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Application of Safety Appurtenances

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A Truck-Mounted Portable Maintenance Barrier

W. LYNN BEASON and HAYES E. ROSS, Jr.

ABSTRACT

A truck-mounted portable maintenance barrier is described. The barrier is designed to provide a reasonable degree of positive protection in short-duration work zones where it is not practical to use conventional barriers. It consists of a steel barrier section supported between two maintenance trucks. The barrier section is towed to the work zone on a specially fabricated transport dolly. On-site deployment can be accomplished by a crew of two men in 15 min or less. The barrier is highly maneuverable in the deployed configuration so that it can be easily repositioned as the work progresses. Three full-scale crash tests were conducted to demonstrate the impact performance of the barrier.

There is an increasing number of high-volume, multilane expressways where it is not practical to stop traffic across all lanes during single-lane maintenance operations. The current approach is to close only the lane under repair and redirect traffic into adjacent lanes. The problem with this approach is that the work zone is adjacent to a traffic lane, which exposes the workers to the risk of being struck by an errant vehicle. This situation is particularly hazardous during times of heavy traffic flow when the loss of even one lane of traffic can create severe local traffic congestion. There is an urgent need to increase the protection of workers in this situation.

In some instances, the nature of the maintenance is such that the work zone is occupied for weeks or months. In such cases, it is possible to install portable concrete barriers (1). In other instances, the time required to accomplish the maintenance is such that it would take substantially more time to deploy portable concrete barriers than it does to perform the maintenance. In addition, the widths of the portable concrete barriers are such that they encroach into either the work zone or the adjacent traffic lane.

Research discussed here was directed toward development of a truck-mounted portable maintenance barrier for use in short-term highway maintenance. The portable maintenance barrier provides a reasonable degree of protection for the workers; it can be easily deployed and, once deployed, it remains highly maneuverable.

Discussions of the concept of the portable maintenance barrier, the performance criteria, and results of both strength and maneuverability tests are presented.

CONCEPT

It is common practice in highway maintenance to station maintenance vehicles in the work-zone lane. This is done to provide ready access to supplies and to prevent unnecessary blockage of additional traffic lanes. A side benefit of this practice is that the maintenance vehicles afford the workers protection from in-lane impacts. The purpose of the research reported here is to develop a barrier system that enhances the protection afforded by in-lane maintenance vehicles. The portable maintenance barrier developed is intended for use in short-term (less than 1 day) maintenance operations such as guardrail replacement, pothole repair, and so on. Major emphasis was placed on developing a barrier that is easily transported and deployed.

The truck-mounted portable maintenance barrier consists of a steel barrier section supported between two trucks as shown in Figure 1. Figure 2 shows planned deployments of the portable maintenance barrier in a work zone. The support trucks provide protection against in-lane impacts, and the barrier section provides protection against lateral impacts. The major components of the portable maintenance barrier are the support trucks, the hitch assemblies, the support members, the barrier section, and the transport dolly (Figure 3). Each of these components is discussed in the following paragraphs.

The support trucks used in the prototype are 5-yd^3 (3.8-m^3) dump trucks. The only modification to the trucks consisted of the installation of frame plates to increase the in-plane stiffnesses of the truck frames so that the support trucks can withstand the design impact without damage. The frame plate is a steel plate 1/2 in. (1.27 cm) thick mounted between the frame members of the truck and the dump bed in a horizontal plane. The frame plate does not interfere with the dump mechanism.

Two different types of hitches were developed to

FIGURE 1 Truck-mounted portable maintenance barrier.

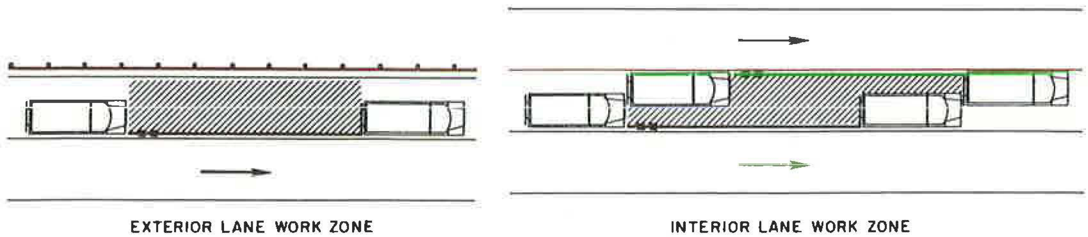


FIGURE 2 Single-lane barrier deployments.

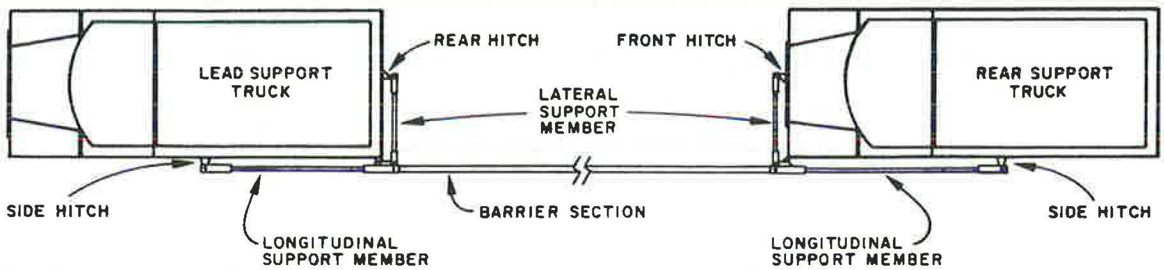


FIGURE 3 Barrier components.

attach the support members to the trucks: a front-rear hitch and a side hitch. Fabrication details of the hitches are presented in Figures 4 and 5. The lead support truck is equipped with a rear and a side hitch, and the rear support truck is equipped with a front and a side hitch. The support members attach to the hitches with pins and bolts.

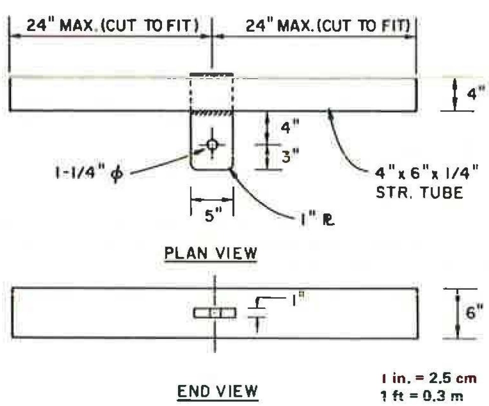


FIGURE 4 Fabrication details of front-rear hitches.

Two types of support members were developed to attach the barrier section to the hitches: longitudinal support members and lateral support members. The support members transfer the impact forces from the ends of the barrier section to the support truck hitches. The longitudinal support members connect the side hitches to the ends of the barrier section. The lateral support members connect the barrier ends of the longitudinal support members to the rear and front hitches of the front and rear support trucks, respectively. Fabrication details of the support members are presented in Figures 6 and 7.

The barrier section is fabricated by using two parallel sections of 6 x 6 x 1/4-in. (15.24 x 15.24 x 0.64-cm) structural steel tubes welded together as shown in Figure 8. The weight of the barrier section is supported by two swivel casters permanently mounted on the underside of the barrier section as shown in Figure 9. In addition, two screw jacks are permanently mounted on the barrier section to aid in handling. The ends of the barrier sections are equipped with single-pin connections that mate with the ends of the longitudinal support members. These connections are designed to allow 180 degrees of yaw and nominal amounts of pitch and roll.

The barrier section is towed to and from the work zone by using a detachable transport dolly (Figure 10). Fabrication details of the transport dolly frame are presented in Figures 11 and 12. The barrier section is loaded and unloaded onto the transport dolly by alternatively using the two barrier section screw jacks. An experienced crew of two men can load or unload the barrier section in 15 min or less. When not in use, the transport dolly is connected to an auxiliary hitch point on the rear of the front truck.

PERFORMANCE CRITERIA

Performance criteria for guardrails, traffic barriers, and other types of highway appurtenances are presented in NCHRP Report 230 (2). The criteria presented in this paper are the result of a consensus involving interested experts and professionals. Although it is not explicitly stated, the primary use of NCHRP Report 230 has been to establish performance criteria for permanent appurtenances.

The proposed portable maintenance barrier is not intended to be permanently deployed. Therefore, it is not exposed to the continual risk associated with a permanent barrier. Further, it must be recognized that maintenance workers are currently working with little or no protection. These factors combine to suggest that it is reasonable to employ performance criteria that are less stringent than those presented in NCHRP Report 230 (2).

The criteria for permanent guardrail installations presented in NCHRP Report 230 are intended to evaluate the following three principal performance factors: structural adequacy, occupant risk, and vehicle after-collision trajectory (2). Permanent guardrail installations must be designed to safely redirect a 4,500-lb (2043-kg) automobile traveling

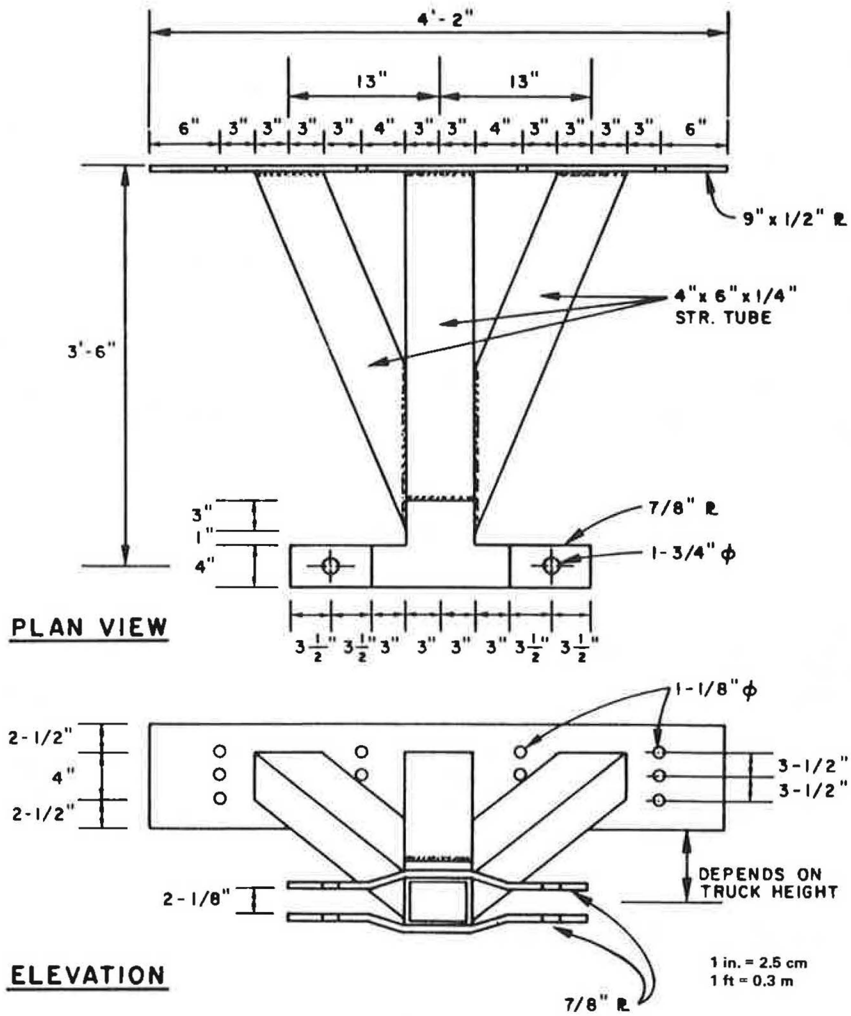


FIGURE 5 Fabrication details of side hitches.

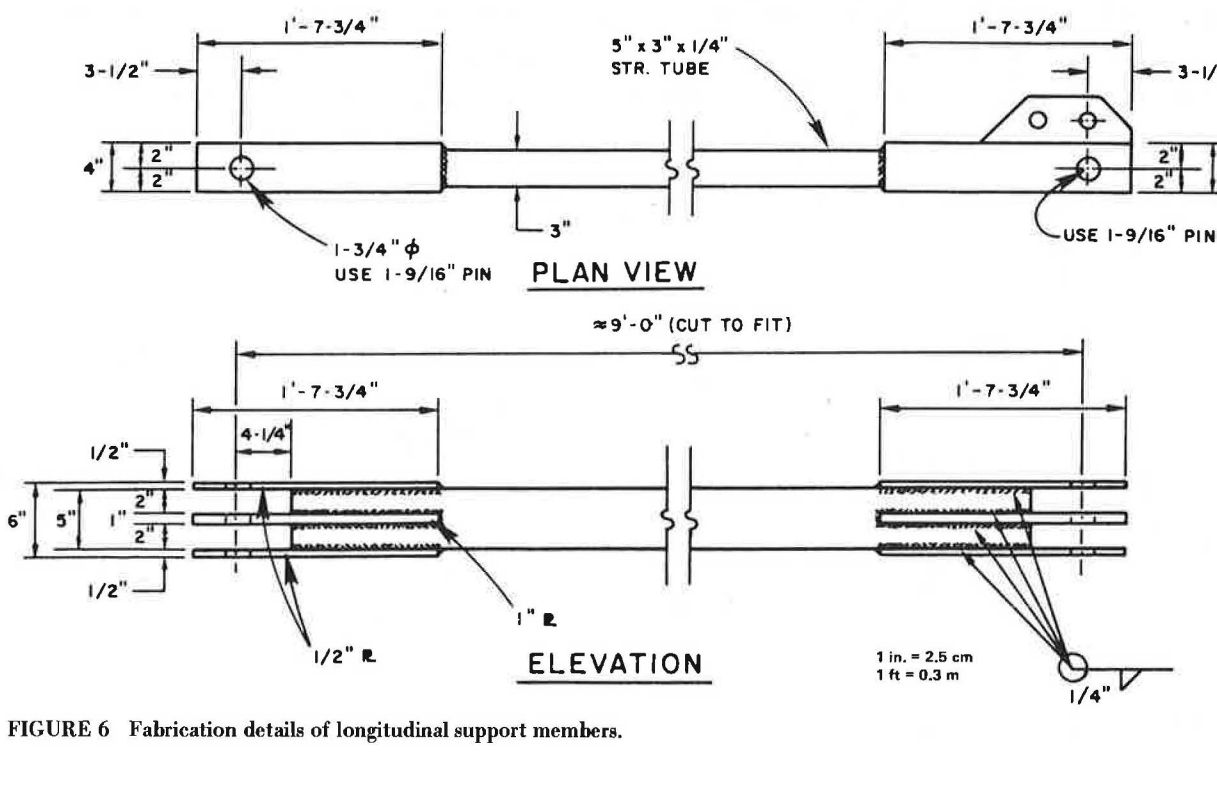


FIGURE 6 Fabrication details of longitudinal support members.

at 60 mph (96.6 km/hr) and impacting at an angle of 25 degrees. In addition, permanent guardrail installations should be able to smoothly redirect compact automobiles [2,250 and 1,800 lb (1022 and 812 kg)] traveling at 60 mph and impacting at an angle of 15 degrees. The first criterion establishes the required strength of the barrier and evaluates the occupant risk factors. The second criterion evaluates the barrier's potential for destabilizing errant automobiles.

On the basis of discussions with engineers from the Texas State Department of Highways and Public Transportation (SDHPT) and the judgments of researchers at the Texas Transportation Institute

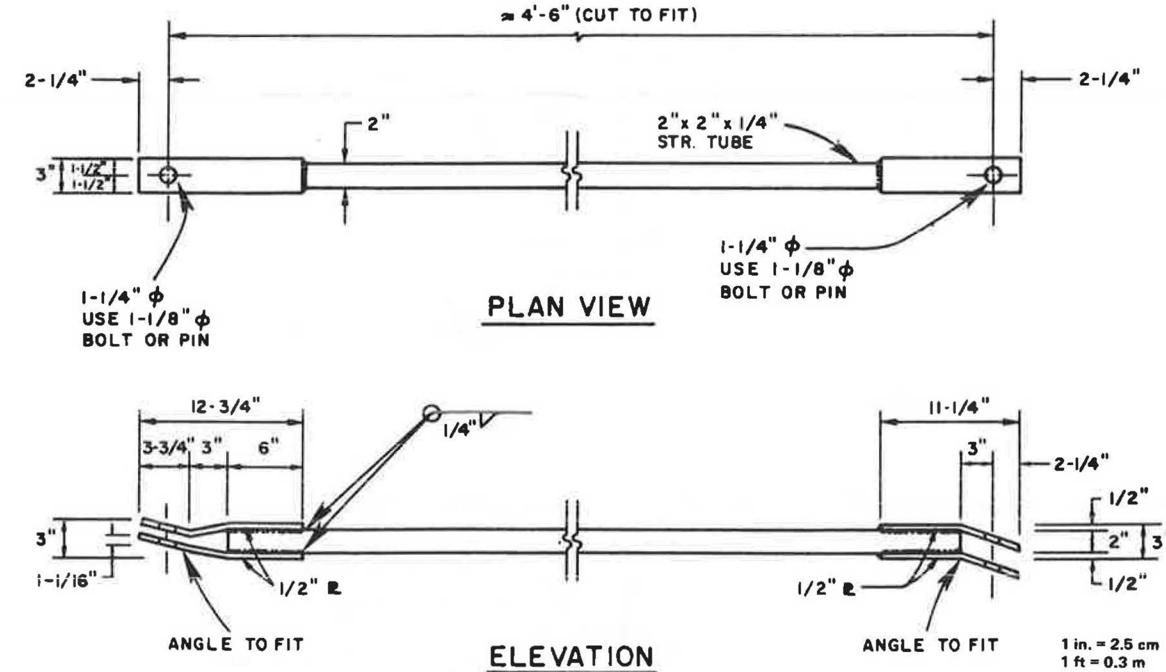


FIGURE 7 Fabrication details of lateral support members.

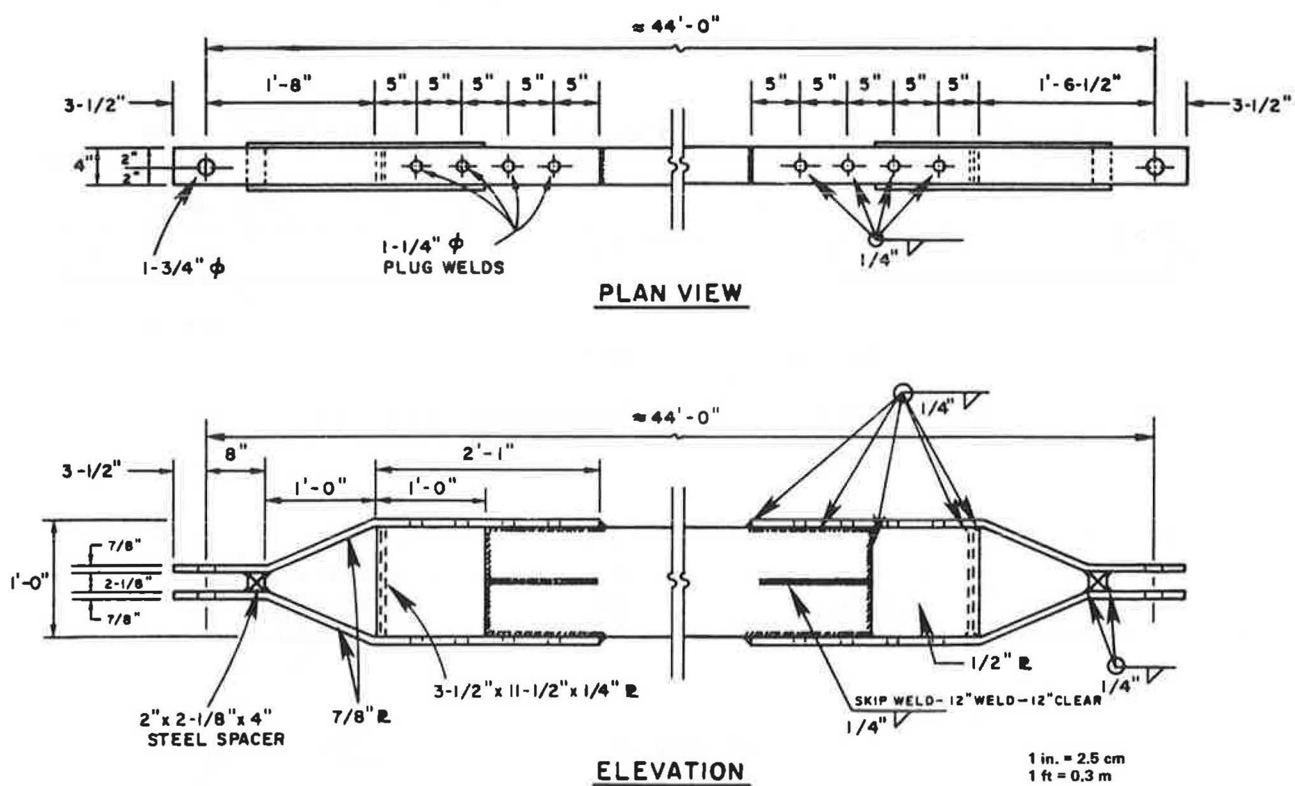


FIGURE 8 Fabrication details of barrier section.

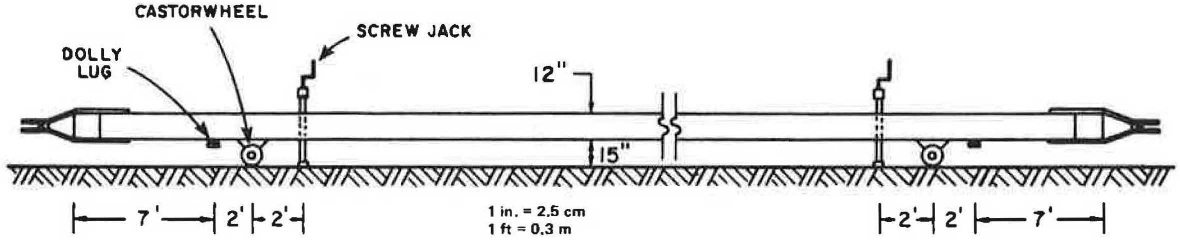


FIGURE 9 Side view of barrier section.

FIGURE 10 Barrier section on transport dolly.

(TTI), the following performance criteria were established for the truck-mounted portable maintenance barrier. It was designed to redirect a 4,500-lb automobile with a velocity of 50 mph (80.5 km/hr) and an impact angle of 15 degrees. The destabilizing potential of the portable maintenance barrier was evaluated by using an 1,800-lb automobile traveling at 50 mph and impacting at an angle of 15 degrees.

TEST RESULTS

Three full-scale crash tests were conducted on the truck-mounted portable maintenance barrier. The purpose of the tests was to establish the redirective capabilities of the barrier section and to determine its destabilizing effect on compact cars. The impact point in all of the tests was loaded one-third of the length of the barrier section ahead of the rear support truck to maximize the flexural loading on the barrier section. The authors recognize that direct impact into the support trucks would be much more serious for the errant vehicle than an impact on the barrier. However, it is their

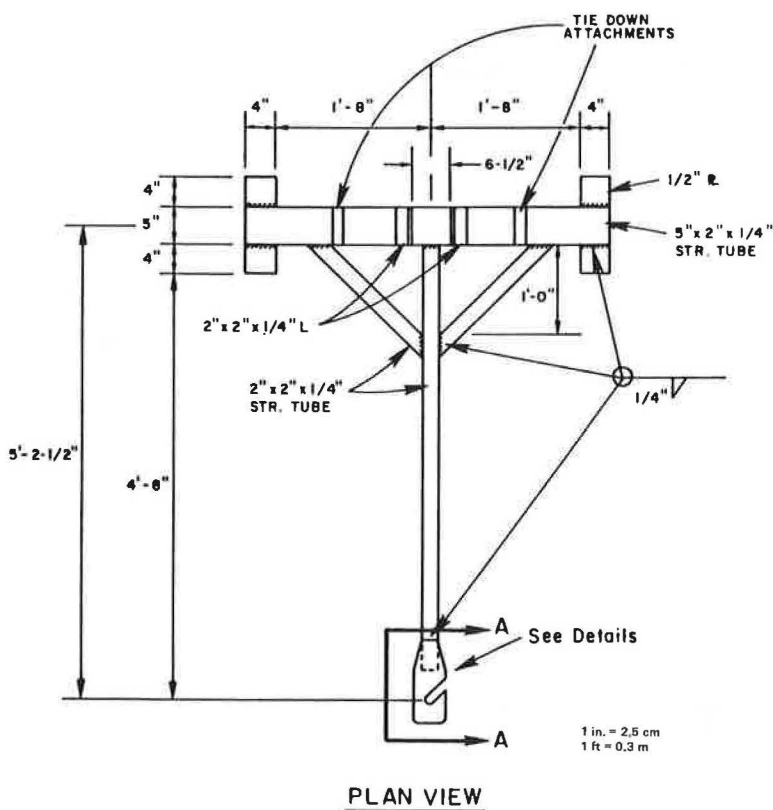


FIGURE 11 Transport dolly: plan view.

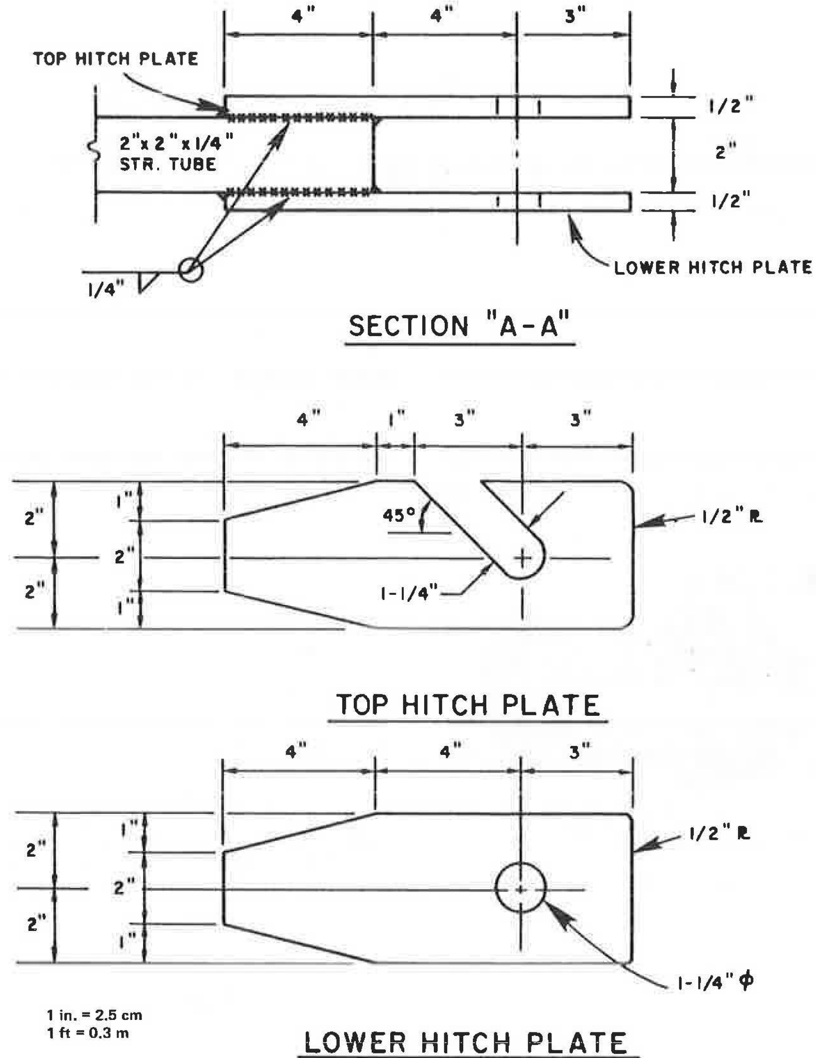


FIGURE 12 Transport dolly: fabrication details.

opinion that such an impact would not be any worse than an impact with a free-standing maintenance vehicle. It is recommended that one of several different types of rear crash cushions be towed behind the rear support truck to reduce the consequences of such a crash.

Table 1 presents a summary of pertinent test statistics. The tests were conducted in order of increasing severity. Complete photographic and accelerometer data are presented elsewhere (3). In addition, tests were conducted to evaluate the maneuverability of the barrier. Short discussions of both the strength and maneuverability tests are presented in the following paragraphs.

TABLE 1 Crash Test Summary

	Test		
	1	2	3
Vehicle weight (lb)	4,500	1,765	4,500
Impact speed (mph)	50.9	50.9	49.7
Impact angle (degrees)	7.3	14.0	15.0
Exit angle (degrees)	0.5	1.3	1.0
Barrier displacement (in.)	11.2	13.0	24.0
Occupant impact velocity (ft/sec)			
Longitudinal	6.7	11.3	10.0
Lateral	0	0	0
Occupant ridedown acceleration (*g*)			
Longitudinal	0.87	1.58	1.34
Lateral	0	0	0
Vehicle damage classification			
Traffic Accident Data	2-RFQ-1	2-RFQ-2	2-RFQ-2
Vehicle Damage Index	02RFMW5	02RFMW6	02RFMW[illegible]

Note: 1 lb = 0.45 kg; 1 mph = 1.61 km/hr; 1 in. = 2.5 cm; 1 ft/sec = 0.3 m/sec.

Full-Scale Crash Tests

In Test 1 a 4,500-lb automobile impacted the barrier with a velocity of 50.9 mph (82.0 km/hr) at an angle of 7.3 degrees. The automobile was smoothly redirected with relatively minor damage. The barrier section sustained 1/2 in. (1.22 cm) of permanent lateral deflection.

In Test 2 a 1,765-lb automobile impacted the barrier with a velocity of 50.9 mph at an angle of 14.0 degrees and was smoothly redirected with relatively minor damage. The barrier section sustained an additional 1/2 in. of permanent lateral deflection.

In Test 3 a 4,500-lb automobile impacted the barrier with a velocity of 49.7 mph (80.0 km/hr) at an angle of 15.0 degrees and was smoothly redirected with only moderate damage. The barrier section sustained an additional 3 in. (7.62 cm) of permanent lateral deflection.

Damage to the impacting vehicles in all three tests consisted of sheet metal damage on the right side and damage to the right front tire and rim. Figure 13 shows the damage done to the vehicle in Test 3 (the most severe impact). The sheet metal damage was the result of contact between the automobile and the barrier section. The damage to the right front wheel occurred when it hit the barrier section support caster as the automobile slid along the barrier section. This occurred because the caster pivoted outward into the wheelpath as the barrier section underwent lateral deformation. This phenomenon occurred in all three tests; however, the failure of the tire and rim did not destabilize the impacting vehicles. It should be noted that following Test 3, the vehicle spare tire was mounted on the right front of the car, which allowed the vehicle to be operated at low speeds. In all instances the measured occupant risk values defined in NCHRP Report 230 were below recommended values.

The same barrier section was used in all three tests with no intermediate repair or straightening. The only damage experienced by the barrier was permanent lateral deflection. On completion of the third test the barrier section had an accumulated lateral deflection of 4 in. (10.2 cm), as shown in Figure 14. The permanent lateral deflection of the barrier section in no way interfered with transport of the barriers. The support trucks, hitches, and support members survived the tests with no damage.

FIGURE 13 Damage to automobile in Test 3.

Figure 14 Accumulated damage to barrier section in Tests 1, 2, and 3.

Maneuverability Tests

In addition to the three crash tests, maneuverability tests were conducted with the truck-mounted portable maintenance barrier. It was found that the barrier section mounted on the transport dolly and hitched to the center of the lead truck for highway transport had handling characteristics similar to those of a tractor-trailor rig of similar length. There were no special problems noted by the drivers in maneuvering the system set up in this fashion.

The fully deployed system (Figure 1) consisting of both trucks, the barrier section, and two drivers had a surprising amount of maneuverability with forward speeds up to 15 mph (24.15 km/hr). When the barrier is deployed in this fashion, the lead truck provides the power. The forward thrust is transferred through the barrier section to the rear truck whose transmission is in neutral. The driver in the lead truck is responsible for controlling the application of power and braking. The responsibility of the driver in the rear truck is to guide the rear truck along the desired path. The only constraint on maneuverability is that the trucks are forced to remain a constant distance apart.

In addition to the general maneuverability tests, the following test was conducted to simulate maneuverability around an obstacle in the work zone. A section of pavement 30 ft (9.15 m) long was marked off between the trucks to simulate an obstacle such as an area of pavement under repair, as shown in Figure 15. Then it was shown that the trucks and barrier can be steered around the 30-ft repair zone without encroaching on it. The maneuverability of the portable maintenance barrier around such obstacles is hindered only by the handling characteristics of the rear truck.

CONCLUSIONS

Recent experiences with injuries and fatalities among SDHPT maintenance personnel suggest that there is a need for increased personnel protection in short-term work zones. One way to reduce the risks

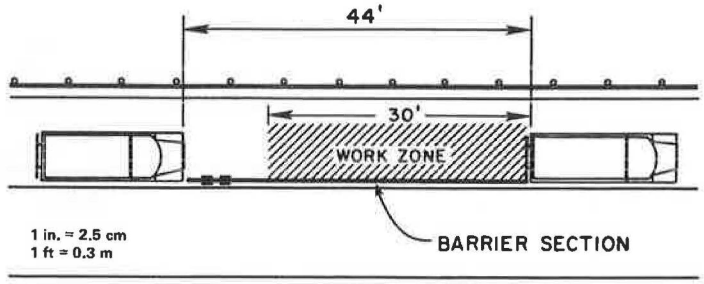


FIGURE 15 Barrier configuration for maneuverability test.

is to use portable maintenance barriers. The problem is that most available portable maintenance barriers require too much set-up time or too much work-zone space. The truck-mounted portable maintenance barrier overcomes both of these difficulties.

The truck-mounted portable maintenance barrier consists of a steel barrier section supported between two trucks. The barrier section was designed to smoothly redirect a 4,500-lb automobile impacting at a velocity of 50 mph and an angle of 15 degrees. These design criteria reflect a consensus among SDHPT and TTI engineers for a portable maintenance barrier. Results of three crash tests conducted on the prototype substantiated that the barrier section can successfully redirect the design impact.

It is clear that if an errant vehicle directly impacts either of the support trucks, the outcome would not be as favorable. However, it is the authors' contention that such an impact would be no more severe than an impact with any other maintenance vehicle. It is recommended that normal procedures involving the use of towed crash cushions and proper delineation of the work-zone hazard be used with the truck-mounted portable maintenance barrier.

The barrier is towed to the work zone on a specially fabricated transport dolly. Experience with the system shows that the barrier can be deployed by an experienced team of two men in less than 15 min. In addition, tests show that the deployed portable maintenance barrier can be easily maneuvered around obstacles that might be encountered in a work zone.

The approximate cost of the barrier system exclusive of the cost of the trucks is $8,000 for a 44-ft (13.42-m) barrier section. This translates to an approximate cost of $182 per foot ($596 per meter). A substantial portion of the fabrication cost is involved in the construction and installation of hitches, support members, and the truck frame plate. However, in the event of a design impact, only the barrier section will have to be replaced or repaired. Therefore, the economics of the system appear to be favorable.

A second version of the portable maintenance barrier has been constructed and delivered to the Houston are a SDHPT office, which plans to put it into service shortly. It is hoped that the system can be easily integrated into routine operations and help to reduce maintenance personnel injuries and fatalities.

ACKNOWLEDGMENTS

This research study was conducted under a cooperative program between the Texas Transportation Institute, the Texas State Department of Highways and Public Transportation, and FHWA. David Hustace, Hunter F. Garrison, and William V. Ward of SDHPT worked closely with the researchers, and their comments and suggestions are appreciated.

REFERENCES

1. D.L. Ivey, H.E. Ross, T.J. Hirsch, C.E. Buth, and R.M. Olson. Portable Concrete Median Barriers: Structural Design and Dynamic Performance. In Transportation Research Record 769, TRB, National Research Council, Washington, D.C., 1980, pp. 20-30.
2. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.
3. W.L. Beason and H.E. Ross. Development of a Truck-Mounted Portable Maintenance Barrier. Research Report 2262-5. Texas Transportation Institute, Texas A&M University, College Station, May 1984.

Minicar Crash Test Evaluation of Longitudinal Traffic Barriers

MAURICE E. BRONSTAD, JARVIS D. MICHIE, and JOSEPH B. MAYER, Jr.

ABSTRACT

The number of small cars in use in the United States is growing rapidly, and the changing characteristics of the vehicle fleet should be considered in highway safety design. Before the series of crash tests described in this paper was performed, few of the current operational barrier systems had been evaluated for the 1,800-lb car test in NCHRP Report 230. Eleven barrier systems were selected for evaluation and findings indicate that all systems met impact test requirements for the 1,800-lb car at 60 mph and a 15-degree angle. The 11 barrier systems included 5 guardrail, 2 median-barrier, and 4 bridge-railing systems.

The number of small cars in use in the United States is growing rapidly, and the changing characteristics of the vehicle fleet should be considered in highway safety design. NCHRP Report 230 (1), published in 1981, tentatively specifies a crash test using an 1,800-lb vehicle as a replacement for the test using a 2,250-lb small car at 60 mph and a 15-degree angle. Before the series of crash tests described in this paper was performed, few of the current operational barrier systems had been evaluated for the 1,800-lb-car test requirements. As the first phase of NCHRP Project 22-4 (2), 11 typical operational barrier designs were selected and then evaluated with regard to dynamic performance with the 1,800-lb car. The results of these evaluations are described.

The 11 barrier systems were selected on the basis primarily of use and on the AASHTO barrier guide (3); they are described in Figures 1 through 3. The test vehicle used in the evaluations was a Honda Civic with a nominal weight of 1,800 lb excluding the side impact dummy (SID) used in all tests. The unrestrained dummy was placed in the front seat on the impact side. Results of the tests were compared with the recommended assessment criteria presented in NCHRP Report 230.

FINDINGS

The 11 tests are briefly described in the following sections; the tests are summarized in Tables 1 through 3, which include an assessment regarding compliance with the recommended evaluation criteria of NCHRP Report 230 (1, Table 6). In judging these tests, the researchers did not consider the values as absolute, and some small exceedance of one value was allowed if all other values were within the recommended limits. Thus, two of the barrier systems that had one test value slightly in excess of the recommended value were given marginal pass ratings. A third resulted in a test failure due to a secondary end treatment impact that resulted in rollover (not considered a system failure).

Test GR-1

Test GR-1 evaluated System G4(2W), a blocked-out W-beam on 6 x 8-in. timber posts. The vehicle was smoothly redirected with a maximum dynamic barrier deflection of 7.7 in. as shown in Figure 4. Damage to the barrier and vehicle was moderate, as shown in Figure 5. The vehicle was operable after coming off the rail, and the barrier was fully serviceable with small permanent deformations. Measured data indicated compliance with the recommended values of NCHRP Report 230.

Test GR-2

Test GR-2 evaluated System G9, a blocked-out thrie beam on steel posts. The test vehicle was smoothly redirected with a maximum dynamic barrier deflection of 6.0 in. as shown in Figure 6. Damage to the barrier and the vehicle was moderate, as shown in Figure 5. The vehicle was operable after the test with mostly sheet metal damage, and the barrier was fully serviceable with negligible permanent deformation. Test values indicated marginal compliance with the recommended occupant risk lateral impact velocity change (ΔV) values of NCHRP Report 230. The test was judged to be successful.

Test GR-3

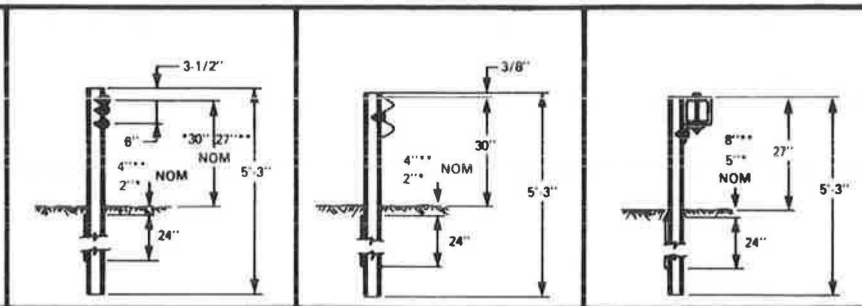
Test GR-3 evaluated System G2, a W-beam on weak steel posts. The vehicle was smoothly redirected with a maximum dynamic barrier deflection of 16.0 in. as shown in Figure 7. Contact with the posts caused the rear of the vehicle to yaw away from the barrier as it left the rail. There was sheet metal and left front wheel and tire damage to the vehicle resulting from contact with the posts. Damage to the barrier was sufficient to reduce the serviceability. One post was completely out of service. Measured test values indicated compliance with NCHRP Report 230.

Test GR-4

Test GR-4 evaluated System G3, a box beam on weak steel posts. The vehicle was smoothly redirected with a maximum dynamic barrier deflection of 6.4 in. as shown in Figure 8. Contact with the posts caused the rear of the vehicle to yaw away from the barrier as it left the rail; the vehicle recontacted the

*BARRIER GUIDE
**AS TESTED

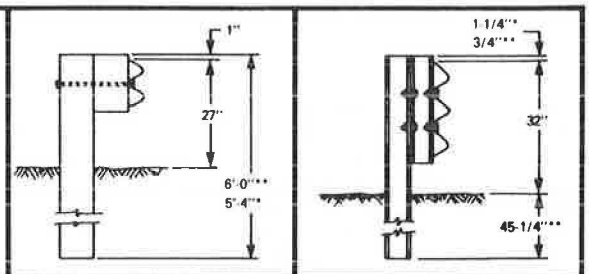
METRIC CONVERSIONS
1 ft. = 0.305 m 1 in. = 25.4 mm 1 mph = 1.61 km/hr 1 lb = 0.454 kg



SYSTEM	G1 CABLE GUARDRAIL	G2 "W" BEAM (STEEL WEAK POST)	G3 BOX BEAM
BARRIER DESCRIPTION			
POST SPACING	16'0"	12'6" NOMINAL	6'4"
POST TYPE	S3 x 5.7 STEEL	S3 x 5.7 STEEL	S3 x 5.7 STEEL
BEAM TYPE	THREE 3/4" DIAMETER STEEL CABLES	STEEL "W" SECTION, 12 GA.	6" x 6" x 0.180" STEEL TUBE
OFFSET BRACKETS	-------	-----	L5" x 3-1/2" x 1/4" STEEL ANGLE, 4-1/2" L
MOUNTINGS	5/16" DIAMETER STEEL HOOK BOLTS	5/16" DIAMETER STEEL BOLT	3/8" DIA. STEEL BOLT (BEAM TO ANGLE)
FOOTINGS	1/4" x 8" x 24" STEEL PLATE WELDED TO POST	1/4" x 8" x 24" STEEL PLATE WELDED TO POST	1/4" x 8" x 24" STEEL PLATE WELDED TO POST

*BARRIER GUIDE
**AS TESTED

METRIC CONVERSIONS
1ft. = 0.305 m 1 in. = 24.4 mm 1 mph = 161 km/hr 1 lb = 0.454 kg

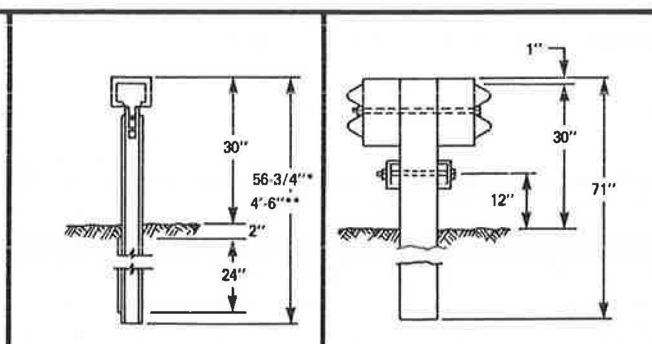


SYSTEM	G4(2W) BLOCKED-OUT "W" BEAM (WOOD POST)	G9 BLOCKED-OUT "THRIE BEAM" (STEEL POST)
BARRIER DESCRIPTION		
POST SPACING	6'3"	6'3"
POST TYPE	6" x 8" DOUGLAS FIR[3]	W6 x 8.5 STEEL
BEAM TYPE	STEEL "W" SECTION, 12 GA	THRIE BEAM, STEEL, 12 GA
OFFSET BRACKETS	6" x 8" x 14" DOUGLAS FIR BLOCK[3]	W6 x 8.5 AND M14 x 17.2, STEEL
MOUNTINGS	5/8" DIAMETER CARRIAGE BOLTS	2 5/8" DIAMETER STEEL BOLTS
FOOTINGS	NONE [3]SOUTHERN PINE (TESTED)	NONE

FIGURE 1 NCHRP Project 22-4 guardrail systems, Phase 1.

*BARRIER GUIDE
**AS TESTED

METRIC CONVERSIONS
1 ft. = 0.305 m 1 in. = 25.4 mm 1 mph = 1.61 km/hr 1 lb. = 0.454 kg



SYSTEM	MB3 BOX BEAM	MB4W BLOCKED-OUT "W" BEAM (WOOD POSTS)
BARRIER DESCRIPTION		
POST SPACING	6'0"	6'3"
POST TYPE	S3 x 5.7	8" x 8" DOUGLAS FIR[3]
BEAM TYPE	8" x 6" x 1/4" STEEL TUBE	TWO "W" SECTION, TWO C6 x 8.2 RUBRAILS
OFFSET BRACKETS	NONE	TWO 8" x 8" x 14" DOUGLAS FIR BLOCKS
MOUNTINGS	STEEL PADDLES	5/8" DIAMETER BOLTS
FOOTINGS	8" x 1/4" x 24" STEEL PLATE WELDED TO POST	NONE [3]SOUTHERN PINE (TESTED)

FIGURE 2 NCHRP Project 22-4 median barrier systems, Phase 1.

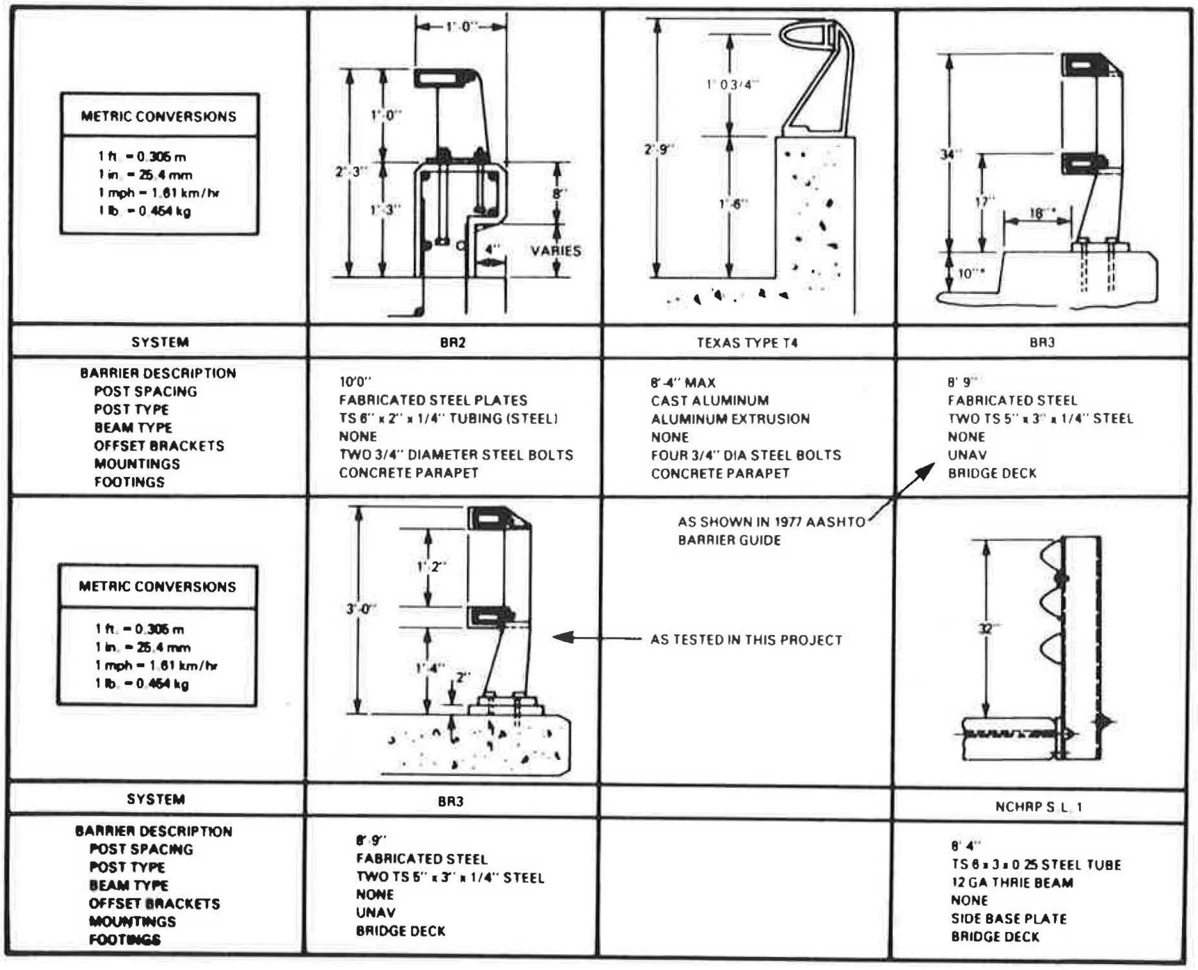


FIGURE 3 NCHRP Project 22-4 bridge-rail systems, Phase 1.

TABLE 1 Summary of Guardrail Crash Tests

	Test No.				
	GR-1	GR-2	GR-3	GR-4	GR-5
Barrier[a]	G4(2W)	G9	G2	G3	G1
Test-vehicle year[b]	1977	1978	1976	1978	1976
Gross vehicle weight (lb)	1,989	1,948	1,857	1,916	1,973
Impact speed (film) (mph)	60.1	59.3	59.7	60.4	60.5
Impact angle (degrees)	15.5	14.4	15.4	15.3	15.8
Impact duration (sec)	0.25	0.22	0.38	0.27	0.84
Maximum deflection (in.)					
Dynamie	7.7	6.0	16.0	6.4	43.4
Permanent	3.2	1.5	11.9	0	Slack cables
Exit angle (degrees)					
Film	-2.1	-3.5	-1.7	4.1	NA
Yaw rate transducer	-1.6	-4.0	-6.0	2.4	1.7
Exit speed (mph)					
Film	54.7	52.3	50.4	49.3	NA
Accelerometer	55.9	52.1	59.0	46.8	43.8
Maximum 50-msec avg acceleration (film/accelerometer)					
Longitudinal	1.8/2.1	3.5/3.1	2.1/2.3	3.2/4.1	2.9/2.1
Lateral	5.9/7.3	6.7/8.1	4.3/6.9	6.7/5.9	2.7/2.2
Occupant risk[c] (film/accelerometer)					
Longitudinal ΔV (fps) (30)	-[d]/-[d]	-[d]/-[d]	15.7/-[d]	-[d]/18.3	12.7/9.8
Lateral ΔV (fps) (20)	19.8/18.6	21.5/20.4	17.0/17.3	18.9/17.8	11.9/10.6
Ridedown acceleration (*g*) (accelerometer)					
Longitudinal (15)	-[d]	-[d]	-[d]	6.2	1.7
Lateral (15)	13.8	10.6	14.7	10.0	8.7
NCHRP Report 230 evaluation					
Structural adequacy (A,D)	Pass	Pass	Pass	Pass	Pass
Occupant risk (E,F,G)	Pass	Pass (marginal F)	Pass	Pass	Fail (E)
Vehicle trajectory (H,I)	Pass	Pass	Pass	Pass	Pass (marginal I)
Barrier damage rating[e]	2	2	3	3	4
Posts not serviceable	None	None	1	2	3

Note: NA = data not available.

[a]AASHTO barrier guide designation (1977).
[b]All tests used a Honda Civic.
[c]Numbers in parentheses are recommended values for NCHRP Report 230.
[d]Occupant did not travel the flail distance.
[e]Barrier damage code: 1, undamaged; 2, fully serviceable, but moderately damaged; 3, reduced service due to damage in impact area; 4, not serviceable in impact area. Damage repair indicated for 3, immediate damage repair for 4.

TABLE 2 Summary of Median Barrier Tests

	Test No.	
	MB-1	MB-2
Barrier[a]	MB4W	MB3
Test-vehicle year[b]	1977	1978
Gross vehicle weight (lb)	1,947	1,979
Impact speed (film) (mph)	58.5	61.6
Impact angle (degrees)	17.2	14.5
Impact duration (sec)	0.24	0.38
Maximum deflection (in.)		
Dynamic	2.5	7.0
Permanent	0	0
Exit angle (degrees)		
Film	-5.3	2.5
Yaw rate transducer	NA	2.6
Exit speed (mph)		
Film	54.7	46.7
Accelerometer	NA	49.2
Maximum 50-msec avg acceleration (film/accelerometer)		
Longitudinal	2.2/NA	3.8/3.8
Lateral	7.4/NA	5.1/5.1
Occupant risk[c] (film/accelerometer)		
Longitudinal ΔV (fps) (30)	-[d]/NA	16.6/13.8
Lateral ΔV (fps) (20)	21.4/NA	16.1/16.9
Ridedown acceleration (*g*) (accelerometer)		
Longitudinal (15)	NA	3.6
Lateral (15)	NA	5.9
NCHRP Report 230 evaluation		
Structural adequacy (A,D)	Pass	Pass
Occupant risk (E,F,G)	Pass (marginal F)	Pass
Vehicle trajectory (H,I)	Pass	Pass
Barrier damage rating[e]	2	3
Post not serviceable	0	3

Note: NA = data not available.

[a]AASHTO barrier guide designation (1977).

[b]All tests used a Honda Civic.

[c]Numbers in parentheses are recommended values for NCHRP Report 230.

[d]Occupant did not travel the flail distance.

[e]Barrier damage code: 1, undamaged; 2, fully serviceable, but moderately damaged; 3, reduced service due to damage in impact area; 4, not serviceable in impact area. Damage repair indicated for 3, immediate damage repair for 4.

TABLE 3 Summary of Bridge-Rail Tests

	Test No.			
	BR-1	BR-2	BR-3	BR-4
Barrier[a]	BR2	Texas Type T4	BR3	NCHRP SL 1
Test-vehicle year[b]	1978	1978	1979	1978
Gross vehicle weight (lb)	1,929	1,980	1,990	1,987
Impact speed (film) (mph)	60.9	61.0	61.0	61.4
Impact angle (degrees)	13.1	15.0	14.2	14.1
Impact duration (sec)	0.24	0.25	0.28	0.32
Maximum deflection (in.)				
Dyamic	0	0	0	17.2
Permanent	0	0	0	6.8
Exit angle (degrees)				
Film	-4.1	-5.6	0.5	-5.5
Yaw rate transducer	0.2	0.3	0.3	-1.6
Exit speed (mph)				
Film	57.9	54.5	51.0	55.9
Accelerometer	55.0	50.0	48.2	58.1
Maximum 50-msec avg acceleration (film/accelerometer)				
Longitudinal	2.7/3.8	1.9/6.1	3.1/6.9	1.8/2.0
Lateral	4.6/10.2	4.8/10.3	6.1/8.0	3.5/6.4
Occupant risk[c] (film/accelerometer)				
Longitudinal ΔV (fps) (30)	-[d]/5.9	-[d]/13.1	12.0/15.8	11.7/8.4
Lateral ΔV (fps) (20)	17.2/16.2	17.5/18.5	19.5/18.0	15.1/17.0
Ridedown acceleration (*g*) (accelerometer)				
Longitudinal (15)	-[d]	2.90	3.5	0.8
Lateral (15)	9.6	14.1	13.2	8.5
NCHRP Report 230 evaluation				
Structural adequacy (A,D)	Pass	Pass	Pass	Pass
Occupant risk (E,F,G)	Pass	Pass	Pass	Pass
Vehicle trajectory (H,I)	Pass	Pass	Pass	Pass
Barrier damage rating[e]	1	1	1	3
Posts not serviceable	0	0	0	2

[a]AASHTO barrier guide designation (1977).

