3 ft; 1 cm = 0.39 in.

^aIncludes 10.97-ms airborne travel.

^bIncludes 510-ms airborne travel.

^cFor 272 ms only; vehicle airborne beyond this point.

Table 4. Lateral movement in CMB.

Joint	Base Movement (cm)				Top Movement, Test 18 (cm)	
	Test 17		Test 18		Downstream	Upstream
	Downstream	Upstream	Downstream	Upstream		
1	0	0	0	0	0	0
2	0	1.3	0	0	1.3	0
3	12.7	10.2	21.6	21.6	27.9	25.4
4	40.6	40.6	25.4	25.4	24.1	27.9
5	1.3	3.8	6.4	6.4	6.4	6.4
6	0	0	0	0	0	0
7	0	0	0	0	0	0

Notes: 1 cm = 0.39 in.

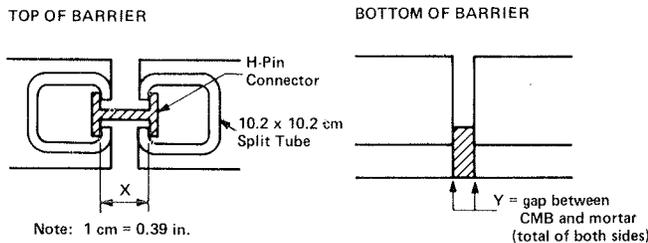
Impact between joints 3 and 4. Movement is displacement from original position, in this case away from traffic. All measurements to base made from reference marks of original position on pavement. All measurements to top made from reference markers erected behind barrier. Discrepancies between top and base movements (test 18) caused by pavement irregularities and barrier moving atop spalled concrete.

Table 5. Joint movement in CMB (test 18).

Joint	X (cm)			Y (cm)			Location
	Before	After	ΔX (cm)	Before	After		
1	5.7	5.7	0	0	0	--	
2	5.7	5.7	0	0	1.0	Front	
3	4.6	5.4	+0.8	0	1.0	Rear	
4	5.2	5.6	+0.4	0	1.3	Rear	
5	5.6	5.7	+0.2	0	0.6	Front	
6	5.4	5.6	+0.2	0	0.3	Front	
7	5.4	5.6	+0.2	0	0	--	

Notes: 1 cm = 0.39 in.
Impact between joints 3 and 4.

Figure 4. Joint movement in CMB.



face, and the left front wheel and bumper dug into the pavement. When it returned to the pavement 21.64 m (71 ft) after impact, the rear of the vehicle yawed sharply right and it rolled over, coming to rest on its wheels about 18.29 m (60 ft) beyond and perpendicular to the barrier.

The vehicle suffered extensive damage during both impact and rollover. Before rollover, it sustained heavy damage to all front-end and right-side sheet metal and to all right-side wheels and suspension parts. In addition, the frame was bent and the windshield broken. The rollover popped out the windshield, dented the roof, and crushed the engine compartment.

The barrier moved laterally a maximum of 27.9 cm (11 in) at the downstream end of the impacted (fourth) section, and there was less movement of the second, third, and fifth sections. Again, no barrier tipping or additional dynamic deflections were detected in the test films. Lateral movements at each joint, measured at both the top and bottom of the barrier, are summarized in Table 4. Widths of joint gaps, measured before and after impact to detect longitudinal movement, are given in Table 5 and shown in Figure 4. The unpinned seventh and eighth sections did not move.

The only significant barrier damage was confined, as in the previous test, to some base corner spalling and some cracks in the impacted section ranging from hairline fractures to cracks 3.2 mm (0.125 in) wide. Joint spalling in this test was noticeably less than in the first because the mortar helped to transfer impact forces across the joints more uniformly.

DISCUSSION AND FINDINGS

Results of the first six tests clearly indicate that the 30.5-cm (12-in) vertical-face timber curb is unable to redirect vehicles even in moderate impacts, although the anchorage system was able to prevent barrier misalignment. In addition, damage to the gasoline tank presents a severe fire hazard. At 3° and 48 km/h (30 miles/h), the vehicle completely vaulted the curb. Use of this curb as a positive barrier, even at very flat angles, thus cannot be justified at speeds above 16 km/h (10 miles/h). In addition, the risk of damage to vehicle sheet metal and suspension damage is great, even for moderate impacts. Gouging and splintering of the curb in several tests emphasize the importance of using only wood that is structurally sound. Tire damage caused by protruding splices indicates the importance of installing these plates flush with the curb face.

The W-beam timber curb, intended to prevent vehicle climb by increasing the curb height to 40.6 cm (16 in) and adding a W-beam to trap the impacting tire, was designed with overlapping joints to eliminate splice plates. The results of two crash tests at 15° indicates that the new design is a significant improvement over 30.5x30.5-cm (12x12-in) curbs. Vehicle climb was greatly reduced by addition of the W-beam rail. Redirection was achieved, and the exit trajectories of 5° and 6° were acceptably shallow. Vehicle decelerations (peak 50 ms) were below 2 g longitudinally in both tests and below 3.4 and 2.0 g laterally in tests 11A and 11B, respectively. Curb damage was minor: several bent anchor pins and one dented rail section. The overlapped, pinned joints proved adequate in test 11A; when a retainer washer was added on each pin in test 11B, deflections were reduced from 27.9 to 7.6 cm (11-3 in), vehicle jump from 25.4 to 10.2 cm (10-4 in), and curb tilt from 30° to 5°.

This design was intended to provide a simple, economical barrier for use in construction zones. It is made up from readily available components and is relatively easy to install and remove. Since the completion of these tests, it has been slightly modified. Timber lengths have been increased to 4.11 m (13 ft 6 in) so that the W-beam and timber joints coincide to allow for removal of one or

several sections to provide access through the curb line. This modified design has been standardized by NYSDOT and is now available for use as a physical barrier on contracts where the anticipated operating speed is 64 km/h (40 miles/h) or less.

Where anticipated speeds are higher than 64 km/h or where more positive protection is required, a stronger barrier is needed. For this reason, a portable CMB was tested to determine its resistance to severe impacts. These two 25° tests were intended as tests of penetration resistance, and lack of satisfactory redirection is not indicative of unsatisfactory performance. It has been established by many testing agencies that CMBs can successfully redirect vehicles impacting at angles up to 15°. At steeper angles, proper redirection is not assured. Although the highest speed attained in the CMB tests was 89 km/h (55 miles/h), it appears that this barrier had sufficient reserve strength to withstand 97-km/h (60-mile/h) impacts, especially with the joints pulled tight and grouted. The low barrier deflection of 27.9 cm (11 in), the lack of barrier roll, and the movement of only four barrier sections on impact, combined with the very light damage to the joint system, all support this conclusion.

The vehicle rollover observed in test 18 is not surprising, because similar reactions have been reported in other such tests. However, both tests confirm that the current NYSDOT design for portable CMBs is sufficiently strong to resist these severe impacts. Results indicate that anchoring the CMB in midstream is unnecessary, because only a very few sections are disturbed even during severe impacts. Joint grouting reduced both joint deflection and corner spalling by stiffening the H-pin joints. Based on the 27.9-cm (11-in) deflection observed in the final test [1919-kg (4230-lb) vehicle at 89 km/h (55 miles/h) and 25°], a maximum barrier deflection of about 38.1 cm (15 in) can be expected for a 2041-kg (4500-lb) vehicle at 97 km/h (60 miles/h) and 25° when the joints are pulled tight and grouted. For ungrouted joints, the maximum deflection would be almost 0.61 m (2 ft). However, where conditions are not likely to result in impact speeds or angles this severe, smaller deflections would result for both the grouted and ungrouted designs. Vehicle decelerations (peak 50 ms) were below 5 g longitudinally for test 18 (test 17 data were not available) and below 12 and 6 g laterally in tests 17 and 18, respectively.

CONCLUSIONS

Based on these tests, the following conclusions appear to be warranted:

1. The 30.5x30.5-cm (12x12-in) timber curb did not redirect vehicles in moderate impacts and cannot be used as a positive barrier. However, the anchorage system tested did prevent barrier movement on impact.
2. The W-beam timber-curb barrier provides satisfactory redirection for impacts at speeds up to 64 km/h (40 miles/h) and 15°.
3. Deflection, joint separation, and curb roll were reduced by adding retaining washers to the top of the steel pins used to anchor the W-beam timber curb.
4. The portable CMB, incorporating NYSDOT's pin-connected joints, is an effective positive barrier for impacts at speeds up to 97 km/h (60 miles/h) and 25°, although satisfactory vehicle redirection cannot be ensured above 15° impact angles.
5. Barrier rotation during impact was

effectively eliminated by the H-pin connectors.

6. CMB deflection and corner spalling were reduced by pulling the joints tight and grouting the lower 15.2 cm (6 in) of each joint, front and rear.

7. The portable CMB that uses the NYSDOT design with grouted joints will deflect only 38.1 cm (15 in) for 97-km/h (60-mile/h), 25° impacts. For less severe impact conditions, this deflection will be less.

8. Except for the terminal sections, this CMB design does not require pinning to the pavement or a wedge of asphalt pavement behind the barrier to provide resistance to severe impacts.

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REFERENCES

1. F.N. Lisle. Evaluation of Timber Barricades and Precast Concrete Traffic Barriers for Use in Highway Construction Areas. TRB, Transportation Research Record 693, 1978, pp. 18-24.
2. M. E. Bronstad and C. E. Kimball, Jr. Temporary Barriers Used in Construction Zones. Southwest Research Institute, San Antonio, Rept. 03-3716-3, Dec. 1977.
3. D. M. Parks and others. Vehicular Crash Tests of Unanchored Safety-Shaped Precast Concrete Median Barriers with Pinned End Connections. Transportation Laboratory, California Department of Transportation, Sacramento, Rept. CA-DOT-TL-6624-1-76-52, Aug. 1976.
4. R. L. Stroughton and others. Vehicular Impact Tests of Precast Concrete Median Barriers with Corrugated Ends and Tensioned Cables. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-AD-TL-78-13, June 1978.
5. D. M. Parks and others. Vehicular Crash Test of a Continuous Concrete Median Barrier Without a Footing. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-CA-TL-6883-77-22, Aug. 1977.
6. M. E. Bronstad and C. E. Kimball, Jr. Crash Test Evaluation of a Precast Interlocked Median Barrier. Southwest Research Institute, San Antonio, Aug. 1974.
7. M. E. Bronstad, L. R. Calcote, and C. E. Kimball, Jr. Concrete Median Barrier Research: Volume 1--Executive Summary. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-RD-77-3, June 1976.
8. M. E. Bronstad, L. R. Calcote, and C. E. Kim-

- ball, Jr. Concrete Median Barrier Research: Volume 2--Research Report. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-RD-77-4, March 1976.
9. M. E. Bronstad and J. D. Michie. Recommended Procedures for Vehicular Crash Testing of Highway Appurtenances. NCHRP, Rept. 153, 1974.
 10. Recommended Procedures for Vehicular Crash Testing of Highway Appurtenances. TRB, Transportation Research Circular 191, Feb. 1978.
 11. R. N. Field and J. L. Beaton. Final Report of

Full-Scale Dynamic Tests of Bridge Curbs and Rails. Materials and Research Department, California Department of Public Works, Sacramento, Aug. 30, 1957.

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Portable Concrete Median Barriers: Structural Design and Dynamic Performance

DON L. IVEY, HAYES E. ROSS, TEDDY J. HIRSCH, C. EUGENE BUTH, AND ROBERT M. OLSON

Types of portable concrete median barriers (CMBs) in use in the United States are described primarily in terms of structural details and the load-bearing characteristics of their end connections. Twelve end-connection designs are analyzed to produce estimates of their resistance to loads in four test conditions: simple tension, shear, yaw moment, and torsion. Rotational connection slack is estimated from the geometric properties of the different end connections. Nine crash tests conducted by four research agencies are examined. These tests cover a range of barrier lengths from 3.81 to 9.14 m (12.5-30 ft) and a range of connection details that vary from low to significant load capacity. The crash tests vary in intensity from a 7° test at 104.6 km/h (65 miles/h) to a 25° test at 99.8 km/h (62 miles/h). Analysis of these tests yields specific conclusions on the performance of CMBs. An energy analysis of portable CMBs during vehicle impacts is presented. Estimates of barrier deflection derived from this analysis check closely with deflections observed during crash tests. A parametric study of the influence of various barrier characteristics, including barrier length and mass, connection slack and strength, and support media static and sliding friction, is also presented. Based on this analysis, portable CMBs can now be designed to provide specific performance characteristics.

The most widely used class of construction-zone barriers with positive redirection characteristics is the portable concrete median barrier (CMB). There are at least as many variations in CMB design as there are states in which it is used. The CMB usually has no mechanical fastening to the ground but relies on mass and sliding friction for translational stability. It is always segmented, and segment lengths vary from 3.05 to 9.14 m (10-30 ft). Segment lengths of 3.05, 3.66, 3.81, 6.10, and 9.14 m (10, 12, 12.5, 15, 20, and 30 ft) have been used. It is in the method of joining these segments that there is the greatest variation.

Figures 1 and 2 show applications of the portable CMB in Virginia and Texas, respectively. In Virginia, the barrier segments are 3.66 m long with a vertical concrete shear key connection design. In Texas, the segments are 9.14 m long, and three no. 8 reinforcing bar dowels form the connection. These two connections and four other representative connections are shown in Figures 3-8. Although only 6 connection designs are shown, 12 have been identified. Of these 12, 5 are variations of the California vertical pin connection shown in Figure 7.

The evolution of the portable CMB was straightforward. The CMB was first produced by forming the barrier in place for a permanent installation. Then, in an effort to reduce costs, precast fabrication was used. This made it

necessary to move barrier segments from place to place. It became obvious that barrier segments could be placed temporarily in construction zones before final placement as permanent barriers. Once this was recognized, the use of portable CMBs became widespread. In early applications, the functional characteristics of the barrier were assumed to be adequate, an assumption that has generally been borne out by field experience.

TEST RESULTS

As the use of the CMB became more widespread, it was subjected to testing by at least four organizations: Southwest Research Institute (SwRI) (1), the Texas Transportation Institute (TTI) (2), the New York Department of Transportation (NYSDOT), and the California Department of Transportation (Caltrans) (3).

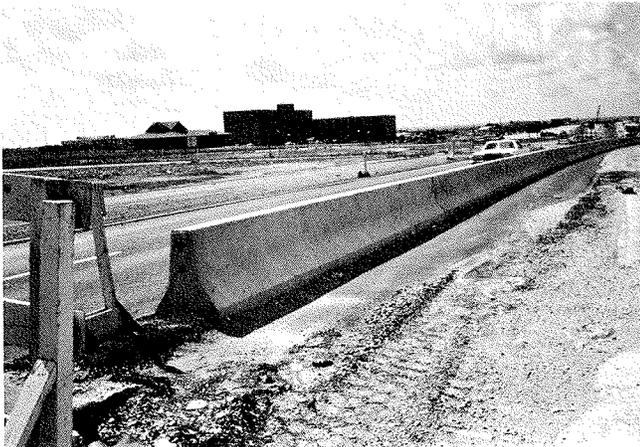
Currently, a total of nine tests have been conducted on barriers that may be considered portable. The results of these tests are summarized in Table 1. In six of the tests, the results were successful at least from the consideration of structural capacity [it must be noted that the test of a 2041-kg (4500-lb) vehicle at 96.5 km/h (60 miles/h) and 25° is a test of structural adequacy, not a test of vehicle reaction from a safety standpoint]. At least three designs have been shown to perform adequately in regard to structural integrity under vehicle collisions at the 96.5-km/h, 25°, 2041-kg energy level. This performance is illustrated in SwRI test CMB-24, TTI test CMB-2, and New York test NY-1. Barrier-segment lengths were 6.10 m (20 ft) for the New York and SwRI tests and 9.14 m (30 ft) for the TTI test. Structural failures occurred in SwRI test CMB-18, where the longitudinal reinforcement of the main section was insufficient to prevent a large portion of one segment from being dislodged; Caltrans test 292, where a main barrier segment was broken; and Caltrans test 293, where one barrier segment was knocked over. As a result of these latter two tests, Caltrans upgraded the design to the one shown in Figure 5 and described in Table 2.

Although many of these tests showed adequate barrier performance, there are a variety of untested designs in use. Some of these are of significantly

Figure 1. Portable CMB in Virginia.



Figure 2. Portable CMB in Texas.



lower strength than the designs tested. In an effort to gain some perspective on the different barrier-connection designs that are in use, 12 designs were subjected to a simple structural analysis. This analysis consisted of determining the ultimate strength of the barrier connection under four loading conditions.

Figure 9 shows a coordinate system used to define these loads. In this system, the x coordinate is coincident with the longitudinal barrier centroidal axis. The y coordinate is vertical and the z coordinate is perpendicular to the longitudinal barrier axis and parallel to the ground plane. The origin of coordinates is at the cross-section center of gravity at a barrier segment joint. The four loading conditions analyzed are the ultimate tensile strength (P), a tensile force in the x direction; the ultimate shear strength (V), the shear in the z direction; the ultimate moment (M), the moment about the y axis; and the ultimate resistance to torsion (T), a torque about the x axis. Each of these ultimate strengths is the strength of the connection, not the strength of the barrier cross section.

Table 2 gives the results of these computations. The various designs are listed sequentially, from the strongest connections to the weakest. The final column in the table denotes the other connection property--i.e., how much one segment of the barrier may rotate with respect to an adjacent segment before significant moment begins to develop about the y axis. In general, the lower the value of the connection slack, the better the barrier performance should be. Stated another way, the lower this value, the lower should be the barrier deflection under a specific impact condition. It seems apparent from Table 2 that a variety of designs with different performance characteristics are represented.

The following performance levels of barriers are considered, based on the amount of kinetic energy associated with the lateral component of vehicle velocity (1 mile = 1.6 km; 1 lb = 0.45 kg):

Figure 3. Virginia vertical tongue-groove connection.

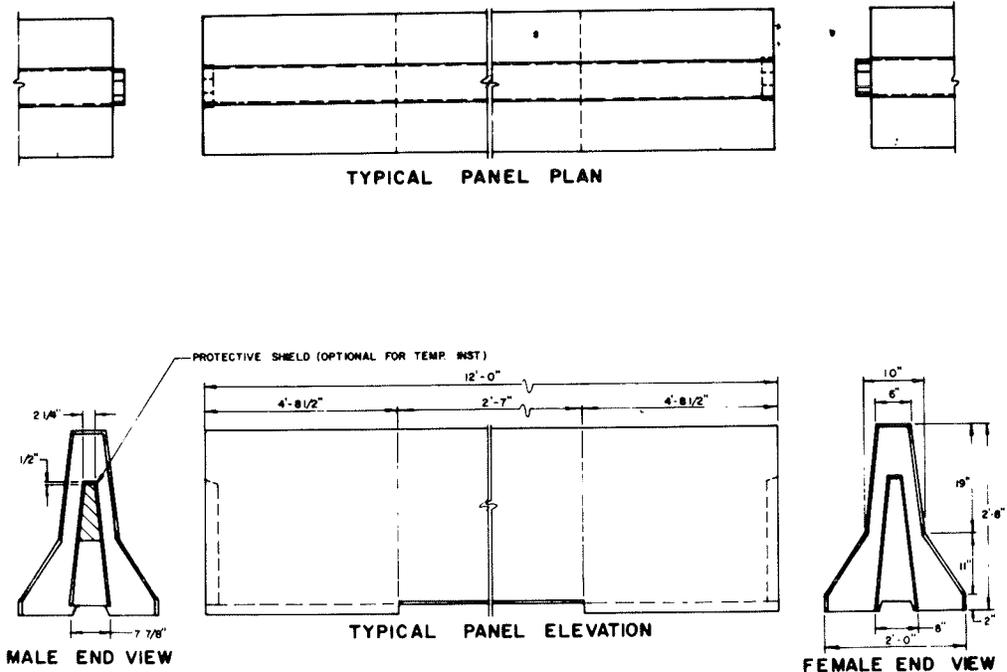


Figure 4. Colorado top hook and rebar.

