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Foreword

Holloway et al. evaluated the performance of a 30-in. New Jersey safety-shape bridge rail to determine if a standard 32-in. New Jersey safety-shape bridge rail would provide satisfactory safety performance after a 2-in. overlay was placed on the adjacent bridge deck. The safety performance was found to be satisfactory. Pfeifer et al. evaluated the performance of the luminaire support 4-bolt breakaway slipbase design originally used in Utah and determined that it performed satisfactorily. Hirsch and Buth report on the development of an aesthetically pleasing, structurally sound bridge that met the safety evaluation criteria and should be safe for use on low-speed (45 mph) roadways. Guidry and Beason report on the development of a low-profile portable concrete barrier (PCB) for low-speed (45 mph) work zones. The PCB is only 20 in. high, which should increase driver visibility.

Lohrey describes the shop fabrication and installation of the Narrow Connecticut Impact-Attenuation System, which is designed to protect drivers from hitting narrow, rigid roadside features in width-restricted hazard areas. Mak et al. discuss a design that is suitable for use with wood-post guardrail systems over low-fill culverts. The design has no shallow embedment posts over the culvert. A nested W-beam rail is used to span the culvert. Ivey et al. review the collision performance, maintenance, costs, and field performance of most commercially available and widely implemented terminals or end treatments for guardrail installations to aid interested parties in selecting the most cost-effective terminals to meet specific needs. Mak et al. summarize the results of crash testing and evaluation of two W-beam guardrail-to-concrete safety-shaped bridge-rail transition designs: the integral end-post design and the separate end-post design. Crowley and Denman discuss the application and misapplication of highway safety appurtenances and propose a solution to the problem of misapplication. Ivey and Marek present the results of a research and compliance testing program to develop a low-cost, high-performance terminal for PCBs and permanent concrete median barriers.

Performance Level 2 Tests on the Missouri 30-in. New Jersey Safety-Shape Bridge Rail

JAMES C. HOLLOWAY, RONALD K. FALLER, BRIAN G. PFEIFER, EDWARD R. POST,* AND DAN E. DAVIDSON

Safety-shape bridge rails are substandard if they are less than 32 in. high, according to Section 2.7.1.2.2 of the AASHTO *Standard Specifications for Highway Bridges*. However, a substandard bridge rail may remain in operation if it passes a safety performance evaluation by full-scale crash testing. Therefore, Nebraska and Kansas pooled their efforts with Missouri to determine whether a 32-in. standard New Jersey safety-shape bridge rail would still provide a satisfactory safety performance if a 2-in. overlay were placed on the adjacent bridge deck. To evaluate the performance of this bridge rail, the Midwest Roadside Safety Facility conducted three full-scale vehicle crash tests on the Missouri 30-in. New Jersey safety-shape bridge rail. Test MS30-1 was conducted with an 18,011-lb single-unit straight truck at 16.1 degrees at 52.5 mph. Test MS30-2 was conducted with a 1,759-lb small automobile at 20.0 degrees at 62.5 mph. Test MS30-3 was conducted with a 5,460-lb pickup truck at 20.0 degrees at 63.5 mph. The test procedures were conducted and reported in accordance with the requirements in NCHRP Report 230. The tests were evaluated in accordance with the PL-2 safety criteria in the AASHTO *Guide Specifications for Bridge Railings*. The safety performance of the Missouri 30-in. New Jersey safety-shape bridge rail was found to be satisfactory according to the AASHTO PL-2 safety criteria.

FHWA currently considers a concrete safety-shape bridge rail substandard if it does not conform to a 32-in. minimum vertical height as stated in Section 2.7.1.2.2 of the AASHTO *Standard Specifications for Highway Bridges* (*1*), which states, "Concrete parapets designed with sloping faces intended to allow vehicles to ride up them under low angle contacts shall be at least 2 feet 8 inches in height." Therefore, a problem would be encountered when bridge decks with an attached 32-in. bridge rail required a 2-in. overlay.

In the past, when an overlay was to be placed on the roadway surface of a bridge deck, FHWA required that the bridge rail be modified so that it would remain in compliance with current specifications (i.e., increase the height of the bridge rail by retrofitting). However, the unmodified bridge rail may remain in operation if the bridge rail passes a safety performance evaluation by full-scale crash testing.

The Missouri Highway and Transportation Department and other highway departments across the Midwest have existing 32-in. standard New Jersey safety-shape bridge rails on decks that need to be resurfaced with a 2-in. concrete overlay. Therefore, Nebraska and Kansas pooled their efforts with Missouri to determine if a 32-in. standard New Jersey safety-shape bridge rail could have a 2-in. overlay placed on the adjacent bridge deck and still provide a satisfactory safety performance.

A safety performance evaluation was conducted on a 30-in. New Jersey safety-shape bridge rail according to test procedures in NCHRP Report 230 (*2*) and the PL-2 performance level evaluation criteria of AASHTO (*1*).

BRIDGE-RAIL DESIGN DETAILS

The installation consisted of a concrete New Jersey safety-shape bridge rail with an overall height of 30 in. and an overall length of 100 ft. The bridge-rail design details are shown in Figure 1, and photographs of the installation before impact

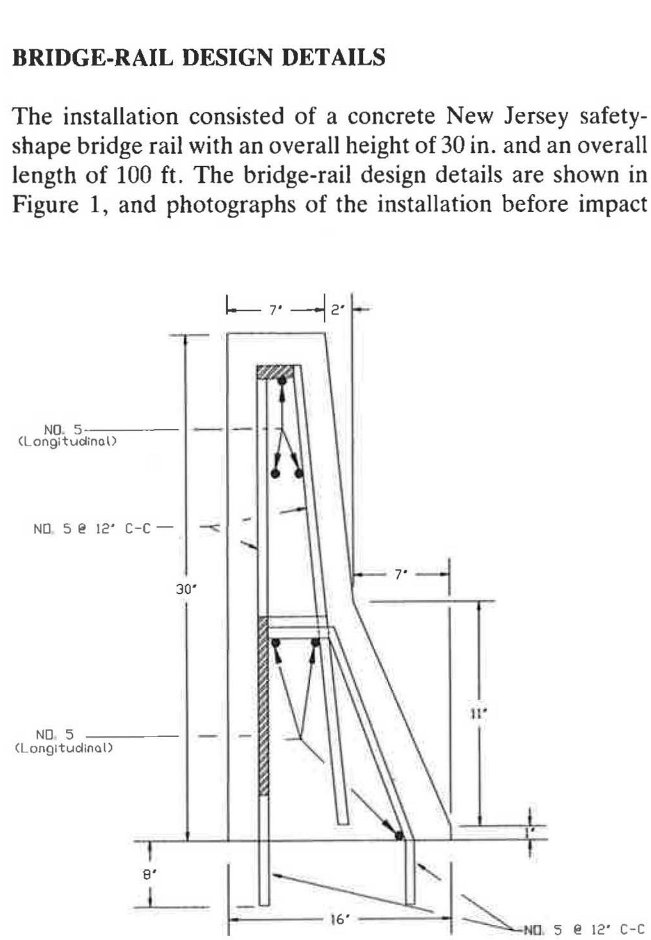


FIGURE 1 Bridge-rail design details.

*Deceased. J. C. Holloway, R. K. Faller, B. G. Pfeifer, and E. R. Post, Midwest Roadside Safety Facility, Civil Engineering Department, W348 Nebraska Hall, University of Nebraska–Lincoln, Lincoln, Nebr. 68588-0531. D. E. Davidson, Design Division, Missouri Highway and Transportation Department, P.O. Box 270, Jefferson City, Mo. 65102.

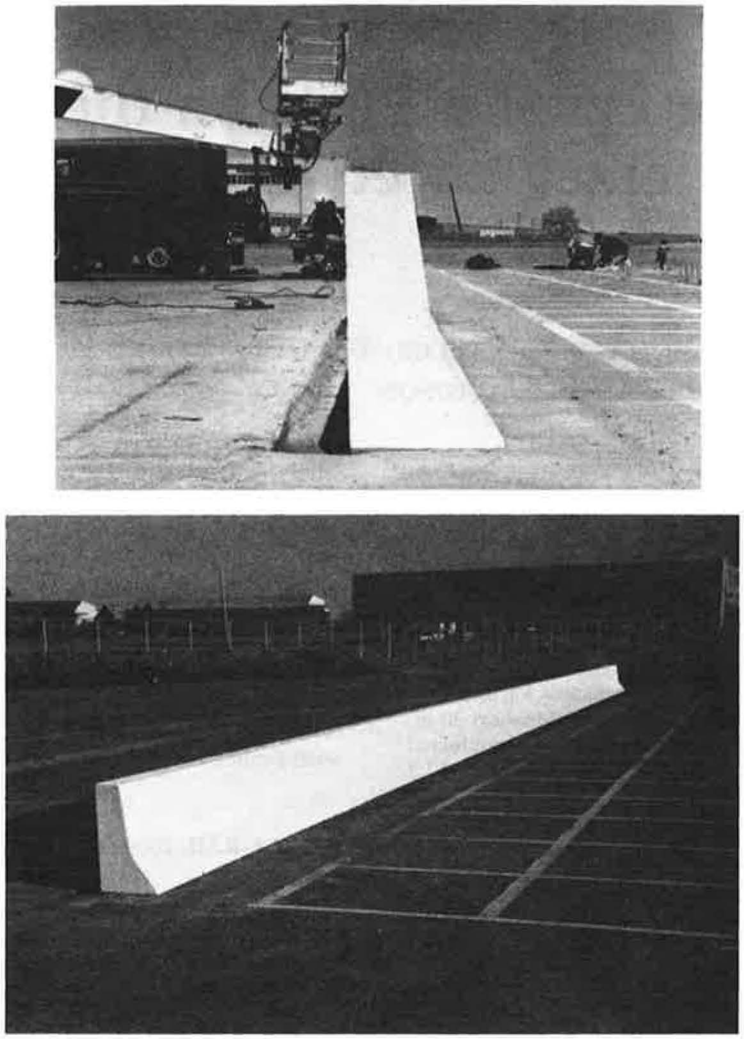


FIGURE 2 Missouri 30-in. New Jersey safety-shape bridge rail.

are shown in Figure 2. The 30-in. bridge rail was constructed by reducing the lower vertical face from 3 in. to 1 in. This construction procedure was accomplished by recessing standard 32-in. steel forms 2 in. below the existing concrete surface. The base width of the installation was 16.0 in., and the top width was 7.0 in.

The bridge rail was not constructed with a simulated concrete bridge deck because only the change in geometry caused by the reduced height was in question. Therefore, the bridge rail was attached to the existing concrete apron with two rows of No. 5 bent rebar spaced at 12-in. centers. The bars were rigidly attached to the apron with an epoxy grout adhesive and embedded 8 in. into the concrete apron surface. The reinforcement details are shown in Figure 1. Grade 60 reinforcing bars were used in all locations. The concrete compressive strength was approximately 6,000 psi.

TEST PARAMETERS

Three full-scale vehicle crash tests were conducted on the Missouri 30-in. New Jersey safety-shape bridge rail to satisfy the AASHTO (*1*) PL-2 performance level. The test vehicles are shown in Figure 3.

Test MS30-1 was conducted with a 18,011-lb single-unit truck at 16.1 degrees, 52.5 mph, and 35 ft from the upstream end. A detailed description of the ballasting procedure used for this test is shown in the test report (*3*). Test MS30-2 was conducted with a 1,759-lb small automobile at 20.0 degrees, 62.5 mph, and 30 ft from the upstream end. Test MS30-3 was conducted with a 5,460-lb pickup truck at 20 degrees, 63.5 mph, and 25 ft from the upstream end.

PERFORMANCE EVALUATION CRITERIA

The safety performance objective of a bridge rail is to reduce the number of deaths of and injuries to occupants of errant vehicles and to protect lives and property on, adjacent to, or below a bridge (*1*). In order to prevent or reduce the severity of such accidents, special attention should be given to four major areas: (a) railing strength to resist impact forces, (b) effective railing height, (c) shape of the face of the railing, and (d) deflection characteristics of the railing (*4*).

FIGURE 3 Test vehicles (*from top*): MS30-1, MS30-2, and MS30-3.

The major concerns about this installation were the reduced height and the change in the shape of the bridge rail face for an AASHTO PL-2 performance level. The other two items listed were not as critical because the tests were performed to evaluate the effects of the geometry change only. The rail must have adequate height in order to prevent vehicles from rolling over the railing. In the case of the small car and the pickup truck the rail must also prevent the vehicle from rolling onto its side away from the railing after redirection.

The performance evaluation criteria used to evaluate the three crash tests were taken from the AASHTO guide (*1*). The test conditions for the required test matrix are presented in Table 1. The safety performance of the bridge rail was evaluated according to three major factors: structural adequacy, occupant risk, and vehicle trajectory after collision. These three evaluation criteria are defined and explained in NCHRP Report 230. The vehicle damage was assessed by the traffic accident scale (TAD) (*5*) and the vehicle damage index (VDI) (*6*).

TEST RESULTS

Test MS30-1

After the initial impact with the bridge rail (35 ft from the upstream end), the right front corner crushed inward, causing the right front fender to ride along the top of the rail. At 0.15 sec after impact, the cab began to ride up the rail, rolling in a counterclockwise direction. At 0.30 sec, the front axle broke away from the frame on the right side and rotated inward underneath the truck. At this time, the cab had a roll angle of approximately 30 degrees counterclockwise, and the box remained level with the bridge rail. The front of the truck extended over the top as it traveled longitudinally along the bridge rail, leaving the front axle assembly on the traffic side of the rail.

At 0.42 sec, the cab began to rotate in the opposite direction (clockwise), with a clockwise yaw motion occurring simultaneously. The box began its clockwise roll motion at the same time. The combined effects of both the clockwise roll and yaw motions caused the rear end to uplift. This yaw motion

TABLE 1 Crash Test Conditions and Evaluation Criteria

Guidelines	Performance Level	Appurtenance	Test Vehicle	Impact Conditions		Evaluation Criteria[1]	
				Speed (mph)	Angle (deg)	Required	Desirable
AASHTO	PL-2	Bridge Rail	Small Automobile	60	20	3.a,b,c,d,g	3. e,f,h
AASHTO	PL-2	Bridge Rail	Pickup Truck	60	20	3. a,b,c,d	3. e,f,g,h
AASHTO	PL-2	Bridge Rail	Medium Single Unit Truck	50	15	3. a,b,c	3. d,e,f,h

[1] - Evaluation criteria explained elsewhere (1).

continued for the remaining length of the rail. During the clockwise roll motion, the cab became level at 0.54 sec, and continued in a clockwise roll motion. At 0.66 sec, the cab and box were rolling in the same direction toward the rail.

At 1.0 sec after impact the entire vehicle (cab and box) had a continuing clockwise roll motion. Coinciding with this roll was positive yawing motion. At 1.04 sec the roll motion of the cab was constant (i.e., the roll angle was not increasing). At approximately 1.10 sec the cab began a sudden redirection in roll motion. This time also signified the front of the cab reaching the end of the rail. At 1.16 sec, as the vehicle exited the bridge rail, the cab was experiencing counterclockwise roll, and the same positive yaw motion continued until 1.40 sec, which was the approximate time that the entire vehicle was free of the bridge rail.

A significant portion of the vehicle had extended over the bridge rail, although there was no physical evidence that the truck touched down behind the bridge rail. It is the authors' opinion that the vehicle would have still been contained had the installation length been longer because the vehicle had obtained a near stable position before reaching the end of the rail, and the positive yaw motion of the vehicle may have kept the vehicle from traveling over the rail. The vehicle may have come to rest on the rail or could have fallen back down onto the roadway.

The vehicle came to rest approximately 183 ft downstream from impact. The vehicle remained upright both during and after the collision. The vehicle trajectory after impact indicated no intrusion into the adjacent traffic lanes. The maximum vehicle rebound distance was 18 ft.

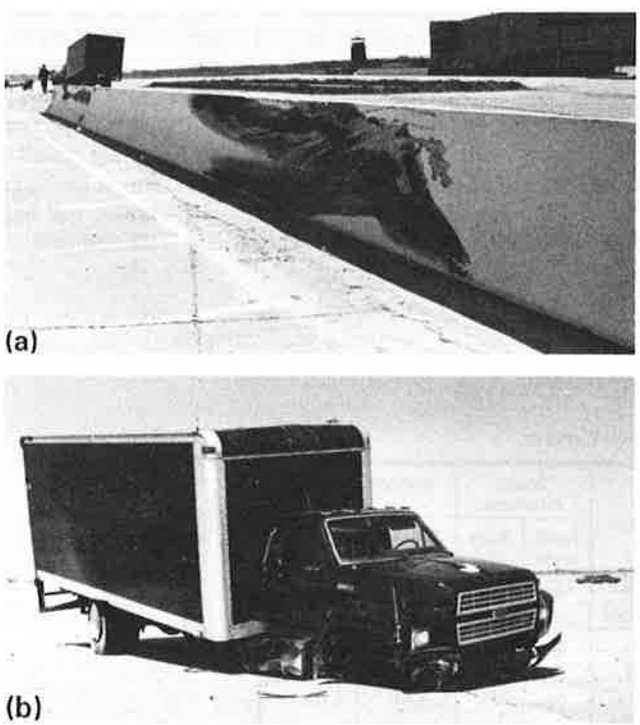


FIGURE 4 Damage to (*a*) bridge rail and (*b*) test vehicle, Test MS30-1.

Bridge rail damage is shown in Figure 4(*a*). Concrete spalling occurred at the point of impact as a result of the right front wheel crushing into the bridge rail. Spalling also occurred along the top of the rail as a result of the undercarriage of the vehicle sliding along the top of the rail. No visible lateral movement of the rail occurred as a result of the collision. Tire marks were visible on the face of the rail for a length of about 17 ft after impact.

Vehicle damage is shown in Figure 4(*b*). Most of the damage occurred to the undercarriage. The front axle assembly was disengaged from its original position. The right rear wheels were damaged, and the drive shaft was separated from the transmission. There was no intrusion or deformation of the occupant compartment.

The longitudinal occupant impact velocity (OIV) was determined to be 11.1 fps, and the lateral OIV was 9.7 fps. The highest 0.010-sec average occupant ridedown decelerations were 2.1 *g* (longitudinal) and 3.0 *g* (lateral). The results of the occupant risk, determined from accelerometer data, are summarized in Figure 5 and Table 2.

A summary of the test and sequential photographs are shown in Figure 5. Additional sequential photographs are shown in Figure 6. The performance of the bridge rail was determined to be satisfactory for this test.

Test MS30-2

After the initial impact with the bridge rail (30 ft from the upstream end), the right front corner crushed inward, causing the corner of the hood to extend over the top of the rail. Following the initial impact, a counterclockwise rolling motion away from the bridge rail occurred. The vehicle became parallel with the bridge rail 0.15 sec after impact and exited at 0.28 sec, which was approximately 20 ft from impact. The continued counterclockwise roll caused the vehicle to become completely airborne at 0.31 sec. It was airborne until the left front wheel touched down at 0.60 sec. The touchdown signified the maximum roll angle; this angle could not be measured, however, because of technical difficulties with the downstream camera. The touchdown also caused the vehicle to roll clockwise toward the rail. The vehicle became level at 0.94 sec. It came to rest approximately 230 ft downstream from impact. The vehicle remained upright both during and after the collision, although moderate roll motion occurred during the test. Vehicle trajectory after impact indicated minimal intrusion into the adjacent traffic lanes. The maximum vehicle rebound distance was 9.5 ft.

The minor bridge rail damage is shown in Figure 7(*a*). The marks on the bridge rail indicated that the vehicle was in contact for approximately 12 ft. No visible lateral movement of the bridge rail occurred.

Vehicle damage is shown in Figure 7(*b*). The damage was mainly to the right front corner, consisting of wheel, bumper, fender, and axle damage. Slight buckling of the roof was also apparent. No intrusion or deformation of the occupant compartment occurred.

The longitudinal OIV was determined to be 11.9 fps, and the lateral OIV was 26.5 fps. The highest occupant ridedown decelerations were 5.5 *g* (longitudinal) and 9.0 *g* (lateral). The results of the occupant risk, determined from film anal-

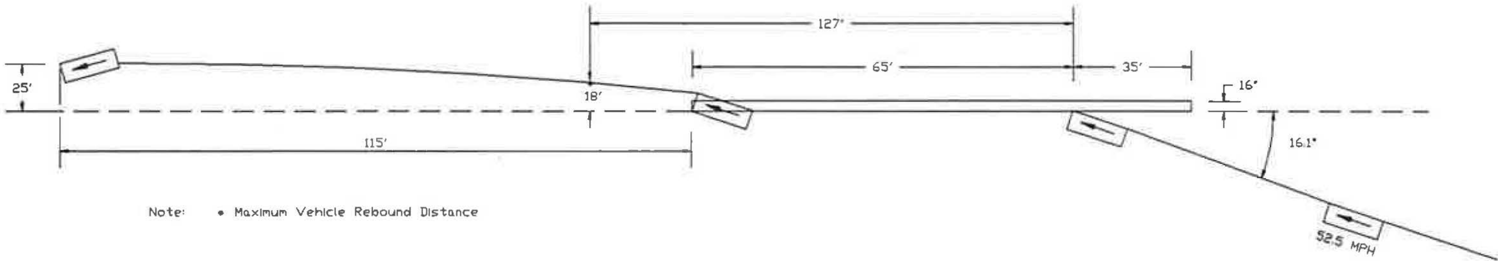
Impact

0.55 sec

1.09 sec

1.64 sec

2.19 sec



· Test Number	MS30-1
· Date	4/15/91
· Installation	30 in. N.J. Safety Shape
· Total Length	100 ft
· Concrete Bridge Rail	
Material	Ne.Special Mix (47-B)
Length	100 ft
Weight	340 lb/ft
Area	2.27 ft^2
Height	30 in.
Lower Vertical Face	1 in.
Middle Inclined Surface	
Length	10 in.
Inclination	55 deg
Upper Inclined Surface	
Length	19 in.
Inclination	84 deg
Base Width	16 in.
Top Width	7 in.

· Vehicle Model	1986 Ford F-700
· Vehicle Weight	
Test Inertia	18,011 lb
Gross Static	18,011 lb
· Vehicle Impact Speed	52.5 mph
· Vehicle Exit Speed	NA
· Vehicle Impact Angle	16.1 deg
· Vehicle Exit Angle	NA
· Vehicle Snagging	None
· Effective Coef. of Friction	NA
· Vehicle Stability	Marginal
· Occupant Impact Velocity	
Longitudinal	11.1 fps
Lateral	9.7 fps
· Occupant Ridedown Deceleration	
Longitudinal	2.1 g's
Lateral	3.0 g's
· Vehicle Damage	
TAD	1-RFQ-3
VDI	01RFWS1
· Vehicle Rebound Distance	18 ft
Bridge Rail Damage	Minor Spalling

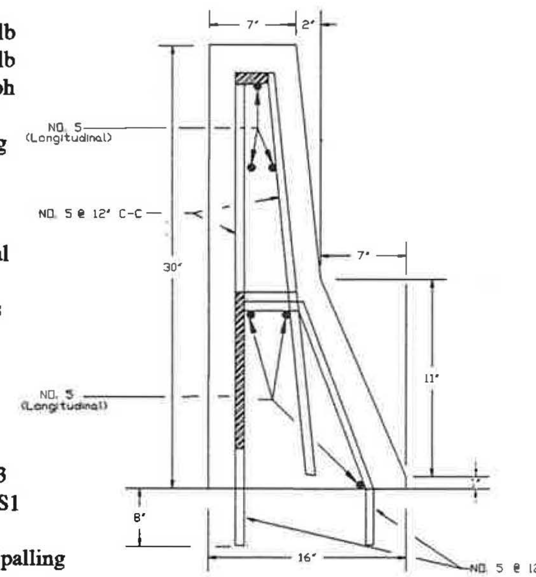


FIGURE 5 Summary and sequential photographs, Test MS30-1.

TABLE 2 Summary of Test Results

Test Item	Test MS30-1	Test MS30-2	Test MS30-3
Vehicle Weight (lb.)	18,011	1,759	5,460
Vehicle Impact Speed (mph)	52.5	62.5	63.5
Vehicle Exit Speed (mph)	NA	55.0	49.0
Vehicle Impact Angle (deg)	16.1	20.0	20.0
Vehicle Exit Angle (deg)	NA	6.6	6.0
Effective Coefficient of Friction	NA	0.11	0.37
Vehicle Rebound Distance (ft)	18.0	9.5	2.5
Vehicle Damage (TAD)	1-RFQ-3	1-RFQ-4	1-RFQ-4
Vehicle Damage (VDI)	01RFWS1	01RFES1	01RFES2
Occupant Impact Velocity (fps)			
Longitudinal	11.1	11.9	16.6
Lateral	9.7	26.5	14.3
Occupant Ridedown Decelerations (g's)			
Longitudinal	2.1	5.5	6.0
Lateral	3.0	9.0	6.6
Did Snagging Occur?	No	No	No

NA = Not Available

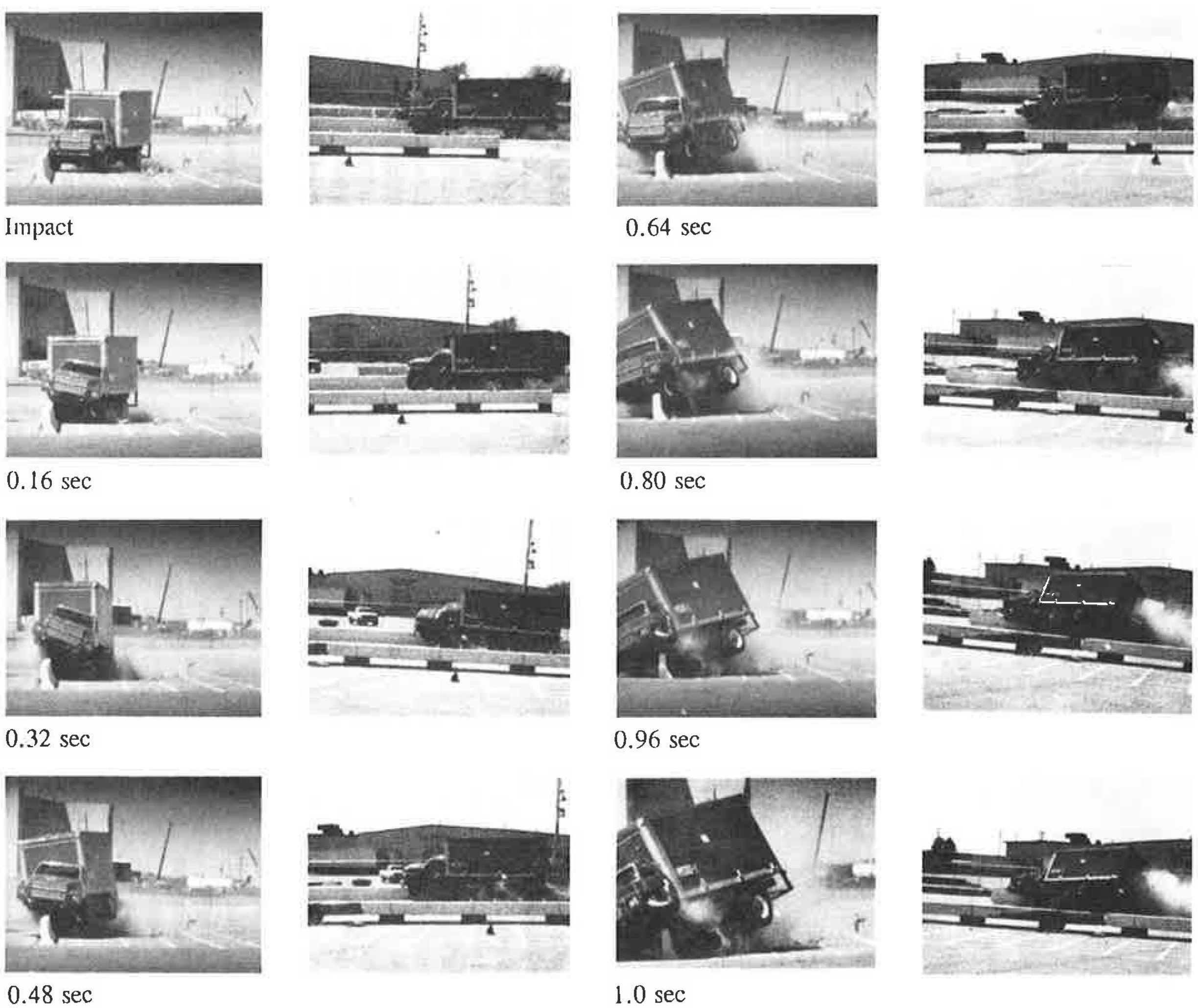
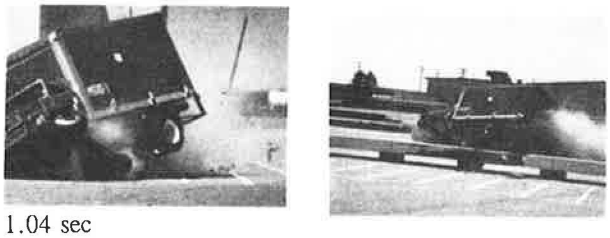
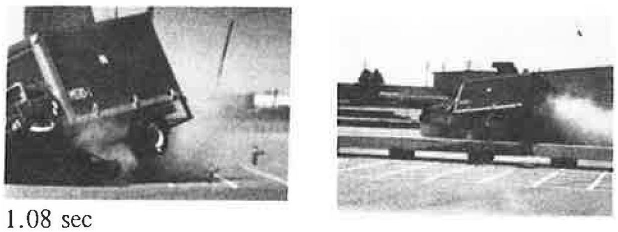


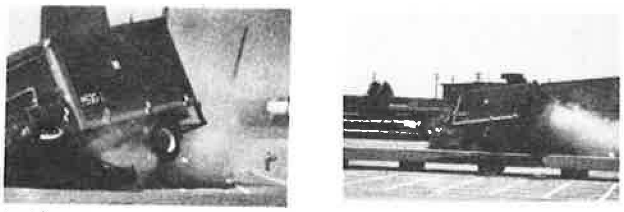
FIGURE 6 Parallel time sequential photographs, Test MS30-1 (*continued on next page*).



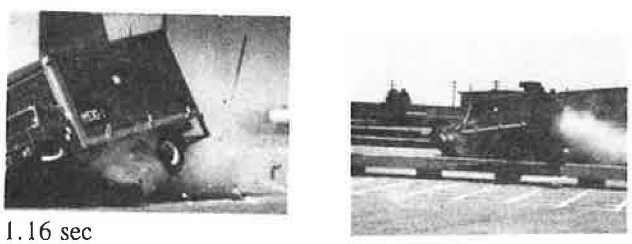
1.04 sec



1.08 sec



1.12 sec



1.16 sec

FIGURE 6 ***(continued).***

FIGURE 7 Damage to (*a*) bridge rail and (*b*) test vehicle, Test MS30-2.

ysis, are summarized in Figure 8, and Table 2. Because of technical difficulties in obtaining accelerometer data, the occupant risk values were determined from film analysis.