[b]All tests used a Honda Civic.

[c]Numbers in parentheses are recommended values for NCHRP Report 230.

[d]Occupant did not travel the flail distance.

[e]Barrier damage code: 1, undamaged; 2, fully serviceable, but moderately damaged; 3, reduced service due to damage in impact area; 4, not serviceable in impact area. Damage repair indicated for 3, immediate damage repair for 4.

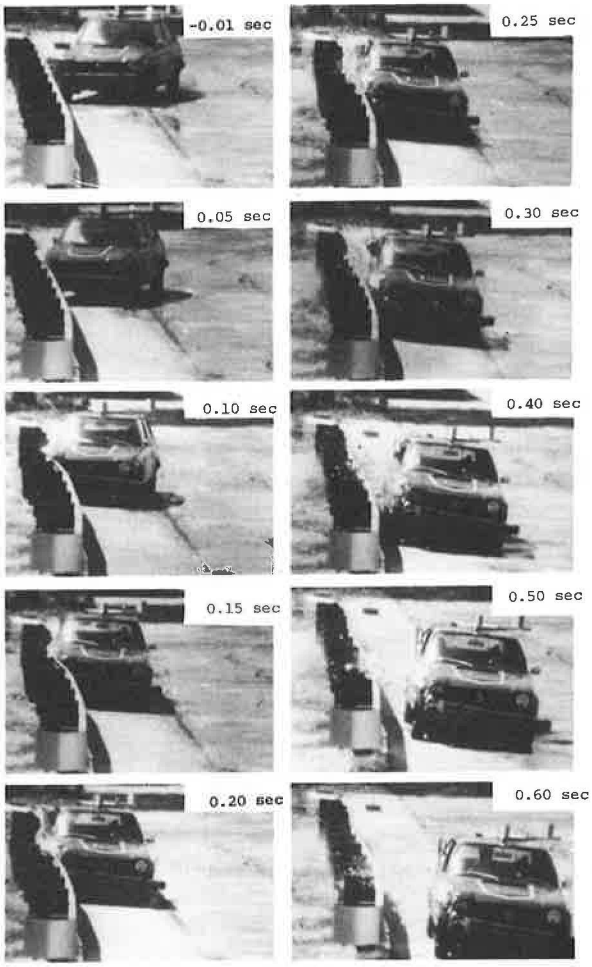


FIGURE 4 Sequential photographs, Test GR-1.

barrier downstream. There was considerable front wheel damage because of contact with the posts; sheet metal damage was extensive in the front quadrant. There was no permanent set in the rail although two posts were completely out of service and another was detached from the rail. Test values measured indicated compliance with NCHRP Report 230. Vehicle and barrier damage are shown in Figure 5.

Test GR-5

Test GR-5 evaluated System G1, a cable on weak steel posts. The vehicle was smoothly redirected with a maximum dynamic barrier deflection of 43.4 in. as shown in Figure 9. The rear of the vehicle yawed away from the barrier as the vehicle left the barrier; the vehicle then recontacted the barrier terminal, snagged, and rolled over. The breakaway feature of the terminal failed to release the cables from the anchorage. Vehicle damage before rollover was confined to sheet metal and the front wheel (because of post contact). Barrier damage was extensive with three posts out of service and cables lying on the ground as shown in Figure 5. Before the rollover the test would have been judged successful except for the 15-mph velocity change criterion of NCHRP Report 230 (1, Table 6, I). This value was slightly exceeded and a marginal pass was indicated.

Test MB-1

Test MB-1 evaluated System MB4W, a blocked-out W-beam on 8 x 8-in. timber posts with channel rub rail. The test vehicle was redirected with a maximum dynamic barrier deflection of 2.5 in. as shown in Figure 10. There was no evidence of vehicle contact with the rub rail. The vehicle sustained side sheet metal and

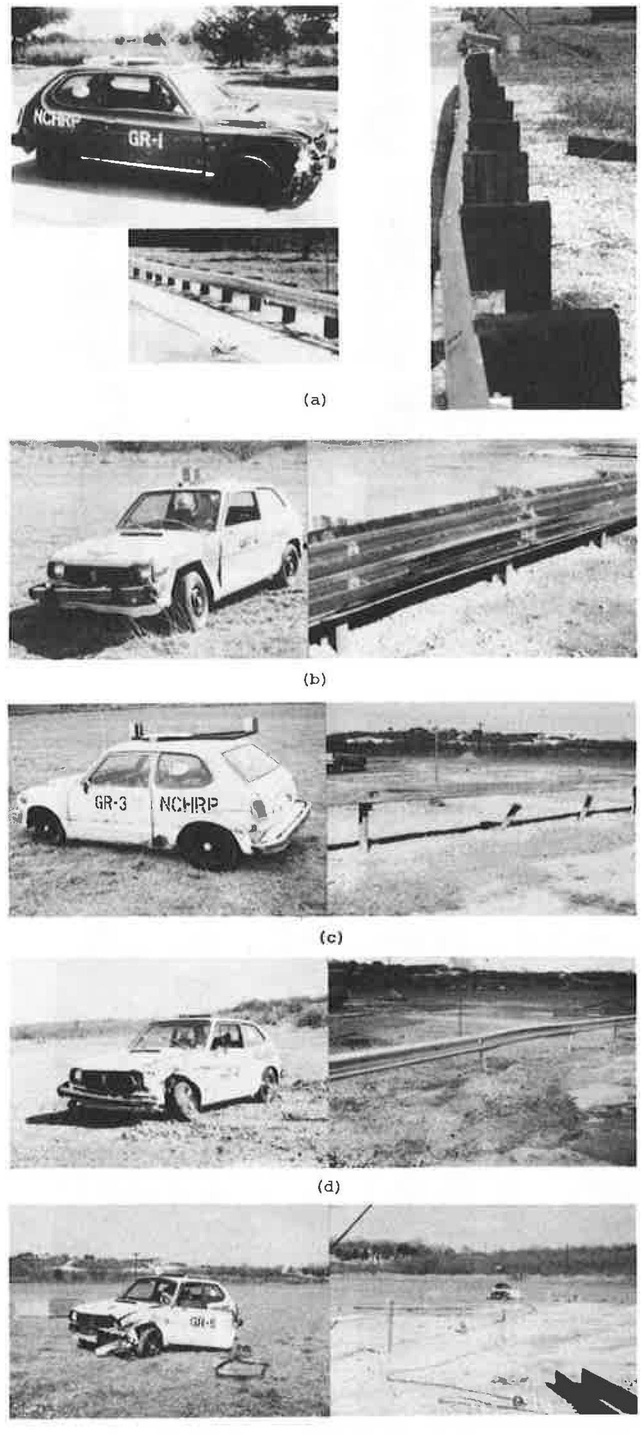


FIGURE 5 Barrier and vehicle damage after guardrail tests: (a) Test GR-1, System G4 (2W); (b) Test GR-2, System G9; (c) Test GR-3, System G2; (d) Test GR-4, System G3; (e) Test GR-5, System G1.

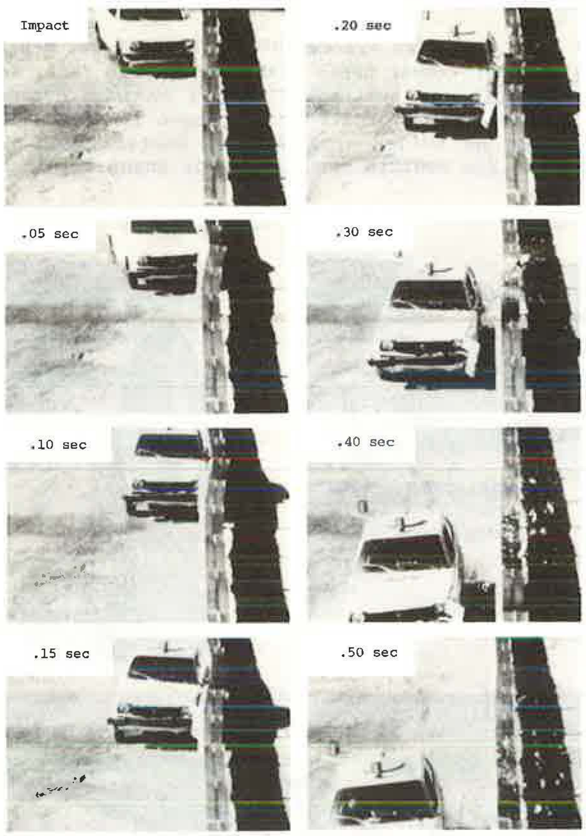


FIGURE 6 Sequential photographs, Test GR-2.

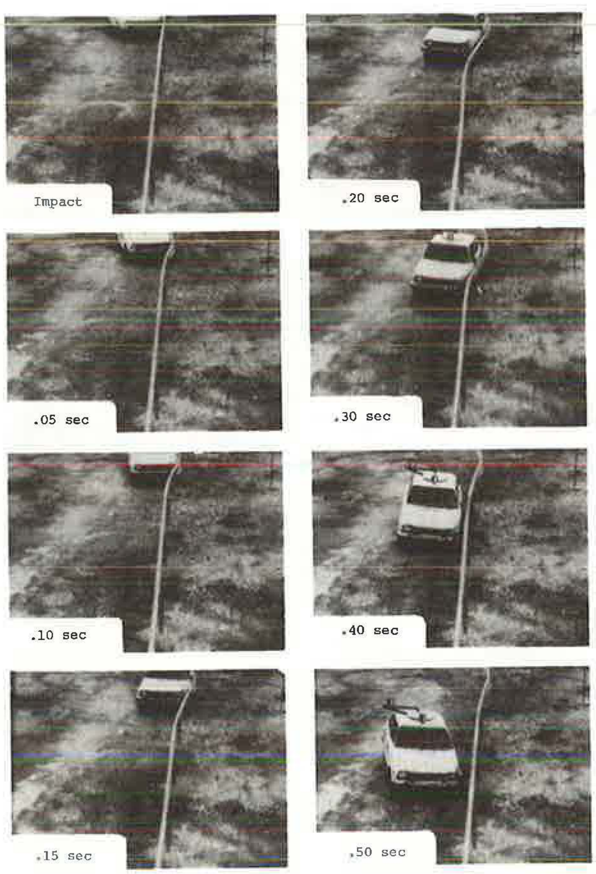


FIGURE 7 Sequential photographs, Test GR-3.

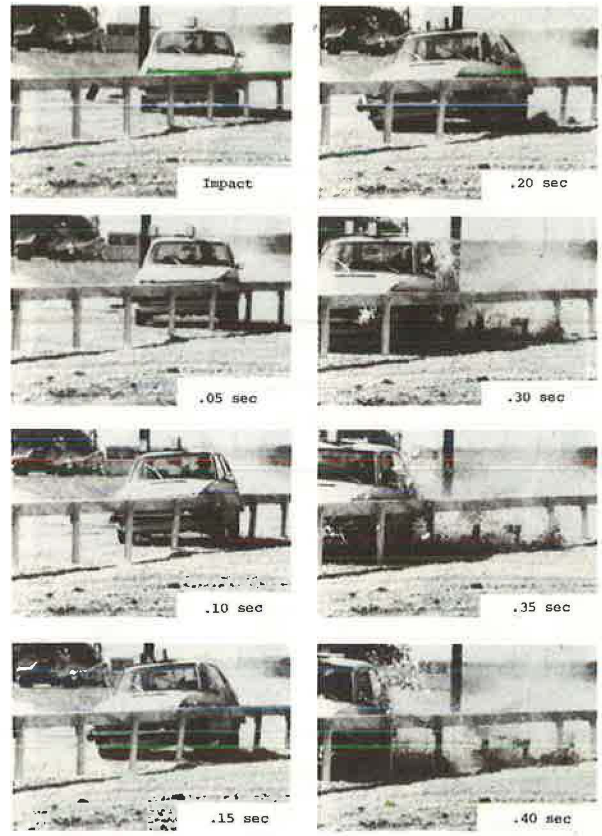


FIGURE 8 Sequential photographs, Test GR-4.

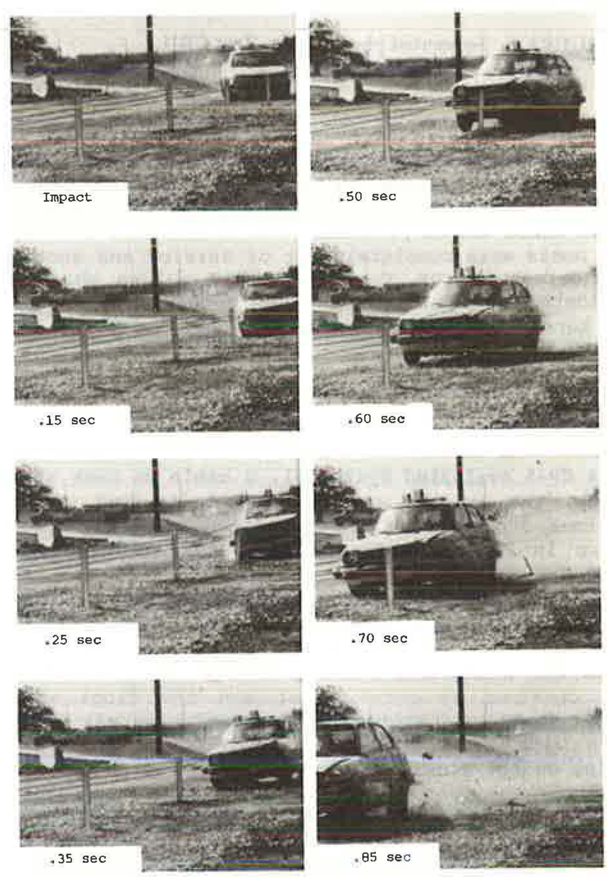


FIGURE 9 Sequential photographs, Test GR-5.

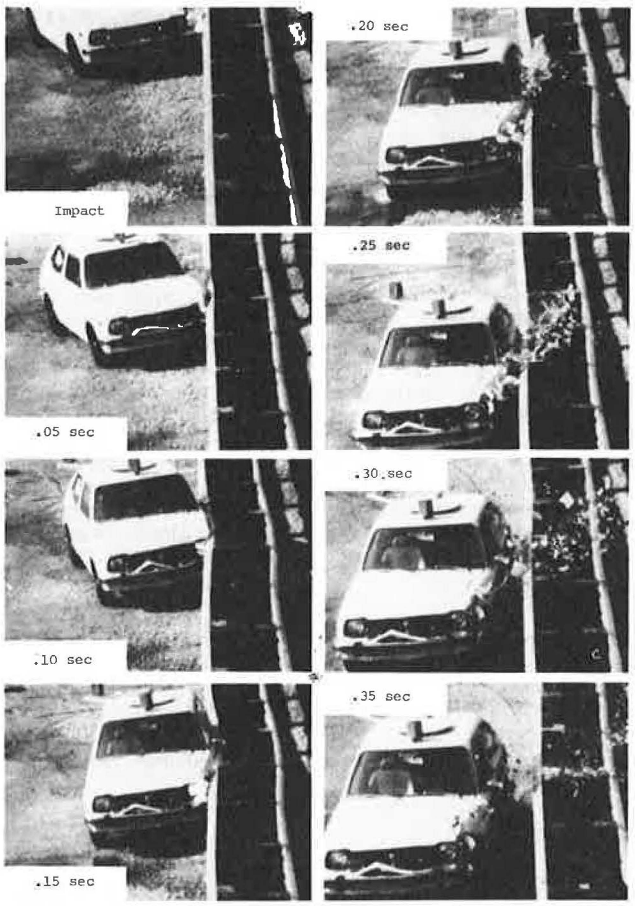
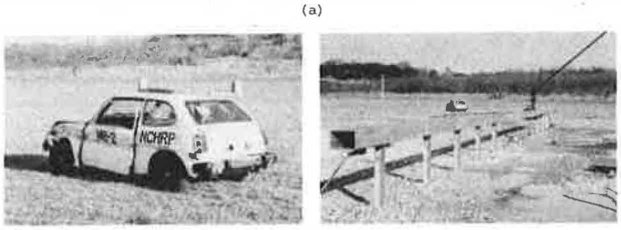


FIGURE 10. Sequential photographs, Test MB-1.

bumper damage; it was operable after the test. Damage to the barrier consisted of local beam deformation at two block-outs as shown in Figure 11. The barrier was fully serviceable with no measurable permanent deformation. On the basis of measured values, the test was judged to be successful although the occupant risk lateral ΔV slightly exceeded the NCHRP Report 230 value.

(a)



(b)

FIGURE 11 Barrier and vehicle damage after median-barrier tests: (a) Test MB-1, System MB4W; (b) Test MB-2, System MB3.

Test MB-2

Test MB-2 evaluated the performance of System MB3, a box beam on weak steel posts. The test vehicle was redirected with a maximum dynamic barrier deflection of 7.0 in. as shown in Figure 12. Because of contact with the posts, the rear of the vehicle yawed away from the barrier as contact with the barrier was lost. Vehicle damage was limited to sheet metal and bumper; all tires remained inflated and the vehicle was operable after the test. Damage to the barrier consisted of three failed posts as shown in Figure 11; there was no permanent set in the rail. Measured values indicated full compliance with the recommendations oF NCHRP Report 230.

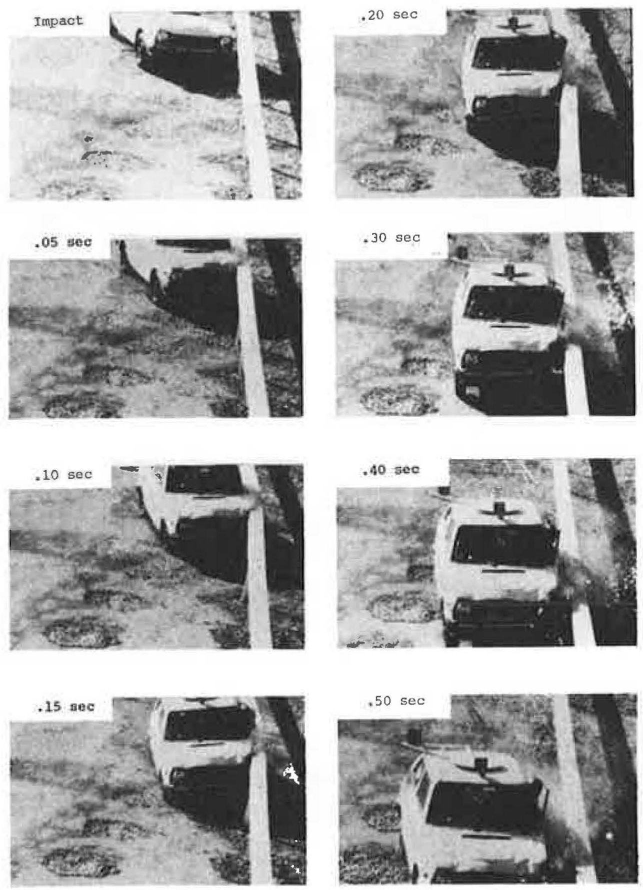


FIGURE 12 Sequential photographs, Test MB-2.

Test BR-1

Test BR-1 evaluated System BR2, a California Type 9, featuring a steel rail mounted on a 15-in.-high parapet [this is 3 in. below the requirement of the AASHTO specifications (4)]. The vehicle was smoothly redirected with no barrier deflection, as shown in Figure 13. No snagging or wedging of the vehicle under the rail was noted. There was sheet metal deformation of the right front and side of the vehicle; the vehicle was operable after the test. No damage to the barrier was noted, as shown in Figure 14. Measured values indicated compliance with NCHRP Report 230.

Test BR-2

Test BR-2 evaluated the Texas Type T4 (aluminum) bridge rail mounted on a parapet 18 in. high. The

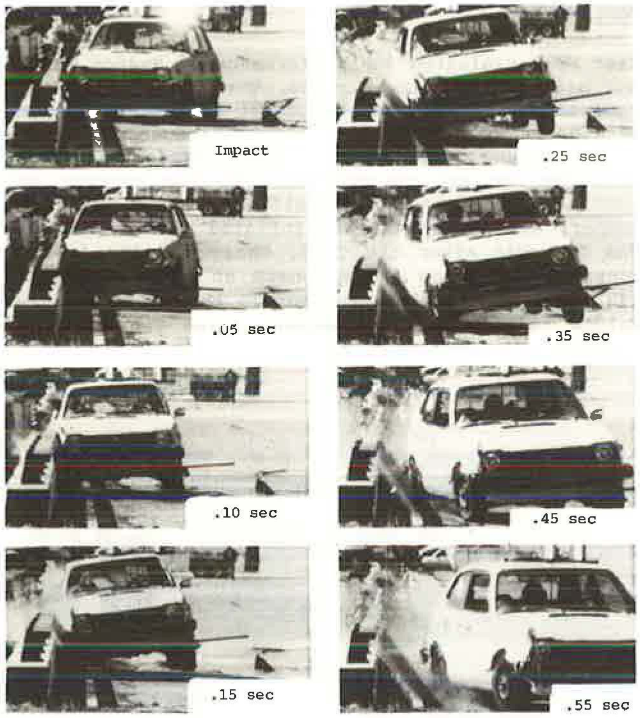


FIGURE 13 Sequential photographs, Test BR-1.

vehicle was smoothly redirected with no barrier deflection and no evidence of snagging, as shown in Figure 15. The vehicle sustained front and side sheet metal damage. All tires remained inflated and the vehicle was considered operable after the test. No damage to the barrier was evident, as shown in Figure 14. Measured values indicated compliance with NCHRP Report 230.

Test BR-3

Test BR-3 evaluated System BR3, a New York box beam bridge rail mounted on a flush deck. The test vehicle was redirected after significant wheel snagging had occurred on the first downstream post as shown in Figure 16. The redirected vehicle remained essentially parallel to the rail for a considerable distance. No barrier deflection was evident, as shown in Figure 14. There was extensive sheet metal damage to the vehicle. A-pillar, windshield, and the right A-frame were significantly damaged. No significant damage to the barrier system was evident. Measured values indicated compliance with NCHRP Report 230.

Test BR-4

Test BR-4 evaluated the NCHRP Service Level 1 bridge-rail system, which uses a thrie beam mounted on breakaway steel posts. The test vehicle was smoothly redirected after a 17.2-in. maximum dynamic barrier deflection as shown in Figure 17. Although the right

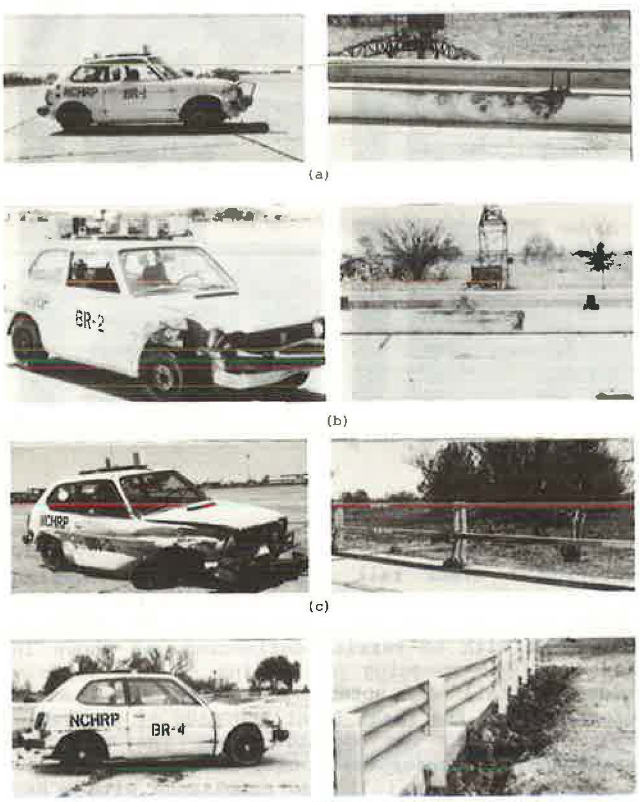


FIGURE 14 Barrier and vehicle damage after bridge-rail tests: (a) Test BR-1, System BR2; (b) Test BR-2, Texas Type T4; (c) Test BR-3, System BR3; (d) Test BR-4, Service Level 1 bridge rail.

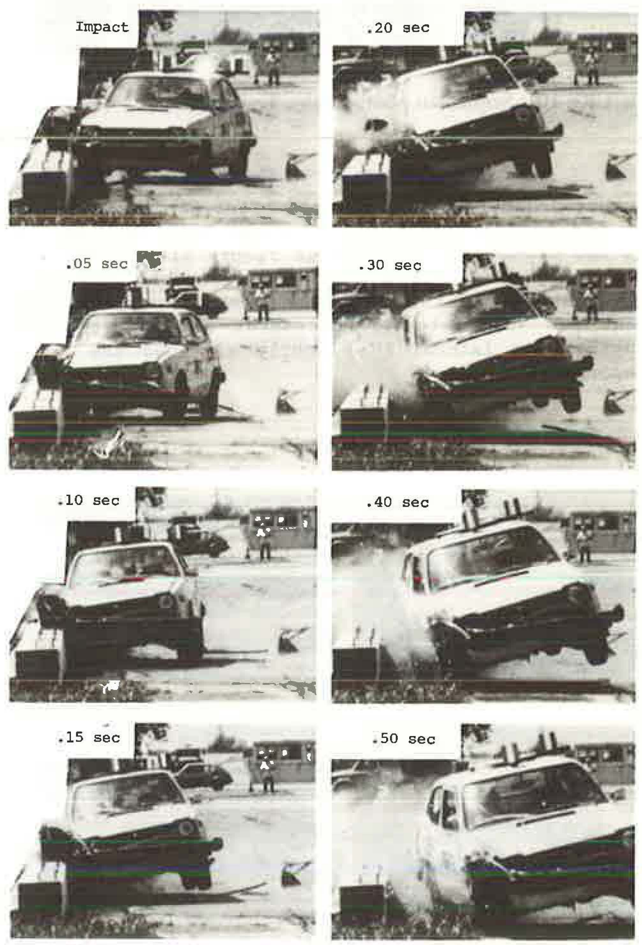


FIGURE 15 Sequential photographs, Test BR-2.

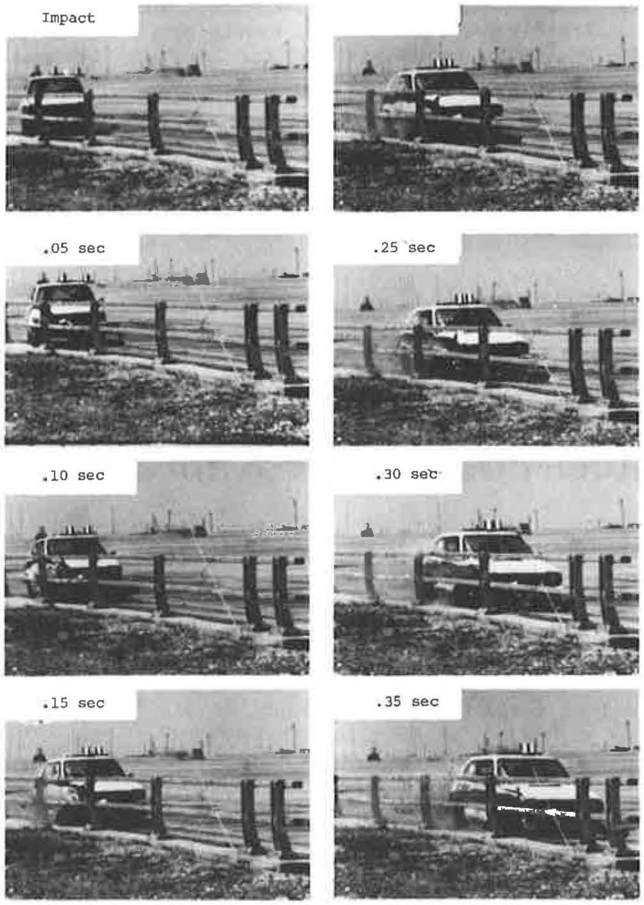


FIGURE 16 Sequential photographs, Test BR-3.

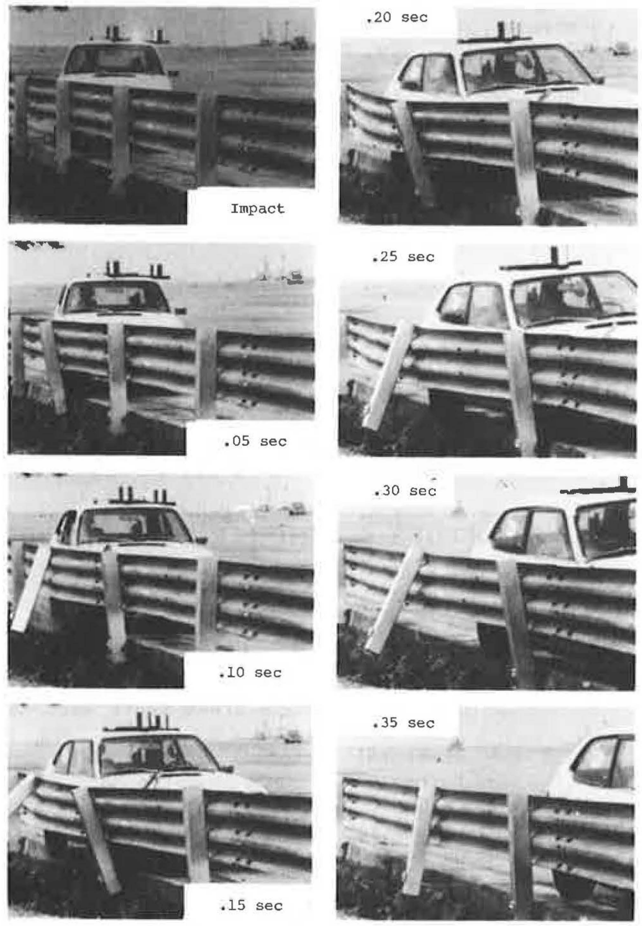


FIGURE 17 Sequential photographs, Test BR-4.

wheels of the vehicle dropped off and below the deck upper surface, they returned to the deck as the redirection continued. The vehicle damage was slight and confined to sheet metal. The vehicle was operable after the test. The barrier damage included one slightly deformed thrie-beam section and two posts that were detached from the base plate, as shown in Figure 14. Measured values indicated compliance with NCHRP Report 230.

CONCLUSIONS

On the basis of findings of the tests described in this paper, the following conclusions have been developed:

1. With minor exceptions, all 11 longitudinal barrier systems evaluated according to NCHRP Report 230, Test 12 (1,800-lb vehicle, 60 mph, and 15-degree angle) performed well and are deemed to have satisfied the assessment criteria. The vehicles remained upright [rollover in System G1 cable guardrail test (Test GR-5) was considered an end-treatment problem], were smoothly redirected, and sustained only moderate damage. Potential modifications to enhance the performance of the barrier systems with the Test 12 conditions are considered unwarranted.
2. With regard to limits of barrier performance with the minicar, Test 12 was not a discerning experiment because all 11 longitudinal barrier systems passed the evaluation criteria. A more discerning test, Test S13 (1,800-lb car, 60 mph, 20-degree angle), is considered desirable to thoroughly evaluate the snagging and possible occupant risk limits of the 11 barrier systems. There is recent evidence that a significant percentage of reported accidents occur where the impact angle exceeds 15 degrees (5).
3. Although the 11 barrier systems have been demonstrated to perform satisfactorily with NCHRP Report 230 minimum matrix Tests 10 and 12, two supplementary but important performance properties have not been evaluated, namely, capability of performing with vehicles that have a high center of gravity such as vans and school buses and the structural limit to contain higher service level loadings.

ACKNOWLEDGMENTS

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Finally, the cooperation and guidance of the NCHRP staff and project panel are acknowledged.

REFERENCES

1. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Ap-

purtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.
2. M.E. Bronstad, J.D. Michie, and J.B. Mayer, Jr. Performance of Longitudinal Traffic Barriers. NCHRP Project 22-4, Phase I Interim Report Draft. TRB, National Research Council, Washington, D.C., April 1984.
3. Guide for Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, D.C., 1977.
4. Standard Specifications for Highway Bridges. AASHTO, Washington, D.C., 1977.

Performance of a Thrie-Beam Steel-Post Bridge-Rail System

JAMES E. BRYDEN and RICHARD G. PHILLIPS

ABSTRACT

Twelve full-scale crash tests were performed to evaluate the performance of a thrie-beam bridge-rail system. The railing consisted of 10-gauge thrie-beam steel rail attached to W6x9 steel posts spaced at 8 ft 4 in. Posts were attached to the deck by using base plates and anchor bolts. The system was tested both with and without a 6-in. curb with the rail at a height of 33 in. (measured from the deck) for both designs. Also tested were transitions from W-beam guiderail on S3x5.7 posts to thrie-beam guiderail on W6x9 posts and from the thrie-beam guiderail to the bridge rail. Tests with both 4,500- and 1,800-lb vehicles showed that the railing system generally meets the recommended performance standards in NCHRP Report 230.

The reconstruction of older structures to replace existing railings with new ones that meet current standards is prohibitively expensive, so the New York State Department of Transportation (NYSDOT) has initiated efforts to improve performance by installing additional railing components. Called "upgradings" or "retrofits," these designs use existing railing and superstructure components to the greatest extent possible and add only the necessary railings, posts, and connectors to achieve the desired performance. Several bridge-railing retrofits have already been tested and are now in use on bridges with discontinuous-panel railings (1,2).

However, some structures do not permit simple attachment of the retrofit to the existing railing system. One solution developed by the Structures Design and Construction Division is shown on Standard Sheet BDD 81-57F (Details for Attaching Thrie-Beam Railing to Bridge Railing). This design mounts 10-gauge thrie-beam railing on new heavy steel posts that are attached to the deck by using an anchor plate and grouted anchor bolts. Analytical procedures used to develop that design indicated the need for W6x25 posts spaced at 4 ft 6 in. Southwest Research Institute has previously tested a similar design that used 12-gauge tubular thrie-beam rail with good results (3). A 12-gauge thrie-beam on W6x9 post spaced at 5 ft that could redirect a 4,500-lb vehicle at 60 mph and 15 degrees was also developed for use as a low-service-level bridge rail (4). The NYSDOT design, which used a single 10-gauge thrie beam, appeared to offer several advantages:

1. Less complex splices are required,
2. Handling is easier because of its lighter weight,
3. Fewer inventory items are required for repair, and
4. Construction and maintenance costs should be lower.

The principal disadvantage of the proposed New York design was heavy steel posts at close spacing; the necessity of grouting so many bolts into the deck would add substantially to the railing's cost. However, other Texas Transportation Institute tests (5) evaluated a railing system composed of two W-beam rails overlapped to form three corrugations similar to a thrie beam. That railing performed adequately when attached to steel posts spaced at 8 ft 4 in., which indicated that it may be possible to increase the post spacing of the New York design.

METHODOLOGY

A total of 12 full-scale tests were conducted in 1982 and 1983 following the recommendations of NCHRP Report 230 (6). These tests were planned to determine maximum permissible post spacing as well as the level of performance provided by this railing system. In addition, tests of proposed transitions to W-beam approach guiderail were needed to ensure their adequate performance. Concrete footings 3 ft wide by 3 ft deep by 40 ft long simulated bridge decks for these tests. Two footings were used: one providing a 6-in. curb and the other simulating a curbless deck. New York's standard bridge deck designs have provided good performance in service, and deck failure has not been noted in severe railing impacts. Thus, it was possible to simplify these tests by using a rigid footing rather than by constructing a more detailed simulated deck.

DESCRIPTION OF BARRIER SYSTEMS

The railing design developed by the Structures Design and Construction Division consisted of 10-gauge thrie beam attached to W6x25 steel posts. The thrie beams are standard 13.5-ft sections, providing 12.5-ft lay lengths with 1-ft splice overlays. The posts were welded to 1-in.-thick base plates, and 1-in.-diameter threaded steel rods were grouted into the deck 10 in. deep for anchor bolts. The railing face was set flush with the curb face for the first test, which placed the centers of the anchor bolts about 3 in. behind the curb. On the basis of a review of the research just discussed, post spacing was increased to 6 ft 3 in. for the first test rather than the 4.5 ft shown on the BDD sheet. Height to the top of the rail was 33 in. above the deck. Four sections of thrie-beam rail were mounted on the simulated bridge, totaling 50 ft in length, and the thrie beam was transitioned to W-beam guiderail on both ends. The W-beam was then anchored to standard concrete foundations, providing a total barrier length of 132 ft. Post size and spacing on the bridge were varied from test to test on the basis of results of the preceding test. Those details are provided in the next section. In addition, this railing system was tested on a curbless deck to determine performance of that configuration. With no curb, the height of the thrie beam was maintained at 33 in. by increasing the post length. The transitions to W-beam guiderail generally consisted of one length of thrie beam extending off the bridge, a tapered transition from three to two corrugations, and then NYSDOT standard W-beam guiderail on S3x5.7 steel posts. Several designs were tested before performance was considered adequate, and details of those designs are also provided in the next section.

RESULTS

Twelve tests were conducted to evaluate the railing system and transition, including three on the bridge railing with curb, five on the transition from W-beam guiderail to thrie-beam approach rail, and two each on the thrie-beam approach rail and bridge railing without curb. Results are summarized in Tables 1 and 2 and discussed in this section.

Railing with Curb

For the first test of the railing with curb, W6x25 steel posts were spaced at 6 ft 3 in. with the face of the thrie beam flush with the curb face (Figure 1). In Test 64, the 4,500-lb Dodge station wagon impacted 1.9 ft downstream from Post 3 at 60.1 mph and 26 degrees. It remained in contact with the barrier for 15.5 ft and was smoothly redirected at an 8-degree exit angle. After departure, the vehicle turned gradually toward the left, achieving an angle of 15 degrees with the barrier. Dynamic barrier deflection was limited to 1.1 ft. The vehicle remained stable throughout the impact, with maximum roll of 15 degrees (clockwise), maximum pitch of 5 degrees (front down), and maximum yaw of 21 degrees (counterclockwise). Peak 50-msec average decelerations were 5.8 g longitudinal and 8.4 g lateral, with occupant impact velocities of 19.4 ft/sec longitudinal and 21.2 ft/sec lateral. Lateral occupant impact velocity thus only slightly exceeded the recommended value of 20 ft/sec 15-degree impacts, and the longitudinal value was well below the recommended limit of 30 ft/sec. Vehicle damage was considered moderate for a 25-degree impact on such a stiff barrier; it included damage to the hood, grill, bumper, right front fender, and right front wheel and suspension, with minor sheet metal damage along the right side. Barrier damage was heavy. Two rail sections were damaged, and four posts--3, 4, 5, and 6--were bent back, with the anchor bolts broken out of the curb. On basis of the results of Test 64, the railing appeared to be stiffer than necessary. Post size thus was reduced to W6x9 with 3/4-in. base plates and post spacing increased to 8 ft 4 in. In addition, the railing was moved back so the face of the thrie beam was 6 in. behind the curb. This provided 9 in. to the center of the anchor bolts and was intended to eliminate the severe anchor-bolt breakout encountered in the first test.

In Test 65, the 4,500-lb Plymouth station wagon impacted 0.8 ft downstream from Post 2 at 58.8 mph and 27 degrees. Contact distance was 17.4 ft, with maximum dynamic deflection of 1.4 ft. The vehicle was again smoothly redirected at a 16-degree exit angle, 4-degree maximum roll, 5-degree pitch, and no measurable yaw. Impact severity was similar to that of the previous test, with peak 50-msec average decelerations of 4.8 g longitudinal and 9.3 g lateral. Occupant impact velocities were 18.2 ft/sec longitudinal and 21.5 ft/sec lateral. Vehicle damage was again moderate and similar to that in Test 64. Barrier damage, however, was significantly less (Figure 2). Two rail sections were again damaged, as well as two posts. However, the posts were bent above the base plates, with no damage to the anchor bolts or curb. Repair thus required simply unbolting and replacing the two damaged posts and no curb repair or anchor-bolt replacement was needed.

Tests 64 and 65 results thus confirmed that post spacing could be increased from the 4 ft 3 in. originally calculated. However, it appeared that the 8-ft 4-in. spacing was about the upper limit with the 10-gauge thrie-beam rail, and wider spacing might permit excessive rail deflection, resulting in pocketing or snagging at posts. To evaluate occupant risk, the barrier was rebuilt with the same design for Test 66. The 1,860-lb Subaru sedan impacted 2.1 ft downstream from Post 2 at 59.6 mph and 14 degrees. Barrier contact was only 8.1 ft, with a maximum dynamic deflection of 0.25 ft. The vehicle was smoothly redirected at a 9-degree exit angle, with maximum roll, pitch, and yaw of 10, 4, and 7 degrees, respectively. Peak 50-msec average decelerations were 3.8 g longitudinal and 11.9 g lateral. Although the lateral 50-msec average value is high compared with TRB Circular 191 criteria (7), lateral occupant impact velocity was 23.4 ft/sec, only slightly exceeding the recommended value in NCHRP Report 230. The 2-ft impact distance was not reached in the longitudinal direction. Barrier damage was superficial, limited to minor dents on the bottom corrugation of the impacted section. Vehicle damage was moderate, including the bumper, grill, hood, right front fender and wheel, and suspension.

W-Beam-to-Thrie-Beam Transition

After completion of the curbed bridge-rail tests, the guiderail-to-bridge-rail transition was tested. It is anticipated that the thrie-beam bridge rail will have maximum application for situations in

TABLE 1 Test Results: Tests 64-69

Item	Test 64	Test 65	Test 66	Test 67	Test 68	Test 69
Point of impact	1.9' downstream from No. 3 bridge post	.8' downstream from No. 2 bridge post	2.1' downstream from No. 2 bridge post	8.75' upstream from W-beam/ transition conn.	4.65' upstream from W-beam/ transition conn.	6.8'upstream from W-beam/ transition conn.
Barrier Length, ft	132	132	132	132	132	132
Vehicle Weight, lb	4500	4500	1860	4500	4500	4600
Vehicle Speed, mph	60.1	58.8	59.6	58.8	59.5	54.4
Impact Angle, deg	26	27	14	25	24	26
Exit Angle, deg	8	16	9	*	*	17
Exit Speed, mph	46.9	48.9	53.5	*	*	33.4
Max. Roll, deg	15	4	10	-15	-14	6
Max. Pitch, deg	5	5	4	6	7	4
Max. Yaw, deg	21	0	7	-11	4	0
Contact Distance, ft	15.5	17.4	8.1	25	23.4	24.3
Contact Time, ms	272	462	234	N.A.	N.A.	617
Deflection, ft						
Dynamic (from film)	1.1	1.4	.25	4.0	2.5	2.5
Permanent (measured)	.85	1.2	.25	4.0	2.5	2.3
Decelerations, g's						
50 ms avg.						
Longitudinal	5.8	4.8	3.8	9.9	9.2	4.5
Lateral	8.4	9.3	11.9	4.4	4.2	6.0
Max. Peak						
Longitudinal	28.5	9.9	10.3	28.3	14.7	9.7
Lateral	28.6	19.0	17.5	45.0	16.5	11.9
Occupant Ridedown						
Longitudinal	-6.9	2.1	2' not reached	12.9	10.7	6.9
Lateral	10.2	11.3	10.2	11.9	8.6	8.7
Occupant Imp. Vel., fps						
Longitudinal (2.0 ft)	19.4	18.2	2' not reached	30.8	25.6	20.8
Lateral (1.0 ft)	21.2	21.5	23.4	14.8	14.6	13.9
Results and Comments	Good redirection; moderate damage to barrier & car	Good redirection; moderate damage to barrier & car	Good redirection; light damage to car, barrier mod.	*Vehicle did not exit	*Vehicle did not exit	Good redirection; light damage to car, barrier mod.

*did not exit.
**film data indicated 45.3 mph.

TABLE 2 Test Results: Tests 70-75

Item	Test 70	Test 71	Test 72	Test 73	Test 74	Test 75
Point of impact	3.5' upstream from W-beam/ transition conn.	2.3' upstream from W-beam/ transition conn.	thrie-beam/ transition connection	3' downstream from thrie-beam/ transition conn.	5.4' downstream from No. 1 bridge post	No. 2 bridge post
Barrier Length, ft	132	132	132	132	132	132
Vehicle Weight, lb	1980	1800	4380	4500	1900	4500
Vehicle Speed, mph	57.8	60.3	57.0	56.5	58.5	59.3
Impact Angle, deg	20	19	28	29	18	29
Exit Angle, deg	*	11°	*	16	6°	14
Exit Speed, mph	*	44.1	*	34.9**	52.8	44.1
Max. Roll, deg	-180	5	-10	9	12	-36
Max. Pitch, deg	10	1	2	4	3	-8
Max. Yaw, deg	71	5	-5	0	0	0
Contact Distance, ft	19.6	20	12.6	16	11.1	24.7
Contact Time, ms	533	360	477	565	259	400
Deflection, ft						
Dynamic (from film)	.5	.5	1.5	1.25	0	1.75
Permanent (measured)	.35	.5	1.33	1.21	0	1.65
Decelerations, g's						
50 ms avg.						
Longitudinal	8.8	6.3	N.A.	5.2	3.0	6.1
Lateral	4.5	6.0	N.A.	8.9	8.9	7.8
Max. Peak						
Longitudinal	19.6	16.2	N.A.	20.4	5.3	14.8
Lateral	15.5	34.0	N.A.	39.0	15.7	18.4
Occupant Ridedown						
Longitudinal	13.0	4.7	N.A.	6.5	2' not reached	-7.6
Lateral	8.7	10.1	N.A.	13.1	8.8	13.9
Occupant Impact Velocity, fps						
Longitudinal (2.0 ft)	29.2	19.3	N.A.	22.0	2' not reached	22.5
Lateral (1.0 ft)	13.9	17.9	N.A.	20.5	18.7	18.0
Results and Comments	Vehicle snagged on heavy-post; rolled over	Good redirection; light damage to barrier, car mod.	Snagged on No. 1 bridge post; data cable sheared	Moderate damage to car & barrier	No curb; good redirection	No curb; good redirection

*did not exit.
**film data indicated 45.3

FIGURE 1 Thrie beam mounted on W6x25 post for Test 64.

FIGURE 2 Bridge-rail damage from Test 65.

which W-beam guiderail is used. Thus, the next test series examined the transition from W-beam guiderail on S3x5.7 steel posts to the thrie-beam approach rail. Standard mounting height on the W-beam had been 33 in. previously, matching the top of the thrie beam. A transition piece was thus needed that compensated for the depth change of about 8 in. on the bottom of the section. Figure 3 shows the first transition tested. The tapered section was 4 ft 2 in. between post bolt holes, and the lower corrugation terminated in a 12-in. taper. A filler piece was added to the exposed end of the lower corrugation. Post spacing of the S3x5.7 steel posts upstream of the taper was reduced to 3 ft 1.5 in. and then to 2 ft 1 in. Two S3x5.7 posts were installed behind the tapered section, and then three W6x9 posts behind the thrie-beam approach rail spaced at 3 ft 1.5 in. The 6-in. concrete curb on the bridge approach was turned under the tapered section on a 10-ft radius and terminated behind the rail.

In Test 67, the 4,500-lb Plymouth station wagon impacted 8.75 ft upstream from the tapered section at 58.8 mph and 25 degrees. Initially, the vehicle had started to redirect until it encountered the tapered transition. At that point, the right front wheel and suspension snagged against the end of the lower thrie-beam corrugation. Dynamic deflection up

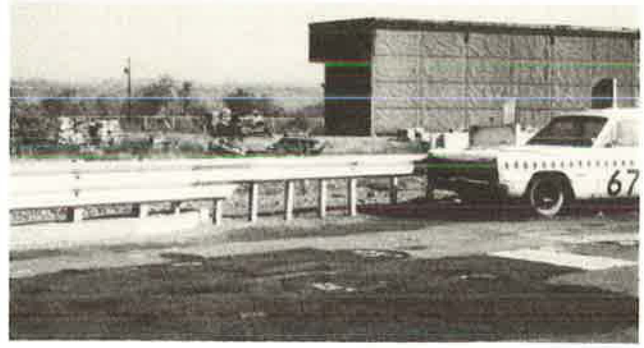


FIGURE 3 W-beam-to-thrie-beam transition evaluated in Test 67.

to that point was about 4 ft, but after the snag the longitudinal load was so great that the W-beam broke in tension at the connection to the tapered section. The W-beam section, tapered transition, and thrie-beam approach section all disconnected from the posts as they were knocked down by the vehicle and rotated back about the first bridge post. The vehicle came to rest against the first bridge post (Figure 4). Peak 50-msec average decelerations were 9.9 g longitudinal and 4.4 g lateral, with occupant impact velocities of 30.8 ft/sec longitudinal and 14.8 ft/sec lateral. The vehicle was heavily damaged, and the entire transition railing was totally destroyed. In addition, the curb was broken and displaced by the impact in the transition area, and the first two bridge posts were bent. Tensile tests were performed on two W-beam samples after the failure. Yield strengths were 59,000 and 62,500 psi with elongations of 23 and 20 percent, both exceeding the specified minimums of 50,000 psi and 12 percent.

For Test 68, the length of the tapered transition was increased to 6 ft 3 in. between mounting bolt holes, with the lower corrugation taper increased to 3 ft (Figure 5). Light posts were again used behind the transition piece spaced at 2 ft 1 in. The increased taper length was intended to reduce the severe snag encountered in Test 67, but it was not successful. The 4,500-lb Ford station wagon impacted at 59.5 mph and 24 degrees 4.6 ft upstream from the tapered section. The vehicle again snagged on the end of the lower corrugation after beginning to redirect. The front wheel and suspension were again trapped between the bottom of the thrie beam and the top of the curb, and the vehicle came to rest against the first bridge post. The rail did not break, but all the posts in the transition area were bent over, and the rail was deflected 2.5 ft. The vehicle traveled 23.4 ft from impact to rest. Peak 50-msec average decelerations were 9.2 g longitudinal and 4.2 g lateral. Occupant impact velocities were 25.6

FIGURE 4 Results of Test 67.

FIGURE 5 W-beam-to-thrie-beam transition evaluated in Test 68.

ft/sec longitudinal and 14.6 ft/sec lateral. The front and right side of the vehicle were heavily damaged.

After Test 68, it became clear that the termination of the lower corrugation presented an insurmountable snag point, and the entire transition was redesigned to include the symmetrical tapered section shown in Figure 6. In this section the middle corrugation drops out and the upper and lower ones continue. This has been adopted as a standard by the American Road and Transportation Builders Association (ARTBA) (8). Because the 8-in. decrease in section depth was split equally between top and bottom, a total height adjustment of 4 in. was required in the top-of-barrier elevation. Sufficient tolerance in the splice bolt holes permitted the tapered section to be tipped up about 1 in., and the remaining 3 in. was gained by lowering the top of the W-beam over one section length. Standard height to the top of the W-beam has since been decreased to 30 in., thus simplifying this connection. As in previous tests, S3x5.7 posts were used with the W-beam and at the beginning of the 6-ft 3-in. tapered section. Mid-length and at the downstream end of the taper, W6x9 posts were substituted to stiffen the transition, and 6-in.-deep blockouts were added to reduce contact with the posts and help maintain rail height during deflection. Finally, the vertical curb radius was replaced with a ramped curb end that terminated the curb 6 ft upstream from the bridge.

FIGURE 6 Symmetrical transition evaluated in Test 69.

In Test 69, the 4,600-lb Cadillac sedan impacted at 54.4 mph and 26 degrees 6.8 ft upstream from the tapered section. This time, the vehicle was smoothly redirected at a 17-degree angle. However, some contact with the heavy posts did occur in spite of the 6-in. blockouts, and exit speed was only 33.4 mph on the basis of accelerometer data. Maximum dynamic deflection was 2.5 ft over the 24.3-ft contact distance. The vehicle turned back toward the rail after exiting and came to rest against the downstream terminal. Maximum roll was 6 degrees, pitch was 4 degrees, and no yaw was observed. Peak 50-msec average decelerations were 4.5 g longitudinal and 6.0 g lateral. Occupant impact velocities were 20.8 ft/sec longitudinal and 13.9 ft/sec lateral. Damage to the vehicle was limited to the right front corner of the grill and bumper, right front fender, and right front wheel. Moderate barrier damage included three light posts and four heavy posts partially bent over, as well as one W-beam, one thrie beam, and the tapered section.

On the basis of the previous test, it appeared that the W-beam-to-thrie-beam transition now performed reasonably well with a large car, although slight post contact was noted. The transition was rebuilt for the small-car test. In Test 70, the 1,980-lb Subaru station wagon impacted at 57.8 mph and 20 degrees 3.5 ft upstream from the tapered section. On impact, the front bumper was under the W-beam, and sheet metal deformation on the front fender was sufficient to allow the front wheel also to protrude under the rail. The S3x5.7 posts collapsed with little resistance when hit by the front of the car, but the bumper and wheel had intruded far enough under the rail that they also contacted the first W6x9 post--in spite of the 6-in. blockout--and a severe snag resulted. On impacting the first heavy post, the car yawed sharply clockwise and rolled over, coming to rest on its roof in front of the barrier. Peak 50-msec average decelerations were 8.8 g longitudinal and 4.5 g lateral, with corresponding occupant impact velocities of 29.2 and 13.9 ft/sec. Damage to the barrier was light, with one light post bent over, three heavy posts dented, and minor scratches on the tapered section. Total barrier contact distance was 19.5 ft, and dynamic deflection was 6 in. The vehicle was heavily damaged.

Although the 6-in. blockout might be adequate in some situations, it did not work well in this case when the bumper and front wheel were able to intrude under the W-beam upstream from the heavy post section. Even though the impact angle was somewhat more severe than the 15 degrees intended, this barrier was considered unacceptable, and design revisions were needed. Testing conducted by the Texas Transportation Institute (9) had shown that a deeper blockout on thrie-beam guiderail performed well in both small sedan and bus tests. Therefore, a welded blockout was added to the heavy posts to increase blockout depth to 14 in. The TTI tests had used an M14x17.2 rolled shape for the blockout. For these tests, 0.25-in. plate was welded to fabricate a blockout closely approximating the rolled shape. In addition, a notch was provided in the lower front of the blockout web. This notch was intended to perform two functions. First, it would provide a collapse mechanism to absorb part of the impact energy, especially with small cars, and second, it would permit the lower corrugation to rotate inward and thus remain more nearly vertical during severe impacts by large vehicles.

In Test 71, the 1,800-lb Honda sedan impacted the barrier 2.8 ft upstream from the tapered section at 60.3 mph and 19 degrees. After 20 ft of barrier contact, the vehicle redirected at an 11-degree exit angle, with maximum 50-degree roll, 1-degree pitch, and 5-degree yaw. Peak 50-msec average decelerations were 6.3 g longitudinal and 6.0 g lateral, with corresponding impact velocities of 19.3 and 17.9 ft/sec. Maximum dynamic barrier deflection was 6 in., and barrier damage was light--two light posts were bent over, one heavy post was dented, and the W-beam and tapered section were scuffed. Even with the 14-in. blockout, the vehicle made slight contact with the second heavy post. Vehicle damage was moderate, confined to the right front corner.