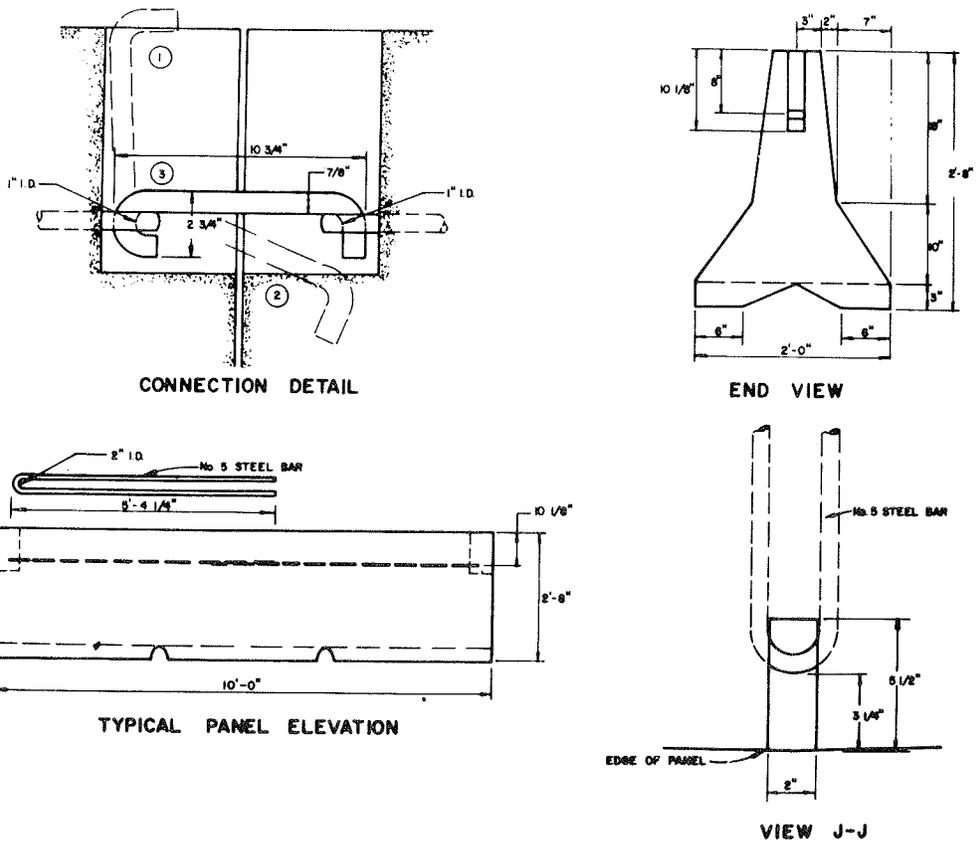
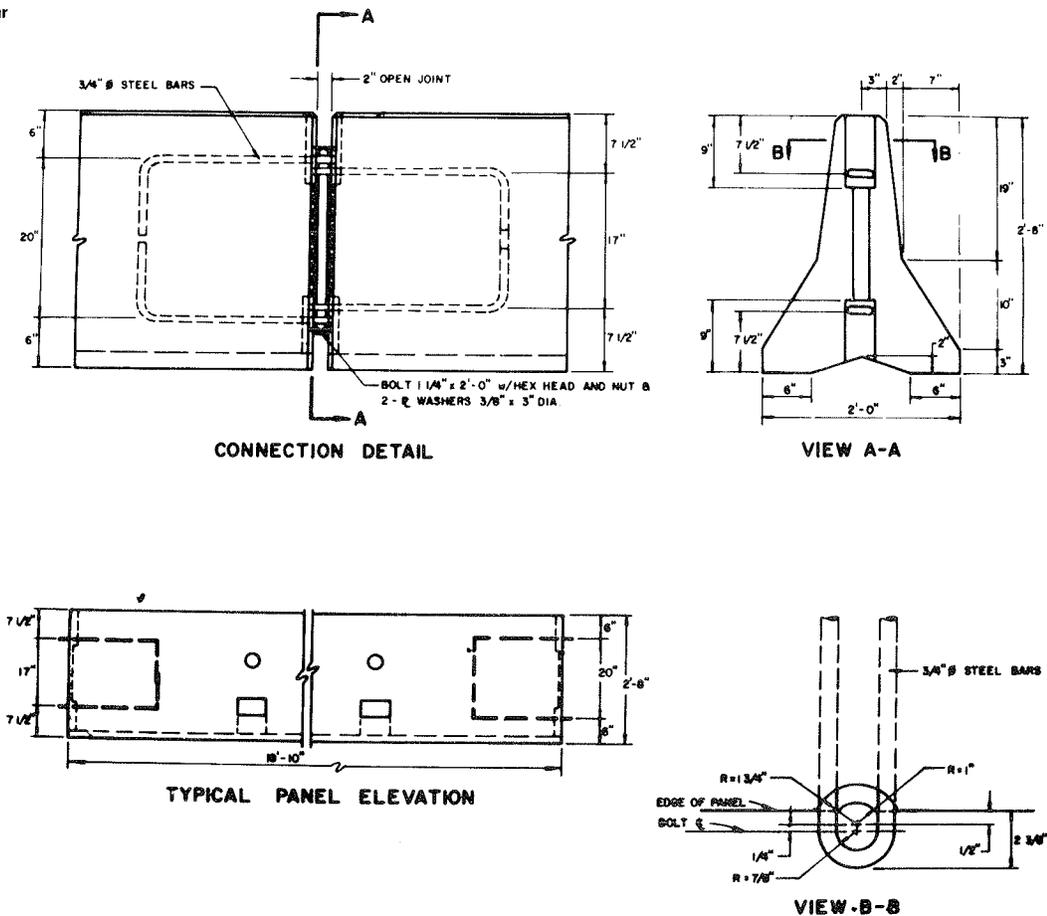


Figure 5. California pin-rebar connection.



Performance Level

Level	Speed (miles/h)	Angle (°)	Weight (lb)
1	60	7	4 500
2	60	15	4 500
3	60	25	4 500
4	60	15	25 000

The tests conducted to date indicate that certain barrier designs are capable of performing at certain levels. Figure 10 implies that the 3.81-m (12.5-ft) barrier tested in Caltrans test 291 will meet performance level 1, the 6.10-m (20-ft) barrier tested in Caltrans test 294 will meet performance level 2, and several other designs, including the New York and SwRI 6.10-m designs and the TTI 9.14-m

Figure 6. New Jersey Welsbach interlock connection.

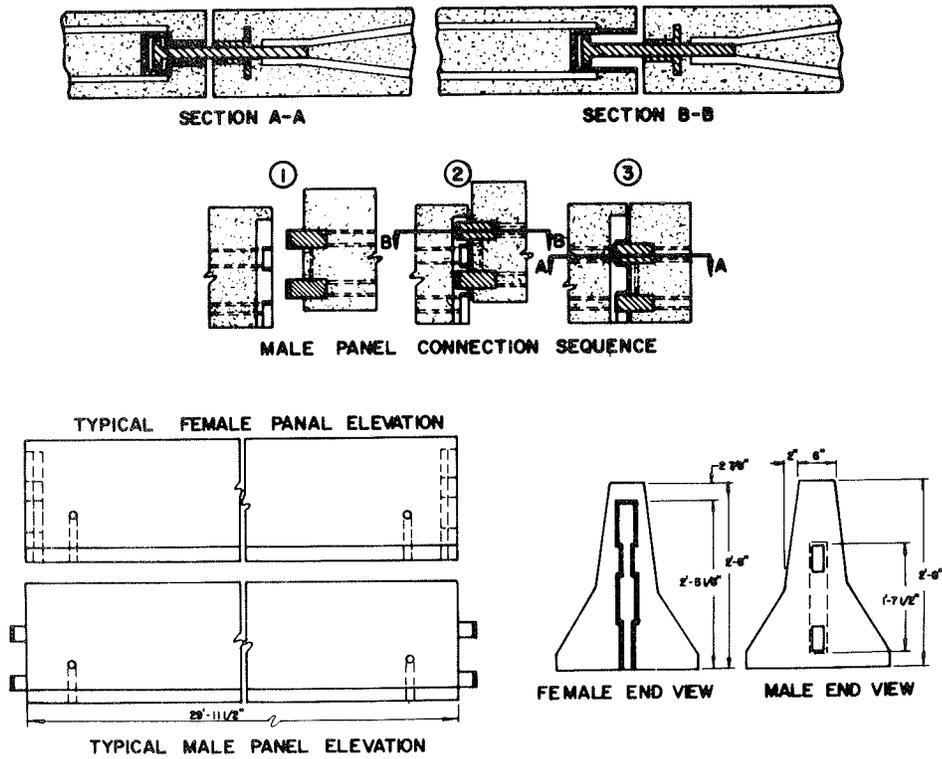


Figure 7. Texas dowel connection.

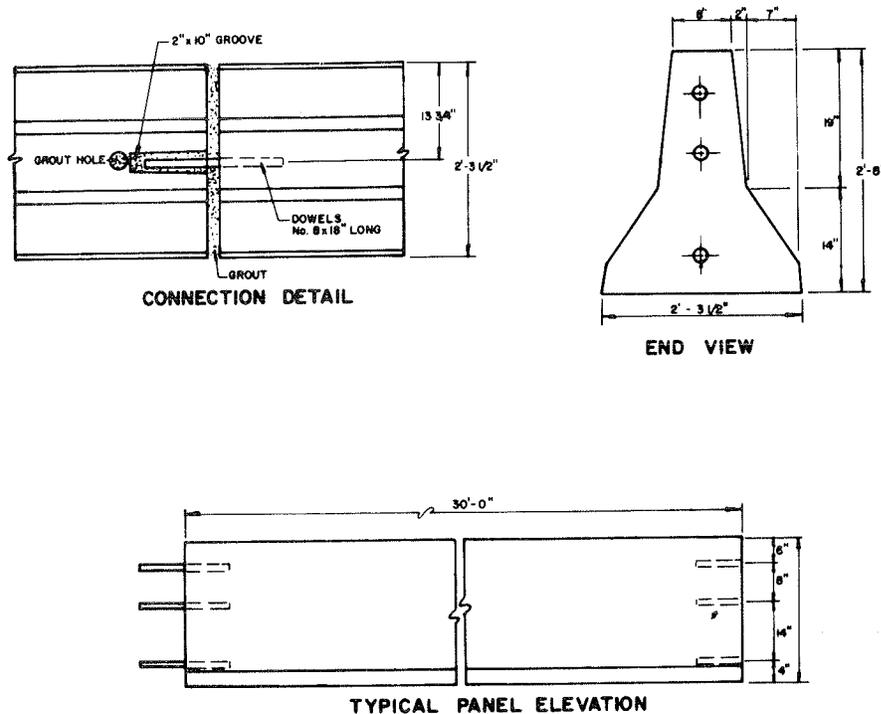


Figure 8. New York "CI" interlock.

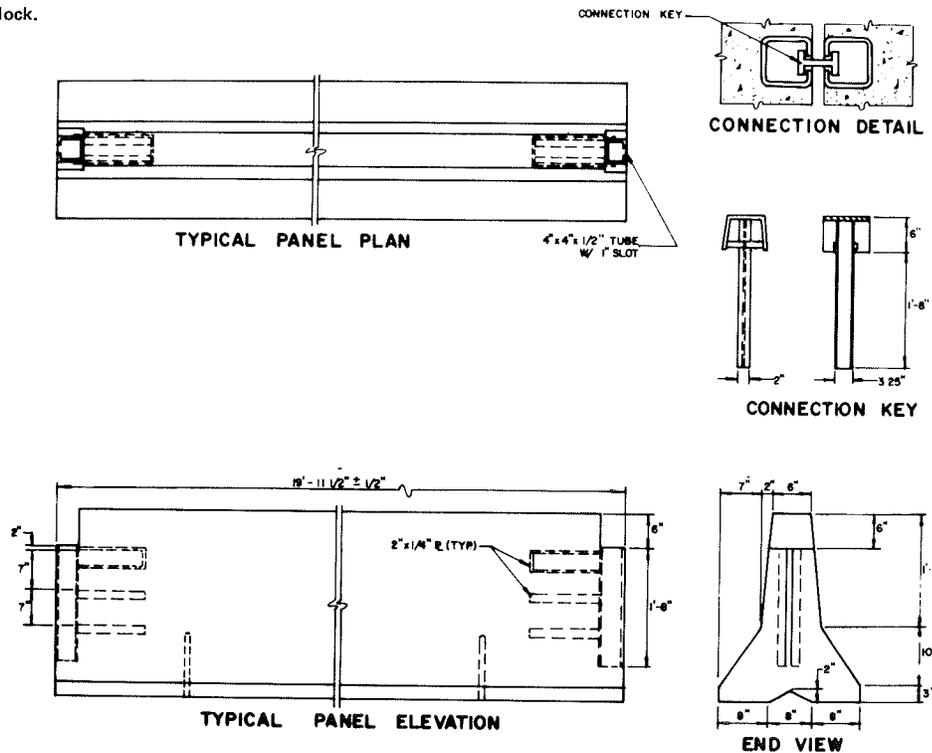


Table 1. Results of tests of portable CMBs.

Testing Agency	Test No.	Condition			Length (ft)	Type of Connection	Moment (kip-ft)	Deflection (ft)	
		Speed (miles/h)	Angle (°)	Weight (lb)				Observed	Computed
Caltrans	291	65	7	4860	12.5	0.875-in pin	9	0.52	0.18
	292	68	28	4860	12.5	0.875-in pin	9	NA ^a	—
	293	66	40	4860	20	1-in pin	12	NA ^a	—
	294	39	25	4700	20	1-in pin	12	0.46	0.53
SwRI	CMB-18	62	25	4500	20	Vertical concrete key connection, 0.25-in plates	6	NA ^a	—
	CMB-24	56	24	4500	20	CI ^b (open)	6	3.42	4.6
NYSDOT	NY-1	53	25	4250	20	CI ^b (open)	96	1.33	2.19
	NY-2	58	25	4230	20	CI ^b (grouted)	96	0.92	0.64
TTI	CMB-2	60	24	4540	30	Dowel (grouted)	50	1.1	0.89

Notes: 1 mile = 1.6 km; 1 lb = 0.45 kg; 1 kip-ft = 1.356 kN·m; 1 ft = 0.3 m.

The Welsbach design included in Table 2 was not included here because the tests conducted used a grout bed and dowels, which precluded displacement of the barrier. The barrier would thus qualify as a rigid system and might be better considered a permanent installation.

^aBarrier segments and/or joints failed, and this precluded the structural integrity required in order for the barrier deflections to be compared with the theoretical deflections. The deflections were thus deemed not applicable.

^bThe steel member embedded in the end of the section has a C-shaped cross section, and the member that slips into the C-shaped section is I-shaped.

(30-ft) design, will meet performance level 3. Stoughton and others (4) have shown that deflections on the order of 0.6 m (2 ft) and greater lead to vehicle ramping and/or straddling of the barrier, although the end result may be containment. There is no general agreement on whether this performance meets the performance criteria of Transportation Research Circular 191 (5). California engineers have taken a conservative view that it may not and have recommended against the use of barriers that deflect to this degree, at least for permanent installations. Another view is that ultimate containment may be sufficient, even though a certain amount of straddling may take place during the impact, and that barrier integrity is the main point of the 2041-kg, 96.56-km/h (4500-lb, 60-mile/h), 25° test.

The data given in Table 3 on the specific tests

as defined by barrier length and joint capacity illustrate compliance with certain performance levels. Although no barrier has been shown by test to meet performance level 4, it is speculated, based on a comparison of New York test CMB-2 and SwRI test CMB-24, that the New York "CI" joint design (Table 1) with a 6.10-m (20-ft) segment length will result in a barrier that would sustain a 15°, 96.56-km/h (60-mile/h) impact by a school bus, the arbitrary definition of performance level 4. It is noted that the New York design tested with a 2041-kg vehicle at 96.5 km/h and 25° resulted in 28 cm (11 in) of barrier deflection. When the energy level is doubled, from level 3 to level 4 [131.9–275.3 kN·m (97.3–203 kip·ft)], the barrier deflection will be considerably greater but probably not more than 0.91 m (3 ft), assuming the joint ductility is sufficient

to develop the specified capacities at a deflection that large.

ENERGY ANALYSIS OF CMB

Some elements of this energy analysis were originally developed by Stoughton in his unpublished analysis of specific CMB tests, which contributed significantly to this research.

A portable CMB subjected to a vehicle impact at or about one of the joints between segments can be analyzed by using the energy method. The analysis is subject to a number of simplifying assumptions. An overhead view of the positions of barrier segments before and after vehicle impact is shown in Figure 11.

The major simplifying assumptions are as follows:

1. Only two segments of the barrier move.
2. The amount of vehicle kinetic energy associated with the lateral component of vehicle velocity is expended in work on the barrier and the vehicle.
3. The complex development of moment in a barrier joint can be approximated as shown in Figure 12.

4. Static and sliding friction between the barrier base and the support media can be approximated as shown in Figure 13.

5. The work done in deforming vehicle structure can be approximated by an equation derived from Figure 14.

The basic energy-balance equation to be used is

$$E\ell = E_m + E_\mu + E_c \tag{1}$$

where

$E\ell$ = amount of kinetic energy associated with the lateral component of vehicle velocity (kip·ft);

E_m = total of E_{m1} , E_{m2} , and E_{m3} , the total work done in rotating barrier joints (kip·ft);

E_μ = work done in sliding two barrier segments through the angle ϕ (kip·ft); and

E_c = work done in deforming the vehicle structure during impact (kip·ft) (Figure 14).

Note that

$$E_{m1} = E_{m1} + E_{m2} + E_{m3} \tag{2}$$

where

Table 2. Structural characteristics of CMB connections.

Connection	Tensile Force P (kips)	Shear Force V (kips)	Moment M^a (kip-ft)	Torsion T (kip-ft)	Rotational Connection Slack ($^\circ$)
Welsbach	270	160	135	95	4
New York I-lock	115	180	96	75	10
California pin and rebar ^b	44	44	37	19	9
California cable posttension	36	20	20	10	0
Texas lapped with bolt	31	22	21	11	0
Minnesota pin and eye bolt	23	23	20	15	14
Idaho pin and wire rope	23	23	19	17	5
Georgia pin and rebar	15	15	12	11	18
Texas dowel					
Calculated	0	51	0	22	NA
As tested	60	51	50		
Oregon tongue and groove	0	41	0	12	NA
Virginia tongue and groove	0	54	0	12	NA
Colorado latch	8	6	7	0 ^c	9

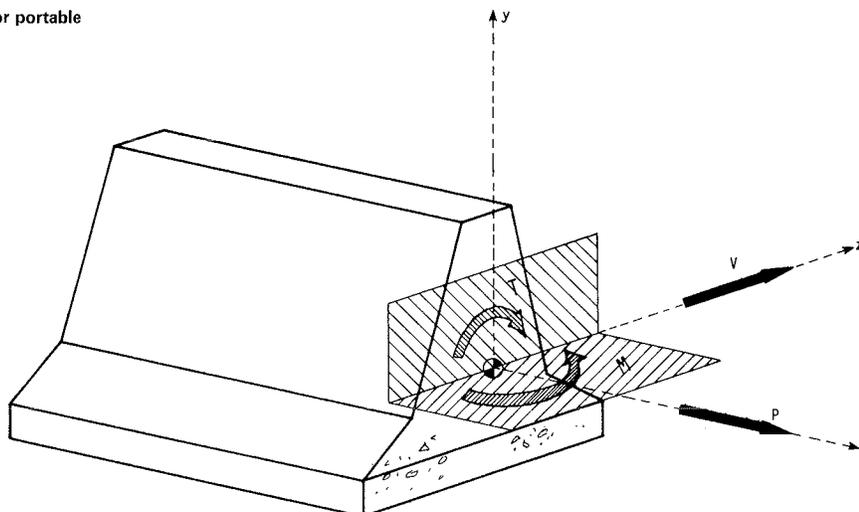
Note: 1 kip = 4.4 kN; 1 kip-ft = 1.356 kN·m.