A summary of the test and sequential photographs are shown in Figure 8. Additional sequential photographs are shown in Figure 9. The performance of the bridge rail was determined to be satisfactory for this test.

Test MS30-3

After the initial impact with the bridge rail (25 ft from the upstream end), the right front corner of the truck was crushed inward. This maximum crushing distance was approximately 2 ft. At 0.13 sec after impact, the right front wheel began to climb up the rail. A parallel position with the bridge rail was obtained at 0.19 sec.

As the vehicle came out of the parallel position with the rail, the front wheels became airborne. At 0.49 sec, the left front wheel touched down, causing the vehicle to skid away from the rail. At 0.91 sec, the vehicle regained a parallel position with the bridge rail, having a lateral offset of approximately 5 ft. The vehicle came to rest approximately 203 ft downstream from the impact. The vehicle remained upright during and after the collision. The vehicle trajectory after impact indicated minimal intrusion into the adjacent traffic lanes. The maximum vehicle rebound distance was 5 ft.

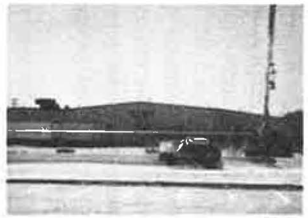
Bridge rail damage is shown in Figure 10(*a*). Damage was minimal. Tire marks and scrapes accounted for the majority of the damage. The marks on the rail were approximately 12 ft long. No visible lateral movement of the bridge rail occurred as a result of the collision.

Vehicle damage is shown in Figure 10(*b*). The damage was mainly to the right front corner of the vehicle. The passenger side door and rear wheel were also slightly damaged. The lower right corner of the windshield was also broken. There was no intrusion or deformation of the occupant compartment.

The longitudinal OIV was determined to be 16.6 fps and the lateral OIV was 14.2 fps. The highest 0.010-sec average occupant ridedown decelerations were 6.0 g (longitudinal) and 6.6 g (lateral). The results of the occupant risk, determined from accelerometer data, are summarized in Figure 11 and Table 2.

A summary of the test and sequential photographs are shown in Figure 11. Additional sequential photographs are shown

Impact

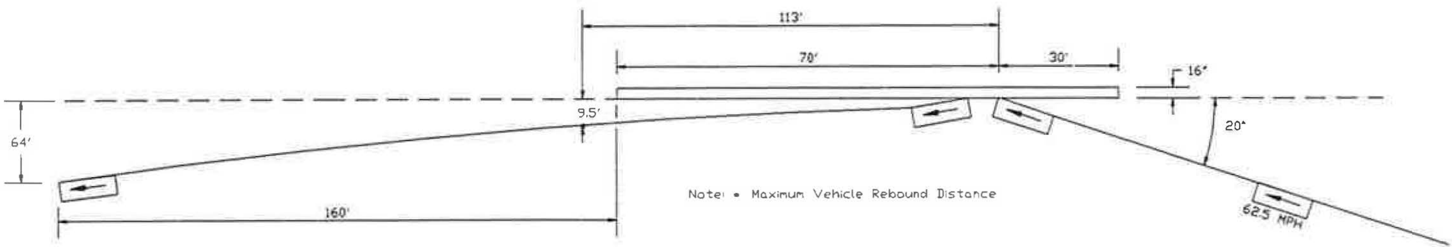


0.31 sec

0.63 sec

0.94 sec

1.25 sec



- Test Number MS30-2
- Date 5/1/91
- Installation . . . . 30 in. N.J. Safety Shape
- Total Length . . . . 100 ft
- Concrete Bridge Rail
  - Material . . . . Ne.Special Mix (47-B)
  - Length . . . . 100 ft
  - Weight . . . . 340 lb/ft
  - Area . . . . 2.27 ft^2
  - Height . . . . 30 in.
  - Lower Vertical Face . . . . 1 in.
  - Middle Inclined Surface
    - Length . . . . 10 in.
    - Inclination . . . . 55 deg
  - Upper Inclined Surface
    - Length . . . . 19 in.
    - Inclination . . . . 84 deg
  - Base Width . . . . 16 in.
  - Top Width . . . . 7 in.

- Vehicle Model . . . . 1984 Dodge Colt
- Vehicle Weight
  - Test Inertia . . . . 1,759 lb
  - Gross Static . . . . 1,759 lb
- Vehicle Impact Speed . . . . 62.5 mph
- Vehicle Exit Speed 55.0
- Vehicle Impact Angle . . . . 20.0 deg
- Vehicle Exit Angle . . . . 6.6 deg
- Vehicle Snagging . . . . None
- Effective Coef. of Friction 0.11
- Vehicle Stability . . . . Marginal
- Occupant Impact Velocity
  - Longitudinal . . . . 11.9 fps
  - Lateral . . . . 26.5 fps
- Occupant Ridedown Deceleration
  - Longitudinal . . . . 5.5 g's
  - Lateral . . . . 9.0 g's
- Vehicle Damage
  - TAD 1-RFQ-4
  - VDI 01RFES1
- Vehicle Rebound Distance . . . . . 9.5 ft
- Bridge Rail Damage . . . . Minor Spalling

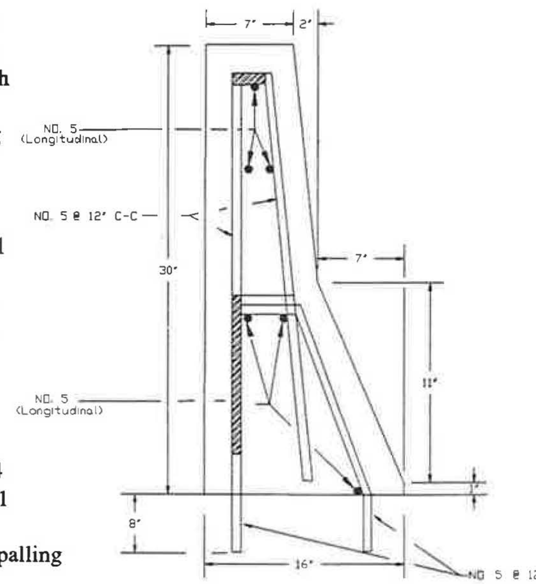


FIGURE 8 Summary and sequential photographs, Test MS30-2.

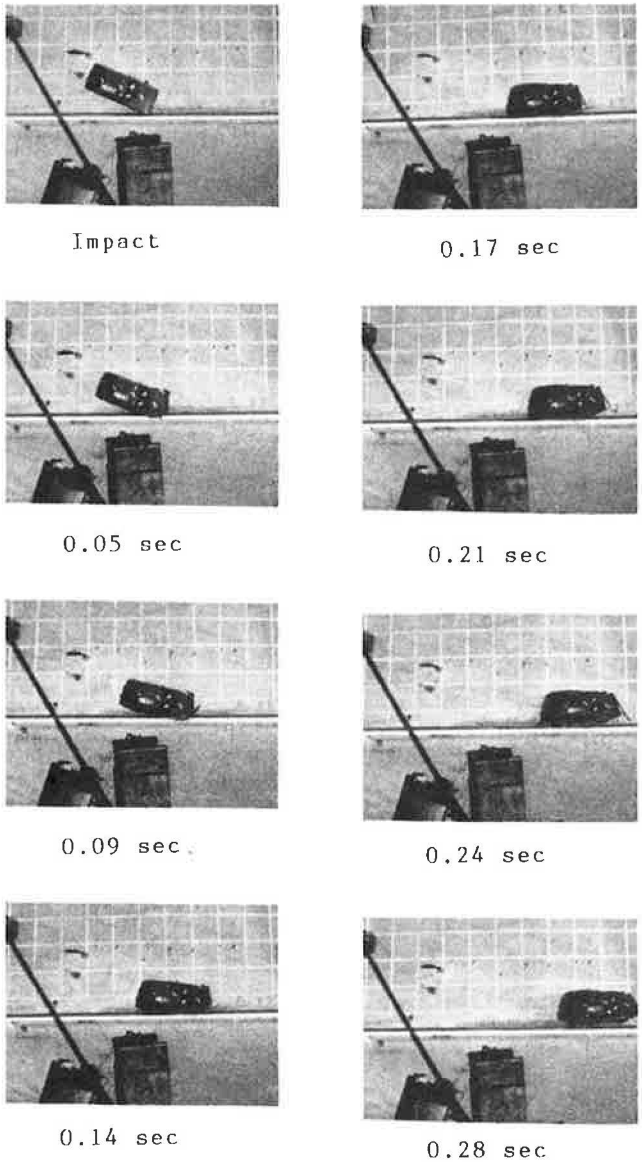


FIGURE 9 Overhead time sequential photographs, Test MS30-2.

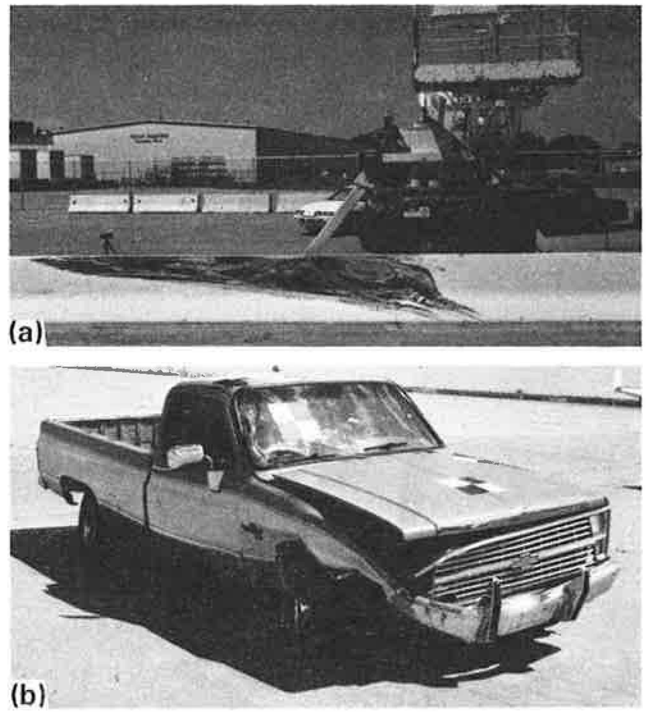


FIGURE 10 Damage to (*a*) bridge rail and (*b*) test vehicle, Test MS30-3.

in Figure 12. The performance of the bridge rail was determined to be satisfactory for this test.

CONCLUSIONS

The PL-2 performance level tests on the 30-in. New Jersey safety-shape bridge rail proved to be satisfactory according to the safety performance criteria given by AASHTO (*1*). The results of all three tests are summarized and presented in Table 2. The analysis of the tests revealed the following:

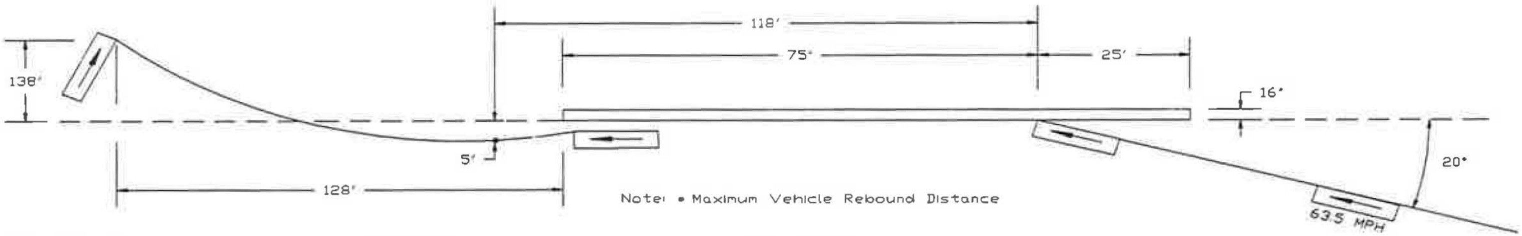
1. The bridge rail contained the vehicles without any visible lateral deflection, although a significant portion of the vehicle did protrude over the top of the bridge rail in Test MS30-1.
2. No detached elements or fragments penetrated the occupant compartments, and their integrity was maintained.
3. The vehicles remained upright both during and after impact, although moderate roll did occur in Test MS30-2.
4. The redirection capability of the bridge rail was determined to be satisfactory.
5. The occupant ridedown decelerations were determined to be satisfactory.
6. The OIVs were determined to be satisfactory, although the OIV for Test MS30-2 was 5 percent greater than the design limit but less than the threshold.
7. The vehicles' exit angles and rebound distances were determined to be satisfactory.

DISCUSSION OF RESULTS

Current practice in state highway departments is to use concrete safety-shape bridge rails with either the standard New Jersey safety shape or the F shape. The standard New Jersey safety shape consists of a 32-in. concrete parapet with a 3-in. lower vertical face. The height above the roadway surface to the slope break point is 13 in. The F shape consists of a 32-in.-high concrete parapet with a 3-in. lower vertical face and a slope break point of 10 in. The Missouri 30-in. New Jersey safety shape consists of a concrete parapet with a 1-in. lower vertical face, and a slope break point of 11 in., which is similar to that of the F shape. This has been shown to reduce vehicle roll. These three bridge rails are shown in Figure 13.

Past research results have shown that if the slope break point is higher than 13 in., the chances of vehicle rollover are increased, particularly for compact and subcompact automobiles (*7*). An example of this is the earlier General Motors

Impact | 0.40 sec | 0.78 sec | 1.17 sec | 1.52 sec



- Test Number MS30-3
- Date 6/14/91
- Installation . . . . 30 in. N.J. Safety Shape
- Total Length . . . . 100 ft
- Concrete Bridge Rail
  - Material . . . . Ne.Special Mix (47-B)
  - Length . . . . 100 ft
  - Weight . . . . 340 lb/ft
  - Area . . . . 2.27 ft^2
  - Height . . . . 30 in.
  - Lower Vertical Face . . . . 1 in.
  - Middle Inclined Surface
    - Length . . . . 10 in.
    - Inclination . . . . 55 deg
  - Upper Inclined Surface
    - Length . . . . 19 in.
    - Inclination . . . . 84 deg
  - Base Width . . . . 16 in.
  - Top Width . . . . 7 in.

- Vehicle Model . . . . 1984 Chevy Silverado
- Vehicle Weight
  - Test Inertia . . . . 5,460 lb
  - Gross Static . . . . 5,460 lb
- Vehicle Impact Speed . . . . 63.5 mph
- Vehicle Exit Speed . . . . 49.1 mph
- Vehicle Impact Angle . . . . 20.0 deg
- Vehicle Exit Angle . . . . 6.0 deg
- Vehicle Snagging . . . . None
- Effective Coef. of Friction 0.37
- Vehicle Stability . . . . Satisfactory
- Occupant Impact Velocity
  - Longitudinal . . . . 16.6 fps
  - Lateral . . . . 14.2 fps
- Occupant Ridedown Deceleration
  - Longitudinal . . . . 6.0 g's
  - Lateral . . . . 6.6 g's
- Vehicle Damage
  - TAD 1-RFQ-4
  - VDI 01RFES2
- Vehicle Rebound Distance . . . . . 5 ft
- Bridge Rail Damage . . . . Minor Spalling

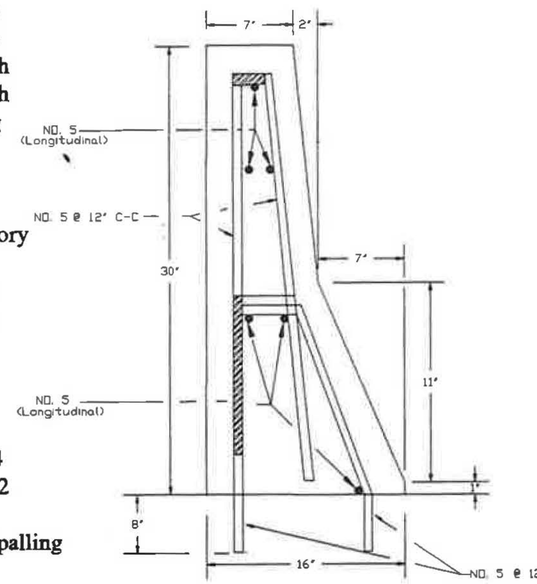


FIGURE 11 Summary and sequential photographs, Test MS30-3.

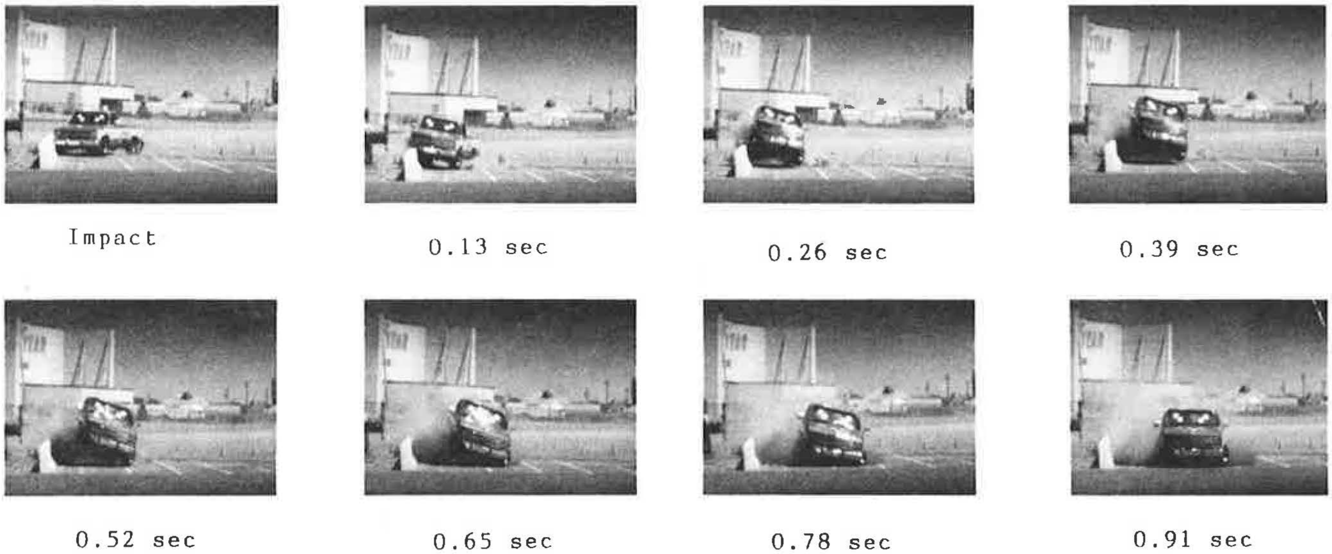


FIGURE 12 Parallel time sequential photographs, Test MS30-3.

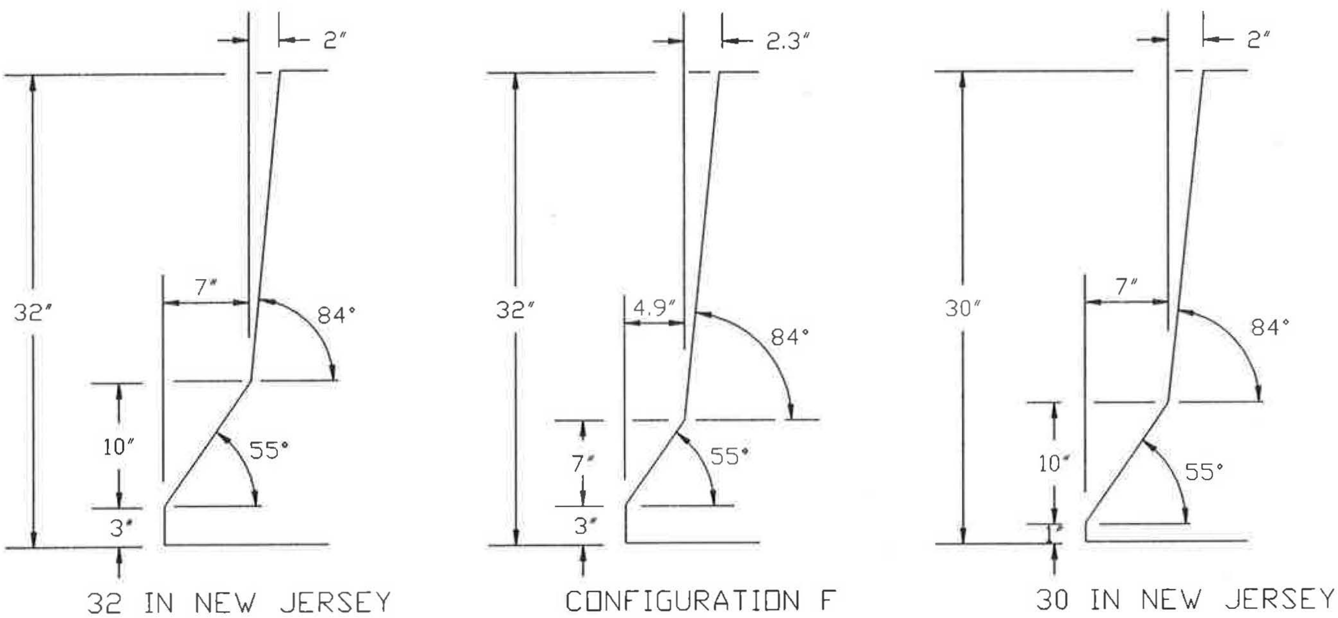


FIGURE 13 Geometric properties of safety-shape bridge rails.

shape, having a slope break point 15 in. above the roadway surface. This system is no longer recommended for use.

To help establish the validity of the 30-in. safety-shape bridge rail, a comparison of safety performance evaluations is presented against other AASHTO PL-2 safety-shape bridge rails (*8,9*). The tests were conducted on the 32-in. standard New Jersey safety-shape bridge rail and the 32-in. F-shape bridge rail. The comparison is shown in Table 3. It was evident that the safety performance results for these shapes and the 30-in. New Jersey safety shape provided similar results. One difference was that the 18,000-lb vehicle test on the 32-in. New Jersey safety shape (Test 7069-12) (*8*) resulted in vehicle rollover, whereas the 18,000-lb tests on the F safety shape (Test 7069-4) (*9*) and the 30-in. New Jersey safety shape (Test MS30-1) (*3*) did not result in vehicle rollovers. This may be explained by the differences in the geometry of the bridge railings, the make and model of the trucks, or even the location of impact.

From the four AASHTO PL-2 bridge railings reported in *Transportation Research Record 1258* (*8*), it was stated that test results indicate that a 32-in. vertical height would be a preferred minimum height. This statement was based upon the fact that only 32-in. bridge railings were tested. However, the authors did recognize that some innovative designs of a lesser height might be able to function in a suitable manner, but must be subjected to full-scale crash testing in order to prove their satisfactory performance. The adequacy of the 30-in. bridge rail was verified by full-scale crash testing. It was

TABLE 3 Comparison of PL-2 Bridge Rail Test Results

TEST ITEMS	TEST INSTALLATION AND TESTING FACILITY								
	32 in. New Jersey Safety Shape (Texas Transportation Institute) (8)			32 in. F-Shape (Texas Transportation Institute) (8)			30 in. New Jersey Safety Shape (Midwest Roadside Safety Facility) (3)		
	Tests Number and Dates			Test Numbers and Dates			Test Numbers and Dates		
	7069-12	3115-3[1]	7069-14	7069-11	7069-3	7069-4	MS30-1	MS30-2	MS30-3
	6/22/88	4/29/81	8/11/88	3/30/88	7/28/87	7/30/87	4/15/91	5/1/91	6/14/91
Vehicle (Year & Model)	1982 GMC SU Truck	1974 Honda	1981 Chevy PU	1982 Ford 7000 SUT	1980 Honda	1981 Chevy PU	1986 Ford F-700 SUT	1984 Dodge Colt	1984 Chevy PU
Vehicle Weight (Gross Static) lb.	18,000	1,968	5,724	18,000	1,966	5,780	18,011	1,759	5,460
Vehicle Impact Speed (mph)	51.6	61.3	57.7	52.1	60.1	65.4	52.5	62.5	63.5
Vehicle Impact Angle (deg)	15.5	20	20.6	14.8	21.4	20.4	16.1	20.0	20.0
Vehicle Exit Speed (mph)	NA	NA	35.8	NA	53.0	56.9	NA	55.0	49.0
Vehicle Exit Angle (deg)	2.0	7.0	.09	0.0	6.2	7.4	NA	6.6	6.0
Effective Coefficient of Friction	NA	NA	0.83	0.12	0.33	0.31	NA	0.11	0.37
Occupant Impact Velocity (fps)									
Longitudinal	13.4	NA	17.8	5.7	19.0	12.5	11.1	11.9	16.6
Lateral	10.2	NA	18.7	8.2	23.7	24.1	9.7	26.5	14.2
Occupant Ridedown Decelerations (g's)									
Longitudinal	3.0	4.4[2]	5.1	1.3	2.1	1.2	2.1	5.5	6.0
Lateral	4.9	10.6[2]	9.2	5.4	4.9	5.9	3.0	9.0	6.6

NA - Not Available
SUT - Single Unit Truck
PU - Pickup
[1] - Testing Performed at Dynamic Science, Inc. (9)
[2] - Maximum Deceleration (50 msec avg.)

the judgment of the authors that the 30-in. standard New Jersey safety-shape bridge rail met the AASHTO PL-2 performance level evaluation criteria. However this does not justify the reduction of heights for standard New Jersey or F-shape bridge railings. To do so would give up a margin of safety for little cost savings and would reduce the potential for safe performance after future overlays.

ACKNOWLEDGMENTS

This study was conducted under a cooperative program between the Midwest Roadside Safety Facility (MwRSF), the Missouri Department of Transportation, the Nebraska Department of Transportation, the Kansas Department of Transportation, the Midwest Transportation Center, and the University of Nebraska Center for Infrastructure Research. The crash tests were conducted by personnel at MwRSF under the direction of the late E. R. Post, to whom this paper is dedicated. This study was one of the last projects in which he was an active participant before his death.

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Full-Scale Crash Tests on a Luminaire Support 4-Bolt Slipbase Design

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The breakaway luminaire support concept has existed for many years and has proven to be an effective safety device. The 4-bolt breakaway slipbase design was originally used in Utah and has been used successfully in 20 years of field implementation. The state transportation departments of Idaho, Montana, Nevada, Utah, and Wyoming requested that the 4-bolt slipbase system be evaluated for possible use on Federal-aid highway projects. Two full-scale vehicle crash tests were performed by the Midwest Roadside Safety Facility to evaluate the system. Both tests had a centerline impact location. Test USBLM-1 was conducted with a 1,800-lb vehicle traveling at 15 mph, and Test USBLM-2 was conducted with a 1,800-lb vehicle traveling at 57.5 mph. The full-scale vehicle crash tests were evaluated according to the performance criteria presented in NCHRP Report 230 and the 1985 AASHTO specifications for structural supports. The Code of Federal Regulations, in which FHWA slightly modified the AASHTO requirement for maximum allowable change in velocity, was also used in the evaluation (23 CFR 625). The tests easily met all of the performance criteria mentioned above. Therefore, the safety performance of the 4-bolt breakaway slipbase design was determined to be satisfactory.

A breakaway device is a mechanism that fractures or yields when struck by a vehicle but is strong enough to withstand static and wind loads. The concept of a breakaway mechanism for highway lighting supports has existed for many years, and extensive testing has been conducted to determine the relative safety of different breakaway designs (*1–7*).

The 3-bolt slipbase design appears to be the most widely used system, and it has undergone extensive testing under a comprehensive program at the Federal Outdoor Impact Laboratory (unpublished data, L.A. Staron, FHWA). However, a 4-bolt slipbase design has been used in Utah for nearly 20 years. During those 20 years of field implementation, the design has proven to be so successful that, in most cases, motorists were able to drive away from the scene of the accident.

The objective of this study was to evaluate the safety performance of the 4-bolt slipbase design for possible use on Federal-aid projects. Two full-scale vehicle crash tests were conducted (*8*) in accordance with the guidelines presented in NCHRP Report 230 (*9*), AASHTO standard specifications (*10*), and the Code of Federal Regulations (*11*), in which the 1985 AASHTO specifications are updated.

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4-BOLT SLIPBASE DESIGN DETAILS

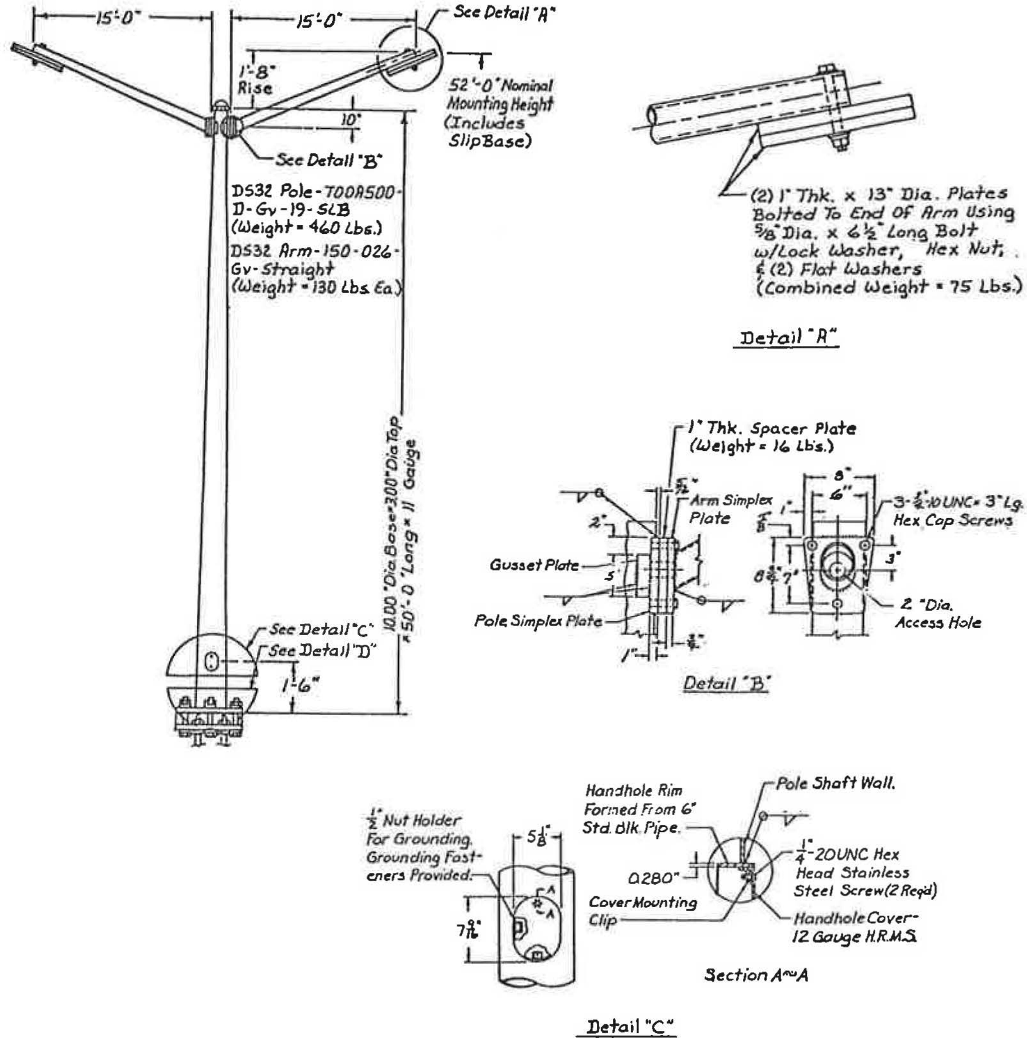
The luminaire support 4-bolt slipbase design details are shown in Figure 1, and photographs of the design are shown in Figure 2. The test article consisted of three major structural components: the luminaire support pole, the two mast arms, and the permanent lower slipbase assembly.