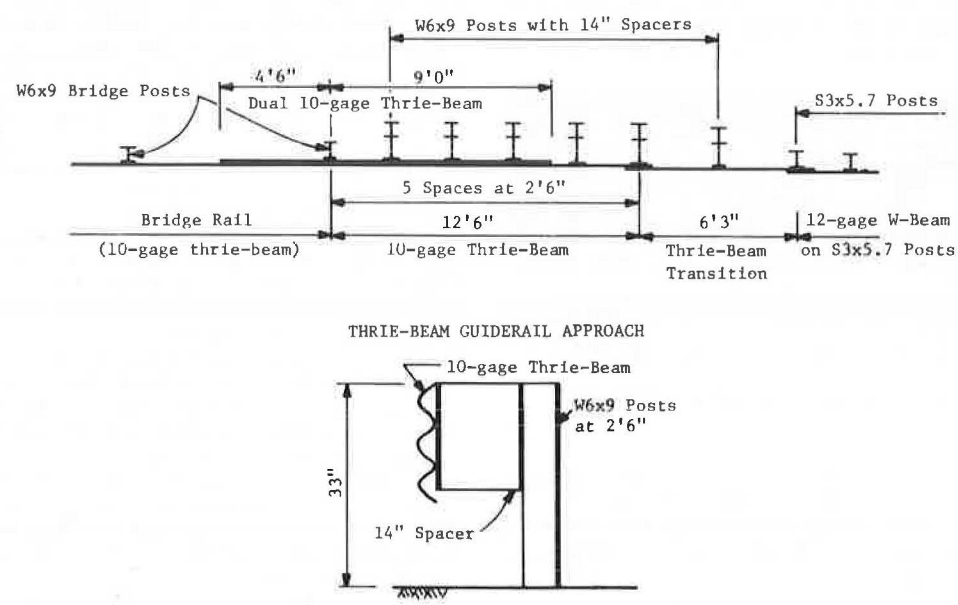
It thus appeared that the symmetrical tapered section combined with the 14-in. blockouts provided adequate performance at the W-beam-to-thrie-beam transition. Because this transition had performed reasonably well with only a 6-in. blockout in the large-car test, it was not considered necessary to retest the deep blockout with a large car. Testing thus proceeded to the thrie-beam bridge approach.

Thrie-Beam Bridge-Approach Section

After the transition from the very flexible W-beam guiderail to the relatively stiff thrie-beam guiderail had been completed, another transition was necessary to the very stiff bridge rail. For the initial design, a single 10-gauge thrie beam spanned from the tapered section to the first bridge post. W6x9 posts with notched 14-in. blockouts were spaced at 3-ft 1.5-in. centers, and one additional W6x9 post was placed at the end of the bridge to provide increased support at that critical point. As in the previous test, the blockout web was notched. In Test 72, the 4,380-lb Plymouth station wagon hit at the downstream end of the tapered section--12.5 ft from the first bridge post--at 57.0 mph and 28 degrees. The barrier deflected 1.5 ft as the vehicle approached the bridge, but this was excessive considering the very stiff railing at the end of the bridge and a pocketing condition developed. In addition to severe pocketing, the right front wheel and suspension snagged on the unblocked post at the end of the curb ramp, and the vehicle stopped very abruptly 20 ft onto the bridge. The car was extensively damaged, including partial collapse of the passenger compartment and roof. Electronic data were lost because of a cable break. In spite of the severe damage to the vehicle, the railing was only moderately damaged, with several posts pushed back, one post bent, and one rail section damaged. In addition, the front flange of the blockouts collapsed against the web notch in the impact area, permitting the lower railing corrugation also to collapse. It appeared that this notch in the blockout contributed to the snag on the post at the end of the ramped curb, because clearance was not maintained between the face of the rail and the posts.

Even if the blockout collapse had not contributed to the snag on the unblocked post, it appeared that with a 1.5-ft deflection, the vehicle would have pocketed at the first bridge post. Thus, it was necessary gradually to increase barrier stiffness in the bridge-approach transition. To accomplish this, a backup section of 10-gauge thrie beam was added behind the thrie-beam approach rail, extending 4.5 ft onto the bridge from the first bridge post and 9 ft upstream into the approach section. Full 12-bolt splices were used at each end to attach it to the front thrie-beam section. The notches in the post blockout were eliminated, and post spacing behind the approach thrie beam was decreased to 2.5 ft. Finally, the last guiderail post at the upper end of the curb ramp was eliminated, because fill conditions at the bridge end would not normally permit it to be installed behind the curb with a blockout. As seen in the previous test, installation on the curb without a blockout presented a potential snag point. Details are shown in Figure 7.

In Test 73, the 4,500-lb Plymouth sedan impacted at 56.5 mph and 29 degrees 3 ft downstream from the tapered section, 9.5 ft from the first bridge post. This time, vehicle redirection was achieved at a 16-degree exit angle after 16 ft of barrier contact and 15-in. dynamic deflection. After exiting, the vehicle turned back toward the rail. Maximum roll was 9 degrees, pitch was 4 degrees, and no yaw was observed. Peak 50-msec average decelerations were 5.2 *g* longitudinal and 8.9 *g* lateral, with corresponding occupant impact velocities of 22.0 and 20.5 ft/sec. The right front corner of the vehicle was extensively damaged, but there was no damage to the passenger compartment. The rail was also extensively damaged, with the posts behind the approach thrie beam pushed back. In addition, the last two guiderail posts twisted 90 degrees, permitting the rail to collapse against the side of the post. Vehicle and barrier damage are shown in Figure 8. Use of heavier bolts to attach the rail to the blockout may have prevented this twisting. In addition, the welds



Note: Spacer used here is similar to spacer detail in Figure 7 of NYSDOT Research Report 118 *(10)*, except 6-in. notch is omitted.

FIGURE 7 Details of final transition design evaluated in Test 73.

FIGURE 8 Vehicle and barrier after impact on guiderail-bridge-rail transition, Test 73.

attaching the first bridge rail to the anchor plate sheared off, permitting that post to deflect back about 1 ft. The welds at the second post cracked but did not separate. There was some concern that this failure at the first post may have helped to reduce the pocketing experienced in the previous test. However, it appears that pocketing would have simply occurred at the second bridge post if deflection of the approach rail had been excessive. It thus appears that the backup rail section and closer post spacing increased transition stiffness enough to prevent pocketing. Performance of the entire bridge-approach rail was now considered satisfactory with both small and large cars.

Tests Without Curb

In Tests 64 and 65, it appeared that part of the impact was absorbed by the 6-in. curb. Eliminating the curb while maintaining rail height at 33 in. thus may result in increased impact forces on the rail. In addition, the 13-in. gap between pavement and rail might permit post snagging, especially by small vehicles with 13-in. wheels. Existing structures where this upgrading system would be used sometimes have no curbs. In addition, it was believed that this rail system might offer good performance on new curbless bridges. Both large- and small-car tests thus were planned to evaluate this design. Other than removing the curb and increasing post length to maintain rail height, the barrier design was the same as that in Tests 65 and 66.

In Test 74, the 1,900-lb Subaru sedan impacted at 58.5 mph and 18 degrees 5.4 ft past the first bridge post. After 11.1 ft of contact, the vehicle exited at 6 degrees with maximum roll of 12 degrees and 3-degree pitch. No measurable vehicle yaw was observed, and no dynamic deflection of the bridge rail. After exiting, the vehicle turned away from the barrier and then continued along essentially a straight line away from the rail. Peak 50-msec average decelerations during impact were 3.0 g longitudinal and 8.9 g lateral. The lateral occupant impact velocity was 18.7 ft/sec, but the 2-ft longitudinal impact distance was not reached. Damage to the barrier was limited to minor scrapes, and the car sustained only light sheet-metal damage.

In Test 75, the final test in this series, the 4,500-lb Buick sedan impacted at 59.3 mph and 29 degrees at the second bridge post. After 24.7 ft of contact and 21-in. dynamic deflection, the vehicle exited at 14 degrees with a maximum roll of -36 degrees (away from the barrier) and -8-degree pitch, but no measurable yaw. Peak 50-msec average decelerations were 6.1 g longitudinal and 7.8 g lateral, with corresponding occupant impact velocities of 22.5 and 18.0 ft/sec. Although the vehicle was redirected, it is apparent that this test, which somewhat exceeded the standard 25-degree impact angle, was at the upper limit of performance for this barrier. Post 3 (the first post past impact) was bent back at the base plate, with a permanent deflection of 20 in. at the top of the post. The square plate washers prevented the rail attachment bolts from pulling through, and the rail also bent back and formed a partial ramp. The vehicle was thus pitched up on redirection, with all four wheels in the air as it left the rail. Two rail sections and two posts were damaged, and extensive damage was sustained by the right front corner of the vehicle (Figure 9).

DISCUSSION AND FINDINGS

On the basis of these tests, it appears that the proposed bridge rails essentially meet the recom-

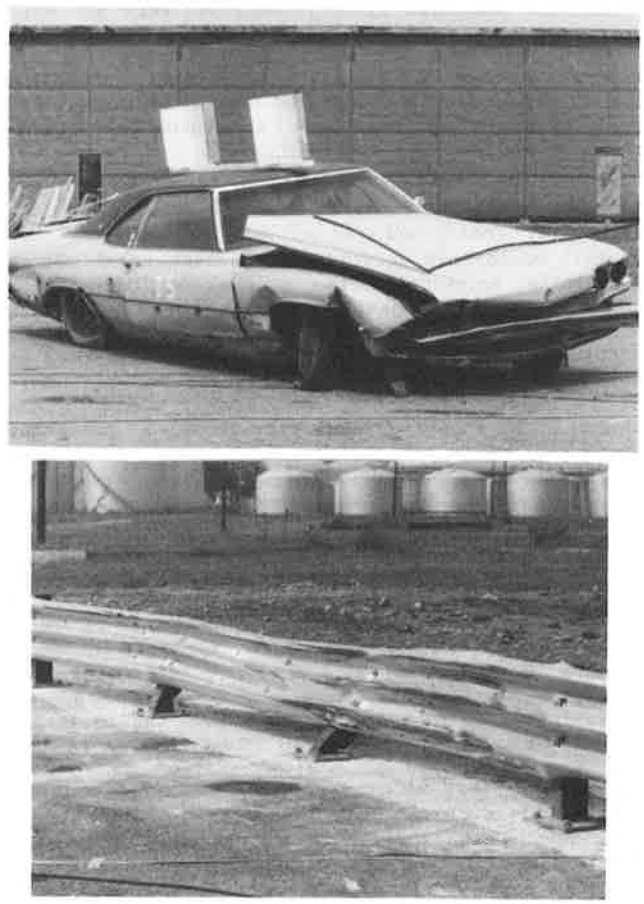


FIGURE 9 Vehicle and barrier after impact on bridge rail, Test 75.

mended evaluation criteria of NCHRP Report 230, both with and without a 6-in. curb. Table 3 of that report recommends tests with 4,500-lb sedans and either a 2,250- or 1,800-lb sedan. For transitions, only a single 4,500-lb test is recommended, but this transition was evaluated at two points with a 4,500-lb sedan and at one with an 1,800-lb sedan.

Test 64 indicated compliance with NCHRP Report 230, but that design was stronger than necessary. Four other tests resulted in clearly unsatisfactory results. The seven remaining tests indicated that the final railing and transition designs were generally in compliance with NCHRP Report 230. All seven tests met Criterion A, which requires smooth redirection. Vehicle trajectory was somewhat marginal in Test 75--the final test of the curbless rail--because of moderate vehicle vaulting. However, considering that the impact angle was 29 degrees rather than the 25 degrees specified, these results appear acceptable. Criteria D and E were easily met in all seven tests.

Criterion F provides suggested values for occupant impact velocities and ridedown accelerations and applies only to the 1,800-lb vehicle. Results of the four successful tests of the 4,500-lb sedan were close to meeting the suggested values, even though impact angles were much more severe. No problem was encountered with the 1,800-lb tests in meeting the suggested longitudinal impact velocity of 30 ft/sec. In fact, in two of three tests, the 2-ft flail space was not reached. Although Tests 71 and 74 were comfortably below the recommended lateral value of 20 ft/sec, it was exceeded slightly in Test 66. However, the 1-ft flail space appears unrealistically large for small sedans, and use of a more realistic number would have reduced this value. Ridedown deceleration values for the three 1,800-lb vehicles were all well below the recommended values. Thus, with two minor exceptions--slight vehicle vaulting in Test 75 and lateral occupant impact velocity slightly exceeding the desired value in Test 66--the bridge-rail and transition designs meet the recommended structural adequacy and occupant risk criteria.

The vehicle trajectory criteria (H and I) are the most difficult to meet. In every test, the vehicle eventually redirected far enough from the barrier that it would have entered the adjacent travel lane, especially considering the narrow shoulders typically found on bridges. However, as pointed out in NCHRP Report 230, this performance factor is difficult to assess. For example, even if redirected at a flat angle such as 7.5 degrees, the vehicle would be 13 ft away from the barrier only 100 ft downstream if no steering input were provided. Thus, the suggested values in Criterion I (exit speed and angle) probably provide a more realistic assessment of performance. The tests generally met the recommended values of no more than 15-mph speed change and exit angle less than 60 percent of impact angle. Two tests were marginal in terms of exit angle (Tests 66 and 69) with Test 66 exceeding the desired value by 0.5 degree and Test 69 by 1.5 degrees. In terms of exit speed, accelerometer data for Test 73 indicated a speed loss of 21.6 mph, but film data, considered more reliable, indicated 11.2 mph, an acceptable value. In Test 71, speed loss was 16.2 mph, slightly exceeding the desired 15-mph value. In Test 69, speed loss was 21 mph, exceeding the desired value by several miles per hour. However, substitution of 14-in. blockouts for the 6-in. blockouts in subsequent tests would help to reduce post contact, and it is expected that a retest at this point on the final design would have resulted in a lesser speed loss.

The four unsuccessful tests--67, 68, 70, and 72--also contributed valuable insight into the performance of guiderail-to-bridge-rail transitions. Tests 67 and 68 dramatically pointed out the snagging potential created by terminating the lower thrie-beam corrugation at the W-beam transition. A symmetrical transition carrying through the lower transition was needed to eliminate this snag. These tests further demonstrated the need for deep blockouts to prevent wheel snag during the transition from light-post to heavy-post systems. Even the 20-in.-deep thrie-beam section was unable to prevent snagging without the 14-in. blockout. Finally, the need to increase lateral stiffness gradually to a level close to that of the bridge rail was evident. Even when all the snag points were eliminated, pocketing occurred at the first bridge post in Test 72. Stiffening the approach section by adding a rail backup section eliminated the pocketing problem in the subsequent test.

The bridge railings and transitions developed in the project appear to perform adequately in full-scale tests and with minor exceptions meet or exceed the recommended performance standards in NCHRP Report 230. The railing system can be used either with a 6-in. curb or on curbless decks, and can be attached on existing decks without contact with existing railing or structural members. The transition developed makes it possible to match the railing to light-post W-beam guiderail on the bridge approach, thus providing a complete barrier system. This system simplifies maintenance inventory problems by using components common to other systems to the greatest extent possible. The 10-gauge thrie beam is used on other NYSDOT railings. By avoiding the use of tubular thrie beam, the splice is simplified and inventory requirements are reduced. With the exception of the bridge posts, 14-in. blockouts, and tapered transition piece, the remaining hardware is common to current guiderail and bridge-rail systems.

Based on the 12 tests performed in this project, the following findings can be stated:

1. Bridge rail consisting of 10-gauge thrie-beam rail mounted on W6x9 steel posts spaced at 8 ft 4 in. generally met NCHRP Report 230 recommended evaluation criteria with or without a 6-in. curb.
2. A W-beam-to-thrie-beam guiderail transition on the bridge approach, including a symmetrical tapered transition piece and 14-in.-deep blockouts, performed satisfactorily.
3. W-beam-to-thrie-beam transitions in which the lower thrie-beam corrugation was dropped resulted in severe snagging.
4. A blockout depth of 14 in. was required in the W-beam-to-thrie-beam transitions and throughout the bridge approach to prevent snagging on the W6x9 posts.
5. Notching the lower front corner of the 14-in. blockout web appeared to reduce performance in the 4,500-lb vehicle test by permitting the lower thrie-beam corrugation to collapse toward the posts. This contributed to the vehicle's snagging on an unblocked post in Test 72.
6. A double layer of 10-gauge thrie beam was required in the bridge approach to provide adequate lateral stiffness at the first bridge post.

For a full explanation of testing procedures, data analysis, and test results, the reader is referred to NYSDOT Research Report 118 (10).

ACKNOWLEDGMENTS

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REFERENCES

1. J.E. Bryden and K.C. Hahn. Crash Tests of Light-Post Thrie-Beam Traffic Barriers. Research Report 85. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, March 1981.
2. J.E. Bryden and K.C. Hahn. Crash Tests of Box-Beam Upgradings for Discontinuous-Panel Bridge Railing. Research Report 92. Engineering Research and Development Bureau, New York State Department of Transportation, Sept. 1981.
3. J.D. Michie and M.E. Bronstad. Upgrading Safety Performance in Retrofitting Traffic Railing Systems. Report FHWA-RD-77-40. Southwest Research Institute, San Antonio, Tex., Sept. 1976.
4. M.E. Bronstad, L.R. Calcote, C.E. Kimball, Jr., and C.F. McDevitt. Development of Retrofit Railings for Through Truss Bridges. In Transportation Research Record 942, TRB, National Research Council, Washington, D.C., 1983, pp. 1-10.
5. R.M. Olson et al. Texas T-1 Bridge Rail Systems. Technical Memorandum 505-10. Texas Transportation Institute, Texas A&M University System, College Station, April 1971.
6. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.
7. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. Transportation Research Circular 191. TRB, National Research Council, Washington, D.C., Feb. 1978.
8. A Guide to Standardized Highway Barrier Rail Hardware. Technical Bulletin 268-B, AGC Standard Form 131. American Road and Transportation Builders Association, Washington, D.C., June 1979.
9. D.L. Ivey, C.F. McDevitt, R. Robertson, C.E. Buth, and A.J. Stocker. Thrie-Beam Guardrails for School and Intercity Buses. In Transportation Research Record 868, TRB, National Research Council, Washington, D.C., 1982, pp. 38-44.
10. Performance of a Thrie-Beam Steel-Post Bridge-Rail System. Research Report 118. Engineering Research and Development Bureau, New York State Department of Transportation, Albany.

Bridge Rail to Contain and Redirect 80,000-lb Tank Trucks

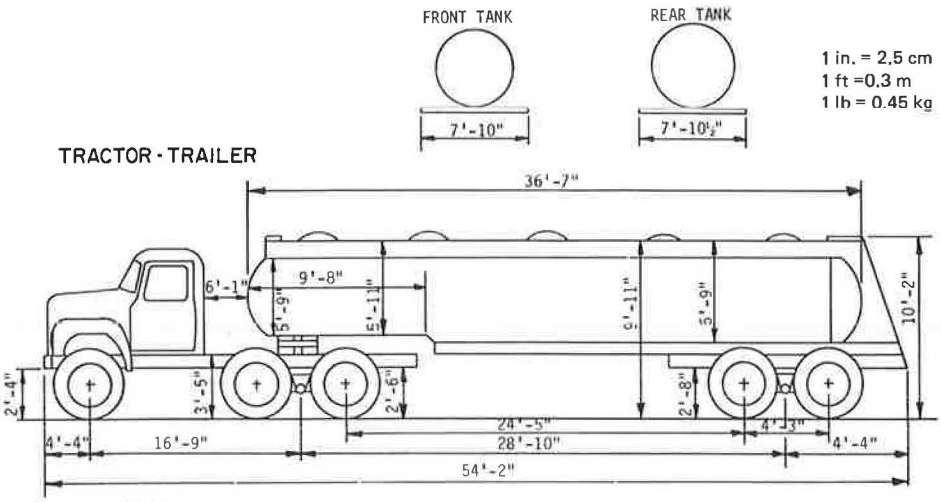
T. J. HIRSCH and W. L. FAIRBANKS

ABSTRACT

A standard Texas Type T5 traffic rail was modified to increase its height and strength to contain and redirect an 80,000-lb (36 297-kg) tank-type tractor-trailer at 50-mph (80.5-km/hr), 15-degree impacts. The height of the concrete parapet was increased to 48 in. (122 cm), and a concrete beam was mounted on concrete posts on the top of the parapet to achieve a total rail height of 90 in. (229 cm). One crash test was conducted on the bridge rail. The truck was contained and smoothly redirected. This test has shown that a bridge rail can redirect heavy tank-type trucks at speeds up to 50 mph and 15-degree impacts. The cost of this rail is estimated at about $125 per foot. Typical passenger car bridge rails in Texas now cost about $35 per foot.

Current bridge rails are designed to restrain and redirect passenger cars only. Collisions of large trucks with these bridge rails have in the past led to catastrophic accidents. Concern for the reduction of the severity of these accidents has led highway designers to devote more attention to the containment and redirection of large trucks at selected locations.

The factors involved in the design of bridge rails to contain and redirect large trucks are not nearly so well understood or researched as those involved in the design of passenger car rails.



EMPTY WEIGHT		LOADED WEIGHT	
Weight on front axle	10,590	Weight on front axle	12,070
Weight on center axles	8,030	Weight on center axles	34,050
Weight on rear axles	9,700	Weight on rear axles	34,000
Total Empty Weight	28,320	Total Loaded Weight	80,120

FIGURE 1 Tractor-trailer loaded dimensions, empty weight, and loaded weight.

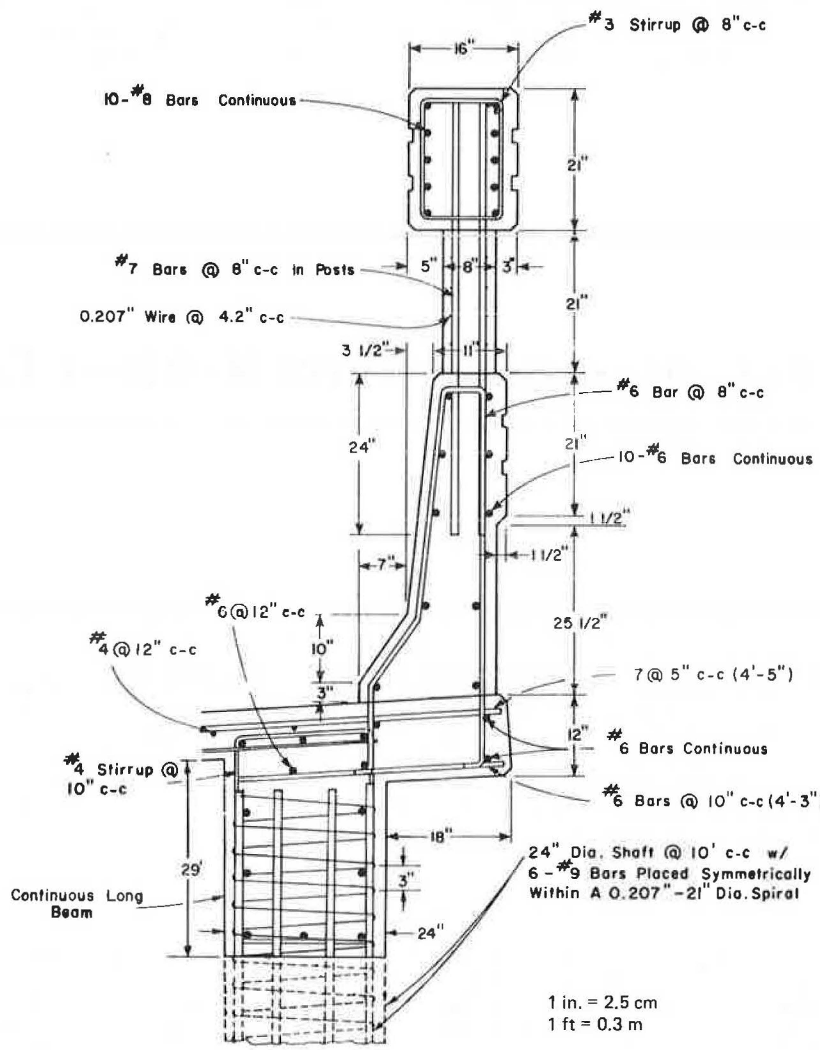


FIGURE 2 Cross section of modified T5 bridge rail and bridge deck.

Therefore, it was the objective of this project to design, build, and test a bridge rail to contain and redirect an 80,000-lb (36 297-kg) tank-type tractor-trailer, as shown in Figure 1. The design was based on data presented elsewhere (1-5).

The rail selected was a modification of the Texas Type T5 traffic rail. The modified T5 rail consists of a concrete safety-shaped parapet 48 in. (122 cm) high and a concrete beam element 16 in. (41 cm) wide and 21 in. (53 cm) deep. The concrete beam is mounted at a height of 90 in. (229 cm) on concrete posts on top of the parapet. The concrete posts are concrete walls 8 in. (20 cm) thick by 5 ft (1.5 m) long located at 10-ft (3-m) center-to-center spacing. This produces 5-ft (1.5-m) openings 21 in. (53 cm) high between posts. The beam element contains a large amount of reinforcing steel, which provides both flexibility and strength and thus minimizes cracking of the concrete and permanent deflection of the rail when impacted by heavy vehicles. The modified T5 concrete parapet can be placed in continuous lengths, which gives good structural continuity and strength. The thickness of the bridge deck below the concrete parapet was increased to 12 in. (30 cm) to minimize cracking.

The beam-and-post design was selected because of its open and aesthetic appearance. The concrete safety-shaped parapet was selected because of its past acceptable safety performance.

DESIGN TECHNIQUE

Earlier tests (1) have shown that the highest forces generated during the redirection of tractor-trailers occur when the tandem axles of the tractor and the front of the trailer impact the bridge railing. With the traffic rails tested in the past, a relatively small part of the total kinetic energy is expended in the redirection of the front axle of the tractor, and the rear tandem axles of the trailer tend to have an even smaller impact. With the knowledge that the total loaded weight on the tandem axles of the tractor would be approximately 34,000 lb (15 426 kg) (Figure 1), it was assumed that 10,000 lb (4540 kg) of this load (empty weight) would probably be transferred to the rail through the wheels and the axles. The remaining 24,000 lb (10 889 kg) (payload) would be transferred to the rail through the trailer.

Accelerometer data from past tests indicated that the tandem axles of the tractor would be subjected to a 50-msec average lateral acceleration of about 6 g. Therefore, equivalent static design forces of 60,000 lb (27 223 kg) (10,000 lb x 6 g) applied at a height of 21 in. and 144,000 lb (65 335 kg) (24,000 lb x 6 g) applied at a height of 84 in. (213 cm) were used to design the rail by using yield line theory for reinforced concrete. These procedures are outlined in Texas Transportation Institute (TTI) Research Report 230-2 (2).

DESCRIPTION OF BRIDGE RAIL AND DECK MODIFICATIONS

The modified T5 rail has a concrete beam 16 in. wide and 21 in. deep mounted on top. This modified bridge rail makes a combination bridge rail 90 in. high suitable to retain large 80,000-lb tank-type trucks or tractor-trailers impacting at 15 degrees and 50 mph. Drawings of this rail are shown in Figures 2, 3, and 4. The size of this bridge rail is compared with a 1979 Ford Thunderbird and the tank-type tractor-trailer in Figure 5. The bridge rail was constructed on a 14-degree curve, and the deck had a superelevation of 0.055 ft/ft (0.017 m/m). The rail was mounted vertically. The bridge rail was constructed in this manner, at the request of the sponsors, to closely simulate an expected installation in San Antonio, Texas.

The concrete parapet was basically a standard Texas Type T5 traffic rail that was heightened to 48 in. and thickened to 11 in. (28 cm) at the top and 20.5 in. (52 cm) at the bottom. It was anchored to the bridge deck by No. 6 stirrups spaced at 8 in. (20 cm) as shown, and 10 No. 8 longitudinal bars were used.

The concrete post was 21 in. high, 8 in. thick, and 5 ft long with 5-ft openings between posts. Each concrete post was anchored to the concrete rail by means of 16 No. 7 bars (8 on the traffic side and 8 on the field side).

The concrete beam on top of the posts was 16 in. wide and 21 in. deep for the entire length of the rail. It contained No. 3 closed stirrups spaced at 8 in. center to center and 10 No. 8 longitudinal bars.

The strength of the Texas standard bridge deck, which is 7 in. (18 cm) thick, was increased in many ways. The dimensions and reinforcement pattern of the standard bridge deck were essentially maintained throughout except in the cantilever portion of the deck. These changes are detailed in Figure 2. The length of the cantilever portion was decreased from 30 in. (76 cm) to 18 in., and the thickness was increased to 12 in. (30.5 cm). The size of the upper transverse bars was increased from No. 5 bars to No.

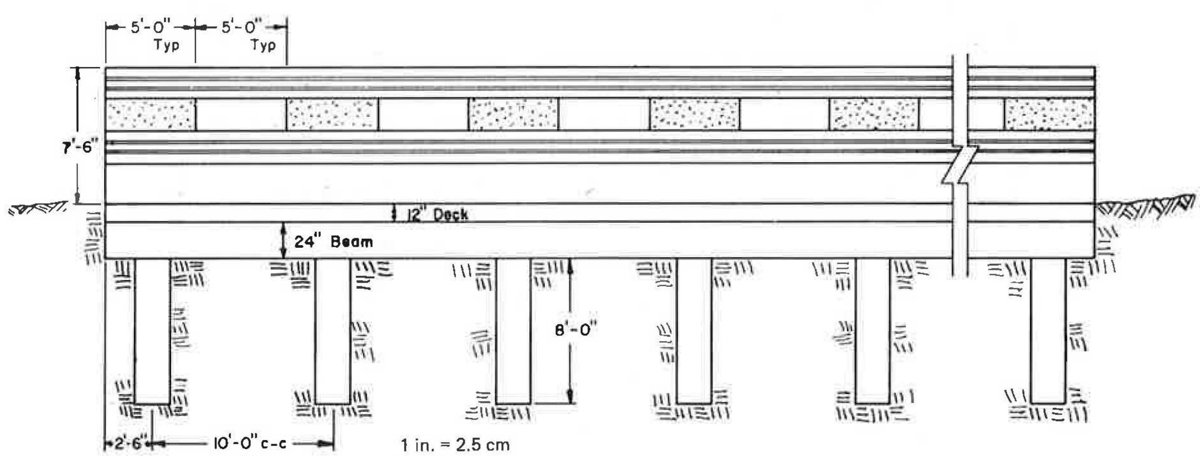


FIGURE 3 Elevation (from field side) of modified T5 bridge rail.

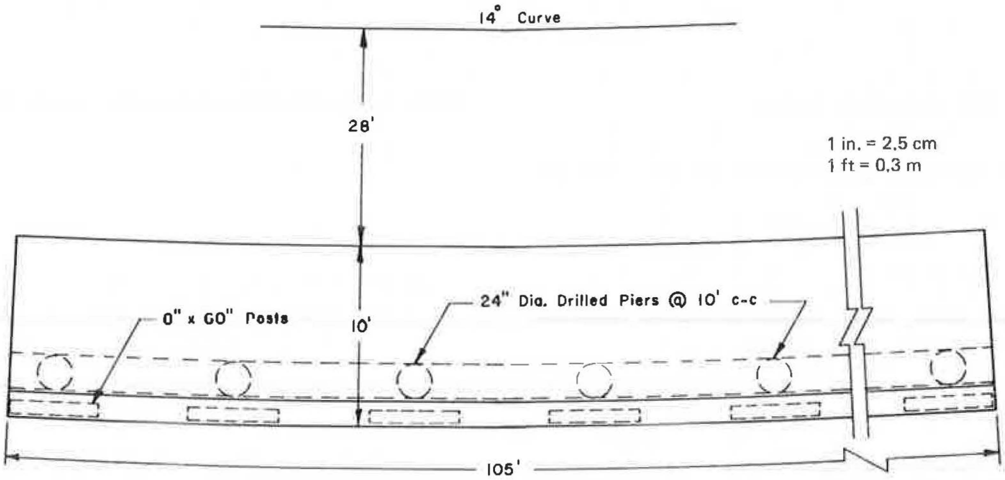


FIGURE 4 Plan view of modified T5 bridge rail, bridge deck, and pier system.

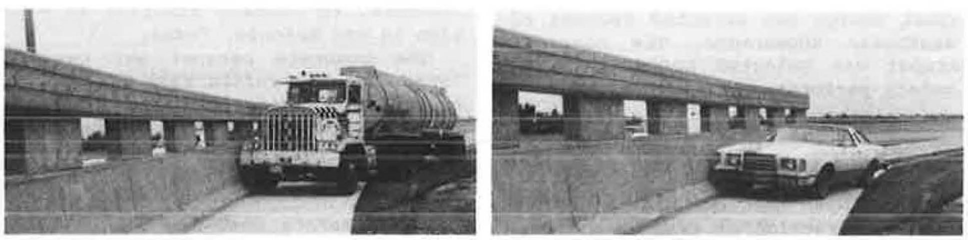


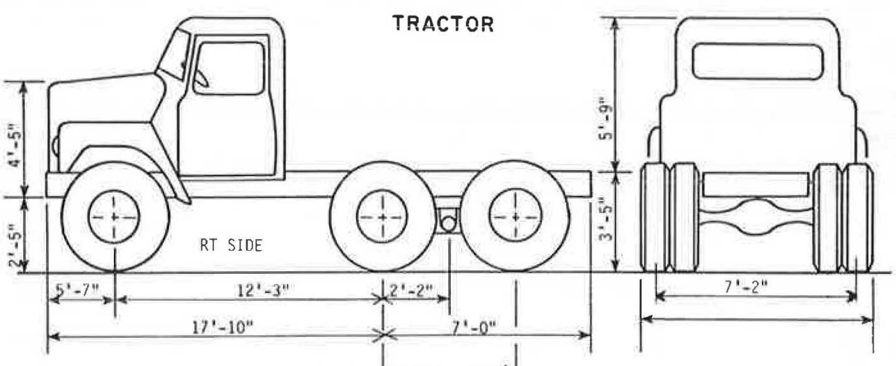
FIGURE 5 Comparison of Thunderbird and 80,000-lb tank truck with modified rail.

7, and the standard 5-in. (12.7-cm) spacing was retained. The size of the lower transverse bars was increased from No. 4 to No. 6, and the standard spacing of 10 in. (25.4 cm) was again retained. The size of the upper and lower longitudinal bars was increased to No. 6 from No. 4 and 5, respectively, and the spacing was increased from 12 to 17.5 in. (44.5 cm).

All reinforcing bars used in both the bridge deck and the rail had a minimum yield strength of 60 ksi (41.4 kN/cm^2). It should be noted that all of the 28-day compressive strengths were well above the minimum specified strength of 3,600 psi (0.25 kN/cm^2); however, the rail would have performed satisfactorily with the minimum 3,600 psi.

TRUCK CRASH TEST

This bridge rail system was designed to contain and redirect an 80,000-lb tank-type tractor-trailer. A simulated bridge deck with this rail system was built at the TTI proving grounds and tested with a 1980 Kenworth tractor-trailer ballasted with water



EMPTY WEIGHT

Weight on front axle:	Left 5,390	Right 5,200	Total 10,590
Weight on rear axles:	L F 2,040	R F 2,040	Total 4,080
	L R 1,960	R R 1,990	Total 3,950
Total Empty Weight			TOTAL 18,620

FIGURE 6 Tractor dimensions and weight after crash test.

to 80,120 lb (36 352 kg). Drawings showing the dimensions of this vehicle along with loaded and unloaded weights on each axle or pair of axles are shown in Figures 1 and 6. Photographs of the truck before and after the test are presented in Figures 7 and 8.

The truck impacted the rail at 51.4 mph (82.7 km/hr) and an angle of 15 degrees. The impact point was at the upstream edge of post 5, and the truck was smoothly redirected and remained upright. Figure 9 shows the bridge rail and test site immediately after the test. The truck entry and exit path can be seen clearly. The truck sustained damage to the right front and right tandem wheels. The cab of the truck remained intact. The trailer body was dented by the impact with the upper beam but did not rupture. The trailer did, however, sustain a small puncture (1/4 in. in diameter) from the exhaust stack of the truck immediately following impact. A summary of the crash test data is shown in Table 1.

The bridge deck supporting the rail was not significantly damaged. It was determined from the overhead film that the upper beam was deflected a maximum of 4 in. (10 cm) and sustained a permanent deflection of 0.6 in. (2 cm). Sequential photographs of the overhead and frontal views of the crash test are shown in Figure 10.

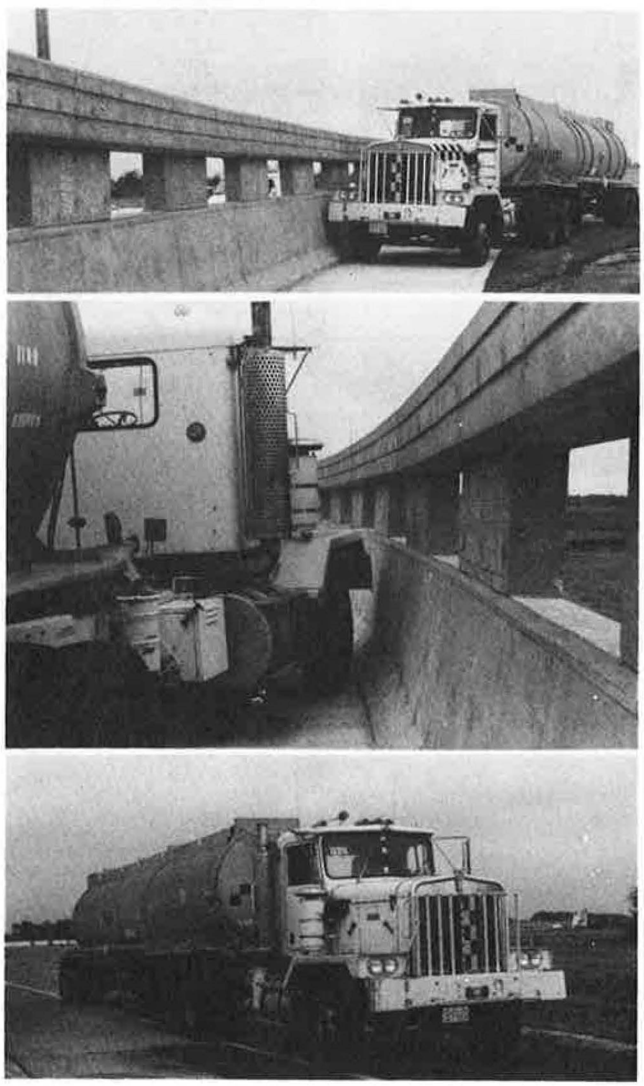


FIGURE 7 Tank truck weighing 80,000 lb before test.

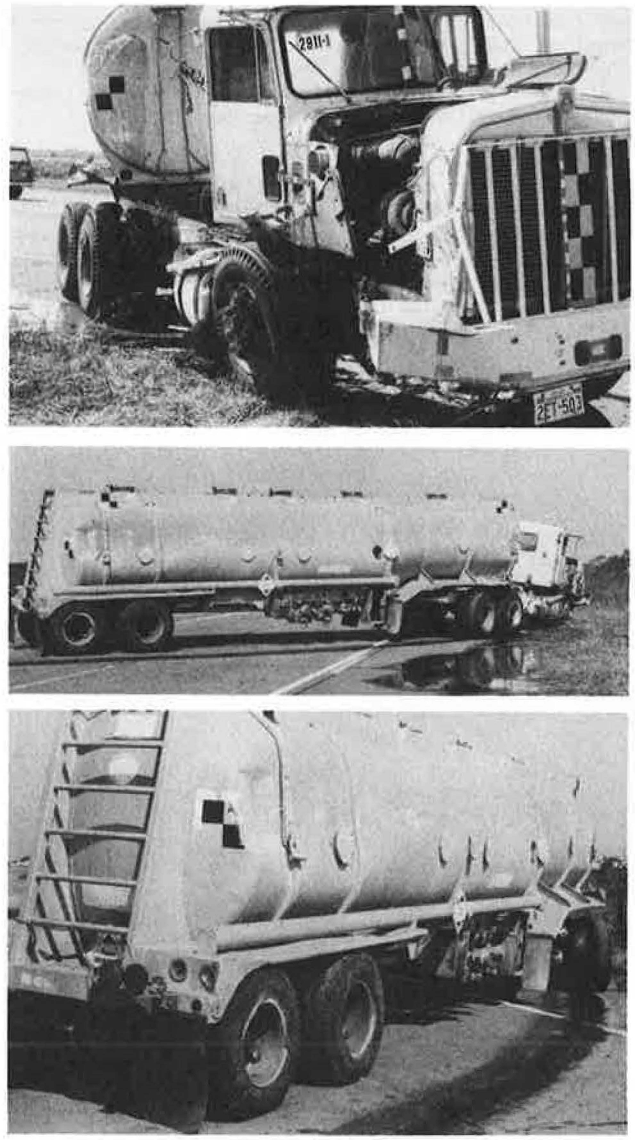


FIGURE 8 Tank truck weighing 80,000 lb after test.

The truck was equipped with roll, pitch, and yaw rate gyroscopes and x, y, and z accelerometers located above the tractor tandem wheels. Graphs of the filtered data from this instrumentation are presented in Figures 11-15.

Other data were gathered on the truck during the test. Maximum positive roll of the tractor tandem axles was 17 degrees from the roll rate gyroscopes and that of the trailer was approximately 15 degrees from the high-speed film. From the accelerometers, the longitudinal and lateral maximum average 0.050-sec accelerations were -1.77 _g_ and 5.54 _g_, respectively.

DISCUSSION OF RESULTS

NCHRP Report 230 (_6_) recommends the following criteria for Test S21 (80,000 lb/50 mph/15 degrees):

1. The test article should smoothly redirect the vehicle; the vehicle should not penetrate or go over the installation;
2. Detached elements, fragments, or other debris

from the test article should not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic; and

3. Vehicle, cargo, and debris should be contained on the traffic side of the barrier.

According to these criteria, the test was a success. The bridge rail contained and smoothly redirected the truck and remained totally intact while doing so.

Impact severity as defined by the occupant flail space approach was also computed from the accelerom-

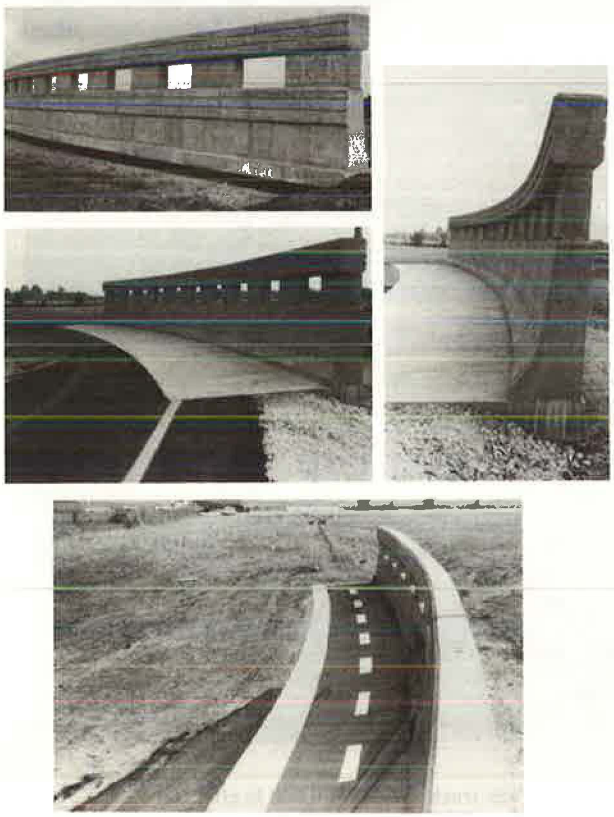


FIGURE 9 Bridge rail before and after test.

TABLE 1 Summary and Results of Crash Tests

Test Parameter	Test Results
Vehicle data	
Type	Tractor-trailer (tank type), 1980 Kenworth
Mass (lb)	80,120
Speed (mph)	51.4
Film data	
Angle (degrees)	
Impact	15
Roll, maximum	
Truck	17
Trailer	15
Barrier displacement (in.) (dynamic)	4.0
Accelerometer data (located over tractor tandem axles), 100-Hz lo-pass maximum flat filter	
Maximum avg 0.050-sec acceleration *(g)*	
Longitudinal	-1.77
Lateral	5.54
Peak acceleration *(g)*	
Longitudinal	10.49
Lateral	18.56

Note: 1 lb = 0.45 kg; 1 mph = 1.61 km/hr; 1 in. = 2.5 cm.

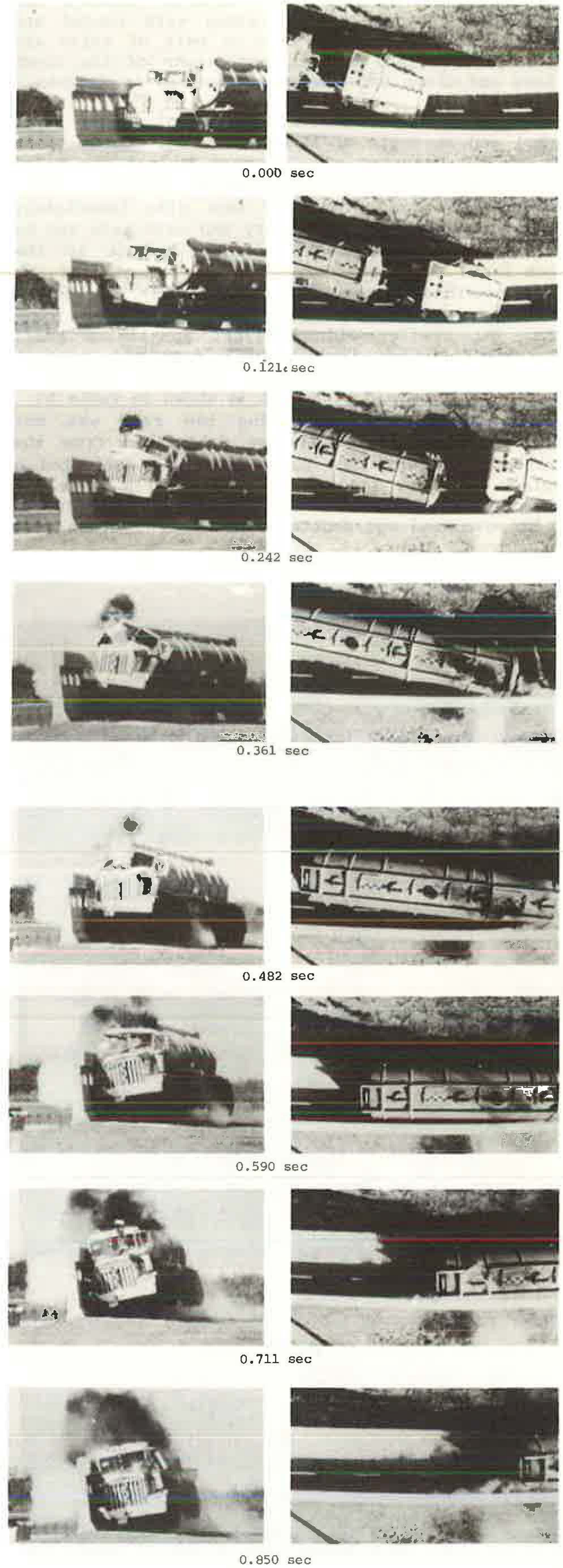


FIGURE 10 Sequential photographs of test.

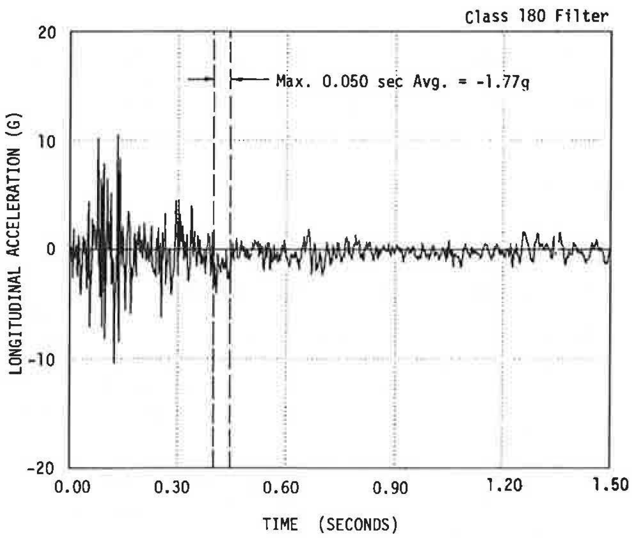


FIGURE 11 Vehicle longitudinal accelerometer trace of test.

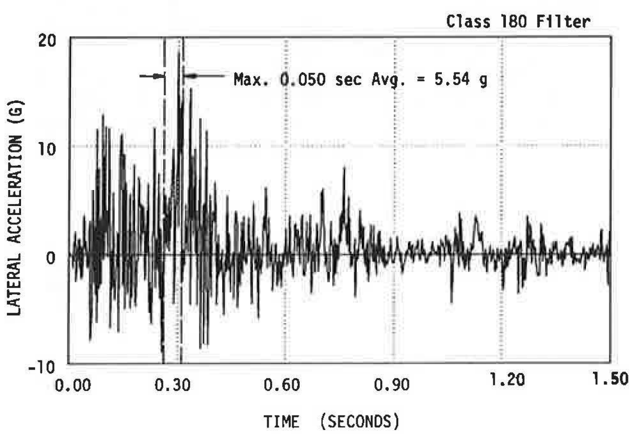


FIGURE 12 Vehicle lateral accelerometer trace of test.

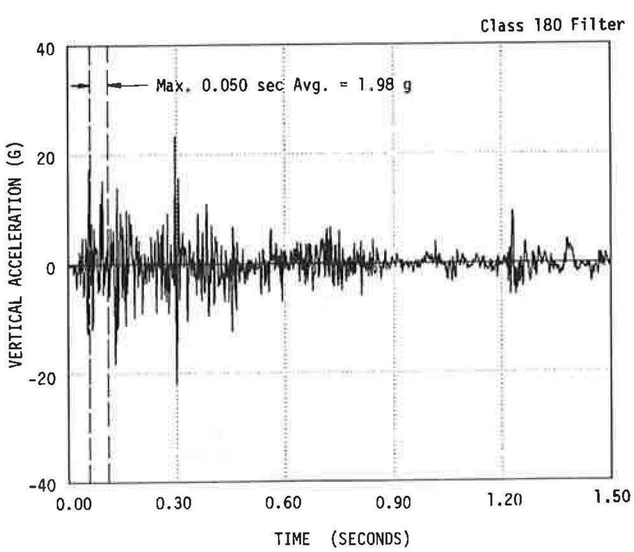


FIGURE 13 Vehicle vertical accelerometer trace of test.

eter data. The recommended threshold values for the flail space evaluation of passenger cars are 40 and 30 ft/sec for the longitudinal and lateral occupant impact velocity, respectively, and 20 g for the highest 10-msec average deceleration after contact. The computed values for this test were well below these recommended values. The longitudinal occupant impact velocity was 7.2 ft/sec, and the highest 10-msec average occupant acceleration after contact was -1.83 g. The lateral occupant impact velocity was 8.03 ft/sec, and the highest 10-msec average acceleration was 11.16 g. Even though these recommended threshold values do not apply to large trucks, they were presented here for comparison purposes.

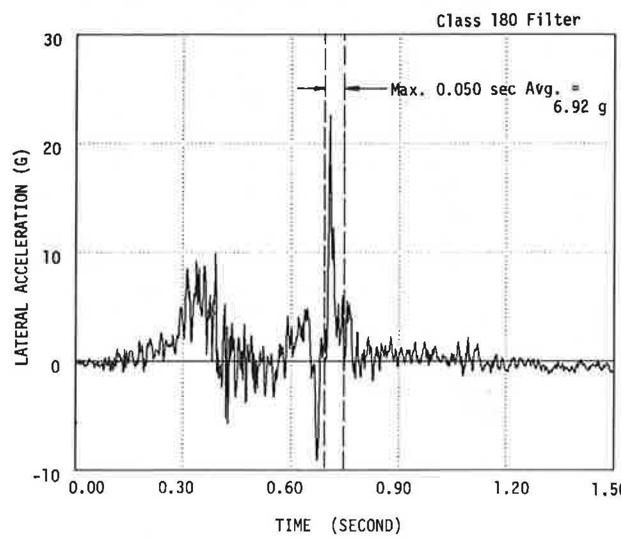


FIGURE 14 Trailer lateral accelerometer trace of test.

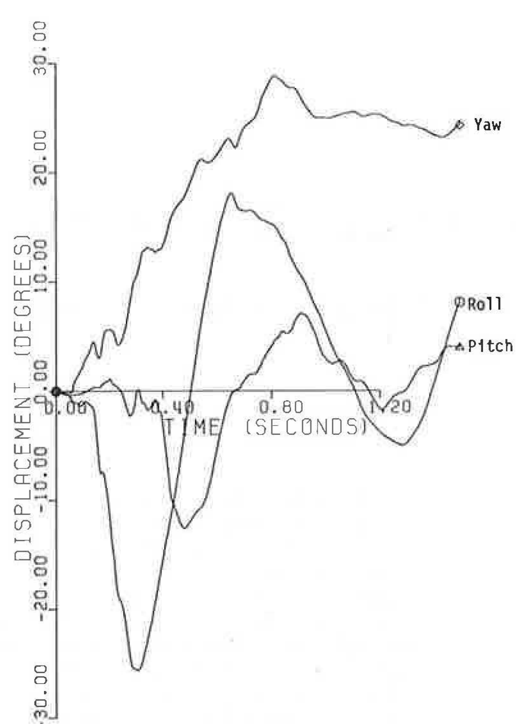


FIGURE 15 Vehicle angular displacement of test.

The upper concrete beam centered at 79.5 in. (202 cm) was designed so that the tank trailer would strike it and be prevented from overturning.

The cross-sectional area of this modified rail is approximately 7.6 ft^2 (0.7 m^2) as compared with approximately 2.6 ft^2 (0.2 m^2) for a standard Texas Type T5 traffic rail. The approximate cost of this modified rail would be about $125 per linear foot, whereas a standard Texas Type T5 traffic rail normally costs about $35 per linear foot.

SUMMARY AND CONCLUSIONS

A standard Texas Type T5 traffic rail concrete safety shape was modified by increasing its height and strength so that it could restrain and redirect an 80,000-lb tank-type truck or tractor-trailer. The height of the concrete parapet was increased to 48 in. A concrete beam element 16 in. wide and 21 in. deep was mounted on concrete posts on top of the concrete parapet to achieve a total rail height of 90 in. The concrete posts were 8 in. thick, 5 ft long, and 21 in. (53 cm) high with 5-ft (1.5-m) openings between the posts. The rail was constructed vertically on a 14-degree curve with the deck superelevated 0.055 ft/ft.

The crash test was conducted on this bridge rail with an 80,120-lb tank-type tractor-trailer impacting the rail at 51.4 mph and at an impact angle of 15 degrees. The vehicle was smoothly redirected.

This test has shown that a bridge rail can be built on a slightly modified Texas standard bridge deck to contain large tank-type tractor-trailer trucks and redirect them without rollover.

ACKNOWLEDGMENT

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REFERENCES

1. T.J. Hirsch and A. Arnold. Bridge Rail to Restrain and Redirect 80,000 lb Trucks. Research Report 230-4F. Texas Transportation Institute, Texas A&M University, College Station, Nov. 1981.
2. T.J. Hirsch. Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks. Research Report 230-2. Texas Transportation Institute, Texas A&M University, College Station, Aug. 1978.
3. T.J. Hirsch. Bridge Rail to Restrain and Redirect Buses. Research Report 230-3. Texas Transportation Institute, Texas A&M University, College Station, Feb. 1981.
4. J.S. Noel, C.E. Buth, and T.J. Hirsch. Loads on Bridge Railings. In Transportation Research Record 796, TRB, National Research Council, Washington, D.C., 1981, pp. 31-35.
5. T.J. Hirsch, J.J. Panak, and C.E. Buth. Tubular W-Beam Bridge Rail. Research Report 230-1. Texas Transportation Institute, Texas A&M University, College Station, Oct. 1978.
6. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.