^a Moment for some segments was calculated by assuming that enough of the concrete faces developed a compressive zone of contact to provide an equal opposing force to the mechanical connection acting in tension. Concrete strength was assumed to be sufficient to develop this force, although in most cases some concrete spalling would be encountered.

^b This design, shown in Figure 5, is not the same as the designs tested in Caltrans tests 291-294. It has been structurally upgraded in comparison with the barriers for which failures were noted during tests.

^c Barrier tilting could unlatch this design, resulting in zero capacities in the preceding three table columns.

Figure 9. Coordinate system for portable CMB connection.



E_{m1} = work done in rotating joint 1 through the angle ϕ (kip·ft) (Figure 12),
 E_{m2} = work done in rotating joint 2 through the angle 2ϕ (kip·ft) (Figure 12), and
 E_{m3} = work done in rotating joint 3 through the angle ϕ (kip·ft) (Figure 12).

The values of E_{m1} , E_{m2} , and E_{m3} can be determined from the following integrals (or numerically from Figure 12): $E_{m1} = \int_0^\phi M d\phi$, $E_{m2} = \int_0^{2\phi} M d\phi$, and $E_{m3} = \int_0^\phi M d\phi$, where ϕ = maximum rotation caused

Figure 10. Functional levels related to combinations of speed and impact angle for 4500-lb automobile.

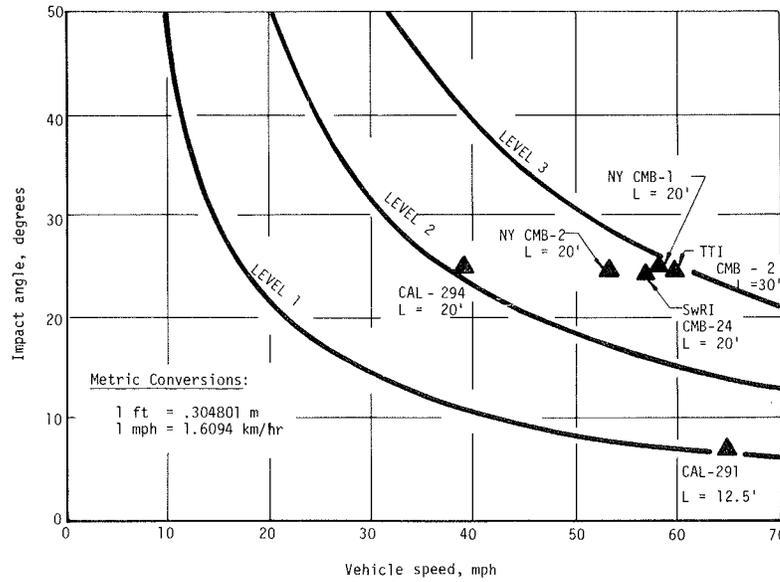


Table 3. Characteristics of portable CMBs required to meet specific functional levels.

Boundary Test Level	Joint Capacity					
	Kinetic Energy ^a (kip-ft)	Segment Length ^b (ft)	Shear (kips)	Moment (kip-ft)	Torsion (kip-ft)	Deflection Limit (ft)
1	8.1	12.5	10	8	8	0.5
2	36.5	20	15	12	12	0.5
3	97.3	20	50	10	15	3.5
4	203	30	50	50	25	3.5
		20 ^c	180	90	70	3 ^d
		30 ^e	100	100	100	2 ^d

Note: 1 kip·ft = 1,356 kN·m; 1 ft = 0.3 m; 1 kip = 4.4 kN.

^a Amount of kinetic energy related to the lateral component of vehicle velocity.

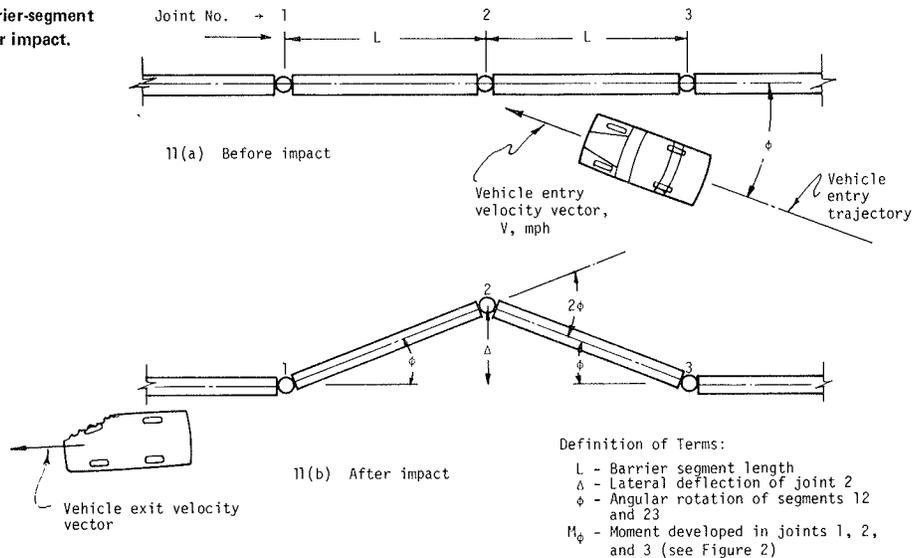
^b For each performance level, only segment lengths whose capacities are confirmed by testing are shown.

^c Although the appropriate function at performance level 4 of a barrier that has these capacities is unconfirmed by test, acceptable performance can probably be based on implications of NYSDOT test CMB-2 and SwRI test CMB-24.

^d Probable.

^e Performance shown acceptable by method of analysis described later in this paper.

Figure 11. Idealized barrier-segment positions before and after impact.



by the impact of segments 1 and 2 and 2 and 3 (rads) and M = development of moment in a joint when it is subjected to an angular deformation of ϕ (kip·ft).

Figure 11 shows that joints 1 and 3 go through an angular deformation of ϕ while joint 2 goes through 2ϕ .

The work done in sliding the barrier segments can be computed by multiplying barrier-segment weight by the amount of friction developed in any interval of sliding movement and summing all these differential portions of work. This value is approximated by the following equation, which can be solved numerically by referring to Figure 13:

$$E\mu = WiL^2 \int_0^\phi u \, d\phi \tag{3}$$

where

Figure 12. Joint moment versus rotation.

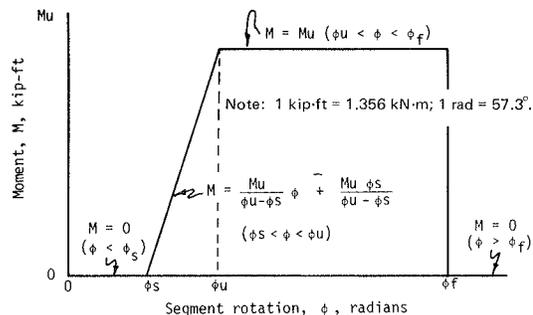


Figure 13. Barrier-support media friction versus barrier-segment rotation.

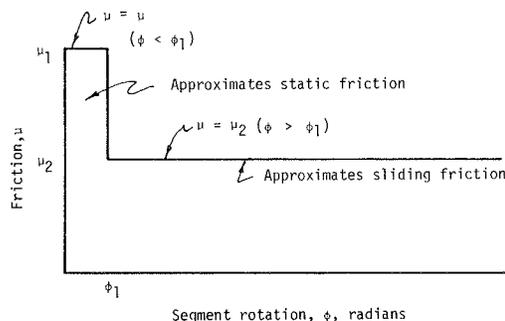
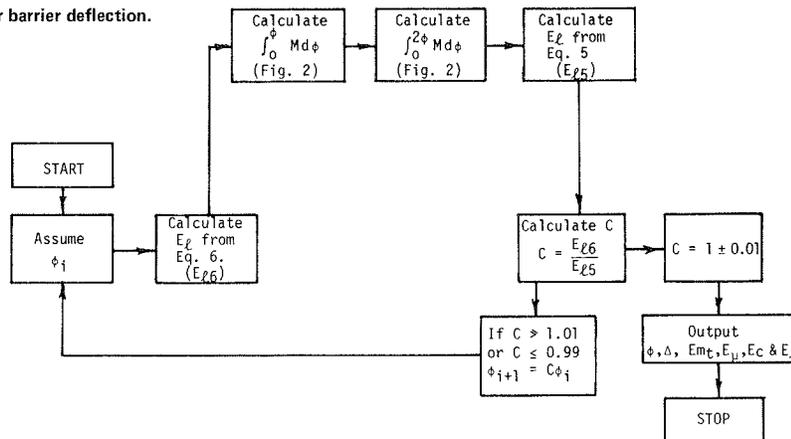


Figure 15. Flow diagram to solve for barrier deflection.



W_i = weight per unit length of the barrier (kip/ft),
 μ = coefficient of friction associated with any movement of the barrier (Figure 13), and
 L = length of a barrier segment (ft).

The work done in deforming the automobile structure (E_C) is approximated by the following equation:

$$E_C = (E\ell/B_1) \{A_1 - [(A_1 - A_2) L \sin \phi] / \Delta \max\} \tag{4}$$

where A_1 , A_2 , and B_1 are constants used in determining E_C (kip·ft) (Figure 14) and $\Delta \max$ is the maximum functional barrier deflection (ft).

By substituting the values of E_{m_t} ($E_{m_1} + E_{m_2} + E_{m_3}$), E_u , and E_C into Equation 2, the following equation can be developed:

$$E\ell = (2 \int_0^\phi M \, d\phi + \int_0^{2\phi} M \, d\phi + W_i L^2 \int_0^\phi \mu \, d\phi) / [1 - (1/B_1) \{A_1 - [(A_1 - A_2) L \sin \phi] / \Delta \max\}] \tag{5}$$

The control value of $E\ell$ is calculated from the following equation:

$$E\ell = (1/2) (W/g) (V \sin \theta)^2 \tag{6}$$

Figure 14. Estimating work done in deforming vehicle structure.

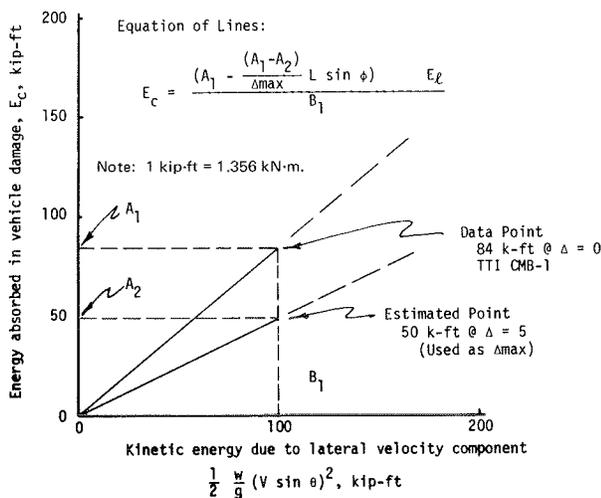


Figure 16. Computed versus observed deflections.

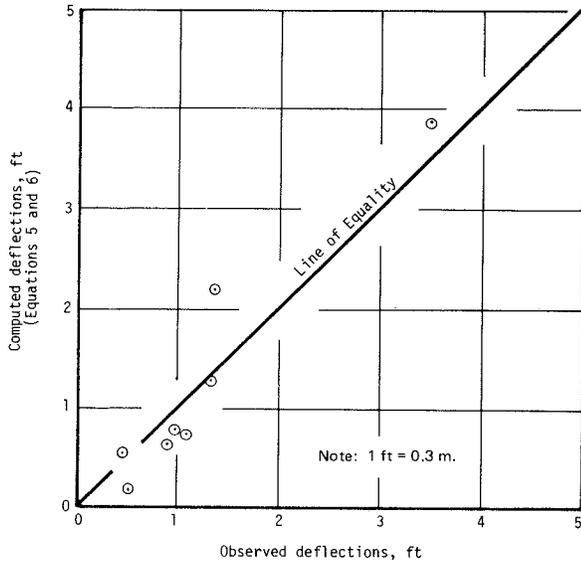
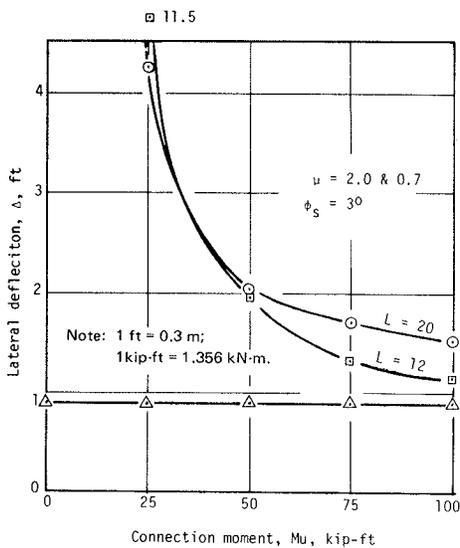


Figure 17. Lateral deflection versus connection moment.



where

- W = vehicle weight (kips),
- g = acceleration of gravity (ft/s²),
- V = vehicle velocity (ft/s), and
- θ = vehicle impact angle (°).

The solution of Equation 5 can be quickly found by using the method of finite differences: assuming a value of ϕ , calculating the value of the right side of the equation, and comparing the calculated value with the known value of $E\delta$ from Equation 6. If $E\delta$ from Equation 5 is greater than $E\delta$ from Equation 6, the value of ϕ is too large. A smaller value should then be estimated and the procedure repeated. If $E\delta$ from Equation 5 is less than $E\delta$ from Equation 6, the value of ϕ is too small and a larger value should be chosen for the next trial. The correct value of ϕ (i.e., the one necessary to balance the equation) will be defined within 1 percent accuracy in 10 iterations if a

Figure 18. Barrier deflection versus connection slack.

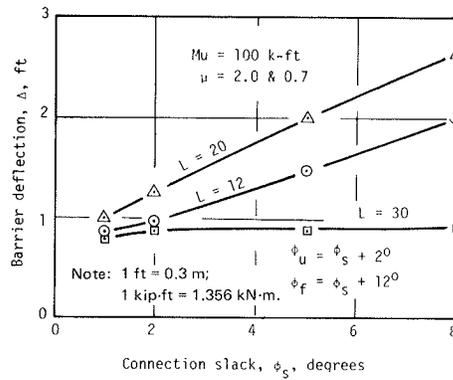


Figure 19. Lateral deflection versus barrier-segment length.

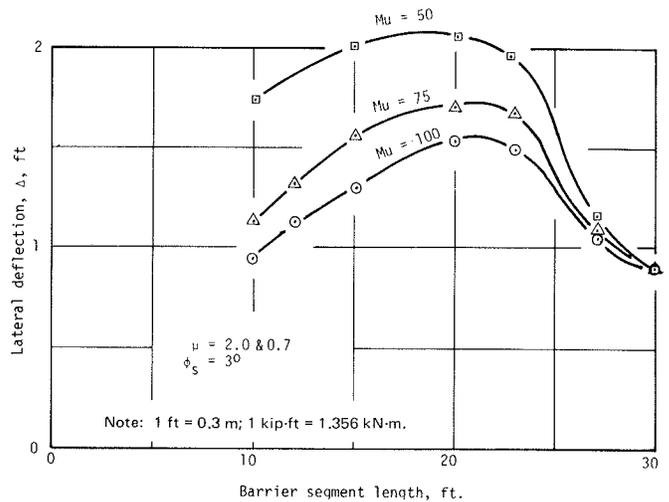
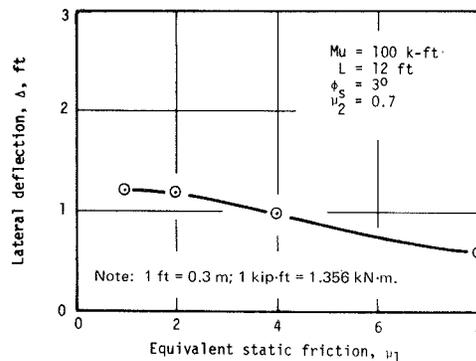


Figure 20. Lateral deflection versus equivalent static friction.



reasonable first estimate of ϕ is chosen. Figure 15 shows a flow chart that illustrates the determination of barrier deflection.

RESULTS OF ENERGY ANALYSIS

The energy analysis developed in the preceding section was performed in several different applications by using different vehicle masses, speeds, and trajectories and various barrier parameters to determine whether the analysis gave

Figure 21. Lateral deflection versus sliding friction.

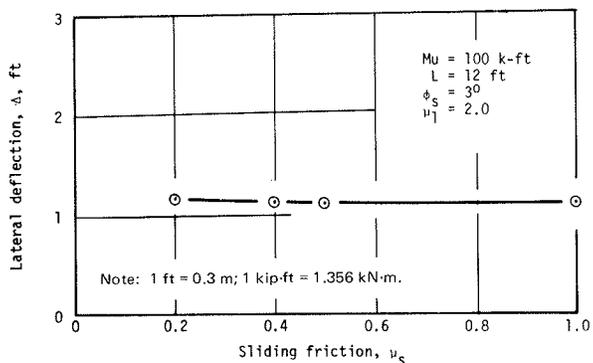
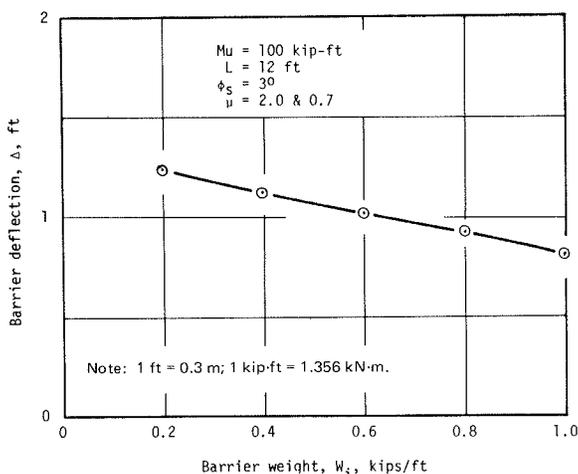


Figure 22. Barrier deflection versus barrier weight.



reasonable results. That exercise was considered successful. The next step in checking this solution was to compare the results of the analysis with the results of actual crash tests. The six tests that were considered applicable for comparison were Caltrans tests 291 and 294, NYSDOT tests 1 and 2, SwRI test CMB-24, and TTI test CMB-2.

Table 1 compares deflection values observed in these tests with values computed by the analytic method. Figure 16 shows that a reasonable correlation is achieved. It was further indicated by the analytic method that the speculated performance of a 6.10-m (20-ft) segment length with a 122-kN·m (90-kip·ft) ultimate moment would be inadequate at an energy level corresponding to performance level 4 [a school bus impacting at 15° and 96.6 km/h (60 miles/h)]. The analysis does indicate that a 9.14-m (30-ft) long barrier segment with a moment capability corresponding to the Welsbach joint design [184.4 kN·m (136 kip·ft)] would perform adequately at performance level 4. The barrier is predicted to deflect approximately 0.61 m (2 ft) during the school-bus impact. Based on comparisons with a fairly small number of tests, it appears that the energy analysis gives reasonable, if perhaps slightly conservative, results. At the higher levels of kinetic energy, the analysis gives deflections slightly greater than those observed in tests.

Parametric studies were conducted to determine, for the first time, barrier sensitivity to the following characteristics: connection moment

capacity, barrier length, connection slack, barrier mass, and barrier-to-ground friction.

Figure 17 shows a fairly low sensitivity to connection moment between 67.8 and 135.6 kN·m (50 and 100 kip·ft) for segment lengths of 3.66 or 9.14 m (12 or 30 ft). When connection moments are less than 67.8 kN·m, sensitivity increases (deflection increases rapidly) between 33.9 and 67.8 kN·m (25 and 50 kip·ft) for barrier lengths of 3.66 and 6.10 m (12 and 20 ft). Barrier segments 9.14 m (30 ft) long are not very sensitive to connection moment at the level 3 test conditions.