The maximum mounting height of the luminaire support pole was 52 ft from the ground to the top of the mast arms. The height to the top of the luminaire pole was 50 ft 4 in. from the ground. The permanent lower slipbase assembly had a stub height (the height remaining after the pole breaks away) of 4 in. The fully assembled test article is shown in Figure 2.

In actual field installations, the permanent lower slipbase assembly is held in place by four cast-in-place 1-in.-diameter galvanized ASTM A449 threaded rods. However, for testing purposes, this assembly was held in place with four 1-in.-diameter × 12-in.-long galvanized ASTM A449 threaded rods doweled into the existing concrete apron with a high-modulus, high-strength epoxy. The embedment depth of the threaded rods was 8.25 in., leaving 3.75 in. extending above the existing concrete surface. The bottom and top surfaces of the permanent lower slipbase assembly were mounted above the existing concrete apron at heights of 1.5 in. and 4 in., respectively. The permanent lower slipbase assembly was manufactured with steel that had a minimum yield strength of 36 ksi. The steel assembly was hot-dipped galvanized in accordance with ASTM standards (ASTM A123). A concrete grout mix was placed below the lower edge of the permanent lower slipbase assembly.

The 50-ft luminaire support was mounted on the permanent lower slipbase assembly with four 1-in.-diameter ASTM A325 slip bolts. The high-strength slip bolts, nuts, and washers were electroplated cadmium in accordance with ASTM standards (ASTM A165). This was used instead of hot-dip galvanizing because it provided a smoother finish, resulting in a much more consistent torque-versus-tension relationship. This also eliminated the need for lubricating the slip bolts. The four slip bolts were torqued to 80 lbf-ft, then released and retorqued to 70 lbf-ft. The Utah Department of Transportation conducted tests that related torque and tension on four 1-in.-diameter A325 high-strength bolts. It was determined that a torque of 70 lbf-ft would develop approximately 4,300 lb of tension per bolt. The results of these tests are shown in Figure 3.

The four 1-in.-diameter slip bolts were held in place in the slots with a keeper plate. The keeper plate conformed to



NOTES

1) KEEPER PLATE CONFORMS TO ASTM DESIGNATION: A446 GRADE A. COATING DESIGNATION: G90.
2) ALL THREADED FASTENERS TO BE GALVANIZED UNLESS OTHERWISE NOTED.
3) POLE AND ARM TO BE GALVANIZED TO ASTM DESIGNATION: A123.
4) ACCESSORIES TO BE GALVANIZED TO ASTM DESIGNATION: A153.

FIGURE 1 Design details of the 4-bolt breakaway slipbase luminaire (*continued on next page*).

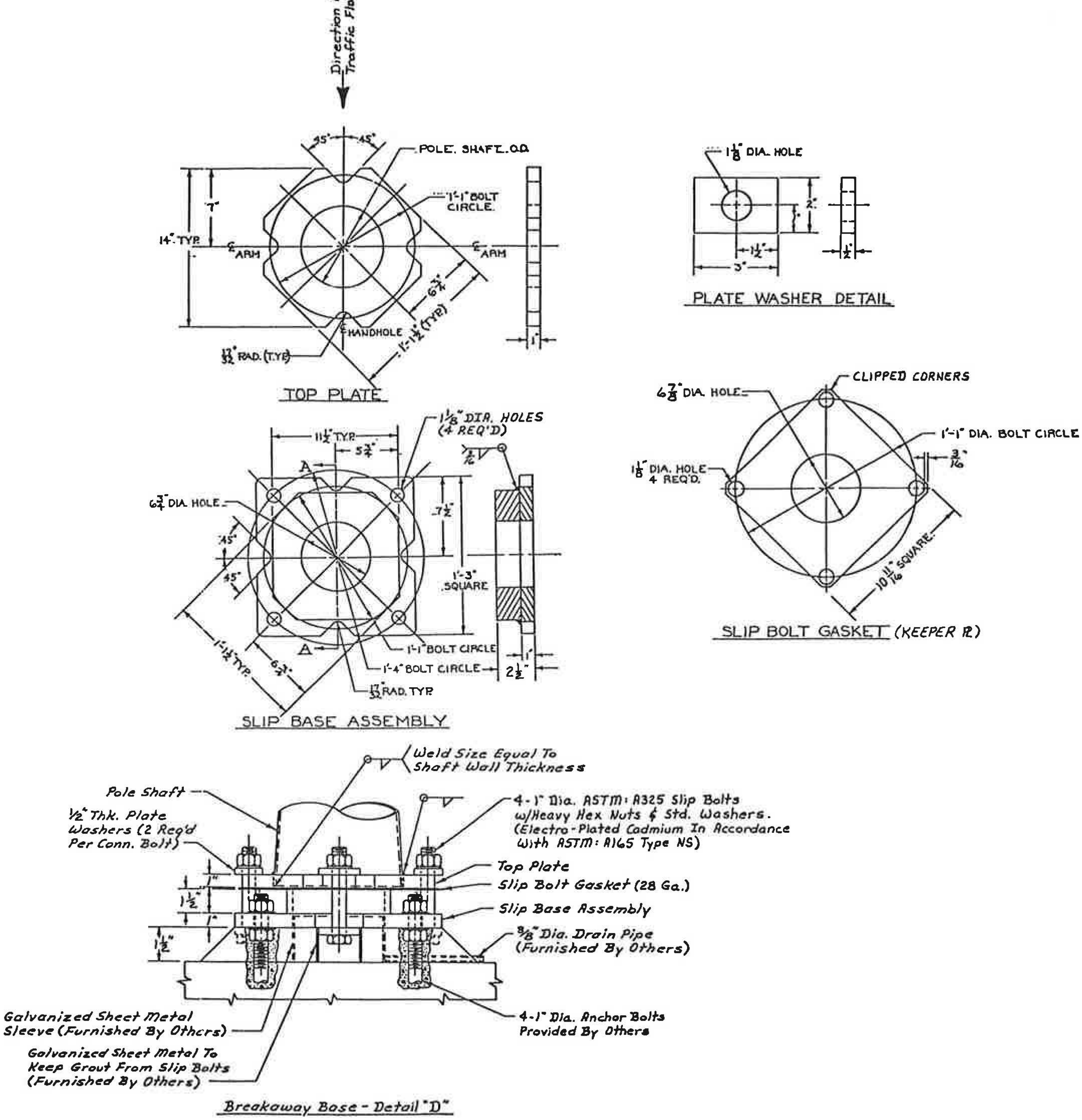


FIGURE 1 *(continued).*

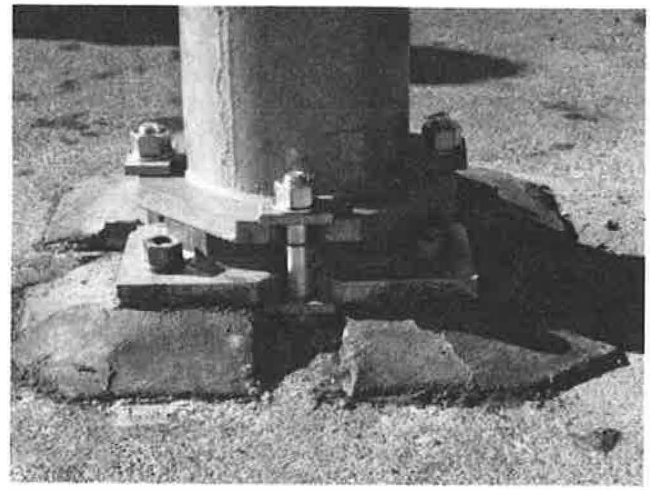


FIGURE 2 4-bolt breakaway slipbase installation.

ASTM A446 Grade A steel with a 0.0149-in. thickness (28 gauge) before coating.

The luminaire support had a diameter of 10 in. at the base, which tapered off to 3 in. at the top. The wall thickness was 0.120 in. (11 gauge). The luminaire support was manufactured with ASTM A595 Grade A steel and hot-dipped galvanized in accordance with ASTM standards (ASTM A123).

The two steel mast arms were attached 10 in. below the top of the luminaire support. The mast arms extended 15 ft outward from the face of the luminaire pole and 1 ft 8 in. above the top of the luminaire support. They extended outward perpendicular to the direction of the impact. Weights (75 lb per mast arm) were mounted on the end of each mast arm to simulate an actual luminaire.

A reinforced handhole opening was located approximately 1 ft 10 in. above the existing concrete apron. The luminaire support was installed so that the handhole opening was located on the side of the luminaire pole opposite that impacted by the test vehicle.

The slipbase was oriented with one of the slip bolts directly

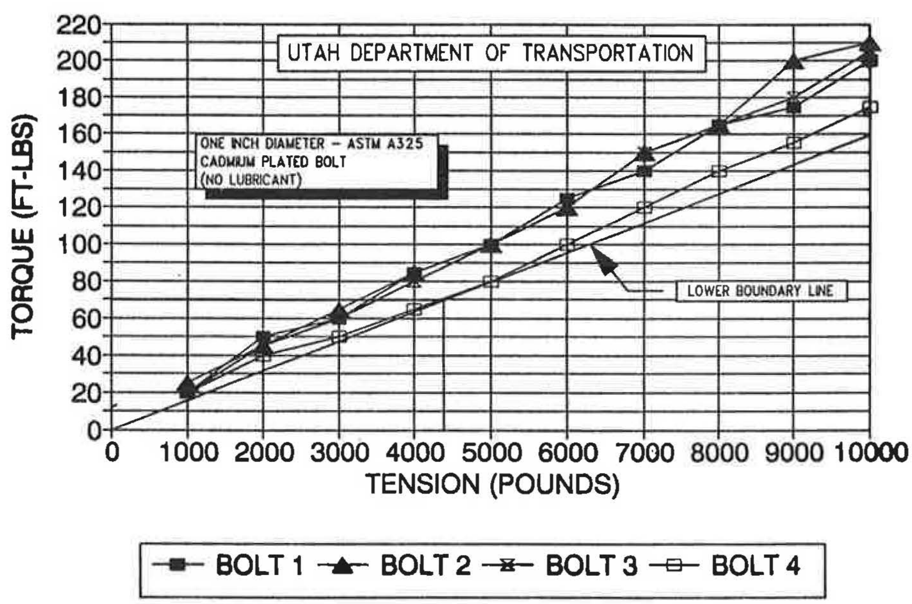


FIGURE 3 Torque-versus-tension tests for A325 slip bolts.

in line with the test vehicle impact location, as shown in Figures 1 and 2. It was anticipated that the highest percentage of accidents would occur with this orientation.

PERFORMANCE EVALUATION CRITERIA

The safety performance objective of a highway appurtenance is to minimize the consequences of an off-road accident. This safety goal is met when the appurtenance allows vehicle occupants to escape major-injury-producing forces. The safety performance of the highway appurtenance was evaluated according to four criteria: (a) breakaway mechanism worthiness, (b) vehicle stability and trajectory, (c) occupant risk, and (d) test object penetration. These factors are defined and explained in NCHRP Report 230 (*9*). Similar criteria are presented by AASHTO (*10*).

The 4-bolt slipbase design was evaluated according to the performance criteria presented in NCHRP Report 230 (*9*), AASHTO standard specifications (*10*), and the Code of Federal Regulations (*11*).

The standards used to evaluate the crash tests were Test Designation Numbers 62 and 63 from NCHRP Report 230 (*9*). These criteria require a 20-mph test in which the vehicle contacts the luminaire support at the center point of the bumper and a 60-mph test in which the impact occurs at the quarter point of the bumper. The location of impact for the 60-mph test was changed from the quarter point to the center point of the bumper under the AASHTO (*10*) recommendation that the quarter point impact may be more stringent than can easily be met under the current state of the art. According to the AASHTO guidelines, acceptable performance under the high-speed, off-center impact may be considered a goal, and acceptance may be based on a centerline, high-speed test.

The safety evaluation criteria are presented in Table 1 (*9–11*). NCHRP Report 230 (*9*) requires that the test article activate in a predictable manner by breaking away or yielding. In addition, detached fragments from the test article should not penetrate or show potential for penetrating the passenger compartment, nor should they present undue hazard to other traffic. The vehicle must remain upright during and after the collision, and the integrity of the passenger compartment must be maintained with essentially no deformation or intrusion. A design value of 15 *g* is recommended for the maximum longitudinal occupant ridedown deceleration (*9*). After the collision, the vehicle should intrude a minimum distance, if at all, into adjacent traffic lanes.

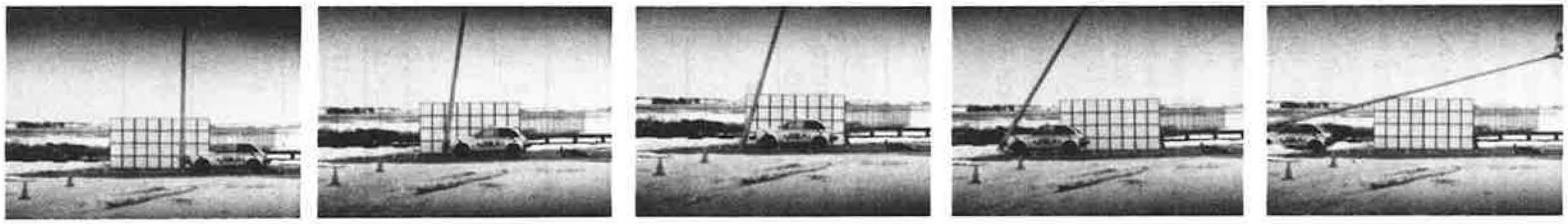
AASHTO specifications (*10*) include the same criteria as NCHRP Report 230 (*9*) except that they also recommend that the change in velocity of the vehicle be less than or equal to 15 ft/sec. FHWA updated that criterion to 16 ft/sec or less (*11*).

After each test, vehicle damage was assessed by the traffic accident data (TAD) scale (*12*) and the vehicle damage index (VDI) (*13*).

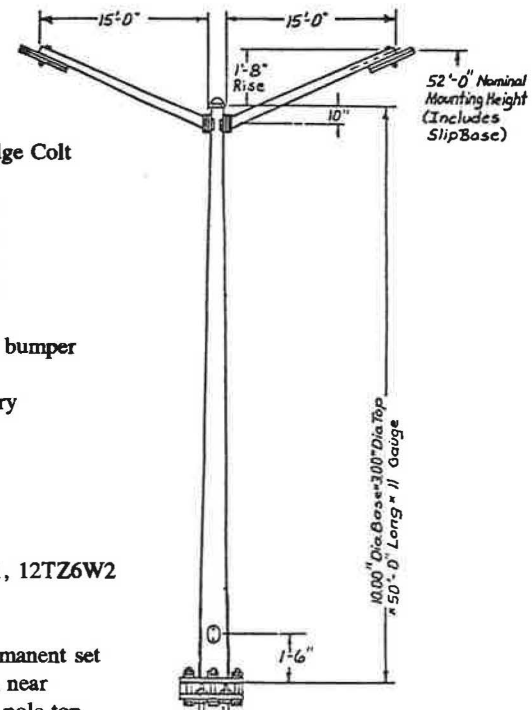
TABLE 1 Performance Evaluation Results

Evaluation Factors	Evaluation Criteria	Test USBLM-1			Test USBLM-2		
		NCHRP 230 (9)	AASHTO (10)	FHWA (11)	NCHRP 230 (9)	AASHTO (10)	FHWA (11)
Structural Adequacy	1. The test article shall readily activate in a predictable manner by breaking away or yielding.	S	S	S	S	S	S
	2. Detached elements, fragments or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.	S	S	S	S	S	S
Occupant Risk	3. The vehicle shall remain upright during and after collision although moderate roll, pitching and yawing are acceptable. Integrity of the passenger compartment must be maintained with essentially no deformation or intrusion.	S	S	S	S	S	S
	4. Longitudinal Occupant Impact Velocity (fps).	7.6 < 15	7.6 < 15	7.6 < 15	14.2 < 15	14.2 < 15	14.2 < 15
	5. Long. Occupant Ridedown Decelerations (g).	3.5 < 15	3.5 < 15	3.5 < 15	1.0 < 15	1.0 < 15	1.0 < 15
	6. Vehicle Change in Velocity (fps).	NA	6.1 < 15	6.1 < 16	NA	13.5 < 15	13.5 < 16
Vehicle Trajectory	7. After collision, the vehicle trajectory and final stopping position shall intrude a minimum distance, if at all, into adjacent traffic lanes.	S	S	S	S	S	S
	8. Vehicle trajectory behind the test article is acceptable.	S	S	S	S	S	S

S Satisfactory
M Marginal
U Unsatisfactory
NA Not Applicable



Impact 0.383 sec 0.583 sec 1.339 sec 2.338 sec



· Test Number	USBLM-1
· Date	1/24/91
· Installation	4-Bolt Breakaway Slipbase Luminaire Support
· Luminaire Height	52 ft-0 in.
· Mast Arm Span Width	30 ft-10 in.
· Luminaire System Weight	902 lbs
· Permanent Lower Slipbase Assembly	
Bolt Type	four 1-in. diameter ASTM A325 bolts
Bolt Circle Diameter	1 ft-4 in.
Stub Height	4 in.
· Luminaire Support Pole	
Base Diameter	10 in.
Top Diameter	3 in.
Length	50 ft
Slipbolt Type	four 1-in. diameter ASTM A325 slipbolts
Bolt Circle Diameter	1 ft-1 in.
Slip Bolt Gasket Thickness	0.0149 in. (28 gauge)
Initial Bolt Torque	80 ft-lbs.
Final Bolt Torque	70 ft-lbs.
Clamping Bolt Force	4 @ 4,300 lbs each

· Vehicle Model	1984 Dodge Colt
· Vehicle Weight	
Curb	1,990 lbs
Test Inertial	1,750 lbs
Gross Static	1,750 lbs
· Vehicle Impact Speed	15.0 mph
· Vehicle Impact Angle	0 deg
· Vehicle Impact Location	Center of bumper
· Vehicle Snagging	None
· Vehicle Stability	Satisfactory
· Occupant Impact Velocity	7.6 fps
· Occupant Ridedown Deceleration	3.5 g's
· Vehicle Change-In-Speed	6.1 fps
· Vehicle Damage	Minimal
TAD	12-FC-2
VDI	12FCEN1, 12TZ6W2
· Vehicle Front-End Deflection	9 in.
· Vehicle Stopping Distance	40 ft
· Luminaire Support Damage	Small permanent set deflection near luminaire pole top
· Final Luminaire Support Location	35 ft to Base

FIGURE 4 Summary and sequential photographs, Test USBLM-1.

TEST PARAMETERS

A 1984 Dodge Colt weighing 1,750 lb was used to evaluate the 4-bolt slipbase design. After Test USBLM-1, the bumper was replaced and the hood repaired so that the vehicle could be used for Test USBLM-2. Both tests were conducted with a centerline head-on impact. Test USBLM-1 was conducted with the vehicle traveling at 15 mph; Test USBLM-2 was conducted with the vehicle traveling at 57.5 mph.

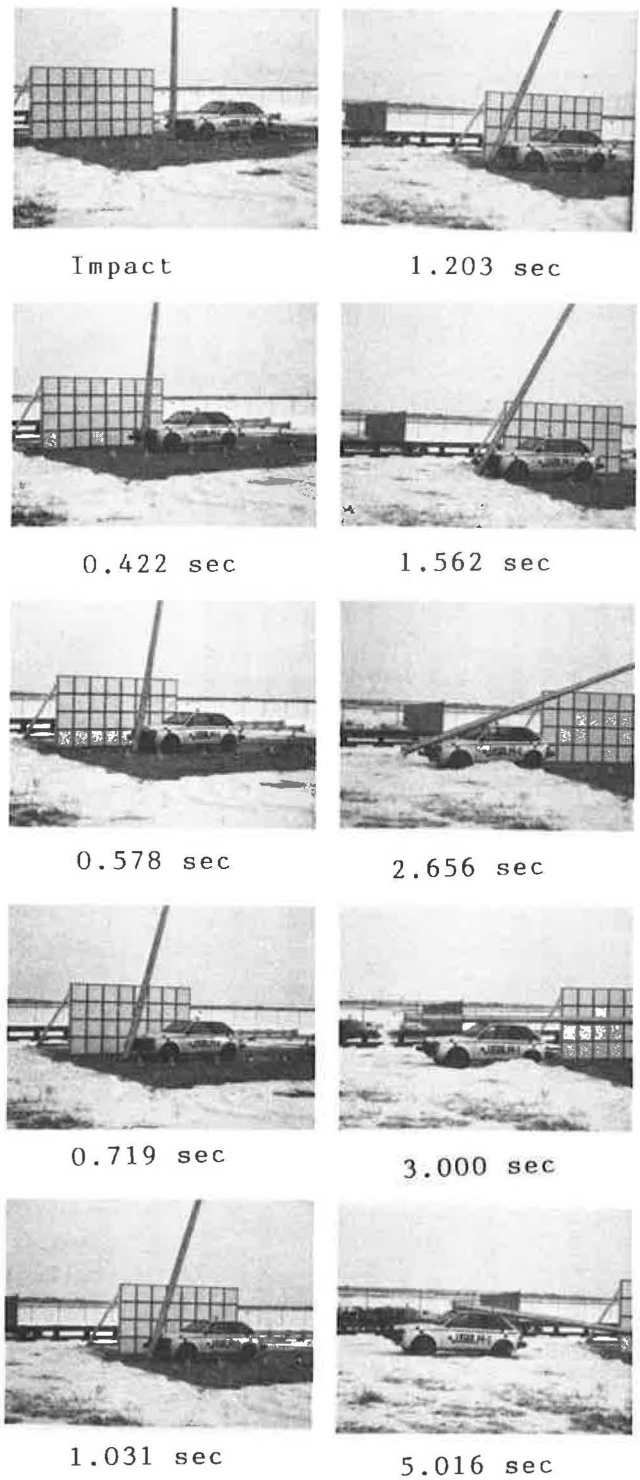


FIGURE 5 Sequential photographs, Test USBLM-1.

TEST RESULTS

Test USBLM-1

A summary of Test USBLM-1 is shown in Figure 4; sequential photographs are shown in Figure 5. The safety evaluation results are presented in Table 1. The test vehicle struck the luminaire support at the center of the front bumper at a speed of 15 mph. This impact speed was less than the target speed of 20 mph because of technical difficulties. Because the speed in this test was only 15 mph, it was more severe than in the 20 mph test because less kinetic energy was available to initiate breakaway.

On impact with the luminaire support, the front bumper of the vehicle crushed inward until approximately 0.08 sec after impact. At that time the luminaire support began to slip from the base. The luminaire support remained approximately vertical until 0.39 sec after impact when the top of the pole started to rotate toward the vehicle. The luminaire support continued to fall toward the vehicle until it hit the roof approximately 2.33 sec after impact. The top of the luminaire support hit

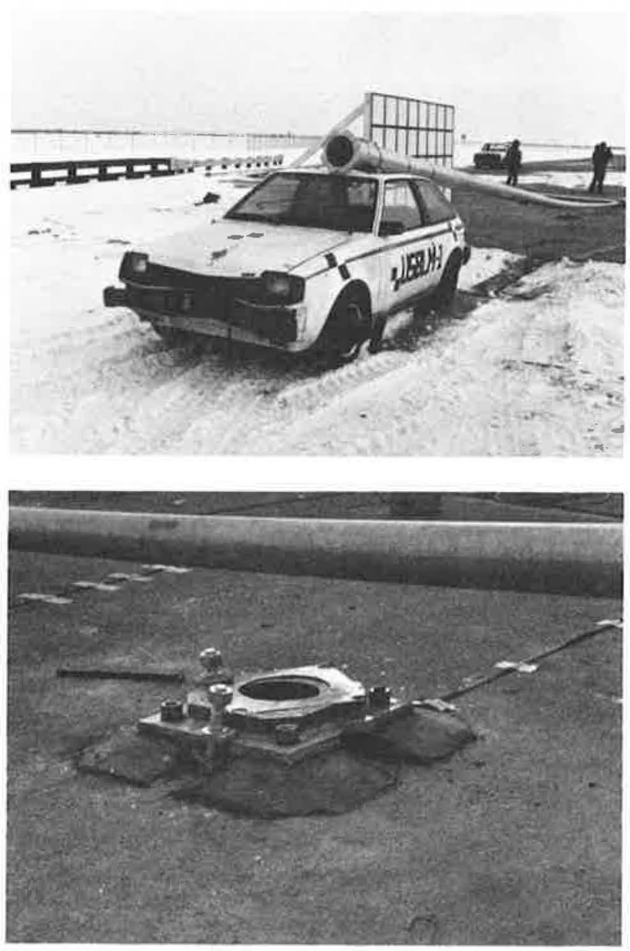
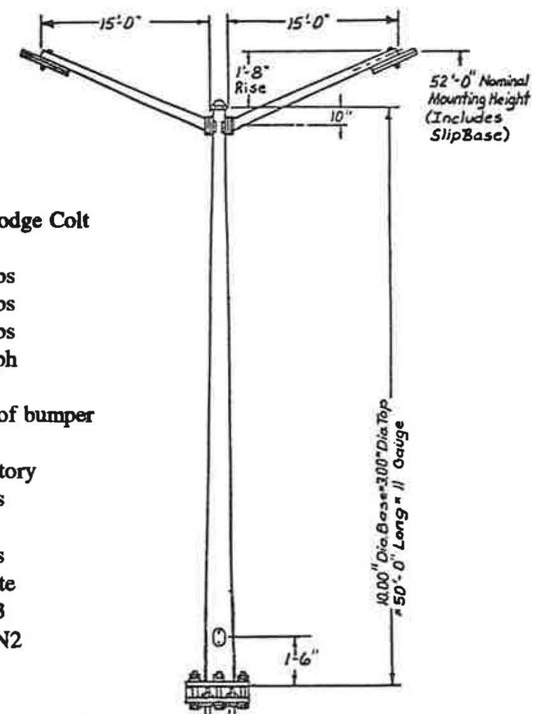


FIGURE 6 Vehicle and installation damage, Test USBLM-1.

Impact 0.188 sec 0.500 sec 0.625 sec 0.906 sec



· Test Number	USBLM-2
· Date	2/1/91
· Installation	4-Bolt Breakaway Slipbase Luminaire
· Luminaire Height	52 ft-0 in.
· Mast Arm Span Width	30 ft-10 in.
· Luminaire Weight	902 lbs
· Permanent Lower Slipbase Assembly	
Bolt Type	four 1-in. diameter ASTM A325 Bolts
Bolt Circle Diameter	1 ft-4 in.
Stub Height	4 in.
· Luminaire Support Pole	
Base Diameter	10 in.
Top Diameter	3 in.
Length	50 ft
Slipbolt Type	four 1-in. diameter ASTM A325 slipbolts
Bolt Circle Diameter	1 ft-1 in.
Slip Bolt Gasket Thickness	0.0149 in. (28 gauge)
Initial Bolt Torque	80 ft-lbs.
Final Bolt Torque	70 ft-lbs.
Clamping Bolt Force	4 @ 4,300 lbs each

· Vehicle Model	1984 Dodge Colt
· Vehicle Weight	
Curb	1,990 lbs
Test Inertial	1,750 lbs
Gross Static	1,750 lbs
· Vehicle Impact Speed	57.5 mph
· Vehicle Impact Angle	0 deg
· Vehicle Impact Location	Center of bumper
· Vehicle Snagging	None
· Vehicle Stability	Satisfactory
· Occupant Impact Velocity	14.2 fps
· Occupant Ridedown Deceleration	1.0 g's
· Vehicle Change-In-Speed	13.5 fps
· Vehicle Damage	Moderate
TAD	12-FC-3
VDI	12FCEN2
· Vehicle Front-End Deflection	12 in.
· Vehicle Stopping Distance	310 ft
· Luminaire Support Damage	Large permanent set deflection near top
· Final Luminaire Support Location	70 ft to Base

FIGURE 7 Summary and sequential photographs, Test USBLM-2.

the ground 2.73 sec after impact. The vehicle stopped 40 ft from the point of impact with the base of the luminaire pole resting on its roof.

The vehicle damage was minimal, with a maximum crushing distance of 9 in. in the bumper and slight damage to the roof. There was no intrusion of the passenger compartment, and the vehicle was repaired for use in Test USBLM-2.

The surface of the steel pole at the point of impact was not dented or deformed. There was only a slight deformation near the top of the support pole caused by the impact with the concrete. The permanent lower slipbase assembly was undamaged.

The damage to the vehicle and the installation is shown in Figure 6. The TAD (*12*) and VDI (*13*) vehicle damage classifications are shown in Figure 4. The occupant impact velocity was determined to be 7.6 ft/sec, which is much less than the 15 ft/sec suggested in NCHRP Report 230 (*9*). The maximum occupant ridedown deceleration was 3.5 *g*, which is less than the 15 *g* recommended in NCHRP Report 230 (*9*). The vehicle change in speed, calculated from impact to first loss of contact, was 6.1 ft/sec, which is lower than the 15 ft/sec required by AASHTO (*10*) and the 16 ft/sec required by FHWA (*11*).

Test USBLM-2

A summary of Test USBLM-2 is shown in Figure 7, and sequential photographs are shown in Figure 8. The safety evaluation results are presented in Table 1. The permanent lower slipbase used for Test USBLM-1 was also used in Test USBLM-2. However, new mast arms, a new pole, and new clamping bolts were used in the second test.

The test vehicle struck the luminaire support at the center of the front bumper at a speed of 57.5 mph. On impact, the front bumper of the vehicle was crushed inward for 0.02 sec. At that time the luminaire support began to slip from the base. At 0.17 sec, the front of the car began to lift up, and it continued on its rear wheels until 1 sec after impact. At 0.87 sec, the luminaire support was approximately 16 ft above and parallel to the ground. The luminaire support impacted the ground at 1.11 sec. The vehicle continued in a straight path until it slid sideways and stopped 310 ft downstream from the base.

The only damage sustained by the vehicle was to the bumper, which had a maximum crushing distance of 12 in. There was no intrusion of the occupant compartment.

The surface of the steel pole at the impact point was not dented or deformed. The support pole was deformed slightly more than in the first test. This deformation occurred at the top of the support pole and was caused by the impact of the pole on the concrete apron. The permanent lower slipbase assembly was not damaged.

The damage sustained by the vehicle and the installation is shown in Figure 9. The TAD (*13*) and VDI (*14*) damage classifications are shown in Figure 7.