Roadside Barriers for Bridge-Pier Protection

JAMES E. BRYDEN and RICHARD G. PHILLIPS

ABSTRACT

Seven full-scale crash tests were conducted to evaluate a concrete bridge-pier protection barrier. This barrier consists of four concrete half-section safety-shape barriers placed in front of the pier and flaring back from the pavement edge. The end of the concrete barrier is protected by a 6 by 6-in. box-beam guiderail bolted to the concrete. The barrier was impacted at various points with either 1,800- or 4,500-lb sedans at 15 and 25 degrees and a speed of about 60 mph. The original design caused vehicles to roll over when the concrete barrier was impacted at 25 degrees near the first bridge pier. The design was modified by extending the box beam across the face of the barrier directly in front of the piers. This eliminated the rollover problem and strengthened the barrier, resulting in performance in compliance with the standards in NCHRP Report 230.

Unprotected concrete bridge piers located near the pavement edge pose a serious hazard to vehicles leaving the roadway at that point. This problem is especially serious on older expressways on which high traffic speeds and volumes occur and bridge piers are located only a few feet from the pavement edge. As part of the effort to upgrade highway safety, it frequently becomes desirable to reduce the hazard presented by these piers. Because their removal would generally be prohibitively expensive, the solution generally is to shield the piers against impact. Impact attenuators may be used effectively in some cases, but their high construction and maintenance costs frequently result in the selection of longitudinal barriers as a more cost-effective alternative. Where shoulder widths are adequate, a variety of flexible steel traffic barriers are available that effectively shield the bridge piers and provide a reasonable level of motorist protection (1). Unfortunately, piers are sometimes located so close to the pavement edge that a nonyielding barrier is needed to provide adequate protection without further reducing the already limited shoulder width.

PURPOSE AND SCOPE

One solution to this problem has been used extensively on the New York State Thruway and to a limited extent by the New York State Department of Transportation (NYSDOT). This design consists of a concrete safety-shape barrier half-section directly in front of the piers, flaring back from the pavement at a 1:8 rate just upstream of the piers. A 6 x 6 x 3/16-in. box-beam guiderail protects the exposed upstream end of the concrete barrier. The guiderail uses the standard NYSDOT terminal on its upstream end, and the downstream end is bolted flush to the face of the concrete barrier. Both the concrete safety shape (2) and box-beam guiderail (3) are tried and proved systems that have been shown to perform well when used separately. However, their combined use raises questions that can best be answered through full-scale crash tests. These tests would determine the impact severity, strength, and redirectional characteristics of this concrete pier-protection barrier.

METHODOLOGY AND BARRIER DESCRIPTION

Seven full-scale crash tests were conducted under this study following the guidelines in NCHRP Report 230 (4). Work was started in 1982, but a performance problem was identified that required design modifications and additional tests in 1983. Five were strength tests with target conditions of 4,500-lb vehicles at 60 mph and 25 degrees. The two remaining tests were to assess occupant risk, with target conditions of 1,800-lb vehicles at 60 mph and 15 degrees.

The barrier system consisted of four 15-ft half-section concrete barriers to which was fastened a 6 x 6 x 3/16-in. box-beam guiderail. Total length of the barrier system was 130.5 ft. The box beam was sup- ported by S3x5.7 steel posts at a height of 30 in. Post spacing in the impact area varied from 2 to 3 ft, and upstream post spacing was maintained at 6 ft. Figure 1 shows details of the barrier as it was erected for Tests 60 through 63. The simulated bridge piers for these four tests were 36-in.-diameter cast-in-place columns. These were situated on

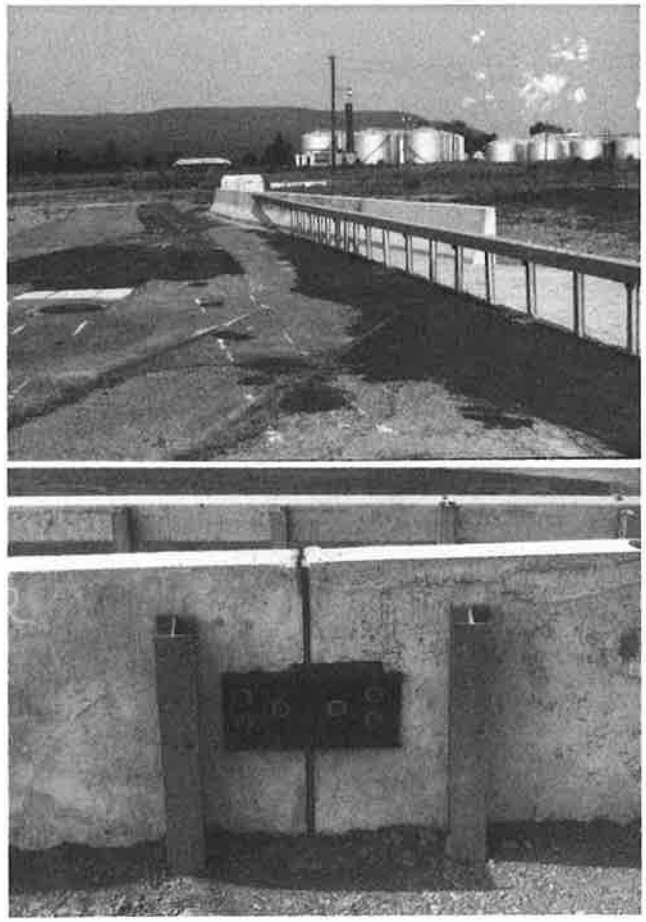

FIGURE 1 Pier-protection barrier for Tests 60 through 63 (above) included continuity connector and W6x9 backup posts for joint support (below).

4-ft-diameter concrete footings about 1 ft below grade.

For Tests 76 through 78 the columns were precast concrete culvert pipe 33 in. in diameter and 8 ft long. The pipes were placed on end with their bottoms about 1 ft below grade, and the excavation was carefully backfilled with compacted soil. The pipes were filled with soil to increase their mass. Changes to the barrier design for the final three tests (Figures 2 and 3) included (a) an earth backfill behind the concrete barrier in place of the heavy posts, (b) continuation of the box beam across Section A, and (c) addition of a 6-in. blockout between the concrete barrier and box beam at Section B.

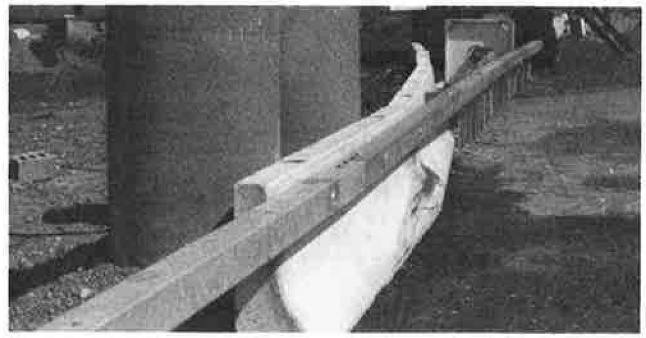

FIGURE 2 Pier-protection barrier for Tests 76 and 77: extending the box beam across Section A.

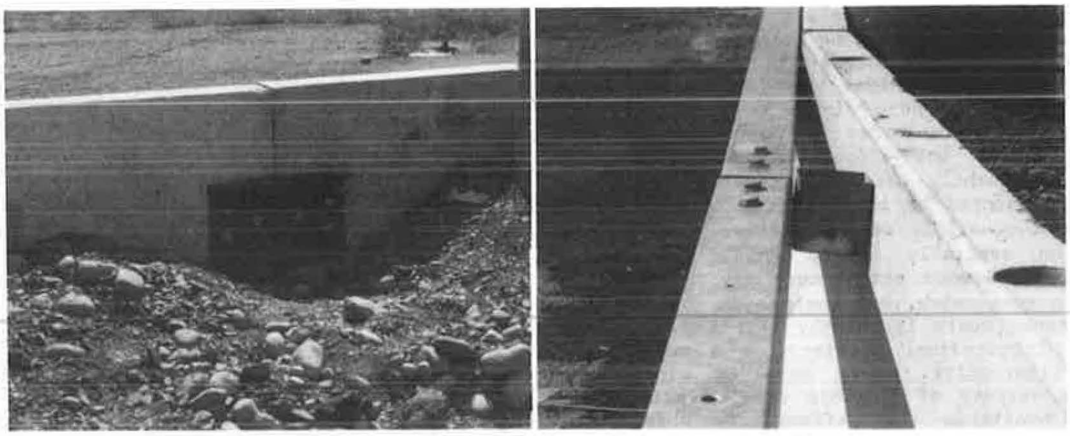

FIGURE 3 Pier-protection barrier for Tests 76 and 77: continuity connectors and earth fill (left), and a 6 by 6-in. blockout at Section B (right).

For all tests the concrete barrier was embedded 8 in. into the ground and continuity connectors were used between sections. The only problem involved with the installation of the barrier system was alignment of the W6x9 backup posts. These had to be driven before the concrete barriers could be positioned. Soil at the test site contained large cobbles in the granular material, which caused a few posts to be slightly misaligned. Hardwood shims filled any gaps resulting between the posts and the back of the concrete barrier. The steel backup posts thus were eliminated for the last three tests, because it appeared that the 6 x 6-in. box beam, combined with the continuity connectors and soil backfill, would provide adequate load transfer across the joints.

RESULTS

Results of the seven full-scale crash tests are summarized in Table 1. The purpose of Test 60 was to ensure that 4-ft clearance between the box beam and the end of the concrete barrier was sufficient to prevent the vehicle from striking the barrier. Standard design deflections for box-beam guiderail are 5 ft with 6-ft post spacing and 4 ft with 3-ft post spacing. Although it was anticipated that adequate deflection control would thus be provided by the 3-ft post spacing, this test was necessary to confirm it.

The 4,450-lb sedan impacted the box beam 55.5 ft downstream from its end (15 ft upstream from the end of the concrete barrier) at 55.7 mph and 25 degrees. Dynamic deflection was 2.6 ft, which confirmed that

TABLE 1 Test Results

Item	Test 60	Test 61	Test 62	Test 63	Test 76	Test 77	Test 78
Point of impact	Box-beam 55.5' downstream from end	Box-beam 12.2' upstream from box beam	Box-beam 12.2' upstream from box beam	Concrete barrier Section B	Box beam & concrete barrier; Section B	Box beam & concrete barrier; Section B	Box beam & concrete barrier; Section B
Barrier Length, ft	130.5	130.5	130.5	130.5	130.5	130.5	130.5
Vehicle Weight, lb	4450	1600	4500	4500	1800	4650	4500
Vehicle Speed, mph	55.7	59.0	54.3	57.1	58.3	61.2	63.7
Impact Angle, deg	25	14	29	26	20	29	30
Exit Angle, deg	9	4	7	9	11	12	9
Max. Roll, deg	-10	5	-9	-46	3	5	-180
Max. Pitch, deg	-2	0	4	-5	4	2	-5
Max. Yaw, deg	0	0	-117	70	0	0	0
Contact Distance, ft	27	21.2	15.6	12.7	12.4	14.9	6.7
Contact Time, ms	749	278	430	635	198	331	482
Deflection, ft							
Dynamic	2.6	.5	.25	N.A.	0	N.A.	N.A.
Permanent	1.7	0	.19	.21	0	.19	.17
Decelerations, g's							
50 ms avg.							
Longitudinal	3.2	4.6	11.1	12.9	10.4	5.9	9.0
Lateral	5.5	8.2	9.7	7.7	14.0	10.9	9.9
Max. Peak							
Longitudinal	9.4	10.2	25.5	34.9	21.1	10.9	-35.9
Lateral	16.1	15.0	20.5	27.6	27.2	17.5	-103.2
Occupant Ridedown							
Longitudinal	6.6	-1.1	6.3	-25.0	4.1	2.3	12.0
Lateral	8.6	8.0	9.3	9.7	6.4	7.8	10.6
Occupant Impact Velocity, fps							
Longitudinal (2.0 ft)	17.6	12.2	30.3	39.0	17.4	16.2	34.2
Lateral (1.0 ft)	16.6	19.9	23.6	19.3	26.5	26.0	23.0
Results and Comments	Good redirection	Good redirection	Good redirection	Vehicle rolled over; heavily damaged	Good redirection	Good redirection	Vehicle rolled over; heavily damaged

the 4-ft clearance was adequate. The vehicle attained a maximum pitch of only 2 degrees and a roll to the right of 10 degrees. (Positive roll is clockwise, positive pitch is nose down, and positive yaw is counterclockwise with respect to the driver's attitude.) Redirection was smooth with an exit angle of 9 degrees and a very slight curve to the right. Damage to the vehicle was moderate, and damage to the box beam was limited to 2 rail sections and 15 posts. Total contact distance was 27 ft. Peak 50-msec average decelerations were 3.2 g longitudinal and 5.5 g lateral. Occupant impact velocities were 17.6 ft/sec longitudinal and 16.6 ft/sec lateral, below those recommended by NCHRP Report 230 for 60-mph, 15-degree impacts. Figure 4 shows the vehicle and barrier after the test. From these results it appears that the barrier performs well at this point.

Test 61 was intended to evaluate barrier performance at the connection between the box beam and the concrete barrier. In addition to determining impact severity and postimpact trajectory, this test investigated the problem of wheel snagging between the bottom of the box beam and the concrete barrier. The 1,600-lb Subaru sedan impacted 12.2 ft upstream from the end of the box beam at 59.0 mph and 14 degrees. The vehicle redirected smoothly with an exit angle of only 4 degrees and followed a curved path to the right back toward the barrier. Virtually no pitch or yaw was observed, and maximum roll was only 5 degrees toward the barrier. Vehicle damage (Figure 5) resulting from the impact was light, although the vehicle was subsequently damaged in a secondary impact with a chain-link-fence arrestor system. Minor sheet-metal snagging occurred on a hex-head bolt used to connect the box beam to the concrete barrier. Peak 50-msec average decelerations were 4.6 g longitudinal and 8.2 g lateral. Occupant impact velocities were 12.2 ft/sec longitudinal and 19.9 ft/sec lateral compared with NCHRP Report 230 recommended maximums of 30 and 20 ft/sec, respectively. This test was thus considered successful.

The impact point in Test 62 was the same as that in Test 61, but the test was designed to evaluate the strength of the barrier system at the connection point. The 4,500-lb Mercury sedan impacted at 54.3 mph and 29 degrees. The vehicle was smoothly redirected with a maximum roll of -9 degrees and maximum pitch of 4 degrees. Departure angle was 7 degrees, but the vehicle gradually curved to the right and yawed about 117 degrees to the right, coming to rest at an angle of 110 degrees along the barrier line. Damage to the vehicle was generally moderate, although the sheet metal on the right front door snagged on a box-beam connection bolt and was peeled off the car (Figure 6). Barrier damage included one bent rail section and two vertical cracks in Section B of the concrete barrier near the end of the box beam. Peak 50-msec decelerations were 11.1 g longitudinal and 9.7 g lateral. Occupant impact velocities were 30.3 ft/sec longitudinal and 23.6 ft/sec lateral. Although impact severity slightly exceeded recommended values for 15-degree impacts, it appeared reasonable for this 29-degree impact. The barrier was thus considered to meet strength and redirectional requirements at this point.

FIGURE 4 Test vehicle and barrier after Test 60.

FIGURE 5 Test vehicle and barrier after Test 61.

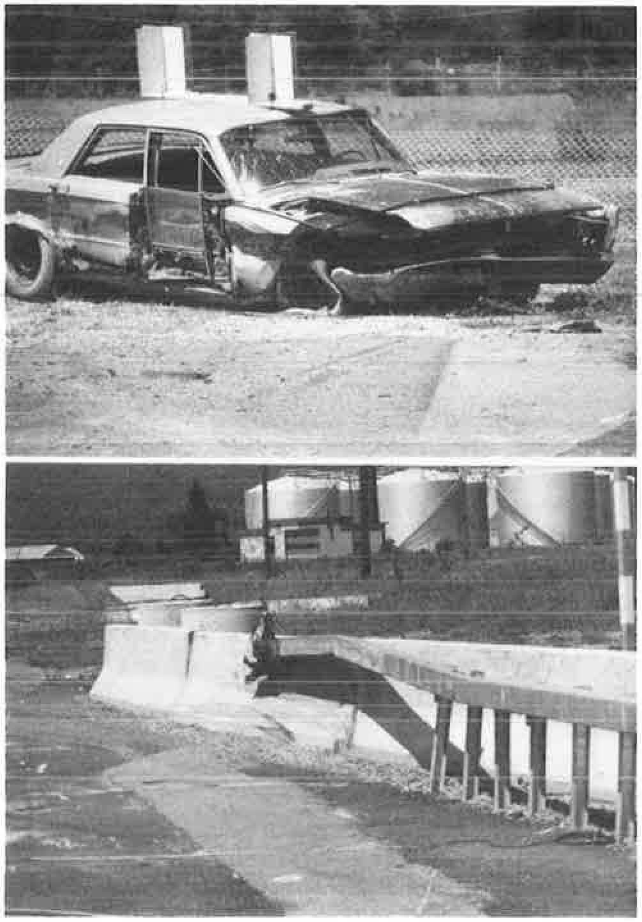

FIGURE 6 Test vehicle and barrier after Test 62.

FIGURE 7 Test vehicle and barrier after Test 63.

Test 63 was intended to test the strength of the connection between barrier Sections A and B, just upstream from the first bridge column. The 4,730-lb Ford station wagon impacts 7.7 ft upstream from the joint at 57.1 mph and 26 degrees. The vehicle quickly climbed to the top of the concrete barrier while rolling 46 degrees counterclockwise. Tire marks at the top of the 4-ft-high simulated bridge columns confirmed that the vehicle would have sustained solid impact with full-height columns. Film measurements showed that the car penetrated about 1.5 ft behind the back of the concrete barrier, or 2 ft behind the barrier face. On leaving the barrier, the vehicle yawed 70 degrees to the left while still rolling away from the barrier. It then rolled sharply toward the right, and on contact with the ground rolled over completely on the right front corner before coming to rest. Peak 50-msec decelerations were 12.9 g longitudinal and 7.7 g lateral, with corresponding occupant impact velocities of 39.0 and 19.3 ft/sec. The vehicle was extensively damaged during the initial impact and subsequent rollover. Damage to the barrier was also extensive, as seen in Figure 7; there were cracks through both Sections A and B and the continuity connector was nearly broken out of the barrier. This test thus indicated that joint strength is marginal for such severe impacts. More important, contact with the columns, high decelerations, and vehicle rollover all confirm that the concrete safety-shape barrier is inadequate for high-angle impacts, especially when fixed objects are located immediately behind the barrier. The vehicle and barrier after the test are shown in Figure 7.

Good performance had been obtained in Tests 61 and 62, where impact had occurred upstream of the end of the box beam. The primary difference between those two tests and Test 63 was that the box beam prevented the vehicle from climbing up the concrete barrier and attaining a very high roll angle. The barrier thus was modified for the 1983 tests by extending the box beam past the columns and bolting it to the face of Section A by using carriage bolts. For the remaining three tests, the height of the simulated bridge piers was increased to 7 ft. Continuity connectors were again attached between sections of concrete barrier, and a 2-ft earth backfill was used behind the barrier in place of the heavy posts used in the previous tests. Because protrusion of the base made it impossible to install posts in front of Section B, a 6-in. steel blockout was added to connect the box beam to the concrete barrier. An 8-in. carriage bolt connected the box-beam rail to the blockout from the front, and an 8 x 3/4-in. hex-head bolt connected the blockout to the concrete barrier.

Test 76 was intended to evaluate the modified design for wheel snag, impact severity, and redirection. The 1,800-lb Honda sedan impacted 4.3 ft upstream from the connection between Sections A and B at 58.3 mph and 20 degrees. Even at this relatively high impact angle, the car was smoothly redirected at an exit angle of 11 degrees, with no appreciable yaw. Maximum roll and pitch were +3 and +4 degrees,

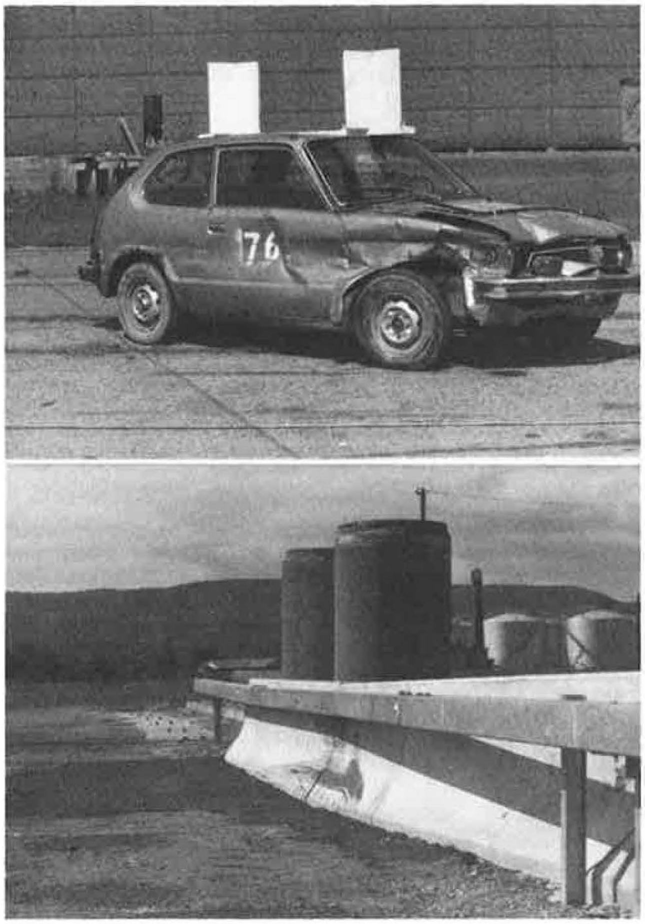

FIGURE 8 Test vehicle and barrier after Test 76.

FIGURE 9 Test vehicle and barrier after Test 77.

respectively. Peak 50-msec average decelerations were 10.4 g longitudinal and 14.0 g lateral. Occupant impact velocities were 17.4 ft/sec longitudinal and 26.5 ft/sec lateral, slightly exceeding the recommended lateral occupant impact velocity. However, considering the high impact angle, these results are not considered unacceptable. Vehicle damage was light in this test, and the barrier had virtually no damage (Figure 8).

Test 77 was a strength test with impact at the same point as that in Test 76. The 4,650-lb Lincoln sedan impacted at 61.2 mph and 29 degrees. The vehicle was smoothly redirected along a 12-degree exit path and curved slightly to the right. Roll and pitch were limited to +5 and +2 degrees, respectively. Peak 50-msec average decelerations were 5.9 g longitudinal and 10.9 g lateral, with corresponding occupant impact velocities of 16.2 and 26.0 ft/sec. Only moderate damage was sustained by the vehicle, and barrier damage was very light (Figure 9).

Tests 76 and 77 confirmed that extending the box beam along the top face of the concrete barrier reduced vehicle roll to a very low level and kept impact forces within tolerable levels. It also increased load transfer between adjoining sections of concrete barrier, greatly reducing barrier damage on severe impacts. It thus appears that the modified barrier provides an acceptable level of protection and will require very little postimpact repair.

Following completion of the two successful tests on the modified barrier, the box beam across Section A was removed and Test 63 was repeated. The objective of this test was to evaluate the severity of contact with the full-height columns. A continuity connector was again used between Sections A and B, but earth backfill was substituted for the backup posts used in Test 63.

The 4,500-lb Chevrolet sedan impacted 3.2 ft upstream from the joint between Sections A and B at 63.7 mph and 30 degrees. It immediately climbed all the way to the top of the barrier and rolled nearly 90 degrees counterclockwise. The vehicle impacted both columns above the top of the barrier and then continued to roll counterclockwise as it left the barrier. It rolled onto its roof and left the barrier on a 9-degree angle, sliding on its roof. The vehicle was severely damaged by the impact with the barrier and piers and the subsequent rollover (Figure 10). Peak 50-msec average decelerations were 9.0 g longitudinal and 9.9 g lateral, with corresponding occupant impact velocities of 34.2 and 23.0 ft/sec. In spite of the severe impact, the barrier experienced only light damage. The earth backfill used for this test was capable of absorbing severe loads without misalignment of the barrier, even without the box beam or 6x8.5 backup posts. Impact conditions in this test were somewhat more severe than the 60-mph, 25-degree intended impact. However, as shown in Test 63, contact with the columns and a similar vehicle trajectory would probably have resulted even at the lower speed and lesser angle.

DISCUSSION AND FINDINGS

On the basis of five successful full-scale tests, it appears that the box-beam concrete-barrier pier-pro-

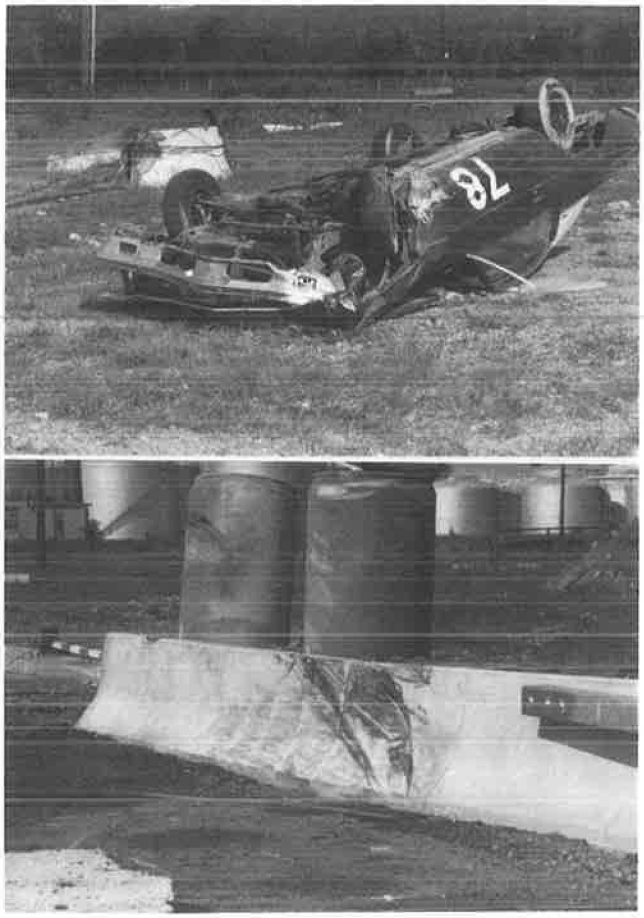

FIGURE 10 Test vehicle and barrier after Test 78.

tection system, as modified, meets the performance criteria of NCHRP Report 230. Structural adequacy was evaluated at three points along the barrier. Test 60 demonstrated that the 4-ft clearance between the box beam and the end of the concrete barrier was adequate. The connection of the box beam to the concrete barrier was also adequate, and continuing the box beam across the front of the bridge pier resulted in adequate strength and containment. Post-impact vehicle trajectories were satisfactory in all five tests. Occupant risk factors slightly exceeded the recommended value in one 1,800-lb vehicle test, but this was attributed to the high 20-degree impact angle. Occupant risk factors for the three large-car tests at 25-degree impact angles were close to or below recommended values for 15-degree tests. The box beam successfully held vehicle roll to a very low level. This may also have increased impact severity somewhat, because very little impact energy was absorbed in lifting the vehicle as normally occurs in concrete-barrier impacts.

The two large-vehicle tests on the concrete barrier in front of the bridge columns--without the box beam--provided strong evidence that this barrier does not provide safe performance for high-angle impacts. In each test, the vehicle climbed to the top of the concrete barrier, developed a very high roll angle, contacted the columns, and rolled over on leaving the barrier. At least a 2-ft clearance behind the barrier face appears necessary to prevent contact with the columns or other rigid objects. This type of behavior has been seen in previous concrete-barrier tests during high-angle impacts (5,6). However, this unsuitable behavior was eliminated by extending the box beam across the front of the piers. A comparison of vehicle redirection with and without the box beam is shown in Figure 11.

Strength of the connection between concrete-barrier sections was also evaluated. The backup posts and continuity connectors were not adequate to provide load transfer in severe impacts (4,500 lb, 60 mph, 25 degrees) and substantial damage to the barrier was experienced. However, with the box beam extended across the concrete barrier joint, adequate load transfer was provided by using the continuity connector and earth backfill. Although the continuity connector was nearly broken out of the barrier in Test 63, addition of the box beam and soil backfill greatly strengthened the system, and the anchorage system used for the connector was entirely adequate without the posts. In Test 78 adequate joint strength was provided by the connector and backfill without either the posts or box beam.

Based on these tests, the following conclusions can be stated:

1. The box-beam concrete-barrier pier-protection system appears to meet NCHRP Report 230 performance criteria when the box beam is extended across the front of the piers.
2. In impacts at 60 mph and 25 degrees, the concrete barrier alone was unable to prevent contact with bridge columns located immediately behind the barrier. At least 2 ft of clearance behind the barrier appears necessary to prevent contact.
3. The continuity connectors and W6x8.5 steel posts provided only marginal strength at the con-

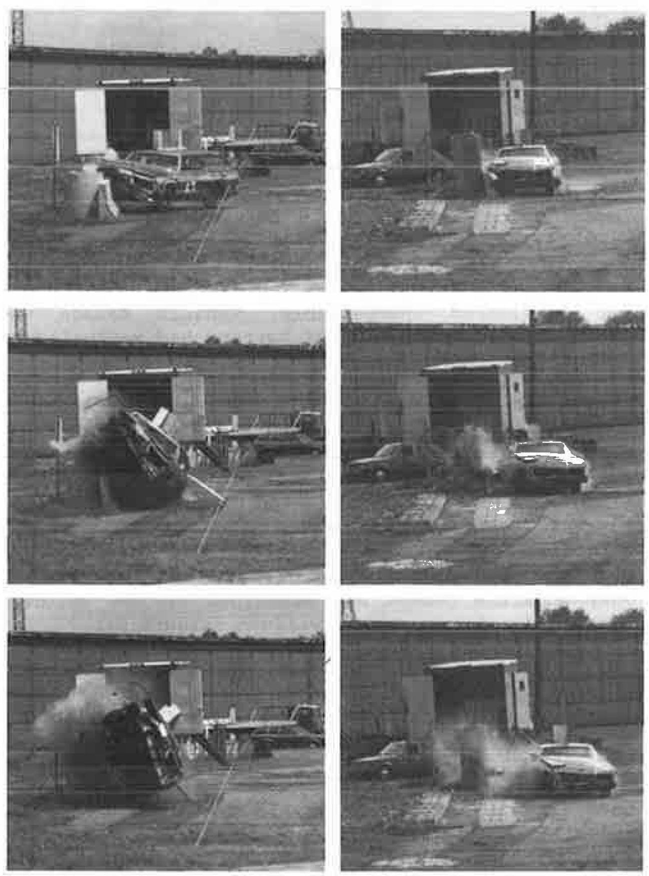

FIGURE 11 Vehicle redirection in Test 63 without box-beam (left) versus Test 77 with box beam (right).

crete barrier joints, because the joint was displaced several inches and the connector broken out by the large-sedan impact.

4. When the box beam was extended across the concrete barrier joint, the continuity connector and earth backfill provided adequate strength without the use of backup posts.

5. Use of carriage bolts instead of hex-head bolts to attach the box-beam rail to the concrete barrier reduced sheet-metal snagging.

6. The 4-ft clearance between the box-beam rail and the end of the concrete barrier is adequate.

For a full explanation of testing procedures, data analysis, and test results, the reader is referred to NYSDOT Research Report 117 (7).

ACKNOWLEDGMENTS

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REFERENCES

1. H.E. Ross. Guide to Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, D.C., 1977.
2. M.E. Bronstad et al. Concrete Median Barrier Research, Vol. 2: Research Report. Report FHWA-RD-77-4. Southwest Research Institute, San Antonio, Tex., March 1976.
3. J.L. Whitmore, R.G. Picciocca, and W.A. Snyder. Testing of Highway Barriers and Other Safety Accessories. Research Report 38. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Dec. 1976.
4. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.
5. K.C. Hahn and J.E. Bryden. Crash Tests of Construction-Zone Traffic Barriers. Research Report 82. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, June 1980.
6. J.S. Fortuniewicz, R.G. Phillips, and J.E. Bryden. Crash Tests of Portable Concrete Median Barrier for Maintenance Zones. Research Report 102. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Dec. 1982.
7. Roadside Barriers for Bridge Pier Protection. Research Report 117. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Dec. 1984.

The Connecticut Impact-Attenuation System

JOHN F. CARNEY III, CHARLES E. DOUGAN, and MARTIN W. HARGRAVE

ABSTRACT

The development of a new crash cushion is described. This impact-attenuation device is composed of steel tubular members formed from straight plate sections, which are bolted together to form a cluster. This device is unique in that it will trap an errant vehicle under most impact conditions. The vehicle will be redirected back out into the roadway only when the impact location is so close to the rear of the system that it is impossible to obtain acceptable energy-dissipation and deceleration-trapping responses because of the proximity of the site hazard. No other attenuation system in use today possesses this capability. In addition, the Connecticut impact-attenuation system exhibits the following characteristics: (a) it satisfies the impact performance standards outlined in Transportation Research Circular 191 and NCHRP Report 230, (b) it is inexpensive to fabricate, (c) the energy-dissipating tubes can be refurbished after impact and reused, (d) there is no flying debris associated with the crash event, and (e) it is constructed of readily available materials.

In May 1982 the Connecticut Department of Transportation (ConnDOT) initiated a research effort to develop a new highway crash cushion constructed of steel tubular members that would possess unique energy-dissipation characteristics. The system concept was an offshoot of the work performed in developing the Connecticut crash cushion (1,2), a truck-mounted attenuator that is currently being employed by ConnDOT field personnel and other state transportation agencies (3). The very favorable accident experience of the portable system (4,5) provided the incentive to apply the same engineering principles to the design and full-scale crash testing of the stationary crash cushion described in this paper.

Crash cushions are currently in widespread use in the United States to bring errant vehicles to a controlled stop when the impact is head on. Under side-impact conditions, systems using fender panels redirect the errant vehicle, even when the impact is near the front of the device. On the other hand, a sand-barrel crash cushion system provides almost no redirection and therefore possesses an inadequate energy-dissipation capacity when the vehicle is directed at the corner of the roadway hazard.

The California Department of Transportation (Caltrans) has recently completed 5 years of monitoring impact attenuators with video systems (6). Their report strongly recommended that further design work be done to make all crash cushions more energy absorbent when subjected to a side impact. The authors of this paper contend that an impact-attenuation device should trap the errant vehicle when it impacts the unit on the side unless the area of the impact on the device is so close to the back of the system that significant energy dissipation and acceptable deceleration responses are unobtainable because of the proximity of the hazard. Only in this situation should the impact-attenuation device redirect the vehicle back into the traffic flow. No energy-absorbing system currently employed possesses these characteristics, and it was the aim of this research project to develop such a system, employing steel tubes as the energy-dissipation components. Steel tubes possess the advantages of low cost, ready availability, and favorable energy-absorbing properties. Model tests conducted at Cambridge University in England (7) verified the analytical approach and ultimately led to two designs for a full-scale system. These two designs were subsequentially crash tested at the Calspan Advanced Technology Center in Buffalo, New York (8). The results of the seven crash tests performed by Calspan demonstrated the potential of the steel-tube attenuator design. The system was further refined during a series of nine crash tests conducted at the Texas Transportation Institute (TTI). These tests documented that this new device offered both redirection and entrapment capabilities, whereas commercially available attenuation systems provide either redirection or entrapment under side-impact conditions.

A technical description of the Connecticut impact-attenuation system (CIAS) is presented, the results of crash tests performed at TTI are documented, and the design changes that evolved during the testing program are outlined in chronological order.

DESCRIPTION OF THE SYSTEM

The CIAS, shown in Figure 1, is composed of 14 tubular members formed from straight (A-36) steel plate sections. These tubes are bolted together, rest on a concrete pad, and are attached to an appropriate backup structure. In order to cope with the redirectional crash test case involving an impact near the rear of the system, steel "tension" straps (ineffective under compressive loading) and "compression" pipes (ineffective in tension) are employed. This bracing system ensures that the crash cushion will respond in a stiff manner when subjected to an oblique impact near the rear of the unit, providing the necessary lateral force to redirect the errant vehicle. On the other hand, the braced tubes retain their unstiffened response when the attenuation system is crushed by impacts away from the back of the device.

The details of the analytical and experimental work that led to the design of the bracing system employed in the CIAS are reported elsewhere (9,10) and will not be repeated here. However, a few quasi-static results reported by Carney and Veillette (10) are reproduced to illustrate the dramatic effect that tension bracing has on the load-deflection response of a steel tube.

Figure 2 shows a tube with symmetrical double tension bracing with its loading rig. Small-scale tubes were tested (outside diameter, 4 in.; wall thickness, 0.087 in.; length, 2 in.) on an Instron 1321 testing machine interfaced with a Hewlett Packard 9825B data acquisition cartridge and plotter system. Before the testing, the tubes were annealed by being heated in an electric furnace for 20 min at 900°C and being allowed to cool slowly. High-tensile-strength steel wire (diameter 0.013 in.) was employed to provide the tension bracing. The wire lacing procedure was carefully done and typically consisted of 25 loops for each stiffener.

Figure 3 shows the theoretical and experimentally determined initial collapse loads obtained, in which P_θ and P_C are the initial collapse loads obtained for the braced and unbraced tubes, respectively. The correlation is considered to be quite good in view of the difficulties associated with accurate placement of the tension bracing. It is of interest to note that from the point of view of stiffness at the onset of collapse, double bracing at 30 degrees represents the optimum condition.

Dimensionless tube load-deflection curves for a wide range of bracing angles are presented in Figure 4 in which δ is the deflection of the tube, D is the outside tube diameter, and P is the applied load. The dramatic effect of the bracing angle on the stiffness and energy-dissipation capacity of the tube (area under P-δ curve) is readily apparent. It is of interest to note that when $\theta \geq 45$ degrees, the bracing does not act in tension during the deformation process and therefore has no effect on the response of the tube. The forces in the tension bracing for θ-values of 0 and 25 degrees are also presented in Figure 4.

It is emphasized that the tension bracing (steel straps) and compression bracing (1.5-in. ID pipe sections) have no effect on the response of the CIAS in head-on impacts. Under this loading, the tension bracing is loaded in compression and buckles. The compression bracing, being welded to the tube at one end only, carries no load during the collapse process because its free end separates from the tube wall when collapse occurs. The internal bracing system is only activated under side-impact conditions.

The effective performance of the CIAS under impact conditions is dependent on the appropriate interaction of the unit with its surroundings. The following peripheral system components are required:

1. A level concrete pad on which the steel tubes rest,
2. A structurally adequate backup structure,
3. Steel skids under the tubes to minimize friction during the collapse process, and

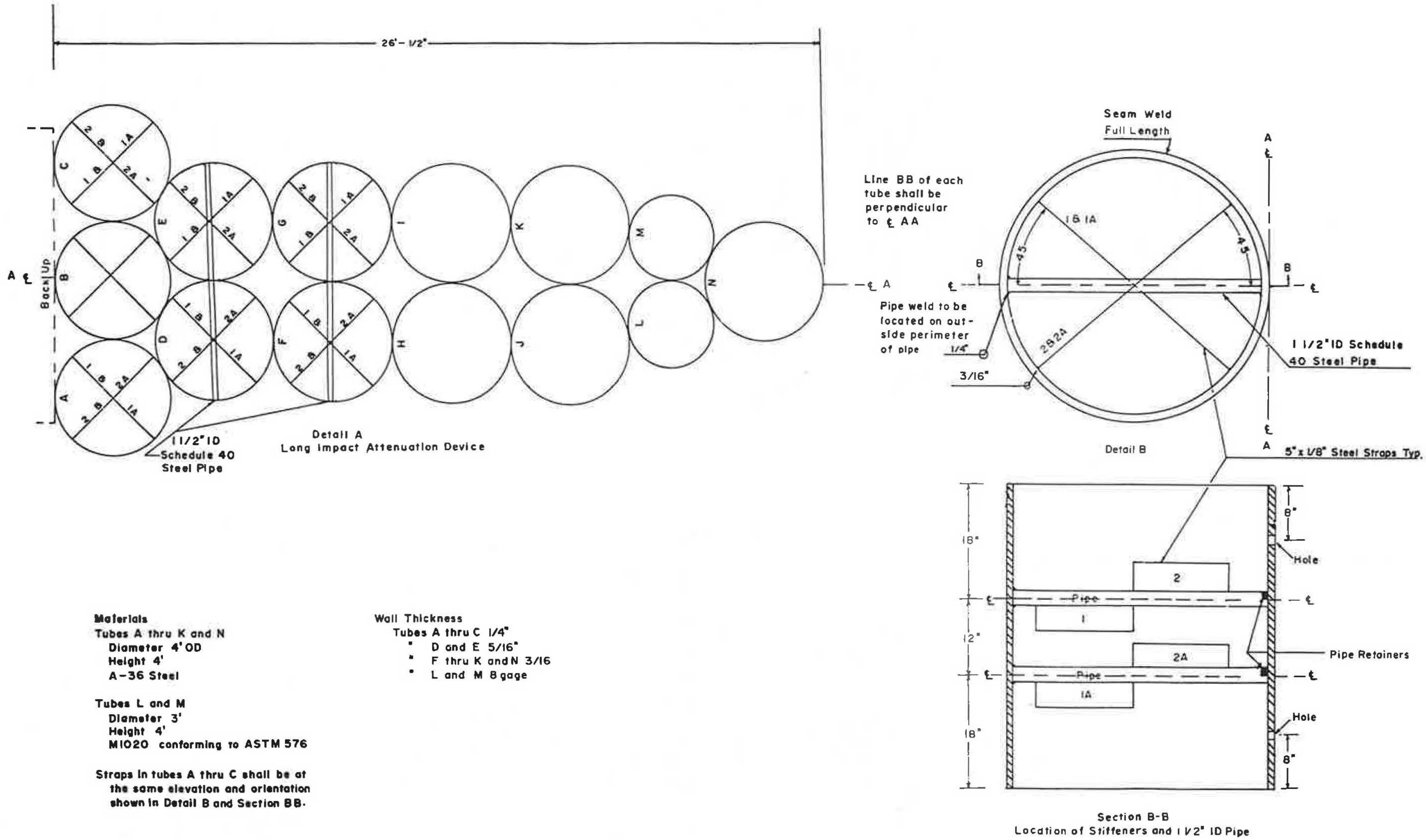

FIGURE 1 Shop fabrication details of CIAS.

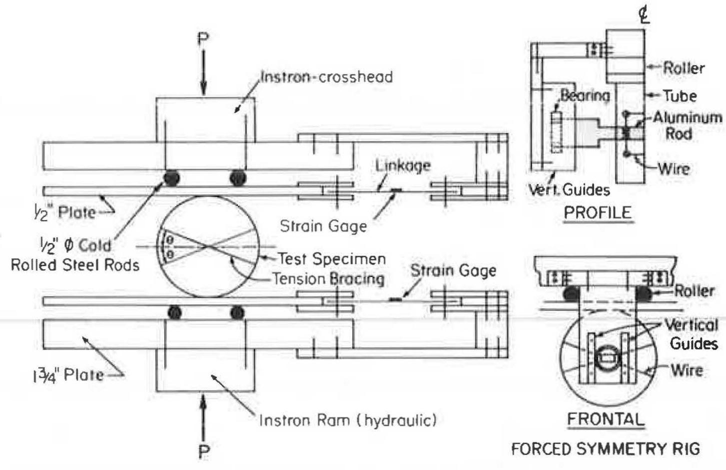

FIGURE 2 Braced tube and loading rig.

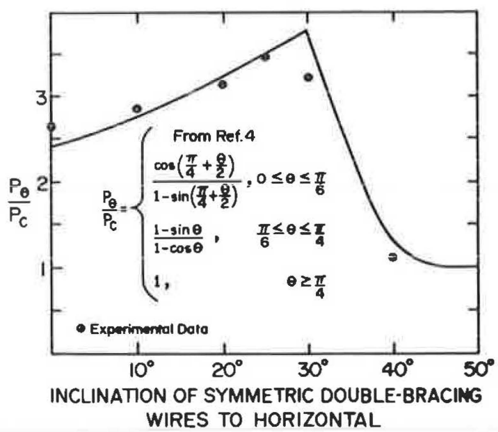

FIGURE 3 Variation in initial collapse loads with double bracing inclination.

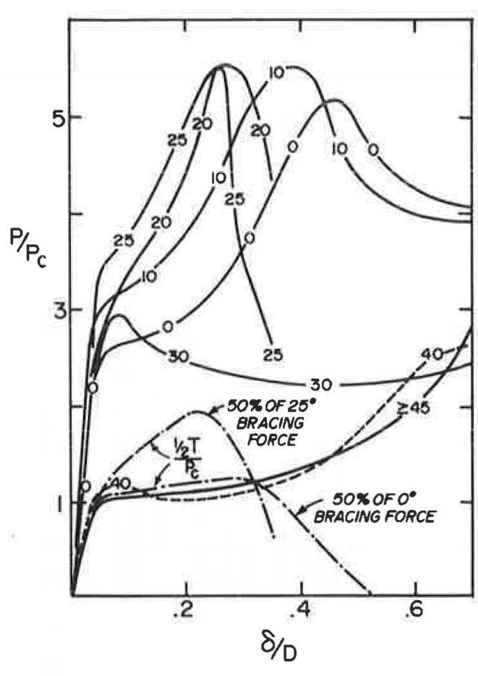

FIGURE 4 Load-deflection curves for double-braced tubes for various values of θ.

4. A vinyl-coated nylon nonlaminated cover to prevent the buildup of snow and ice in winter.

The complete design drawings for the CIAS may be obtained from Charles E. Dougan at ConnDOT.

The CIAS system is designed so that the tubes can be reused, even after an impact causing significant collapse of the system. Research has demonstrated that individual tubes can be reshaped and reused with an attendant saving in material cost. Thus, the CIAS has a potentially longer service life as compared with some of the conventional impact attenuators now in use. Test 9 described in the next section of the paper was conducted to show that the crash-test performance of the CIAS is unaffected when refurbished sections are used in the design. The data obtained verify that a CIAS constructed with refurbished cylindrical members meets all criteria set forth in NCHRP Report 230 and Transportation Research Circular (TRC) 191 (11,12).

CRASH-TEST PROGRAM

A total of nine full-scale crash tests were conducted at TTI. The crash tests were evaluated in accordance with the standards set forth in both TRC 191 and NCHRP Report 230. A summary of the nine crash tests is presented in Table 1. The complete individual crash-test reports and system design modifications made during the testing program have been described elsewhere (13).

It can be seen from Table 1 that design modifications took place during the first five crash tests. The major developments were as follows:

1. The height of the collapsing tubes was increased from 36 to 48 in. to eliminate vehicle ramping problems encountered in Tests 1 and 3.
2. The cover design was modified. Cellular plastic covers were replaced with a polyvinyl cover design. The polyvinyl cover remains attached to the crash cushion during the collapse and will prevent snow and ice from accumulating in the tube system in winter.
3. Steel skids were installed under the CIAS to reduce friction force buildup during the collapse process.
4. The tension stiffening system was modified, some tube thicknesses were changed, and an additional row of tubes was added to soften the impact response of the system.

TABLE 1 Summary of Crash-Tests Results

								Vehicle Deceleration Data *(g)*						
						Occupant Impact Velocity[a] (ft/sec)		Occupant Ridedown Peak[b] (10-msec Avg)		Peak 50-msec Avg[c]				
Test No.	Vehicle Weight (lb)	Impact Speed (mph)	Angle of Impact (degrees)	Point of Impact	Vehicle Stopping Distance (ft)	Longitudinal	Lateral	Longitudinal	Lateral	Longitudinal	Resultant	Avg over Entire Event[c]	Vehicle Damage Classification[d] (TAD)	Comments
1	4,500	59.9	0	Nose	NA	29.8	NA	13.7	NA	9.7	NA	NA	12FD2	Vehicle vaulted onto CIAS because of high center of gravity of vehicle and large friction forces developed at the rough concrete pad's surface; unit will rest on steel skids in future tests to reduce friction
2	1,800	59.8	0	Nose	13.4	34.9/39.2[e,f]	8.3	14.5	1.9	14.5	14.5	6.2	12FD3	Cellular plastic covers performed unsatisfactorily; new cover design used in subsequent tests
3	4,500	60.0	20	Along side	NA	28.2[e]	10.4	16.6	3.0	7.4	NA	NA	11FL4	Vehicle vaulted because of high center of gravity; tube heights increased from 36 to 48 in. to solve ramping problem
4	4,500	60.4	20	Along side	18.1	27.6[e]	11.5	20.6	1.5	13.3	13.5	6.5	11FD3	Stable impact response obtained; vehicle trapping achieved
5	4,500	61.7	0	Nose	19.5	29.7	NA	30.8	NA	12.7	12.9	6.3	12FD3	Polyvinyl cover design deemed satisfactory; no cover used in subsequent tests
6[g]	4,500	58.0	15	Corner of test hazard	NA	32.0	14.3[e]	9.6	11.6	9.5/6.6[h]	10.0	3.7/1.9[h]	11FL6	CIAS design now complete (see Figure 1) and used for Tests 6-9
7[g]	4,500	61.4	0	Nose	23.0	25.5[e]	4.5	12.6	0.9	10.4	10.4	5.2	12FD3	Excellent test results
8[g]	1,800	60.9	0	Nose	16.0	30.96/34.66[f]	NA	12.8	NA	11.6	11.6	5.7	12FD3	Excellent test results
9[g]	4,500	61.6	0	Nose	22.0	26.7	NA	12.8	NA	9.4	9.5	5.8	12FD2	Excellent test results; refurbished tubes employed

Note: NA = not applicable.

[a]NCHRP Report 230 recommends a longitudinal occupant impact velocity limit $[\Delta V_{Limit}]$ of 40 ft/sec/(acceptance factor). If the acceptance factor is set at 1.33, then $[(\Delta V)_{Design}]_{Long.}$ = 30 ft/sec. It recommends a lateral occupant impact velocity limit of 30 ft/sec (acceptance factor). If this acceptance factor is taken as 1.5, then $[(\Delta V)_{Design}]_{Lat.}$ = 20 ft/sec.

[b]NCHRP Report 230 recommends longitudinal and lateral occupant ridedown acceleration limits $[(a)_{Limit}]$ of 20 *g* /(acceptance factor) based on the highest 10-msec averages beginning with occupant impact. If the acceptance factor is set at 1.33 , then $(a)_{Design}$ = 15 *g* .

[c]For direct-on impacts, TRC 191 specifies a maximum average vehicle deceleration of 12 *g* as calculated from vehicle impact speed and passenger compartment stopping distance. When the test article functions by redirecting the vehicle, the maximum resultant 50-msec vehicle deceleration is specified to be 12 *g* when the impact angle is 15 degrees or less.

[d]Damage scale specified according to procedures developed by the Traffic Accident Data Project of the National Safety Council.

[e]Occurs first.

[f]The first impact velocity value is associated with the measured distance that the occupant would travel before impacting the compartment interior (1.25 ft). The second impact velocity value corresponds to an assumed occupant travel distance of 2 ft.

[g]Two longitudinal and two lateral accelerometers were employed. Occupant impact velocities and decelerations are average values.

[h]Lateral acceleration value.

The CIAS design was finalized following Test 5. No additional modifications were made during Tests 6 through 9, which are described as follows:

1. Test 6 (August 9, 1983)
   a. System tested: The impact attenuator tested was that shown in Figure 1.
   b. Test vehicle: A Plymouth Salon (1978) impacted the Connecticut attenuator at 58.0 mph and 15 degrees, directed at the rear corner of the system (Figure 5). The vehicle weighed 4,500 lb with 2,482 lb on the front axle and 2,018 lb on the rear axle.
   c. Test results: The crash cushion smoothly redirected the vehicle. Figures 6-8 show the CIAS and the vehicle after Test 6 (see Table 1 for measured decelerations). This test demonstrated that the tube-stiffening system provides the lateral resistance required to redirect a vehicle under these severe test conditions.

2. Test 7 (August 11, 1983)
   a. System tested: Same as that in Test 6 (see Figure 1).
   b. Test vehicle: A Plymouth Salon (1978) impacted the attenuator at 61.4 mph and 0 degrees. The vehicle weighed 4,500 lb with 2,460 lb on the front axle and 2,040 lb on the rear axle. Views of the test vehicle and the CIAS before the test are shown in Figures 9 and 10.

FIGURE 5 Vehicle alignment before Test 6.

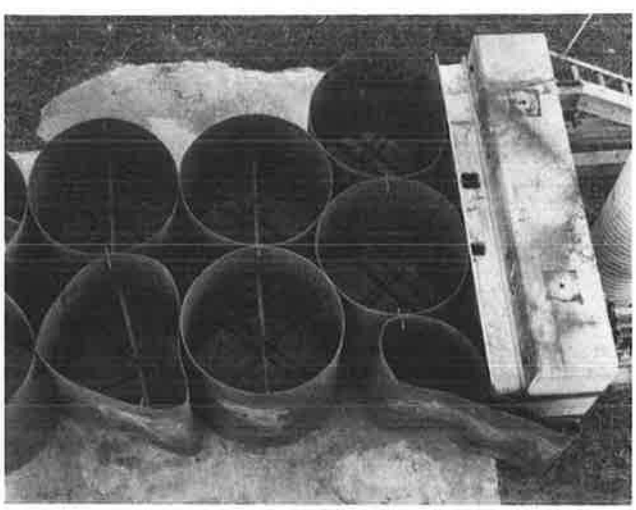

FIGURE 6 Top view of CIAS after Test 6.

   c. Test results: The vehicle collapsed the attenuator almost completely, as shown in Figure 11 (see Table 1 for measured decelerations). The front end of the car sustained an average crush of 13.5 in. (Figure 12). All occupant risk values in this test were well below the guidelines of both TRC 191 and NCHRP Report 230.

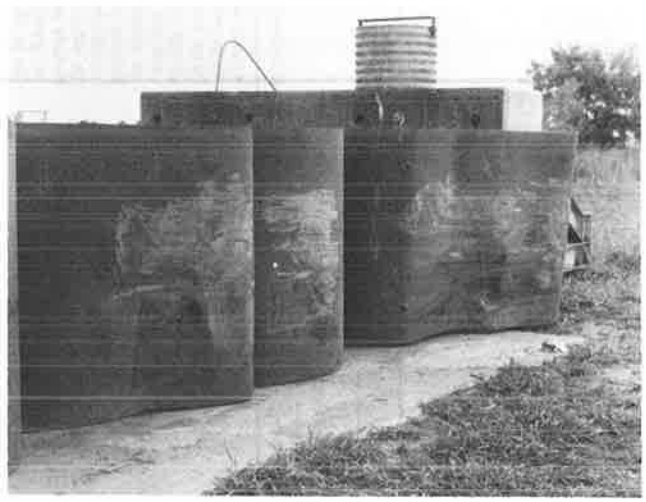

FIGURE 7 Side view of CIAS after Test 6.

FIGURE 8 Side view of CIAS showing test vehicle after Test 6.

FIGURE 9 Side view of CIAS before Test 7.

3. Test 8 (October 4, 1983)

a. System tested: Same as that in Tests 6 and 7 (see Figure 1).

b. Test vehicle: A Honda Civic (1977) impacted the attenuator at 60.9 mph and 0 degrees. The vehicle weighed 1,800 lb with 1,069 lb on the front axle and 731 lb on the rear axle (Figure 13).

c. Test results: The vehicle fully collapsed the first five rows of the attenuator, but the back two rows of the system were deformed only slightly (Figures 14 and 15). The front end of the vehicle sustained an average crush of 9 in. (Figure 16). Two occupant impact velocities are reported in Table 1 for this test. As in Test 2, they correspond to occupant travel distances of 1.25 and 2.0 ft.

FIGURE 10 Test vehicle before Test 7.

FIGURE 13 Test vehicle before Test 8.