Sensitivity to connection slack--that is, the amount one segment can rotate with respect to an adjacent segment before significant yaw moment is produced--lessens as segments become longer. Figure 18 shows an increase in deflection from 0.30 to 0.61 m (1-2 ft) as connection slack goes from 2° to 8° for a segment length of 3.66 m (12 ft); for a 9.14-m (30-ft) segment length, this sensitivity is reduced to an increase of only 14 percent [0.21-0.27 m (0.7-0.9 ft)] as connection slack increases from 2° to 8°.

A fascinating, if perhaps predictable, phenomenon occurs when the influence of barrier-segment length is determined for different values of connection moment capacity. Figure 19 shows that barrier deflection first increases with segment lengths up to a maximum of about 6.10 m (20 ft) and then decreases for segment lengths longer than 6.10 m. This phenomenon becomes less pronounced for smaller values of moment capacity and disappears entirely for a zero moment capacity. At zero moment capacity, deflection becomes continuously smaller as segment lengths increase. The reason for this is the conflicting influences of moment capacity and friction. When joint moment capacities are high, the influence of relatively large joint rotations at fairly small deflections produces significantly higher energy absorption for short segment lengths. In contrast, as segments become longer--6.10-9.14 m (20-30 ft)--the joint rotation at a given deflection becomes smaller and the influence of barrier energy absorption attributable to friction becomes dominant.

The influence of static and sliding friction on barrier deflection is shown in Figures 20 and 21, respectively. The influence of positive connection techniques, such as the dowels provided by California on some temporary installations, can be accommodated by selecting an appropriately high value of static friction. The influence of barrier mass, although perhaps academic at this stage, is shown in Figure 22. It is possible to increase the mass of portable CMBs by using heavyweight aggregate, but this is not likely to be economically justified.

At this stage, the energy analysis appears to be a useful tool in predicting barrier structural capacities. It cannot, however, predict vehicle response. This must be inferred from actual test data. Nor does it predict whether connections have the necessary shear or torsion characteristics to avoid the development of structural discontinuities. Fortunately, these requirements can be determined by standard analytic methods once barrier lateral deflection is accurately predicted.

SUMMARY AND CONCLUSIONS

Portable CMBs used in construction zones throughout the United States exhibit a wide range of structural details and performance characteristics. In many cases, designs are fabricated and applications made on the assumption of adequate performance. Of primary importance is the fact that there has not been a reliable analytic method by which to design

these barriers. The information and analyses included here allow the following conclusions:

1. Some portable CMBs in use have significantly restricted performance capacity.
2. Portable CMBs can be designed to resist high-intensity vehicle impacts.
3. The energy analysis presented here is a relatively simple and useful tool in the design of portable CMBs. This analysis indicates the following: (a) Connection yaw moment capacity is critical if barrier deflections are held to reasonable levels; (b) slack in barrier connections increases barrier deflection significantly; (c) for barriers with significant yaw moment capacity, the 6.10-m (20-ft) segment length is the least effective in reducing barrier deflection; (d) increasing static barrier support friction will significantly reduce barrier deflection; and (e) over a practical range of values, barrier sliding friction has little effect on barrier deflection.

ACKNOWLEDGMENT

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elements of the energy analysis reported here, is gratefully acknowledged.

The contents of this paper reflect our views, and we are responsible for the facts and accuracy of the data presented. The contents do not reflect the views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

REFERENCES

1. M.E. Bronstad and C.E. Kimball, Jr. Temporary Barriers Used in Construction Zones. Southwest Research Institute, San Antonio, Task Rept., Dec. 1977.
2. T.J. Hirsch and E.L. Marquis. Crash Test and Evaluation of a Precast Concrete Median Barrier. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. 223-1, Oct. 1975.
3. D.M. Parks and others. Vehicular Crash Tests of Unanchored Safety-Shaped Precast Concrete Median Barrier with Planned End Connections. California Department of Transportation, Sacramento, Aug. 1976.
4. R.L. Stoughton and others. Vehicular Impact Tests of Precast Concrete Median Barriers with Corrugated Ends and Tensioned Cables. California Department of Transportation, Sacramento, June 1978.
5. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. TRB, Transportation Research Circular 191, Feb. 1978.

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Evaluation of the Performance of Portable Precast Concrete Traffic Barriers

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An evaluation of the portable precast concrete traffic barrier as a device for separating high-speed vehicle traffic and construction activities is presented. The evaluation included (a) a review of the literature on the performance of concrete "safety shape" barriers and (b) an examination of traffic operations and safety characteristics in a construction zone where the portable barriers were used. The literature review revealed that in using the barrier (a) the end of the barrier should never be exposed to oncoming traffic; (b) the barrier joints must be tight for the barriers to act as a system; (c) the longitudinal axis of the barriers should be placed parallel to the roadway, except when the barrier system is started with a flare; and (d) the barrier system must have lateral support to prevent vehicle penetration. For conditions at the study site, it was found that (a) there was an average of 49 vehicle contacts with the barrier for every reported accident in which the barrier was involved; (b) there was a definite tendency for motorists to stay out of the barrier lane, but avoidance of the barrier lane decreased as volume increased; and (c) with an 88-km/h (55-mile/h) posted speed limit, vehicle speeds were reduced only slightly when the barriers were in place.

This paper summarizes the results of a study requested in August 1976 by the Virginia Department of Highways and Transportation to evaluate the performance of the portable precast concrete traffic barrier (PCTB) used to separate freeway traffic and construction activities in the widening of the Virginia Beach-Norfolk Expressway (1). The evalua-

tion consisted of (a) a review of the literature on the performance of concrete "safety-shape" barriers and (b) an examination of traffic-safety and traffic-operations characteristics during the widening of the Virginia Beach-Norfolk Expressway (VA-44). The literature was searched by the Highway Research Information Service of the Transportation Research Board.

The widening project on which the PCTBs were used covered the westerly 9.91-km (6.16-mile) portion of VA-44. The traffic volume on VA-44 ranges from 45 000 vehicles/day in the winter months to 95 000 vehicles/day in the summer months. The widening consisted of adding a median lane in each direction to an existing four-lane limited-access road. The widening project was divided into three sections of approximately 3 km (2 miles) each. Work on the first section was started in September 1976. As each section was completed, the PCTB units were moved to the next section. The new lanes were opened for traffic when possible, and all portions were in service by June 1978.

The PCTB units used on VA-44 had the New Jersey safety-shape profile. They were 0.61 m (24 in) wide, 0.81 m (32 in) high, and 3.66 m (12 ft) long

Figure 1. Precast concrete traffic barriers.

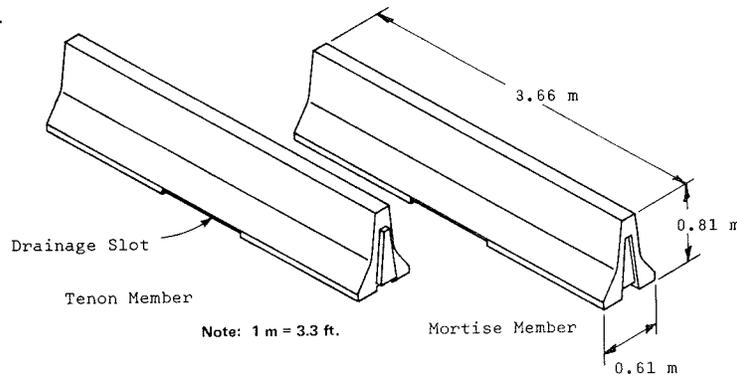
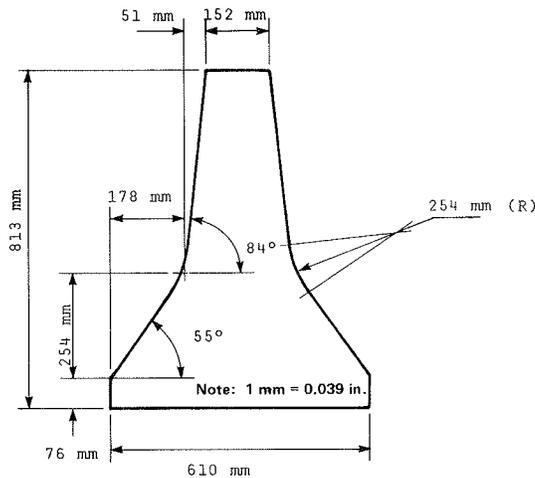


Figure 2. Profile of New Jersey "safety-shape" CMB.



California in 1946 provided the initial insight into its performance capabilities. Based on these experiences, New Jersey highway officials developed a specially contoured profile to give vehicle redirection capabilities to the concrete barrier. The earliest New Jersey barriers were only 0.46 m (18 in) high but, when it was found that vehicles climbed these barriers, the height was increased to the present 0.81 m (32 in). The width and thickness were made sufficient to prevent the barrier from fracturing or overturning when struck by a vehicle (2).

Today's standard New Jersey barrier, often referred to as the safety-shape barrier, is 0.81 m (32 in) high and has a 0.61-m (24-in) base and a 0.15-m (6-in) top width (see Figure 2). It incorporates a 55° batter-curb face with an upper portion (stem) at 84° from the horizontal.

The theory of concrete-barrier performance is relatively simple. When a vehicle strikes a barrier like that shown in Figure 2 at an angle of less than 15°, the initial contact is between the 76-mm (3-in) vertical curb and the vehicle tire. This contact deforms the tire and tends to slow the vehicle. The front wheel then climbs up the 55° batter-curb face, and the vehicle is lifted from the roadway. The lifting of the vehicle dissipates some of its kinetic energy and places it in such a position that the redirecting forces perpendicular to the barrier can be applied to its suspension system. At a low angle of impact, there is usually no contact between the side of the car and the barrier. If the impact speed is high and the impact angle is more than a few degrees, the vehicle may climb up the 55° sloped face to its intersection with the upper portion of the barrier. As the front portion of the vehicle wheel contacts the upper (near-vertical) portion of the barrier, the wheel is turned parallel to the barrier's longitudinal axis and the vehicle is redirected. Depending on the impact speed and angle, the vehicle may continue to climb up the near-vertical portion of the barrier before returning to the roadway.

and weighed approximately 2200 kg (4800 lb). The joints were of the tongue-and-groove design. Individual sections were either a tenon member (male-male) or a mortise member (female-female), which facilitated the removal of a member at the midsection when necessary (see Figure 1).

The PCTBs were placed 150 mm (6 in) from the edge of the existing roadway to allow room for construction reference points. The passing lane during construction was 2.9 m (9.5 ft) wide, and the traffic lane remained at 3.9 m (12 ft). Lateral support was provided only on bridge decks, where the units were placed on channels bolted to the decks. The barrier system was introduced at the start of the work area by either a 91-m (300-ft) barrier taper or by a sand-filled plastic barrel crash cushion.

BARRIER

Design Characteristics

The PCTB is a portable barrier designed to restrain and redirect impacting vehicles with minimal damage to the vehicle or injury to its occupants. Its use as a temporary barrier followed the successful use of the concrete median barrier (CMB) on permanent installations. The ability of the PCTB to safely restrain and redirect impacting vehicles arises from the design characteristics of its forerunner, the CMB.

Use of the CMB in Louisiana in 1942 and in

Performance Characteristics

As used to separate traffic from construction activities, the PCTB has many characteristics that can significantly alter its performance from that described above. The following is a brief summary of some but not all of the characteristics that should be considered in using a PCTB system. For characteristics that are not covered in this paper, the reader is referred to a 1977 American Association of State Highway and Transportation Officials (AASHTO) publication for permanent barrier installations (3).

Figure 3. General Motors, New Jersey, and configuration F barrier profiles.

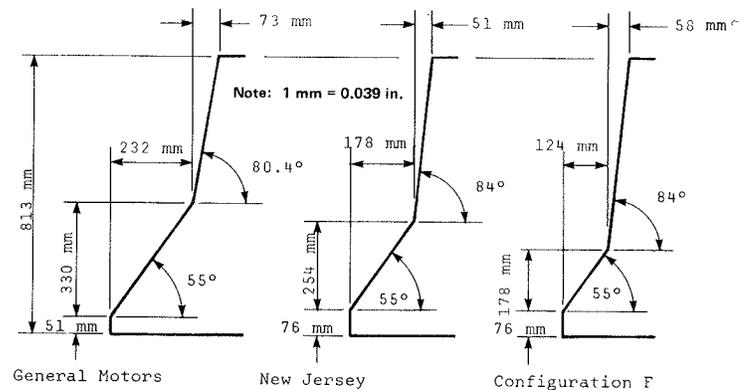
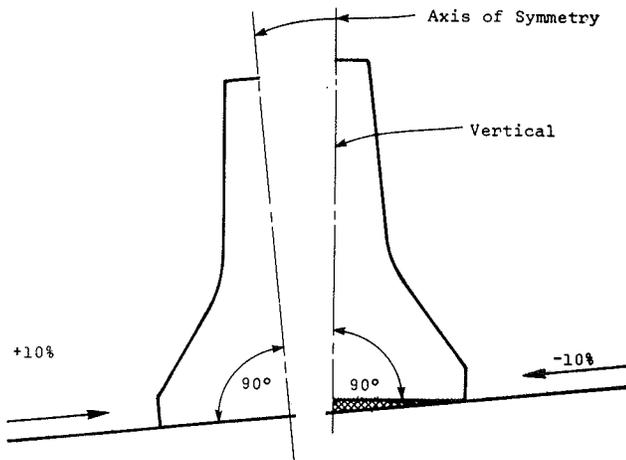


Figure 4. Preferred barrier orientation on superelevated surfaces.



Crash tests have shown that the shape of the concrete barrier can significantly affect the performance of any impacting vehicle. Eight barrier profiles, including the General Motors, New Jersey, and configuration-F designs (see Figure 3), were evaluated by the Southwest Research Institute (SwRI). The tests indicated that all designs performed well in restraining vehicles from penetrating the barrier and most did not cause major damage to the impacting vehicles. However, the General Motors design did cause rollover of a subcompact vehicle at an impact speed of 92 km/h (57 miles/h) and an angle of 16°. Generally, those designs that incorporated a low batter-curb height (the configuration-F and New Jersey designs) were found to be least likely to cause rollover of subcompact cars (4). The batter-curb height is the vertical distance from the bottom of the barrier to the intersection of the 55° batter curb and the near-vertical portion of the barrier. Minicar crash tests performed in England by the Transport and Road Research Laboratory (TRRL) confirmed the crash test results obtained by SwRI. However, in a TRRL crash test, a minicar rolled over when it struck a New Jersey barrier at an impact speed of 113 km/h (70 miles/h) and an angle of 20°. The TRRL study also found that, when a 75-mm (3-in) overlay was placed in front of the New Jersey barrier, a minicar impacting at 113 km/h and 20° did not roll over. The overlay results in a batter-curb height of 254 mm (10 in), which is equal to that of the configuration-F design (5).

The potential for high-angle impacts increases as

the distance between the travel way and a CMB increases. Based on vehicle accelerations at impact, the Texas Transportation Institute recommended that concrete barriers should not be installed more than 3.7 m (12 ft) from the edge of the roadway because of the potential of a high-angle impact (6). The AASHTO publication cited above (3) also suggests that rigid barriers should not be used more than 4.6 m (15 ft) from the edge of the roadway. These references emphasize that the barriers are designed to safely redirect a vehicle that strikes a barrier at an angle of 15° or less.

The movement of the temporary barrier on impact can also significantly affect vehicle performance. Crash strength tests performed by the California Department of Transportation (Caltrans) showed that barrier roll and excessive lateral movement contributed to vehicle vaulting and hazardous vehicle trajectories. The report on the tests stressed the need for tight barrier joints to reduce the potential hazards produced by barrier movement (7). Crash strength tests conducted by SwRI demonstrated that temporary barriers offer substantially less resistance to impact than barriers that are restrained by a continuous foundation. The report on the study by Bronstad and others (4) recommended that temporary barriers should have some continuous support. If this is not possible, the barrier mass and base friction must provide lateral restraint, and the joints must be capable of transmitting yaw movement to adjacent barriers. The authors also noted that, among roll, yaw, and lateral displacement, roll is the most important barrier movement to consider because it induces vehicle ramping (4).

In computer-simulated vehicle crashes conducted by SwRI, the orientation of the barrier axis of symmetry was identified as a factor that contributes to vehicle rollover. In that analysis, only +10 percent and -10 percent superelevations were used (see Figure 4). A subcompact vehicle rolled over when it struck a New Jersey barrier, the axis of symmetry of which was oriented perpendicular to a roadway surface with a -10 percent superelevation. After the analysis was completed, the researchers concluded that the preferred orientation of the barrier axis of symmetry is "perpendicular to the roadway when the traffic is going up the super and vertical when the traffic is going down the super" (4). The orientation of the barrier axis of symmetry should be considered when PCTBs are to be placed on a surface that slopes away from the roadway. If it is determined that the surface must be graded before installation of the barrier, it is important to ensure that (a) vertical support is provided across the full width of the barrier to reduce barrier roll and (b) the toe of the barrier is not placed below the approach surface level.

The manner in which a barrier system is introduced can significantly affect the severity of impact by a vehicle. The end of a barrier system should never be exposed to oncoming traffic; it should be removed or shielded from errant vehicles. The end can be removed from a roadway by flaring the system. The AASHTO Guide (3) provides information on the rate at which a barrier system should be flared from the roadway. Flare rates (number of meters parallel to the roadway per meter perpendicular to the roadway), which are a function of operating speeds, are given below (1 km = 0.62 mile; 1 m = 3.3 ft):

Operating Speed (km/h)	Flare Rate (m/m)
113	20
97	17
80	14
64	11

According to the AASHTO Guide, the purpose of the flare is "(1) to locate the barrier and its terminal as far from the traveled way as is feasible, (2) to redirect an errant vehicle without serious injuries to its occupants, and (3) to minimize a driver's reaction to a hazard near the traveled way" (3). The end of a barrier flare should be located beyond the clear zone line discussed in Chapter 3 of the AASHTO Guide. However, if this is impracticable, the end should be shielded from errant vehicles. This can be accomplished by flaring the barrier system behind a guardrail system in such a manner as to avoid the possibility of a vehicle hitting the end or of pocketing if it hits the guardrail. Another method of making the end of a barrier system crashworthy would be to install a crash cushion or some other appropriate impact-attenuating device.

In summary, the performance of an impacting vehicle can be significantly affected by (a) the profile of the barrier, (b) the distance between the travel way and the barrier as it relates to the angle of impact, (c) excessive barrier movement, (d) the orientation of the barrier axis of symmetry, (e) the barrier system flare rate, and (f) the exposed end of the barrier system.

CONSTRUCTION ZONE

The field evaluation of the PCTB consisted of examination of (a) construction-zone traffic accidents, with emphasis on those in which a vehicle came into contact with the PCTB, and (b) the effects of construction on characteristics of traffic operations, such as traffic volume, vehicle speed, and lateral placement.

Traffic-Safety Analysis

The purpose of the traffic-safety analysis was to determine the effect of the VA-44 construction work on the traffic-safety environment with emphasis on the performance of the PCTB. There were three phases: (a) an examination of data on traffic accidents on VA-44 during construction to determine the effects of construction on the characteristics of traffic accidents, (b) a study of reported accidents on VA-44 in which vehicles came into contact with the PCTBs, and (c) an analysis of the tire marks on barriers involved in accidents to obtain an indication of the effectiveness of the PCTB in safely redirecting vehicles.

Accidents

In the first phase, an attempt was made to compare accidents that occurred before and during construc-

tion to determine the magnitude of the traffic-safety problem associated with construction. However, 48 percent of the accidents before construction involved vehicle contact with barriers that were used on two ramp-improvement projects. Thus, the effects of construction on traffic accidents could not be determined with any degree of reliability, and the analysis was limited to the following summary of during-construction accident data.