The occupant impact velocity was determined to be 14.2 ft/sec, which is less than the 15 ft/sec suggested in NCHRP Report 230 (*9*). The maximum occupant ridedown deceleration was 1 *g*, which is much less than the 15 *g* recommended in NCHRP Report 230 (*9*). The vehicle change in velocity, calculated from impact to first loss of contact, was 13.5 ft/sec,

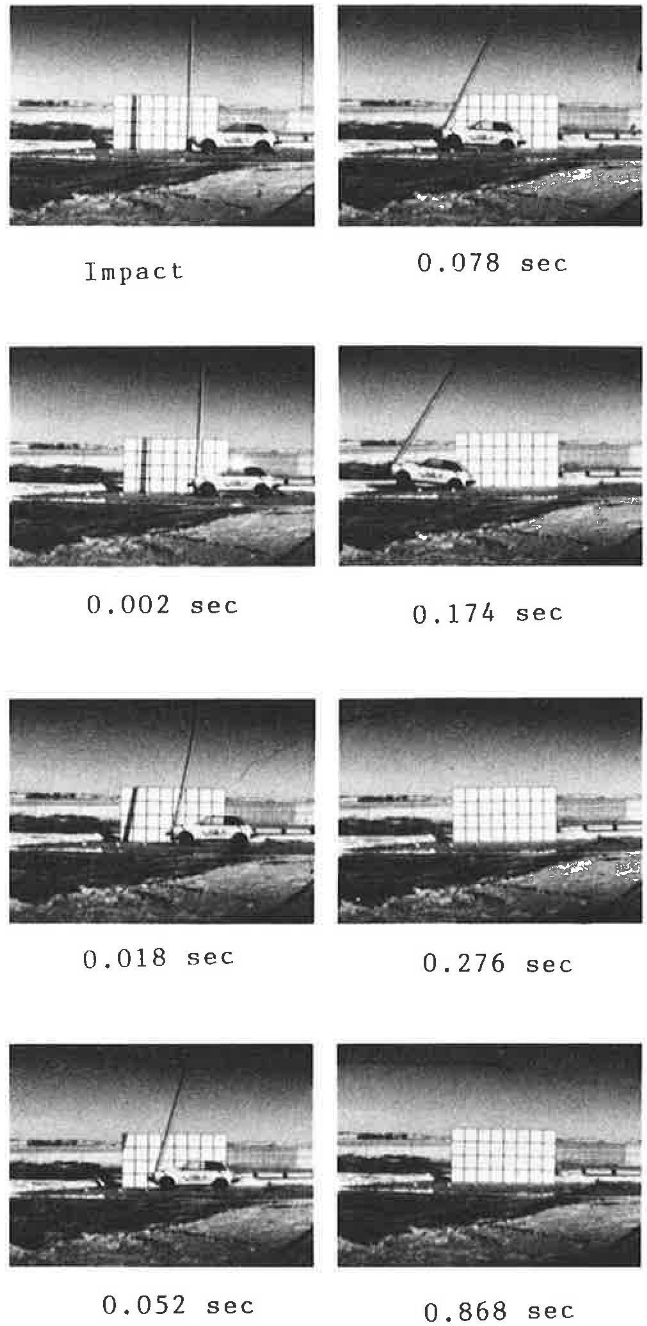


FIGURE 8 Sequential photographs, Test USBLM-2.

which is lower than the 15 ft/sec required by AASHTO (*10*) and the 16 ft/sec required by FHWA (*11*).

CONCLUSIONS

Two full-scale crash tests were conducted to evaluate the safety performance of the 4-bolt breakaway slipbase design. The analysis of the crash tests revealed the following:

- The test article activated in a predictable manner by breaking away.

FIGURE 9 Vehicle and installation damage, Test USBLM-2.

- Detached elements, fragments, and other debris from the test article did not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
- The vehicle remained upright during and after the collision, and the integrity of the passenger compartment was maintained.
- The longitudinal occupant impact velocities for Tests USBLM-1 (7.6 ft/sec) and USBLM-2 (14.2 ft/sec) were less than the 15 ft/sec recommended in NCHRP Report 230 (*9*).
- The longitudinal occupant ridedown decelerations for Test USBLM-1 (3.5 *g*) and Test USBLM-2 (1 *g*) were less than the 15 *g* recommended in NCHRP Report 230 (*9*).
- The changes in vehicle speed for Test USBLM-1 (6.1 ft/sec) and Test USBLM-2 (13.5 ft/sec) were less than the 15 ft/sec required by AASHTO (*10*) and the 16 ft/sec required by FHWA (*11*).

On the basis of this analysis, the results of Tests USBLM-1 and USBLM-2 are acceptable according to the guidelines established in NCHRP Report 230 (*9*), AASHTO standard specifications (*10*), and the Code of Federal Regulations (*11*). Therefore, the use of the 4-bolt slipbase design is recommended for use in Federal-aid projects.

ACKNOWLEDGMENTS

This study was conducted under a cooperative program between the Midwest Roadside Safety Facility, the University of Nebraska Center for Infrastructure Research, the Midwest Transportation Center, and the state transportation departments of Idaho, Montana, Nevada, Utah, and Wyoming. The crash tests were conducted by personnel at the Midwest Roadside Safety Facility. The 4-bolt slipbase system was provided by Valmont Industries of Valley, Nebraska.

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The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Utah Department of Transportation or FHWA. This report does not constitute a standard, specification, or regulation.

Aesthetically Pleasing Combination Pedestrian-Traffic Bridge Rail

T. J. HIRSCH AND C. E. BUTH

Railings have been developed to withstand impact loads from vehicles of ever-increasing size; however, aesthetic considerations have been overshadowed by safety and structural requirements. The objective of this study was to develop aesthetically pleasing, structurally sound alternative railings for cities and urban areas. A new concrete combination pedestrian-traffic bridge rail, Texas Type C411, is presented. The bridge rail was constructed of reinforced concrete 42 in. high × 12 in. thick. It contains 6-in.-wide × 28-in.-high openings at 18-in. center-to-center longitudinal spacing. The combination pedestrian-traffic bridge rail was located on a 6-ft-wide sidewalk with an 8-in.-high curb separating it from traffic. The rail was developed for use on urban streets on which speed limits are 45 mph or less. Service Level 1 in NCHRP Report 230 and Performance Level 1 in the 1989 AASHTO *Guide Specifications for Bridge Railings* were considered inappropriate. A different test matrix was being considered under NCHRP Project C22-7, Update of Roadside Safety Hardware Crash Test Specifications, and NCHRP Project 12-33, Development of a Comprehensive Bridge Specification and Commentary, at the time these tests were conducted. It was decided to use a 4,500-lb automobile at 45 mph and a 25-degree impact angle and a 1,900-lb automobile at 45 mph and a 20-degree impact angle. The new C411 bridge rail performed well under these two crash tests. The results of the crash tests easily met the standard safety evaluation criteria. The C411 should be safe for use on low-speed (45 mph or less) roadways.

Railings have been developed to withstand impact loads from vehicles of ever-increasing size; however, aesthetic considerations have been overshadowed by safety and structural requirements. Engineers often fail to recognize the impact of structures on the landscape, particularly in cities and urban areas. Architects and developers often propose aesthetically pleasing railings that engineers cannot accept because of structural inadequacies. The objective of this study was to develop aesthetically pleasing, structurally sound alternative railings.

This study was an attempt to develop one or more new concrete, steel, and aluminum railings or combination railings, some with a curb and sidewalk.

A new open-type concrete combination pedestrian-traffic bridge rail, Texas Type C411, is presented here.

DESCRIPTION OF TEXAS TYPE C411 BRIDGE RAIL

The bridge rail was constructed of reinforced concrete 42-in. high × 12-in. thick. It contains 6-in.-wide × 28-in.-high openings at 18-in. center-to-center longitudinal spacing. The combination pedestrian-traffic bridge rail was located on a 6-ft-wide sidewalk with an 8-in.-high curb separating it from traffic. Figures 1 – 3 show an elevation, cross section, and plan view of the C411 rail. The sidewalk deck is a 7.75-in.-thick typical Texas bridge slab design in accordance with AASHTO specifications (*1*).

A photograph of the bridge rail installation before crash testing is shown in Figure 4. The installation was 47-ft 4-in. long. The three pilasters are not the super-strong posts that they appear to be. They contain styrofoam blocks 10.5 in. × 13 in. × 31 in. (void), which means that the pilasters are similar to the 6-in. × 28-in. openings. Thus, use of pilasters is optional; they did not contribute to the strength of the bridge rail as built and crash tested.

This bridge rail was designed using a failure mechanism (or yield line) method of analysis (*2*). The design strength of the concrete was $f_c = 3{,}600$ psi, and the yield strength of reinforcing steel was $f_y = 60{,}000$ psi. The top beam was nominally 7-in. wide and 10- to 12-in. thick ($b = 7$ in. and $d = 8.25$ in.), yielding an ultimate moment capacity of 20.0 kip-ft. The posts were 10-in. wide and 10-in. thick ($b = 10$ in. and $d = 8$ in.), yielding an ultimate moment capacity of 20.6 kip-ft. With a moment arm of 3.5 ft, each post could resist a lateral load of about 5.9 kips. A summary of the failure mechanism analysis of the strength of the C411 bridge rail is shown in Figure 5. The failure load would be about 51.4 kips or more over five spans or a 7.5-ft length of bridge rail, as shown in Figure 6.

CRASH TESTS

The Texas Type C411 was developed for use on urban streets on which the speed limit is 45 mph or less. Selection of the crash-test matrix posed a problem. Using Service Level 1 from NCHRP Report 230 (*3*) would require use of a 4,500-lb automobile at 60 mph and a 15-degree impact angle for the strength test and a 1,800-lb automobile at 60 mph and a 20-degree impact angle for geometry evaluation. Using Performance Level 1 from the 1989 AASHTO *Guide Specifications for Bridge Railings* (*4*) would indicate testing with a 5,400-lb pickup truck at 45 mph and a 20-degree impact angle and a 1,800-lb automobile at 50 mph and a 20-degree impact angle.

These documents were being revised under NCHRP Project C22-7, Update of Roadside Safety Hardware Crash Test Specifications, and NCHRP Project 12-33, Development of a Comprehensive Bridge Specification and Commentary. Researchers on these two projects were considering the severity level test matrix shown in Table 1 at the time the tests in this

Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843.

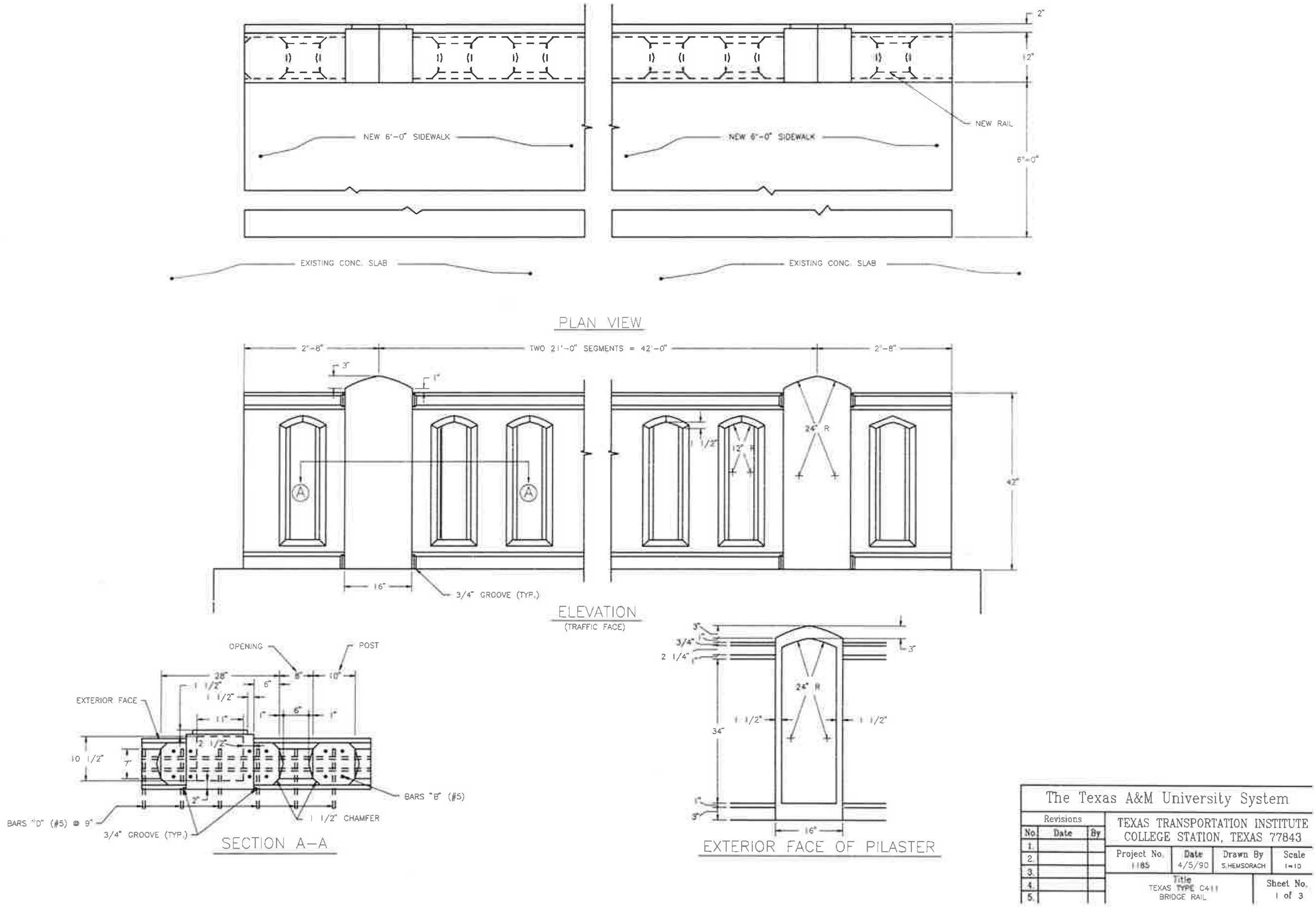
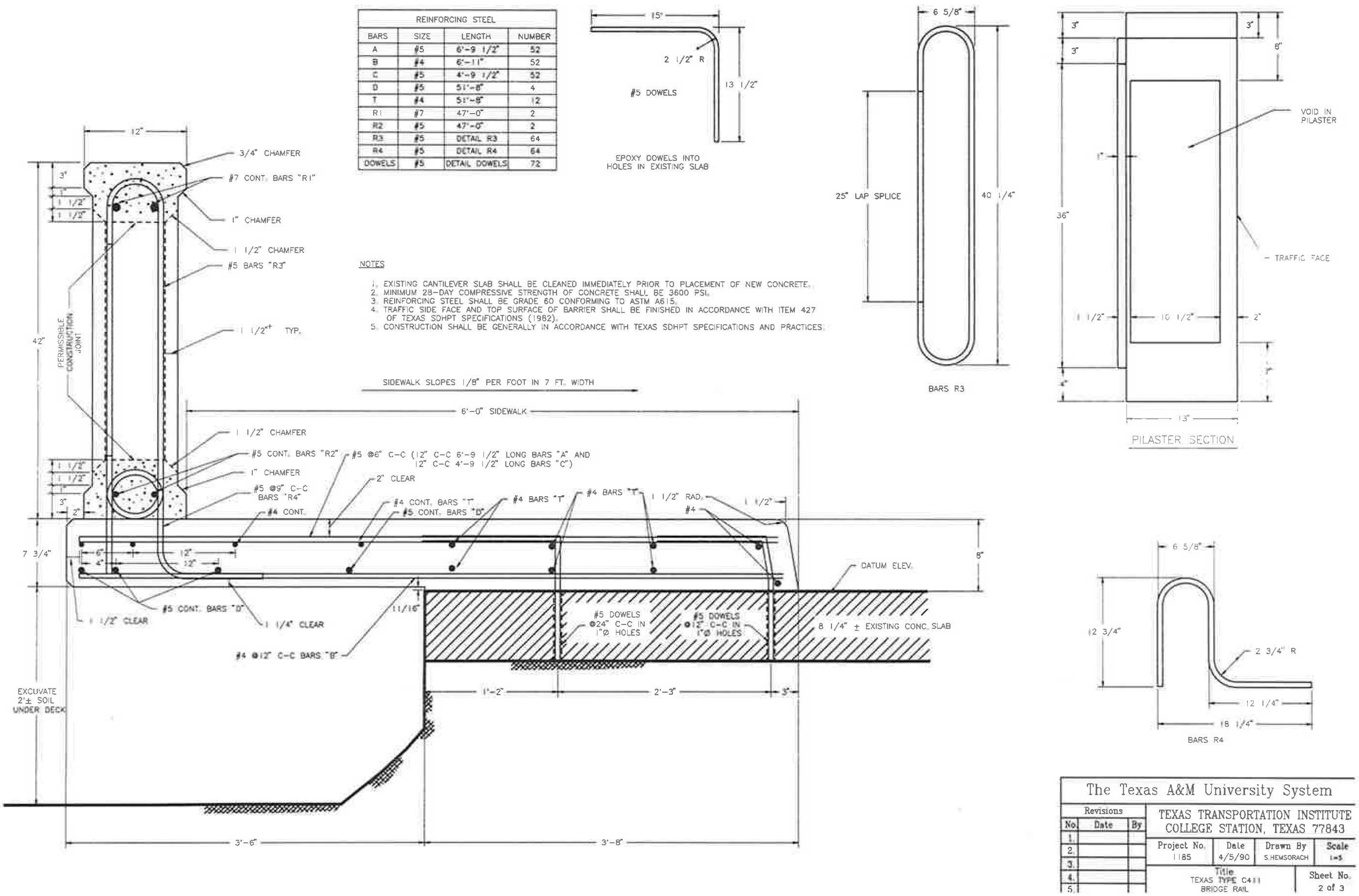


FIGURE 1 Plan and elevation of Texas Type C411 bridge rail.



REINFORCING STEEL			
BARS	SIZE	LENGTH	NUMBER
A	#5	6'-9 1/2"	52
B	#4	6'-11"	52
C	#5	4'-9 1/2"	52
D	#5	51'-8"	4
T	#4	51'-8"	12
R1	#7	47'-0"	2
R2	#5	47'-0"	2
R3	#5	DETAIL R3	64
R4	#5	DETAIL R4	64
DOWELS	#5	DETAIL DOWELS	72

NOTES

1. EXISTING CANTILEVER SLAB SHALL BE CLEANED IMMEDIATELY PRIOR TO PLACEMENT OF NEW CONCRETE.
2. MINIMUM 28-DAY COMPRESSIVE STRENGTH OF CONCRETE SHALL BE 3600 PSI.
3. REINFORCING STEEL SHALL BE GRADE 60 CONFORMING TO ASTM A615.
4. TRAFFIC SIDE FACE AND TOP SURFACE OF BARRIER SHALL BE FINISHED IN ACCORDANCE WITH ITEM 427 OF TEXAS SDHPT SPECIFICATIONS (1982).
5. CONSTRUCTION SHALL BE GENERALLY IN ACCORDANCE WITH TEXAS SDHPT SPECIFICATIONS AND PRACTICES.

The Texas A&M University System

TEXAS TRANSPORTATION INSTITUTE
COLLEGE STATION, TEXAS 77843

Revisions		
No.	Date	By
1.		
2.		
3.		
4.		
5.		

Project No.	Date	Drawn By	Scale
1185	4/5/90	S.HEMSORACH	1=5

Title: TEXAS TYPE C411 BRIDGE RAIL — Sheet No. 2 of 3

FIGURE 2 **Cross section of Texas Type C411 bridge rail.**

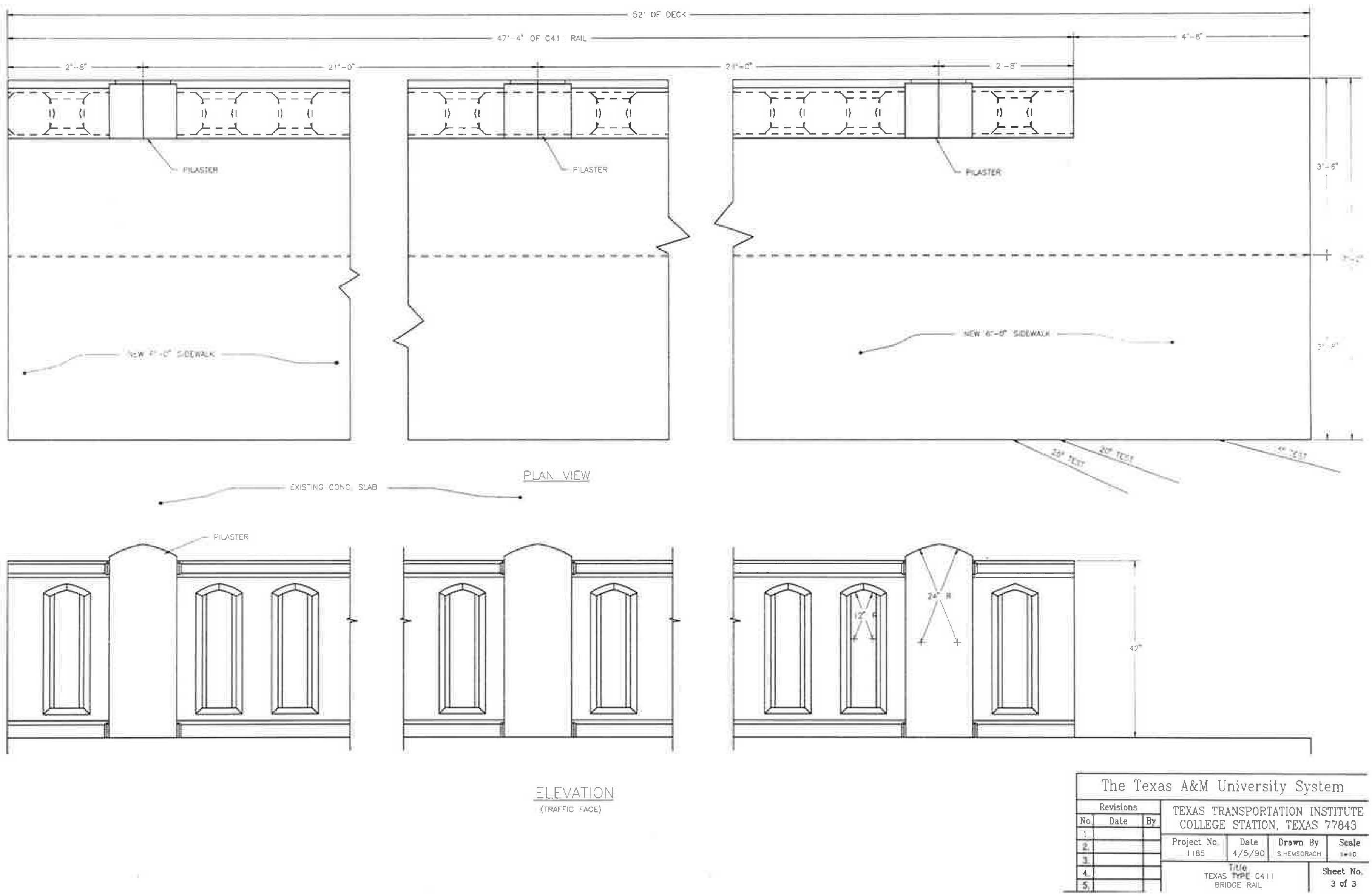


FIGURE 3 Plan and elevation of Texas Type C411 bridge rail.

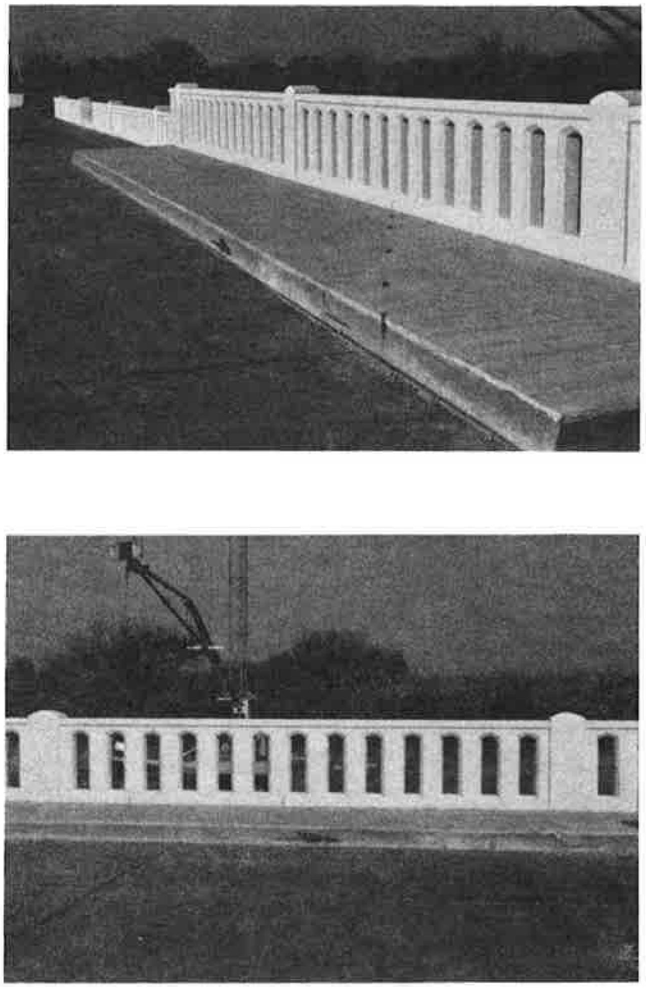


FIGURE 4 Bridge rail before Test 1185-5.

study were conducted. Therefore, Severity Level 2 from Table 1 was chosen. A 4,500-lb automobile (not a pickup) at 45 mph and a 25-degree impact angle and a 1,900-lb automobile at 45 mph and a 20-degree impact angle were used.

Description of Test 1185-5

A 1982 Honda Civic (Figures 7 and 8) was directed into the bridge rail installation using a reverse tow and guidance system. Test inertia mass of the vehicle was 1,800 lb (808 kg), and its gross static mass was 1,970 lb (894 kg). The height to the lower edge of the vehicle bumper was 15.0 in. (38.1 cm); the height to the top of the bumper was 20.5 in. (52.1 cm). The vehicle was free-wheeling and unrestrained just before impact.

The speed of the vehicle at impact with the curb was 45.5 mph (73.2 km/hr); the angle of impact was 20.1 degrees. The vehicle impacted the curb approximately 8 ft (2.4 m) from the end of the sidewalk. As the vehicle rode up the curb, the right front wheel twisted counterclockwise, and as it rode onto the sidewalk, the vehicle redirected slightly. At 0.322 sec, the vehicle, traveling at 43.0 mph (69.2 km/hr), struck the bridge rail 28 ft (8.5 m) from the end of the rail. The impact angle was 17.8 degrees. The vehicle was airborne at this time and began to redirect significantly at 0.371 sec. At 0.626 sec the vehicle was traveling parallel with the bridge rail at a speed of 34.1 mph (54.9 km/hr), and at 0.632 sec the rear of the vehicle hit the bridge rail. Traveling at 32.2 mph (51.8 km/hr), the vehicle lost contact with the bridge rail at 0.761 sec. The exit trajectory was 2.7 degrees. The front of the vehicle rode off the sidewalk at 0.781 sec and touched ground at 0.932 sec after impact. The brakes were then applied and the vehicle yawed counterclockwise and subsequently came to rest 105 ft (32.0 m) from and 25 ft (7.6 m) in front of the point of impact.

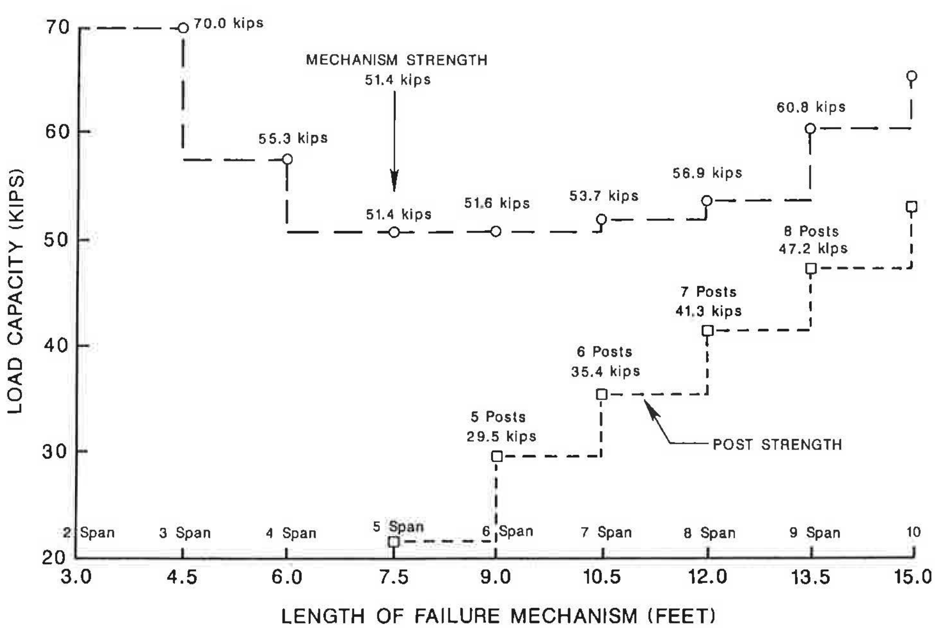


FIGURE 5 Failure mechanism analysis of Texas C411 bridge rail.

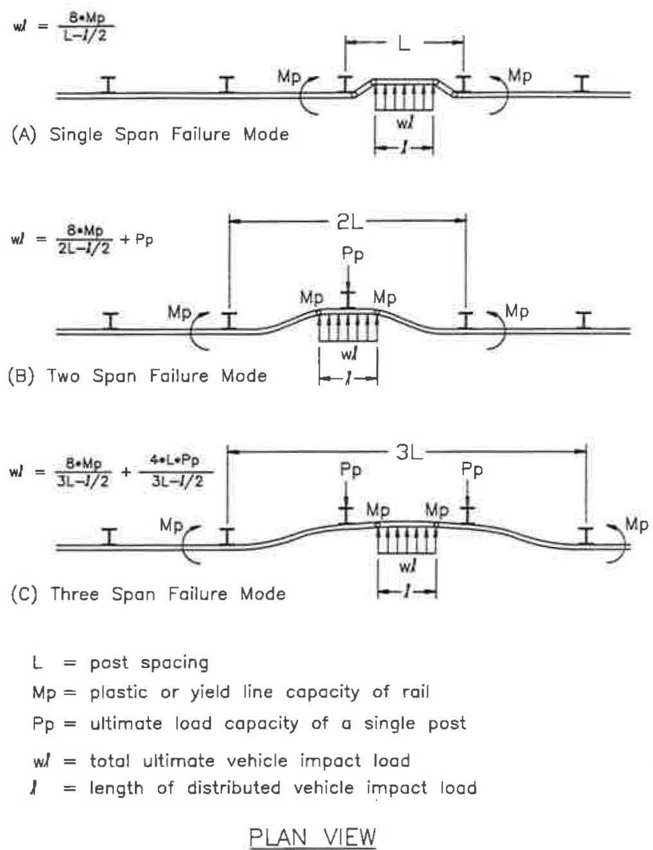


FIGURE 6 Possible failure modes for beam and post barriers.