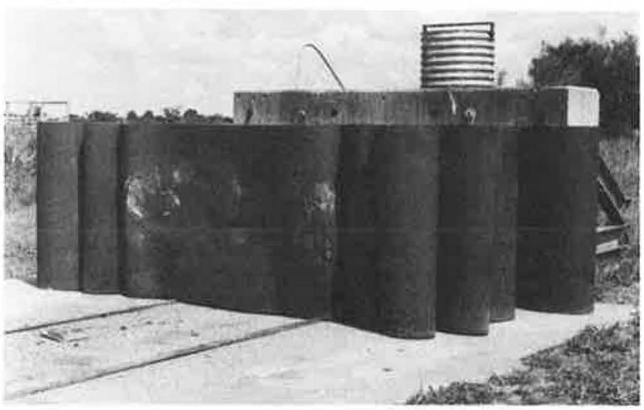

FIGURE 11 Front angular view of CIAS after Test 7.

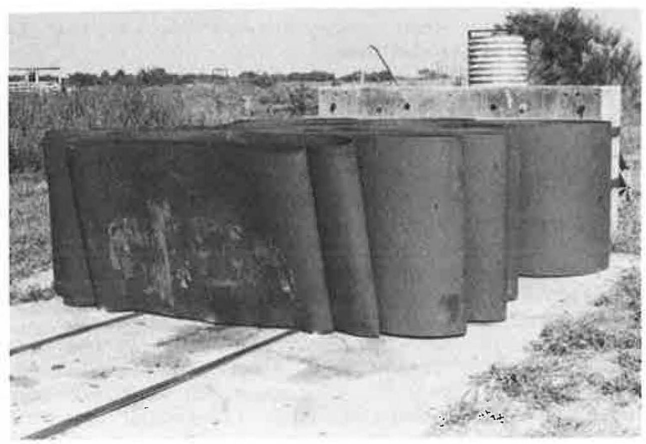

FIGURE 14 Side view of collapsed system, Test 8.

FIGURE 12 Frontal damage sustained by vehicle in Test 7.

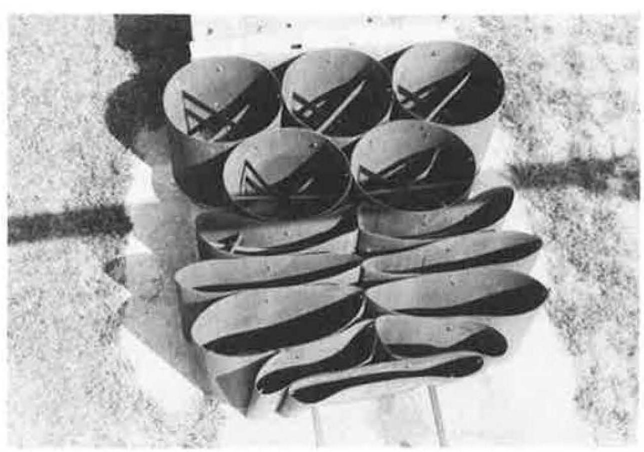

FIGURE 15 Top view of collapsed system, Test 8.

FIGURE 16 Damage sustained by vehicle, Test 8.

4. Test 9 (October 6, 1983)
   a. System tested: Same as that in Tests 6-8 (see Figure 1).
   b. Test vehicle: A Chrysler Newport (1979) impacted the attenuator at 61.6 mph and 0 degrees. The vehicle weighed 4,500 lb with 2,358 lb on the front axle and 2,142 lb on the rear axle.
   c. Test results: This test is a repeat of Test 7 with refurbished CIAS materials. The unit was composed of 14 tubes used in previous crash tests to demonstrate that restored tubes would exhibit the same energy-dissipation behavior as virgin tubular sections.

Two major points were considered during the planning of the restoration process. First, the cost was to be held down without sacrificing quality. Second, the final process was to be one that could be practically performed on an attenuator in actual use after an impact.

The 4-ft-high tubes were available from four earlier crash tests: Tests 4, 5, 6, and 7. The prior location of each steel tube used in the restored unit and the action taken to correct the damage are summarized in Table 2. Six tubes had no previous damage: five of these (J, K, L, M, and N) were left undamaged from Test 6; the other (C) had not been used before. It contained thin 1/8-in. straps that were replaced with the correct pipes and straps. Three tubes (B, D, and E) were rerounded by placing hydraulic jacks inside them. Two tubes (F and G) were only slightly out of round. Neither contained bracing, but the addition of the needed pipes and straps made the tubes round again.

The remaining three tubes (A, H, and I) were rerolled by a commercial metal fabricator. Before the tubes could be rerolled, all protrusions such as seam welds and bracing had to be removed and ground smooth. Tube A had not been severely damaged. The

TABLE 2 Summary of Refurbished Tubes Used in Crash Test 9

Restored Unit Tube	Prior Location	Corrective Action
A	A, Test 6	Rerolled and steel added
B	B, Test 6	Rerounded with jacks
C	E, unused	Bracing replaced with correct type
D	D, Test 5	Rerounded with jacks
E	E, Test 6	Rerounded with jacks
F	H, Test 5	Rerounded with steel addition
G	I, Test 6	Rerounded with steel addition
H	J, Test 7	Rerolled
I	K, Test 7	Cut and reverse rolled
J	J, Test 5	Left undamaged
K	K, Test 6	Left undamaged
L	L, Test 5	Left undamaged
M	M, Test 5	Left undamaged
N	N, Test 5	Left undamaged

FIGURE 17 Tube A mounted in rolling mill.

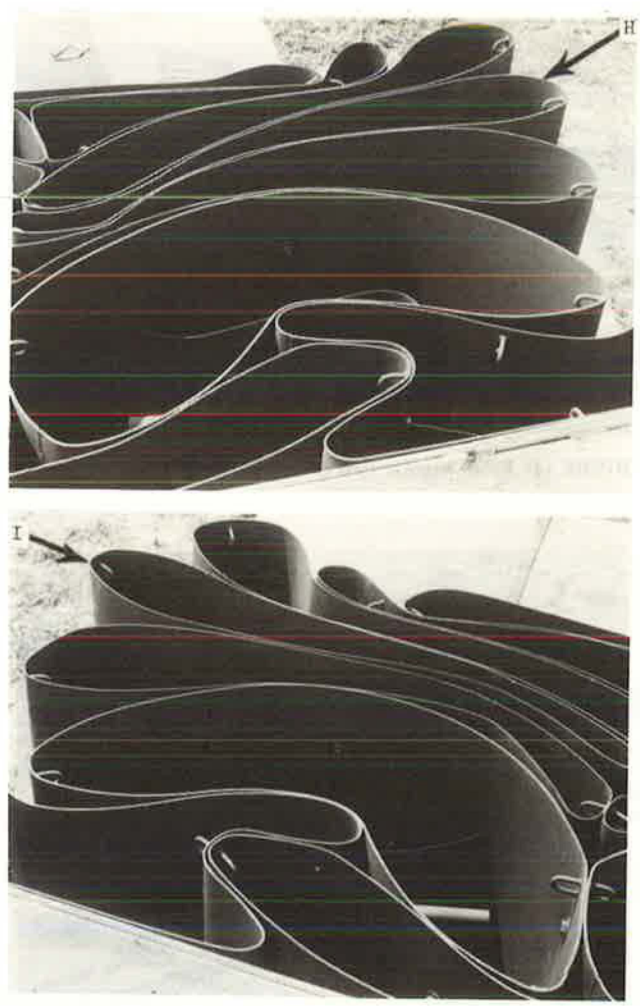

FIGURE 18 Tubes H and I after Test 7.

pipes and straps were removed and it was rerolled from the existing semiround shape. Figure 17 shows Tube A mounted in a rolling mill before it was rerolled. After 1.5 hr of rerolling, new pipes and straps were added to the tube.

Figure 18 shows Tubes H and I, which had been severely deformed, before each was removed from the previous attenuator. After 3 hr of rerolling, Tube H looked as it does in Figure 19. A different method of rerolling was tried for Tube I: it was cut along the seam with a torch. Figure 20 shows Tube I after cutting. It was then flattened, rolled on the reverse side, and rewelded. A total of 2.5 hr was spent cutting, rolling, and welding the tube.

It can be seen from Figures 21 and 22 and Table 1 that the system's response in Test 9 was essentially identical to that in Test 7. The only discernible difference to be reported concerns the relative stiffness of the front ends of the test vehicles employed in Tests 7 and 9. The Chrysler Newport was significantly stiffer than the Plymouth Salon used in Test 7, sustaining an average crush of only 8 in. This very successful test proves that collapsed tubes can be economically restored and used again in the CIAS without affecting system performance.

DISCUSSION

The final design of the CIAS evolved during the first phase of the testing program. No design changes were made subsequent to the fifth crash test. Tests 6-9 all exhibited excellent performance characteristics with respect to both NCHRP Report 230 and TRC 191.

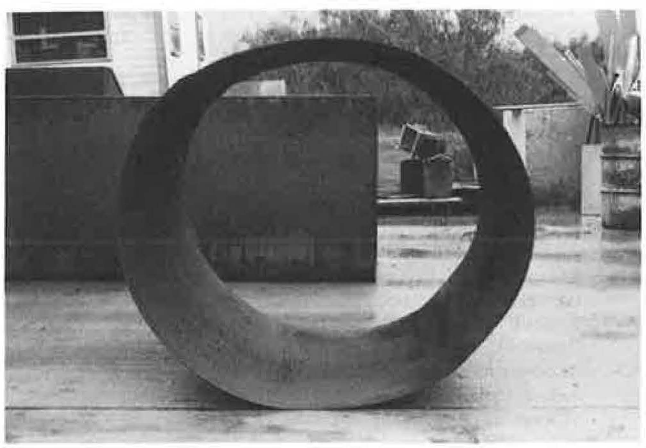

FIGURE 19 Tube H after rerolling.

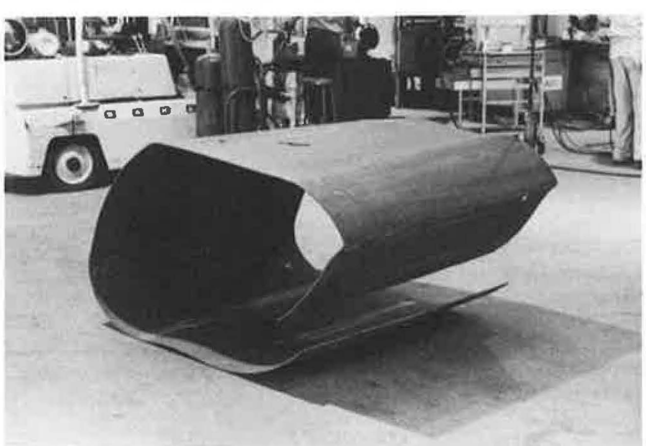

FIGURE 20 Tube I after cutting.

FIGURE 21 Before Test 9: test vehicle (top) and top view of CIAS (bottom).

The CIAS possesses unique trapping and redirectional characteristics. An extensive full-scale crash testing program (16 tests) (8) has verified the effectiveness of the system, which has a unit fabrication cost of $4,200. ConnDOT installed four such systems in the field in 1984. The locations were selected by the Office of Research and other affected units based on field experience. ConnDOT research personnel will monitor the performance of the CIAS, working closely with maintenance, design, traffic, and law enforcement personnel to obtain sufficient data to evaluate the effectiveness of the system. A 3-year performance evaluation is planned during which a frequent regular inspection routine will be set up. The inspectors will be equipped with cans of spray paint to cover scrape marks on the tubes caused by minor hits. With such a procedure, brush-type hits can be easily detected.

ConnDOT has produced a short narrated color film to document the construction of the units, highlight the crash-testing program, describe how the system is installed in the field, and summarize available performance data. Information regarding this film can be obtained from the second author of this paper.

SUMMARY AND CONCLUSIONS

The development of a new crash cushion is described in this paper. This impact-attenuation device is composed of steel tubular members formed from

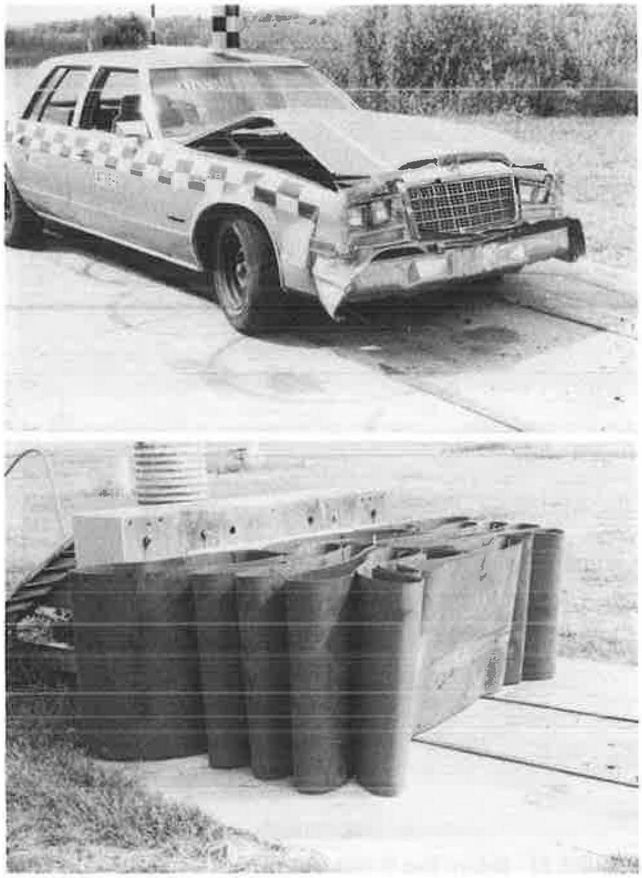

FIGURE 22 After Test 9: damage sustained by test vehicle (top) and side view of collapsed system (bottom).

straight plate sections, which are bolted together to form a cluster. This device is unique in that it will trap an errant vehicle under most impact conditions. The vehicle will be redirected back out into the roadway only when the impact location is so close to the rear of the system that it is impossible to obtain an acceptable energy-dissipation and deceleration-trapping response because of the proximity of the site hazard. No other attenuation system in use today possesses this capability.

In addition, the Connecticut impact-attenuation system exhibits the following characteristics:

1. It satisfies the impact performance standards outlined in TRC 191 and NCHRP Report 230;
2. It is inexpensive to fabricate;
3. It is inexpensive to repair after impact (Test 9 demonstrated that collapsed tubes can be restored to their original circular configurations and reused), and the energy-dissipating tubes can be refurbished and reused;
4. There is no flying debris associated with the crash event; and
5. It is constructed of readily available materials.

ACKNOWLEDGMENT

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REFERENCES

1. J.F. Carney III. Experimental Evaluation of a Portable Energy-Absorbing System for Highway Service Vehicles: Final Report for Phase I. Report 402-1-77-3. Connecticut Department of Transportation, Wethersfield, Jan. 1977.
2. J.F. Carney III. Experimental Evaluation of a Portable Energy-Absorbing System for Highway Service Vehicles: Final Report. Report 402-F-79-1. Connecticut Department of Transportation, Wethersfield, Dec. 1978.
3. Portable Crash Cushion Saves Lives of Maintenance Crews. TR News, No. 107, July-Aug. 1983, p. 4.
4. J.F. Carney III and D.A. Larsen. Accident Experience with Connecticut Crash Cushion. Report 343-16-80-19. Connecticut Department of Transportation, Wethersfield, Dec. 1980.
5. R.V. Anderson. Crash Attenuator Saves Life of Minnesota Maintenance Man. TR News, No. 104, Jan.-Feb. 1983.
6. P. Brown, P. Geib, and R. Leonard. Monitoring Field Installations of Impact Energy Attenuators by Videotape, Phase II. Report FHWA/CA/07/80/01. California Department of Transportation, Sacramento, July 1980.
7. J.F. Carney III. Report on Model Tests Conducted at the University of Cambridge, England. Report 722-1-80-13. Connecticut Department of Transportation, Wethersfield, July 1980.
8. J.F. Carney III. Development and Experimental Evaluation of a Steel-Tube Vehicle Impact Attenuation System: Final Report. Report 722-F-81-13. Connecticut Department of Transportation, Wethersfield, Sept. 1981.
9. S.R. Reid, J.F. Carney, III, and S.L.K. Drew. Energy Absorbing Capacities of Braced Metal Tubes. International Journal of Mechanical Sciences, Vol. 25, No. 9-10, 1983, pp. 649-667.
10. J.F. Carney III and J.R. Veillette. Impact Response and Energy Dissipation Characteristics of Stiffened Metallic Tubes. Structural Impact and Crashworthiness, Vol. 2, 1984, pp. 564-575.
11. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. Transportation Research Circular 191. TRB, National Research Council, Washington, D.C., Feb. 1978.
12. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.
13. J.F. Carney III and C.E. Dougan. Summary of the Results of Crash Tests Performance on the Connecticut Impact-Attenuation System. Report 876-1-83-13. Connecticut Department of Transportation, Wethersfield, Dec. 1983.

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Structural Performance Levels for Portable Concrete Barriers

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ABSTRACT

There is a significant variation in the structural performance of different types of portable concrete barriers (PCBs) because of variations in connection strengths. Results of 20 full-scale crash tests on PCBs are examined and relationships between PCB connection strength and structural performance are established. On the basis of this information, five different service levels are proposed to classify PCB structural performance. These service levels are based on estimates of connection shear, torsion, and bending strength. This information can be used to estimate the structural performance of existing barriers or it can be used as a guide in barrier design.

During the past several years the use of the portable concrete barrier (PCB) as a longitudinal construction-zone barrier has become widespread. The construction-zone PCB consists of several precast PCB segments that are transported to the construction zone and connected end to end. The cross-sectional geometries of the PCB segments are patterned after the popular concrete median barrier safety shape. The concrete median barrier has proven to be an acceptable permanent barrier for many applications, and experience has shown that PCBs have performed well as construction barriers (1).

In general, the strength of PCB connections is much less than that of the PCB segments away from the connections. Therefore, the overall strength of the PCB is controlled by the strength properties of its connections. A survey of different types of connections in use reveals that there is a significant variation in their respective structural capacities. Hence, there is a significant variation in the potential structural performance of PCBs. Variations of PCB performance have been predicted by using computer simulations and have been observed in full-scale crash tests (1,2).

A straightforward procedure is presented to estimate the structural performance of PCBs on the basis of the strength properties of the connections. To do this, five different structural performance levels are defined based on the energy associated with the lateral component of velocity of the impacting vehicle. This energy is termed the impact severity (IS) in NCHRP Report 230 (3). Then existing full-scale PCB impact tests are examined and the IS for each test is calculated. In addition, the strength properties of the various connections represented in the crash tests are estimated by using simplified structural analyses. These results are then combined to make conservative estimates of the connection strength properties necessary to achieve each level of the structural performance scale. The issue of vehicle stability is not addressed.

PCB STRUCTURAL PERFORMANCE CRITERIA

Of primary concern in assessing the structural performance of PCBs is their relative capability to redirect impacting vehicles. Three service levels for classifying the strength of longitudinal barriers are recommended in NCHRP Report 230 (3). The authors used these three service levels in combination with two additional ones to develop the five PCB structural service levels presented in Table 1, which are based on the mass, velocity, and angle of impact of the most severe impact that the barrier is capable of withstanding.

IS is given as follows:

$$IS = 1/2\ (w/g)\ (V_i \sin \theta)^2 \qquad (1)$$

where

V_i = impact velocity,
w = weight of the impacting vehicle,
g = acceleration of gravity, and
θ = angle of impact (3).

The impact severity is a convenient measure of the relative severity of automobile impacts. In general, the impact severity may not always be an accurate indicator of the impact forces. However, for barriers of similar construction and stiffness, such as PCBs, it is a reasonable indicator of the relative magnitude of these forces. The minimum IS values corresponding to the five structural performance levels in the rating system are presented in Table 1.

TABLE 1 PCB Service Levels Compared with NCHRP 230 Service Levels

PCB Service Level	Corresponding NCHRP Level (3)	Collision Characteristics: Weight (kips)	Speed (mph)	Angle (degrees)	IS (kip-ft)
A	-	4.5 or 3.5	45 or 60	15	20.4
1	1	4.5	60	15	36.5
2A	2	4.5	60	25	97.3
2B	-	20	60	15	161.1
3	3	40	60	15	322.2

FULL-SCALE CRASH-TEST DATA

During the past 10 years, a total of 20 full-scale crash tests have been conducted on different PCBs.

TABLE 2 Summary of PCB Tests

Testing Agency	Test No.	Test Conditions Speed (mph)	Angle (degree)	Weight (kips)	Segment Length (ft)	Static Deflection (ft)	Data Point No.	Test Results and Comments
TTI	TX-1	60.9	17.8	4.5	15	0.9	1	Smooth redirection; negligible barrier damage
TTI	TX-2	55.9	26	4.51	15	1.3	2	Smooth redirection; negligible barrier damage
TTI	3825-7	59.2	25	4.5	12	1.8	3	Smooth redirection; slight barrier damage
TTI	3825-6	60.1	24	4.5	12	1.8	4	Vehicle redirected but rolled after recontact with pavement subsequent to primary collision; slight barrier damage
TTI	3825-5	60.7	25	4.5	12	1.6	5	Smooth redirection; slight barrier damage
TTI	3825-9	63.4	25	4.51	12	6.5	6	Smooth redirection; side plates failed; slight barrier damage
TTI	3825-8	57.7	15	20.0	15	1.8	7	Bus redirected but rolled 90 degrees onto side after collision; slight barrier damage
TTI	CMB-2	60.0	24	4.54	30	1.1	8	Smooth redirection; negligible barrier damage
Caltrans	291	65	7	4.86	12.5	0.5	9	Smooth redirection; slight barrier damage
Caltrans	292	68	23	4.86	12.5	1.9	10	Vehicle redirected but penetrated over top of barrier and slid sideways along top; segment fractured; major barrier damage
Caltrans	293	66	40	4.86	20	NA	11	Vehicle penetrated and rolled; segment tipped over; major barrier damage
Caltrans	294	39	25	4.7	20	0.5	12	Smooth redirection; steel vertical connection rods severely bent; significant barrier damage
SWRI	CMB-18	62	25	4.5	20	NA	13	Vehicle redirected; flexural failure in the segments; major barrier damage
SWRI	CMB-24	56	24	4.5	20	3.4	14	Vehicle redirected; joint failures; significant barrier damage
New York	NY-17	53	25	4.25	20	1.3	15	Smooth redirection; slight barrier damage
New York	NY-18	58	25	4.23	20	0.9	16	Vehicle redirected but rolled after recontact with pavement subsequent to primary collision; slight barrier damage
New York	NY-44	65	25	4.3	8	1.4	17	Vehicle redirected but subsequently rolled; slight barrier damage
New York	NY-45	66	15	2.18	8	0.3	18	Vehicle redirected but could have rolled; slight barrier damage
New York	NY-46	61	25	4.35	8	0.6	19	Vehicle redirected; slight barrier damage
New York	NY-47	61	15	2.18	20	0.3	20	Vehicle smoothly redirected; no significant barrier damage

Note: TTI = Texas Transportation Institute; Caltrans = California Department of Transportation; SWRI = Southwest Research Institute.

These tests were conducted by independent research organizations in California, Texas, and New York (2). General descriptions of the test conditions and results are presented in Table 2.

The structural performance of each PCB included in Table 2 is classified as either good or poor on the basis of NCHRP Report 230 criteria and the level of damage experienced by the barrier. The PCBs identified by data point numbers 6, 10, 13, and 14 were judged to exhibit poor structural performance. The remainder of the PCBs were judged to have exhibited good structural performance.

Previous evaluations of crash-test data have shown that the strength of PCBs is controlled to a large extent by the structural properties of the connections. The important structural properties of the connections are shear, bending, and torsional resistance (1,2). Further, it has been shown that acceptable estimates of these structural properties can be achieved by using the structural details of the connections and simplified structural analysis techniques (2).

There are seven different basic connection configurations represented in Table 2. General details of these seven connection configurations are presented in Figures 1-7. By using these details and specific connection details available from the respective testing agencies, estimates of the structural properties of the connections associated with each of the 20 crash tests were calculated (2). These data are presented in Table 3. Included in Table 3 are calculated values of IS and estimates of the connection slack in degrees. The connection slack is defined as the joint rotation before the connection exhibits significant flexural resistance. Excessive connection slack can result in excessive barrier deflection during an impact.

Figures 8, 9, and 10 are plots of connection strength versus impact severity. In some cases, the connection strength was greater than that required to resist the impact force. The performance of these PCBs is plotted as open triangles. In the other cases, the connection strength was less than that required to resist the impact forces. The performance of these PCBs is plotted as solid triangles. As would be expected, there is a boundary between the good performance data points and the poor performance data points for each strength property. This boundary corresponds to the minimum connection strength required to resist an impact of a given severity. The precise location of the boundary is not always well defined by the available test data.

In the absence of more definitive information, conservative locations for the boundaries between good and poor performance are determined by defining a lower-bound envelope on the good data points with two lines. These lines are located by using three control points for each structural property. The locations of these control points are based on the 20 data points discussed earlier, related information, and the goal of reaching conservative strength requirements. Logically, the magnitude of the shear, flexural, and torsional connection capacities required to resist a given level of impact must increase as the impact severity increases. The greater the impact severity, the more connection strength that is required. This trend is evident in the boundary lines indicated in Figures 8, 9, and 10 with dashed lines. The rationale behind location of the three control points for each structural property is discussed in the following.

For the lowest service level (A), the characteristics of the Virginia tongue and groove connection were used as control points. Details of the Virginia tongue and groove connection are similar to those of the partial tongue and groove with side plates (Figure 2), except that there are no side plates. The structural properties of this connection were calculated to be 32 kips for shear, 0 kip-ft for moment, and 7 kip-ft for torsion (2). These values were assigned to the service level A control points in each graph. This may appear arbitrary,

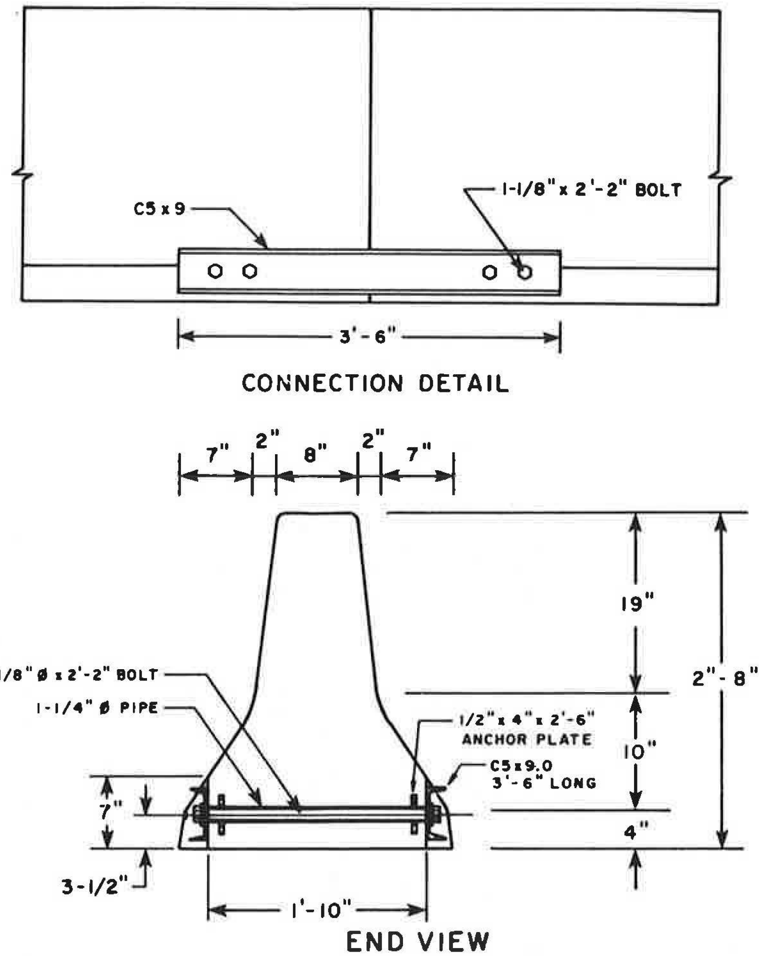

FIGURE 1 Side plate or side channels (channel splice).

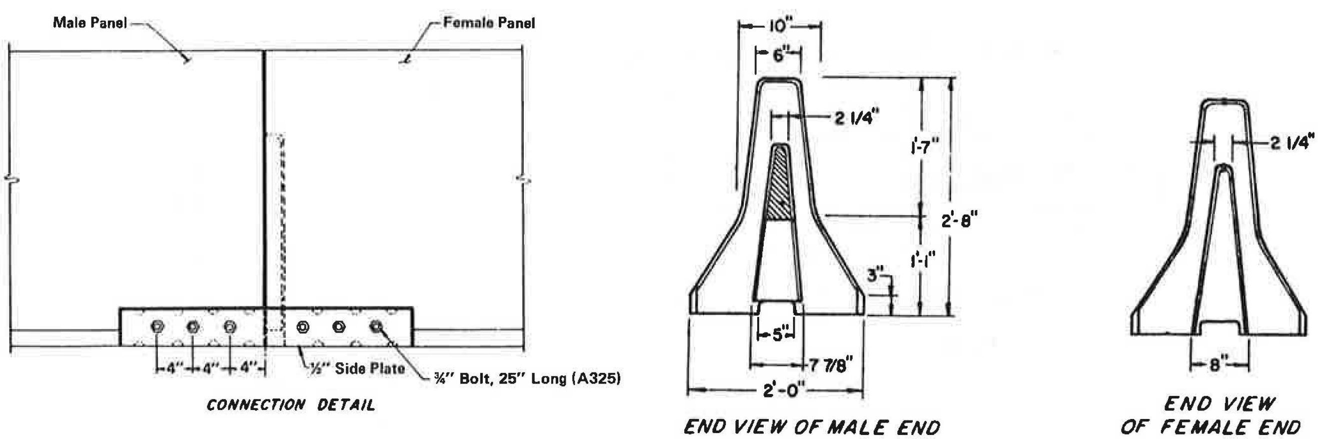

FIGURE 2 Partial tongue and groove and side plates.

because the Virginia tongue and groove connection has not been subjected to formal testing. However, it is the opinion of the authors that this barrier will meet at least service level A criteria based on favorable field performance reported in the literature (4).

In a related research project, Buth measured the maximum normal force between an impacting automobile and barrier for level 2A and 2B impacts (5). These data provide upper limits for the shear forces that must be resisted by connections at these impact levels. These measured upper-limit shear forces appear to be consistent with the shear strength data presented in Figure 10. Therefore, these two points are used in combination with the Virginia tongue and groove point to define the boundary line between good and poor shear strength performance. This boundary line is presented in Figure 10.

Examinations of data points 3, 4, 5, and 6 in Tables 2 and 3 show that a good connection performance at a level 2A test can be achieved with a nominal moment capacity of 50 kip-ft. These data

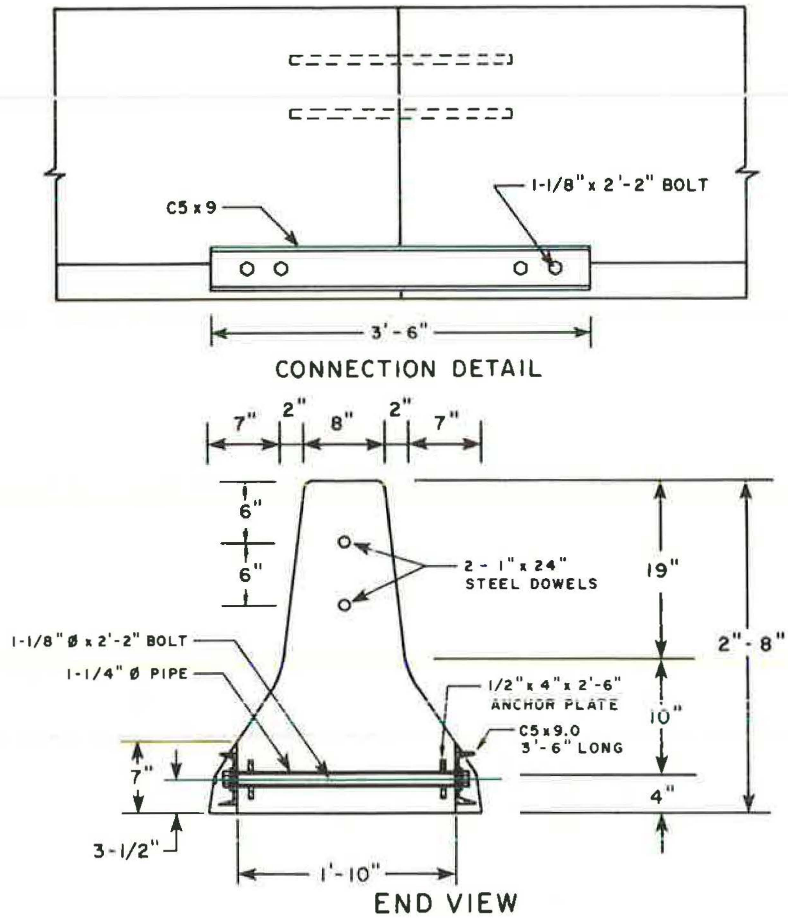

FIGURE 3 Side plate or side channels with steel dowel shear connection (channel splice).

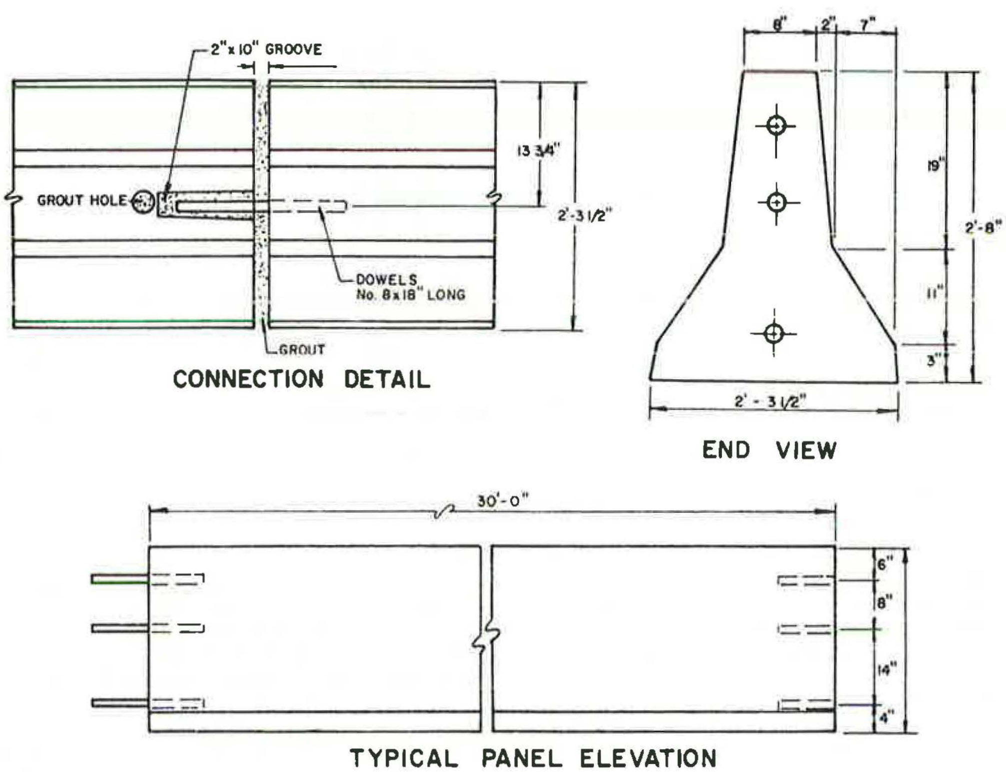

FIGURE 4 Grouted dowels (steel dowel joint).

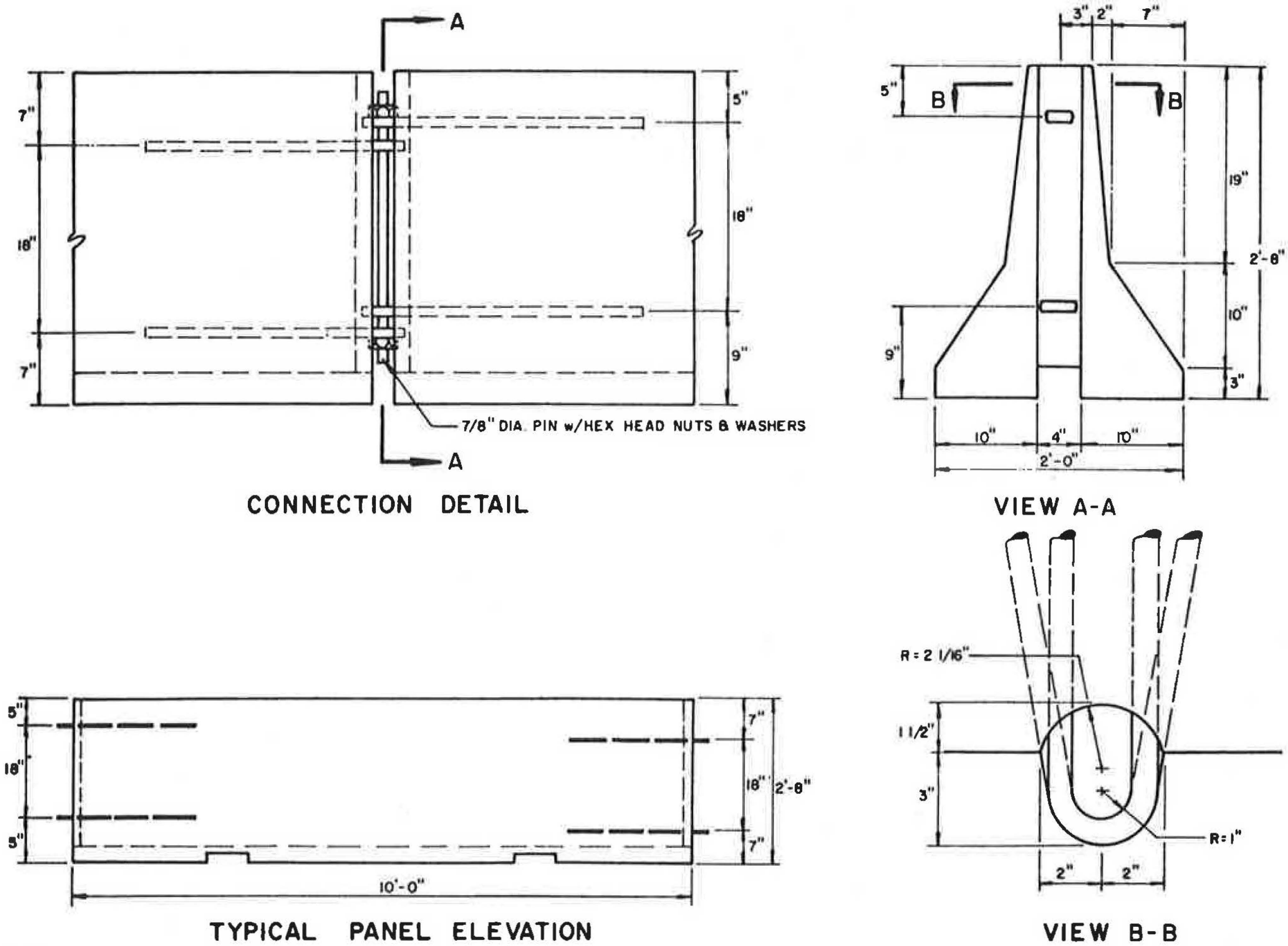

FIGURE 5 Vertical steel pin (pin and rebar).

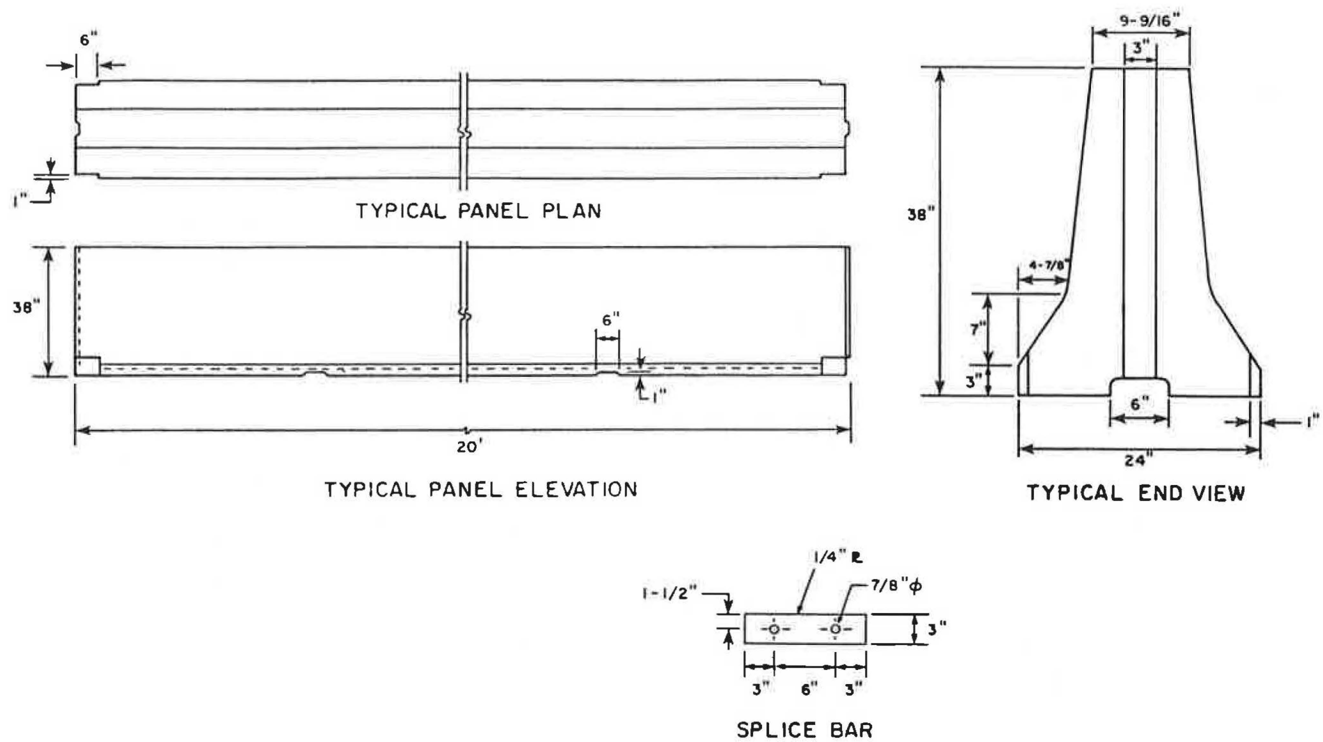

FIGURE 6 Tongue and groove and side plates.

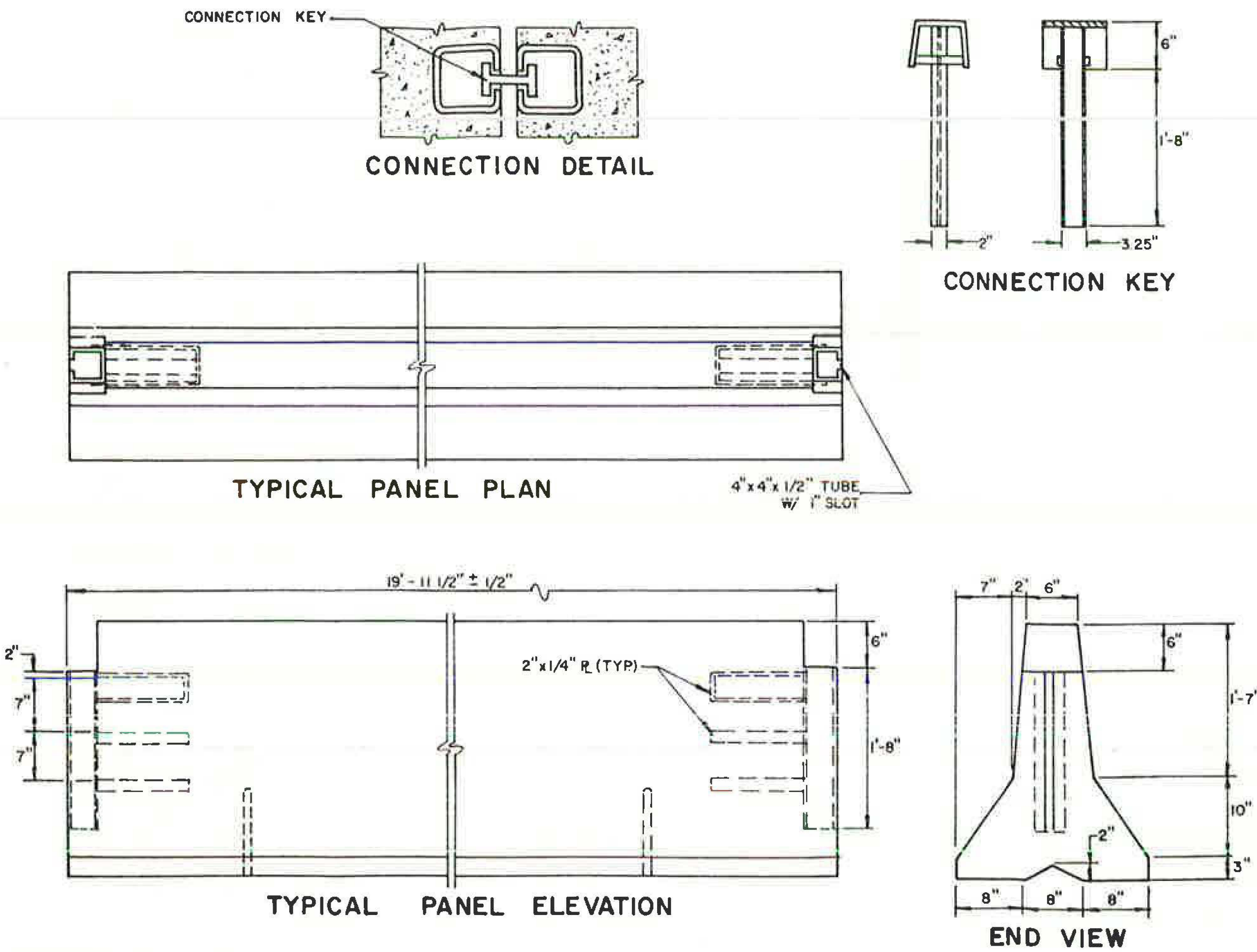

FIGURE 7 Vertical I-beam.

TABLE 3 Summary of PCB Connection Properties

Data Point No.	Connection Description	Connection Slack (degrees)	Connection Capacities: Shear (kips)	Moment (kip-ft)	Torsion (kip-ft)	IS[a] (kip-ft)
1	Side plates (3 ft 6 in. x 5 in x 1/2 in., steel) (Figure 1)	5	90	117	53	52.1
2	Side channels (C5 x 9 x 3 ft 6 in., steel) (Figure 1)	3	90	117	53	90.5
3	Partial tongue and groove and side plates (3 ft 0 in. x 4 in. x 1/2 in. steel) (Figure 2)	3	76	103	67	94.1
4	Partial tongue and groove and side plates (3 ft 0 in. x 4 in. x 3/8 in. steel) (Figure 2)	3	57	77	52	89.8
5	Partial tongue and groove and side plates (3 ft 0 in. x 4 in. x 1/2 in. steel) (Figure 2)	3	38	52	37	98.9
6	Partial tongue and groove and side plates (3 ft 0 in x 4 in. x 1/8 in. steel) (Figure 2)	3	19	26	22	108.2
7	Side channels (C5 x 9 x 3 ft 6 in. steel) (Figure 3) plus three no. 8 x 18 in. steel rebar dowels	3	135	117	73	149.2
8	Three grouted dowels (no. 8 x 18 in.) (Figure 4)	0	60	50	37	90.3
9	Vertical steel pin (7/8 in. ø x 26 in.) (Figure 5)	9	46	31	35	10.2
10	Vertical steel pin (7/8 in. ø x 26 in.) (Figure 5)	9	46	31	35	114.6
11	Vertical steel pin (1 in. ø x 26 in.) (Figure 5)	8	55	40	42	292.2
12	Vertical steel pin (1 in. ø x 26 in.) (Figure 5)	8	55	40	42	42.6
13	Tongue and groove and side plates (12 in x 3 in. x 1/2 in. steel) (Figure 6)	3	27	9	16	103.2
14	Tongue and groove and side plates (12 in x 3 in. x 1/4 in. steel) (Figure 6)	3	27	9	16	77.8
15	Vertical I-beam (3 1/4 in. x 2 in.) (Figure 7)	10	208	61	87	71.2
16	Vertical I-beam (3 1/4 in. x 2 in.) (Figure 7)	0	208	61	87	86.3
17	Vertical I-beam (3 1/4 in. x 2 in.) (Figure 7)	10	208	61	87	108.4
18	Vertical I-beam (3 1/4 in. x 2 in.) (Figure 7)	10	208	61	87	21.2
19	Vertical I-beam (3 1/4 in. x 2 in.) (grouted joints) (Figure 7)	0	208	61	87	96.6
20	Vertical I-beam (3 1/4 in. x 2 in.) (Figure 7)	10	208	61	87	18.1

[a]The IS is calculated by using the data presented in Table 2 and Equation 1.

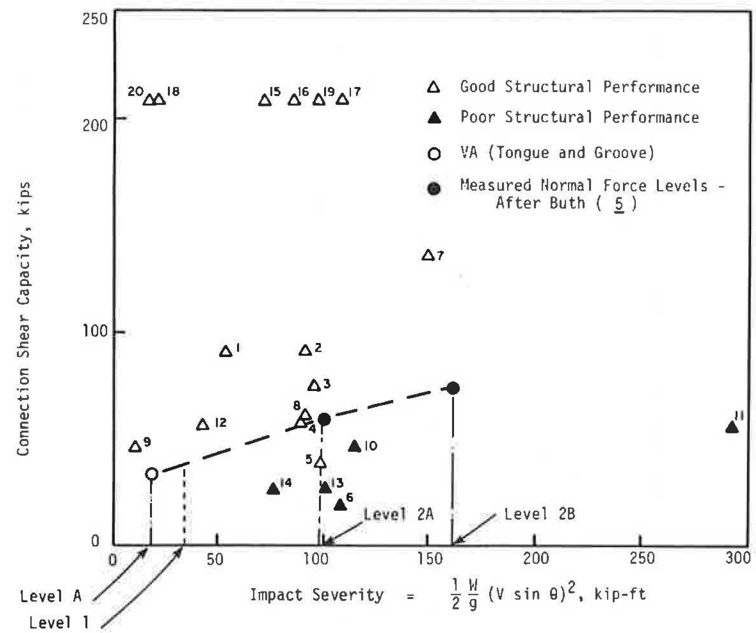

FIGURE 8 Shear capacity of connection versus impact severity.

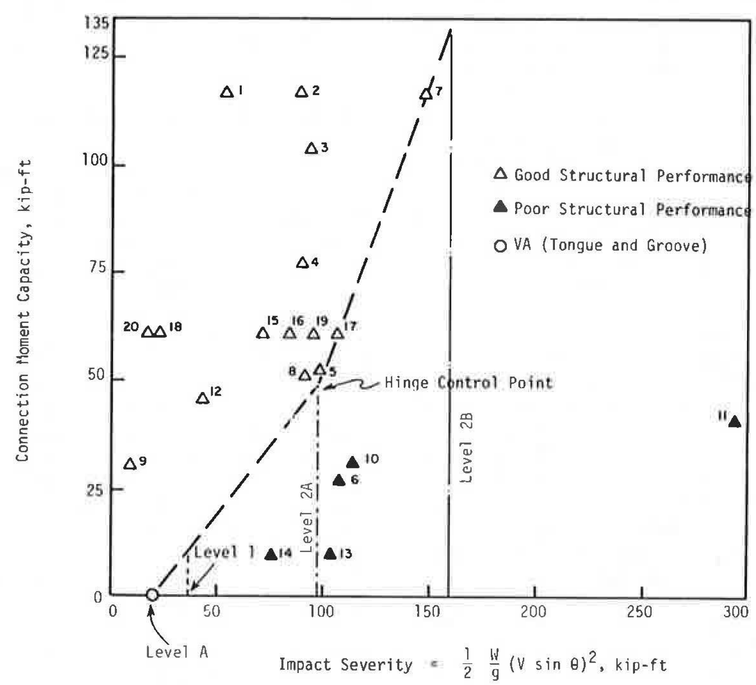

FIGURE 9 Moment capacity of connection versus impact severity.

points were used to establish a conservative control point as shown in Figure 9. Another control point for the moment capacity boundary line is established by using either point 17 or point 7. These control points are combined with the Virginia tongue and groove control point to establish boundary lines for moment capacity. By setting the boundary lines in this manner it appears that the required moment capacities for the service levels above 2A are probably quite conservative.

Further, examinations of data points 3, 4, 5, and 6 show that a nominal torsion capacity of 40 kip-ft is required to achieve a service level of 2A. This value was used as a control point. The second control point is established by data point 7. The use of data point 7 as the second control point may result in overdesigning barriers at the 2B impact energy level. The gap between points 7 and 11 through which the boundary line must pass is wide. The placement chosen here is likely to be highly conservative. The third control point is again established with the Virginia tongue and groove point. As with the flexural capacity, the torsional boundary line probably overestimates to some degree the required torsional strength for most connections.

PCB PERFORMANCE CRITERIA

The information presented in the previous section provides a relationship between the strength prop-

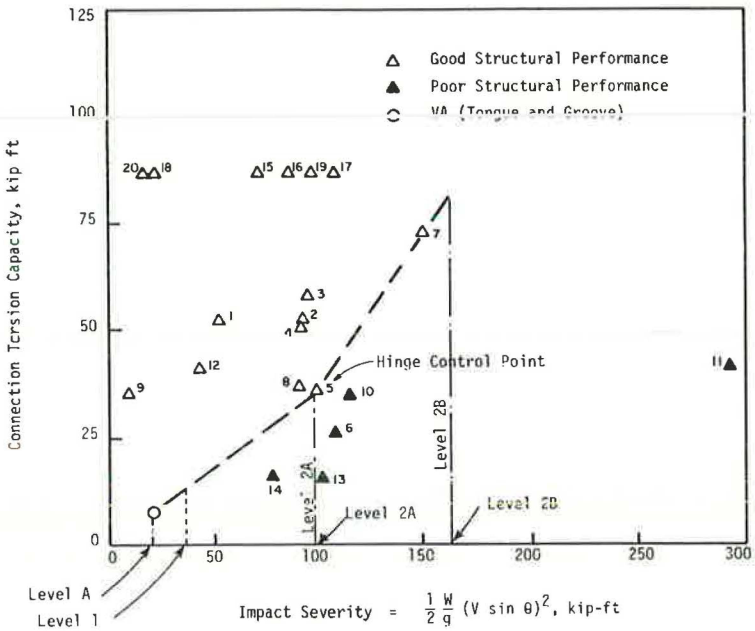

FIGURE 10 Torsion capacity of connection versus impact severity.

erties of PCB connections and the IS. This information allows the PCB structural service levels to be stated in terms of the estimated strength properties of the connections as shown in Table 4, which can be used to estimate the structural performance of existing PCBs. In addition, the information can be used as a design guideline for PCB connections. However, use of this information is not intended to supplant the need for full-scale testing.

In addition to strength considerations, adequate barrier performance often depends on the lateral deflection during impact. Experience suggests that a PCB may not perform adequately if the lateral deflections are greater than 2 ft (2). Further, the permissible lateral deflection based on available work-zone space varies significantly from site to site. The amount of lateral deflection that a particular PCB experiences has been shown to be primarily a function of three factors: the moment capacity of the connection, the amount of slack in the connection before development of the flexural resistance, and the length of the PCB segment (1,2). Guidance regarding the calculation of barrier deflection with variations of the three factors listed previously is available elsewhere (1,2).