The analysis of the during-construction accident data revealed that there were 39 accidents during the nine-month study period, of which none were fatal accidents and 10 were injury accidents. The total accident rate (accidents per 100 million vehicle miles of travel) was 136.8, and the injury accident rate was 35.1. Driver inattention was identified as the major cause in 25 of the 39 accidents (64.1 percent), and driving under the influence was listed for 7 of the accidents (17.9 percent). Six accidents (15.4 percent) involved vehicle contact with the concrete barrier. Twenty-four accidents (61.5 percent) were of the rear-end type. Twenty-two of the 72 vehicles (30.6 percent) involved in the construction accidents were traveling at a speed of less than 32 km/h (20 miles/h) before impact. The high percentage of rear-end accidents and the slow-moving vehicles are indicative of stop-and-go traffic. Of the 72 vehicles involved in these accidents, 60 (83.3 percent) were cars and 10 (13.9 percent) were single-unit trucks. These figures approximate the normal traffic mix of 84 percent cars and 15 percent single-unit trucks (8). A review of the accidents by location found no clustering of accidents at any specific location.

Accidents Involving a Barrier

The second phase in the traffic-safety analysis included a summary of data for the 10 reported accidents in which a vehicle came in contact with the PCTB units between September 22, 1976, when the first concrete barriers were placed on VA-44, and December 31, 1977. Because the small sample size precluded any in-depth statistical analysis of the data, only a general summary is given here.

Of the 10 reported accidents that involved vehicle contact with a barrier, 3 were injury accidents and 7 were property-damage-only accidents. Five of the impacting vehicles remained in the lane adjacent to the barrier after contacting the barrier, 2 infringed on the adjacent lane, and 3 crossed into the adjacent lane, with 1 hitting an adjacent vehicle. Two vehicles that struck the barrier at a high impact angle (>15°) rolled over. The barrier was struck first in 6 of the 10 accidents; 3 of the remaining accidents were rear-end accidents, and 1 vehicle hit a guardrail before hitting the barrier. In 6 of the 10 accidents, only 1 vehicle was involved, and, in the remaining 4, only 1 vehicle in each accident contacted the barrier.

The typical barrier displacement caused by vehicle contact was less than 0.3 m (1 ft), but one incident in which a van struck the barrier at an estimated speed of 88 km/h (55 miles/h) and an angle of 45° resulted in a barrier displacement of 2.4 m (8 ft). Concerning this accident, it should be noted that (a) the impact conditions were severe compared with those under which the strength tests on the permanent barriers are made (9), (b) the van rolled over after impact, and (c) the van did not enter the construction area.

Tire Marks

The purpose of the examination of tire marks was to determine the effectiveness of the PCTB used on VA-44 in safely redirecting vehicles. The principle on which this determination was based is that the profile of the PCTB is designed to safely redirect vehicles that strike the barrier at a shallow angle of incidence. If the PCTB performed as anticipated, a traffic-accident analysis would not identify those vehicles that hit it and were safely redirected but evidence of the vehicle's involvement would remain on the face of the PCTB in the form of tire marks. Thus, a correlation between vehicle involvements with a barrier and traffic accidents in which a vehicle contacted the barrier should give an indication of the effectiveness of the PCTB in safely redirecting vehicles. It should be noted that the correlation between vehicle involvements and barrier-involved accidents can be affected by any factor that can affect barrier performance. These factors include the profile of the barrier, the orientation of the axis of symmetry, the tightness of the joints, the distance from the travel way to the barrier, and the flare rate. Thus, the correlation is a measure not only of the particular design of the barrier used but also of the conditions under which it is used.

In this analysis, tire marks on the barrier were identified, photographed, and cataloged as to roadway location. Scuff marks on the 76-mm (3-in) vertical curb were identified during the logging operation but were not included in the number of vehicle involvements, since they could have been made by the side of the tire and thus might not be an indication of vehicle climb on the barrier. During the course of the study, there was evidence of 243 vehicle involvements with the barrier above the 76-mm vertical curb and 5 reported accidents in which a vehicle came in contact with the barrier. This information reduces to an average of 49 vehicle involvements with the barrier for every reported accident in which the barrier was involved. The rate of 49 to 1 is used here as the rate of barrier effectiveness.

In an attempt to determine the significance of the rate of barrier effectiveness found on VA-44, a literature search was initiated to identify any similar data. This search found a 1976 SwRI study (4) that reported that on two Indiana roadway sections there were 99 vehicle involvements that resulted in 32 reported accidents, or a rate of barrier effectiveness of 3 to 1. However, the barriers in Indiana were not temporary barriers but New Jersey-shaped CMBs located in a median that varied in width from 1.2 to 3.4 m (4-11 ft). Since the New Jersey profile was used on VA-44 and on the Indiana roadways, the difference in the rates of effectiveness suggests that the rates were significantly affected by the distance between the travel way and the barrier.

The tire-mark data can also be used in evaluating the performance of an installed barrier system. A review of the VA-44 tire-mark data by roadway location revealed that the end of the barrier flare adjacent to the travel way at the start of the work area was the most-often-hit "point location" on the barrier system. There were 27 vehicle involvements at these locations (11 percent of the total). It appears that the drivers of the vehicles did not expect the barriers to encroach into their lane to the extent that they did. This indicates that there is a need to adequately warn and physically move traffic before the introduction of a barrier system that encroaches on or in some way reduces the lane width.

Roadway alignment also appears to be associated with the frequency of vehicle involvements with the barrier. On VA-44, there was an average of 9.7 vehicle involvements with the barriers per million vehicle miles of exposure. Barriers located in the left-hand curves had an average rate of 12.4. Tangent sections had an average rate of 8.7 and right-hand curves (barrier on left) an average rate of 7.1. A notable deviation from the average involvement rates was the rate of 32.3 on a right-hand curve that was preceded by a 2.7-km (1.7-mile) tangent section. Seventy-one percent of the involvements at this curve were located in the first 150 m (500 ft) of the curve. Time-lapse film taken at this location showed that drivers tended to start turning after their vehicles were already in the curve, where they were closer to the barrier than they were in the tangent section. This observation may indicate that additional lane width or the shifting of the lane away from the barrier is required under these circumstances to compensate for the delayed driver maneuvers.

Traffic-Operations Analysis

One of the concerns in using the PCTB was its potential adverse effects on traffic operations, particularly the effect of physically reducing the median lane width from 3.7 to 2.9 m (12-9.5 ft) by placing the barrier on the left-hand edge of the normal travel way. To evaluate the effects of this condition, experiments were designed to examine a variety of traffic characteristics with and without the barrier under various traffic volumes, roadway alignments, and lighting conditions. Specifically, the analysis consisted of identification and examination of the effects of the barriers on trends in average vehicle speeds, lane distribution, and lateral placement. Data were collected at three sites: a tangent section, a right-hand curve, and a left-hand curve.

Vehicle Speed

The analysis of vehicle speeds was undertaken to partially identify driver reaction to the barrier. It was assumed that any serious traffic-flow problems would show up in the speed analysis.

As the data given in Table 1 show, average vehicle speeds decreased in both lanes during construction. In both lanes, the average decrease was slightly more than 3.2 km/h (2 miles/h), and the average speed decreased significantly in all of the data-collection periods (t -test, $p > 0.99$). The data in Table 1 also show that the barrier had a similar effect on vehicle speeds in all three alignment conditions and under all traffic volumes. The one exception to this, the speed reduction in the right-hand curve at 8:00 a.m., was caused by the fact that the traffic volume approached the roadway capacity. Finally, the data in Table 1 show no discernible trends in the change-in-speed variance as a result of the barrier. This information taken together indicates that, although drivers are undoubtedly aware of the construction activity--as the average speed reductions show--both lanes were affected equally and the influence was small under the conditions examined.

Lane Distribution

It was assumed that the presence of the barrier and the consequent reduction in the median-lane width would make motorists using that lane somewhat uncomfortable. It was reasonable, therefore, to hypothesize that there would be some observable

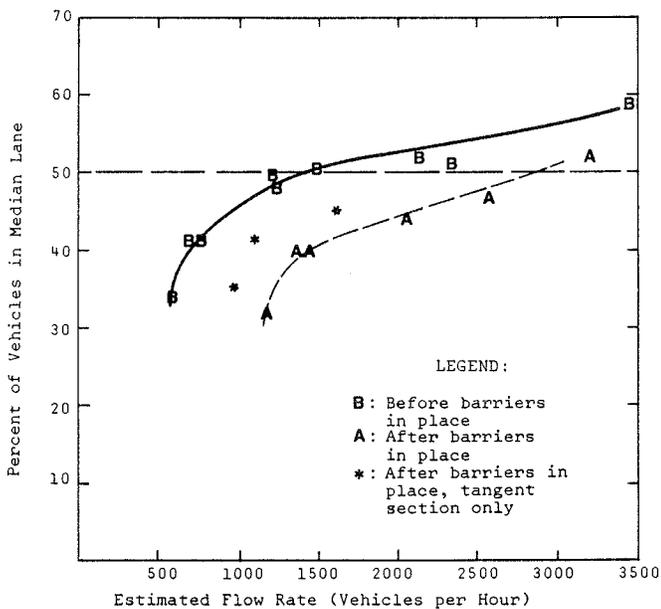
Table 1. Average vehicle speed.

Site No.	Site	Lane	Time	Without Barrier		With Barrier	
				Avg Speed (km/h)	Variance (km ² /h)	Avg Speed (km/h)	Variance (km ² /h)
1	Tangent section	Median	8:00 a.m.	98.0	52.6	97.4	51.0
			12:00 noon	98.3	42.7	92.1	42.5
			10:00 p.m.	95.6	43.3	95.0	45.1
	Shoulder	8:00 a.m.	91.4	97.9	90.6	76.7	
		12:00 noon	91.9	73.0	85.5	53.1	
		10:00 p.m.	89.6	55.7	86.9	59.1	
2	Right-hand curve	Median	8:00 a.m. ^a	92.1	25.4	80.8	22.0
			4:00 p.m. ^a	95.6	33.4	91.1	39.1
			10:00 p.m.	95.0	50.0	91.9	44.8
	Shoulder	8:00 a.m. ^a	88.4	44.8	78.5	30.8	
		4:00 p.m. ^a	89.5	55.4	85.3	47.4	
		10:00 p.m.	89.8	54.9	86.7	57.8	
3	Left-hand curve	Median	8:00 a.m.	96.6	32.6	95.1	53.1
			12:00 noon ^a	94.3	39.1	92.4	46.9
			10:00 p.m.	93.8	47.1	91.4	42.5
	Shoulder	8:00 a.m.	90.0	44.3	87.2	68.9	
		12:00 noon ^a	87.7	49.7	86.3	57.0	
		10:00 p.m.	88.2	52.3	85.5	45.8	

Note: 1 km = 0.62 mile; 1 km²/h = 0.38 mile²/h.

^aAverage flow rate in both lanes more than 2000 vehicles/h.

Figure 5. Lane distribution.



tendency in motorists to avoid using the median lane during construction. This hypothesis was examined by analyzing the distribution of traffic between the two lanes before and after the barrier was in place.

As expected, the distribution of traffic between the lanes changed with traffic volume. Figure 5 shows that at low traffic volumes drivers exhibited a definite tendency to stay out of the median lane when the barrier was present. As the traffic volume increased, however, a larger percentage of motorists chose the left lane, no doubt because of its higher speed even when the barrier was present. Figure 5 also shows that there was a larger percentage of vehicles in the median lane in the tangent section than in either curve section. This finding suggests that motorists feel relatively more comfortable traveling immediately adjacent to the barrier in tangent sections than in curved sections.

Lateral Placement

The analysis of lateral vehicle placement was concerned with the position of vehicles with and without the barriers as a function of roadway alignment, traffic volume, and lighting conditions. At all of the sites examined, the existing centerline was left in place in anticipation of the tracking problems vehicles would have with the concrete centerline joint if the centerline were shifted. In addition, before construction, standard centerlines and edge lines were in use. No edge line was placed next to the barrier after it was installed, but standard barrier delineators (reflectors and barricade warning lights) were used throughout the project.

Lateral placements for all three sites are summarized in Table 2. In all cases, lateral placements were measured from the outside wheel of the vehicle to the edge of the pavement or to the bottom of the barrier, as shown in Figure 6.

Before construction, the lateral placement in the shoulder lane averaged 1.0 m (3.3 ft) and the variance was relatively large--0.13 m² (1.40 ft²). Before construction, the larger shoulder-lane lateral placements were observed at night and at site 3; the smaller lateral placements were observed at site 2 and during periods of high traffic volume (i.e., when flow rates were greater than 2000 vehicles/h).

During construction, the average lateral placement in the shoulder lane decreased an average of 0.2 m (0.6 ft) to an average of 0.8 m (2.7 ft). However, as before construction, the larger lateral placements were observed at night and at site 3 and the smaller lateral placements at site 2 and during periods of high traffic volume. These findings suggest that, whereas the barrier did result in a shift of traffic to the right (i.e., away from the median lane), it did not mask the natural tendency of the shoulder-lane vehicles to (a) "cut the corners" in both the curved sections, (b) travel closer to the centerline at night, and (c) stay farther to their side of the roadway during peak periods.

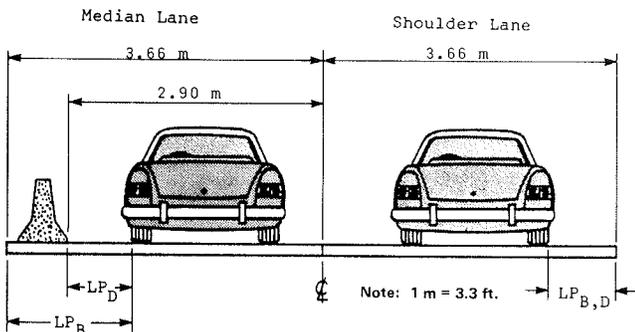
Although all of the reductions in lateral placement were significant (t-test, p > 0.99), they were greatest at night [0.23 m (0.75 ft)] and least in the tangent section [0.15 m (0.48 ft)]. Table 2 also indicates that variance in lateral placement changed very little after the placement of the bar-

Table 2. Lateral placement.

Site No.	Site	Lane	Time	Without Barrier ^a		With Barrier ^b	
				Mean Lateral Placement (m)	Variance (m ²)	Mean Lateral Placement (m)	Variance (m ²)
1	Tangent section	Median	8:00 a.m.	0.93	0.11	0.91	0.05
			12:00 noon	0.98	0.14	0.90	0.06
			10:00 p.m.	0.92	0.14	0.88	0.06
		Shoulder	8:00 a.m.	0.94	0.14	0.89	0.13
			12:00 noon	0.98	0.12	0.86	0.12
			10:00 p.m.	1.26	0.14	1.00	0.12
2	Right-hand curve	Median	8:00 a.m. ^c	0.87	0.08	0.88	0.05
			4:00 p.m. ^c	0.90	0.11	0.98	0.08
			10:00 p.m.	1.02	0.13	0.94	0.10
		Shoulder	8:00 a.m. ^c	0.88	0.11	0.69	0.11
			4:00 p.m. ^c	0.87	0.12	0.68	0.13
			10:00 p.m.	1.16	0.14	0.94	0.12
3	Left-hand curve	Median	8:00 a.m.	0.78	0.11	0.88	0.09
			12:00 noon	0.73	0.11	0.91	0.07
			10:00 p.m.	0.77	0.13	0.77	0.07
		Shoulder	8:00 a.m.	1.05	0.12	0.89	0.15
			12:00 noon	1.08	0.12	0.87	0.14
			10:00 p.m.	1.32	0.14	1.11	0.15

Note: 1 m = 3.3 ft; 1 m² = 10.76 ft².
^a Measured to the edge of the pavement.
^b Measured to the bottom of the barrier.
^c Average flow rate in both lanes more than 2000 vehicles/h.

Figure 6. Lateral placement.



LP_B = Lateral Placement Before Construction

LP_D = Lateral Placement During Construction

riers. This suggests that, although motorists in the shoulder lane were undoubtedly keenly aware of the shift in the median-lane vehicles, they did not feel unduly cramped.

The average lateral placement in the median lane was 0.88 m (2.9 ft) both before and during construction. As in the shoulder lane, there was still a tendency for motorists to "cut the corner" at site 3, but this phenomenon was less pronounced when the barrier was in place. There was no consistent corner-cutting at site 2 with or without the barrier. At site 1, motorists drove slightly closer to the barrier than they did to the edge of the pavement when the barrier was not in place.

The most dramatic evidence of driver reaction to the presence of the barrier was the large and consistent reduction in lateral-placement variance in the median lane from an average of 0.12 to 0.07 m² (1.30-0.73 ft²). This reduction in the variance indicated that during construction motorists were paying much more attention to their lane position and, therefore, to their steering tasks.

CONCLUSIONS

Literature Review

1. CMBS with low batter-curb heights can safely

redirect cars that impact at high speeds and at angles of less than 15°.

2. Excessive movement of temporary barrier units can contribute to vehicle vaulting and hazardous vehicle trajectories.

3. The joints in a temporary barrier system must be tight if the individual units are to perform as a system.

4. A temporary barrier system must have lateral support to prevent vehicle penetration.

5. The improper orientation of the barrier axis of symmetry can contribute to vehicle rollover.

6. A temporary barrier system should not be placed more than 3.7-4.6 m (12-15 ft) from the edge of the roadway because of the potential for high-angle impacts.

7. The longitudinal axis of the barriers should be placed parallel to the roadway, except when the barrier system is introduced with a flare, and the flare rate should not exceed the rate given in the table on page 33.

8. The end of the barrier system should never be exposed to oncoming traffic; it should be removed from the travel way by flaring or be made crashworthy by installing a crash cushion or some other appropriate impact-attenuating device.

Field Study

1. A comparison of the frequency and characteristics of traffic accidents before and during the widening of VA-44 was not completed in this study because 48 percent of the accidents in the before period were associated with other construction activities.

2. Vehicles contacted the concrete barriers in 15.4 percent of the accidents reported as occurring during construction.

3. In the 10 reported accidents that involved barriers, 5 of the impacting vehicles remained in the lane adjacent to the barrier after contacting the barrier, 2 infringed on the adjacent lane, and 3 crossed into the adjacent lane with 1 hitting an adjacent vehicle.

4. Of the 10 vehicles that contacted the barriers in reported accidents, 2 rolled over; both struck the barrier at a high impact angle (>15°).

5. There was evidence of 49 vehicle contacts with the barrier for every reported accident in which the barrier was involved.

6. Based on observed tire marks, the end of the barrier flare adjacent to the travel way at the start of the work area was the most-often-hit "point location" in the barrier system.

7. There was an average of 9.7 vehicle involvements with the temporary barriers on VA-44 per million vehicle miles of exposure.

8. There was a definite tendency for motorists to stay out of the barrier lane during construction, but this avoidance decreased as the traffic volume increased.

9. Driver awareness of the construction zone was demonstrated by a 3.2-km/h (2-mile/h) reduction in average speed and a lower variance in lateral placement in the barrier lane.

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REFERENCES

1. F.N. Lisle and B.T. Hargroves. Evaluation of the Performance of Portable Precast Concrete Traffic Barriers. Virginia Highway and Transportation Research Council, Charlottesville, Nov. 1978.
2. E.C. Lokken. Concrete Safety Barrier Design. Transportation Engineering Journal, Feb. 1974, pp. 151-169.
3. Guide for Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, DC, 1977.
4. M.E. Bronstad, L.R. Calcote, and C.E. Kimball. Concrete Median Barrier Research. Southwest Research Institute, San Antonio, March 1976.
5. V.J. Jehu and L.C. Pearson. Impacts of European Cars and a Passenger Coach Against Shaped Concrete Barriers. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. 801, 1977.
6. H.E. Ross, Jr. Impact Performance and a Selection Criterion for Texas Median Barriers. Texas Transportation Institute, Texas A&M Univ., College Station, April 1974.
7. D.M. Parks and others. Vehicular Crash Tests of Unanchored Safety-Shaped Precast Concrete Median Barriers with Pinned End Connections. California Department of Transportation, Sacramento, Aug. 1976.
8. Average Daily Traffic Volumes on Interstate, Arterial, and Primary Routes. Virginia Department of Highways and Transportation, Richmond, 1977.
9. M.E. Bronstad and J.D. Michie. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. NCHRP Rept. 153, 1974.