(D) Four Span Failure Mode

$$wl = \frac{8M_p}{4L-l/2} + \frac{4P_pL}{4L-l/2} + P_p$$

(E) Five Span Failure Mode

$$wl = \frac{8M_p}{5L-l/2} + \frac{12P_pL}{5L-l/2}$$

(F) Six Span Failure Mode

$$wl = \frac{8M_p}{6L-l/2} + \frac{12P_pL}{6L-l/2} + P_p$$

(G) Seven Span Failure Mode

$$wl = \frac{8M_p}{7L-l/2} + \frac{24P_pL}{7L-l/2}$$

(H) Eight Span Failure Mode

$$wl = \frac{8M_p}{8L-l/2} + \frac{24P_pL}{8L-l/2} + P_p$$

The equations above for the ultimate horizontal load capacity (*wl*) satisfy all equations of static equilibrium. A simpler equation (which does not quite satisfy equations of static equilibrium for forces and moments in the beam) is as follows:

(I)

$$wl = \frac{8M_p}{NL-l/2} + \sum_{0}^{NL} P_p$$

where N = number of spans in the failure mechanism.

Equation (I) was used to analyze this rail (Figure 5).

TABLE 1 Test Severity Levels, Vehicles, Weights, Angles, and Speeds

Test Vehicle Description and Impact Angles			Severity Level (SL) and Test Speed mph					
Vehicle Description	W (kips)	θ (degrees)	SL-1	SL-2	SL-3	SL-4	SL-5	SL-6
Small Automobile	1.9	20	30	45	60	60	60	60
Pickup Truck or Sports Wagon Truck	4.5	25	30	45	60	60*	60*	60*
Medium Single Unit Truck	18.0	15				50		
Van Type Tractor-Trailer	80.0	15					50	
Tank Type Tractor-Trailer	80.0	15						50

*These tests should be conducted unless it can be conclusively shown that these tests would be no more severe than the small automobile test (above) and the truck test (below).

FIGURE 7 Vehicle before Test 1185-5.

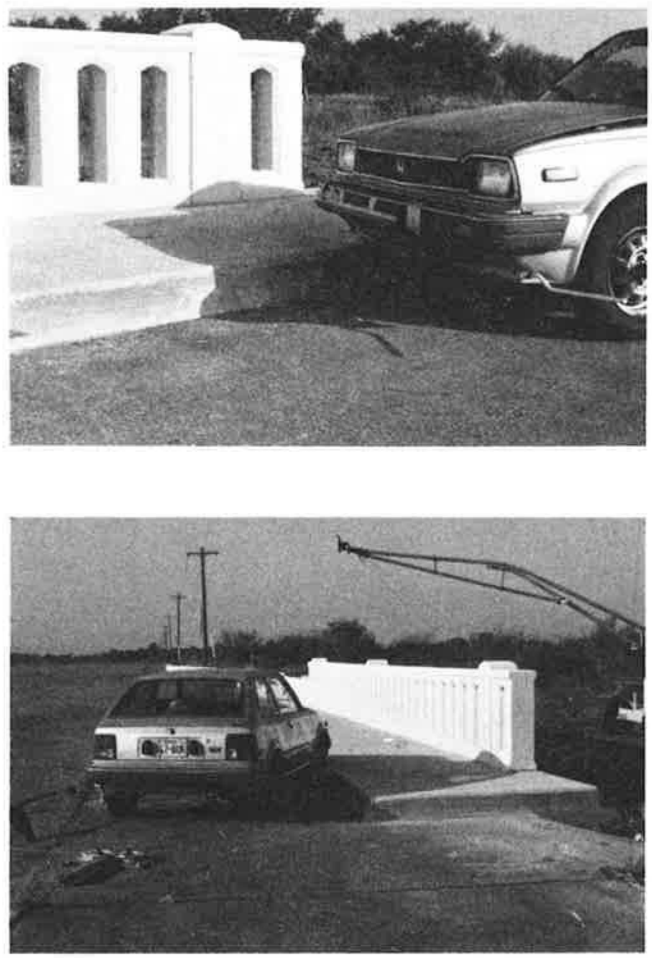


FIGURE 8 Vehicle and bridge rail installation geometrics for Test 1185-5.

As can be seen in Figure 9, the rail received minimal cosmetic damage. Tire marks on the face of the bridge rail extended 11 ft (3.4 m) from the point of impact. Some scraping and gouging occurred along the edges of the portholes.

The vehicle sustained moderate damage to the right side, as shown in Figures 10 and 11. Maximum crush at the right front corner at bumper height was 9.0 in. (22.9 cm). The right front and rear rims were bent and the tires damaged. There was damage to the hood, grill, bumper, right front and rear quarter panels, right door, and rear bumper.

Results of Test 1185-5

The speed of the vehicle as it struck the curb was 45.5 mph (73.2 km/hr), and the angle of impact was 20.1 degrees. The vehicle was traveling at 43.3 mph (69.7 km/hr) when it struck the bridge rail at an angle of 17.8 degrees. The speed of the vehicle as it was parallel to the bridge rail was 34.1 mph (54.9 km/hr). Exit speed was 32.2 mph (51.8 km/hr), and exit trajectory was 2.7 degrees. Occupant impact velocity was 12.1 ft/sec (3.7 m/sec) in the longitudinal direction and 7.3 ft/sec (2.2 m/sec) in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were $-4.6\,g$ (longitudinal) and 7.4 g (lateral). These data and other pertinent information from the test are summarized in Figure 12.

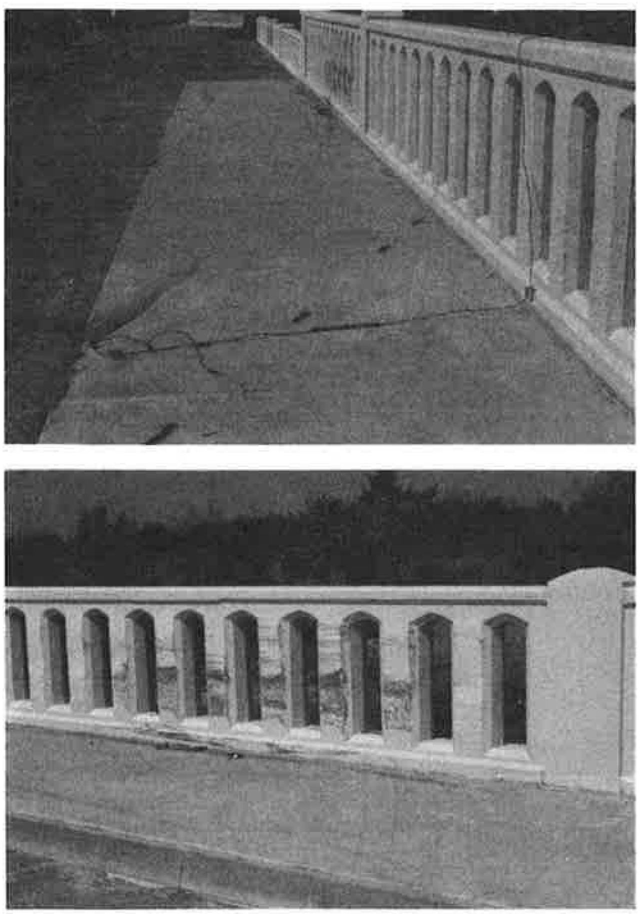


FIGURE 9 Bridge rail after Test 1185-5.

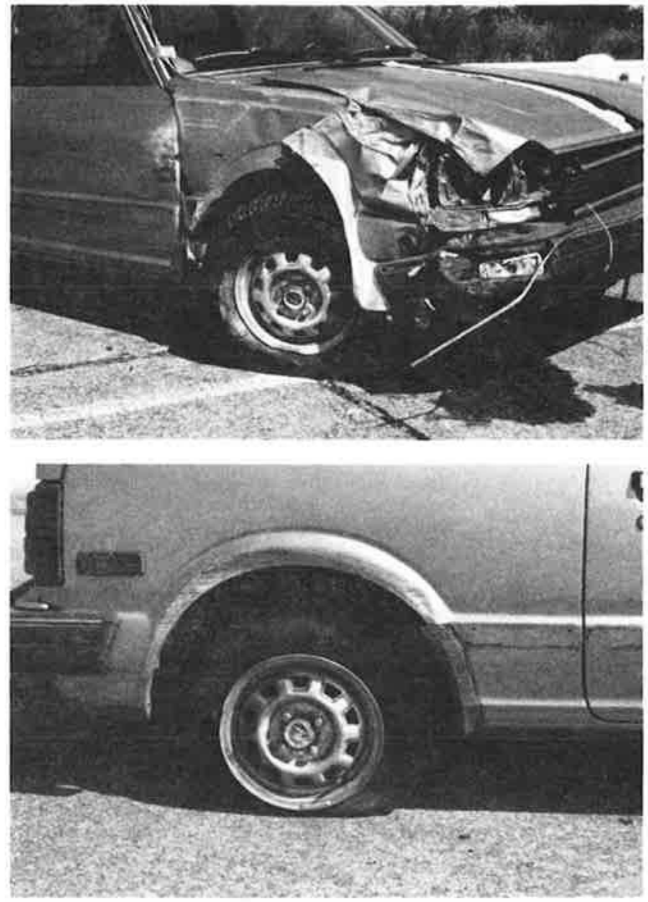


FIGURE 10 Damage to right side of vehicle, Test 1185-5.

Conclusions

The bridge rail contained and smoothly redirected the test vehicle with no lateral movement or cracking. There were no detached elements or debris to present undue hazard to other vehicles. The vehicle remained upright and relatively stable during the collision. The occupant-compartment impact velocities and 10-m/sec occupant ridedown accelerations were within the usual recommended limits. The vehicle trajectory at loss of contact indicated no intrusion into adjacent traffic lanes.

Description of Test 1185-6

A 1982 Oldsmobile 98 (Figures 13 and 14) was directed into the bridge rail installation using a reverse tow and guidance system. Test inertia mass of the vehicle was 4,500 lb (2,043 km). The height to the lower edge of the vehicle bumper was 12.25 in. (31.1 cm); the height to the top of the bumper was 20.75 in. (52.7 cm). The vehicle was free-wheeling and unrestrained just before impact.

The speed of the vehicle at impact was 47.0 mph (75.6 km/hr); the angle of impact was 25.4 degrees. The vehicle struck the curb approximately 5.75 ft (1.75 m) from the end of the sidewalk. As the right front tire rode up the curb and onto the sidewalk, the vehicle redirected slightly. At 0.177 sec, the

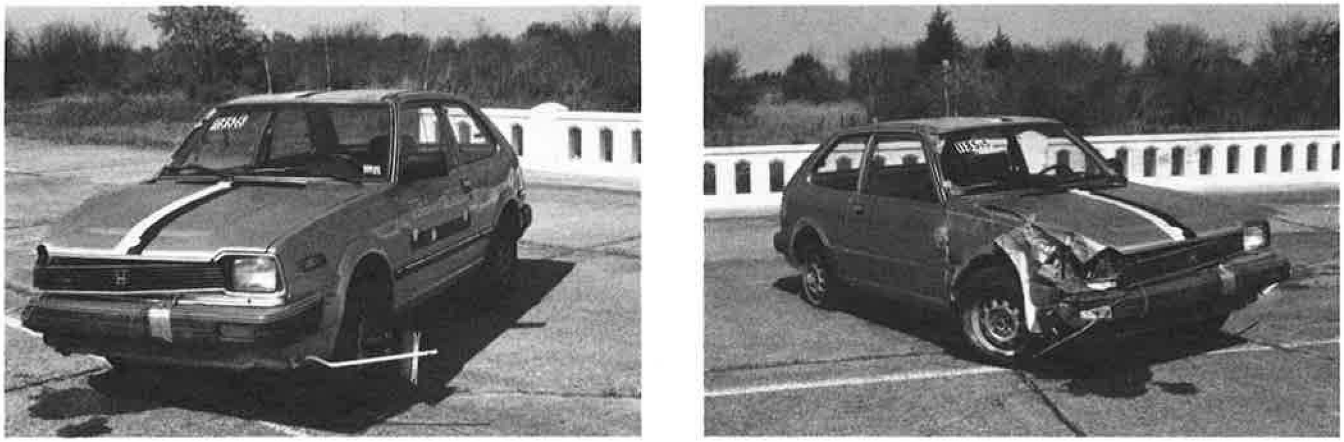
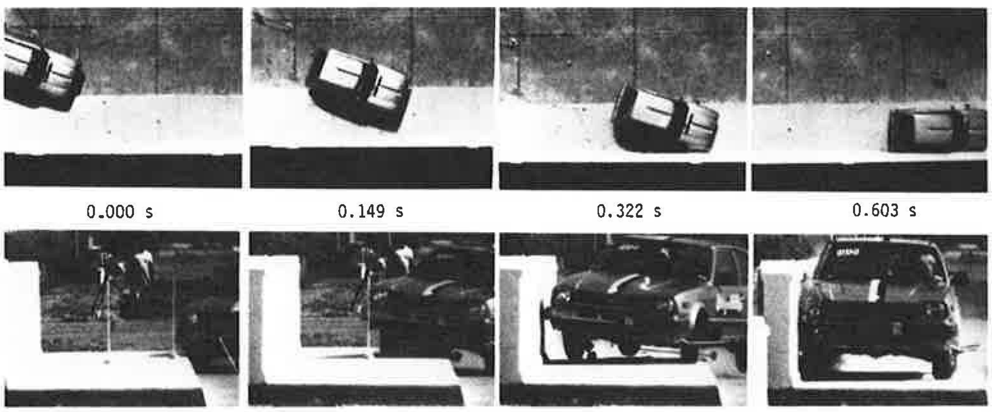


FIGURE 11 Vehicle after Test 1185-5.



Test No. 1185-5
Date 07/03/90

Test Installation . . . Texas Type C411
Bridge Rail
Installation Length . . 52 ft (16 m)

Vehicle . . . . 1982 Honda Civic
Vehicle Weight
Test Inertia . . . . . 1,800 lb (817 kg)
Gross Static . . . . . 1,970 lb (894 kg)
Vehicle Damage Classification
TAD 01RFQ4
CDC . . . . 01FREK1 & 01RFES3
Maximum Vehicle Crush . 9.0 in (22.9 cm)

Impact Speed . . . 45.5 mi/h (73.2 km/h)
Impact Angle . . . 20.1 degrees
Speed at Parallel . 34.1 mi/h (54.9 km/h)
Exit Speed . . . . 32.2 mi/h (51.8 km/h)
Exit Trajectory . . 2.7 degrees
Vehicle Accelerations
(Max. 0.050-sec Avg)
Longitudinal . . -3.6 g
Lateral . . . . . 4.0 g
Occupant Impact Velocity
Longitudinal . . 12.1 ft/s (3.7 m/s)
Lateral . . . . . 7.3 ft/s (2.2 m/s)
Occupant Ridedown Accelerations
Longitudinal . . -4.6 g
Lateral . . . . . 7.4 g

FIGURE 12 Summary of results, Test 1185-5.

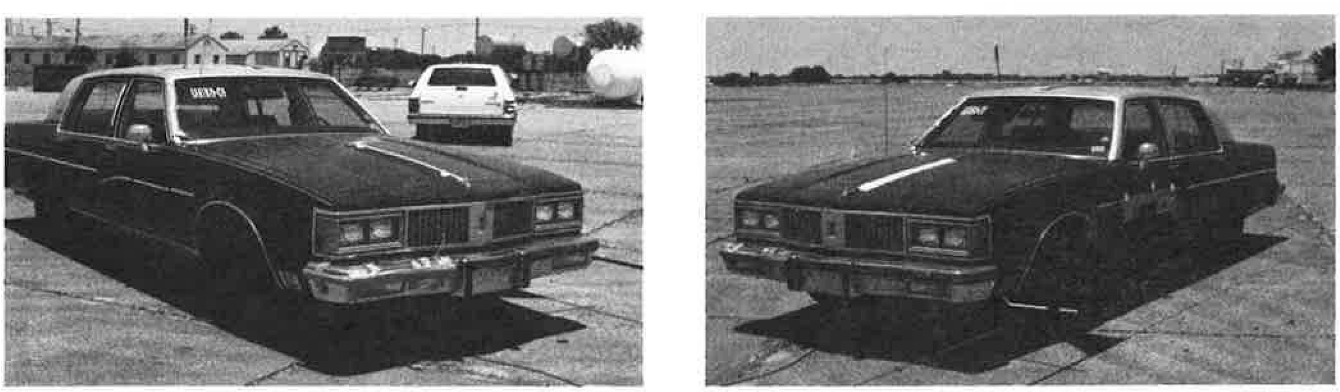


FIGURE 13 Vehicle before Test 1185-6.

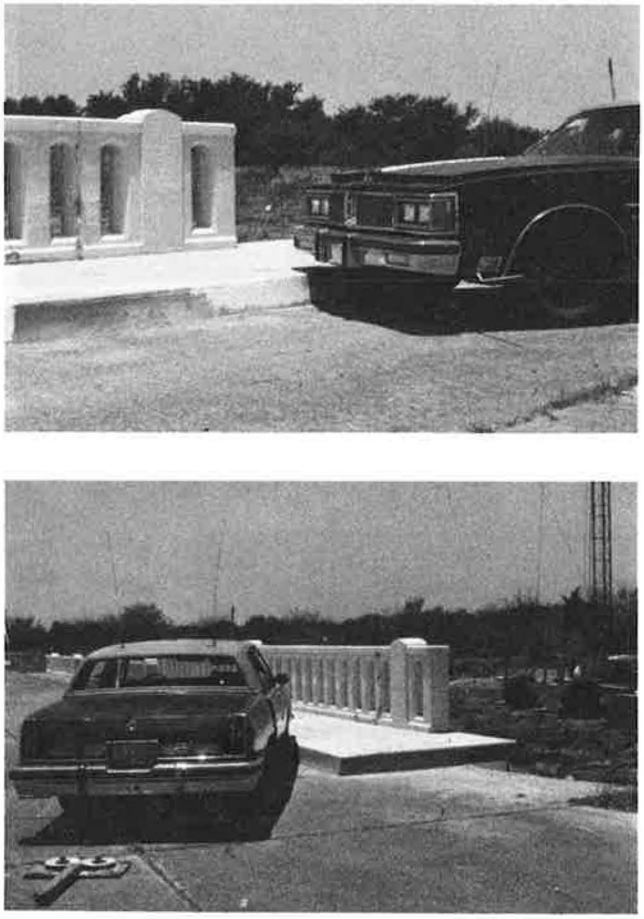


FIGURE 14 Vehicle and bridge rail installation geometrics for Test 1185-6.

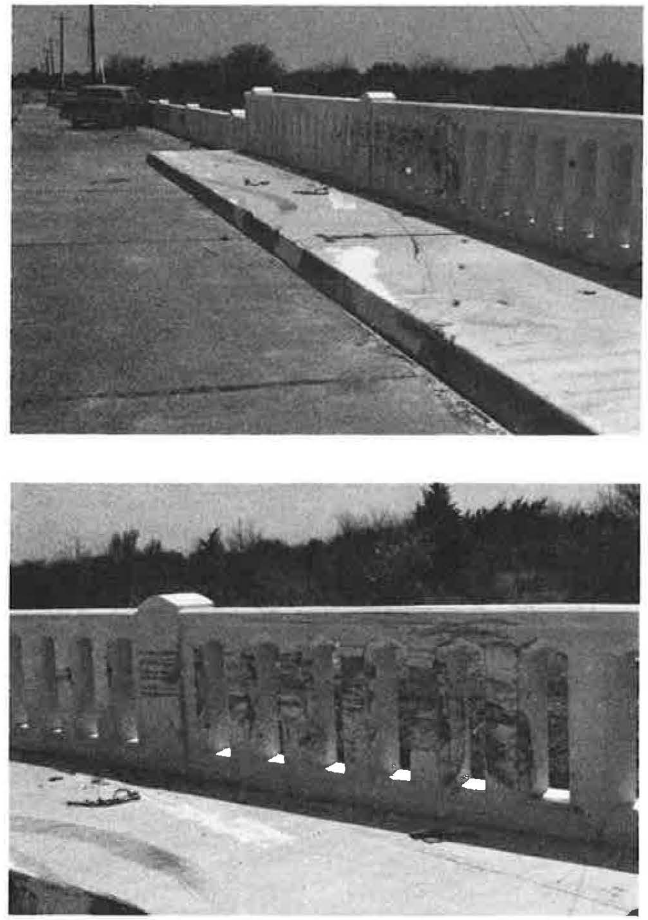


FIGURE 15 Bridge rail after Test 1185-6.

vehicle impacted the bridge rail 14 ft (4.3 m) from the end of the rail, traveling at 46.7 mph (75.1 km/hr). The angle was 22.7 degrees. The right front wheel and tire were mangled in the porthole at 0.237 sec, and the vehicle began to redirect significantly at 0.246 sec. At 0.492 sec, the vehicle was traveling parallel with the bridge rail at a speed of 32.1 mph (51.6 km/hr), and at 0.567 sec the rear of the vehicle hit the bridge rail. Traveling at 28.9 mph (46.5 km/hr), the vehicle lost contact with the bridge rail at 0.658 sec. Exit trajectory was 3.5 degrees. The undercarriage of the vehicle bottomed out on the curb at 0.744 sec, and the vehicle rode off the sidewalk at 1.466 sec after impact. The brakes were then applied and the vehicle yawed clockwise and subsequently came to rest 105 ft (32.0 m) from the point of impact.

As can be seen in Figure 15, the rail received moderate cosmetic damage. Tire marks on the face of the bridge rail extended 15 ft (4.6 m) from the point of impact. Some scraping and gouging occurred along the edges of the portholes.

The vehicle sustained damage to the right side (Figures 16 and 17). Maximum crush at the right front corner at bumper height was 14.0 in. (35.6 cm). The floorpan and subframe of the vehicle were bent, and the right A-arm, tie rod, and sway bar were damaged. The windshield was cracked, and the roof bent. The right front and rear rims were bent, and the tires were damaged. There was damage to the hood, grill, front and rear bumpers, radiator and fan, right front and rear quarter panels, and right front and rear doors.

Results of Test 1185-6

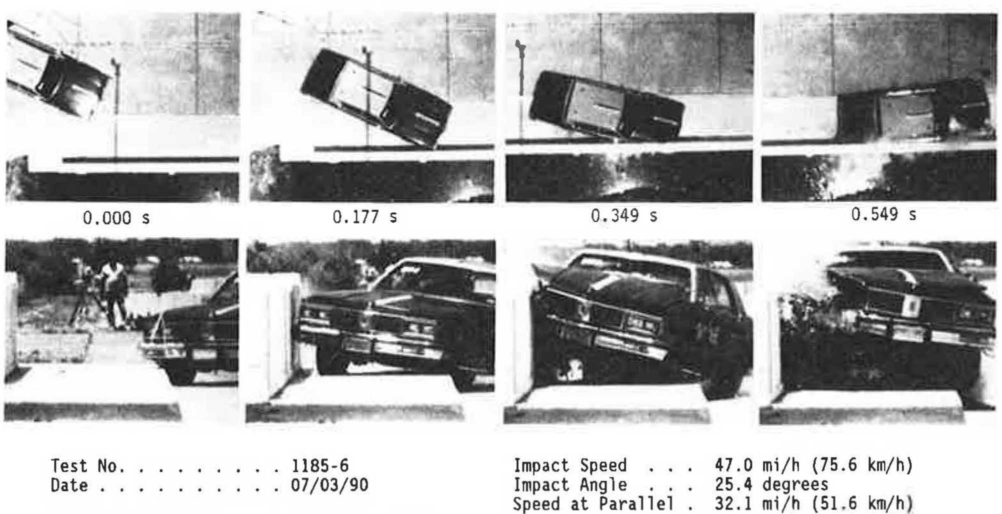
The speed of the vehicle as it struck the curb was 47.0 mph (75.6 km/hr); the angle of impact was 25.4 degrees. Traveling at 46.7 mph (75.1 km/hr), the vehicle struck the bridge rail at 22.7 degrees. The speed of the vehicle as it was parallel to the bridge rail was 32.1 mph (51.6 km/hr). Exit speed was 28.9 mph (46.5 km/hr), and exit trajectory was 3.5 degrees. Occupant impact velocity was 23.2 ft/sec (7.1 m/sec) in the longitudinal direction and 17.1 ft/sec (5.2 m/sec) in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were $-4.8\ g$ (longitudinal) and $8.5\ g$ (lateral). These data and other pertinent information from the test are summarized in Figure 18. These data were further analyzed to obtain 0.050-sec average accelerations versus time. The maximum 0.050-sec averages measured at the center of gravity were $-6.6\ g$ (longitudinal) and $6.2\ g$ (lateral).

Conclusions

The bridge rail contained and smoothly redirected the test vehicle with no lateral movement or cracking. There were no

FIGURE 16 Damage to right side of vehicle, Test 1185-6.

FIGURE 17 Vehicle after Test 1185-6.



Test No.	1185-6
Date	07/03/90
Test Installation . . .	Texas Type C411 Bridge Rail
Installation Length . .	52 ft (16 m)
Vehicle	1982 Oldsmobile 98
Vehicle Weight	
Test Inertia	4,500 lb (2,043 kg
Vehicle Damage Classification	
TAD	01RFQ6
CDC	01FREK3 & OFRFES3
Maximum Vehicle Crush .	14.0 in (35.6 cm)

Impact Speed . . .	47.0 mi/h (75.6 km/h)
Impact Angle . . .	25.4 degrees
Speed at Parallel .	32.1 mi/h (51.6 km/h)
Exit Speed	28.9 mi/h (46.5 km/h)
Exit Trajectory . .	3.5 degrees
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal . .	-6.6 g
Lateral	6.2 g
Occupant Impact Velocity	
Longitudinal . .	23.2 ft/s (7.1 m/s)
Lateral	17.1 ft/s (5.2 m/s)
Occupant Ridedown Accelerations	
Longitudinal . .	-4.8 g
Lateral	8.5 g

FIGURE 18 Summary of results, Test 1185-6.

detached elements or debris to present undue hazard to other vehicles. The vehicle remained upright and relatively stable during the collision. The occupant-compartment impact velocities and 10-m/sec occupant ridedown accelerations were within the standard recommended limits. The vehicle trajectory at loss of contact indicates no intrusion into adjacent traffic lanes.

SUMMARY AND CONCLUSIONS

This study was probably the first time a combination pedestrian-traffic bridge rail mounted on an 8-in.-high curb and 6-ft-wide sidewalk has been designed and crash tested. This rail was developed for use where city streets pass over Federal-aid or Interstate highways or other hazards. The combination pedestrian-traffic rail would only be exposed to vehicles traveling at moderate speeds (45 mph or less). The low-service-level crash test in NCHRP Report 230 (*3*) (SL-1) calls for a 4,500-lb automobile traveling 60 mph and impacting at a 15-degree angle and a second test with a 1,800-lb automobile impacting at 60 mph and a 20-degree angle. The low-performance-level test in the 1989 *Guide Specifications for Bridge Railings* (*4*) (PL-1) calls for a crash test with a 5,400-lb pickup impacting at 45 mph and a 20-degree angle and a second test with a 1,800-lb automobile impacting at 50 mph and a 20-degree angle. Neither of these crash tests was appropriate for this type of traffic rail and its intended location on low-speed urban streets.

Table 1, which is now being considered in two NCHRP-AASHTO research projects, was appropriate because the 1,900-lb automobile and 4,500-lb pickup would impact at 45 mph at 20-degree and 25-degree angles, respectively. (Note that a 4,500-lb automobile was used instead of a pickup.)

The 8-in.-high curb, 6-ft-wide sidewalk, and 42-in.-high combination pedestrian-vehicle bridge rail performed well in the two crash tests. Appendix F of the 1977 *Guide for Selecting, Locating, and Designing Traffic Barriers* (*5*) presents automobile trajectory data predicted by the HVOSM computer model when an automobile strikes curbs of various heights.

The data generated by the HVOSM computer model of an automobile predicted that the two vehicles used in this study would vault so that the bumpers would be 14 in. to 18 in. higher than normal when they struck the bridge rail behind the 8-in.-high curb and 6-ft-wide sidewalk. The crash-test results showed that the Honda bumper was only 4 in. higher than normal and the Oldsmobile bumper was only 3.5 in. to 7.5 in. higher (the bumper was 3.5 in. higher on initial impact and continued to climb to 7.5 in. higher 0.25 sec later). The normal bumper height when parked on a level surface of the Honda was 20.5 in. and that of the Oldsmobile was 20.75 in. During the Oldsmobile test the right front and rear tires blew out, and the wheel rims were bent during the impact with the curb.

In the strength test with the 4,500-lb vehicle at 47.0 mph and a 25.4-degree angle, the changes in speed and angle after the impact with the curb until the impact with the rail were only −0.3 mph and −2.7 degrees. The conclusion is that the effect of the 8-in.-high curb and 6-ft-wide sidewalk on the vehicle impact with the bridge rail in this area was not as significant as originally believed. Different vehicles at different speeds and angles will behave differently.

Although the crash-test variables were not those recommended in the crash-test matrix in NCHRP Report 230 or

TABLE 2 Safety Evaluation of Crash Test 1185-5

Usual Safety Evaluation Criteria		Test Results		Pass/Fail
Must contain vehicle		Vehicle was contained		Pass
Debris shall not penetrate passenger compartment		No debris penetrated passenger compartment		Pass
Passenger compartment must have essentially no deformation		Minimal deformation		Pass
Vehicle must remain upright		Vehicle did remain upright		Pass
Must smoothly redirect the vehicle		Vehicle was redirected		Pass
Effective coefficient of friction (9)				
μ	Assessment	μ	Assessment	
0 - .25	Good	.55	Marginal	Pass
.26 - .35	Fair			
> .35	Marginal			
Shall be less than				
Occupant Impact Velocity - fps		Occupant Impact Velocity - fps		Pass
Longitudinal	Lateral	Longitudinal	Lateral	
30	25	12.1	7.3	
Occupant Ridedown Accelerations - g's		Occupant Ridedown Accelerations - g's		Pass
Longitudinal	Lateral	Longitudinal	Lateral	
15	15	-4.6	7.4	
Exit angle shall be less than 12 degrees		Exit angle was 2.7 degrees		Pass

TABLE 3 Safety Evaluation of Crash Test 1185-6

Usual Safety Evaluation Criteria		Test Results		Pass/Fail
Must contain vehicle		Vehicle was contained		Pass
Debris shall not penetrate passenger compartment		No debris penetrated passenger compartment		Pass
Passenger compartment must have essentially no deformation		Minimal deformation		Pass
Vehicle must remain upright		Vehicle did remain upright		Pass
Must smoothly redirect the vehicle		Vehicle was redirected		Pass
Effective coefficient of friction (9)				
μ	Assessment	μ	Assessment	
0 - .25	Good	.51	Marginal	Pass
.26 - .35	Fair			
> .35	Marginal			
Shall be less than				
Occupant Impact Velocity - fps		Occupant Impact Velocity - fps		Pass
Longitudinal	Lateral	Longitudinal	Lateral	
30	25	23.2	17.1	
Occupant Ridedown Accelerations - g's		Occupant Ridedown Accelerations - g's		Pass
Longitudinal	Lateral	Longitudinal	Lateral	
15	15	-4.8	8.5	
Exit angle shall be less than 15 degrees		Exit angle was 5.0 degrees		Pass

the 1989 *Guide Specifications for Bridge Railings*, the test results are compared in Tables 2 and 3 with the standard safety evaluation criteria presented in those documents. The results of the crash tests indicate that the C411 bridge rail should be safe for use on low-speed (45 mph or less) roads.