Barrier connection strengths and barrier deflections are not the only factors that need to be considered in determining the safety performance of PCBs. For a full evaluation of safety, applicable sections of NCHRP Report 230 should be considered (3). Especially important is the criterion of rolling. Achieving structural connection adequacy and limiting deflection will not, in all cases, prevent vehicle rolling, as the testing to date illustrates.

TABLE 4 PCB Structural Service Levels

PCB Service Level	IS (kip-ft)	Minimum Shear Strength (kips)	Minimum Torsional Strength (kip-ft)	Minimum Flexural Strength (kip-ft)
A	20.4	30	10	0
1	36.5	40	15	10
2A	97.3	60	40	50
2B	161.1	75	80	130
3[a]	322.2	150	160	260

[a]The strength values for this interval are highly speculative. They were determined by multiplying the strength values for service level 2B by the ratio of the impact severities of service levels 3 and 2B.

COMPARISONS OF DIFFERENT CONNECTION DETAILS

Most PCB connections in use today can be placed into one of 10 different generic categories, arbitrarily designated C1 through C10. Specific details of these categories were presented by Ivey and Buth (2). In Table 5 the strength characteristics of the 10 different connection categories are presented, which were determined by using a uniform set of material strength properties (2). These relative strengths do not necessarily represent the strength of any particular design. Each connection could be made stronger or weaker by using different materials. The purpose of this exercise is to compare generic types of connections, not specific connection designs.

It may be seen that connections C1, C2, and C3 are rated as service level A because they lack significant moment capacities. Connections C4 and C5 are rated as service level 1, and connections C6, C7, C8, and C9 are rated as service level 2A. Connection C10 is qualified as service level 2B. Examination of Table 5 suggests that the classification is dominated by the moment capacity requirement for levels 1 and higher.

Connection C10 is the only connection analyzed that appears to meet service level 2B. This does not mean that it is the only design that can meet 2B. Connections C6 through C9 could all be designed to meet the 130-kip-ft moment capacity. Likewise, specific connections could be designed to be weaker than indicated in Table 5. Before a particular connection is advocated for a given level of service, a specific analysis of that connection should be made. In addition, other safety-related issues, such as vehicle roll stability, should be addressed. This is particularly true for the higher service levels.

CONCLUSIONS

PCBs have become increasingly popular as longitudinal construction-zone barriers in the past few years.

TABLE 5 Strength Characteristics of Connection Types

Connection Designation	Connection Name	Strength Characteristics[a] Shear (kips)	Moment (kip-ft)	Torsion (kip-ft)	Estimated Service Level
C1	Tongue and groove	32	0	7	A
C2	Steel dowel	60	0	37	A
C3	Grid slot	60	0	30	A
C4	Top T-lock	190	11	56	1
C5	Lapped joint	47	22	24	1
C6	Pin and rebar	85	57	60	2A
C7	Vertical I-beam	210	61	87	2A
C8	Bottom T-lock	590	66	370	2A
C9	Channel splice	67	80	36	2A
C10	Welsbach	160	139	94	2B

[a]These strength characteristics were calculated by using average material strength *(2)*. In many cases these levels are not the same as those for specific designs used in some states.

Examinations of in-service experiences and results of full-scale crash tests show that the strength of PCBs is primarily a function of the PCB connection strength. Further, an examination of the wide variety of different types of PCBs in use around the country reveals a wide variation in PCB connection strength properties, which ultimately leads to a wide variation in PCB performance. Five different service levels are presented in this paper to quantify the structural performance of PCBs on the basis of shear, torsion, and bending strength of the connections. By using these service levels, the expected structural performance of 10 different types of generic connections in common use was classified.

The information contained in this paper can be used to estimate the structural performance of existing barriers or it can be used as a guide in the design of PCB connections. The service levels do not address the stability of the impacting vehicle.

ACKNOWLEDGMENTS

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REFERENCES

1. D.L. Ivey, H.E. Ross, T.J. Hirsch, C.E. Buth, and R.M. Olson. Portable Concrete Median Barriers: Structural Design and Dynamic Performance. In Transportation Research Record 769, TRB, National Research Council, Washington, D.C., 1980, pp. 20-30.
2. D.L. Ivey and C.E. Buth. Barriers in Construction Zones. Research Report 3825. Texas Transportation Institute, Texas A&M University, College Station, March 1984.
3. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.
4. F.N. Lisle and B.T. Hargroves. Evaluation of the Performance of Portable Precast Concrete Traffic Barriers. In Transportation Research Record 769, TRB, National Research Council, Washington, D.C., 1980, pp. 30-37.
5. C.E. Buth, J.S. Noel, A.G. Arnold, and T.J. Hirsch. Safer Bridge Railings. Draft Final Report. Texas Transportation Institute, Texas A&M University, College Station, Feb. 1981.

Emergency Opening System for Authorized Vehicle Lanes

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ABSTRACT

An emergency opening system (EOS) for an authorized vehicle lane was developed and crash tested. The design consisted of two steel box tubes mounted on top of each other. The beams were supported by pins at the ends connected to modified concrete median barrier sections. Factors considered in the development of the system were ease of operation and ability to redirect errant vehicles. Three full-scale crash tests were conducted to evaluate the impact behavior of the design. All the occupant risk values as well as the vehicle trajectory hazard were below recommended values for all the crash tests. In addition, the EOS was still operational after the first two tests. In the third test, the anchorage system for the downstream concrete median barrier failed; however, damage to the EOS was slight.

A $52 million project is under way in Houston to install an authorized vehicle lane (AVL) down the center of Interstate 45. This AVL will provide buses, vanpools, and other authorized traffic with an expressway free from normal traffic congestion over a distance of 13.1 miles (21.1 km). Concrete median barriers (CMBs) will be used to separate traffic within the AVL from the normal I-45 traffic. Limited access to the AVL will ensure smooth flow uninterrupted by unauthorized vehicles. However, in the event of a mechanical problem, minor breakdown (e.g., a flat tire), accident, or other emergency, this limited access will also impede the wrecker or other emergency equipment and cause major traffic congestion. Such an eventuality makes the implementation of a gate or emergency opening system (EOS) for the AVL essential.

The design of an EOS for a CMB involves several key parameters. The EOS must function as a median barrier in its ability to safely redirect errant vehicles and stop them from entering adjacent traffic lanes. This should be achieved without endangering the driver during vehicle redirection. At the same time, the operator of the emergency vehicle must be able to open the EOS into the AVL. This requires that the EOS either be lightweight or include provision for mechanical or electrical devices to aid in its operation. Furthermore, it would be desirable to have an EOS that would remain operational following moderate impacts with little or no maintenance. Guidelines and designs also are needed to properly transition the CMB on both the upstream and downstream ends of the EOS. An EOS meeting these requirements was designed, fabricated, and tested at the Texas Transportation Institute (TTI) proving grounds. Details of the EOS and descriptions of the tests and system performance are presented in the following sections.

EMERGENCY OPENING SYSTEM

The EOS must perform as a median barrier in its ability to safely redirect errant vehicles and stop them from entering adjacent traffic lanes. Furthermore, it must able to be opened and closed by the operator of the emergency vehicle. Finally, the barrier should be relatively inexpensive to build and maintain, and it should not be too difficult to install. Consultation with several state highway departments found that there was no system now in operation that would satisfy all these requirements.

The strength of the EOS was achieved by using two square steel tubes mounted on top of each other and separated vertically by 1.38 in. (3.5 cm). The tubes were mounted between two modified CMB sections 30 ft (8.9 m) long that were separated 30 ft. The details of the EOS design and operation are given in Figures 1 and 2. The size and orientation of the steel members were selected on the basis of information from a computer analysis. The EOS was analyzed with a computer program developed to study the behavior of an automobile striking a deformable barrier of general configuration (1). In the computer program, a dynamic, inelastic large displacement structural analysis problem is solved in two dimensions by using a step-by-step method. The automobile is modeled as a plane body of arbitrary shape surrounded by inelastic springs. During impact, the automobile slides along the barrier. Forces between the automobile tires and the pavement as well as the interaction forces between the automobile and the barrier are taken into account. The barrier is modeled as an assemblage of beams, posts, springs, and damping devices with loads applied to the barrier only at the nodes. For the purposes of this study the barrier was modeled as a system of 20 beam elements. Impact with a large, 4,500-lb (2040-kg) vehicle traveling at 60 mph (96.6 km/hr) and 25 degrees was investigated. The joint loads, 250 kips (34.6 kN) axial and 50 kips (6.9 kN) lateral shear, and deflections from this simulation were used to design all the appurtenances of the EOS.

The connections and supports of the EOS were designed by using the applicable standards (2,3) to transmit and contain the peak loads obtained in the computer simulation. The details of the EOS design have been presented elsewhere (4). The system consists of a 30-ft-long steel beam section, which is pinned at each end to a 30-ft-long modified CMB. A 3.25-in. (8.3-cm) diameter steel pin in quadruple shear transfers the load at each end of the EOS through tongue plates to a base plate bolted to the CMB. Further details of the system, in both as-tested and modified configurations, are available elsewhere (4).

Tests were conducted after the EOS was fabricated to demonstrate the ease of operation by a single emergency vehicle operator. The complete EOS tested was 90 ft (27.4 m) long and cost approximately

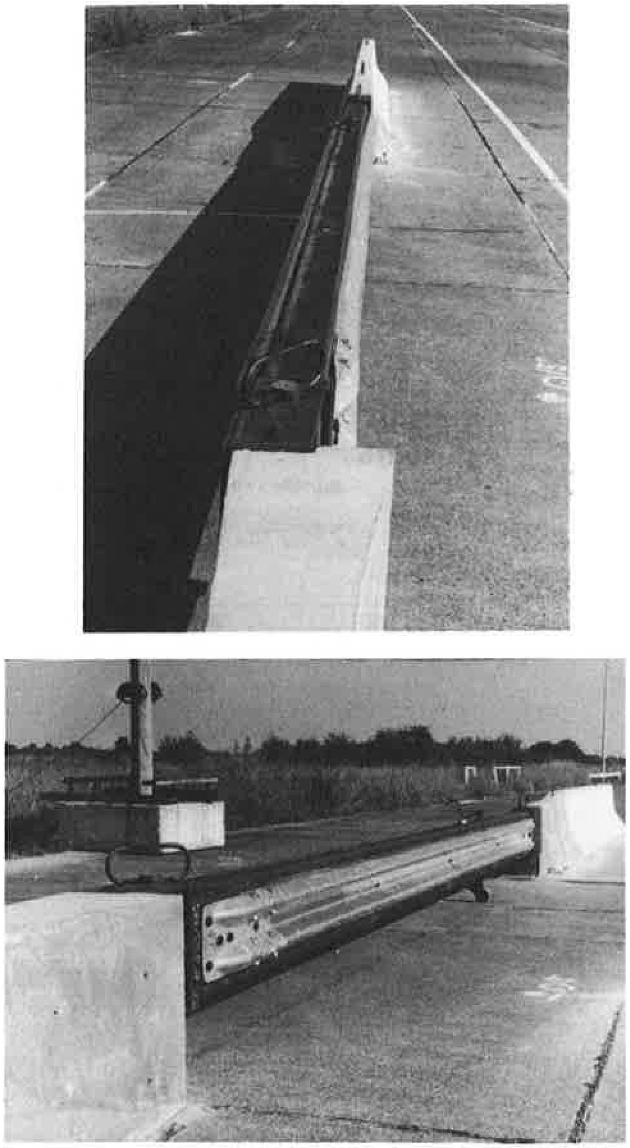


FIGURE 1 Emergency opening system.

$19,300. The cost included two 30-ft-long modified CMB sections. At a cost of $215 per foot ($705 per meter), the barrier compares favorably with other alternatives. The average cost of repairing the EOS after three full-scale crash tests was approximately $300. This value does not include the cost to replace the downstream CMB section after the third test.

TEST DESCRIPTION

Instrumentation

Test vehicles were equipped with triaxial accelerometers mounted near the center of gravity. Yaw, pitch, and roll were sensed by on-board gyroscopic instruments. The analog signals were telemetered to a base station for recording on magnetic tape and display on a real-time strip chart. Provision was made for transmission of calibration signals before and after the test, and an accurate time reference signal was simultaneously recorded with the data.

Tape switches near the impact area were actuated by the vehicle to indicate elapsed time over a known distance to provide a quick check of impact speed. The initial contact also produced an "event" mark on the data record to establish the instant of impact.

High-speed motion pictures were obtained from various locations, including overhead, to document the events and provide a time-displacement history. Film and electronic data were synchronized through a visual-electronic event signal at initial contact.

Crash Test Results

Three full-scale crash tests, designed to evaluate the limits of performance of the barrier, were conducted on the EOS. The vehicle impact point for Tests 1 and 3 was 6 ft (1.8 m) upstream from the downstream end of the gate system. This point of impact should cause the maximum forces on the CMB anchorage system and the maximum forces on the steel gate to the CMB section connection. In addition, this impact point should give the greatest possibility of vehicle snag on the barrier. The impact point for Test 2 was 6 ft upstream from the midpoint of the gate. This point of impact should cause maximum beam deflections and maximum forces in the beam. The tests are summarized in Table 1.

Test 1

In the first test, an 1,800-lb (815-kg) Honda Civic 1200 (1977) impacted the EOS 6 ft upstream from the downstream end of the steel gate system at 55.2 mph (88.8 km/hr) and 15 degrees. Figure 3 shows sequential photographs of this test. The test vehicle was smoothly redirected. The vehicle exit angle and speed were 5.5 degrees and 48.0 mph (77.3 km/hr), respectively. The occupant impact velocities were 14.15 ft/sec (4.31 m/sec) longitudinal and 16.42 ft/sec (5.00 m/sec) lateral. The peak 50-msec average acceleration was 4.27 $\underline{g}$ longitudinal (Figure 4) and 7.52 $\underline{g}$ lateral (Figure 5). All the occupant risk values as well as the vehicle trajectory hazard are below recommended values (5) for this type of test.

The test vehicle and installation after the test are shown in Figures 6 and 7. Damage to the vehicle occurred when the W-beam corrugation dragged the front bumper down and the left front tire snagged on one corner of the downstream CMB section. The vehicle damage consisted of sheet metal damage to the left front fender, a flattened left front tire, and a left front tire rim that was bent from the impact with the CMB. The EOS was damaged by having the paint scraped off the W-beam at the impact point and some surface cracking in the downstream end of the CMB. The only repair to the gate was repainting the W-beam at the impact point. The EOS was still operational after this test, which was considered a success based on the barrier safety performance and the relatively light damage incurred by the system.

Test 2

In Test 2 the strength of the gate system was examined. A 4,500-lb (2040-kg) Plymouth Grand Fury (1977) impacted the EOS 6 ft upstream from the midpoint of the steel gate at 60.7 mph (97.7 km/hr) and 25.25 degrees. Figure 8 shows sequential photographs of this test. The test vehicle was smoothly redirected. The occupant impact velocities were 18.89 ft/sec (5.76 m/sec) longitudinal and 22.77 ft/sec (6.94 m/sec) lateral. The vehicle exit angle was 4 degrees and the vehicle exit velocity was 47.96 mph (77.2 km/hr). The peak 50-msec average acceleration was 5.77 $\underline{g}$ longitudinal and 9.32 $\underline{g}$ lateral. The

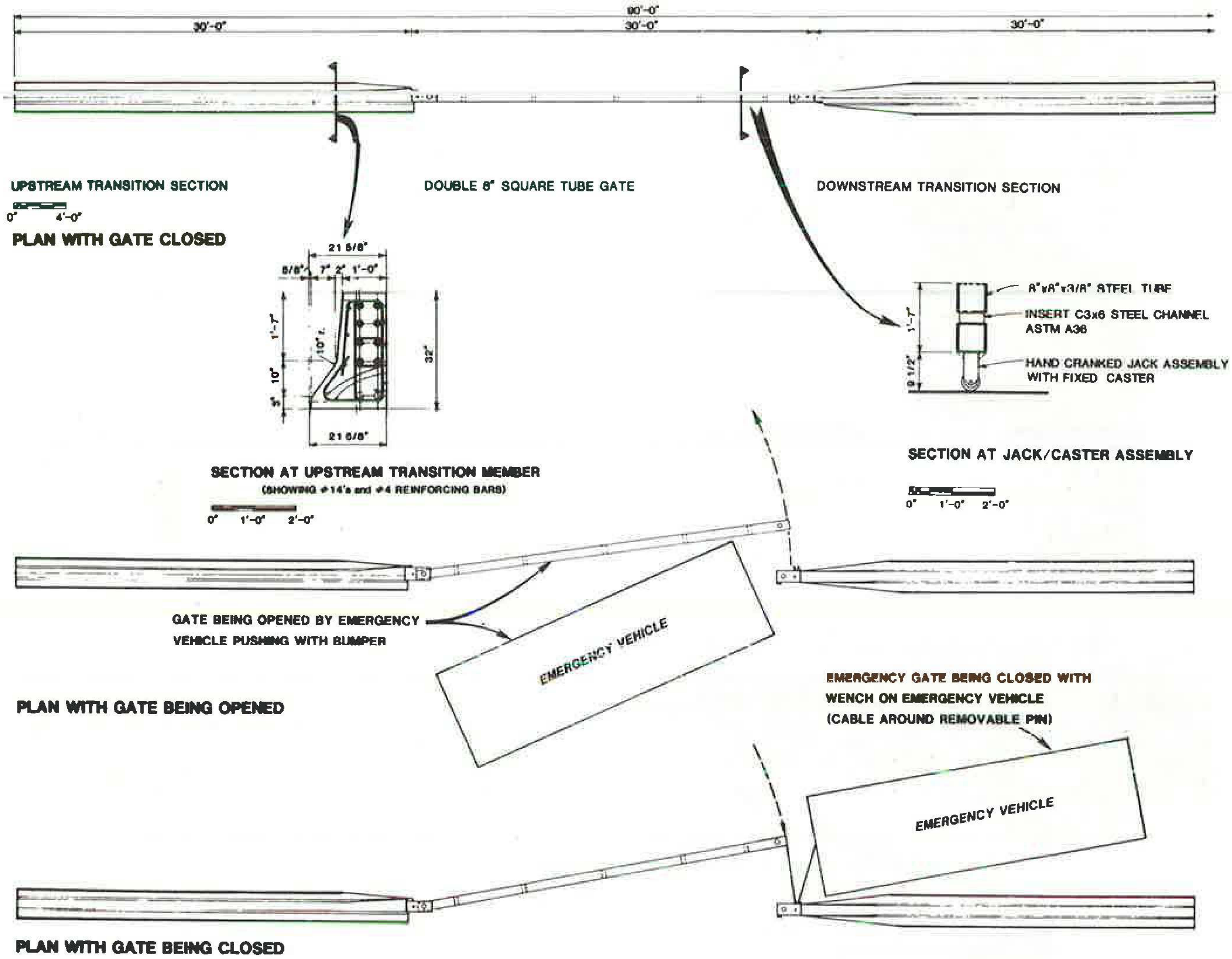


FIGURE 2 EOS in operation.

TABLE 1 Summary of Crash Tests

	Test 1	Test 2	Test 3
Vehicle weight (lb)	1,800	4,500	4,500
Impact speed (mph)	55.2	60.7	60.04
Impact angle (degrees)	15.0	25.25	25.5
Exit speed (mph)	48.0	47.96	39.01
Exit angle (degrees)	5.5	4.0	1.75
Maximum beam deflection (in.)			
Dynamic	3.36	17.16	30.84
Permanent	0.0	1.63	23.88
Maximum CMB movement (in.)			
Dynamic	2.04	15.12	31.68
Permanent	0.0	3.75	24.00
Maximum CMB roll (degrees)	0.0	3.5	9.0
Maximum CMB yaw (degrees)	0.0	0.0	5.5
Occupant impact velocity (ft/sec)			
Longitudinal	14.15	18.89	25.62
Lateral	16.42	22.77	20.54
Vehicle accelerations (*g*)			
Occupant ridedown			
Longitudinal	1.49	8.21	4.11
Lateral	10.83	7.78	6.99
Peak 50-msec avg			
Longitudinal	4.27	5.77	8.59
Lateral	7.52	9.32	8.32
Vehicle damage classification			
Traffic Accident Data	10LFQ4	11LFQ5	11FL6
Vehicle Damage Index	10LFEW3	11LDEW4	11FDAW6

Note: 1 lb = 0.45 kg; 1 mph = 1.61 km/hr; 1 in. = 2.5 cm; 1 ft/sec = 0.3 m/sec.

vehicle accelerations were within acceptable limits (5) for this type of test. The longitudinal occupant impact velocity was also within acceptable limits, but the lateral occupant impact velocity exceeded the recommended value, although it was less than the limiting value. In addition, this type of test was not required to meet the NCHRP (5) criteria.

The damage incurred by the test vehicle and installation is shown in Figures 9 and 10. The vehicle sustained sheet metal damage to the left front fender. The EOS damage included the W-beam on the vehicle impact side of the gate, which had to be replaced, and noticeable flexural cracking in the CMB sections. The permanent beam deflection was 1.63 in. (4.1 cm). The gate could still be opened after this test, which was considered very successful based on the safety performance of the system.

Test 3

In Test 3 the strength of the beam-to-CMB connection was examined. A 4,500-lb Plymouth Grand Fury (1977) impacted the EOS 6 ft upstream from the downstream end of the steel gate system at 60.04 mph (96.6 km/hr) and 25.5 degrees. Figure 11 contains sequential photographs of this test. The test vehicle was smoothly redirected. The vehicle exit angle was 1.75

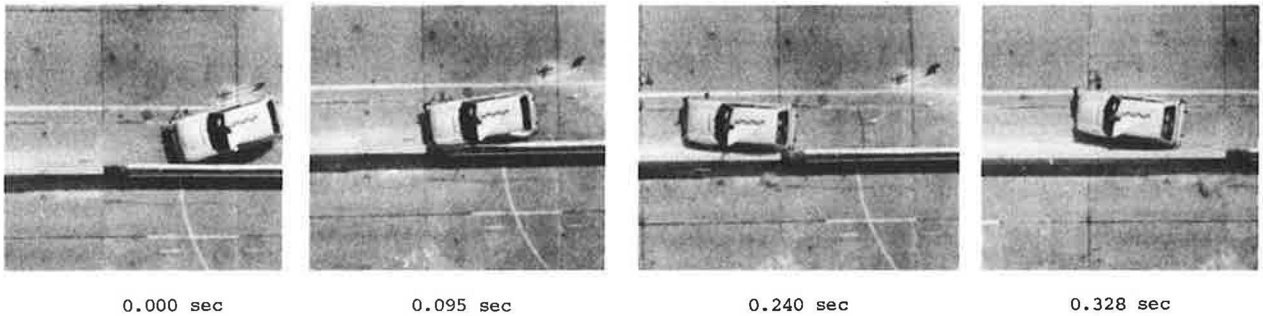


FIGURE 3 Sequential photographs, Test 1.

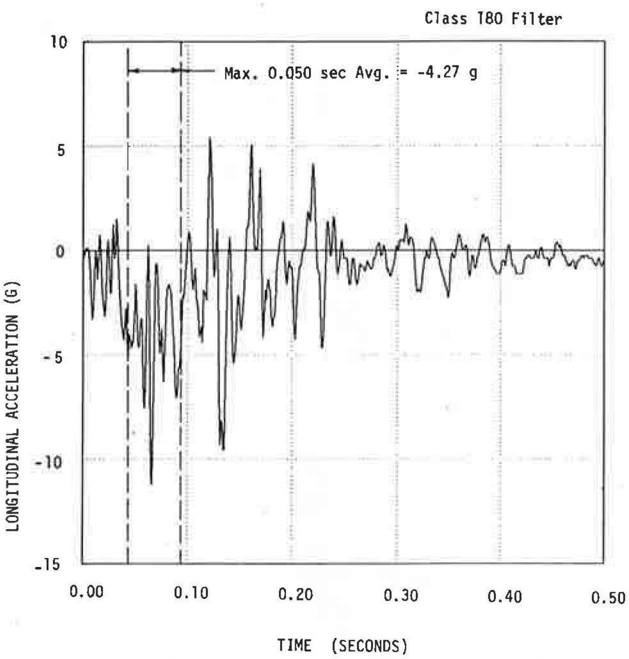


FIGURE 4 Vehicle longitudinal acceleration trace, Test 1.

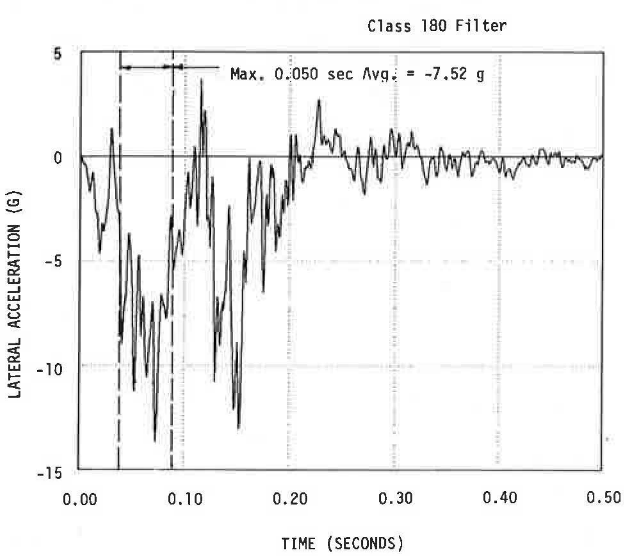


FIGURE 5 Vehicle lateral acceleration trace, Test 1.

FIGURE 6 Test vehicle after Test 1.

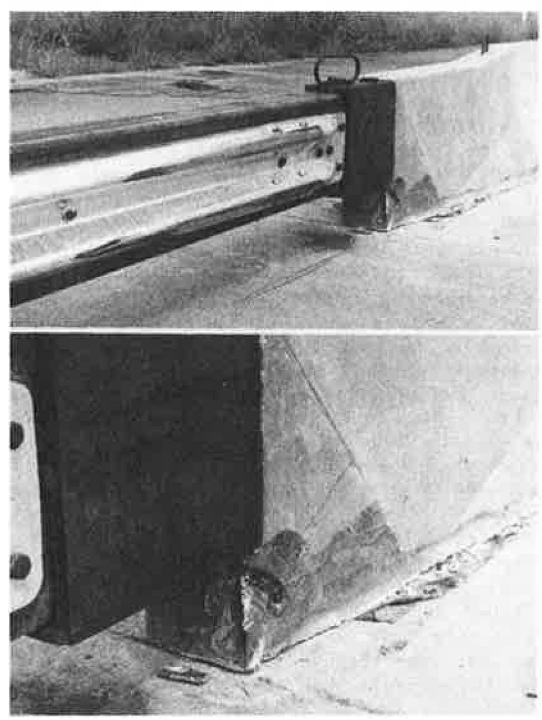
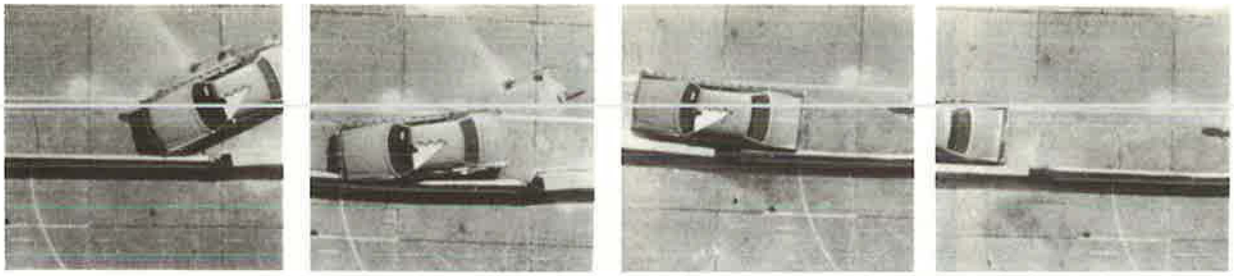


FIGURE 7 Test installation after Test 1.



0.000 sec 0.163 sec 0.409 sec 0.579 sec

FIGURE 8 Sequential photographs, Test 2.

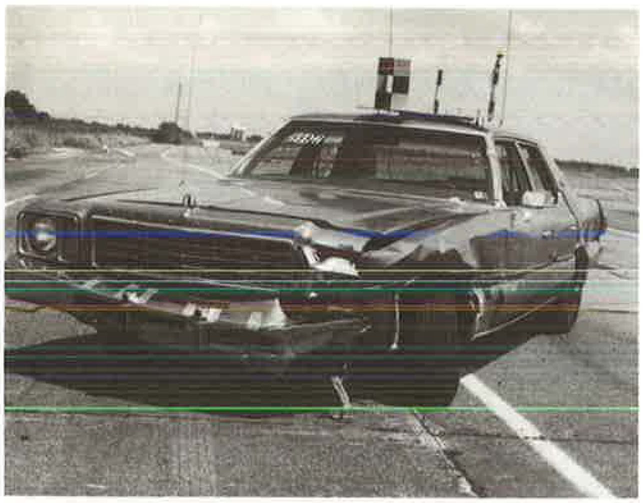


FIGURE 9 Test vehicle after Test 2.

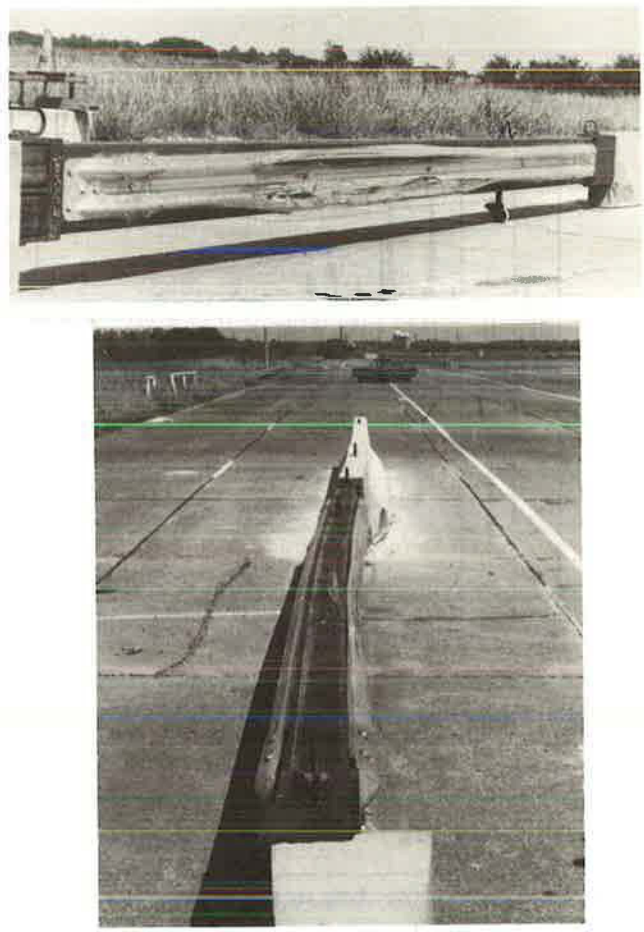


FIGURE 10 Test installation after Test 2.

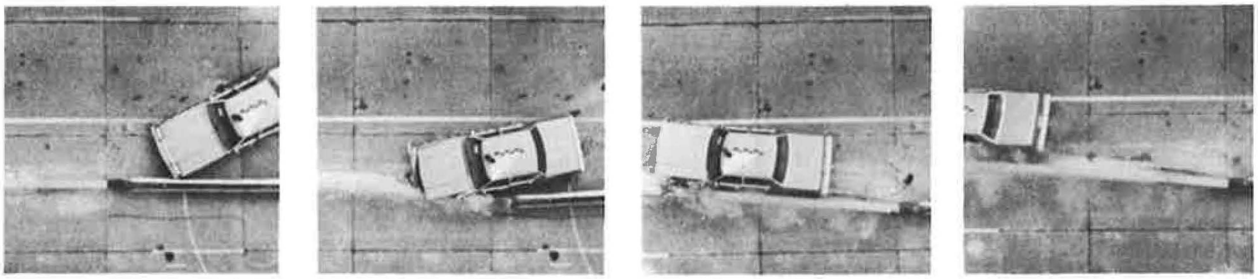
degrees and the vehicle exit speed was 39.01 mph (62.8 km/hr). The occupant impact velocities were 25.62 ft/sec (7.81 m/sec) longitudinal and 20.54 ft/sec (6.26 m/sec) lateral. The peak 50-msec average acceleration was 8.59 g longitudinal and 8.32 g lateral. The vehicle accelerations were within acceptable limits (5) for this type of test. The lateral occupant impact velocity was also within recommended limits, but the longitudinal occupant impact velocity exceeded the recommended value, although it was less than the limiting value. In additional, this type of test was not required to meet the NCHRP criteria (5).

Damage incurred by the vehicle and test installation for Test 3 is shown in Figures 12 and 13. The test vehicle was severely damaged in this test when it snagged on the downstream CMB section. The permanent deflection of the gate was 23.88 in. (60.66 cm). The gate section of the EOS sustained damage to the W-beam on the impact side of the tubes, which had to be replaced. The downstream CMB section was severely damaged because of flexural cracking and failure of one of the anchor rods in the concrete. The upstream CMB section was also severely damaged because of flexural cracking. In addition, the gate could not be opened because the metal tubes were binding about the pin connections. However, this test was still considered a success because of the barrier's safety performance and because the vehicle did not penetrate the barrier. The damage to the barrier would be minimized if proper anchorage were achieved, and although the forces on the vehicle would increase, they should not exceed those of a CMB.

SUMMARY AND CONCLUSIONS

An EOS for an AVL was developed and crash tested. The system, as shown in Figures 1 and 2, consisted of two steel box tubes mounted on top of each other. The steel beams were supported by pin connections to modified CMB sections. Factors considered in the development of the EOS were ease of operation and ability to redirect errant vehicles.

Three full-scale crash tests were conducted to evaluate the impact behavior of the design. In the first test, an impact severity test, a small vehicle was smoothly redirected. In Test 2, a beam-strength



0.000 sec 0.191 sec 0.476 sec 0.690 sec

FIGURE 11 Sequential photographs, Test 3.

test, a large vehicle was smoothly redirected. The connection strength was tested in the third test, in which a large vehicle was redirected. All the vehicle accelerations were below recommended values for all the crash tests. In addition, all the occupant impact velocities were within acceptable limits. Even though the lateral occupant impact velocities for Tests 2 and 3 exceeded the recommended value, they fell below the limiting value. Furthermore, this type of test was not required to meet NCHRP Report 230 criteria (5). In addition, the EOS was still operational after the first two tests. The anchorage system for the downstream CMB failed in Test 3, which caused the hinge mechanism on the gate to bind. With adequate anchorage for the CMB support sections, the as-tested design would remain operational after three successive severe hits.

The full-scale crash tests showed that the system tested can be used by an emergency vehicle to gain immediate access to an AVL. In addition, the tests showed the barrier's safety performance characteristics. Finally, with proper measures to protect oncoming traffic, the EOS could be adapted for use on any highway system that is separated by CMBs.

Several modifications in the EOS were recommended, on the basis of observations during the test program, to improve the operation and performance of the system. These modifications are enumerated in TTI Report 105-1F (4), and the intent of the major changes is summarized as follows:

1. To further reduce maintenance, the W-beams, end shoes, and the side straps have been eliminated (which causes the EOS gate repair cost per crash to drop essentially to zero);
2. To improve postimpact operation of the system, an improved anchorage system is being implemented for the CMB (which will eliminate binding of the gate after a crash and also reduce damage to the CMB);

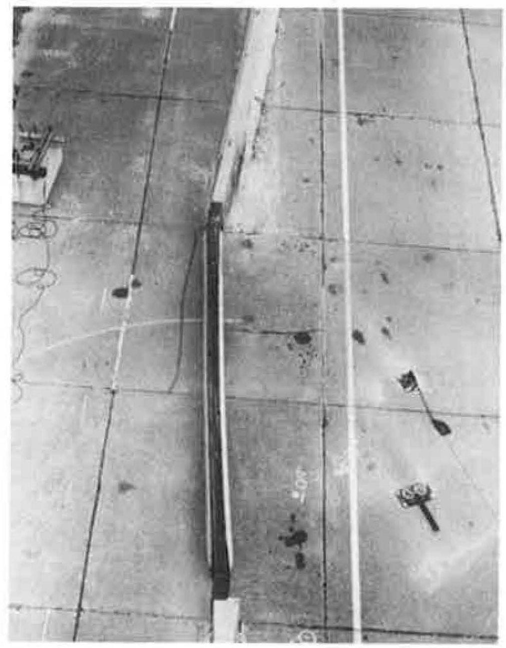


FIGURE 12 Test vehicle after Test 3.

FIGURE 13 Test installation after Test 3.

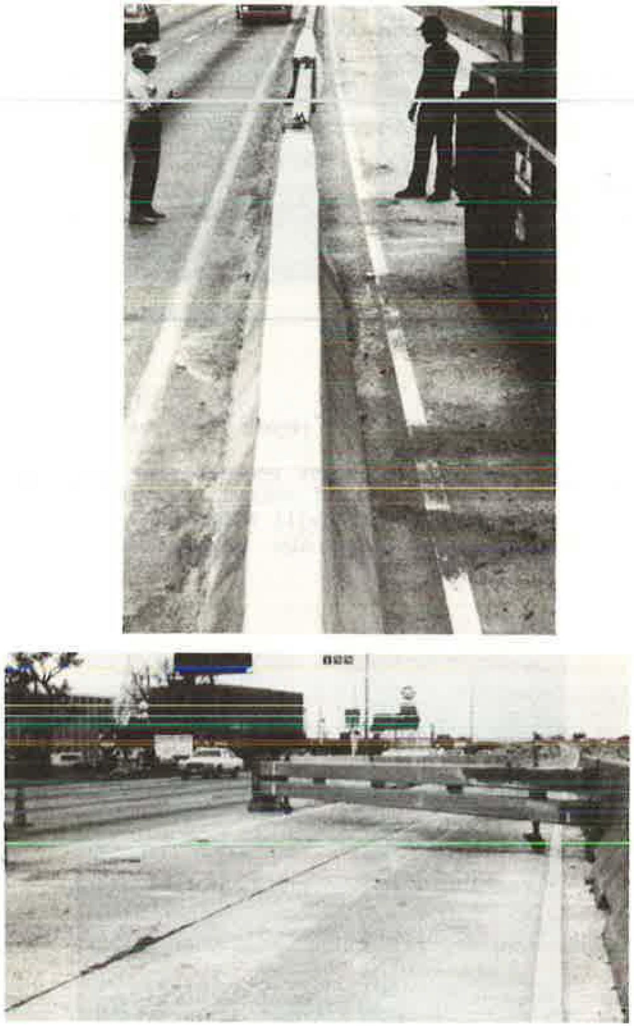


FIGURE 14 EOS as implemented.

3. To reduce the snagging potential of the EOS, a smoother transition section has been designed; and
4. To allow the gate to be opened with greater ease and from either end, the caster system has been rearranged.

At the writing of this paper the concepts and modified designs presented here are being implemented on the I-45 AVL project in Houston, Texas (Figure 14).

ACKNOWLEDGMENTS

This research study was conducted by TTI for the Texas State Department of Highways and Public Transportation (SDHPT). Technical liaison and guidance was provided by William V. Ward and James G. Darden III of the Houston Urban Project of SDHPT. The crash tests were carried out by personnel of the Highway Safety Research Center of TTI. This paper is based on a Master of Engineering thesis by John W. Strybos.

REFERENCES

1. G.H. Powell. Barrier VII: A Computer Program for Evaluation of Automobile Barrier Systems. FHWA, U.S. Department of Transportation, April 1973.
2. Building Code Requirements for Reinforced Concrete. ACI 318-77. American Concrete Institute, Detroit, Mich., 1977.
3. Manual of Steel Construction. American Institute of Steel Construction, Chicago, Ill., May 1981.
4. J.W. Strybos, J.R. Morgan, and H.E. Ross, Jr. Emergency Opening System for an Authorized Vehicle Lane. Research Report 105-1F. Texas Transportation Institute, Texas A&M University System, College Station, March 1984.
5. J.D. Michie. Recommended Procedure for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., March 1981.

New Jersey Breakaway Sign Testing

W. M. SZALAJ and R. L. HOLLINGER

ABSTRACT

Simulated and actual crash tests were conducted on a New Jersey breakaway sign structure. The tests were aimed at isolating and modifying those aspects of the system that were causing excessive damage to components as a result of vehicular impact and thus made it necessary to return the sign structure to the shop for repairs rather than reerect it in the field with a few parts changed. Before beneficial modifications were incorporated into the standard specifications, full-scale instrumented vehicular crash tests were also conducted, which confirmed that the modified system functioned well and demonstrated compliance with the latest safety standards as specified in NCHRP Report 230.

The New Jersey breakaway sign support system, used on large ground-mounted signs, was developed around 1968 in an effort to reduce damage to vehicles and injury to their occupants. The breakaway concept is based on two components: the breakaway couplings and the load-concentrating (LC) washers (Figures 1 and 2). The combination of the necked-down section of the couplings and the eccentric loading applied by the LC washers provides the sign structure with the ability to withstand wind loading and at the same time to easily break away under vehicle impact. The concept is based on the application of the wind load to the post in a horizontal direction, which results in a bending moment at the base of the support. A counteracting rotational moment, which cancels, or substantially minimizes, the wind-induced bending moment, is developed by the LC washer's eccentricity. However, when a vehicle impacts one of the sign's support posts (18 in. above the ground), the LC washers are not effective in cancelling the vehicle-induced bending because of the reduced moment arm (about one-tenth the wind-induced moment). As a result, the post and its base are moved in the direction of impact, which causes the couplings to bend and break at the necked-down section. The post then moves from its foundation and rotates about the unimpacted post out of the way of the errant vehicle. The post is restrained to the sign panel by a metal cable (with a shock-absorbing device) that prevents the post from flying completely free after impact. The restraint causes the post to rotate horizontally as well as vertically about the unimpacted post (Figure 3).

Vehicle crash tests conducted in 1970 (1) demonstrated that the system functioned with vehicle change in momentum well under the FHWA desirable safety criteria limit of 750 lb-sec. After several years of actual roadside experience, however, it was determined that the system was not performing as desired, although no deaths or serious injuries occurred. In each accident investigated, there was some type of mechanical malfunction, and as a result, the sign had to be returned to the shop for repairs rather than be reerected in the field with minor repairs.

A committee was formed and charged with the responsibility to review the field experience with the breakaway signs. The committee considered several possible deficiencies within the design, including the shock absorber, as causes of the poor field

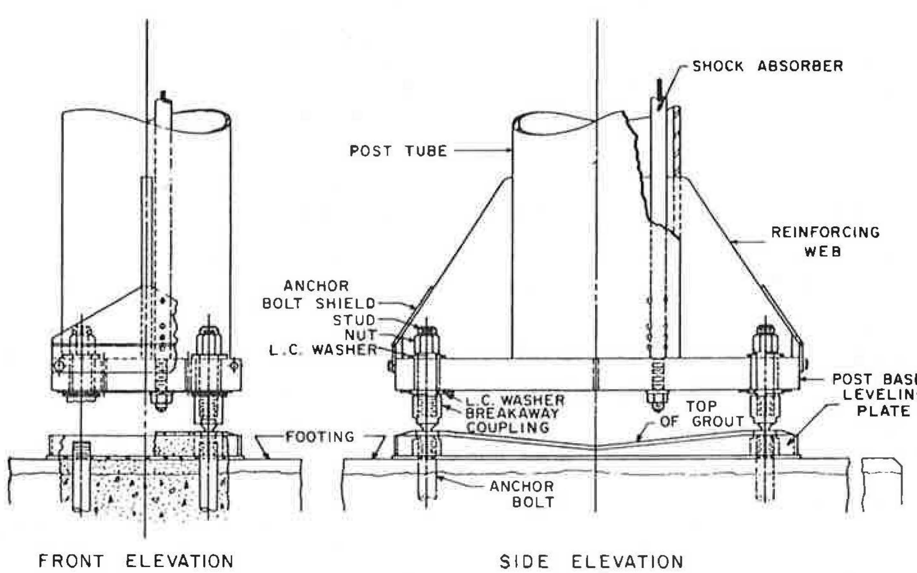


FIGURE 1 Breakaway base detail.

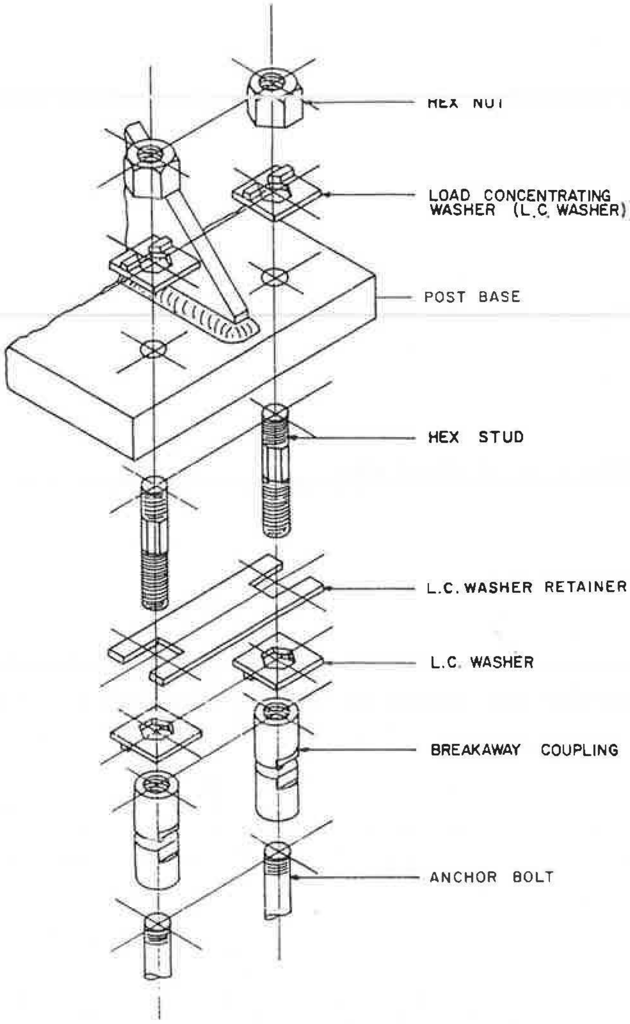


FIGURE 2 Breakaway coupling assembly.

performance. The committee modified the shock absorber design, as shown in Figure 4, and included it in the Standard Details (2) as of November 1974. The committee also suggested several other minor modifications to reduce hardware damage and recommended the testing program to isolate additional problems, verify the functioning of the modified design, and demonstrate conformance with safety standards (3-5).

STUDY PROCEDURES

The testing program was planned to proceed in three phases. Phase 1 was intended to identify and modify those items that prevented proper functioning of the system. Phase 2 was to confirm that the system, as modified, complies with nationally accepted safety standards. Phase 3 was to observe the modified sign structure under real accident conditions. (This phase was later dropped because in the 10-year experience with breakaways, no single structure has been struck more than once. When Phase 3 was proposed, an assumption had been made that certain structures, particularly those located in gore areas, would be impacted on a frequent basis. That assumption, however, was shown to be wrong.)

For Phase 1, a breakaway sign structure consisting of a sign panel 6 ft high by 12 ft wide and two 8-in. diameter support posts was erected. A truck equipped with a wire rope cable was used to pull one of the sign posts to simulate a vehicular impact. The impact transfer device (Figure 5), a wire rope sling, was wrapped around the post's base plate and pretensioned to stay in position. Once the couplings had broken and the post began to rotate forward, the cable sling fell to the ground and the post continued to rotate as under actual impact. High-speed cameras were used to photograph the sign structure operation during the event so that those aspects that prevented proper functioning of the system could be identified and modified.

Phase 2 was planned to be conducted by an independent testing agency utilizing more sophisticated techniques to certify compliance with national standards.

RESULTS AND DISCUSSION

Phase 1 consisted of five tests. In the first test, conducted with the test sign conforming to the existing plans so that data could be collected to identify the problem, several potential problems were spotted. One was the slipping of the channel frame on the impacted post, which is attached to the sign panel by clips (Figure 6). A second problem was the jamming of the post top pin (Figures 6 and 7), which must drop from its position under impact. To prevent sign panel slippage, the number of sign clips used was doubled (Figure 6) for the later tests. The jamming of the pin was a major concern because it could explain many other problems associated with the malfunction of the structure, such as loosened or broken sign panel clips, bent connecting plate, broken connecting-plate U-bolts, and miscellaneous weld failures. A suggestion to change the pin shape from cylindrical to conical was investigated and selected for further tests (Figure 7). The final simulated tests demonstrated that the conical post top pin released effectively without damage to the connecting hardware. Based on the test results, it was concluded that the system functioned acceptably as modified with the increased sign clip arrangement and the conical post top pin.

Phase 2 was begun by utilizing actual vehicles to impact a sign structure. Momentum change was determined from data collected from high-speed film and accelerometers. The effort was contracted to the Federal Aviation Administration (FAA) Experimental Center in Pomona, New Jersey. At the time, Transportation Research Circular (TRC) 191 (3) was the document listing the procedures for vehicle crash testing of highway appurtenances. This document required use of 2,250-lb vehicles and both high-speed (60-mph) and low-speed (20-mph) impacts.

Results of the tests indicated momentum changes in excess of the requirements of TRC 191. An investigation into why the momentum change was much greater than that documented when the system was originally tested in 1970 led to the discovery that the breakaway couplings did not meet the specification for hardness. When it was attempted to produce couplings that complied with the specifications, it was discovered that heat-treating to increase the hardness resulted in tensile strength above the maximum allowable in the specification. In the course of solving the hardness-tensile problem, a characteristic that greatly improved the breakaway function of the couplings was discovered--toughness, which, it was determined, should be quite low for good operation.

Investigation of available steels led to the discovery that steels processed with an elevated-temperature-draw (e.t.d.) process have the desired tensile strength to assure that the system can with-

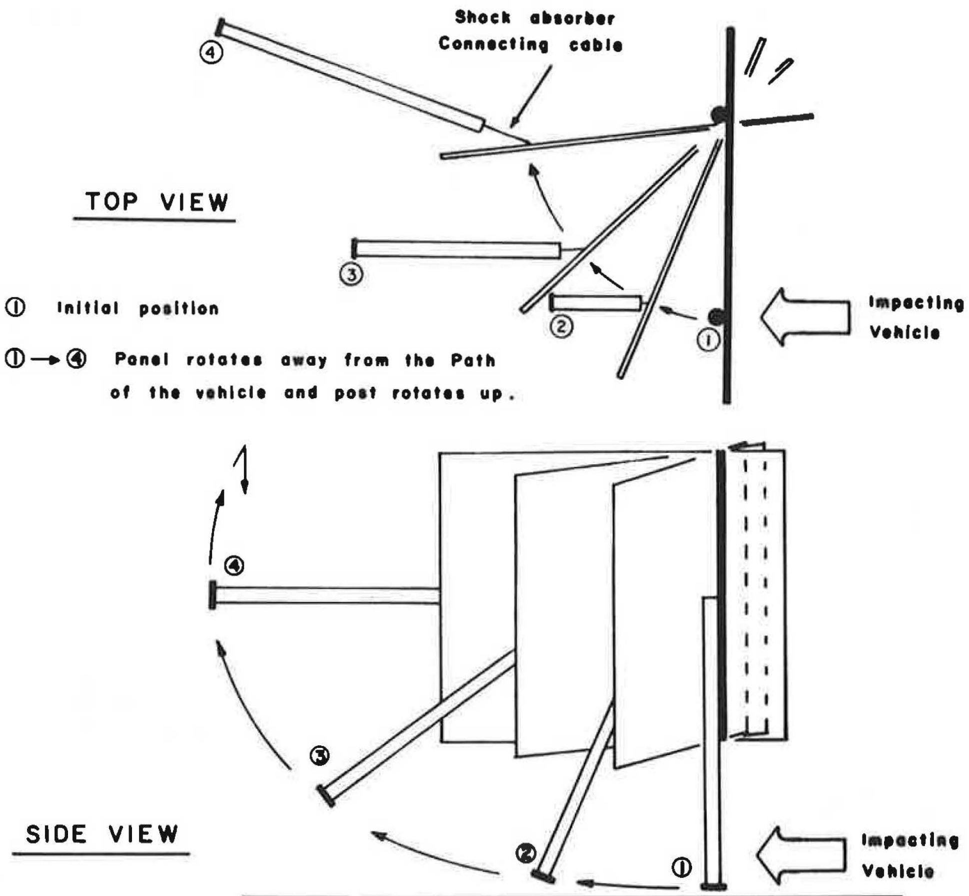


FIGURE 3 New Jersey breakaway sign support system: typical action during impact.

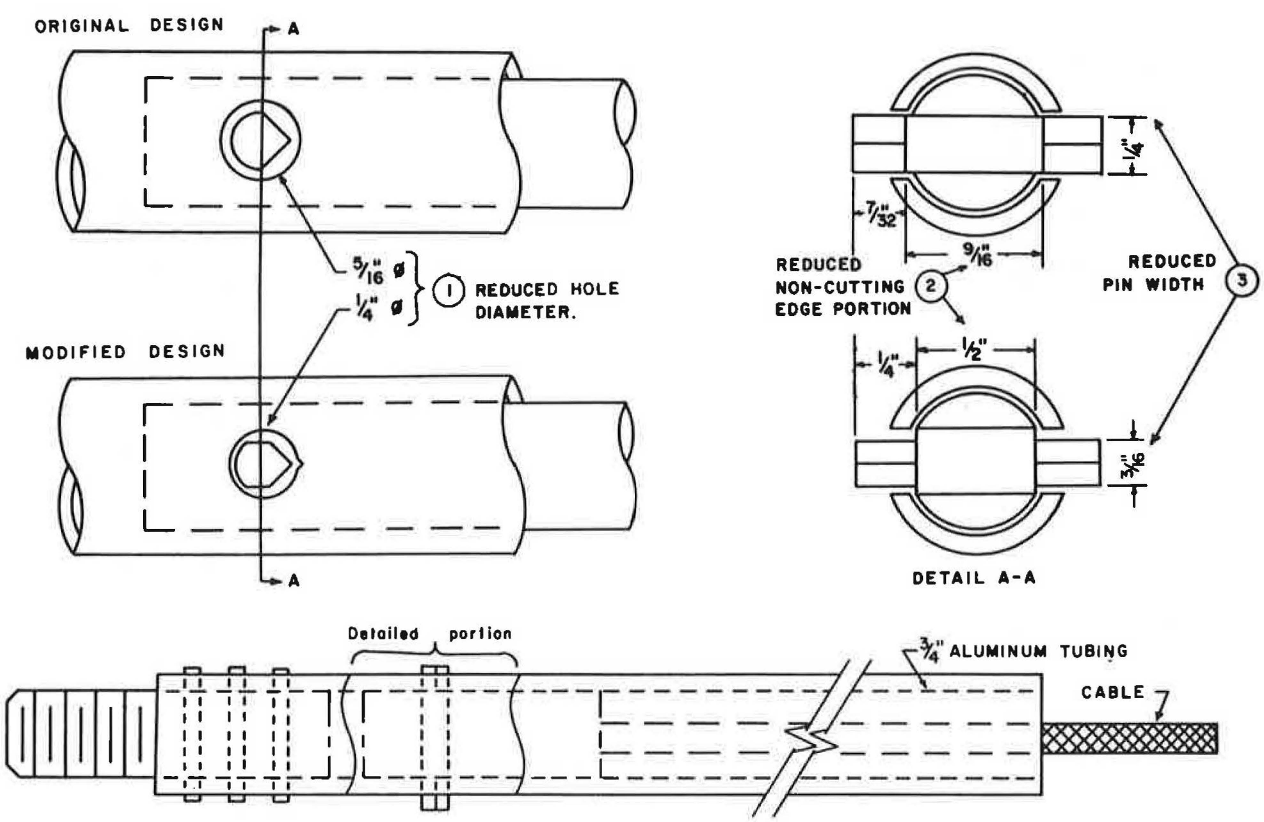


FIGURE 4 Shock absorber.

stand design wind loads and low toughness to ensure low-energy fracture on impact. A sample of a steel referred to as e.t.d. 4150-X, detailed in Table 1, was obtained and breakaway couplings were machined for testing. Laboratory tests conducted on these couplings indicated a high probability of desirable operation under vehicular impact. It should be noted that the critical section design of the couplings results in a neck-strengthening effect, which increases the coupling tensile strength by about 20 percent. Hence, the resulting coupling ultimate tensile strength will be in the range of 195,000 to 225,000 psi.