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Cost-Effectiveness of Guardrail-Bridgerail Transition Improvements: Double W-Beam Versus Decreased Post Spacing

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A study of the performance characteristics of two guardrail-bridgerail transition systems is reported. The two systems were the American Association of Highway and Transportation Officials (AASHTO) stiff-post system and the Nebraska Department of Roads (NDR) "double W-beam" system. The AASHTO system provides larger-sized posts on reduced spacings, whereas the NDR system uses another length of guardrail alongside the face of the existing guardrail with uniform 1.01-m (6-ft 3-in) post spacings. The NDR system eliminates the difficulty of increasing the stiffness of existing systems because of the concrete bridge abutments and/or wing walls that restrict the placement of additional posts on reduced spacings. The objective of the study, which was limited in scope, was to use the BARRIER VII computer program to compare the cost-effectiveness of the two systems. The study considered the effects of impacts with the guardrail transitions by two different-sized automobiles under all possible combinations of speed and angle. The findings show that a reasonable doubt exists as to the cost-effectiveness of the AASHTO system under a wide range of impact conditions. Specifically, the stiff-post system produced more injury-type accidents and resulted in larger exit angles, which increased concern about secondary collisions with other vehicles. The structural adequacy of the guardrail-bridgerail connection in both systems was the single most important design element.

The current accepted practice in designing W-beam approach guardrail is to increase the stiffness of the guardrail by decreasing the post spacing and using larger-sized posts adjacent to a bridge

structure. This design practice was established from the results of a limited number of full-scale crash tests of a large-sized automobile weighing 2041 kg (4500 lb) under extreme impact conditions [97 km/h (60 miles/h) and 25°].

In attempting to upgrade existing systems, the Nebraska Department of Roads (NDR) has often found that it was difficult to increase the stiffness of approach guardrail by adding posts because of the extended concrete foundation footings. As a compromise, NDR has designed a transition section in which the stiffness of the guardrail is increased by installing another length of guardrail alongside the face of the existing guardrail.

The NDR design has been questioned by some engineers because its performance has not been verified by full-scale crash tests. The objective of this study was to conduct a study of limited scope in which computer-model simulations would be used to ascertain the cost-effectiveness of decreasing the post spacing adjacent to a bridge structure in comparison with using the NDR "double W-beam" design. This study takes into consideration the effects of impacts by different-sized

Figure 1. Existing double W-beam approach guardrail.

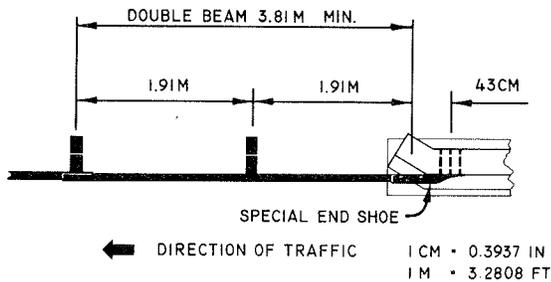
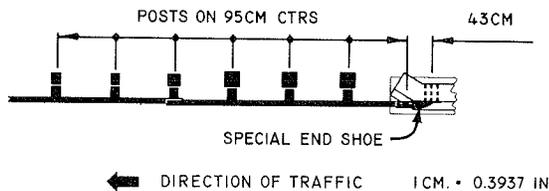


Figure 2. Guardrail improvement alternative.



automobiles with the approach guardrail under all possible speed and angle combinations.

STUDY SITE

The highway used as a study site is classified as a two-lane major arterial rural state highway that will carry a design hourly vehicle volume of 400-750. The traffic lanes are 3.66 m (12 ft) wide, and the paved shoulders are 2.44 m (8 ft) wide.

Details of the type 4 bridge approach guardrail are shown in Figure 1. The double section of guardrail extends over a length of 3.81 m (12.5 ft) and is bolted to 15x20-cm (6x8-in) posts spaced 1.91 m (6.25 ft) on centers. A "special" end shoe is used to connect the guardrail to the concrete bridge parapet.

A plan view of the proposed improvement alternative is shown in Figure 2. This design is very similar to the T1 design of the American Association of State Highway and Transportation Officials (AASHTO) (1, p. 43). The six posts adjacent to the bridge have a reduced post spacing of 1 m (3 ft 1.5 in) on centers, whereas the last three posts have larger [25x25-cm (10x10-in)] timbers.

COMPUTER SIMULATION MODEL

During the past three decades, many highway organizations have relied heavily on experience and judgment in the design of roadside appurtenances; full-scale trial-and-error tests were often conducted to determine the feasibility of these appurtenances. Significant advancements in technology and an increase in safety have evolved from these efforts. However, this type of design approach appears to be insufficient by itself because one or more full-scale tests were required to effectively evaluate the influence of any one variable. Conducting many full-scale tests can be both time-consuming and costly.

Mathematical model simulation provides a rapid and economical method of investigating the many variables involved in a run-off-the-road automobile collision or maneuver. A limited number of full-scale tests can then be conducted to confirm

the simulation results. When supplemented by experience, judgment, and testing, model simulation can be a very helpful tool in achieving efficient and safe designs.

The BARRIER VII program developed by Powell (2,3) was used in this study to determine the dynamic effect of an automobile interacting with a traffic barrier system. The traffic barrier is idealized as a plane framework composed of inelastic one-dimensional elements of a variety of types. The automobile is idealized as a plane rigid body surrounded by a cushion of springs. A large-displacement dynamic structural analysis problem is solved by numerical methods.

The analysis is two-dimensional in the horizontal plane. Out-of-plane effects, which include vertical displacements of both the automobile and the barrier, are not considered. The automobile slides along the barrier, and the effects of normal force, friction forces, and wheel drag forces are considered in determining its motion. The necessary input data consist of the barrier configuration, the properties of the barrier members and the automobile, and the velocity and trajectory of the automobile before impact. Output consists of barrier member forces, barrier deflections, time histories of automobile positions, and velocities and acceleration of the automobile.

A final comment should be made about the BARRIER VII program. It is a two-dimensional program and therefore placed limitations on this study. BARRIER VII cannot predict the roll motion of the vehicle, wheel snagging, or vehicle vaulting, nor will it predict situations in which the vehicle could break through the guardrail. In all BARRIER VII simulations, the rail will return to the elastic state, even though at times there may be sufficient plastic hinges formed so as to create a local mechanism. As far as this study was concerned, all guardrail performance runs were based on successful guardrail tests.

The output results from BARRIER VII that were of direct interest in this study were vehicle accelerations, exit angles, dynamic deflections, forces in the rail member adjacent to the guardrail-parapet connection, and damage to the guardrail system. The results for all impact combinations are given in Table 1.

In determining damage to the guardrail system, the BARRIER VII program will show whether a post has failed. Rail damage can be assessed based on the deflections that occur in the system. The length of rail reported as damaged is in increments of 3.81 m (12.5 ft), since this would be the minimum length of rail that could realistically be replaced.

It was felt that the structural adequacy of the guardrail-parapet connection could be predicted by using the force histories that the BARRIER VII program outputs. If any tensile force in the rail member directly adjacent to the parapet connection reached 355.8 kN (80 000 lbf) and was maintained for a few time steps, it would be assumed to cause the connection to reach yield and then fail.

PROBABILITY OF INJURY

The criteria used in the majority of the evaluations conducted during the past decade on the safety aspects of roadside-hazard improvements were based on levels of vehicle acceleration that would be tolerable to an unrestrained vehicle occupant. One method used to accomplish this task was the definition of a severity index, computed as the ratio of the measured resultant automobile acceleration to the resultant "tolerable" automobile acceleration. An improvement that resulted in a severity-index

Table 1. Results of BARRIER VII simulation.

Speed (km/h)	Angle (°)	Existing System				Stiffened Post			
		Severity Index ^a	Maximum Tensile Force in End Rail (kN)	Vehicle Exit Angle (°)	Probability of Injury ^b	Severity Index ^a	Maximum Tensile Force in End Rail (kN)	Vehicle Exit Angle (°)	Probability of Injury ^b
1021-kg Automobile									
64	10	0.48	0.4	1.5	0.19	0.55	0.4	1.8 ^c	0.22
	15	0.78	0.9	2.9	0.30	0.88	0.4	3.7 ^c	0.35
	20	1.03	5.8	5.4	0.41	1.26	0.9	7.5 ^c	0.50
	25	1.16	36.5	10.9	0.46	1.81	5.8	16.0 ^c	0.72
80	10	0.63	0.4	1.7	0.25	0.76	0.0	1.9	0.30
	15	0.95	7.1	3.4	0.38	1.24	0.9	4.4	0.50
	20	1.26	25.8	6.4	0.50	1.86	2.7	9.0 ^c	0.74
	25	1.27	92.5	7.7	0.51	1.96	51.2	14.6 ^c	0.78
97	10	0.82	1.8	1.3	0.33	1.00	0.9	2.1	0.40
	15	1.28	10.2	3.4	0.51	1.41	0.9	4.4	0.56
	20	1.34	82.7	6.0	0.54	2.21	21.8	9.8	0.88
	25	1.56	136.1	6.0	0.62	2.24	85.8	15.7	0.90
1732-kg Automobile									
64	10	0.45	0.9	3.3	0.18	0.55	1.8	2.9 ^c	0.22
	15	0.67	1.8	7.7	0.27	0.87	16.0	6.4 ^c	0.35
	20	0.72	7.6	10.0	0.29	1.25	74.7	14.5 ^c	0.5
	25	0.84	78.3	14.0	0.34	1.12	123.2	21.6 ^c	0.45
80	10	0.61	0.9	3.3	0.24	0.75	5.3	3.1	0.30
	15	0.87	2.2	6.7	0.35	1.23	53.4	7.2 ^c	0.49
	20	0.83	64.9	10.8	0.32	1.31	92.1	15.4	0.52
	25	1.02	128.1	16.3	0.41	1.33	222.8	25.0 ^c	0.53
97	10	0.80	0.4	2.8	0.32	1.04	9.8	3.2	0.42
	15	0.97	23.1	8.9	0.39	1.43	106.3	8.8	0.57
	20	2.12	101.9	13.1	0.95	1.31	228.2	13.8	0.52
	25	3.09	171.7	21.5	1.00	1.58	321.1	28.0	0.63

Note: 1 km = 0.62 mile; 1 kN = 225 lbf.

^aComputed by using Equation 1. ^bObtained from Post and others (4). ^cSecondary impact.

value of 1 or less was considered to be safe; an improvement resulting in a severity-index value greater than 1 was considered to be unsafe. The work reported here expands the existing technology to include the probability of occurrence of roadside injury-type accidents. An in-depth discussion of a tentative relationship between severity index and the probability of occurrence of injury-type accidents is given by Post and others (4).

CONCEPT OF SEVERITY INDEX

The severity of the collision of an automobile with a traffic barrier is expressed in terms of a severity index (SI). The SI is computed as the ratio of the measured or computed resultant automobile acceleration to the resultant tolerable automobile acceleration that defines an ellipsoidal surface. This ratio can be expressed mathematically as follows (5,6):

$$SI = \frac{G_{total\ auto}/G_{total\ occupant}}{\sqrt{(G_{long}/G_{XL})^2 + (G_{lat}/G_{YL})^2 + (G_{vert}/G_{ZL})^2}} \quad (1)$$

where

- G_{total auto} = resultant automobile acceleration,
- G_{total occupant} = resultant tolerable acceleration,
- G_{long} = automobile acceleration along longitudinal x-axis,
- G_{XL} = tolerable acceleration along x-axis,
- G_{lat} = automobile acceleration along lateral y-axis,
- G_{YL} = tolerable acceleration along y-axis,

G_{vert} = automobile acceleration along vertical z-axis (= 0), and
 G_{ZL} = tolerable acceleration along z-axis.

The computations of SI in this paper are based on accelerations tolerable to an unrestrained occupant, and the automobile accelerations are averaged over a time duration of 50 ms. The relation between SI and injury levels is discussed in detail by Post (4).

RESULTS OF BARRIER VII SIMULATIONS

Automobiles of two sizes were used in this study: the standard-sized vehicle [1732 kg (3820 lb)] and the increasingly popular compact vehicle [1021 kg (2250 lb)]. Three impact speeds--64, 80, and 97 km/h (40, 50, and 60 miles/h) and four impact angles--10°, 15°, 20°, and 25°--were considered.

Point of Vehicle Impact

For all combinations of impact conditions, the initial impact location was 6.68 m (21.9 ft) upstream from the concrete parapet connection. The single impact location was chosen so that there would be adequate time and distance for successful redirection of the automobile under all conditions considered in the study, if indeed redirection were to occur. In the case of the lower speeds and lower impact angles, it would have been possible to move the initial impact location closer to the parapet and still have successful redirection. It was felt that there would be a trade-off in relation to the degree of hazard in these cases when the existing system was compared with the stiff-post system. The stiff-post system would yield significantly higher accelerations, whereas in the existing system it

seemed likely that forces would occur in the rail near the parapet connection that would be large enough to cause failure of the guardrail-parapet connection. In the stiff-post system, higher severities occur because of significant increases in acceleration; in the existing system, higher severities occur because of the increased likelihood of impact with the parapet. It seemed reasonable, therefore, to select a single location of impact for all impact conditions.

Guardrail-Parapet Connection

One point raised in the preceding discussion that needs to be examined is the importance of the guardrail-parapet connection. This connection is required to withstand a 355.8-kN (80 000-lbf) load. As the data given in Table 1 indicate, the tensile forces in the guardrail adjacent to the connection become quite high in some cases. It becomes very important then that the design engineer look carefully at the structural details of the connection. This means not only making sure that there are an adequate number of bolts and a structurally adequate rail for the connection but also ensuring the strength of the parapet that will be receiving these rather large forces. Whenever the connection fails, there is an almost certain chance of vehicle impact with the parapet and a 100 percent probability of injury.

The critical consideration in the design of the guardrail transition is the guardrail-parapet connection. Thus, if upgrading of a transition section were required and if there were any question about the structural adequacy of the guardrail-parapet connection, the stiff-post system would appear to be the best solution because it develops smaller tensile forces in the rail at the connection than does the existing system, which decreases the chance for connection failure.

Characteristics of Vehicle Redirection

The redirection characteristics of the two systems considered in this study were important, since a higher exit angle after impact with a guardrail increases the chance of the vehicle being redirected into traffic in the opposing lane. It was interesting to note that the stiff-post system generates higher exit angles than the existing system. This can be explained by the fact that larger normal forces are developed in the stiff-post system between the guardrail system and the automobile than are developed in the existing system. These larger forces tend to redirect the automobile at a higher yaw rate than that found in the existing system. In some cases, secondary impacts occurred in interactions between the stiff-post system and the vehicle. The guardrail was initially in contact with the front portion of the car, and the large normal forces quickly increased the yawing motion until the rear portion of the vehicle suddenly struck the rail. The cases in which secondary impact occurred are noted in Table 1.

Figure 3 shows a typical comparison between the redirection characteristics of a vehicle interacting with the stiff-post system and one interacting with the existing system. The data were obtained from simulations performed by using the compact automobile at 64 km/h (40 miles/h) and 25°. The point being monitored was the center of gravity of the vehicle.

Deviations in Severity Indices

In two cases, the SI deviated from a consistent pattern. For the 1732-kg (3820-lb) automobile impacting the existing system at 20° and 25°, the SIs were reported as being larger than for the same vehicle impacting the stiff-post system. An apparent explanation for this was that, at these large encroachment angles, the large vehicle had penetrated far enough into the guardrail system that it was "picking up" the contribution of the stiffness of the guardrail-parapet connection more than did the vehicle under the other impact combinations. The maximum accelerations for these two unique cases, then, were occurring at a later time during the interaction with the guardrail system than in the other cases.

IMPACT-CONDITION PROBABILITIES

The impact-condition probabilities were computed by combining distributions of vehicle speeds and encroachment angles. The vehicle-speed distribution was obtained from an analysis of spot-speed data collected on two-lane major arterial rural highway sections by NDR. It was determined that vehicle speeds on these sections were normally distributed with a mean speed of 89.2 km/h (55.4 miles/h) and a standard deviation of ± 7.4 km/h (± 4.6 miles/h). The impact-angle distribution was that reported by Hutchinson and Kennedy for median encroachments (7).

Assuming that these two distributions were completely independent, they were combined. The combined distribution of vehicle speeds and impact angles was then used to compute impact-condition probabilities (8). These probabilities indicate that the most likely impact condition is a speed-angle combination of 89-105 km/h (55-65 miles/h) and $< 7.5^\circ$.

The point mass model presented by Ross (9) was used to determine that some high-speed, high-angle impacts were not possible. However, because of a lack of encroachment data on speed-angle combinations to support this conclusion, it was decided that adjustment of the impact-condition probabilities to account for the apparent impossibility of high-speed, high-angle impacts was not warranted.

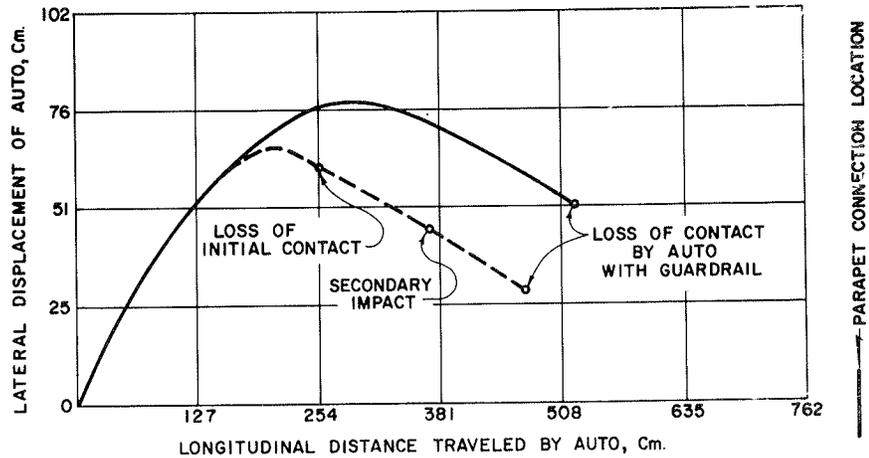
EVALUATION OF IMPROVEMENT ALTERNATIVES

Roadside safety improvement programs must compete with other ongoing highway programs for the limited funds available. The cost-effectiveness method of analysis was used to compare alternatives to making the transition from the semirigid W-beam guardrail to the rigid concrete bridge parapet. The cost-effectiveness method is a management tool intended to provide the highway administrator with a means of evaluating safety-improvement alternatives on a common data base to realize the greatest return on the investment to reduce injury accidents.

Cost-Effectiveness Analysis

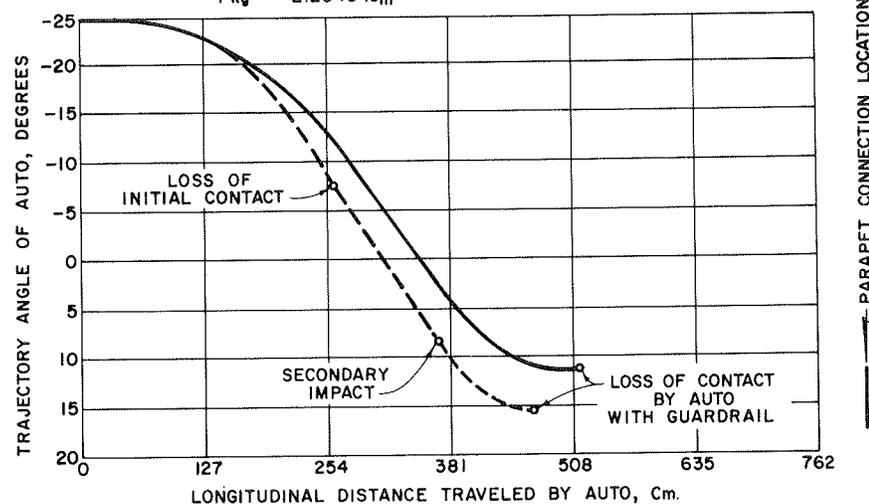
The cost-effectiveness analysis conducted in this study was based on the cost-effectiveness priority approach formulated by Glennon (10) and implemented in Texas for managing roadside safety improvement programs on both non-controlled-access and controlled-access highways (11). The following cost-effectiveness measure was used in this approach: Cost-effectiveness = annualized cost of improvement per unit hazard reduction achieved = cost to eliminate one injury (fatal or nonfatal) accident. The measure of effectiveness was defined

Figure 3. Trajectories of 1021-kg automobile under impact conditions of 64 km/h and 25°.