ACKNOWLEDGMENTS

This research was conducted under a cooperative program among the Texas Transportation Institute, the Texas State Department of Highways and Public Transportation, and FHWA. Dean Van Landuyt, John J. Panak, and Van M. McElroy were closely involved in all phases of the study.

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Development of a Low-Profile Portable Concrete Barrier

TODD R. GUIDRY AND W. LYNN BEASON

A low-profile portable concrete barrier (PCB) has been developed for use in low-speed [approximately 45 mph (73 km/hr) or less] work zones. The purpose of the low-profile barrier is to shield the work zone and redirect errant vehicles while improving visibility. The low-profile barrier has a total height of only 20 in. (50.8 cm), whereas most current PCBs have a total height of 32 in. (81.28 cm). The primary advantage of the reduced height of the low-profile PCB is that driver visibility is significantly increased. This enhanced visibility should provide drivers with safer conditions and reduce the number of accidents. The performance of the barrier was demonstrated through the results of two full-scale crash tests. Based on the results of these crash tests, the low-profile barrier is recommended for immediate use under appropriate conditions.

As many cities show continued growth, so do their existing roadway systems. As a result, roadway work zones have become commonplace. The work zones disrupt the continuity of traffic flow and thus introduce a hazard for both motorists and workers. As such, work zones are often segregated and delineated by longitudinal barriers capable of redirecting errant vehicles.

Boundaries of work zones are often defined by the use of reflective barrels or portable concrete barriers (PCBs). These systems work well for vehicles traveling along the major roadway through the work zone. However, if cross-traffic access is required, sight-distance problems often occur. A typical example of this problem would occur where openings in the longitudinal barrier are provided to allow cross-traffic access from parking lots and intersecting roads. The heights of typical longitudinal barriers reduce the ability the cross-traffic visibility. This is especially a problem at night, when the barrier obstructs the ability of drivers to see oncoming headlights.

In many cases, the driver of the cross-traffic vehicle must pull into the mainstream of the roadway before being able to see the headlights of oncoming vehicles. This situation has led to many accidents. The objective of this research was to develop a low-profile PCB short enough to alleviate the sight-distance problem while still maintaining a credible redirective ability. This was accomplished by first studying the geometrics of the situation. Studies were then conducted to establish theoretical barrier performance limits for low-profile barriers of various heights. This information was integrated into the workable low-profile barrier design discussed in this report.

The remainder of this report deals with the development, full-scale testing, and recommendations for the use of the new low-profile PCB.

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DEVELOPMENT OF LOW-PROFILE PCB

The purpose of this research was to develop a low-profile segmented PCB for use in low-speed [45 mph (73 km/hr) or less] applications. The design goals for the low-profile PCB are as follows. The low-profile barrier should be short enough so that the barrier does not cause a sight-distance problem for cross traffic. The new low-profile PCB should be capable of redirecting errant vehicles over an appropriate range of vehicle weights, speeds, and impact angles. Texas Department of Transportation (TxDOT) engineers requested that the maximum lateral deflection of the barrier should be held to a minimum. These issues are addressed in the remainder of this section.

It was decided that an unobstructed line of sight between the cross-traffic driver's eye and the center of the headlight of the oncoming vehicle provides the boundary for acceptable barrier performance. To study the sight-distance problem, it was necessary to define headlight heights and other related geometric constraints as described below.

A random survey of 100 vehicles was conducted to establish the range of typical headlight heights. In this study, the headlight height was defined as the measured distance between roadway surface and the center of the headlight. The headlight heights varied for different makes and models of vehicles. Of importance, however, is the range that encompassed most of the vehicle headlights heights and the minimum headlight height. Most of today's cars have headlight heights between 24 and 28 in. (61 cm and 71 cm). None of the vehicles measured had headlight heights less than 24 in. (61 cm). In addition, AASHTO's *A Policy on Geometric Design of Highways and Streets, 1990* suggests that the minimum allowable headlight height is 24 in. (61 cm) (*1*). Therefore, the minimum headlight height of 24 in. (61 cm) was used in the sight-distance analysis.

In addition to the headlight height, it was necessary to know the eye height of the driver of the cross-traffic vehicle. AASHTO requires a driver's design eye height of 42 in. (107 cm) (*1*). Hence, this value was used to generate the results discussed here.

Many other variables affect the sight-distance problem, including the offset of the oncoming vehicle and the offset of the cross-traffic vehicle to the barrier, as shown in Figure 1. Further, the situation depicted in Figure 1 can occur in conjunction with three different geometric conditions: (a) constant slope—flat terrain, (b) sag curve, and (c) crest curve. These geometric conditions are shown in Figure 2.

Simplified geometric analyses were conducted for each of these geometric conditions and a wide range of offset con-

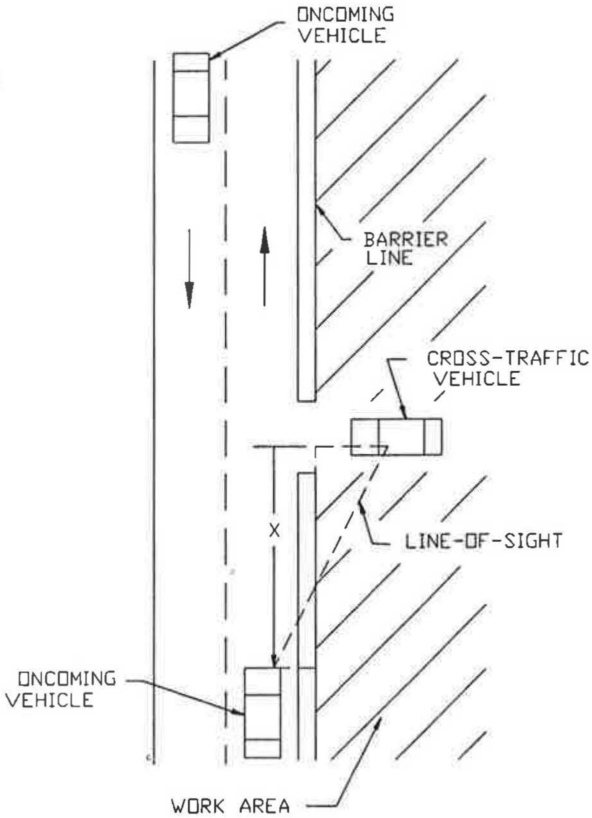


FIGURE 1 Geometry of sight-distance problem.

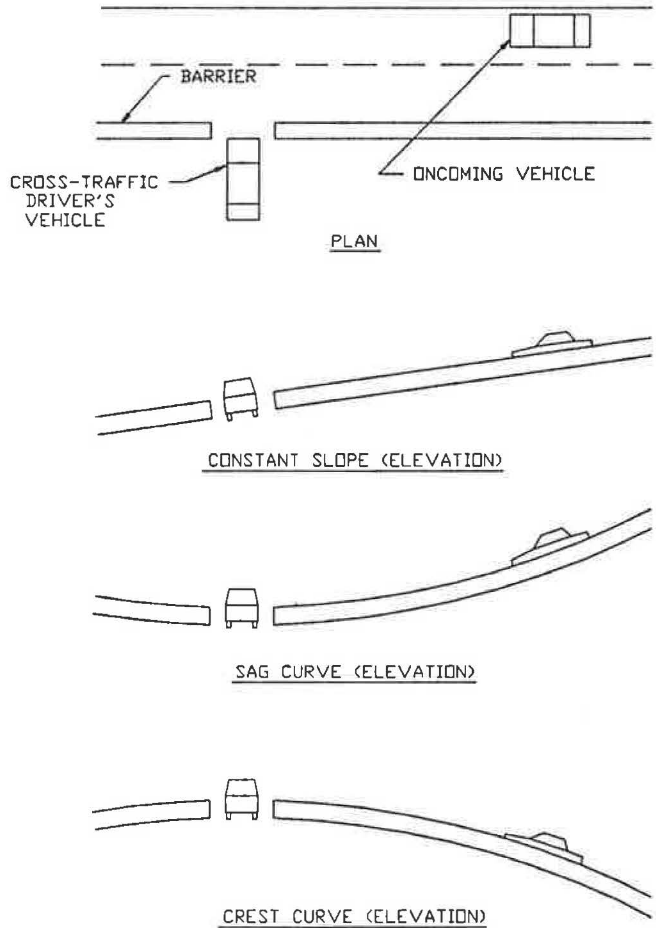


FIGURE 2 Categories of study.

ditions. It was found that the sight distance of cross-traffic drivers is unlimited as long as the barrier height is less than 24 in. (61 cm) (the minimum headlight height) for both constant slope and sag vertical curves. However, in the case of crest vertical curves it was found that the sight distance of cross-traffic drivers is significantly increased by the use of barrier heights less than 24 in. (61 cm). The degree of limitation in this latter case depends to a large extent on the geometric conditions assumed. AASHTO sets limits for crest vertical curve design parameters based on driver comfort, visibility, and stopping sight distance (*1*). These limiting parameters result in minimum curve radii for given design speeds. Cross-traffic driver analyses were done for 45 mph (73 km/hr) AASHTO requirements. In addition, headlight offsets of 2, 14, and 26 ft (0.61, 4.3, and 7.9 m) were examined to represent one, two, and three lanes of oncoming traffic. The AASHTO design stopping sight-distance for a vehicle traveling at 45 mph (73 km/hr) is 325 ft (99 m) (*1*). Results from this analysis showed that a barrier height of 20 in. (51 cm) provided sufficient vision of one or both headlights for the above conditions. Therefore, an overall barrier height of 20 in. (51 cm) is acceptable for 45 mph (73 km/hr) applications. Although the 20 in. (51 cm) barrier meets AASHTO requirements, the cross-traffic driver's visibility is further improved if the barrier height is reduced. On the basis of this sight-distance analysis it was determined to develop a low-profile barrier that is 20 in. (51 cm) tall or shorter.

The first step in the design process was to define appropriate collision criteria for the low-profile barrier in cooperation with TxDOT engineers. After discussion with TxDOT engineers, test conditions were established. Since the low-profile barrier is intended for use in urban work zones where speeds are limited to 45 mph (73 km/hr), it was determined that 45 mph (73 km/hr) provides a reasonable test speed for all conditions. Because of the potentially hazardous consequences associated with failure to redirect, the remainder of the crash test parameters were selected to reflect relatively severe conditions. Therefore, the strength test was established to be a ¾-ton pickup impacting at 45 mph (73 km/hr) at an angle of 25 degrees. It is believed that this test represents a severe set of impact conditions for the proposed application. The stability test was determined to be a 1,800-lb (817-kg) small automobile impacting at 45 mph (73 km/hr) at an angle of 20 degrees. These angles are consistent with current strength and stability tests for full-service barriers.

After the test criteria were established, the research was focused on determining the minimum barrier height that is required to achieve the desired goal.

Preliminary barrier analyses were conducted using computer simulations. The computer program used was HVOSM (Highway-Vehicle-Object-Simulation-Model) (*2*). The RD-2 version of HVOSM was used in the study; modifications developed by researchers at the Texas Transportation Institute (TTI) were incorporated in this version. The TTI modifications permit the structure of the vehicle to interact with the sloped faces of a multifaced rigid barrier. Studies of rigid New

Jersey concrete median barriers (CMBs) made with this modified version of HVOSM have been reasonably successful.

A ¾-ton-pickup computer model was not available at the outset of the project; consequently, a large car model was used in its place for the HVOSM simulations. The simulation results suggested that the minimum acceptable barrier height is 18 in. (46 cm) for the impact criteria discussed above. At 18 in. (46 cm), the large automobile remained stable. At a barrier height of 16 in. (41 cm), the large automobile rolled over the barrier. Since a ¾-ton pickup has a higher center of gravity than a large automobile, it was judged that the barrier should be taller than 18 in. (46 cm). In addition, a barrier height of 20 in. (51 cm) is acceptable for previously mentioned visibility requirements. Therefore, a barrier height of 20 in. (51 cm) was established on the basis of these results and engineering judgment. The authors believe that a barrier height of 20 in. (51 cm) is close to the minimum acceptable height for this application.

In reviewing previous automobile tests on the New Jersey CMB and the single-slope CMB it can be seen that the stability of an impacting vehicle is significantly affected by the shape of the barrier face (*3–5*). Both the New Jersey and single-slope CMBs have sloped sides. The sloped sides induce upward-acting vertical forces on the impact side of the vehicle. These forces, in combination with tire interaction, cause the impact side of the vehicle to rise. This vertical rise imparts a roll motion to the vehicle. The severity of the roll motion depends on the vehicle properties and impact conditions. If the roll motion is severe enough, the vehicle will experience full rollover.

Results of full-scale crash tests show that the impact sides of automobiles, pickups, and suburban-type vehicles do not have a tendency to rise if the barrier face is vertical (*4*). This is the case because the impact forces have relatively small vertical components, and the tire-barrier interaction forces alone are not sufficient to force the impact side of the vehicle to rise. The result is little or no roll motion away from the barrier.

Because of the reduced height of the low-profile barrier, it is important to control the upward vertical displacement of the impact side of the vehicle so that the vehicle does not vault over the barrier. Therefore, a negative slope was cast into the impact surface of the low-profile barrier to prevent vertical displacement of the impact side of the vehicle. The negative slope significantly changes the tire barrier interaction, thus reducing the tendency for the vehicle to rise because of this mechanism. In addition, the vertical component of the impact force acts in a downward direction on the vehicle, which further restricts the tendency for the impact side of the vehicle to rise. Using engineering judgment and simplified analyses it was determined that a negative slope of 1:20 would provide the desired effect.

Keeping the lateral deflections of the barrier to a minimum required an adequate combination of barrier weight and connection moment capacity. The effects of barrier weight and connection moment capacity on the lateral deflections of the low-profile barrier were studied using a simulation program called Simulation of Articulated Barrier Systems (SABS) (*6*). SABS yields deflections of segmented PCBs based on force versus time data derived from similar crash tests. For this study, deflections were determined for barrier segment lengths of 20, 25, and 30 ft (6.1 m, 7.6 m, and 9.1 m). The weight of the barrier was somewhat constrained to be in the 500 to 600 lb/ft (745 to 894 kg/m) range, given the geometric constraints discussed previously. For barrier weights in this range, and a 100,000 ft-lb (136,000 N-M) connection moment capacity, the deflections are approximately the same for all three segment lengths. Therefore, no significant advantage is given by using 25 or 30 ft (7.6 or 9.1 m) segments over the 20 ft (6.1 m) segment for this connection moment capacity. In addition, using a shorter segment allows a reduced turning radius while enhancing barrier maneuverability. Although barrier segments shorter than 20 ft (6.1 m) were not examined (because TxDOT criteria were met with 20 ft lengths), it is believed that shorter segments would probably work in other applications.

These barrier segments are moved by using adequate steel rebar placed though holes located 4 ft from the end of each segment. Chains can be connected to the rebar and the segment can be moved by forklift or light crane. On the basis of these results it was concluded that a combination of a barrier weight of approximately 550 lb/ft (819 kg/m) for a 20-ft (6.1-m) segment and a 100,000-ft.-lb (136,000-N-M) moment connection capacity would appropriately limit lateral deflections to less than 6 in. (15.2 cm). The barrier segment moment capacity is in excess of 100,000 ft-lb (136,000 N-M). As such, maximum lateral barrier deflections are forced to occur at the system's weakest points (i.e., at the barrier segment connections).

On the basis of the previous discussions, the barrier height was established at 20 in. (51 mm), the minimum barrier weight was set at 550 lb/ft (819 kg/m), and the slope of the barrier face was set at a negative 1:20. The resulting barrier cross section is shown in Figure 3. The outline of the New Jersey PCB is also presented in Figure 3 for comparison purposes. The low-profile barrier shape yields an actual weight of approximately 560 lb/ft (834 kg/m).

Several different connection schemes were considered for the new low-profile PCB, including those previously used on many conventional PCBs. However, none of these existing connection details was appropriate. Therefore, a new con-

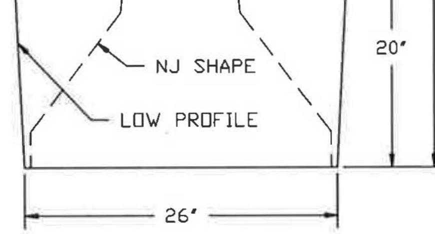


FIGURE 3 Low-profile PCB cross section.

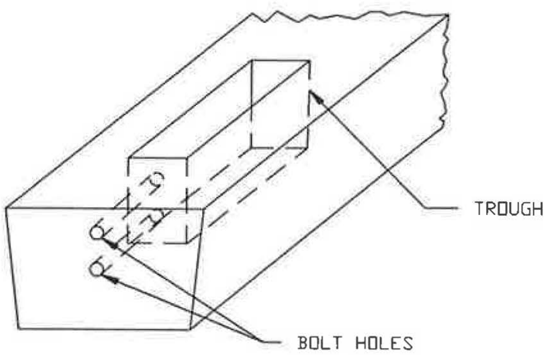


FIGURE 4 Connection details.

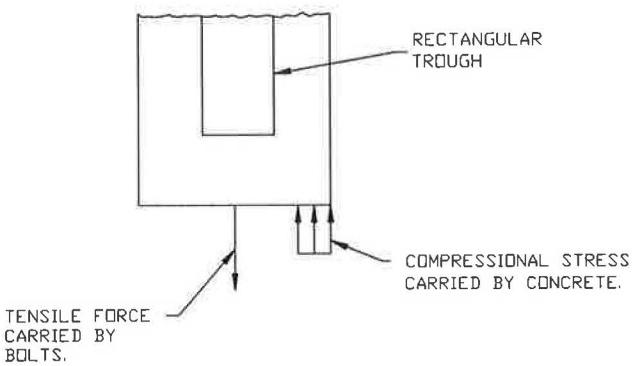


FIGURE 5 Connection loading.

nection detail was developed, as shown in Figure 4. The connection is accomplished by aligning the ends of two barrier segments and inserting two ASTM A36 bolts through the connection holes that are recessed into a rectangular trough that is cast into the end of each segment. This trough allows the bolts to be removed and inserted freely. Drainage in the trough is provided by a hole 1 in. (2.54 cm) in diameter that runs from the bottom of the trough to the barrier drainage slot. When the connection is loaded, a moment develops between the tensile force in the bolts and the compressive force in the extreme concrete fibers, as shown in Figure 5. This connection results in a moment capacity slightly in excess of 100,000 ft-lb (136,000 N-M).

The tolerances in the connection holes were set so that the barrier can be assembled on roadways with moderate vertical and horizontal curves. The barrier connection can tolerate angles up to 4 degrees in both the horizontal and vertical directions. This means that barrier segments 20 ft. (6.1 m) in length can be used to turn horizontal curves with radii of curvature of 150 ft. (46 m).

Complete fabrication details for the new low-profile barrier are shown in Figure 6.

FULL-SCALE CRASH TESTS

Two full-scale crash tests were conducted on the low-profile PCB to evaluate its performance relative to structural adequacy, occupant risk, and vehicle exit trajectory. The first test involved a 4,500-lb (2,043-kg), ¾-ton pickup that impacted the PCB at 45 mph (73 km/hr) at an encroachment angle of 25 degrees. The second test involved a 1,800-lb (817-kg) compact car that impacted the PCB at 45 mph (73 km/hr) at an encroachment angle of 20 degrees.

The tests were conducted using six 20-ft- (6.1-m-) long low-profile concrete segments connected together to form a 120 ft (36.4 m) longitudinal barrier. The segments were placed on the existing concrete surface at the TTI Proving Ground with no positive attachment to the roadway surface.

In both full-scale crash tests, the vehicles impacted the 120 ft (36.4 m) longitudinal barrier at a point located approximately 5 ft (1.5 m) upstream of the middle barrier segment joint. This impact point was chosen to provide the most critical impact situation with respect to both strength and snagging. Test statistics for the two crash tests are summarized in Table 1.

Results from Test 1

In this test, a 1984 GMC Sierra 2500 pickup was directed into the PCB. The test inertia weight of the vehicle was 4,500 lb (2,043 kg), and its gross static weight was also 4,500 lb (2,043 kg). The height to the lower edge of the vehicle bumper was 17.5 in. (44.4 cm), and the height to the upper edge was 26.5 in. (67.3 cm). The vehicle was directed into the barrier using a reverse cable tow and guidance system. The vehicle was freewheeling and unrestrained just before impact.

The vehicle was traveling at a speed of 44.4 mph (71.4 km/hr) when it impacted the barrier. The impact angle was 26.1 degrees. Immediately after impact, the bumper of the vehicle rode up on top of the barrier. At approximately 23 msec after impact, the left front tire impacted the barrier. The barrier began to move laterally at 66 msec, and the vehicle began to redirect at 71 msec after initial impact. The right front tire became airborne at 117 msec, the left front at 133 msec, and the right rear at 217 msec. At approximately 357 msec, the vehicle was traveling parallel to the barrier with a speed of 37.0 mph (59.5 km/hr), and the rear of the vehicle impacted the barrier shortly thereafter. The vehicle exited the barrier at 768 msec, traveling virtually parallel with the barrier at a speed of 34.8 mph (56 km/hr). Sequential photographs of the test are presented elsewhere (7).

Damage to the barrier is shown in Figure 7. The maximum lateral movement of the barrier was 5 in. (12.7 cm) at the impacted (center) joint. At the impacted connection, vehicle bumper interaction resulted in slight damage to the upper edge of the barrier. One segment downstream experienced a shallow delamination. These damages exposed no reinforcing steel and are not considered to be structurally significant.

The vehicle (shown in Figure 8) sustained minimal damage to the left side; however, the floorpan and frame were bent, and the A-arms were damaged. There was also damage to the front bumper, left front quarter panel, left door, left rear quarter panel, and rear bumper. The wheelbase on the left side was shortened from 131.5 in. (3.3 m) to 120.75 in. (3.1 m).

Data from the electronic instrumentation were digitized for evaluation and posttest processing. As stated previously, the impact speed was 44.4 mph (73 km/hr), and the angle of impact was 26.1 degrees. Occupant risk evaluation criteria are described in *NCHRP Report 230*, and limits are placed on these criteria for acceptable performance for tests conducted with 1,800-lb (817-kg) vehicles (*8*). These limits do not apply to tests conducted with 4,500-lb (2,043-kg) vehicles, but they were computed for information purposes. The occupant impact velocity was 21.2 ft/sec (6.5 m/sec) in the longitudinal direction and 16.0 ft/sec (4.9 m/sec) in the lateral direction. The highest 0.010 sec average occupant ridedown accelerations were -6.0 *g* (longitudinal) -11.4 *g* (lateral). These and other pertinent data from this test are presented in Figure 9. Angular displacement data and vehicular accelerations versus time data are presented elsewhere (*7*). The maximum 0.050-sec average accelerations measured near the center of gravity of the vehicle were -5.6 *g* (longitudinal) and -7.7 *g* (lateral).

After impact, the vehicle redirected and did not penetrate, vault, or roll over the barrier. The barrier moved laterally 5 in. (12.7 cm). There were no detached elements or debris to show potential for penetration of the occupant compartment or to present undue hazard to other vehicles. The vehicle remained upright and stable during contact with the barrier and after exiting the test installation. The vehicle trajectory at loss of contact indicates minimum intrusion into adjacent traffic lanes.

Results from Test 2

In this test a 1981 Honda Civic was directed into the low-profile PCB deployed in a temporary configuration. The test inertia weight of the vehicle was 1,800 lb (817 kg), and its gross static weight was 1,965 lb (892 kg). The height to the lower edge of the vehicle bumper was 14.0 in. (35.6 cm), and the height to the upper edge was 19.5 in. (49.5 cm). The vehicle was directed into the barrier using a cable reverse tow and guidance system. The vehicle was freewheeling and unrestrained just before impact.

The vehicle was traveling at a speed of 45.7 mph (73.5 km/hr) when it impacted the barrier. The impact angle was 21.3 degrees. At approximately 27 msec after impact, the left front

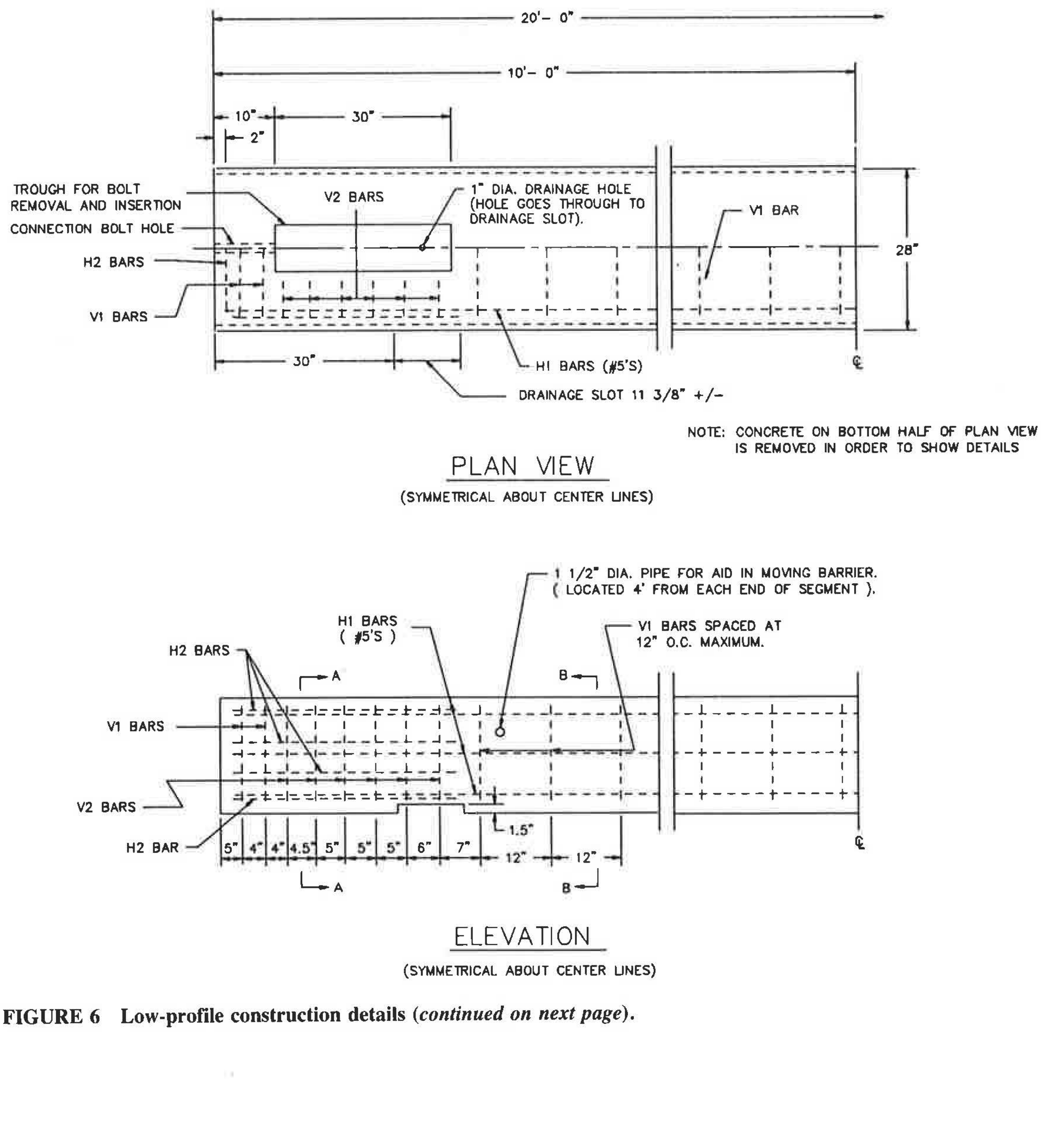
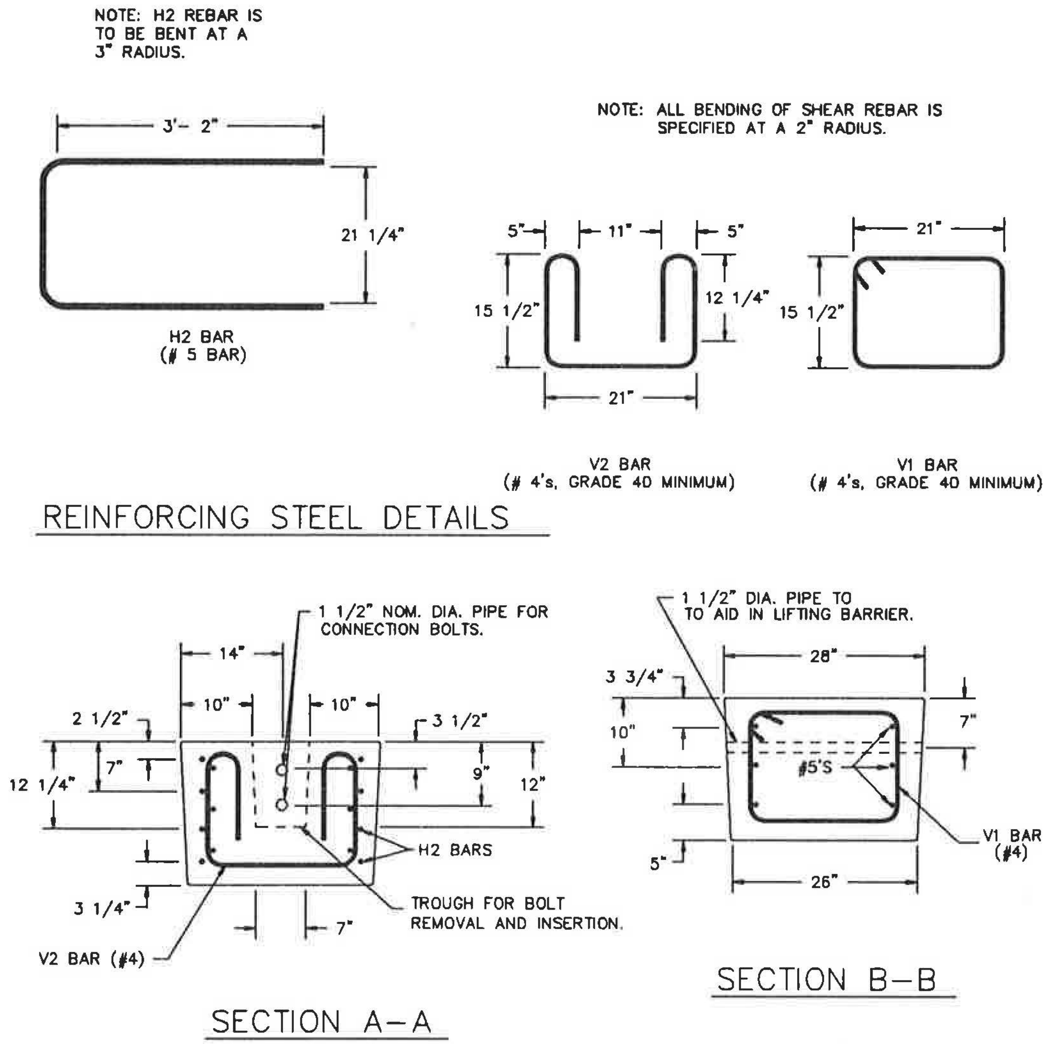


FIGURE 6 Low-profile construction details (*continued on next page*).