A pilot test was conducted by using a Chevrolet Chevette that was pushed into a test sign mounted on couplings made from the e.t.d. 4150-X steel. Data collected from film and vehicle damage showed insignificant damage to vehicle and structure and resulted in a momentum change well under the desirable safety limit of 750 lb-sec, and Phase 2 testing was thus resumed.

Unfortunately, during the time that a complying

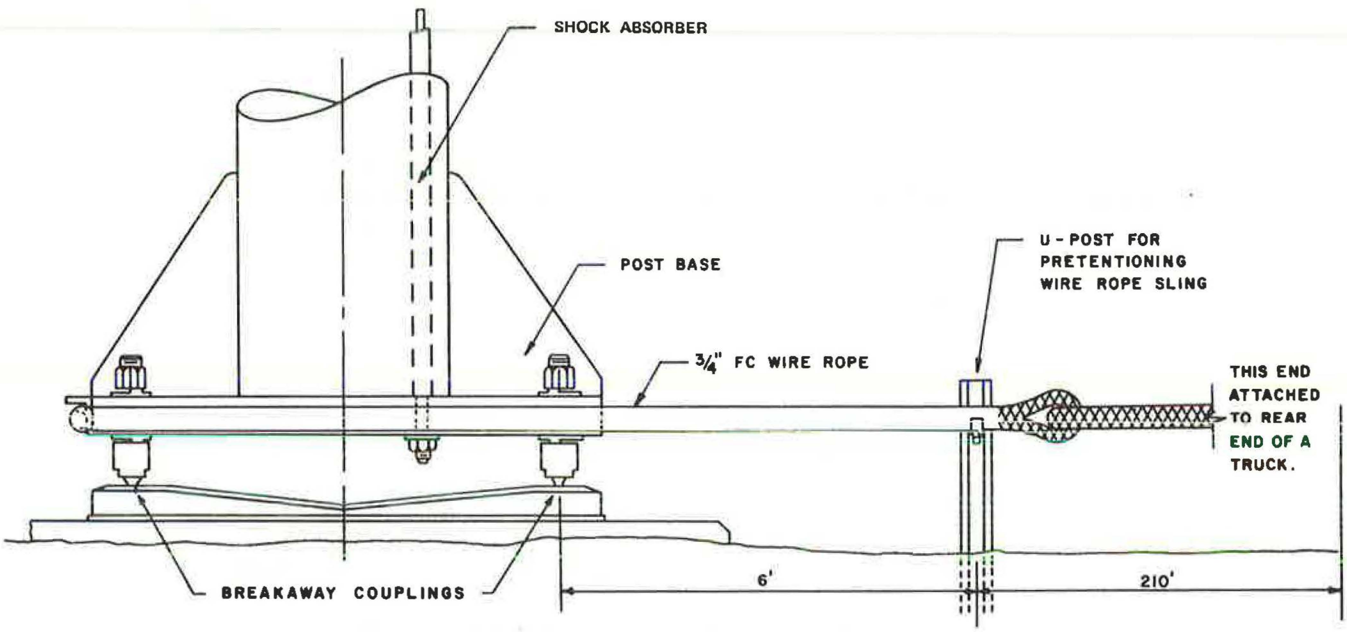


FIGURE 5 Impact transfer device used for simulating impacts.

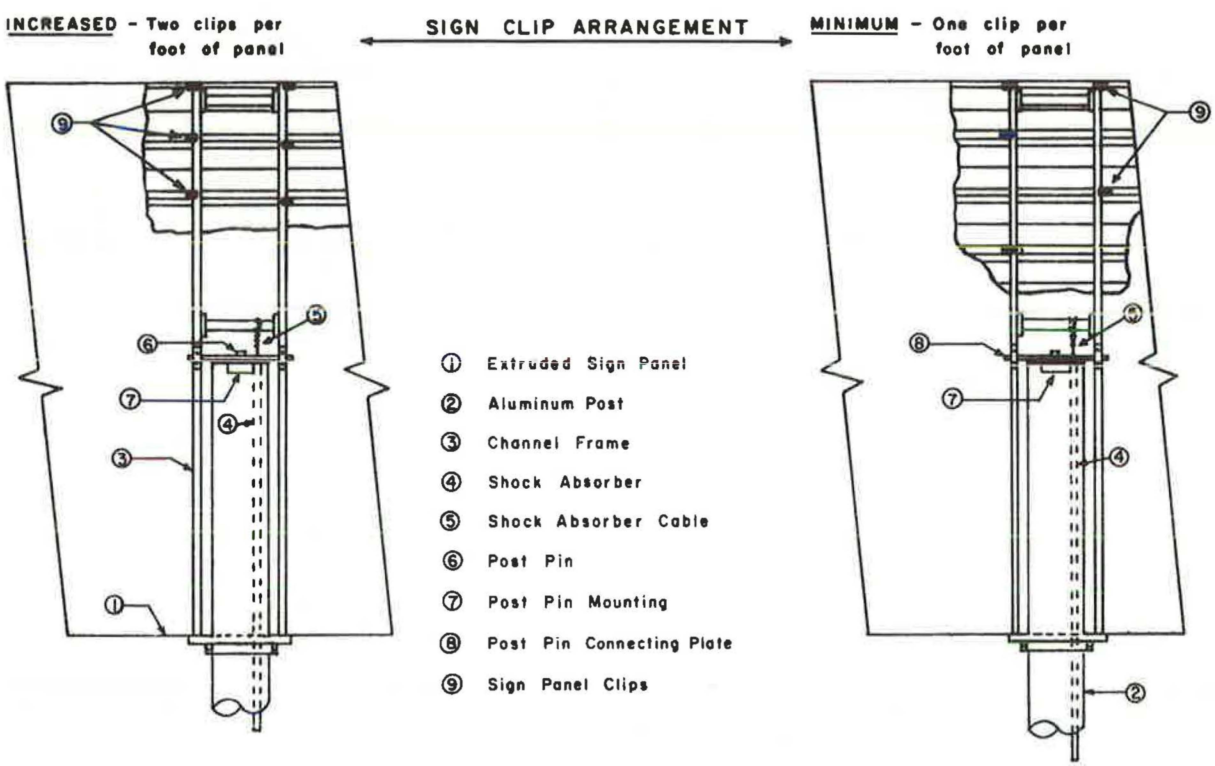


FIGURE 6 Sign panel attachment detail.

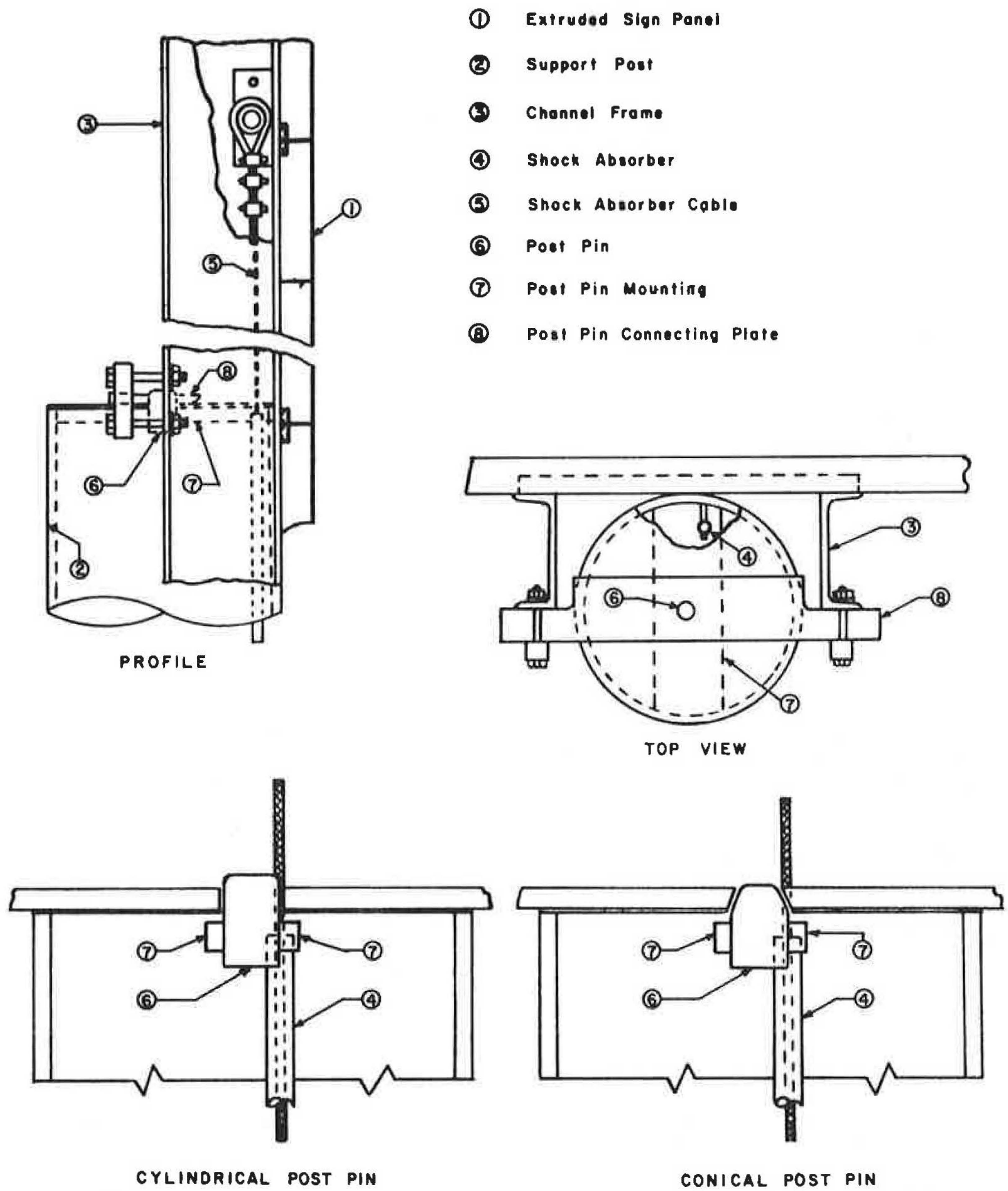


FIGURE 7 Post top connection detail.

TABLE 1 Mechanical and Chemical Properties for e.t.d. 4150-X Steel

Item	Amount
Chemical composition (%)	
Carbon	0.48 minimum
Manganese	0.75/1.00
Phosphorus	0.035 maximum
Sulfur	0.040 maximum[a]
Silicon	0.15/0.35
Chromium	0.80/1.10
Molybdenum	0.15/0.25
Tellurium or selenium	0.01 or 0.035
Mechanical property	
Tensile strength (psi)	165,000–185,000
Yield strength (psi)	155,000 (minimum)
Elongation (%)	9 mean (13 maximum)
Reduction of area (%)	34 mean (40 maximum)
Machinability (%)	56 of C-1212
Toughness (ft-lb)	10 (maximum at 70 degrees)

Note: e.t.d. 4150-X is a product of the LaSalle Steel Company, Hammond, Indiana.

[a]When tellurium is added, sulfur may be 0.04/0.06 percent.

steel was being investigated, the FAA facility was reorganized and testing could not be continued there. Southwest Research Institute (SWRI) was selected to conduct the full-scale testing, now under the guidelines of NCHRP Report 230. The revised testing procedures now required use of 1,800-lb vehicles instead of 2,250-lb ones. There was some concern about the use of the lighter vehicles because the pilot test had used a 2,250-lb Chevette. The concern proved to be unwarranted when an additional pilot test conducted with a Volkswagen Rabbit weighing 1,800 lb also resulted in a vehicle change of momentum well within the standards.

Three full-scale vehicle crash tests were conducted on a sign structure with a 14 x 18-ft panel mounted on two 12-in.-diameter support posts. The three crash tests were conducted with late-model Honda Civic sedans in the 1,800-lb weight class. Test conditions corresponded to Tests 62 and 63 of NCHRP Report 230 and an additional test similar to Test 63 but at a 25-degree angle. The three tests

TABLE 2 SWRI Test Conditions and Results

	Test		
	NJ-1	NJ-2	NJ-3
Test vehicle year[a]	1977	1978	1978
Vehicle weight (lb)	1,771	1,812	1,743
Impact speed (film) (mph)	20.8	59.9	61.4
Impact location	Left support	Right support	Right support
Impact angle[b] (degrees)	0	0	25
Offset distance[c] (in.)	0	15	22
Impact duration (sec)	0.24	0.09	0.115
Exit speed (mph)			
Film	15.5	54.5	54.6
Accelerometer	15.9	53.9	54.1
Change in momentum (lb-sec)			
Film	429	445	541
Accelerometer	402	508	571
Maximum 50-msec avg acceleration (*g*) (accelerometer)			
Longitudinal	-3.5	-6.4	-5.6
Lateral	-0.2	2.1	1.0
Occupant risk[d] (ΔV)			
Longitudinal (ft/sec) (15)	7.8	9.9	11.4
Lateral (ft/sec) (15)	0.5	-1.9	-1.0
a_{long} (15)	n/a	n/a	n/a
a_{lat} (15)	n/a	n/a	n/a

Note: n/a = occupant did not travel specified distance.
[a]All test vehicles were Honda Civics.
[b]Angle from axis perpendicular to sign panel plane.
[c]Distance from vehicle to pole centerline, positive to left.
[d]Numbers in parentheses are recommended values for NCHRP Report 230 *(4)*.

conducted demonstrated full conformance with the safety requirements of TRC 191 and NCHRP Report 230. The test conditions and results are summarized in Table 2.

There was some concern that the conical post top pin design might allow high wind loads to cause the sign panel to ride up and off the pin. A review of the potential problem indicated that this is very unlikely to happen except under some very unusual combinations of terrain and wind speed and direction. The use of a taut shock absorber cable connection, as currently required, should prevent such an occurrence and no problem is expected.

IMPLEMENTATION OF FINDINGS

The full-scale validation tests conducted at SWRI (6) confirmed that the modified breakaway sign system functions well within safety standards and with minimal hardware damage. It is hence recommended that the conical post top pin design and the special low-toughness material (e.t.d. 4150-X) be incorporated into the New Jersey breakaway sign standard drawings and specifications. The increased sign clip arrangement, which was also found to be a desirable modification, is already included in the standard specifications.

Because the modified New Jersey breakaway sign system has not been used except in testing, monitoring of field installations to ensure proper functioning in high winds is desirable.

SUMMARY AND CONCLUSIONS

The New Jersey breakaway sign support system was designed to break away on impact to reduce vehicle damage and prevent occupant injury. Accident experience has indicated that changes could be made to improve the performance of the breakaway sign structure by reducing the amount of sign repair needed after a vehicular impact occurred.

Several important modifications were made and the system was tested under various simulated and actual impact test conditions. Based on available literature, the modified New Jersey breakaway sign support is at this time the only breakaway system to have been tested in full compliance with the latest testing procedures (NCHRP Report 230) and to demonstrate compliance with the latest safety evaluation criteria.

Because the modified system's performance was well under the current safety limits and resulted in minimal damage to the sign structure, the modified system, which includes changes made to the post top pin connection and the breakaway coupling material, is recommended for use.

ACKNOWLEDGMENTS

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REFERENCES

1. N.E. Shoemaker. Full Scale Dynamic Testing of the New Jersey Breakaway Sign Structure. Cornell

Aeronautical Laboratory, Inc., Buffalo, N.Y., Sept. 1970.
2. Standard Details. New Jersey Department of Transportation, Trenton, July 1976.
3. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. Transportation Research Circular 191, TRB, National Research Council, Washington, D.C., Feb. 1978.
4. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230, TRB, National Research Council, Washington, D.C., March 1983.
5. AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals. FHWA Notice N5040.20. FHWA, U.S. Department of Transportation, July 14, 1976.
6. J.B. Mayer, Jr. and M.E. Bronstad. Full Scale Vehicular Crash Testing of a Modified NJ Breakaway Sign Structure. Southwest Research Institute, San Antonio, Tex., Nov. 1983.

Analysis of Accidents Involving Breakaway-Cable-Terminal End Treatments

JERRY G. PIGMAN, KENNETH R. AGENT and TOM CREASEY

ABSTRACT

This paper includes an analysis of 50 accidents involving breakaway-cable-terminal (BCT) end treatments and 19 accidents involving median-breakaway-cable-terminal (MBCT) end treatments as used in Kentucky. The primary data base consisted of Kentucky accident records for the years 1980-1982; selected accidents were included that occurred before 1980 and after 1982. An attempt was made to document each accident with a police report, photographs, and a maintenance repair form. Results showed that the BCT end treatment performed properly in 60 percent of the accidents; that is, the end treatment performed as it was designed, with the wooden posts breaking away or the guardrail redirecting the vehicle. Only five impacts were known to involve small cars and the BCT performed improperly in four of those accidents. It should be noted that the BCT used in Kentucky is similar to the design tested and evaluated as part of the NCHRP studies and included in the AASHTO barrier guide. The primary difference was that before 1982, most BCTs in Kentucky were installed so that the last 125 ft of rail were placed on a simple curve (4.5 degrees) and there was a 6-ft offset rather than a parabolic flare with a 4-ft offset. However, Kentucky's MBCT design utilizes two BCTs joined together at the end section, and it varies considerably from the design tested as part of the NCHRP studies. The MBCT end treatment performed properly in 50 percent of the accidents. Problems related to stiffness of the end treatment are most apparent when impact angles are shallow. A recommendation was made to remove any existing MBCT designs from gore locations and replace them with crash cushions. A turned-down end treatment design was proposed for consideration at median installations.

The performance of guardrail end treatments has been a subject of concern to highway engineers for many years. A concerted effort was begun in the mid-1960s to evaluate guardrail design and recommend warrants for guardrail use. The work was funded through NCHRP Project 15-1 and a review of current practice was performed by Cornell Aeronautical Laboratory (1). The next study funded by NCHRP was a compilation of recommended practices for locating, designing, and maintaining guardrails and median barriers (2). Results reported from the study were based on a comprehensive literature review, a state-of-the-art survey, and the advice of a selected group of experts. It was noted that ramped end treatments were found to cause test vehicles to launch, roll, and tumble.

The next study in the series under NCHRP Project 15-1 included results of 25 full-scale crash tests and summarized the relative performance of the designs tested (3). Eight full-scale tests were performed on end terminal designs; six involved ramped designs, one was performed on a flared end treatment, and one on a blunt end terminal. With the exception of one test, the vehicles were launched, rolled, and tumbled in the ramp-terminal tests. In the flared-terminal test, the vehicle penetrated the rail and decelerated in an acceptable manner. For the blunt-terminal test, the vehicle sustained major front-end damage, was launched, and landed on top of the rail. It was concluded that all designs tested as part of the research were hazardous and development of a safer end treatment was the highest-priority item for subsequent research.

The fourth in a series of studies as a part of NCHRP Project 15-1 was a synthesis of information on warrants, service requirements, and performance criteria for all traffic-barrier systems (4). Emphasis was placed on the center or "length-of-need" section rather than on the terminal sections.

The last of five documents reporting on research that originated as NCHRP Project 15-1 dealt with guardrail end design and included results of full-scale tests on hydraulic-post guardrail design and concepts for improved end designs (5). Results included in NCHRP Report 118 were 12 new guardrail terminal and transition concepts, one of which was the breakaway cable terminal (BCT). Three full-scale crash tests were performed to evaluate the dynamic performance of the BCT. The BCT concept was shown to be an effective terminal for W-beam guardrail systems and appeared to be a significant improvement over either the turned-down or blunt-nose terminal. It was noted that for end-on impacts, the BCT performed in a manner similar to that of crash cushions. Maximum average vehicle deceleration permissible for crash cushions is 12 *g* and average deceleration values for end-on impacts into the BCT were only 2.5 and 3.4 *g*. Those tests were conducted with 4,100-lb test cars, and it was noted that higher deceleration values should be experienced for smaller test vehicles. Advantages of the flared over the nonflared terminal for end-on impacts were demonstrated in the crash tests. Stabilization of the end nose was achieved by using either steel diaphragms or vermiculite concrete to spread the beam loads over a large frontal area. As a result of the tests conducted and documented in NCHRP Report 129, the BCT was recommended for immediate installation for field evaluation.

The work of the Southwest Research Institute (SWRI) on guardrail end treatments was extended as NCHRP Project 22-2. Included were 25 full-scale crash tests to develop prototype end designs with emphasis on the BCT (6). Three tests of the BCT with subcompact cars were also performed. High rates of deceleration were measured during impacts with the small cars. Results indicated that the BCT neither eliminated nor increased the danger during small-car end-terminal collisions. Modifications to the end treatment were made to include a concrete footing and a drilled hole in the second post. Additional modifications were made to increase the size of the concrete footing, which had failed in one of the earlier tests. Overall results confirmed the recommendation for immediate trial implementation.

Development of the BCT for median barriers followed the research on BCTs for guardrails (7). Test results showed that the median barrier performed acceptably for the steel box-beam median barrier and the blocked-out W-beam median barrier with both steel and wood posts. It was also noted that installation of the BCT for guardrails was encouraged by FHWA as part of the National Experimental and Evaluation Program (Notices HNG-32, December 11, 1972, and HHO-31, May 24, 1973).

Additional research conducted as part of NCHRP Project 22-2 included component testing, analytical simulation, and full-scale crash testing to further develop earlier BCT designs (8). Several modifications were made, including the use of slip-base steel posts, a reduction in the size of wood posts from 8 x 8 in. to 6 x 8 in., and elimination of diaphragms in the nose section. It was noted that more than 12 states had installed BCTs as of March 1976.

An update on development of the BCT was reported by NCHRP in May 1978 (9). Several problems were reported, both in service and during subsequent experimental programs. Those problems included removal of the fractured wood post from the concrete footing, cost of BCT components, and snagging of a subcompact vehicle's underside by steel-post BCTs. Modifications made were such that the BCT was judged to perform satisfactorily for most vehicle impact conditions. It was noted that 30 states had adopted the guardrail BCT as a standard and that there was less widespread use of the median-barrier BCT.

By November 1980 it was reported by NCHRP that nearly 100,000 BCT end treatments had been installed in more than 40 states (10). Problems continued to occur with the removal of broken posts and with installations where the 4-ft flare was not obtained. It was emphasized that lack of the 4-ft flare could result in spearing of vehicles during head-on impacts.

Documentation of field performance of BCT and median-breakaway-cable-terminal (MBCT) end treatments has been relatively scarce since the testing by SWRI. A study by the New Jersey Department of Transportation had the objective of evaluating in-service performance of BCTs (11). A total of 13 vehicular impacts into BCTs was evaluated and results were compared with full-scale crash tests previously conducted by SWRI. The in-service experience was similar to the initial tests by SWRI, and the BCT was recommended for flared-guardrail installations. A significant problem was spearing of small cars during end-on impacts when the end had not been flared. Reinforcement of the unstiffened buffer end on straight guardrail sections was recommended. Replacement of the two 12.5-ft sections with one 25-ft section also was recommended.

The MBCT end treatment as designed and tested by SWRI has had limited use. Installations are known to have been made in New Jersey and North Carolina. New Jersey has installed approximately 40 MBCTs and there has been only one reported accident (E. Dayton, New Jersey Department of Transportation, July 1982, unpublished data). A large automobile struck the device, and it performed as designed. Only one accident has been reported involving a MBCT in North Carolina (M. Bronstad, SWRI, unpublished data). The terminal was impacted end-on by a full-size sedan and performed properly, even though it was damaged extensively.

A recently completed survey by the Transportation Research Program at the University of Kentucky revealed that the BCT was the most common end treatment used; 40 states use this treatment to some degree (12). In 24 states, only the BCT is used for terminating roadside steel-beam guardrails. Some form of the MBCT was used in 16 states.

BCT AND MBCT USE IN KENTUCKY

Kentucky was one of the first states to install BCTs; the first installations were made in 1974. Through 1983 the total number of installations made and included in the Kentucky Department of Highway's summary of unit bid prices was 3,633. The average cost for each was $515. A summary of BCT installations and costs for 1974-1983 is presented in Table 1. The BCT is the current recommended standard in Kentucky for all fills and solid rock cut sections that have an adequate recovery zone behind the guardrail. It should be noted that several BCTs without the parabolic flare have been installed in Kentucky. Before 1982 most BCTs were installed with the last 125 ft of rail placed on a simple curve (4.5 degrees) and an offset of 6 ft. In 1982 Kentucky's standard drawing for BCT installations was revised to reflect a parabolic flare over the last 37.5 ft with a 4-ft offset at the end. A recent installation of a BCT in Kentucky with a 4-ft offset and parabolic flare is shown in Figure 1. Shown in Figure 2 is a BCT installed by using the 4.5-degree simple curve with an offset of about 6 ft. Significant problems can occur if the end is not flared. Only a few accidents were found that involved a BCT without the designed offset. When the BCT end treatment is installed with the designed flare and offset, impacts with the end may result in very acceptable performance, as shown in Figure 3. This BCT was constructed by using the 4.5-degree simple curve as the method to achieve the desired offset.

The MBCT has not been installed in Kentucky as extensively as the BCT. For the period 1974 through 1983, a total of 573 was installed as a part of new construction or reconstruction projects and the average cost was $627 per installation (Table 1). Kentucky's design utilizes two BCTs joined together at the end section as shown in Figure 4. It was noted earlier that head-on impacts into unflared BCTs could result in spearing of the vehicle. Similar problems are associated with head-on impacts into Kentucky's MBCT design (Figure 5). There appears to be little uniformity nationwide in the types of designs used for MBCT end treatments. Only a few states adopted the MBCT for use as it was designed and tested by SWRI. A typical installation using that design is shown in Figure 6. It should be noted that the BCT and MBCT evaluated in this study are the types used in Kentucky. Although the BCT now used in Kentucky is very similar to the design tested, evaluated, and recommended as part of the NCHRP studies (5), the MBCT varies considerably from the MBCT design recommended as part of the NCHRP studies (7,8).

TABLE 1 Summary of BCT and MBCT Installations by Year

	BCT		MBCT	
Year	No.	Avg Unit Price ($)	No.	Avg Unit Price ($)
1974	285	668	2	700
1975	443	617	98	742
1976	421	446	63	590
1977	541	423		
1978	229	444	73	545
1979	350	482	101	574
1980	244	516	10	680
1981	160	519	14	657
1982	498	572	90	636
1983	462	487	122	631
Total	3,633	515[a]	573	627[a]

Note: Numbers and unit prices were tabulated from contracts awarded.
[a]Weighted average.

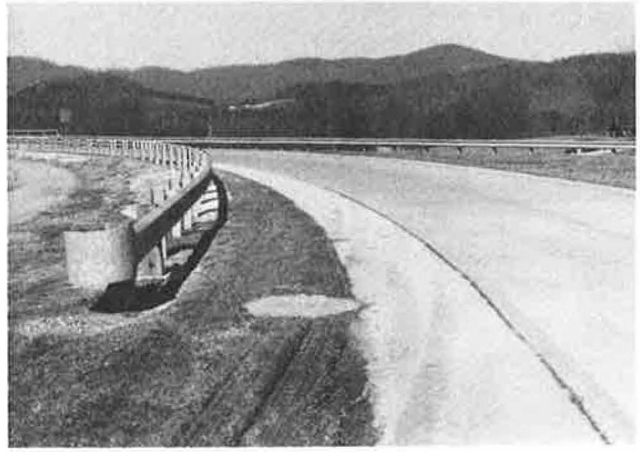

FIGURE 2 BCT installed on 4.5-degree simple curve with offset of 6 ft.

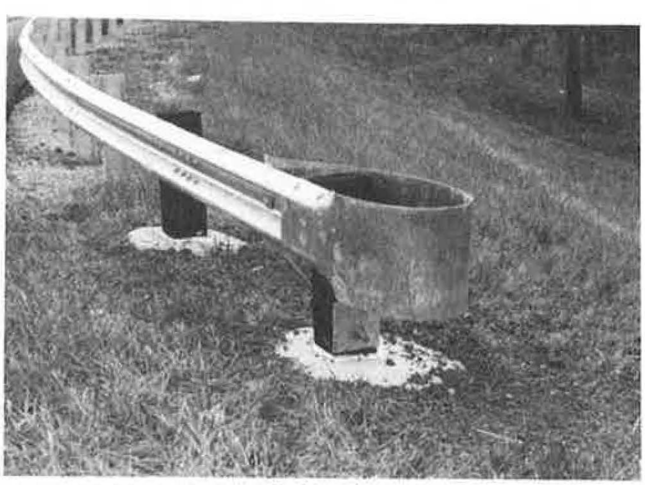

FIGURE 1 BCT end treatment (Kentucky's Type 4).

FIGURE 3 Proper performance of BCT end treatment.

FIGURE 4 MBCT end treatment (Kentucky's Type 6).

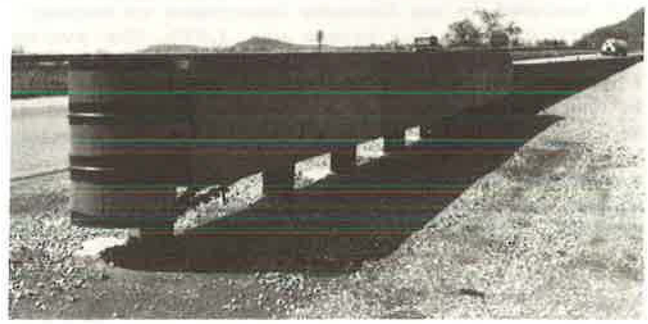

FIGURE 6 MBCT end treatment (similar to design tested by SWRI).

FIGURE 5 Spearing of vehicle by MBCT end treatment.

DATA COLLECTION

Data collection for this study involved several phases. Initially, reports of accidents involving all types of safety barriers were collected for 1980. Those barriers included crash cushions, earth mounds, concrete median barriers, and four types of guardrail end treatments: BCT, MBCT, buried (turned down), and blunt. Accident reports were made available through the Accident Surveillance Section of the Division of Traffic, Kentucky Department of Highways. It was decided to search for BCT and MBCT accidents for 1981 and 1982. An inventory of all Kentucky routes having BCT and MBCT installations was used; accident reports pertaining to those routes were reviewed and appropriately selected. Thus a 3-year data base for accidents involving BCT and MBCT end treatments was established.

The next step involved making arrangements with maintenance personnel within the Kentucky Department of Highways so that the study team could be notified when accidents occurred involving BCT or MBCT end treatments. A liaison was appointed for each highway district to supply information concerning guardrail and end-treatment installations and repairs. On-site investigations were made before the guardrail was repaired, and photographs were taken to document the performance and damage of the end treatment. In some instances, photographs of the vehicle were made available through police or other agencies.

Additional accidents involving guardrails were discovered on trips or when accident reports were being searched for other purposes. An effort was made to combine photographs with the accompanying accident reports. However, some accidents involving guardrail ends went unreported. In other cases, the guardrail was repaired before photographs could be taken.

The resulting data base consisted of all known accidents involving BCT and MBCT end treatments since the beginning of those installations. This consisted of a search of accident records for the years 1980, 1981, and 1982 and use of selected accidents before 1980 and after 1982. There was a total of 69 accidents. Time did not permit the investigation of all accidents before 1980 on routes containing BCT and MBCT installations. Correspondence with the district offices eliminated the need to search the records of all accidents occurring after 1982. Information obtained on other types of safety barriers mentioned earlier was used in another phase of the study.

The sample used in the final analysis of data contained verified accidents involving BCT and MBCT end treatments. When possible, each accident was documented with a police report, photographs, repair report, and any other pertinent information. However, not all information could be obtained for every accident.

RESULTS

Data for a total of 69 BCT or MBCT end-treatment accidents were obtained. The majority of accidents (50) involved a BCT. The earliest accident date was May 1976 and the latest was April 1984. Limited repair cost data were available. The average repair cost at eight BCT locations was approximately $700, with a range of about $430 to $920. A wide range of repair costs would be expected because of the difference in damage. The cost to repair one MBCT end treatment was about $890. The repair costs are higher than the original installation costs.

The possible sources of information concerning the accidents included accidents reports, photographs, and repair forms. An accident report was obtained for 51 of the 69 accidents, photographs were obtained for 33 accidents, and a repair form was found for 20 accidents. All three sources of information were found for only six accidents. Both an accident report and photographs were found for 18 accidents. Following is a discussion of the results from the analysis of BCT and MBCT end-treatment accidents.

BCT End-Treatment Accidents

Performance of BCT end treatments was determined for each accident. In addition to end-treatment performance, information concerning vehicle size, impact severity, impact angle, guardrail placement, initial vehicle contact area, vehicle action after impact, and end-treatment damage was analyzed. Subjective judgment was use to determine many of those variables. A detailed description of each accident was included as an appendix in the full report (13) from which this paper was prepared. Sketches were drawn to show the angle of impact when that information was known.

End-treatment performance, when it could be determined, was defined as either proper or improper. Proper performance resulted when the end treatment performed as designed, with the wooden posts breaking away or the guardrail redirecting the vehicle. Of the 50 accidents studied, the BCT end treatment was judged to have performed properly in 30 (60 percent).

Because many of the BCT end treatments were not installed with an offset of 4 ft and a parabolic flare over a distance of 37.5 ft, further analysis was made to document the configuration of the BCT as it was installed. End-treatment configuration was categorized as one of the following:

1. Simple curve: A 4.5-degree simple curve is used to extend the standard section of guardrail to the terminal section. The last 125 ft of guardrail are installed on this 4.5-degree curve to obtain an offset of 6 ft at the end.
2. Parabolic flare: The terminal section is offset 4 ft with a parabolic flare over the last 37.5 ft (type that was tested, evaluated, and recommended as part of NCHRP studies).
3. Straight: The terminal section is placed at the end of a standard section of guardrail with very little or no offset.

Results of categorizing the end-treatment configurations are as follows:

End-Treatment Configuration	No.	Percent
Simple curve	38	76
Parabolic flare	8	16
Straight	4	8
Total	50	

An analysis of the data was made to relate performance to BCT end-treatment configuration (Table 2). It was determined that in 23 of 38 accidents (61 percent) the end treatment performed properly when it was installed on a 4.5-degree simple curve. When the end treatment was installed on a parabolic curve, performance was rated proper in five of eight (63 percent) accidents. For installations that were classified as straight, performance was rated proper in two of four (50 percent) accidents. It is worth noting the specifics of the three accidents involving a BCT end treatment with a parabolic flare that resulted in improper performance:

TABLE 2 Performance Related to BCT End-Treatment Configuration

End-Treatment Configuration	Proper Performance		Improper Performance	
	No.	Percent	No.	Percent
Simple curve	23	61	15	39
Parabolic flare	5	63	3	37
Straight	2	50	2	50
Total	30	60	20	40

1. A small car hit the BCT at a moderate angle and overturned;
2. A single-unit truck struck the BCT with its left fender, spun 90 degrees, and overturned; and
3. A large car broke through both wood posts and several metal posts before overturning.

In seven other accidents, the vehicle overturned after impacting the end treatment (six involved a BCT installed on a simple curve and one involved a straight BCT). Only one accident involved spearing of a vehicle. A 1974 Capri went out of control and skidded 210 ft and impacted a BCT installed on a simple curve. Impact was on the driver's door; the vehicle was penetrated by the rail and continued for 20 ft before coming to rest.

Presented in Table 3 is a summary of impact severity cross-tabulated with end-treatment configuration and related to performance. A severe impact was one sufficient to cause heavy or extensive damage to the guardrail, disabling damage to the vehicle, and with injury severity classified as fatal or incapacitating. Nonsevere was classified as slight or moderate damage to the guardrail, functional or nonfunctional damage to the vehicle, and slight or no injury. The data show that proper performance was much higher for nonsevere impacts (73 percent) as compared with severe impacts (55 percent). For end sections installed on a simple curve, there was 55 percent proper performance in severe impacts compared with 86 percent in nonsevere impacts. Even though the sample was small, severe accidents involving the parabolic flare resulted in proper performance in only 57 percent of the accidents (four of seven). As noted previously, in the three cases of improper performance involving a parabolic flare, the vehicle overturned after impacting the end treatment.

Impact angle was cross-tabulated with end-treatment configuration and related to performance in Table 4. A higher percentage of improper performance was noted for impacts at shallow angles (15 degrees or less) than for moderate-to-sharp angles (greater than 16 degrees). At shallow angles, the BCT installed on a simple curve performed properly less

TABLE 3 Impact Severity Related to BCT End-Treatment Performance

Impact Severity	End-Treatment Configuration	Proper Performance		Improper Performance	
		No.	Percent	No.	Percent
Severe	Simple curve	17	55	14	45
	Parabolic flare	4	57	3	43
	Straight	1	100	–	–
	Subtotal	22	55	17	45
Nonsevere	Simple curve	6	86	1	14
	Parabolic flare	1	100	–	–
	Straight	1	33	2	67
	Subtotal	8	73	3	27

TABLE 4 Impact Angle Related to BCT End-Treatment Performance

Impact Angle	End-Treatment Configuration	Proper Performance		Improper Performance	
		No.	Percent	No.	Percent
Shallow	Simple curve	11	50	11	50
	Parabolic flare	2	67	1	33
	Straight	–	–	1	100
	Subtotal	13	50	13	50
Moderate-sharp	Simple curve	8	75	4	25
	Parabolic flare	2	50	2	50
	Straight	–	–	–	–
	Subtotal	10	63	6	37

frequently (50 percent) than it did when impacted at moderate-to-sharp angles (75 percent). This could be related to the stiffness of the BCT end section when installed without the parabolic flare, a condition that would be worse for impacts at shallow angles. For impacts into an end treatment installed on a parabolic flare, performance was proper in two of three accidents at shallow angles and two of four at moderate-to-sharp angles.

The results of comparing damage to the various end-treatment configurations with performance are presented in Table 5. End-treatment damage was classified as either slight to moderate or heavy to extensive. Generally, slight-to-moderate damage was deflection of the rail, bending of both posts or the breaking away of one, and/or movement of the concrete footing. Heavy-to-extensive damage was breaking away of both posts and/or breaking of both posts with damage to the rail beyond the second post. When all end-treatment types were combined, performance results were nearly the same for slight-to-moderate and heavy-to-extensive end-treatment damage. For BCT end treatments installed on a simple curve, performance was proper in 7 of 10 accidents (70 percent) when end-treatment damage was slight to moderate and in 11 of 18 accidents (61 percent) when damage was heavy to extensive. Even though only a small sample of accidents was available for end treatments with the parabolic flare, it was found that performance was better for accidents in which end-treatment damage was heavy to extensive (three of four) as compared with slight-to-moderate damage (one of two).

Presented in Table 6 is a summary of performance when vehicle size was cross-tabulated with end-treatment configuration. Five impacts involved small cars and the end treatment performed properly in only one of the collisions. For impacts involving large automobiles, the end treatment performed properly in 14 of 24 accidents (58 percent). For those accidents involving large automobiles, performance

TABLE 5 End-Treatment Damage Related to BCT End-Treatment Performance

End-Treatment Damage	End-Treatment Configuration	Proper Performance		Improper Performance	
		No.	Percent	No.	Percent
Slight-moderate	Simple curve	7	70	3	30
	Parabolic flare	1	50	1	50
	Straight	1	50	1	50
	Subtotal	9	64	5	36
Heavy-extensive	Simple curve	11	61	7	39
	Parabolic flare	3	75	1	25
	Straight	1	50	1	50
	Subtotal	15	63	9	37

TABLE 6 Vehicle Size Related to End-Treatment Performance

Vehicle Size	End-Treatment Configuration	Proper Performance		Improper Performance	
		No.	Percent	No.	Percent
Small automobile	Simple curve	1	25	3	75
	Parabolic flare	–	–	1	100
	Straight	–	–	–	–
	Subtotal	1	20	4	80
Large automobile	Simple curve	11	55	9	45
	Parabolic flare	2	67	1	33
	Straight	1	100	–	–
	Subtotal	14	58	10	42
Truck	Simple curve	3	60	2	40
	Parabolic flare	–	–	1	100
	Straight	–	–	–	–
	Subtotal	3	50	3	50

was proper for 11 of 20 when the BCT included a simple curve and 2 of 3 when the BCT included a parabolic flare. In the six accidents involving trucks, performance was rated proper in three cases. In all three cases of improper performance involving trucks, the vehicle overturned.

It should be noted that vehicle size information was available in sufficient detail to categorize only 35 of the 50 BCT accidents. However, in six other accidents, it was determined that the vehicle was an automobile of unknown size. Performance was rated proper in all six of those accidents; five were at locations where the BCT was a simple curve and one involved a straight BCT.

Data relating the most severe injury in each accident with end-treatment configuration are as follows:

End-Treatment Configuration	No. of Accidents by Severity: Fatal	Injury	Property Damage
Simple curve	5	20	7
Parabolic flare	–	4	1
Straight	–	–	2

There were five fatal accidents and all of these occurred at locations where the BCT had been installed on a simple curve. Of the 24 injury accidents, 8 involved incapacitating injuries and all of these were at locations where the BCT was a simple curve. Injury accidents involving BCTs installed on a parabolic flare resulted in less severe injuries than those involving the simple curve. For accidents in which injury severity was known, 5 of 39 (13 percent) resulted in a fatality. A substantial percentage of accidents (33 percent) resulted in either a fatality or an incapacitating injury. Of the five fatal accidents, one involved spearing of a small car and three involved overturning of the vehicle; in the fifth a car broke one post and then spun counterclockwise 180 degrees.

Improper performance was generally associated with one of the following occurrences: (a) the vehicle hit the end treatment and was stopped when the posts did not break, (b) the vehicle overturned as it hit the end and the post did not break as designed, or (c) a concrete footing moved, which prevented the posts from breaking. There was one instance in which the BCT end treatment (simple curve) speared the vehicle. Other researchers have shown that the BCT has failed to perform properly when impacted head on by small cars. Head-on crash tests performed by SWRI (14) showed that small cars performed satisfactorily in 30-mph tests but not in 60-mph tests. Instances of spearing are usually the result of an impact with an end treatment that has no flare. As will be shown, such a problem may occur when an MBCT end treatment installed in a gore location is impacted.

An analysis of injury severity as compared with end-treatment performance was made. This showed performance to be proper more frequently in accidents where there were no injuries or the injuries were not severe. Injury severity was also compared with end-treatment damage, and it was found that injuries generally were more severe when damage was greater.

MBCT End-Treatment Accidents

Performance was determined for 12 of the accidents involving an MBCT end treatment, with 6 (50 percent) rated as proper performance. Only two of eight severe impacts (25 percent) resulted in proper performance, whereas all four nonsevere impacts were termed proper. Impact angles were classified as either shallow or moderate. For both impact angles, only two of five accidents (40 percent) resulted in proper performance. All accidents (three) in which heavy or extensive guardrail damage resulted and in which performance was also rated resulted in improper performance. Only two accidents in which vehicle size was known involved a small car. Both accidents involved collision with an MBCT placed in a gore and resulted in improper performance in which the end speared the vehicle (Figure 5).

Of 12 accidents in which injury severity was known, 9 (75 percent) resulted in some type of injury and 5 (42 percent) resulted in either a fatality or incapacitating injury. There were two fatal accidents, both the result of spearing when a small vehicle impacted an MBCT in a gore area. The vehicle received disabling damage in 11 of 12 accidents (92 percent). Impact severity was classified as severe in 14 of the 19 accidents (74 percent). Collisions with either small or large automobiles resulted in severe impacts. There were no known accidents involving either a single-unit or combination truck. Six of the 10 accidents (60 percent) in which damage was known resulted in either heavy or extensive guardrail damage.

The MBCT end treatment has been used on medians and at least one gore location. For those accidents in which performance could be rated, both gore accidents were classified as giving improper performance whereas 4 of 10 median-location accidents were classified as resulting in improper performance.

CONCLUSIONS

The analysis of the accidents investigated shows that any accident involving collision with a guardrail end is potentially severe. The BCT end treatment performed properly in most accidents (60 percent); that is, the end treatment performed as designed: the wooden posts broke away or the guardrail redirected the vehicle. This percentage of proper performance occurred even though the BCT was found to have been installed with a parabolic flare in only 8 of the 50 accidents investigated. Most BCT end-treatment configurations evaluated included those installed on a 4.5-degree simple curve with an offset of approximately 6 ft at the end (38 installations) and those installed basically straight with a very small or no offset (4 installations). Only five impacts involved small cars and the BCT end treatment performed properly in only one of these accidents. Improper performance of the BCT was generally related to either failure of the posts and guardrail to break away as designed, causing the vehicle to stop abruptly or overturn, or excessive movement of a concrete footing that prevented the posts from breaking. One accident involved spearing of the vehicle. Performance was not as good when the impact angle was shallow. Poor performance for shallow impact angles involving BCTs and the problem exhibited by MBCT end treatments impacted head on show that a flare is necessary. Any installation of a BCT end treatment without proper flare provides the potential for spearing of a vehicle during a shallow-angle impact.

Evaluation of the performance of Kentucky's BCT end treatment indicates that it may be used where geometrics permit, that is, when a 4-ft flare can be obtained with a 10:1 slope in front and a sufficient recovery area, not exceeding a 3:1 slope, behind. Slopes referred to here are based on general guidelines for BCT design as noted in the survey of other states performed by the Kentucky Transportation Research Program (12) and from the AASHTO barrier guide (15). Where those geometrics are not present, the turned-down end treatment proposed in the previous report should be used (12).

The MBCT end treatment performed properly only 50 percent of the time. The problem appears to be related to the stiffness of the end treatment and is most apparent when the MBCT is used in a gore area where impact angles are shallow. Two fatal accidents occurred when the end treatment speared a small vehicle after a head-on collision in a gore area. If these two accidents involving a MBCT placed in a gore area are excluded, then in 6 of the 10 accidents (60 percent) involving MBCTs in medians, there was proper performance.

The MBCT design as used in Kentucky should be removed from gore locations. The recommended replacement at gore locations would be a crash cushion. Because of the stiffness of the MBCT and the problems associated with impacts at shallow angles, consideration should be given to modification or elimination of Kentucky's MBCT design. It is important that consideration be given to the need for crash testing any new or modified designs for median end treatments before they are used in the field. The importance of evaluating even minor modifications to safety barriers was stressed in NCHRP Report 230 (16).

The question as to the best end treatment that may be used for shoulder and median installations has not been resolved. A continued in-field performance evaluation of the BCT, MBCT, and new turned-down end treatments through in-depth analysis of accidents is warranted. This type of performance evaluation would provide valuable information for future decisions concerning the most crashworthy end treatment to use.

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REFERENCES

1. N.J. Deleys and R.R. McHenry. Highway Guardrails--A Review of Current Practice. NCHRP Report 36. HRB, National Research Council, Washington, D.C., 1967.

2. J.D. Michie and L.R. Calcote. Location, Selection, and Maintenance of Highway Guardrails and Median Barriers. NCHRP Report 54. HRB, National Research Council, Washington, D.C., 1968.
3. J.D. Michie and L.R. Calcote. Guardrail Performance and Design. NCHRP Report 115. TRB, National Research Council, Washington, D.C., 1971.
4. J.D. Michie and M.E. Bronstad. Location, Selection, and Maintenance of Highway Traffic Barriers. NCHRP Report 118. TRB, National Research Council, Washington, D.C., 1971.
5. J.D. Michie and M.E. Bronstad. Guardrail Crash Test Evaluation--New Concepts and End Designs. NCHRP Report 129. TRB, National Research Council, Washington, D.C., 1972.
6. Evaluation of Breakaway Cable Terminals for Guardrails. NCHRP Research Results Digest 43. TRB, National Research Council, Washington, D.C., 1972.
7. Development of a Breakaway Cable Terminal for Median Barrier. NCHRP Research Results Digest 53. TRB, National Research Council, Washington, D.C., 1973.
8. Breakaway Cable Terminals for Guardrails and Median Barriers. NCHRP Research Results Digest 84. TRB, National Research Council, Washington, D.C., March 1976.
9. Modified Breakaway Cable Terminals for Guardrail and Median Barriers. NCHRP Research Results Digest 102. TRB, National Research Council, Washington, D.C., May 1978.
10. A Modified Foundation for Breakaway Cable Terminals. NCHRP Research Results Digest 124. TRB, National Research Council, Washington, D.C., Nov. 1980.
11. R.F. Baker. Breakaway Cable Terminal Evaluation. Report FHWA/NJ-81/001. New Jersey Department of Transportation, Trenton, May 1980.
12. J.G. Pigman and K.R. Agent. Survey of Guardrail End Treatment Usage. Report UKTRP-83-23. Transportation Research Program, University of Kentucky, Lexington, Oct. 1983.
13. J.G. Pigman, K.R. Agent, and T. Creasey. Analysis of Accidents Involving Breakaway-Cable-Terminal End Treatment. Report UKTRP-84-16. Transportation Research Program, University of Kentucky, Lexington, June 1984.
14. C.E. Kimball, Jr., M.E. Bronstad, and L. Meczkowski. Evaluations of Guardrail Breakaway Cable Terminals. Report FHWA-RD-82-57. Southwest Research Institute, San Antonio, Tex., 1982.
15. Guide for Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, D.C., 1977.
16. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP Report 230. TRB, National Research Council, Washington, D.C., 1981.

Analysis of Accidents Involving Crash Cushions

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ABSTRACT

This paper is an analysis of 127 accidents involving crash cushions in Kentucky. The primary data base was for the period 1980-1982, with some additional data before and after this period. An attempt was made to document each accident with a police report, photographs, and a repair form. The largest number of accidents (63) involved a Hi-Dro cell cushion or cluster, followed by 33 accidents involving a Guardrail Energy-Absorbing Terminal (G.R.E.A.T.) crash cushion, 19 with a temporary G.R.E.A.T. system, 10 with sand barrels, and 2 with steel drums. Average repair cost was lowest for the Hi-Dro cell cushion ($392) and highest for the Hi-Dro cell cluster ($2,839). Other repair costs were $1,886 for the G.R.E.A.T. system, $887 for sand-barrel installations, and $1,760 for steel-drum installations. For those accidents in which performance was noted, crash cushions performed properly 85 percent of the time. Instances of improper performance generally involved either rebounding of a vehicle into or across the adjacent roadway or overturning of a vehicle. All the various types performed well. Results from the cost-effectiveness analysis show that crash cushion installations produce a benefit/cost ratio in the range of 1.0-2.0.

Hazardous fixed objects located within the driving environment continue to present severe safety problems to errant vehicles and their drivers. When the roadway is wholly or partially on structure, the gore area is characterized by bridge abutments or massive bases for sign supports. Bridge piers and other fixed objects located in medians have previously been inadequately shielded by guardrail or not shielded at all. In addition, roadways with narrow medians separated only by guardrail have proven to be ineffective and the source of many severe or fatal accidents. More recent designs have incorporated the concrete median barrier. At other locations where guardrail is deemed adequate, the breakaway-cable-terminal guardrail end treatment is now being used.

On the basis of the 1978 revision of the Handbook of Highway Safety Design and Operating Practices (1), highway traffic barriers may be classified into two general groups: (a) longitudinal barriers, such as guardrails, concrete median barriers, and bridge railings, which redirect vehicles away from roadside hazards; and (b) crash cushions, which incorporate various methods to reduce the rate of deceleration for vehicles (1). Running off the road has been shown to account for approximately 40 percent of all fatal accidents, and collisions with fixed objects are frequently the culmination of the out-of-control vehicle's trip (2). Recent design standards have emphasized the need to install barriers only when the consequence of striking a barrier is less than that of striking the object being shielded. This problem of barrier overuse can be of considerable consequence in gore areas where past research has shown that the rate of accidents is approximately four times that of run-off-the-road accidents at other locations (3). Gore areas that are not or cannot be modified to provide favorable terrain and unobstructed recovery zones have been recognized as misfits in the environs of the highway. Bridge piers in narrow medians and openings between parallel bridges on divided highways are also potential hazards from which the driver should be protected. Crash cushions are an alternative means of shielding the errant vehicle at these types of locations.

Analyses of accidents involving crash cushion impacts have shown these installations to be very effective. A study by FHWA in 1973 included analysis of 188 crash cushion installations in 36 states (4). It was determined that there were 5 fatalities in a total of 393 accidents. It was also found that the total accident experience increased because of a reduction of clear area in the gores and a higher accident reporting level in the after period. Installation and maintenance costs were also reported from the study in 1973. Installation costs were lowest for the sand-barrel installations and the liquid-cell clusters and highest for the steel-drum installations.

Another analysis of accidents involving crash cushions was performed by the Texas Transportation Institute (TTI) (5) that included 135 steel drums and sand barrels. Included were 400 impacts over a 7-year period. Results from crash experience showed that elimination of the redirection panels on steel-drum crash cushions at sites with low probability of angular impacts would improve the safety and reduce construction and maintenance costs.

The design and evaluation of crash cushions began in Kentucky in 1970 with the installation of a sand-barrel system and a liquid-cell system. Following those installations, a survey of the Interstate system was made and the result was a list of 23 gore sites that were considered to be candidates for crash cushion installations or other types of improvements (6). Barriers were installed at 16 sites, and 7 sites were contour graded. Accident experience was monitored at five crash cushion locations in Kentucky from 1970 through 1972 (6). Included were three sand-barrel installations and two liquid-cell installations. At one sand-barrel installation, there were 24 police-investigated accidents during a 37-month period before the barrier was installed. After installation, there were only four accidents in a 24-month period, all minor ones as compared with two fatalities and seven incapacitating injuries before installation. Increased recovery area and the conspicuous nature of the sand barrels were determined to be responsible for the large decrease in accidents. At another sand-barrel installation, a considerably different accident history resulted. In a 36-month period before installation, 33 police-investigated accidents were reported. After installation, 18 accidents occurred in a 18-month period. Reduced recovery area was determined to be the primary cause of the continued high number of accidents. Modifications were made to the gore area so that the more compact liquid-cell unit could be installed and the result was a significant decrease in the number of accidents.

CRASH CUSHION USE IN KENTUCKY

Crash cushions were first installed in Kentucky in 1970. During that year, three sand-barrel systems (Kentucky's Type II) were installed at an average cost of $3,583 per unit and three Hi-Dro cell systems (Kentucky's Type IV) were installed at an average cost of $6,844 per unit. Average unit costs were obtained from tabulations of contracts awarded by the Kentucky Department of Highways and from records of installations by state personnel. Prices for other types of crash cushions did not vary as much, even though the sample of locations was relatively small.