NOTE: ALL DATA POINTS ARE WITH RESPECT TO AUTO CENTER OF GRAVITY
 1 km/h = .6214 mph
 1 cm = .3937 in
 1 kg = 2.2046 lb_m

— EXISTING SYSTEM
 - - - STIFFENED SYSTEM



as the difference between the hazard indices before and after an improvement, expressed in terms of the number of fatal and nonfatal accidents per year. Thus, in order to apply the cost-effectiveness priority approach in this analysis, it was necessary to compute the hazard index for each improvement and its annualized costs.

Hazard Index

The hazard index was computed for the improvement alternative by using the following equation:

$$H = [E_f(D)(P)(L)/5280] (0.60 H_1 + 0.40 H_2) \quad (2)$$

where

- H = hazard index for each improvement alternative (injury accidents per year);
- E_f = encroachment frequency (7);
- D = directional traffic split = 1/2;
- P = lateral impact probability at some offset distance (7);
- L = effective length of guardrail transition = 7.62 m (25 ft);
- H_1 = hazard-index contribution for impacting ve-

- hicles that weigh more than 1021 kg (2250 lb) (assumed to be 60 percent) = $\sum_{\theta} \sum_v [(SP)(PI)]$;
- H_2 = hazard-index contribution for impacting vehicles that weigh less than 1021 kg (assumed to be 40 percent) = $\sum_{\theta} \sum_v [(SP)(PI)]$;
- SP = impact-condition probability for each combination of speed and angle (8);
- PI = injury-accident probability for each combination of speed and angle severity index for a certain size of vehicle (4);
- θ = vehicle impact angle = 10°, 15°, 20°, and 25°; and
- v = vehicle impact speed = 64, 80, and 97 km/h (40, 50, and 60 miles/h).

Encroachment Frequency

Knowledge of the frequency at which vehicles encroach on the roadside is very limited. Therefore, the encroachment frequency used by Glennon (10) was assumed to be applicable for the purpose of this analysis. The average daily traffic (ADT) for the study site was assumed to be 7500 vehicles/day, which will result in an encroachment frequency of

Table 2. Results of cost-effectiveness evaluation of two bridge guardrail approaches.

Alternative	Lateral Offset Distance (m)	Lateral-Impact Probability	Hazard Index (injury accidents/year)	Capital Costs ^a		Collision Maintenance Costs ^a (\$/year)	Cost-Effectiveness
				Dollars	Dollars per Year		
Existing double W-beam	2.74	0.94	0.0034	—	—	1	—
Reduced post spacing and larger-sized posts	2.74	0.94	0.0041	540	59	1	Not cost-effective ^b

Note: 1 m = 3.3 ft.

^aAnnualized costs were based on a 20-year service life, 9 percent interest rate, and zero salvage value (capital recovery factor = 0.1095).

^bNot cost-effective because $H_{impr} > H_{exist}$.

$$E_f = 1.1 + (0.000415)ADT \quad (3)$$

where $E_f = 4.2$ encroachments/year/mile.

Probability of Lateral Impact

Given that an encroachment has occurred, the probability of a vehicle impacting a roadside obstacle decreases as the distance from the edge of the traveled roadway increases. Probabilities of lateral impact were obtained from the relationship used by Glennon (10).

Collision Maintenance Costs

The collision maintenance cost was computed for the improvement alternative by using the following equation:

$$CM = [E_f(D)(P)(L)/5280] (0.60 CM_1 + 0.40 CM_2) \quad (4)$$

where

- CM = annualized collision maintenance cost,
- CM₁ = annualized collision maintenance cost contribution for vehicles that weigh more than 1021 kg (2250 lb) = $\sum \sum [(SP)(CS)]$,
- CM₂ = annualized collision maintenance cost contribution for vehicles that weigh less than 1021 kg = $\sum \sum [(SP)(CS)]$, and
- CS = annualized collision maintenance cost for each combination of impact speed and angle.

All of the remaining terms in Equation 4 have been previously defined in Equation 2.

Evaluation

As defined earlier, cost-effectiveness was described as the annualized cost of an improvement per unit of hazard reduction achieved. The measure of effectiveness was defined as the difference between the hazard indices before and after an improvement. Effectiveness can be computed by using the following equation:

$$E = H_{exist} - H_{impr} \quad (5)$$

where

- E = effectiveness (hazard reduction),
- H_{exist} = hazard index of existing system, and
- H_{impr} = hazard index of stiffened system.

The annualized improvement costs consider both capital costs and collision maintenance costs. Normal maintenance costs were assumed to be small and were neglected. The costs can be computed from the following equation:

$$C = CI_{impr} + CM_{impr} - CM_{exist} \quad (6)$$

where

- C = annualized cost of improvement,
- CI_{impr} = annualized capital cost of improvement,
- CM_{impr} = annualized collision maintenance cost of improvement, and
- CM_{exist} = annualized collision maintenance cost of existing system.

The results of the cost-effectiveness evaluation are summarized in Table 2. As Equation 5 indicates, the improvement alternative was not cost-effective because there was no reduction in the hazard index; in fact, the stiff-post system not only did not exhibit a reduction in the hazard index but even indicated a slight increase. This indicates the probability of a higher incidence of injury accidents.

SUMMARY AND CONCLUSIONS

This study on the cost-effectiveness of guardrail-bridgerail transition areas was conducted by the University of Nebraska in cooperation with the Nebraska Department of Roads.

The NDR requested the research study in order to gain more insight into the performance characteristics of two guardrail transition systems: the AASHTO stiff-post system and the existing NDR double-beam system. The stiff-post system increases the stiffness of the guardrail by reducing the post spacings, whereas the NDR system installs another length of guardrail alongside the face of the guardrail and uses uniform post spacings of 1.91 m (6.25 ft) on centers.

The NDR system has been questioned by some engineers because its performance has not been verified by full-scale crash tests. The objective of this study was to use the BARRIER VII computer program to ascertain the cost-effectiveness of the stiff-post system in comparison with the existing NDR double-beam system. The study considered the effects of impacts of the guardrail transition area by two vehicles of different sizes under all possible combinations of impact speed and angle. The significant findings were as follows:

1. The stiff-post system was not cost-effective because it produced more injury-type accidents.
2. The stiff-post system resulted in larger exit angles, thereby increasing the possibility of secondary collisions with other vehicles.
3. Since the stiff-post system produces lower tensile forces in the guardrail, it would perform more effectively than the NDR system if the connection between the guardrail and bridgerail could not be made to meet minimum structural

requirements. It is not, however, recommended that decreased post spacing be substituted for a structurally adequate connection because the tension capability in the guardrail is the single most important design element.

The results of this study will be used in the formulation of NDR policy on guardrail design, installation, and maintenance. The methodology and procedures developed will be included in NDR design procedures and will increase the ability of the department to evaluate new systems through the cost-effectiveness calculations based on BARRIER VII simulations.

Based on the results of this study, there has been shown to be reasonable doubt as to the cost-effectiveness of the stiff-post system under a wide range of impact conditions. A more detailed examination of the total effectiveness of the stiff-post system is needed. Further research should be conducted to compare the performance characteristics of the two systems by means of full-scale testing and computer model simulations.

REFERENCES

1. Guide for Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, DC, 1977.
2. G. H. Powell. BARRIER VII: A Computer Program for Evaluation of Automobile Barrier Systems. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-RD-73-51, April 1973.
3. G. H. Powell. Computer Evaluation of Automobile Barrier Systems. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-RD-73-73, Aug. 1970.
4. E. R. Post and others. Cost-Effectiveness of Driveway Slope Improvements. TRB, Transportation Research Record 685, 1979, pp. 14-19.
5. H. E. Ross and E. R. Post. Criteria for Guardrail Need and Location on Embankments: Volume 1--Development of Criteria. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 140-4, April 1972.
6. G. D. Weaver, E. L. Marquis, and R. M. Olson. Selection of Safe Roadside Cross Sections. NCHRP, Rept. 158, 1975.
7. J. W. Hutchinson and T. W. Kennedy. Medians of Divided Highways: Frequency and Nature of Vehicle Encroachments. Univ. of Illinois, Urbana-Champaign, Engineering Experiment Station Bull. 487, 1966.
8. E. R. Post and others. Guardrail Utilization: Cost-Effectiveness Computer Program to Analyze W-Beam Guardrail on Fill Slopes. Civil Engineering Department, Univ. of Nebraska-Lincoln, Res. Rept. TRP-004-78, Vols. 1 and 2, 1978.
9. H. E. Ross, Jr. Impact Performance and a Selection Criterion for Texas Median Barriers. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 140-8, April 1974.
10. J. C. Glennon. Roadside Safety Improvement Programs on Freeways: A Cost-Effectiveness Priority Approach. NCHRP, Rept. 148, 1974.
11. G. D. Weaver, D. L. Woods, and E. R. Post. Cost-Effectiveness Analysis of Roadside Safety Improvements. TRB, Transportation Research Record 543, 1975, pp. 1-15.

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Conflicts Between Vehicle Traffic and Utility Facilities

DON H. JONES

Conflicts between vehicle traffic and utility facilities are examined in a before-and-after study of a heavily used section of a four-lane major arterial in Knoxville, Tennessee. At the beginning of the study, many utility facilities were located in the roadway and at the back of the curb in close proximity to moving traffic. Some of the underground facilities in the roadway were relocated under or to the far side of the sidewalk, and all utility poles were relocated to the far side of the sidewalk and to one side of the street. Prior to the relocation, there was a high incidence of vehicle collisions with aboveground utility facilities and considerable traffic delays caused by maintenance of underground facilities on this section of highway. In the five years since the relocation, no collisions with utility facilities have been reported. Other factors examined include accidents involving the failure of pavement cuts made when underground facilities were repaired or installed, delays resulting from utility operations, and user costs resulting from traffic delays and accidents in which utility facilities are involved. The study shows not only that serious conflicts exist in certain locations but also that considerable improvement can be achieved through cooperation in planning, design, and operations between highway agencies and utility agencies.

Utility facilities are a part of the transportation system, moving important products, such as water, gas, electricity, and communications, from sources to destinations. Problems occur when utility facilities are brought into direct contact with other transportation facilities, such as the highway and street network. When highways and utilities are merged, conflicts arise that are not easily or economically resolved.

Highways and utilities serve two totally different purposes. The highway and street system is for the purpose of moving people (as well as freight and goods) at high rates of speed and in heavy vehicles. Utility facilities move commodities--water, electricity, sewage, or communications--that are different in nature and for a different purpose.

Utilities have always shared rights-of-way with highways. This need, long recognized by law, varies only slightly from state to state with regard to occupancy and installation rights. Rights and needs to share rights-of-way are generally and mutually recognized; the difficulty occurs when service to one mode is interrupted or interfered with by the opposite mode. Furthermore, other factors, such as aesthetics and crowding, tend to aggravate the perceived adverse relation when conditions such as interrupted service develop.

This paper discusses the problems encountered in conflicts between highways and utilities and their possible solutions. The problems are presented from the following viewpoints:

1. Utilities are generally installed on existing highway rights-of-way.
2. The health, safety, and welfare of the

motoring public are of paramount concern.

3. The problems involved, whether perceived, actual, or a combination of both, are solvable.

4. Roadways and utilities are maintained by two different groups that are interested in serving their own personal interests.

The data and information presented here are based on a study begun in Knoxville, Tennessee, in 1963 (1). An extensive report dealing with the conflicts between utility facilities and highway traffic was published in 1969. A major relocation of the communication lines was begun in 1968. This consisted of relocating the overhead lines and some underground lines in the middle of the street to an area under the sidewalk. The overhead power lines were relocated in 1974. These changes brought about a substantial reduction in conflicts with vehicle traffic and an improvement in the appearance of the community. The study included a citywide sample of accidents involving utility facilities and focused on a heavily used major arterial that seemed to have an unusually high accident rate, particularly of accidents involving utility poles.

The city of Knoxville is reasonably representative and has a population of about 150 000, which is continuing to increase rapidly. The city has a good mixture of diverse occupations, including a large industrial commercial base. It is a hub city for banking and wholesale-retail businesses and is surrounded by a large agricultural base. Knoxville supports a large community of science- and research-oriented citizens and also has a university campus with an enrollment that is among the 20 largest in the country.

UTILITY LOCATION POLICY

The general policy for sharing rights-of-way has been for utilities to locate as near the roadway as possible. In urban areas, underground facilities are usually located under the pavement and overhead lines are at the back of the curb or very near the edge of the pavement. Since highways and streets are usually designed and built to serve only vehicle traffic, the utility is viewed as a trespasser, and any installation or maintenance has the potential for disrupting traffic. Utilities usually view their work as being necessary and of short duration and see the problem as being one of lack of patience on the part of highway users. Although there has been a marked improvement in recent years in cooperation between highway users and utility groups, historically there has been a lack of understanding by both groups of each other's needs and problems. Relations are strained by such things as vehicles knocking down utility poles and fire hydrants and utility companies blocking traffic for maintenance and installation purposes. The utility group views as too harsh the requirements and regulations governing backfill procedures, signing, and flagging, and the highway-street group views the utilities as trying to short-circuit the standard operating procedures and sound engineering practices normally used in highway operations.

Group meetings to discuss the reasons for certain specifications or mutual concerns are not the standard mode of operation. Although meetings do occur occasionally and some are productive, most are scheduled as a result of a prospective change in the utility's plant or in the roadway, and the meeting usually takes place only between the utility representative and the highway utility officer. Many cities lack a formal and separate street organization. Many diverse engineering operations are placed under a public works administrator who is

responsible for several areas, including streets and sanitation and storm sewers. Other utilities, such as gas, electricity, and sometimes water, are separate entities in the management scheme. As a result, there is little, if any, coordination of street and utility operations.

NATURE OF THE CONFLICTS

The fact that any conflict occurs between highway vehicles and utility facilities is sufficient to warrant an examination of the causes, kinds, and extent of the conflicts and policies on utility installation and maintenance. Before this study began, however, there was an unproved theory that a large number of accidents involved utility facilities. As a result of that theory, an effort was begun by highway department representatives in that region of Tennessee where the study was conducted to have all relocated utility facilities placed at the back of the sidewalk or along the right-of-way lines as far away from the traveled lanes as possible. Unusually strong resistance was encountered--partly because of typical resistance to change, partly because of economic reasons, and partly because of utility company concern that they would eventually be forced from highway rights-of-way. This resistance might not have been so great had documentation of accidents and their causes been a part of the evaluation and planning process. From these rather difficult beginnings, the study discussed here was begun.

An examination of vehicle accident records for the specified study section for a number of years revealed many accidents involving utility facilities (1). The following table gives the number of accidents for each type of utility facility over a six-year period:

Type of Facility Involved	Number of Accidents
Utility poles	103
Fire plugs	8
Bus benches	1
Mailboxes	7
Signs	6
Guy wires	2
Telephone booths	2

Table 1 gives data on utility-facility accidents for four locations over a six-month period. The data indicate that a disproportionately high number of these conflicts involved utility poles and that a disproportionately high number of personal injuries and fatalities occurred in collisions with utility poles (see Figure 1). A survey of newspaper articles also revealed that the conflicts were not just related to recent times and/or high-volume highways and streets (2, p. 1). One should also note that the selected study site had at least a 30-year history of vehicle accidents involving utility facilities, especially poles.

Observations during the long study period also substantiated suspected conflicts of a different nature, particularly those involving pavement cuts (see Figure 2). Pavement cuts in urban areas are rather common. During the study period, numerous failures of the restored cuts were noted. A search of accident records did not indicate a high incidence of accidents involving the failure of repaired pavement cuts. However, through a careful search of newspaper articles and interviews with police traffic officers, the reason for the lack of data was determined: Very few accidents of this type were formally reported.

Accidents involving burst tires, wheels knocked

Table 1. Comparison of accidents involving utility facilities for six-month period at four Knoxville locations.

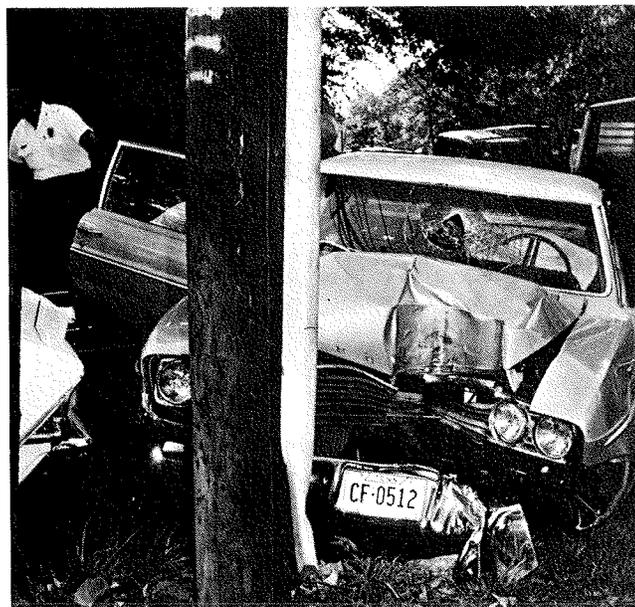
Location	Total Number of Accidents	Accidents Involving Utility Facilities			Number of Drunk Drivers Involved
		Number	Personal Injury	Fatal	
Kingston Pike	66	17	8	2	1
Broadway	325	9 ^a	5	0	0
Chapman Highway	236	7 ^b	5	0	1
Clinton Highway	122	4 ^c	2	0	1

^a Involved utility facilities located at the back of the curb.

^b Involved utility poles located at the back of the curb.

^c All vehicles struck utility poles in the median area.

Figure 1. Low-speed head-on collision with a utility pole that resulted in personal injuries.



out of alignment, or ruined shocks were rarely reported. These types of accidents usually did not involve personal injury. Most were reported by telephone. The caller would give the location of the pavement failure and indicate only that there was a hole in the pavement. A call to the utility company involved would usually result in the quick dispatch of a truck and crew to place a saw horse with a flashing lantern as a warning to motorists. Repairs were begun as soon as possible.

Such problems do not occur often at a given location and, after one or two return trips for repairs, the serious, hazardous failures no longer seem to occur. Yet, as Figure 2 indicates, poorly repaired pavement cuts sometimes create a serious problem that can result in serious accidents involving personal injury (3, p. B-1). It may be better in the long run to install all underground facilities in locations other than directly under the traffic lanes. On the other hand, proper methods of backfilling and pavement patching would eliminate this type of problem.

Probably the most visible encounter between utilities and the traveling public involves delays resulting from the maintenance of utility facilities (see Figure 3). Accidents that occur in this situation are usually slow-moving, rear-end

Figure 2. Failure of pavement over new installation of a sanitary sewer line that resulted in several accidents involving property damage and minor personal injury.



Figure 3. Traffic queue caused by maintenance of a utility facility.



collisions or occasionally sideswipe or lane-change collisions. Accident reports rarely note a utility maintenance operation as a contributing factor (4, p. 10). The most obvious result of these delays is economic, because of lost time and higher operating cost.

The nature of the conflict between vehicles and utility facilities, as noted in the University of Tennessee study (1), generally consists of collisions with such objects as poles, fire hydrants, and phone booths; vehicles running into depressions caused by collapsing patches of pavement; and delays caused by utility maintenance, repair, and installation. A literature search provided little information to support the study findings, since few research projects in this area had been conducted at that time. However, a few research studies that support the findings have been conducted since 1970, most relating to utility poles (5-7). Some of the reports cited quote statistics that indicate that a serious problem does indeed exist.