GENERAL NOTES

1. ALL CONCRETE SHALL BE CLASS A, C, OR H, UNLESS OTHERWISE SPECIFIED.
2. ALL REINFORCING STEEL SHALL BE GRADE 60, UNLESS OTHERWISE SPECIFIED.
3. CHAMFER END EDGES 3/4".

FIGURE 6 *(continued).*

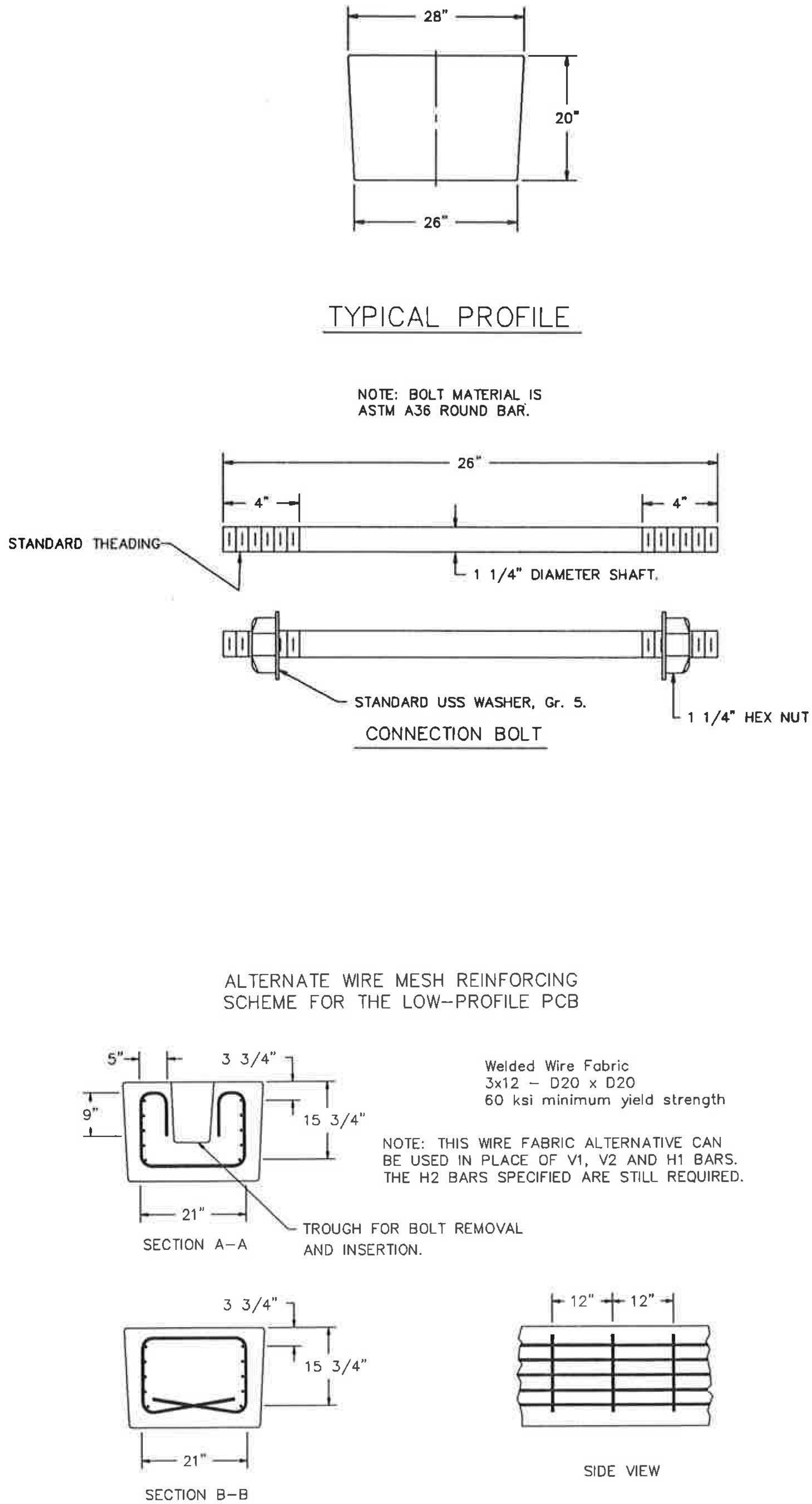


FIGURE 6 ***(continued)*****.**

TABLE 1 Summary of Crash Test Results

Test No.	9901F-1	9901F-2
Vehicle Weight, lb (kg)	4500(2043)	1800(817)
Impact Speed, mph (km/hr)	44.4(71.4)	45.7(73.5)
Impact Angle, degrees	26.1	21.3
Exit Angle, degrees	0.0	7.4
Displacement, in (cm)	5.0(12.7)	0.0(0.0)
Occupant Impact Velocity ft/s (m/s)		
Longitudinal	21.2(6.5)	11.7(3.6)
Lateral	16.0(4.9)	18.6(5.7)
Occupant Ridedown Acceleration g's		
Longitudinal	-6.0	-1.1
Lateral	-11.4	-8.7
Vehicle Damage Classification		
TAD	11FL1	11LD3
CDC	11FLLK1 & 11LDLW1	11FLEK2 & 11LDEW3

tire impacted the barrier, and at 40 msec the vehicle began to redirect. The right side of the vehicle began to lift at 125 msec. At approximately 174 msec, the vehicle was traveling parallel to the barrier at a speed of 39.6 mph (63.7 km/hr). The rear of the vehicle impacted the barrier at 202 msec, and the vehicle exited the barrier at 366 msec, traveling 7.4 degrees away from the barrier at a speed of 38.2 mph (61.5 km/hr). Sequential photographs of the test are presented elsewhere (*7*).

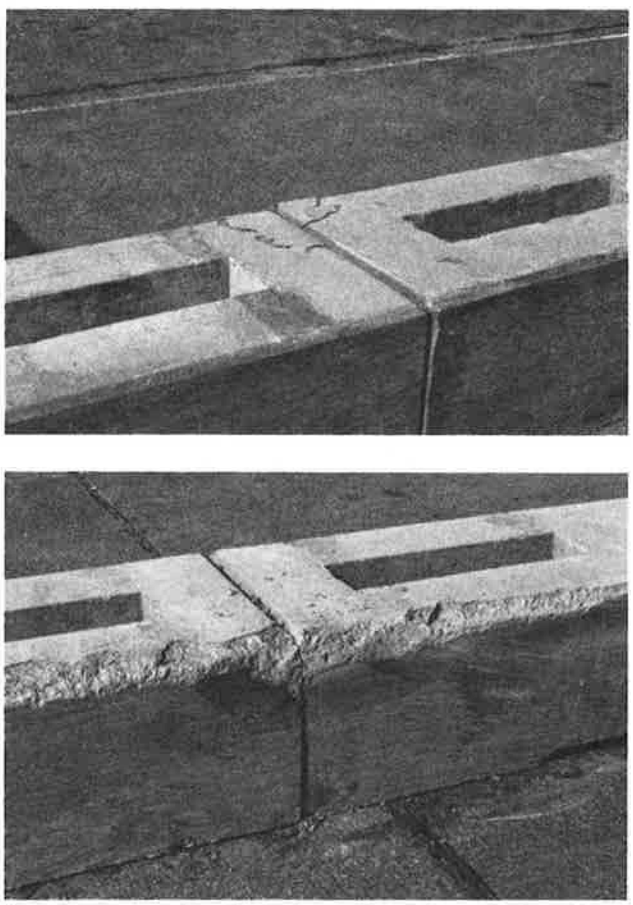


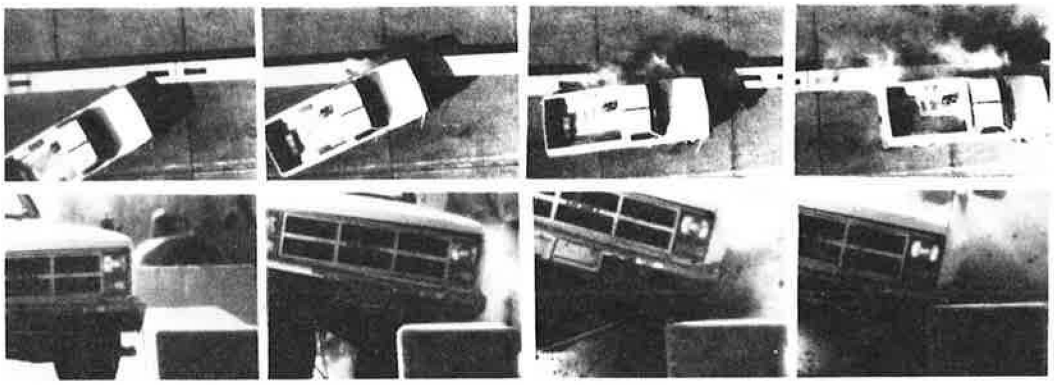
FIGURE 7 Damage at joints, Test 1.

FIGURE 8 Vehicle after Test 1.

The barrier received no significant damage, as shown in Figure 10. There was no measurable lateral movement of the barrier. The vehicle sustained moderate damage to the left side, as shown in Figure 11. The left strut and stabilizer bar were damaged. The front bumper, grill, left front quarter panel, left door, left rear quarter panel, and rear bumper were also damaged.

Data from the electronic instrumentation were digitized for evaluation and posttest processing. As stated previously, the impact speed was 45.7 mph (73.5 km/hr), and the angle of impact was 21.3 degrees. Occupant risk evaluation criteria are described in *NCHRP Report 230,* and limits are placed on these criteria for acceptable performance for tests conducted with 1,800-lb (817 kg) vehicles impacting at 15-degree angles (*8*). These limits do not apply to this set of test conditions; they were computed for information only. The occupant impact velocity was 11.7 ft/sec (3.6 m/sec) in the longitudinal direction and 18.6 ft/sec (5.7 m/sec) in the lateral direction. The highest 0.010-sec average occupant ridedown accelerations were $-1.1\ g$ (longitudinal) and $-8.7\ g$ (lateral). These and other pertinent data from this test are presented in Figure 12.

Vehicle angular displacements and vehicular accelerations versus time traces filtered at 300 Hz are presented elsewhere (*7*). The maximum 0.050-sec average accelerations measured near the center of gravity of the vehicle were $-4.5\ g$ (longitudinal) and $-9.1\ g$ (lateral).



Test No	9901F-1
Date	01/17/91
Test Installation	Low Profile Barrier
Installation Length	120 ft (37 m)
Maximum movement	5 in. (12.7 cm)
Vehicle	1984 GMC Pickup
Vehicle Weight	
Test Inertia	4,500 lb (2,043 kg)
Gross Static	4,500 lb (2,043 kg)
Vehicle Damage Classification	
TAD	11FL1
CDC	11FLLK1 & 11LDLW1
Maximum Vehicle Crush	3.0 in. (7.6 cm)
Impact Speed	44.4 mi/h (71.4 km/h)
Impact Angle	26.1 degrees
Speed at Parallel	37.0 mi/h (59.5 km/h)
Exit Speed	34.8 mi/h (56.0 km/h)
Exit Trajectory	0 degrees
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal	-5.6 g
Lateral	-7.7 g
Occupant Impact Velocity	
Longitudinal	21.2 ft/s (6.5 m/s)
Lateral	16.0 ft/s (4.0 m/s)
Occupant Ridedown Accelerations	
Longitudinal	-6.0 g
Lateral	-11.4 g

FIGURE 9 Summary of results for Test 1.

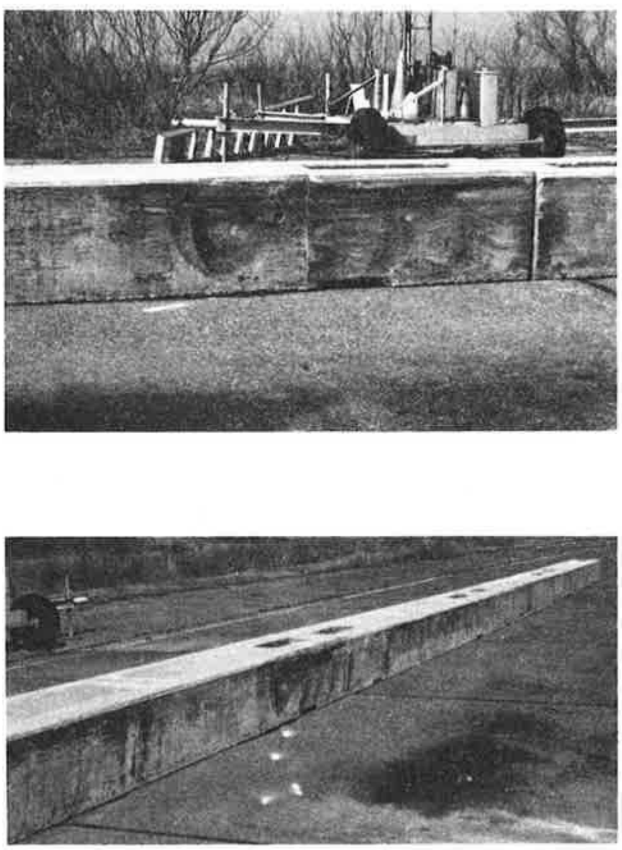


FIGURE 10 Barrier after Test 2.

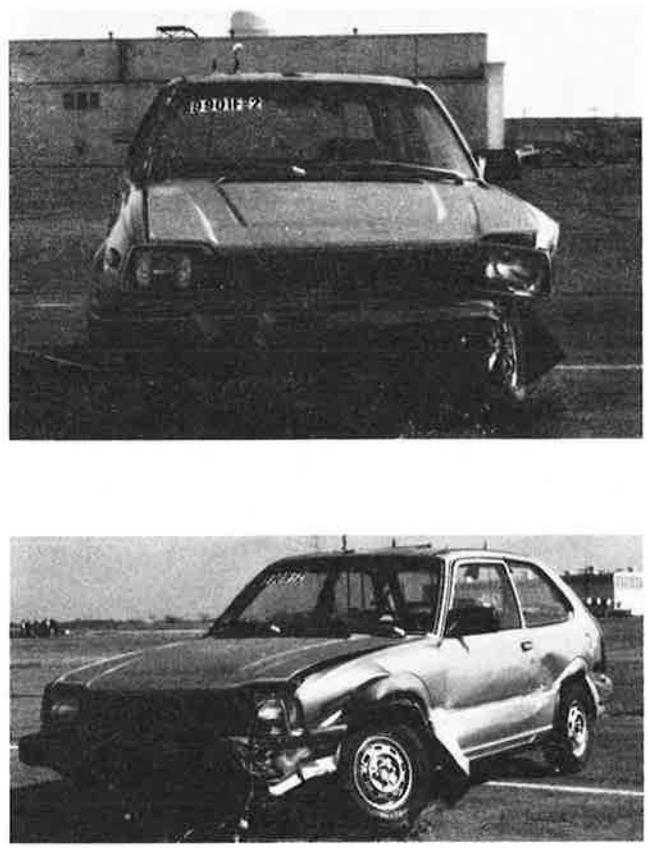
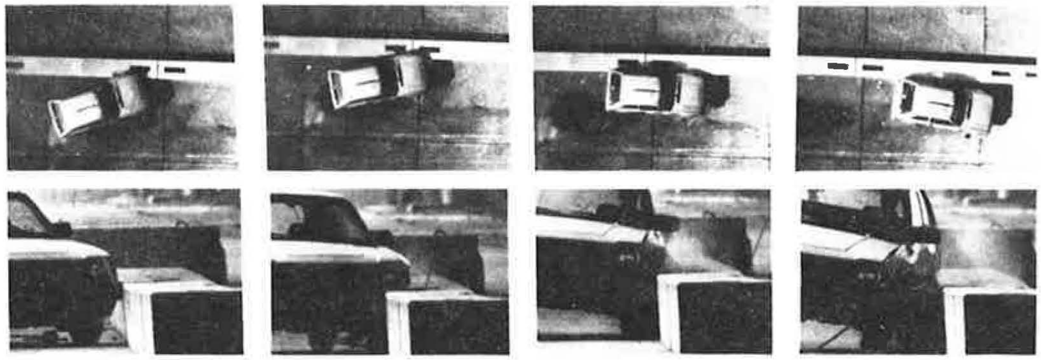


FIGURE 11 Vehicle after Test 2.



Test No	9901F-2
Date	01/25/91
Test Installation	Low Profile Barrier
Installation Length	120 ft (37 m)
Maximum movement	0 in. (0 cm)
Vehicle	1981 Honda Civic
Vehicle Weight	
Test Inertia	1,800 lb (817 kg)
Gross Static	1,965 lb (892 kg)
Vehicle Damage Classification	
TAD	11LD3
CDC	11FLEK2 & 11LDEW3
Maximum Vehicle Crush	8.0 in. (20.3 cm)

Impact Speed	45.7 mi/h (73.5 km/h)
Impact Angle	21.3 degrees
Speed at Parallel	39.6 mi/h (63.7 km/h)
Exit Speed	38.2 mi/h (61.5 km/h)
Exit Trajectory	7.4 degrees
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal	-4.5 g
Lateral	-9.1 g
Occupant Impact Velocity	
Longitudinal	11.7 ft/s (3.6 m/s)
Lateral	18.6 ft/s (5.7 m/s)
Occupant Ridedown Accelerations	
Longitudinal	-1.1 g
Lateral	-8.7 g

FIGURE 12 Summary of results for Test 2.

After impact, the vehicle redirected and did not penetrate, vault, or roll over the barrier. There was no measurable movement of the barrier. There were no detached elements or debris to show potential for penetration of the occupant compartment or to present undue hazard to other vehicles. The vehicle remained upright and stable during impact with the barrier and after exiting the test installation. There was no deformation or intrusion into the occupant compartment. The vehicle exited the barrier traveling 7.4 degrees away from the barrier. The vehicle trajectory at loss of contact indicates minimum intrusion into the adjacent traffic lanes.

CONCLUSIONS

A low-profile PCB has been developed; it is designed for impacts ranging from 1,800-lb (817-kg) compact automobiles to 4,500-lb (2,043-kg), ¾-ton pickups. The test conditions for the ¾-ton pickup were 45 mph (73 km/hr) at a 25-degree encroachment angle. The test conditions for the small car were 45 mph (73 km/hr) at a 20-degree encroachment angle. It is believed that these are severe test conditions for the urban application in which vehicle speeds are limited to 45 mph (73 km/hr). The tests prove that the barrier can withstand these impacts without any vaulting or rolling of the vehicle and without any significant damage to the barrier.

In both full-scale crash tests, the vehicles were smoothly redirected. The largest deflection of the barrier was 5 in. (12.7 mm), which resulted from the impact of the ¾-ton pickup. No measurable deflection occurred in the small-car test. All test results fell within acceptable limits of occupant and vehicle accelerations according to *NCHRP Report 230* (*8*). Therefore, the low-profile PCB is recommended for immediate use.

The primary advantage of the low-profile PCB is that it significantly improves the site distance situation for the drivers attempting to enter or exit a work zone delineated with PCB barriers. The critical site-distance situation was judged to be the lateral visibility of a cross-traffic driver attempting to enter the work zone at night. Specifically, the new low-profile PCB was designed to not interfere with the sighting of headlights of oncoming traffic at night. In addition, the daytime visibility is significantly improved. The improved visibility provided by the use of the low-profile PCB will allow drivers to see oncoming vehicles at night and during the day and to avoid a potentially hazardous situation. In addition to this advantage, a reasonable level of safety in the work zone is maintained by preventing the intrusion of errant vehicles into the work area.

TxDOT engineers believe there are also permanent uses for the low-profile barrier in urban situations and in some areas adjacent to freeways. The PCB can be easily converted to permanent use including slip forming the shape without connections or permanently anchoring the barrier to the roadway.

The new low-profile barrier presents a major advance for urban work zones in which vehicle speeds are limited to 45 mph or less. It is perceived that there is a need for a similar low-profile barrier for higher speed applications. Although the redirective capabilities of the 20-in. (51-cm) low-profile PCB may not be sufficient for use in high-speed work zones, it is believed that a 24-in. (61-cm) version of the low-profile barrier would be able to redirect a 4,500-lb (2,043-kg) vehicle impacting at an angle of 25 degrees and a speed of 60 mph (96 km/hr). Therefore, it is suggested that future research efforts be directed toward the development and testing of a 24-in. (61-cm), full-service, low-profile barrier. In addition, a significant effort is ongoing to develop an end treatment for the new low-profile PCB that will not inhibit required cross-traffic visibility.

ACKNOWLEDGMENTS

This study was conducted under the sponsorship of the Texas State Department of Highways and Public Transportation. Mark Marek, Harold Cooner, and Gary Humes of TxDOT worked closely with the researchers. In addition, the efforts of Hayes E. Ross and Don L. Ivey made the research possible. Their comments, suggestions, and cooperative spirit are appreciated.

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Construction of the Narrow Connecticut Impact-Attenuation System at Five High-Hazard Locations

ERIC C. LOHREY

In an ongoing effort to develop improved vehicular impact-attenuation devices, the Connecticut Department of Transportation has designed, tested, and installed in the field a new and unique crash cushion know as the Narrow Connecticut Impact-Attenuation System (NCIAS). NCIAS is the third and latest device to be introduced to the family of Connecticut impact attenuators. Like the first truck-mounted device (Connecticut Crash Cushion) and the second wide stationary device (Connecticut Impact-Attenuation System), steel cylinders of various wall thicknesses are used as the energy-absorbing medium in NCIAS. Unlike the first two devices, NCIAS incorporates eight steel cylinders that are arranged and connected in a single row to protect motorists from striking narrow and rigid roadside features, such as bridge piers and blunt ends of concrete longitudinal barriers. The completed system is 3-ft wide × 24-ft long, which facilitates its use in width-restricted hazard areas. The shop fabrication of NCIAS units and their subsequent construction and installation at five high-hazard expressway locations in Connecticut are described in this paper, the fourth in a series of publications on NCIAS. NCIAS has been approved by FHWA for use as an experimental safety appurtenance on Federal-aid highway projects and has been installed at two locations in Tennessee in addition to the five locations described here. The operational and safety performance of these installations will be monitored for a 3-year field evaluation period.

In a continuing effort to improve the safety of the highway environment, research personnel from the Connecticut Department of Transportation (ConnDOT) have introduced a new and unique vehicular crash cushion, known as the Narrow Connecticut Impact-Attenuation System (NCIAS). NCIAS is the third in a series of cylindrical steel impact-attenuation devices designed by John F. Carney III and developed by ConnDOT in cooperation with FHWA. In the mid-1970s, a mobile, truck-mounted attenuator (TMA), using four steel cylinders as the energy absorbing medium, was crash tested and subsequently used in the field by ConnDOT. This TMA became known as the Connecticut Crash Cushion and is now in widespread use by ConnDOT and other highway agencies for protection of slow-moving and stationary maintenance and construction operations (*1*).

Using the crushable-steel-cylinder concept, Carney designed a stationary crash cushion consisting of 14 steel cylinders arranged in a wedge-shaped cluster. This design, which became known as the Connecticut Impact-Attenuation System (CIAS), passed a complete crash-test and field-evaluation program conducted by ConnDOT during a 5-year period (*2*). Because of the outstanding safety performance during the first 2 years of field service, CIAS was designated operational by FHWA in 1986. Since its initial installations in 1984, CIAS has been extremely successful in preventing injuries to occupants of impacting vehicles, regardless of the severity of the accident (*3,4*).

Because of these favorable results, work began on the development of a crash cushion for use at width-restricted locations (i.e., locations too narrow for installation of CIAS). Based on scale-model impact tests, Carney designed a system of eight steel cylinders arranged in a single row (*5*). This design was then built to size and subjected to a full-scale crash-test program under the guidelines of NCHRP Report 230 (*6*). After a few design changes during the test program, the system, NCIAS, successfully satisfied the performance requirements and was subsequently approved by FHWA for field deployment as an experimental crash cushion (*7*). Both CIAS and NCIAS are eligible for installation on Federal-aid highway projects. In addition, all three Connecticut systems are nonproprietary; any government agency or highway authority can fabricate and use them without restriction. Each has been patented by the state of Connecticut in cooperation with FHWA.

The shop fabrication and field installation of five NCIAS units at high-hazard sites in Connecticut are described here. The objective is to provide a guide to familiarize users with field construction and installation procedures. Because NCIAS is new and unique, state engineers may be hesitant to deploy the system because of the uncertainty of unforeseen construction difficulties. Issued as specific documentation of previously installed units, this paper is intended to encourage the use of NCIAS and to supplement the other development reports. In addition to the five Connecticut sites described herein, two units have also been installed in Tennessee (*8*). These locations will be closely monitored to evaluate the safety and operational performance of NCIAS under field conditions.

DESCRIPTION OF NCIAS

NCIAS consists of five basic groups of components. Figure 1 shows a plan-view schematic diagram of NCIAS; major components are labeled.

Connecticut Department of Transportation, Office of Research and Materials, 280 West Street, Rocky Hill, Conn. 06067.

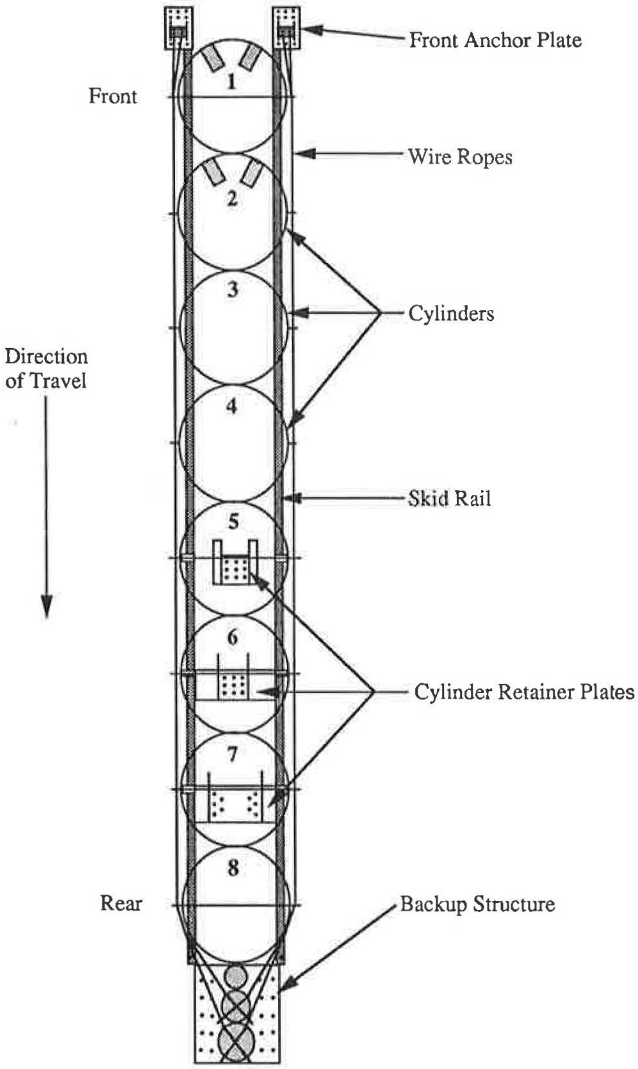

FIGURE 1 NCIAS plan-view schematic.

Concrete Base Pad and Barrier Curb End Treatment

NCIAS must be securely anchored to a sound concrete pad to ensure proper performance. The pad used by ConnDOT is 30 ft long, 10 ft wide, and designed to resist heavy uplifting and overturning loads, which may be incurred during severe side impacts. The pad design used by Tennessee DOT incorporates a concrete block, deadweight approach, which is used with other commercially available crash cushions (*8*). The design selected for a specific location should be based on the existing site characteristics.

A tapered nose piece (barrier curb end treatment) was constructed to provide a smooth redirecting transition from NCIAS to the protected object. The nose piece is designed to be attached to a standard, single, 24- × 32-in. concrete barrier curb section, but can be connected to other structures, such as bridge piers and parapets. The end treatment can be precast or cast-in-place, depending on site conditions.

Anchored Components

These parts are semipermanently bolted to the base pad with ⅞-in. chemically anchored studs and are intended to remain undamaged during a system impact. The following parts are included in this group.

The free-standing backup structure is anchored at the rear of the system and consists of three concrete-filled pipes. It serves as the rear anchorage for the wire ropes and provides a smooth transition between NCIAS and the tapered end treatment.

Two skid rails are positioned parallel to the longitudinal centerline of the system and anchored at each end. They allow the energy-absorbing cylinders to slide freely during the collapsing process.

Three cylinder retainer plates are anchored along the longitudinal centerline of NCIAS; they are positioned in Cylinders 5–7, as shown in Figure 1. They provide lateral stiffness to NCIAS during side impacts, but do not restrict the system's collapse during head-on impacts.

Finally, two front anchor plates, located in the front of the system, serve as the front anchorage for the wire ropes.

Cylinders

Eight steel cylinders are used as the energy-absorbing material in NCIAS. All cylinders are 3 ft in diameter and 4 ft high; they have wall thicknesses ranging from ⅛ to ⅜ in. Cylinders 1 and 2 contain box-beam members, which force the cylinders to wrap around and capture the hood of an impacting vehicle. Cylinders 5, 6, and 7 have retainer clips welded to their inside walls; these clips engage with the retainer plates during side impacts. Other internal cylinder components include stiffening pipes inside Cylinders 5, 6, and 7. These pipes also provide lateral rigidity to the system during side impacts, but do not hinder the collapse during a head-on impact. Cylinder 8 contains a combination tension/compression pipe, which provides stiffness to NCIAS under all impact conditions. The cylinders weigh between 200 and 600 lb, and each is numbered on the inside wall with its position in the array. They are connected to each other with two ⅞- × 2-in. bolts, and the rear cylinder (Cylinder 8) is attached to the backup structure with four ⅞-in. nuts.

Wire Ropes

To control lateral deflection of NCIAS and provide a smooth redirecting response under side-impact conditions, two 1-in.-diameter wire ropes are placed along each side of the system. Each wire rope passes through eyebolts on the sides of Cylinders 1 and 8 and through U-bolts on the sides of Cylinders 2–7. At the front of the assembly, each wire-rope end has a closed-swage-socket fitting for attachment to the front anchor plates, and each end at the rear has a threaded stud fitting for attachment to the backup structure.

Cover

To prevent the buildup of snow, ice, and debris inside the cylinders, a vinyl-coated polyester cover is attached to the top of the cylinder array. The cover has sewn-on straps, which fold down and attach to the cylinders by means of chrome-plated steel clips and aluminum pop rivets. NCIAS units in snow-free areas need not have a cover, provided that they are checked periodically to ensure that no debris collects in the cylinders.

All fabricated parts are made from standard carbon grade steel in readily available shapes. Current prices for the individual parts are presented in the section on costs.