Crash cushion installations were relatively infrequent during the early 1970s, with the exception of several Hi-Dro cell clusters installed at toll booths. Recent crash cushion installations in Kentucky have been almost exclusively the Guardrail Energy-Absorbing Terminal (G.R.E.A.T.) System. Presented in Table 1 is a summary of crash cushion installations by year for the period 1970 through

TABLE 1 Summary of Crash Cushion Installations by Year

	Crash Cushion Type							
				IV				
Year	I	II	III	Cluster	Cushion	V	VI	VI-T
1970		3			3			
1971								
1972				12				
1973				12		1		
1974				6				
1975				6	4		6	
1976				16			15	
1977				10	2		14	8
1978		1		10			20	6
1979				7			20	20
1980				2	7		2	10
1981				20			5	10
1982				2			17	26
1983							22	59
Total		4		103	16	1	121	139

Note: Crash cushion types are defined as follows: I, Energite module inertial barrier; II, Fitch-type energy-absorbing barrier system; III, Hi-Dro cell-type energy-absorbing barrier system; IV, Hi-Dro cushion-type energy-absorbing barrier system; V, steel crash-cushion-type energy-absorbing barrier system; VI, Guardrail Energy-Absorbing Terminal (G.R.E.A.T.) system; VI-T, Guardrail Energy-Absorbing Terminal (G.R.E.A.T.)-temporary system.

1983. Numbers of crash cushions were obtained from tabulations of contracts awarded by the Kentucky Department of Highways. From Table 1, it may be seen that 384 crash cushions were installed during the 14-year period. Many of the temporary G.R.E.A.T. systems were installed for short periods of time on construction projects and then reused. There have been four sand-barrel systems and only one steel-drum system installed in Kentucky. A total of 119 Hi-Dro cell systems have been installed; 103 are clusters installed at toll booths and 16 are cushions installed at other locations such as gore areas and bridge piers. As noted, most of the recent crash cushions have been the G.R.E.A.T. type, and they now total 121. In addition, there are a large number of temporary G.R.E.A.T. installations (a total of 139).

DATA COLLECTION

Initially, police reports of accidents involving crash cushions were collected for 1980, 1981, and 1982. The accident reports were made available through the Accident Surveillance Section of the Division of Traffic of the Kentucky Transportation Cabinet. An inventory of all Kentucky routes having crash cushion installations was used, and accident reports pertaining to these routes were reviewed and appropriately selected. This established a 3-year data base for accidents involving all types of crash cushions.

The next step involved obtaining photographs to aid in the documentation process. When the accident report indicated that photographs had been taken at the scene, a request was made by telephone or in writing to the reporting police agency. Some photographs were obtained through communication with maintenance officials from each highway district, either through written correspondence or through notification that would allow the study team members to investigate the scene shortly after the accident had occurred. When available, repair forms also were obtained from maintenance officials. Therefore, an individual accident possibly could be documented by a police accident report, photographs, and a repair form. However, most cases could not be documented this thoroughly.

Finally, some accidents occurring either before 1980 or after 1982 were included for the purpose of strengthening the sample size. These cases were either already in possession before the beginning of the study or were discovered in the search process. In all, information on 127 accidents involving crash cushions was obtained.

RESULTS

Data for a total of 127 crash cushion accidents were included in the analysis. A summary of accident locations and information available is given in Table 2. A detailed description of each crash cushion accident was presented in an appendix to the full report (7), in which a narrative describing the accident, an accident diagram (when sufficient information was available), and photographs, when available, were included.

The largest number of accidents (63) involved a Hi-Dro cell cushion or cluster (Type IV). Of these 63, 41 involved Hi-Dro cell cluster installations on the toll road system. This was followed by 33 accidents involving a G.R.E.A.T. crash cushion (Type VI) and 19 with a temporary G.R.E.A.T. (Type VI-T). There were 10 accidents involving sand barrels (Type I or II) and 2 involving steel drums (Type V).

The large majority of accidents occurred from 1980 through 1982: 42 in 1980, 28 in 1981, and 25 in 1982. There were 16 accidents from the period before 1980 and 16 after 1982.

The largest number of accidents occurred in District 6 (49 accidents) followed by District 5 (31 accidents). This was expected because those two districts had the largest number of crash cushions. Four districts had no crash cushion accidents, and two districts had only one accident each.

Repair costs were available for several of the accidents included in this analysis as well as others. The lowest average repair cost was $395 for 45 repairs of the Hi-Dro cell crash cushion. One accident required replacement of the Hi-Dro cell crash cushion at a cost of about $11,000. This compares with an average cost of $2,839 for 19 repairs of Hi-Dro cell clusters. The average cost of 52 repairs to sand-barrel installations was $887. This includes repairs over the past 10 years, and costs for the most recent repairs have averaged about twice that amount. The average cost of 20 repairs to G.R.E.A.T. crash cushion installations was $1,886. The average cost to repair three steel-drum installations was $1,760.

The possible sources of information concerning the accidents included accident reports, photographs, and repair forms. Accident reports were obtained for 125 of the 127 accidents. Photographs were obtained for 19 accidents, and a repair form was obtained for 28 accidents. All three sources of information were obtained for only nine accidents. Following is a discussion of the results from analyses of crash cushion accidents.

Crash Cushion Performance

A summary of the performance of crash cushions for each accident is given in Table 3. In addition to crash cushion performance, information concerning type of crash cushion, vehicle size, impact severity, type of impact, crash cushion placement, initial vehicle contact area, vehicle action after impact, and crash cushion damage is given. Subjective judgment was used to determine many of these variables. A description of the variable categories is given in Table 4.

Performance was rated in 116 of the accidents as either proper or improper. In proper performance the crash cushion performed as designed with the impact energy fully attenuated in head-on, broadside, and angle collisions. For sideswipe impacts, proper performance was defined as the condition when the vehicle was redirected at a shallow angle back into the adjacent traffic lane. In six accidents, insufficient information was available to rate performance. The other five accidents involved impact with a high-speed heavy truck in which the crash cushion was destroyed. Performance was not rated in those accidents because the crash cushions were not designed for such impacts, so a "does not apply" category was used. Performance of the crash cushions was judged to be very good; 85 percent of the collisions resulted in proper performance.

The detailed analysis of the data given in Table 3 is summarized in Table 5. Crash cushion performance was determined as a function of type of crash cushion, vehicle size, impact severity, and type of impact.

All types of crash cushions were found to have a high percentage of proper performance. Performance was termed improper in only 17 accidents. The problem was related primarily to rebounding of the vehicle into or across the roadway at a sharp angle or to rolling over of the vehicle. One of these two vehicle actions occurred in 14 of the 17 improper-

TABLE 2 Summary of Accident Locations and Information Available

Accident No.	District	County	Route	Milepoint	Crash Cushion Type	Date	Information Available: Accident Report	Photographs	Repair Forms
001	2	Christian	Pennyrile	11.8	IV	11/21/82	X		
002	2	Christian	Pennyrile	11.7	IV	2/05/80	X		
003	2	Henderson	Audubon	10.2	IV	12/12/82	X		
004	2	Hopkins	Western Kentucky	24.4	IV	6/01/81	X		
005	2	Hopkins	Western Kentucky	24.4	IV	12/17/82	X		
006	2	Hopkins	Western Kentucky	24.4	IV	12/31/82	X		
007	2	Hopkins	Western Kentucky	24.4	IV	7/07/83	X		
008	2	Hopkins	Western Kentucky	24.4	IV	7/26/83	X		
009	2	Muhlenberg	Western Kentucky	58.0	IV	10/26/83	X	X	X
010	2	Muhlenberg	Western Kentucky	58.0	IV	1/14/82	X		
011	2	Muhlenberg	Western Kentucky	58.0	IV	6/21/80	X		
012	2	Muhlenberg	Western Kentucky	58.0	IV	5/07/80	X		
013	2	Ohio	Green River	47.9	IV	11/26/81	X		
014	2	Ohio	Western Kentucky	47.8	IV	7/01/83	X		X
015	2	Webster	Western Kentucky	62.6	IV	7/21/83	X	X	X
016	2	Webster	Pennyrile	62.6	IV	12/18/82	X	X	X
017	2	Webster	Pennyrile	62.6	IV	6/16/82	X	X	X
018	2	Webster	Pennyrile	62.6	IV	1/06/81	X		
019	2	Webster	Pennyrile	62.6	IV	7/22/80	X		X
020	3	Barren	Cumberland	3.1	IV	5/22/81	X		X
021	3	Butler	Green River	13.8	IV	11/14/80	X		X
022	4	Grayson	Western Kentucky	107.0	IV	8/04/82	X		
023	4	Grayson	Western Kentucky	107.0	IV	11/23/80	X		
024	4	Grayson	Western Kentucky	107.0	IV	3/17/80	X		X
025	4	Grayson	Western Kentucky	107.0	IV	3/01/80	X		X
026	4	Grayson	Western Kentucky	107.0	IV	4/18/77	X	X	X
027	4	Grayson	Western Kentucky	107.0	IV	10/11/79	X		
028	4	Grayson	Western Kentucky	107.0	IV	5/29/80	X		X
029	4	Grayson	Western Kentucky	107.0	IV	7/17/81	X		X
030	4	Grayson	Western Kentucky	107.0	IV	10/30/81	X		
031	4	Grayson	Western Kentucky	107.0	IV	1/04/83	X	X	X
032	4	Grayson	Western Kentucky	107.0	IV	10/14/83	X		X
033	4	Grayson	Western Kentucky	107.0	IV	3/20/84	X		
034	4	Nelson	Bluegrass	33.3	IV	12/23/79	X		
035	4	Nelson	Bluegrass	33.3	IV	11/17/84	X	X	X
036	4	Nelson	Bluegrass	9.7	IV	2/04/82	X		X
037	4	Nelson	Bluegrass	33.3	IV	4/15/81	X		
038	4	Nelson	Bluegrass	33.7	IV	11/18/80	X		
039	5	Franklin	US-421	3.0	VI	3/04/82	X		
040	5	Franklin	KY-676	Unknown	VI	3/17/80	X	X	
041	5	Henry	I-71	37.1	VI-T	10/15/81	X		
042	5	Jefferson	I-64	2.7	V	8/29/82	X	X	X
043	5	Jefferson	I-64	2.7	V	4/11/80	X		
044	5	Jefferson	I-64	4.5	IV	3/17/80	X		
045	5	Jefferson	I-65	123.5	VI	1/30/84	X		
046	5	Jefferson	I-65	133.0	IV	3/09/83	X		
047	5	Jefferson	I-65	136.3	IV	10/09/82	X		
048	5	Jefferson	I-65	136.4	IV	10/02/82	X		
049	5	Jefferson	I-65	133.0	IV	7/29/82	X	X	
050	5	Jefferson	I-65	133.0	IV	6/24/82	X		
051	5	Jefferson	I-65	136.5	IV	6/10/82	X	X	X
052	5	Jefferson	I-65	133.0	IV	4/27/82	X		
053	5	Jefferson	I-65	136.5	IV	4/26/82	X		
054	5	Jefferson	I-65	136.4	IV	2/09/82	X		
055	5	Jefferson	I-65	136.3	IV	11/05/81	X		
056	5	Jefferson	I-65	136.7	IV	2/07/81	X		
057	5	Jefferson	I-65	125.0	VI-T	12/09/80	X		
058	5	Jefferson	I-65	126.0	VI	12/05/80	X		
059	5	Jefferson	I-65	125.0	VI-T	12/04/80	X		
060	5	Jefferson	I-65	136.7	IV	4/01/80	X		
061	5	Jefferson	I-65	123.5	VI	1/31/78	X	X	X
062	5	Jefferson	I-264	7.5	II	11/12/82	X		X
063	5	Jefferson	I-264	19.9	VI	9/26/82	X		
064	5	Jefferson	I-264	19.1	II	9/26/82	X		X
065	5	Jefferson	I-264	7.5	II	8/27/82	X		X
066	5	Jefferson	I-264	19.9	VI	5/22/82	X		
067	5	Jefferson	I-264	7.5	II	2/09/82	X		
068	5	Jefferson	I-264	11.0	VI	7/27/80	X		
069	5	Jefferson	I-264	9.1	VI	3/06/80	X		
070	6	Campbell	I-275	77.0	VI	10/16/81	X		
071	6	Campbell	KY-9	13.7	VI	3/14/81	X		
072	6	Campbell	KY-9	13.7	VI	3/02/81	X		
073	6	Campbell	KY-9	13.7	VI	2/15/81	X		
074	6	Campbell	KY-9	13.7	VI	1/25/81	X		
075	6	Campbell	KY-9	13.7	VI	1/19/81	X		
076	6	Campbell	KY-9	13.7	VI	1/14/81	X		
077	6	Campbell	KY-6	13.7	VI	1/11/81	X		
078	6	Campbell	KY-9	13.7	VI	10/30/80	X		
079	6	Campbell	KY-9	13.7	VI	9/04/80	X		
080	6	Campbell	KY-9	13.7	VI	7/23/80	X		
081	6	Campbell	KY-9	13.7	VI	4/30/80	X		
082	6	Campbell	KY-9	13.7	VI	3/23/80	X		
083	6	Campbell	KY-9	13.7	VI	2/05/80	X		
084	6	Campbell	KY-9	13.7	VI	12/17/77	X	X	

TABLE 2 continued

Accident No.	District	County	Route	Milepoint	Crash Cushion Type	Date	Information Available: Accident Report	Photographs	Repair Forms
085	6	Campbell	KY-9	13.7	VI	11/06/76	X		X
086	6	Campbell	KY-9	13.7	VI	3/07/77	X		X
087	6	Campbell	KY-9	13.7	VI	5/14/77		X	X
088	6	Campbell	KY-9	13.7	VI	5/26/77		X	X
089	6	Kenton	I-75	191.3	II	12/22/72	X		X
090	6	Kenton	I-75	191.3	II	10/26/72	X		X
091	6	Kenton	I-75	191.3	II	4/29/72	X		X
092	6	Kenton	I-75	191.3	II	5/17/72	X		X
093	6	Kenton	I-75	191.3	II	6/03/72	X		X
094	6	Kenton	I-75	191.3	II	8/04/72	X		X
095	6	Kenton	I-75	191.3	IV	1/18/84	X		
096	6	Kenton	I-75	191.3	IV	12/23/83	X		
097	6	Kenton	I-75	191.3	IV	10/23/83	X	X	
098	6	Kenton	I-75	184.7	VI	12/27/81	X		
099	6	Kenton	I-75	191.3	IV	11/08/81	X		
100	6	Kenton	I-75	191.3	IV	8/15/81	X		
101	6	Kenton	I-75	184.0	VI	6/18/81	X		
102	6	Kenton	I-75	191.3	IV	4/11/81	X		
103	6	Kenton	I-75	191.4	IV	3/13/81	X		
104	6	Kenton	I-75	186.7	VI-T	11/18/80	X		
105	6	Kenton	I-75	186.5	VI-T	11/11/80	X		
106	6	Kenton	I-75	186.6	VI-T	11/11/80	X		
107	6	Kenton	I-75	186.7	VI-T	11/09/80	X		
108	6	Kenton	I-75	186.9	VI-T	11/08/80	X		
109	6	Kenton	I-75	186.7	VI-T	10/05/80	X		
110	6	Kenton	I-75	186.8	VI-T	9/30/80	X		
111	6	Kenton	I-75	186.5	VI-T	9/26/80	X		
112	6	Kenton	I-75	187.0	VI-T	9/21/80	X		
113	6	Kenton	I-75	191.3	IV	9/06/80	X		
114	6	Kenton	I-75	191.3	IV	8/30/80	X		
115	6	Kenton	I-75	190.6	VI-T	6/10/80	X		
116	6	Kenton	I-75	190.6	VI-T	6/07/80	X		
117	6	Kenton	I-75	184.1	VI	5/15/80	X		
118	6	Kenton	I-75	188.0	VI-T	5/04/80	X		
119	7	Anderson	Bluegrass	58.8	IV	9/19/81	X		
120	7	Anderson	Bluegrass	58.8	IV	4/17/81	X		
121	7	Anderson	Bluegrass	58.8	IV	4/13/80	X	X	
122	7	Fayette	I-75	116.9	VI-T	10/04/81	X		
123	7	Fayette	I-75	112.0	VI-T	7/02/80	X		
124	7	Fayette	I-75	113.5	VI-T	9/23/79	X	X	
125	7	Scott	I-75	128.3	VI-T	8/13/83	X	X	
126	11	Harlan	US-119	14.0	VI	12/17/81	X		
127	12	Pike	US-23	Unknown	VI	2/13/83	X	X	

performance accidents. Of the three accidents with Hi-Dro cell crash cushions in which there was improper performance, all involved a rollover. Of the seven G.R.E.A.T. crash cushion accidents with improper performance, three were rollover and four involved a rebound. Two of the three accidents with a temporary G.R.E.A.T. installation with improper performance involved a rebound and in the other the temporary G.R.E.A.T. crash cushion was knocked from its base by the impact. Of the two sand-barrel accidents with improper performance, one involved a rebound and in the other the vehicle impacted the bridge abutment. In one of the two Hi-Dro cell cluster accidents with improper performance, a large automobile knocked the cluster from its brace and impacted the abutment in front of the toll booth. The other accident with improper performance involved a rebound into a light pole. Except for five heavy-truck accidents in which the crash cushions were destroyed, the crash cushions prevented the vehicles from impacting the shielded object with two exceptions. One exception occurred when a vehicle hit a sand-barrel installation next to a back corner, which allowed impact with a bridge abutment. The other was the improper performance of the Hi-Dro cell cluster.

When vehicle size was analyzed as to performance, the percentage with proper performance was high for all vehicle types. All but one nonsevere impact was rated proper. The one improper nonsevere impact involved a rebound. Performance was also high for all types of impact. Improper performance was higher for angle than head-on impacts because of the higher possibility of rebound and rollover.

In most instances, crash cushion damage was not known. In those accidents in which damage was documented, it was judged to be either moderate or heavy. The most common location for crash cushion accidents (55 accidents) was gore areas, where various types of crash cushions were used. There were 41 accidents at toll booth locations, all of which involved a Hi-Dro cell cluster. There were 19 accidents in construction zones, all involving a G.R.E.A.T. temporary crash cushion. There were seven accidents at the termination of a concrete median barrier and five at a bridge pier, primarily involving G.R.E.A.T. crash cushions. Usually the initial vehicle contact area was the front (62 accidents); this was followed by the right front (25 accidents) and the left front (11 accidents).

The primary vehicle action after impact was that the vehicle was stopped by the crash cushion (52 accidents). The second most common action was that the vehicle rebounded left or right (23 accidents). In six accidents, the vehicle overturned. In the remaining accidents with a known vehicle action after impact, the vehicle either continued in the same direction (12 accidents), spun clockwise or counterclockwise (7 accidents), or ramped (1 accident).

TABLE 3 Crash Cushion Performance

Crash Cushion Type	Accident No.	Vehicle Size Category	Impact Severity	Type of Impact	Crash Cushion Placement	Initial Vehicle Contact Area	Vehicle Action After Impact	Crash Cushion Performance	Crash Cushion Damage
IV	001	Auto-L	Severe	Head-on	Toll	1	Rb-R	Proper	Unknown
IV	002	Auto-U	Nonsevere	Head-on	Toll	1	Stop	Proper	Slight
IV	003	Auto-L	Nonsevere	Angle	Toll	4	Rb-R	Proper	Unknown
IV	004	Auto-L	Severe	Angle	Toll	2	Rb-L	Proper	Unknown
IV	005	SUT	Nonsevere	SS	Toll	6	Stop	Proper	Slight
IV	006	Auto-L	Nonsevere	Angle	Toll	2	Cont	Proper	Unknown
IV	007	Comb	Nonsevere	SS	Toll	6	Cont	Proper	Slight
IV	008	Auto-L	Nonsevere	Angle	Toll	1	Stop	Proper	Slight
IV	009	Comb	Severe	Angle	Toll	2	Stop	Proper	Moderate
IV	010	Auto-L	Nonsevere	SS	Toll	2	Stop	Proper	Slight
IV	011	Auto-U	Nonsevere	Head-on	Toll	1	Stop	Proper	Unknown
IV	012	Auto-U	Nonsevere	Angle	Toll	2	Rb-L	Proper	Unknown
IV	013	Auto-S	Nonsevere	Angle	Toll	2	Stop	Proper	Slight
IV	014	Comb	Severe	BSD	Toll	7	Unknown	Unknown	Slight
IV	015	Auto-L	Severe	Head-on	Toll	1	Bb-L	Improper	Extensive
IV	016	Auto-S	Nonsevere	Angle	Toll	4	Rb-R	Improper	Slight
IV	017	Comb	Nonsevere	SS	Toll	6	Rb-L	Proper	Slight
IV	018	Auto-L	Nonsevere	SS	Toll	4	Stop	Proper	Unknown
IV	019	Auto-U	Nonsevere	SS	Toll	2	Rb-L	Proper	Slight
IV	020	Auto-L	Nonsevere	Angle	Toll	4	Rb-R	Proper	Slight
IV	021	Auto-L	Nonsevere	Head-on	Toll	1	Stop	Proper	Slight
IV	022	Comb	Nonsevere	SS	Toll	2	Cont	Proper	Unknown
IV	023	Auto-L	Nonsevere	Unknown	Toll	–[a]	Unknown	Proper	Slight
IV	024	Comb	Nonsevere	SS	Toll	2	Cont	Proper	Slight
IV	025	Auto-L	Nonsevere	SS	Toll	2	Stop	Proper	Slight
IV	026	Auto-L	Severe	Head-on	Toll	1	SP-CCW-90	Proper	Moderate
IV	027	Auto-L	Nonsevere	Angle	Toll	1	Stop	Proper	Slight
IV	028	Comb	Nonsevere	SS	Toll	6	Stop	Proper	Slight
IV	029	Auto-U	Severe	BSD	Toll	3	SP-CCW-180	Proper	Moderate
IV	030	Comb	Nonsevere	SS	Toll	6	Stop	Proper	Slight
IV	031	Auto-L	Severe	Head-on	Toll	1	Rb-R	Proper	Heavy
IV	032	Comb	Nonsevere	SS	Toll	6	Cont	Proper	Slight
IV	033	Comb	Nonsevere	SS	Toll	6	Stop	Proper	Unknown
IV	034	Auto-U	Nonsevere	Angle	Toll	1	Cont	Proper	Slight
IV	035	Comb	Nonsevere	SS	Toll	6,7	Cont	Proper	Slight
IV	036	Auto-L	Severe	Angle	Toll	–[a]	Rb-R	Proper	Moderate
IV	037	Comb	Nonsevere	SS	Toll	2	Cont	Proper	Unknown
IV	038	Comb	Nonsevere	SS	Toll	6,7	Cont	Proper	Slight
V	044	Comb	Severe	BSD	Gore	5	Stop	Proper	Unknown
IV	046	Auto-L	Unknown	Head-on	Gore	1	Stop	Proper	Unknown
IV	047	Auto-U	Nonsevere	Head-on	Gore	1	Stop	Proper	Unknown
IV	048	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Unknown
IV	049	Auto-L	Severe	Head-on	Gore	2	Over	Improper	Heavy
IV	050	Auto-U	Nonsevere	BSD	Gore	5	Unknown	Proper	Unknown
IV	051	Comb	Severe	Angle	Gore	4	Cont	DNA[b]	Heavy
IV	052	Auto-U	Severe	Head-on	Gore	1	Stop	Unknown	Unknown
IV	053	Comb	Severe	Head-on	Gore	1	Stop	Unknown	Unknown
IV	054	Auto-L	Severe	Head-on	Gore	2	Unknown	Proper	Unknown
IV	055	Comb	Severe	Angle	Gore	2	Unknown	Proper	Unknown
IV	056	Auto-L	Severe	Head-on	Gore	1	Over	Improper	Unknown
IV	060	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Unknown
IV	095	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Heavy
IV	096	Auto-U	Severe	Head-on	Gore	1	Stop	Proper	Unknown
IV	097	Comb	Severe	Angle	Gore	1	Unknown	Proper	Heavy
IV	099	Auto-U	Severe	Head-on	Gore	1	Stop	Proper	Unknown
IV	100	Auto-L	Severe	Head-on	Gore	–[a]	Unknown	Proper	Unknown
IV	102	Auto-L	Severe	Head-on	Gore	1	Unknown	Proper	Moderate
IV	103	Auto-L	Severe	Head-on	Gore	4	Stop	Proper	Unknown
IV	113	Auto-L	Severe	SS	Gore	2	Over	Improper	Unknown
IV	114	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Unknown
IV	119	Auto-U	Severe	Head-on	Toll	2	SP-CW-90	Proper	Unknown
IV	120	Auto-U	Severe	Angle	Toll	1	Rb-L	Proper	Unknown
IV	121	Comb	Severe	Angle	Toll	2,3	Cont	Proper	Heavy
VI	039	Auto-L	Severe	Head-on	CMB	1	Stop	Proper	Unknown
VI	040	Auto-L	Severe	Unknown	CMB	4	Unknown	Proper	Heavy
VI	045	Comb	Severe	Head-on	BP	1	Stop	DNA	Extensive
VI	058	Auto-U	Severe	Unknown	BP	–[a]	Over	Improper	Unknown
VI	061	Comb	Severe	BSD	BP	8	Stop	DNA	Extensive
VI	063	Auto-U	Severe	Head-on	Gore	1	Unknown	Proper	Unknown
VI	066	Auto-U	Severe	SS	Gore	5	Unknown	Proper	Unknown
VI	068	Auto-L	Nonsevere	BSD	CMB	3	Stop	Proper	Slight
VI	069	Auto-S	Severe	BSD	CMB	3	Over	Improper	Unknown
VI	070	Auto-L	Severe	Head-on	Gore	1	Unknown	Proper	Moderate
VI	071	Auto-U	Severe	SS	Gore	2	Rb-L	Improper	Moderate
VI	072	Auto-U	Severe	Head-on	Gore	1	Stop	Proper	Moderate
VI	073	Auto-U	Severe	Angle	Gore	2	Unknown	Proper	Moderate
VI	074	Auto-U	Severe	Angle	Gore	2	Rb-L	Improper	Moderate
VI	075	Auto-U	Nonsevere	SS	Gore	3	Unknown	Proper	Moderate
VI	076	Auto-U	Nonsevere	SS	Gore	2	Cont	Proper	Unknown
VI	077	Auto-S	Nonsevere	Head-on	Gore	1	Stop	Proper	Moderate
VI	078	Auto-S	Severe	Head-on	Gore	1	Stop	Proper	Moderate
VI	079	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Moderate
VI	080	Auto-U	Severe	Head-on	Gore	2	Rb-R	Improper	Unknown

TABLE 3 continued

Crash Cushion Type	Accident No.	Vehicle Size Category	Impact Severity	Type of Impact	Crash Cushion Placement	Initial Vehicle Contact Area	Vehicle Action After Impact	Crash Cushion Performance	Crash Cushion Damage
VI	081	Auto-L	Severe	Angle	Gore	2	Rb-L	Improper	Moderate
VI	082	Auto-U	Nonsevere	Unknown	Gore	2	Unknown	Proper	Unknown
VI	083	Auto-U	Severe	Unknown	Gore	1	Over	Improper	Unknown
VI	084	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Moderate
VI	085	SUT	Nonsevere	Angle	Gore	3	Rb-L	Proper	Moderate
VI	086	Auto-S	Severe	Angle	Gore	2,3	Rb-L	Proper	Moderate
VI	087	Unknown	Unknown	Unknown	Gore	–[a]	Unknown	Unknown	Moderate
VI	088	Unknown	Unknown	Unknown	Gore	–[a]	Unknown	Unknown	Moderate
VI	098	Auto-L	Severe	Unknown	CMB	2	Unknown	Unknown	Unknown
VI	101	Auto-L	Severe	Head-on	Gore	1	Unknown	Proper	Unknown
VI	117	Auto-L	Severe	Head-on	CMB	1	Stop	Proper	Unknown
VI	126	Auto-L	Severe	Angle	BP	4	Unknown	Proper	Unknown
VI	127	Auto-L	Severe	Head-on	CMB	1	Stop	Proper	Heavy
VI-T	041	Comb	Severe	Angle	TCB	1	Rb-R	Proper	Unknown
VI-T	057	Auto-U	Severe	Head-on	TCB	1	Stop	Proper	Unknown
VI-T	059	Auto-L	Severe	Head-on	TCB	1	SP-CCW-180	Proper	Unknown
VI-T	104	Auto-S	Severe	Angle	TCB	4	SP-CCW-90	Proper	Unknown
VI-T	105	Auto-L	Severe	Head-on	TCB	1	Stop	Proper	Unknown
VI-T	106	Auto-U	Severe	Head-on	TCB	1	Unknown	Proper	Unknown
VI-T	107	Auto-L	Severe	SS	TCB	5	Unknown	Proper	Unknown
VI-T	108	Auto-S	Severe	Head-on	TCB	1	Rb-L	Improper	Unknown
VI-T	109	Auto-L	Severe	Head-on	TCB	1	Stop	Proper	Unknown
VI-T	110	Auto-L	Nonsevere	SS	TCB	5	Unknown	Proper	Slight
VI-T	111	Auto-L	Severe	Angle	TCB	4	Unknown	Proper	Unknown
VI-T	112	Auto-U	Severe	Angle	TCB	4	Rb-R	Improper	Unknown
VI-T	115	Auto-U	Severe	Head-on	TCB	1	Stop	Proper	Heavy
VI-T	116	Auto-L	Severe	Head-on	TCB	1	Unknown	Proper	Unknown
VI-T	118	Auto-L	Severe	Head-on	TCB	1	Stop	Proper	Unknown
VI-T	122	Auto-L	Severe	Head-on	TCB	1	SP-CW-90	Proper	Unknown
VI-T	123	Comb	Severe	Head-on	TCB	1	Stop	Proper	Unknown
VI-T	124	Auto-L	Severe	SS	TCB	5	Rb-R	Improper	Heavy
VI-T	125	Comb	Severe	Head-on	TCB	1	Ramp	DNA	Heavy
II	062	Auto-S	Severe	Head-on	Gore	1	Stop	Proper	Heavy
II	064	Comb	Severe	Head-on	BP	1	Stop	DNA	Extensive
II	065	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Heavy
II	067	Auto-L	Severe	Angle	Gore	1	Stop	Improper	Unknown
II	089	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Moderate
II	090	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Heavy
II	091	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Heavy
II	092	Auto-L	Severe	Head-on	Gore	1	SP-CW-90	Proper	Heavy
II	093	Auto-L	Severe	Head-on	Gore	1	Stop	Proper	Heavy
II	094	Auto-L	Severe	Head-on	Gore	1	Rb-L	Improper	Heavy
V	042	Auto-L	Severe	Head-on	Gore	1	Unknown	Proper	Heavy
V	043	Auto-L	Severe	Head-on	Gore	1	Unknown	Proper	Unknown

Note: Refer to Table 4 for definition of variable categories.
[a]Unknown. [b]Does not apply; crash cushions are not designed to attenuate impacts of large or heavy trucks.

Vehicle Size and Impact Severity

Information concerning vehicle size and impact severity is presented in Table 6. Impact severity for crash cushion accidents is high; 68 percent of the impacts are termed severe. If the less severe toll booth accidents are excluded, 85 percent of the remaining collisions are rated as severe. This is reflected in the vehicle damage; 66 percent of the impacts result in disabling vehicle damage. This percentage is increased to 86 percent if toll booth accidents are excluded.

The percentage of accidents involving an injury was high (38 percent), as would be expected. The proportion involving either a fatality or incapacitating (severe) injury was 16 percent. When toll booth accidents are excluded, the percentage of injury accidents increases to 46 and fatal or severe injury accidents to 19. Although these percentages are high, they are substantially lower than those determined for accidents involving a breakaway-cable-terminal (BCT) guardrail end treatment. In BCT accidents, the proportion of injury-producing accidents was determined to be 71 percent, whereas 29 percent resulted in a fatality or severe injury (8). This comparison illustrates the better performance of a crash cushion versus a BCT end treatment.

There were four fatal accidents involving crash cushions. Three involved a Hi-Dro cell crash cushion and one involved a Hi-Dro cell cluster. One involved a head-on collision of a large car with a Hi-Dro cell crash cushion in a gore. The car rolled over after impact, partially ejecting the driver. The second involved an angle collision of a large truck with a crash cushion in a gore. The crash cushion was destroyed because it was not designed for a high-speed impact with such a large vehicle. The truck continued on and the cab eventually vaulted over a bridge railing. The third fatal accident occurred when a van sideswiped a crash cushion and then overturned. The fatal accident involving the Hi-Dro cell cluster occurred when a large car hit the cluster head-on, knocked the cluster from its brace, and hit the abutment in front of the toll booth. The percentage of injury accidents was lower for trucks (26 percent) compared with large cars (41 percent). There were only nine small cars in the sample, but the percentage of injury accidents involving these vehicles was substantially higher (67 percent) than that for either trucks or large cars.

Cost-Effectiveness Analysis

In order to determine the cost-effectiveness of crash cushion installations in Kentucky, an analysis

TABLE 4 Description of Variable Categories

Variable	Category	Description
Vehicle size	Auto-L	Full or mid-size passenger car, full-sized pickup truck, van
	Auto-S	Compact or subcompact car, small pickup truck
	Auto-U	Automobile, unknown size
	SUT	Single-unit truck (two-axle, six tires or larger)
	Comb	Combination tractor and semitrailer or full trailer
Impact severity	Severe	Impact sufficient to cause heavy or extensive damage to crash cushion, disabling damage to vehicle, and/or fatal or incapacitating injury (injury severity 1 or 2)
	Nonsevere	Functional or nonfunctional to vehicle, slight or moderate damage to crash cushion, and/or nonincapacitating, possible, or no injury (injury severity 3, 4, or 5)
Type of impact	Head-on	At a shallow angle (15 degrees or less) with front end of vehicle
	Angle	At a moderate or sharp angle (16 degrees or greater) with front, right front, or left front of vehicle
	BSD	Broadside, impact at a shallow angle (15 degrees or less) with left or right side of vehicle
	SS	Sideswipe, impact to side of crash cushion with side of vehicle
	Unknown	Cannot be determined from available data
Injury severity	1	Fatal
	2	Incapacitating injury
	3	Nonincapacitating injury
	4	Possible injury
	5	No injury
Vehicle action after impact	Stop	Stopped by crash cushion
	SP-CW-D	Spun clockwise D degrees
	SP-CCW-D	Spun counterclockwise D degrees
	Over	Overturned
	Ramp	Ramped
	RB-L	Rebounded left
	RB-R	Rebounded right
	Cont	Continued in same direction
Crash cushion performance	Proper	Crash cushion performed as designed; impact energy fully attenuated in head-on, broadside, and angle collisions; for sideswipe impacts, vehicle redirected at a shallow angle back into adjacent traffic lane
	Improper	Performance other than as designed
	DNA	Does not apply
Crash cushion damage	Slight	Damage insufficient to affect performance should crash cushion be struck again before repairs are made
	Moderate	Up to 50 percent damage
	Heavy	Between 50 and 100 percent damage; rendered useless
	Extensive	Total destruction of crash cushion in addition to damage to protected structure behind crash cushion
Vehicle damage	1	No damage
	2	Nonfunctional damage
	3	Functional damage
	4	Disabling damage
Crash cushion contact area	End	End of crash cushion
	Side	Side of crash cushion
Crash cushion placement	Toll	Protecting toll booth at toll plaza
	Gore	Area between roadway split
	BP	Protecting median bridge pier
	CMB	Terminating concrete median barrier
	TCB	Terminating temporary concrete barrier in construction zone
Initial vehicle contact area	1	Front
	2	Right front
	3	Right side
	4	Left front
	5	Left side
	6	Right side of trailer
	7	Left side of trailer
	8	Bottom of trailer
Vehicle make	AMC	American Motors
	Buick	Buick
	Chev	Chevrolet
	Dodge	Dodge
	Ford	Ford
	Frtln	Freightliner
	GMC	General Motors
	Intl	International
	Kenw	Kenworth
	Linc	Lincoln
	Mack	Mack
	Merc	Mercury
	Olds	Oldsmobile
	Pblt	Peterbilt
	Plym	Plymouth
	Pont	Pontiac
	Toyo	Toyota
	Volks	Volkswagen
	White	White
	Dia	Diamond
Vehicle style	2-Dr-Sd	2-door sedan
	4-Dr-Sd	4-door sedan
	SW	Station wagon
	PU	Pickup
	SD	Sedan
	Semi	Combination tractor and semitrailer
	Truck	Truck (single unit)
	Van	Van

TABLE 5 Detailed Analysis of Crash Cushion Performance

		Proper Performance		Improper Performance	
Variable	Category	No.	Percent	No.	Percent
Crash cushion type	IV				
	Cushion	18	86	3	14
	Cluster	38	95	2	5
	VI	21	75	7	25
	VI-T	15	83	3	17
	II	5	71	2	29
	V	2	100	0	0
Vehicle size	Auto-S	6	67	3	33
	Auto-L	51	88	7	12
	Auto-U	22	73	7	27
	Truck	20	100	0	0
Impact severity	Severe	61	79	16	21
	Nonsevere	37	97	1	3
Type of impact	Head-on	49	88	7	12
	Angle	22	81	5	19
	Broadside	4	80	1	20
	Sideswipe	21	88	3	12

Note: Refer to Table 4 for definition of variable categories.

was made that included installation costs, maintenance repair costs, and accident savings resulting from these installations. Installation costs were obtained, when available, from average unit bid prices prepared by the Kentucky Department of Highways. Additional installation cost summaries were obtained from other reports when data were not available for Kentucky (9,10). Average installation costs used for this analysis are presented in Table 7. Installation costs were tabulated for all crash cushions installed in Kentucky. Maintenance repair costs were available from repair forms used by Department of Highways employees responsible for repair of damaged crash cushions. As part of the arrangement with maintenance employees in each highway district, repair information was provided along with accident reports for collisions occurring during the study period. Repair costs were also tabulated for all data available since the first crash cushion installations in 1970. Values used for the 3-year analysis period were annual averages since installa-

TABLE 6 Vehicle Size and Impact Severity

Crash Cushion Type	Accident No.	Vehicle Year	Vehicle Make	Vehicle Style or Model	Vehicle Size	Impact Severity	Injury Severity	Vehicle Damage	Crash Cushion Damage
IV	001	76	Olds	Cutlass	Auto-L	Severe	5	4	Unknown
IV	002	76	Chev	2-Dr-Sd	Auto-U	Nonsevere	5,5,5	2	Slight
IV	003	77	Olds	Cutlass	Auto-L	Severe	4,5	4	Unknown
IV	004	81	Ford	PU	Auto-L	Severe	5	4	Unknown
IV	005	72	Chev	Truck	SUT	Nonsevere	5	1	Slight
IV	006	78	Ford	PU	Auto-L	Nonsevere	5	4	Unknown
IV	007	78	Intl	Semi	Comb	Nonsevere	5	1	Slight
IV	008	73	Olds	2-Dr-Sd	Auto-L	Nonsevere	5	2	Slight
IV	009	83	Pblt	Semi	Comb	Severe	5	2	Moderate
IV	010	67	Plym	4-Dr-Sd	Auto-L	Nonsevere	5	3	Slight
IV	011	77	Buick	2-Dr-Sd	Auto-U	Nonsevere	5	1	Unknown
IV	012	79	Pont	2-Dr-Sd	Auto-U	Nonsevere	4	3	Unknown
IV	013	72	Chev	Vega	Auto-S	Nonsevere	5	2	Slight
IV	014	79	Kenw	Semi	Comb	Severe	5	4	Slight
IV	015	79	Ford	PU	Auto-L	Severe	1	4	Extensive
IV	016	81	Chev	Chevette	Auto-S	Severe	3,3	4	Slight
IV	017	78	Intl	Semi	Comb	Nonsevere	5	2	Slight
IV	018	77	Linc	2-Dr-Sd	Auto-L	Nonsevere	5,5,5,5,5	2	Unknown
IV	019	79	Pont	2-Dr-Sd	Auto-U	Nonsevere	5	2	Slight
IV	020	71	Chev	El Camino	Auto-L	Nonsevere	5	4	Slight
IV	021	68	Dodge	2-Dr-Sd	Auto-L	Nonsevere	5	2	Slight
IV	022	75	Frtln	Semi	Comb	Nonsevere	5	2	Unknown
IV	023	73	Ford	Sd	Auto-L	Nonsevere	5	1	Unknown
IV	024	69	Frtln	Semi	Comb	Nonsevere	5,5	1	Slight
IV	025	77	Ford	SW	Auto-L	Nonsevere	5	1	Slight
IV	026	69	Dodge	Van	Auto-L	Severe	4	4	Moderate
IV	027	70	Pont	4-Dr-Sd	Auto-L	Nonsevere	5	3	Slight
IV	028	71	GMC	Semi	Comb	Nonsevere	5	2	Slight
IV	029	78	Chev	Sd	Auto-U	Severe	4	4	Moderate
IV	030	73	GMC	Semi	Comb	Nonsevere	5	1	Slight
IV	031	78	Merc	2-Dr-Sd	Auto-L	Severe	2,2	4	Heavy
IV	032	74	Dia	Semi	Comb	Nonsevere	5	2	Slight
IV	033	78	Pblt	Semi	Comb	Nonsevere	5,5	2	Unknown
IV	034	73	Buick	Sd	Auto-U	Nonsevere	5	2	Slight
IV	035	75	White	Semi	Comb	Nonsevere	5	2	Slight
IV	036	69	Olds	4-Dr-Sd	Auto-L	Severe	5	3	Moderate
IV	037	78	Ford	Semi	Comb	Nonsevere	5	2	Unknown
IV	038	78	Pblt	Semi	Comb	Nonsevere	5	1	Slight
IV	044	77	Intl	Semi	Comb	Severe	5	4	Unknown
IV	046	74	Linc	Sd	Auto-L	Unknown	5	3	Unknown
IV	047	79	Dodge	Sd	Auto-U	Nonsevere	4	2	Unknown
IV	048	74	Chev	2-Dr-Sd	Auto-L	Severe	5	4	Unknown
IV	049	73	Pont	Catalina	Auto-L	Severe	1	4	Extensive
IV	050	79	Ford	Sd	Auto-U	Nonsevere	5	2	Unknown
IV	051	79	Mack	Semi	Comb	Severe	1	4	Heavy
IV	052	81	Chev	Sd	Auto-U	Severe	2,3	4	Unknown
IV	053	79	GMC	Semi	Comb	Severe	2,5	4	Unknown
IV	054	75	Chev	Sd	Auto-L	Severe	2,5	3	Unknown
IV	055	71	GMC	Semi	Comb	Severe	5	4	Unknown
IV	056	73	Chev	Sd	Auto-L	Severe	3	4	Unknown
IV	060	80	Ford	T-Bird	Auto-L	Severe	5	4	Unknown
IV	095	74	Buick	2-Dr-Sd	Auto-L	Severe	2	4	Extensive
IV	096	76	Ford	Sd	Auto-U	Severe	5	4	Unknown
IV	097	–[a]	Frtln	Semi	Comb	Severe	5	4	Extensive
IV	099	76	Chev	2-Dr-Sd	Auto-U	Severe	3	4	Unknown

TABLE 6 continued

Crash Cushion Type	Accident No.	Vehicle Year	Vehicle Make	Vehicle Style or Model	Vehicle Size	Impact Severity	Injury Severity	Vehicle Damage	Crash Cushion Damage
IV	100	72	Pont	2-Dr-Sd	Auto-L	Severe	5,5	4	Unknown
IV	102	79	Dodge	Diplomat	Auto-L	Severe	4	4	Unknown
IV	103	-[a]	Olds	Sd	Auto-L	Severe	5,5	4	Unknown
IV	113	74	Ford	Van	Auto-L	Severe	1,3,3	4	Unknown
IV	114	76	Chev	Monte Carlo	Auto-L	Severe	4	4	Unknown
IV	119	79	Ford	Sd	Auto-U	Severe	2	4	Unknown
IV	120	79	Chev	PU	Auto-U	Severe	2	4	Unknown
IV	121	79	White	Semi	Comb	Severe	5	3	Heavy
VI	039	77	AMC	Pacer	Auto-L	Severe	5,5,5	4	Unknown
VI	040	75	Merc	2-Dr-Sd	Auto-L	Severe	5	4	Heavy
VI	045	84	Pblt	Semi	Comb	Severe	2	4	Heavy
VI	058	74	Chev	Unknown	Auto-U	Unknown	5	4	Unknown
VI	061	-[a]	Kenw	Semi	Comb	Severe	3	4	Extensive
VI	063	78	Ford	PU	Auto-U	Severe	5	4	Unknown
VI	066	80	Merc	Sd	Auto-U	Nonsevere	5	4	Unknown
VI	068	67	Ford	PU	Auto-L	Nonsevere	5	2	Slight
VI	069	76	Chev	Monza	Auto-S	Severe	2	4	Unknown
VI	070	68	Plym	2-Dr-Sd	Auto-L	Severe	4	4	Unknown
VI	071	77	Pont	2-Dr-Sd	Auto-U	Severe	4,4	4	Unknown
VI	072	77	Pont	2-Dr-Sd	Auto-U	Severe	5	4	Unknown
VI	073	76	Chev	2-Dr-Sd	Auto-U	Severe	2,4,5	4	Unknown
VI	074	79	Ford	2-Dr-Sd	Auto-U	Severe	3,3	4	Unknown
VI	075	75	Olds	2-Dr-Sd	Auto-U	Nonsevere	5,5,5,5	4	Unknown
VI	076	78	Ford	2-Dr-Sd	Auto-U	Nonsevere	5	2	Unknown
VI	077	77	Ford	Mustang	Auto-S	Nonsevere	4	2	Unknown
VI	078	78	Ford	Mustang	Auto-S	Severe	3	4	Unknown
VI	079	78	Merc	Cougar	Auto-L	Severe	4	3	Unknown
VI	080	80	Plym	2-Dr-Sd	Auto-U	Severe	5	4	Unknown
VI	081	77	Chev	Nova	Auto-L	Severe	4	4	Unknown
VI	082	75	Chev	2-Dr-Sd	Auto-U	Nonsevere	5	2	Unknown
VI	083	71	Chev	2-Dr-Sd	Auto-U	Severe	4	4	Unknown
VI	084	75	Dodge	PU	Auto-L	Severe	5	2	Heavy
VI	085	71	Ford	Truck	SUT	Nonsevere	5	3	Moderate
VI	086	70	VW	2-Dr-Sd	Auto-S	Severe	5	4	Moderate
VI	087	-[a]	Unknown	Unknown	Unknown	Unknown	-[a]	-[a]	Moderate
VI	088	-[a]	Unknown	Unknown	Unknown	Unknown	-[a]	-[a]	Moderate
VI	098	72	Ford	2-Dr-Sd	Auto-L	Severe	2,2	4	Unknown
VI	111	70	Olds	2-Dr-Sd	Auto-L	Severe	5	4	Unknown
VI	117	68	Ford	2-Dr-Sd	Auto-L	Severe	2	4	Unknown
VI	126	77	Ford	PU	Auto-L	Severe	5	4	Unknown
VI	127	76	Merc	Montego	Auto-L	Severe	5	4	Heavy
VI-T	041	80	Intl	Semi	Comb	Severe	3	4	Unknown
VI-T	057	79	Chev	PU	Auto-U	Severe	5	4	Unknown
VI-T	059	71	Ford	4-Dr-Sd	Auto-L	Severe	5	4	Unknown
VI-T	104	78	Ford	Pinto	Auto-S	Severe	2	4	Unknown
VI-T	105	78	Chev	Monte Carlo	Auto-L	Severe	4	4	Unknown
VI-T	106	78	Dodge	2-Dr-Sd	Auto-U	Severe	3	4	Unknown
VI-T	107	78	Chev	2-Dr-Sd	Auto-L	Severe	5,5,5,5	4	Unknown
VI-T	108	80	Toyo	2-Dr-Sd	Auto-S	Severe	4,5	4	Unknown
VI-T	109	77	Ford	LTD	Auto-L	Severe	5	4	Unknown
VI-T	110	77	Pont	2-Dr-Sd	Auto-L	Nonsevere	5,5	2	Unknown
VI-T	111	77	Chev	SW	Auto-L	Nonsevere	5	4	Unknown
VI-T	112	80	Chev	2-Dr-Sd	Auto-U	Severe	5	4	Unknown
VI-T	115	79	Pont	2-Dr-Sd	Auto-U	Severe	4	4	Extensive
VI-T	116	69	Dodge	Sd	Auto-L	Severe	5,5	4	Unknown
VI-T	118	73	Pont	2-Dr-Sd	Auto-L	Severe	5,5	4	Unknown
VI-T	122	69	Plym	4-Dr-Sd	Auto-L	Severe	2	4	Unknown
VI-T	123	78	Kenw	Semi	Comb	Severe	3,3	4	Unknown
VI-T	124	77	Ford	T-Bird	Auto-L	Severe	5,5,5,5,5	3	Extensive
VI-T	125	83	White	Semi	Comb	Severe	4	4	Extensive
II	062	75	Volks	2-Dr-Sd	Auto-S	Nonsevere	5	3	Unknown
II	064	78	Intl	Semi	Comb	Severe	5	4	Unknown
II	065	71	Ford	PU	Auto-L	Severe	5	4	Unknown
II	067	78	Chev	Sd	Auto-U	Severe	2	4	Unknown
II	089	62	Ford	2-Dr-Sd	Auto-L	Severe	5	4	Moderate
II	090	66	Olds	4-Dr-Sd	Auto-L	Severe	2	4	Heavy
II	091	66	Plym	Sd	Auto-L	Severe	5	4	Heavy
II	092	67	Chev	Sd	Auto-L	Severe	5	4	Heavy
II	093	64	Ford	Sd	Auto-L	Severe	5	3	Heavy
II	094	65	Chev	SW	Auto-L	Severe	3	4	Heavy
V	042	70	Chev	4-Dr-Sd	Auto-L	Severe	2	4	Unknown
V	043	72	Dodge	PU	Auto-L	Severe	5	4	Unknown

Note: Refer to Table 4 for definition of variable categories.
[a]Unknown.

tion. A summary of these average costs is included in Table 7.

Accident savings were determined by calculating the reductions in injuries that resulted because collisions were with crash cushions rather than with a fixed, non-energy-absorbing object such as a bridge abutment. For this analysis, accident data were summarized for collisions with bridge abutments during the period 1980 through 1982. There were 394 accidents of this type and the average cost per accident was calculated. The costs for each fatal ($200,000), each non-fatal-injury ($8,000), and each property-

TABLE 7 Summary of Installation and Repair Costs

Crash Cushion Type	Annualized Installation Cost[a] ($)	Avg Installation Cost ($)	Avg Repair Cost per Accident ($)
I and II	641	3,937	887
IV			
Cushion	3,225	19,824	392
Cluster	779	4,788	2,839
V	1,082	6,650	1,760
VI	1,968	12,098	1,886
VI-T	2,338	14,369	1,886

[a]Installation costs amortized over a 10-year period at a 10 percent rate.

damage-only ($1,090) accident were those reported by the National Safety Council for 1982 (11). The cost per accident for each reported collision involving a bridge abutment was found to be about $21,000. Similarly, the cost per accident for each collision with a crash cushion in 1980 through 1982 (95 accidents) was found to be about $11,000. Therefore, the saving per accident was determined to be $10,000. The total savings would be $950,000 over a 3-year period or an annual saving of about $317,000.

Installation costs were amortized over a 10-year period at a 10 percent interest rate and the annual costs were determined for each type of crash cushion. Average annual installation costs for the analysis period were determined to be $274,707. Total repair costs for the three-year period were $178,506, or $59,502 per year. The result was an average cost of approximately $334,000 per year. Comparing the average annual accident savings of $317,000 with the average annual cost of $334,000 yields a benefit/cost ratio of approximately 1.0. It should be noted that additional savings likely resulted in the form of reduced accident costs because of nonreported accidents involving crash cushions. In many cases, crash cushions are capable of absorbing an impact or redirecting a vehicle without disabling the vehicle. The result is reduced accident severity when this is compared with the consequences of impacting a rigid object such as a bridge abutment. However, these successful impacts were not included in the cost-effectiveness analysis because no accident report was filed.

Another approach to evaluate the cost-effectiveness of crash cushion installations is application of accident reduction factors obtained from a national survey conducted by the Kentucky Transportation Research Program as part of another study (12). Several states reported reduction factors for crash cushions and those reductions averaged approximately 75 percent for fatal accidents and 50 percent for injury accidents. When these factors are applied to the numbers of various types of crash cushion accidents for the period 1980 through 1982, the expected reduction in fatal and injury accidents may be estimated. If no crash cushions had been installed, there would have been 9 more fatal accidents and 36 more injury accidents expected as well as 45 fewer property-damage-only accidents. This would have resulted in an annual accident cost saving of approximately $680,000. With an average annual cost of about $334,000 for crash cushion installation and repair cost, the benefit/cost ratio would be about 2.0.

Therefore, the range of benefit/cost ratios for crash cushions would be from 1.0 to 2.0, depending on what approach is used to estimate the reduction in accidents. The conservative estimate of 1.0 was obtained when the severity of crash cushion accidents was compared with the severity of bridge abutment accidents in Kentucky. The higher benefit/cost ratio of 2.0 resulted when the severity of crash cushion accidents was compared with the reductions expected because crash cushions were installed.

CONCLUSIONS

An analysis of accidents involving crash cushions, which includes Hi-Dro cell, G.R.E.A.T., G.R.E.A.T.-T, sand-barrel, and steel-drum types, indicates that the crash cushions have been performing their function properly (85 percent proper performance). Vehicles have generally been stopped by the crash cushions. The instances of improper performance have generally involved either the rebounding of the vehicle into or across the adjacent roadway or the overturning of the vehicle. All the various types have performed well.

Accident severity was high but less than that for similar impacts into BCT guardrail end treatments (8). This illustrates the increase in impact attenuation of a crash cushion over a guardrail end treatment.

Results from the cost-effectiveness analysis show that crash cushion installations produce a benefit/cost ratio in the range of 1.0 to 2.0.

It is recommended that the use of crash cushions be continued at locations where they are cost-effective. Primary examples of these locations include (a) gore areas on elevated structures; (b) other gore areas where guardrail end treatments must be joined together; (c) bridge piers in narrow medians at high-speed, high-volume locations; and (d) the ends of concrete barrier walls. Any of the types studied could be used, depending on site geometrics.

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REFERENCES

1. Handbook of Highway Safety Design and Operating Practices. FHWA, U.S. Department of Transportation, 1978.
2. Fatal Accident Reporting System. National Center for Statistics and Analysis, National Highway Traffic Safety Administration, U.S. Department of Transportation, 1980.
3. Highway Design and Operational Practices Related to Highway Safety. AASHO, Washington, D.C., 1967.
4. J.G. Viner and C.M. Boyer. Accident Experience with Impact Attenuation Devices. Report FHWA-RD-73-71. FHWA, U.S. Department of Transportation, 1973.
5. T.J. Hirsch et al. Summary of Crash Cushion Experience in Texas--Four Hundred Collisions in Seven Years on One Hundred Thirty-Five Installations. Report 223-2F. Texas Transportation Institute, Texas A&M University, College Station, 1975.
6. J.G. Pigman, W.M. Seymour, and D.L. Cornette. Experimental Installations of Impact-Attenuating Devices. Report 359. Division of Research, Kentucky Department of Highways, Frankfort, 1973.

7. J.G. Pigman, K.R. Agent, and T. Creasey. Analysis of Accidents Involving Crash Cushions. Research Report UKTRP-84-17. Kentucky Transportation Research Program, University of Kentucky, Lexington, 1984.
8. J.G. Pigman, K.R. Agent, and T. Creasey. Analysis of Accidents Involving Breakaway-Cable-Terminal End Treatments. Research Report UKTRP-84-16. Kentucky Transportation Research Program, University of Kentucky, Lexington, 1984.
9. J.B. Rollins et al. Assessment of Techniques for Cost-Effectiveness of Highway Accident Countermeasures. Report FHWA-RD 79-53. Texas Transportation Institute, Texas A&M University, College Station, 1979.
10. R. Lizak. Impact Attenuators: A Market Analysis. National Aeronautics and Space Administration, 1978.
11. Estimating the Cost of Accidents. National Safety Council, Chicago, Ill., 1982.
12. Accident Reduction Factors Survey. Kentucky Transportation Research Program, University of Kentucky, Lexington, 1984.