EVALUATION OF CONFLICTS

The original study cited here examined a 3.25-km (2-mile) section of a major arterial that was

Figure 4. Typical collision diagram for a location that has a high incidence of conflicts between utility poles and vehicles.

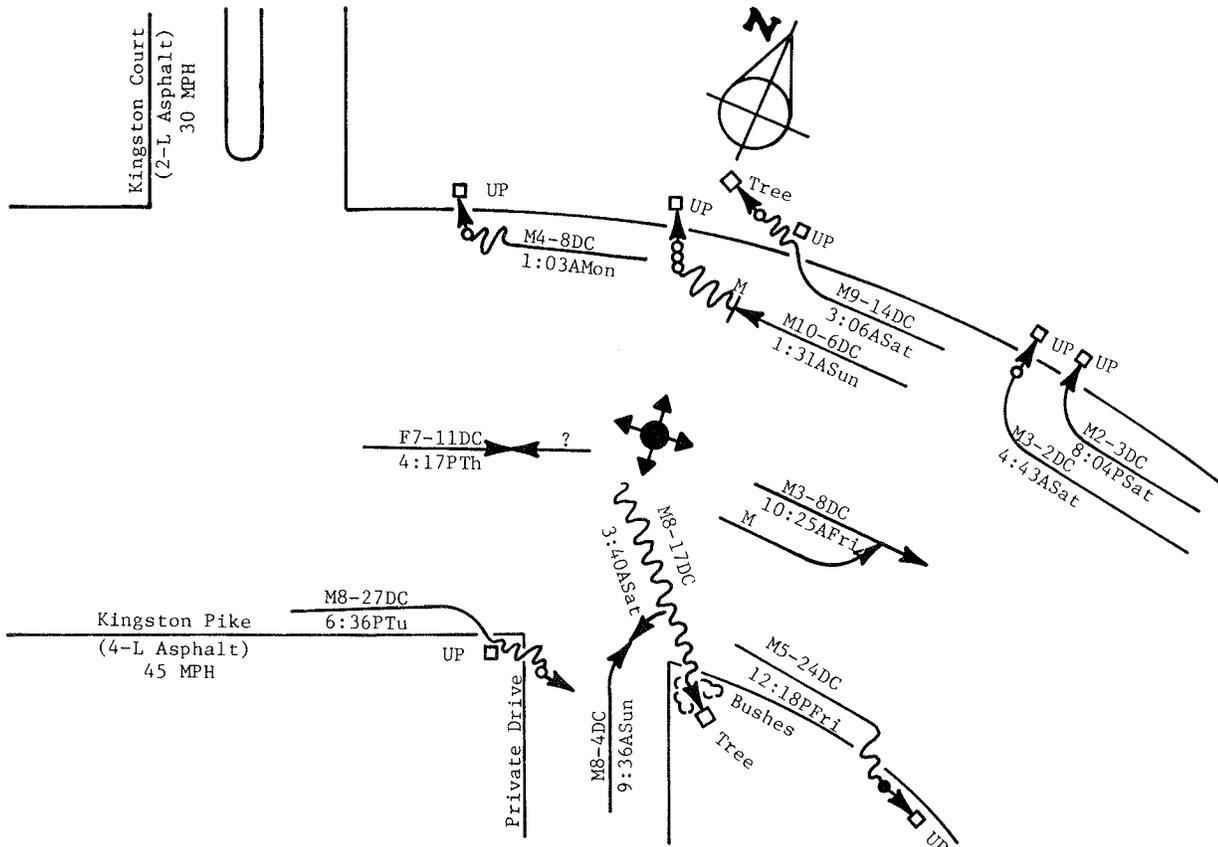
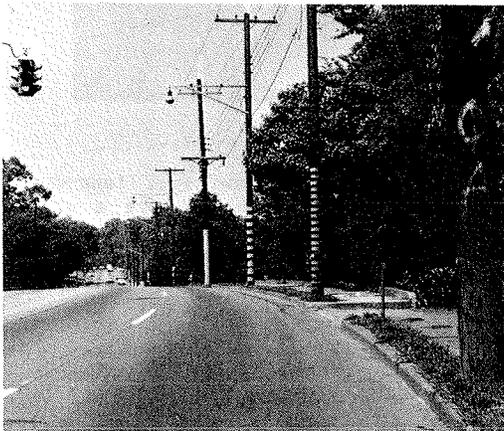


Figure 5. Utility poles located too close to the roadway on a superelevated curve and painted in an attempt to reduce accidents.



purported to have a high incidence of accidents involving utility facilities. The collision diagrams prepared from police accident reports for that highway section bore out the suspicions (1). Figure 4 and the following table illustrate the accident data for the period from January 18 to October 14, 1968:

Category	Day	Night	Total
Total accidents	5	6	11
Fatalities	1	0	1
Injuries	1	6	7
Property damage only	3	2	5
Utilities involved	2	5	7

An analysis of the accident data consistently pointed to one thing--that utility facilities were located too close to moving traffic. Utility poles, guy wires, and fire hydrants were usually placed at the back of curbs and occasionally in the street. They were often placed only 15 cm (6 in) from the pavement edge on the outside of nonsuperelevated curves (see Figure 5). Of course, those utility facilities located under the pavement not only disrupted the flow of traffic when they were repaired but also presented a traffic hazard when improperly compacted backfill failed under pavement patches.

The utility companies were somewhat aware of the conflicts between vehicles and their facilities. In some cases, where particular poles were often struck, extra precautions had been taken to protect the facilities with heavy concrete barriers (see Figure 6), steel posts concreted in the ground, or other devices. Many of these barriers were placed along routes where posted speeds were 45 miles/h or higher. A utility pole is a fixed object that will break when struck with enough momentum, although usually not before destroying the striking vehicle and its contents. A concrete barrier is a fixed object that is almost indestructible (Figure 6).

In analyzing the accident data, it was difficult to determine whether the struck utility facility was a primary or secondary accident factor. Herein lies the basis for the argument that the accidents were the fault of the drivers and not the utility facility. In accidents that involve pavement failures over underground utility facilities, there is little doubt as to who is at fault (although responsibility, liability, and negligence may be factors to be decided through legal proceedings). But collisions that involve roadside objects such as

Figure 6. Protection of utility poles with heavy concrete encasements that created a much worse hazard for vehicles.



poles and fire hydrants are another matter. It can be claimed that the poles blocked the view or interfered with sight distance.

The evidence seems to indicate that struck poles are secondary factors in many vehicle accidents. Some typical examples are a pole being struck after control of the vehicle is lost on wet pavement or a pole being hit by a vehicle that is forced off the road or out of a lane by another vehicle that is attempting a lane change. The utility pole is considered a primary factor when, for example, the operator of a vehicle that pulled out from a driveway or a side street into the path of an oncoming vehicle reports that the view was blocked by utility poles at the edge of the pavement or the running lane. In one incident, a pole was knocked down by one vehicle and immediately struck by another (1).

A great deal of disagreement may arise over the causes of accidents occurring in a queue that is the result of a utility operation such as that shown in Figure 3. Direct collisions with utility barricades, with utility vehicles parked at the scene, or with parts of the facility--e.g., manhole covers--may constitute primary or secondary involvement of the utility facility. These types of collisions are also well documented in the University of Tennessee study (1).

Accidents that occur well back in a queue are not so easily reconciled. Would they have occurred if the queue were not present or if the utility operation were not present? However, the only concern here is that the accident occurred in a queue that resulted from a utility operation and interfered with or disrupted the movement of traffic.

The argument over who creates the problem seems endless. The utility company can often and justly argue that utility facilities, such as poles, are frequently destroyed by vehicles, so that large

areas are left without vital power and communications. The counter argument may be that the highway has paramount rights and utilities should be placed somewhere else. And so the costly problem goes on.

COST STUDIES

Deriving the cost involved in conflicts between highways and utilities is not easy. The Texas Department of Highways and others (8) have developed procedures for inventorying roadside hazards and for developing safety improvement alternatives. Wright and Robertson (6) developed procedures for determining probable sites with high rates of accidents that involve roadside hazards and for setting priorities for modifications. Jones (1) developed some procedures for costing out accidents and delays resulting from vehicle-utility conflicts. The procedures are cumbersome and fragmented and often require on-site studies. On-site analysis can feasibly be conducted by the utility agency involved, but utilities seem reluctant to conduct such studies. Yet good bases are available for developing excellent cost estimates.

For delays caused by queuing around utility operations, one must begin with traffic-volume studies. If these are not already available, they must be conducted to derive average daily traffic, peak-hour volume, weekday volume, etc. Cooperating utility companies can provide dates, the nature of the operations, and the costs for crews, equipment, and materials.

In studies of the cost of delays, some typical data must be gathered--e.g., the average number of stops per vehicle while the vehicle is in the queue and before it passes the point of interference. The total effect on all directions of traffic movement must be determined. Some idea of the average number of collisions between vehicles and utility-related equipment must also be developed. If on-site analyses are made, all of the factors are included as data collected at the site; however, the dependability of the data must be considered and appropriate factors included, if necessary.

Again, most utility companies can provide actual cost data for each operation, including fringe benefits and overhead cost. For accidents involving property damage, personal injury, and fatalities, cost data are published intermittently by the National Safety Council, the Federal Highway Administration (FHWA), the National Highway Traffic Safety Administration (9), and others. These data can be used in costing accidents. Otherwise, insurance claims, court awards, and out-of-court settlements would have to be evaluated, which is not an easy task.

The cost of accidents involving utility facilities was determined for a nine-month period in 1968 over the 3.25-km (2-mile) study section by using cost data and appropriate interest rates. This cost amounted to about \$136 000. Today, the cost of the same accidents would probably approach \$500 000. Two incidents that involved substantial delays were costed out at an average of \$1655/incident in highway user costs (1). These were substantial costs for 1968. In developing such cost analyses, attention must be given to the category in which the costs are placed. Rarely are benefits or utility enhancements involved, except for the possible extension of service life for the one pole, hydrant, or other unit involved. The questions of when to apply interest rates and what rates to apply also require attention, especially today.

Figure 7. Study area after major relocation of utility poles.

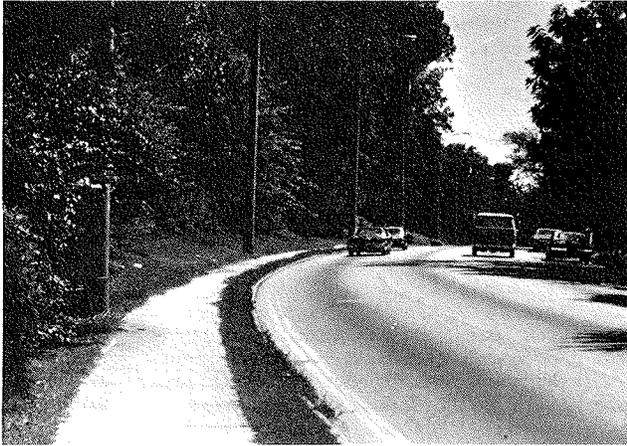


Table 2. Comparison of personal injuries and fatalities before and after completion of major 1974 relocation of utility poles.

Year	Average Daily Traffic	Personal Injuries	Fatalities
1963	27 470	8	0
1964	24 940	8	1
1965	20 900	3	1
1966	20 860	18	0
1967	19 550	11	0
1968	22 143	16	2
1974	25 230		
Before		4	1
After		0	0
1975	28 190	0	0
1976	28 650	0	0
1977	25 570	0	0
1978	26 600	0	0
1979 ^a	32 090	0	0

^aThrough August.

REMEDIAL MEASURES

The study completed in 1968 pointed out that, in the first nine months of that year, 16 persons were injured and 2 were killed when vehicles struck utility poles over the 3.25-km highway section studied. The study also noted that this section of highway had probably the highest rate of accidents involving utility facilities in the city of Knoxville. The study also pointed out that there were other sections of highways within the urban area where the rates of such accidents approached this one and that special attention should be given to particular locations. The study received widespread publicity in the local news media. City officials, including the city traffic engineer, and officials of the utility agencies conducted a number of discussions and planning sessions. Estimates for some recommended adjustments to the utility facilities, such as the cost of placing overhead lines underground, were developed. Many other alternatives were also examined in the study, such as locating utility facilities in alleys and putting utility vaults under sidewalks and special utility strips.

In the period from 1968 to the present, action was taken in the four areas described below.

Relocation of Overhead Lines

The owners of overhead communication lines assigned

first priority to the highway section under study and, slipping some other proposed improvements back in their program, began immediately to relocate overhead lines underground and to remove some troublesome facilities from underground locations in the middle of the street to an area under the sidewalk or to the far side of the sidewalk. This work was begun in 1968 and completed in about 1970. It resulted in a substantial improvement in the costly disruptions of traffic that had been caused by frequent maintenance of such communication facilities. It did not, however, result in the removal of a single utility pole, the only utility elements involved in vehicle collisions in which personal injuries and fatalities were reported.

Adjustments to Utility Facilities

Some welcome financial relief came in the form of the Traffic Operations to Increase Capacity and Safety (TOPICS) program. No TOPICS money was used in the 3.25-km study area; however, in other locations chosen for redesign under the TOPICS program, utility facilities were a major consideration and were relocated if they were considered to present a hazard. But many problems emerged in the TOPICS projects where substantial adjustments to utility facilities were considered necessary. The utility agencies were not compensated for adjustments to their facilities if they were on public rights-of-way. The utility agencies felt, in some ways, that they had been "had" again. But more effort was made under this program to point out and explain the necessity of relocating poles from corners, from just behind the curb, and from the middle of the highway. Another problem that arose under the TOPICS program was the virtual impossibility of attributing specific portions of improvements in traffic flow and accident rates to the utility adjustments because so many other elements at the selected sites were changed. However, improvements in traffic flow and accident rates at each site where utility facilities were adjusted were so great that it was felt that some contributions had been made by these adjustments.

Relocation of All Utility Poles

Relocation of all utility poles on the 3.25-km study section did not occur for some time. By 1971, the recommended relocation of utility poles appeared to be a dead issue. But a new city mayor was elected that year, and he immediately resurrected the study and mandated the relocation of all utility poles in the 3.25-km highway section as part of a beautification and safety program. The final approved plan included the relocation of all poles to one side of the street, to an area behind the sidewalk, about 1.83 m (6 ft) from the permanent edge. Large steel poles were used. Considerable thought was given to blending the poles and the overhead lines into the surrounding environment of well-landscaped estates, shrubs, and walls. The spacing of the new poles was to be increased as much as possible over the maximum spacing of 45.7 m (150 ft) for the existing poles. Special attention was to be given to sites that were considered to be especially hazardous.

The city agreed to bear most of the cost of the relocation, which was completed in 1974 (see Figure 7). Ironically, before the relocation was completed and the old wooden poles were removed, four personal injuries and one fatality occurred. The good news, however, is that the total accident rate dropped substantially after the relocation was completed. No more utility poles have been struck to date, even

though average daily traffic has almost doubled. Table 2 gives a comparison of accidents involving utility facilities and resulting in personal injuries and fatalities for the years 1963-1968 and 1974 through August 1979. The data are taken from 1963-1968 traffic records of the Knoxville Police Department and from 1974-1979 accident records in the Tennessee Department of Transportation (DOT) computer bank.

Other Programs

Other programs that have helped are the program of high-accident-site studies and the Governor's Highway Safety Program. Generally, no funds have been available to assist utility agencies in relocating or adjusting facilities that were located on public property. But these programs have created an awareness of hazardous conditions and have pinpointed locations where accidents or serious injuries and fatalities consistently occur.

Most utility agencies respond favorably to good documentation and supporting evidence, since they share the concerns of the highway agencies for highway and community safety. FHWA has considered the possibility of reimbursing utility agencies for the adjustments and relocations required to improve conditions at sites where accident rates are high. Under current law, however, FHWA can only participate in such activities to the extent to which the states participate. As a result, in many states reimbursement is approved only if the utility facilities are located on private property, in which case 100 percent of the cost involved is reimbursed to the utility. In many states, and especially in Tennessee, if the facilities are located on public property, the utility agency must bear the total cost except on Interstate highway projects (10).

CONCLUSIONS

During the past 15 years, there has been considerable evidence that conflicts do occur between vehicle traffic and utility facilities when the two are in very close proximity. However, utility agencies and highway groups (traffic engineers, utility relocation personnel, and safety program administrators) still seem reluctant to discuss openly and candidly such issues as legal requirements, financial constraints, hazardous or potentially hazardous conditions, and cooperation through joint planning.

Many roadway sites are made extremely hazardous by the location of utility facilities. Minor adjustments or relocations can often solve the problem; in some situations, however, such as that presented in this paper, a costly and extensive relocation may be the only practicable solution. Through cooperative effort and financial participation, adequate solutions can be found.

Today, a great deal of emphasis is placed on the cost-effectiveness of almost any activity. Although procedures are available for determining the cost-effectiveness of utility adjustments or relocations to alleviate vehicle-utility conflicts, this should not always be the sole criterion. When the potential for personal injuries and fatalities exists, corrective action must be taken. Customer service is not going to be improved by a utility agency adjusting its facilities to alleviate a traffic problem, and benefits to the highway user are not going to accrue to the utility. But it is necessary and beneficial to the community for the utility to pursue every avenue of potential financial assistance.

There is a need for closer interaction between

agencies, beginning at the local planning and development stage and continuing through regular group meetings, so that a feeling of mutual cooperation can be developed. Both utility agencies and highway agencies have the potential to help each other, but this potential is not always fully realized.

There seems to be more awareness today of potential conflicts between vehicles and utility facilities and operations. There also seems to be more cooperation between agencies to improve potentially hazardous conditions. This is probably the result of continuing research efforts, publications, both favorable and unfavorable publicity through the news media, numerous safety programs, and more and more lawsuits challenging both utility agencies and highway agencies. No matter what the reasons, the increased cooperation is welcome.

It is strongly recommended that highway-utility liaison groups be established to develop better communications. Perhaps representatives of the utilities could be appointed to the highway safety committees that are active in many states in the review of hazardous locations. Utility companies could be provided copies of accident reports for cases in which utility facilities are involved. Such programs could result in greater cooperation and assist in locating funding for some of the changes considered necessary for safe highways and unhampered operation of utilities.

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The views and opinions expressed here are solely my own, and I am responsible for the content and the accuracy of the data presented.

REFERENCES

1. D.H. Jones. Traffic Accidents, Traffic Delays, and Highway Capacity as Related to the Installation of Utility Facilities Within Highway Rights-of-Way. Univ. of Tennessee, Knoxville, thesis, 1969.
2. U.T. Instructor's Son Is Killed in Accident. Knoxville News-Sentinel, Sept. 29, 1940.
3. Lyons View Chuckhole Takes Toll. Knoxville News-Sentinel, Nov. 18, 1968.
4. Pike Collision Jams Traffic. Knoxville News-Sentinel, Jan. 11, 1968.
5. L. Graf, J.V. Boos, and J.A. Wentworth. Single-Vehicle Accidents Involving Utility Poles. TRB, Transportation Research Record 571, 1976, pp. 36-43.
6. P.H. Wright and L.S. Robertson. Priorities for

- Roadside Hazard Modification: A Study of 300 Fatal Roadside-Object Crashes. Insurance Institute for Highway Safety, Washington, DC, March 1976.
7. J.J. Labra. Reports to TRB Committee A2A07, Committee on Utilities, Regarding Breakaway Utility Poles. Southwest Research Institute, San Antonio, TX, 1977 and 1978.
 8. G.D. Weaver, E.R. Post, and D.L. Woods. Procedures Manual for Roadside Hazard Inventory and Safety Improvement Alternatives. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. TTI-2-8-72-11-1, Jan. 1979.
 9. B.M. Faigin. 1975 Societal Costs of Motor Vehicle Accidents. National Highway Traffic Safety Administration, U.S. Department of Transportation, Dec. 1976.

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