NCIAS SITE SELECTION

NCIAS is suitable for placement in front of a variety of narrow rigid roadside hazards, such as bridge piers, parapets, and exposed ends of longitudinal concrete barriers. Since NCIAS includes a stand-alone backup structure, it does not rely on other objects for anchorage. The situation ideally suited for deployment of NCIAS is a highway bifurcation divided by a concrete, Jersey-shaped barrier. Tests have shown that when these barrier ends are vertically sloped, the result is an extremely dangerous ramping response when impacted. For this reason, crash cushions are the best protection for these locations. A maximum hazard width of 2 ft is permitted at NCIAS installation sites. If longitudinal space is available, wider hazards can be tapered down to a single 24- × 32-in. barrier curb section. All site appurtenances and their orientation to adjacent travel lanes must conform to AASHTO guides (*9,10*).

To date, NCIAS has been installed at five hazardous expressway locations in Connecticut. These areas were selected on the basis of their physical suitability for NCIAS and the accident histories of previous crash cushions at these locations. All five sites are in gore areas formed by exit ramps and mainline expressways where exposed bridge-parapet ends present the rigid hazard. Data for each site are presented in Table 1.

INSTALLATION

Site Preparation

The first step in the NCIAS installation process is to set up a safe and proper work zone around the hazard location. At the five sites discussed here, temporary concrete barriers were placed in a V-shaped array, which enclosed and protected the entire pad excavation area. Pointing toward oncoming traffic, the vertex of the barrier arrangement was shielded with an array of sand modules. These modules were the same ones that previously existed at the sites, except at Site 2, where new ones were required. At Site 2, a Hi-Dro Sandwich System was removed and salvaged for spare parts. In addition to the shielding of personnel in the work area, it is imperative that construction-zone hazards are not introduced to the motoring public.

Pad Construction

Once the sites were prepared for installation, the NCIAS pad location at each was established by first constructing a centerline coinciding with the center of the fixed hazard. The centerline extended upstream from the fixed hazard toward the vertex of the gore area and was equidistant from the two travelway edges. The 10- × 30-ft pad was then placed axisymetrically about the centerline with the rear edge oriented toward the hazard end treatment. Once established, the

TABLE 1 Site Information

Site Number	Route and Direction	Milepost	Average Daily Traffic	Exit Number and Direction	Exit Destination	Town	Type of Attenuator Replaced
1	2 Eastbound	5.15	55,800	7 Left	Route 17 Southbound	Glastonbury	Sand Module Barrier
2	I-91 Southbound	0.14	84,900	1 Right	Route 34 Westbound	New Haven	Hi-Dro ® Sandwich System
3	8 Northbound	26.30	43,800	28 Right	Union Street	Naugatuck	Sand Module Barrier
4	I-84 Eastbound	31.85	80,300	19 Right	Route 8 Southbound	Waterbury	Sand Module Barrier
5	I-84 Eastbound	31.92	77,300	20 Left	Route 8 Northbound	Waterbury	Sand Module Barrier

boundaries of the pad were sawn and its area excavated to the proper depth. After the excavation was cleared out and properly compacted, the reinforcing steel cage was placed in conformance with the project plans. The position of the rebars in the top mat was adjusted to be clear of the anticipated locations of the anchor bolt holes. This was done to avoid hitting the steel when drilling the anchor bolt holes in the finished concrete. It was only marginally successful because of difficulty in holding the locations of the rebar to such close tolerances while people were walking on the cage during concrete placement. At Sites 2–4, the mainline and ramp are at different elevations, which created the need for a sloped pad. Because a level surface is desired under crash cushions, these three pads were constructed level under NCIAS and tapered down to the lower elevation on the appropriate side. Once the rebars were placed and positioned correctly, concrete was poured in each excavation, vibrated, and leveled to the surrounding elevations. The pad concrete was then allowed to cure for at least 7 days before it was drilled.

NCIAS Layout on Pad

The anchored components of NCIAS must be properly positioned on the pad to ensure that no gaps are created between the end treatment of the barrier curb and NCIAS. To begin, the backup structure was placed on the pad flush with the end treatment and centered. Using the actual base plate of the backup structure as a template, the 20 anchor-bolt holes were marked on the pad. The locations of the remaining anchored components could then be measured off the pad centerline and the backup structure. The actual components were used as templates for marking the hole locations. For the first installations, the cylinders were placed on the skid rails to ensure that the retainer plates and front anchor plates were in their proper positions relative to the cylinders. The retainer clips that are welded to the inside of Cylinders 5, 6, and 7 must align properly with their respective retainer plates. The front anchor plates must also be positioned properly. Once all anchor-bolt-hole locations were marked out on the pad, all components were removed so the holes could be drilled.

Anchoring of Base Components

The base components of NCIAS are anchored to the pad with ⅞-in.-diameter chemically anchored bolts, available from several manufacturers. With the locations marked on the pad, the bolt holes were drilled to the proper depth. The ASTM A325 anchor bolts used at the sites are 12 in. long, so the holes were drilled 9½ to 10 in. deep. An air-powered rotary-impact hammer drill with a 1-in.-diameter bit was used for the holes to provide the best adhesion surface for the anchor resin. Conventional rotary drills are not recommended because concrete powder becomes embedded in the walls of the holes; air-impact drills blow the powder out. When reinforcing steel was encountered during drilling, an impregnated diamond or carbide core bit of the same diameter was required to penetrate though the bar. These water-cooled bits worked well, but required much more drilling time than holes in which

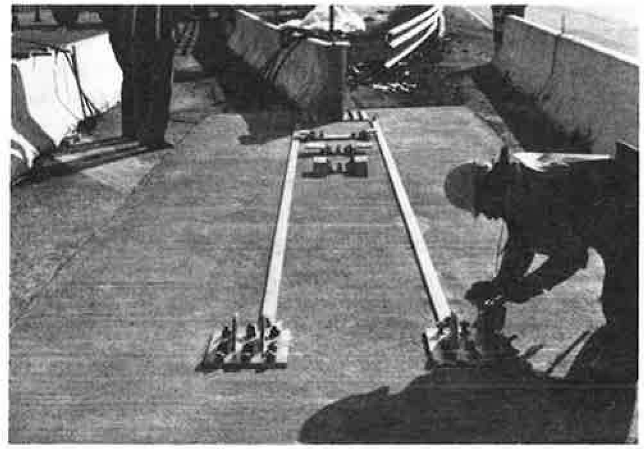

FIGURE 2 Anchored components.

no steel was encountered. For this reason, the rebars should be moved clear of the bolt locations before the concrete pad is poured. On the average for the five sites, steel rebars were struck in about 10 percent of the holes.

Once the holes were drilled to the proper depth, they were blown out with compressed air to remove all dust and debris. The anchor bolt resin was then mixed and poured into the holes in strict conformance with the manufacturer's instructions. The anchored components were left in place to hold the anchor bolts while the resin cured. Most common anchor resins set rapidly, so care must be taken to ensure that each bolt is in its proper position. The resin was allowed to cure overnight; the anchor bolts were then tightened with the anchored components in place. All nuts, with washers, were loaded to a minimum torque of 75 lbf-ft, and 1 of every 10 was proof-loaded to 125 lbf-ft to ensure solid anchorage of the bolts. As with the lugs of car tires, the nuts were tightened in a crisscross pattern to obtain uniform contact pressure between the plates and the pad. The finished pad with all anchored components securely fastened is shown in Figure 2. NCIAS is designed such that the cylinders sustain all the damage to the system when it is struck by a vehicle. The anchored components are intended to remain undamaged through many impacts, depending on the individual site conditions. Because the anchored components are key to the system's performance, they must be thoroughly inspected for damage and loose anchorage after each impact.

System Assembly

The next step in the construction of NCIAS is placement of the energy-absorbing cylinders on the skid rails. The cylinders may be placed individually, in small clusters, or as one cluster. Each cylinder has two lifting rings welded to its inside wall for use with a chain and shackles. For the installations discussed here, all eight cylinders were connected together and placed on the skid rails as a single unit. Once on the skid rails, Cylinders 6 and 7 required maneuvering in order to position their retainer clips properly under the flanges of their respective retainer plates. It is very important that these parts engage correctly for NCIAS to function as intended. Once the cylinders were in position, all connections were securely tightened, and Cylinder 8 was connected to the bolts on the backup structure and secured there with washers and nuts.

Four high-strength wire ropes are placed along the sides of NCIAS. The wire ropes have fittings on both ends and are made in two different lengths. The longer wire ropes are connected to the inside positions of the front anchor plates, passed through the top set of cylinder eyebolts and U-bolts, and attached to the rear-most pipe of the backup structure. The shorter wire ropes are connected to the outside positions of the front anchor plates, passed through the lower set of cylinder eyebolts and U-bolts, and attached to the middle pipe of the backup structure. The front wire-rope fittings are closed swage sockets for connection to a solid 2-in.-diameter pin supplied with the front anchor plates. The rear fittings are threaded studs, which pass through cross pipes in the backup structure and are tightened with nuts on the back. The wire ropes must be as tight as possible, but the eyebolts on Cylinders 1 and 8 must not be bent.

A cover should be placed on top of NCIAS installations in northern regions where snow and ice may collect in the cylinders. The cover is made of a tough polyester-reinforced vinyl fabric with straps sewn to it. The end of each strap has a steel clip, which is fastened to the cylinders with aluminum pop rivets. These rivets shear under impact, leaving the cover undamaged. The three rear-most cover clips are bolted to Cylinder 8 to prevent the cover from flying away in the event that all rivets are sheared during a severe impact. A detailed, step-by-step assembly procedure that includes the cover attachment and other maintenance requirements is contained in the NCIAS Maintenance Manual (*11*). Photographs of the completed installations are shown in Figures 3-7.

Exit Signs

The Connecticut NCIAS installations are at exit gore areas, which require exit signs. The standard Connecticut exit sign is supported by two 3-lb/ft U-channel posts. At Sites 3 and 4, the sign posts were embedded into the pavement, straddling a single concrete barrier curb section at the rear of the system

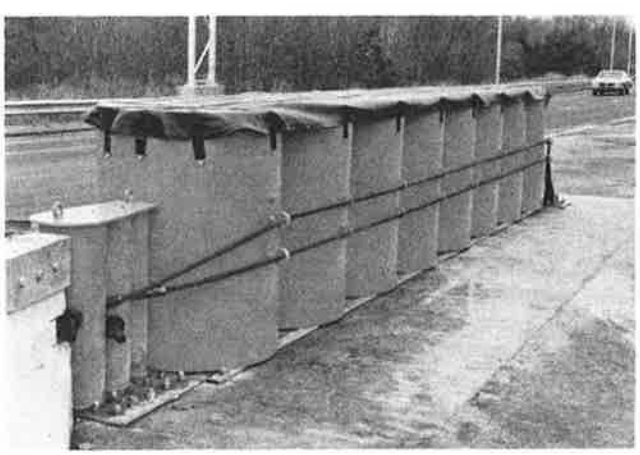

FIGURE 3 Site 1.

FIGURE 4 Site 2.

FIGURE 5 Site 3.

FIGURE 6 Site 4.

(see Figures 5 and 6). Although these posts comply with breakaway standards, it was decided to change the sign supports at these sites in order to prevent them from interfering with the safety performance of NCIAS and the concrete barrier curb. The new configuration uses a narrower exit sign on a single post support that is mounted atop the backup struc-

FIGURE 7 Site 5.

ture with four bolts. In addition to improving safety, the new sign mount may reduce repair efforts because it is less likely to be damaged in the raised position.

COSTS

A major goal of the Connecticut attenuator program is to provide alternative, safety-compliant crash cushions at reduced costs. Because NCIAS and CIAS are nonproprietary and fabricated from readily available materials, they offer the safety performance of modern devices with lower unit costs. For these five installations, the shop fabrication of NCIAS and their installation were completed under separate contracts with ConnDOT. A breakdown of the latest contract prices for fabrication of each complete NCIAS unit and for spare parts is presented in Table 2. The pad construction and NCIAS installation were included as a single bid item in the construction contract.

When compared with other commercially available crash cushions, NCIAS is competitive in a cost-versus-safety analysis. For many years, the sand-module attenuators have offered a convenient, inexpensive method of protecting highway hazards. However, the sand systems do not provide the same level of safety performance as the newer mechanical devices, including NCIAS. In fact, many safety-improvement projects involve replacing sand attenuators with more modern crash cushions. Recent ConnDOT records show that average contractor bid prices for the supply and installation of other high-speed crash cushions have ranged from $25,000 to $38,000. Of course, each installation is unique and has different site-preparation requirements. Using the data presented in Table 2, highway agencies can compare the cost of NCIAS with those of other modern devices.

TABLE 2 NCIAS Cost Breakdown

Complete NCIAS Unit	$ 7,280.00
Pad Construction and NCIAS Installation	$ 10,000.00
Total Installed Cost	$ 17,280.00
Spare Parts	
Cylinder 1	$ 610.00
Cylinder 2	$ 590.00
Cylinder 3	$ 600.00
Cylinder 4	$ 600.00
Cylinder 5	$ 660.00
Cylinder 6	$ 700.00
Cylinder 7	$ 730.00
Cylinder 8	$ 640.00
Retainer Plate for Cylinder 5	$ 242.00
Retainer Plate for Cylinder 6	$ 242.00
Retainer Plate for Cylinder 7	$ 242.00
Front Anchor Plate with Pin	$ 70.00
Skid Rails	$ 250.00
Backup Structure	$ 1,110.00
Wire Rope with End Fittings	$ 195.00

CONCLUSION

By describing the first five NCIAS installations, this paper familiarizes potential users with the methodology required to construct the device to perform as designed. It is extremely important that NCIAS and all roadside safety features be field constructed such that their safety performance at least equals that of the controlled crash tests. This paper is also intended to encourage highway agencies to install NCIAS for the collection of field performance data. As outlined in NCHRP Report 230 (*6*), field evaluation of experimental safety features is an integral part of the overall development and approval of these devices. On the basis of the installation of the NCIAS units described here, it is concluded that savings in material costs have been realized. Because it is nonproprietary, NCIAS can be purchased through competitive bids for less than other modern devices. Since no major construction problems were encountered, the installation costs for NCIAS are considered to be approximately equal to those of most present-day crash cushions. As the field evaluation progresses, cost figures for the repair of damaged systems, as well as safety-performance benefits, will be included to complete a cost analysis of NCIAS.

RECOMMENDATIONS

NCIAS is currently designated as experimental by the Geometric and Roadside Design Branch of FHWA. All sites are being closely monitored for a 3-year field evaluation of safety performance, durability, and operational characteristics. To date, none of the installations in Connecticut or Tennessee have been impacted by an errant vehicle. When such an accident occurs, data on occupant injury, vehicle damage, replacement part costs, and required repair labor will be quantified and compared with those of other widely used systems. It is anticipated that NCIAS will achieve operational status when enough field performance data are collected and evaluated. It is recommended that transportation agencies consider using NCIAS at appropriate locations as opportunities arise. Many federally funded, safety improvement projects require the installation of crash cushions for the shielding of unremovable roadside hazards. With its stand-alone backup structure, NCIAS can be anchored in front of many types of narrow expressway gore areas in which a concrete barrier separates the mainline from the exit ramp. Other applications include the shielding of bridge piers in median areas, large-sign supports, median terminals, and bridge abutments. Many of these locations may currently be underprotected and in need of a crash cushion. As use of NCIAS increases, more field performance data can be obtained to complement the research efforts completed to date.

It is also recommended that research continue on the development of additional impact-attenuation devices for protection of hazards not suitable for CIAS or NCIAS. Many exit gore areas contain hazards that are too wide for use of NCIAS and too narrow for use of the CIAS. Work has begun on testing of a family of CIAS designs called the Generalized CIAS. With the use of a computerized design program, a hazard width and design speed can be entered to obtain a suitable cylinder arrangement to accommodate the site conditions (*12,13*). The extreme designs resulting from the program (small and large length/width ratios) are undergoing full-scale crash tests. If successful, a large variety of CIAS-type crash cushions will be available for use at almost all hazardous gore areas. Since the current safety performance guidelines of NCHRP Report 230 will soon be updated, development of new systems and upgrading of existing ones will be needed to keep pace with increasing safety requirements.

ACKNOWLEDGMENTS

The author gratefully acknowledges the support of ConnDOT and FHWA.

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Long-Span Nested W-Beam Guardrails over Low-Fill Culverts

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A problem arises when it is necessary to continue a roadside guardrail across a low-fill box or pipe culvert. Full embedment of the guardrail posts is not possible over the culvert because of the shallow soil cover. Previous crash testing has demonstrated that posts with short embedment depths can be pulled out from the ground and subsequently fall into the path of the vehicle's tires, resulting in snagging or vaulting of the vehicle. For steel-post guardrails, one approach that has been successfully tested is bolting of the base plates of the short steel posts to the top of the box culvert. However, this design is not applicable to wood-post guardrail systems unless the posts are replaced with steel posts for the segment over the low-fill box culvert. This design also requires specially fabricated steel posts and additional labor for installation, resulting in considerably higher installation and maintenance costs. The results of a study to develop a design that is suitable for use with wood-post guardrail systems over low-fill culverts are summarized here. A computer simulation study was conducted to evaluate various alternative designs. The best design was then further evaluated through full-scale crash testing. The final design that was developed and successfully crash tested has no shallow embedment posts over the culvert and uses a nested W-beam rail to span the culvert. The design was tested for span lengths of 12 ft 6 in. (3.81 m) and 18 ft 9 in. (5.72 m). The additional costs associated with implementing this system are relatively low, consisting of two or three 12.5-ft (3.81-m) sections of W-beam rail elements and a little more labor. It is believed that the same design can be used for steel-post guardrail systems over low-fill culverts.

Roadside guardrails are often used in conjunction with culverts to prevent errant vehicles from running off the edge of the culvert. A problem arises when a roadside guardrail must continue across a low-fill culvert. Full embedment of the guardrail posts is not possible over the culvert because of the shallow soil cover. Previous crash testing has demonstrated that posts with short embedment depths can be pulled out from the ground and subsequently impacted by the vehicle's tire, which could result in snagging or vaulting of the vehicle, with potentially disastrous results (*1*).

For a steel-post guardrail system, one design that has been successfully crash tested involves welding base plates to the short steel posts and then bolting them to the top of the concrete box culvert (*1*). This eliminates the potential for the short posts to be pulled out from the ground and increases their load-carrying capacity. To use this design for wood-post guardrail systems, steel posts must be used instead of wood for the barrier segment over the low-fill box culvert. This design also requires specially fabricated steel posts and additional labor for installation, resulting in considerably higher installation and maintenance costs.

A study was conducted at the Texas Transportation Institute to develop a design that is suitable for use with wood-post guardrail systems over low-fill culverts. The study was jointly sponsored by the Washington State Department of Transportation and the State of Idaho Transportation Department under a pooled-fund study administered by FHWA (*2–5*). First, a computer simulation study was conducted to evaluate several designs. The best design was then further evaluated through full-scale crash testing. The results of this study are summarized here.

COMPUTER SIMULATION STUDY

Four designs were evaluated in the computer simulation study:

1. Single W-beam guardrail with one short post midspan over culvert,
2. Nested W-beam guardrail with one short post midspan over culvert,
3. Single W-beam guardrail with long span across culvert, and
4. Nested W-beam guardrail with long span across culvert.

The culvert was assumed to have a minimum soil cover of 18 in. (45.7 cm). The embedment depth for the one short post midspan over the culvert was also assumed to be 18 in. (45.7 cm). The guardrail system was assumed to be a standard G4(2W) strong-post blocked-out W-beam guardrail system, with standard 6-in. × 8-in. × 6-ft (15.2-cm × 20.3-cm × 1.82-m) wood posts placed on either side of the span that bridged the culvert.

The Barrier VII simulation program (*6*) was the primary tool used in the evaluation of the alternative guardrail designs. It is a two-dimensional simulation program that models vehicular impacts with deformable barriers. The program employs a sophisticated barrier model that is idealized as an assemblage of discrete structural members possessing geometric and material nonlinearities. The available structural members include beams, cables, posts, springs, columns, links, and damping devices. The vehicle is idealized as a plain rigid body surrounded by a series of discrete inelastic springs.

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The Barrier VII program has been shown to be capable of accurately predicting barrier deflections for a wide variety of flexible barrier designs, even under severe impact conditions. Although it cannot be used directly to evaluate the effect of wheel and post interaction and rail rupture, it can be used to predict their occurrence. The amount of wheel snag can be inferred from the position of the vehicle tire and the deflected position of a post. Rail failure can be predicted by evaluating the maximum strain in the rail and comparing the computed values to the rated ductility.

The impact conditions used for the simulation runs consisted of a 4,500-lb (2,041-kg) vehicle impacting the guardrail at a velocity of 60 mph (96.6 km/hr) and an angle of 25 degrees. This is in accordance with the impact conditions outlined in NCHRP Report 230 (*7*) for evaluating the structural integrity of longitudinal barrier systems.

More than 200 computer simulation runs were conducted to evaluate the four systems. For each system, the span length (i.e., spacing over the culvert) was systematically varied in order to identify the maximum safe span length. For each span length, the point of impact was varied along the barrier to allow identification of the critical impact locations. The maximum safe span length for each system was determined using both structural and functional failure criteria. The structural failure criterion was based on how much the rail element yielded. This was quantified in terms of the maximum strain in the rail. A typical W-beam rail has a minimum yield stress of 50 ksi (345 Mpa) and a minimum ductility of 12 percent.

The functional failure criterion was based on the maximum allowable deflection for the rail at a given point (e.g., a post). The deflection limit imposed on the guardrails with shallow embedment posts was 24 in. (61 cm) because the short post should remain partially embedded at all times during the impact and should not be pulled completely from the soil. The limit for the systems without short posts was 30 in. (76.2 cm). The deflection limit was based primarily on the values obtained from other strong-post barrier systems that have been successfully tested (*8*). The designs had to satisfy these evaluation criteria to be considered for further evaluation.

One of many factors that influence the maximum deflection of a barrier system is the number of predicted post failures. In the Barrier VII model, complete failure of a post is assumed if the computed displacement along either of the principal axes exceeds a user-specified limit. This represents separation of the rail from the post or withdrawal of the post from the ground. The post is idealized with elastic/perfectly plastic behavior, hence the post will form a plastic hinge at its base and yield at constant load until the specified limit is reached. Based on results of previous static and dynamic testing of guardrail posts (*9–11*), a limit of 18 in. (45.7 cm) was used for the standard guardrail post and 14 in. (35.6 cm) for the short post. Note that, in the case of the standard post, the limit is more representative of separation from the rail than of a loss of load-carrying capacity. The short post, on the other hand, experiences a significant reduction in load-carrying capacity at 14 in. (35.6 cm) as a result of partial withdrawal from the ground and excessive rotation.

Summaries of the simulation results for the four systems evaluated are presented in Table 1. For both systems with one short post over the culvert (Systems 1 and 2), the maximum deflection exceeds the allowable limit of 24 in. (61 cm), even for the shortest simulated span of 10.5 ft (3.2 m).

For System 3, with a long-span single W-beam rail and no shallow embedment posts over the culvert, the maximum rail deflections exceed the allowable limit of 30 in. (76.2 cm) for all span lengths simulated. In addition, the structural limitation of the W-beam rail is exceeded for the 10.5 ft (3.2 m) span length. A 19 percent strain was predicted for the W-beam rail element, which exceeds the rated minimum ductility of 12 percent, indicating that rupture of the rail element is imminent.

System 4, with a long-span nested W-beam rail and no shallow embedment posts over the culvert, appeared to be the best of the four systems evaluated. The data in Table 1 show that neither the functional nor the structural limitations are exceeded for spans up to 18.5 ft (5.64 m). Longer span lengths are not recommended because of the increased probability of pocketing on the full-strength system and the probability of the impacting vehicle under- or overriding the rail element.

Additional simulation runs were conducted to determine the minimum length of nested rail required to maintain the structural integrity of the rail. A nested rail has twice the area and, consequently, twice the tensile capacity of a single rail. The single W-beam rail will therefore yield at lower loads and produce more elongation in the rail. Simulation runs were conducted for various lengths of nested rail, and the maximum strains were calculated at the transition point from single to nested rail. The longer the length of the nested rail, the lower the tensile forces at the transition point and, thus, the lower the strain in the single W-beam rail. On the other hand, longer nested rails would mean higher material and labor costs.

The simulation results indicated that a minimum of 25 ft (7.62 m) of nested W-beam rail should be used in conjunction with the 12.5-ft (3.81-m) span over the culvert. This would require two 12.5-ft (3.81-m) rail sections or one 25-ft (7.62-m) section. The nested rail would span over the culvert and one post span on either side of the culvert. Note that this system would have a splice in the middle of the long span when 12.5-ft (3.81-m) rail sections were used. If this detail is not desirable, three sections of nested rail can be used for a total length of 37.5 ft (11.43 m) of nested rail. One section would span the culvert, and an additional section would be added on either side of the long span. This would place all splice locations at a post, a common feature of standard guardrail design.

For a 18.75-ft (5.72-m) span length, a minimum of 37.5 ft (11.4 m) of nested W-beam rail is required. This would require three 12.5-ft (3.81-m) rail sections. The nested rail would span over the culvert, one post downstream of the culvert, and two posts upstream of the culvert. The long span would require a splice because the W-beam rail would be only 12.5 ft (3.81 m) long. When 25-ft (7.62-m) rail elements are used, two sections would be required for a total nested length of 50 ft (15.2 m). The nested rail would span over the culvert, two posts downstream of the culvert, and three posts upstream of the culvert, with a splice in the long span.

On the basis of the results of the simulation study, the design with a long-span nested W-beam rail and no shallow embedment posts over the culvert was considered the best

TABLE 1 Summary of Computer Simulation Results

Length of Span (ft)	Maximum Rail Deflection (in)	No. of Posts Failed*	Rail Yielded in Tension (Y/N)	Maximum Strain (%)
SYSTEM 1 SINGLE W-BEAM GUARDRAIL WITH ONE SHORT POST MIDSPAN OVER CULVERT				
10.5	29.3	3	Y	8.3
12.5	31.4	2	Y	8.9
14.5	32.7	2	Y	4.4
SYSTEM 2 NESTED W-BEAM GUARDRAIL WITH ONE SHORT POST MIDSPAN OVER CULVERT				
10.5	26.1	2	Y	4.7
12.5	26.1	2	Y	0.8
14.5	26.3	2	Y	1.9
16.5	27.3	2	Y	2.8
SYSTEM 3 SINGLE W-BEAM GUARDRAIL WITH LONG SPAN ACROSS CULVERT				
10.5	32.1	2	Y	19.0
12.5	32.2	1	Y	14.2
14.5	34.1	1	Y	7.3
SYSTEM 4 NESTED W-BEAM GUARDRAIL WITH LONG SPAN ACROSS CULVERT				
14.5	26.6	1	Y	0.5
16.5	28.1	1	N	---
18.5	28.1	1	N	---

* Including the short post for systems 1 and 2.

design and was recommended for further evaluation under full-scale crash testing.

FULL-SCALE CRASH TESTING

The long-span nested W-beam guardrail design with no shallow embedment post over the culvert was tested for span lengths of 12.5 ft (3.81 m) (Test 7147-2) and 18.75 ft (5.72 m) (Test 7147-5). The crash test procedures and evaluation criteria were in accordance with requirements outlined in NCHRP Report 230 (*7*). The results of the two crash tests are summarized in Table 2; the tests are described briefly here.

Test 7147-2

The test installation was 150-ft- (45.7-m-) long and included 87.5 ft (26.7 m) of standard G4(2W) strong-post, blocked-out, W-beam wood post guardrail, a 25-ft (7.6-m) turned-down end anchorage on the downstream end, and a 37.5-ft (11.4-m) breakaway cable terminal (BCT) anchorage on the upstream end. The standard guardrail installation included 6-in. × 8-in. × 6-ft (15.2-cm × 20.3-cm × 1.82-m) wood posts with 6-in. × 8-in. × 14-in. (15.2-cm × 20.3-cm × 25.6-cm) wood blockouts, which were spaced 6 ft 3 in. (1.91 m) center to center. The W-beam rail elements used were 12 gauge galvanized steel sections, 12 ft 6 in. (3.81 m) in length.

As shown in Figure 1, a 12-ft 6-in. (3.81-m) span was constructed in the center of the test installation to simulate the long clear span over a low-fill culvert. The minimum length of 25 ft (7.62 m) of nested W-beam rail was used, which allowed for nested rail over the culvert and one post span on either side of the culvert. Since 12-ft 6-in. (3.8-m) W-beam rail elements were used, the splice in the 25 ft (7.62 m) of nested rail was in the middle of the long span instead of at a post. Photographs of the completed test installation are shown in Figure 2.

A 1981 Cadillac Fleetwood was used for the crash test. The empty weight of the vehicle was 4,500 lb (2,043 kg), and its test weight was 4,670 lb (2,120 kg). An Alderson Research Laboratories Hybrid II 50th percentile male anthropometric dummy was placed in the driver's seat and restrained with lap and shoulder belts. The vehicle impacted the barrier approximately 1 ft (30.5 cm) downstream from Post 12 (upstream post for the long span over the simulated culvert) at a speed of 62.7 mi/hr (100.9 km/hr) and an angle of 24.5 degrees.

Note that the impact point was selected to produce maximum barrier dynamic deflection and maximum potential for pocketing and wheel snagging at the downstream post of the long span (Post 13). The impact point was not intended to

TABLE 2 Summary of Test Results

Description	Span Length 12 ft 6 in (Test 7147-2)	18 ft 9 in (Test 7147-5)
Test Vehicle	1981 Cadillac Fleetwood	1982 Oldsmobile Regency 98
Test Weight, lb (kg)	4670 (2120)	4670 (2120)
Impact Speed, mi/h (km/h)	62.7 (100.9)	60.9 (98.0)
Impact Angle, deg.	24.5	25.1
Point of Impact, ft (m) Downstream of Upstream Post of Long Span	1.0 (0.31)	2.9 (0.88)
Exit Speed, mi/h (km/h)	42.2 (67.9)	44.2 (71.1)
Exit Angle, deg.	11.0	10.4
Velocity Change[1], mi/h (km/h)	20.5 (33.0)	16.7 (26.9)
Occupant Impact Velocity[2]		
Longitudinal, ft/s (m/s)	17.8 (5.4)	14.7 (4.5)
Lateral, ft/s (m/s)	15.9 (4.8)	14.2 (4.3)
Occupant Ridedown Acceleration[2]		
Longitudinal, g	- 6.5	- 3.5
Lateral, g	12.9	9.7
Length of Rail Contact, ft (m)	23.5 (7.2)	25.0 (7.6)
Maximum Dynamic Rail Deflection, ft (m)	3.1 (0.9)	3.2 (0.9)
Maximum Permanent Deformation, in (cm)	29.0 (73.7)	30.0 (76.2)
Maximum Vehicle Crush, in (cm)	13.0 (33.0)	8.0 (20.3)

Notes. [1] The velocity change was higher than the recommended value of 15 mi/h (24.1 km/h) in both tests, but the vehicle was judged not to be a hazard to adjacent traffic lanes.

[2] According to NCHRP Report 230 guidelines, the occupant risk criteria are not applicable for 4,500-lb passenger car crash tests.

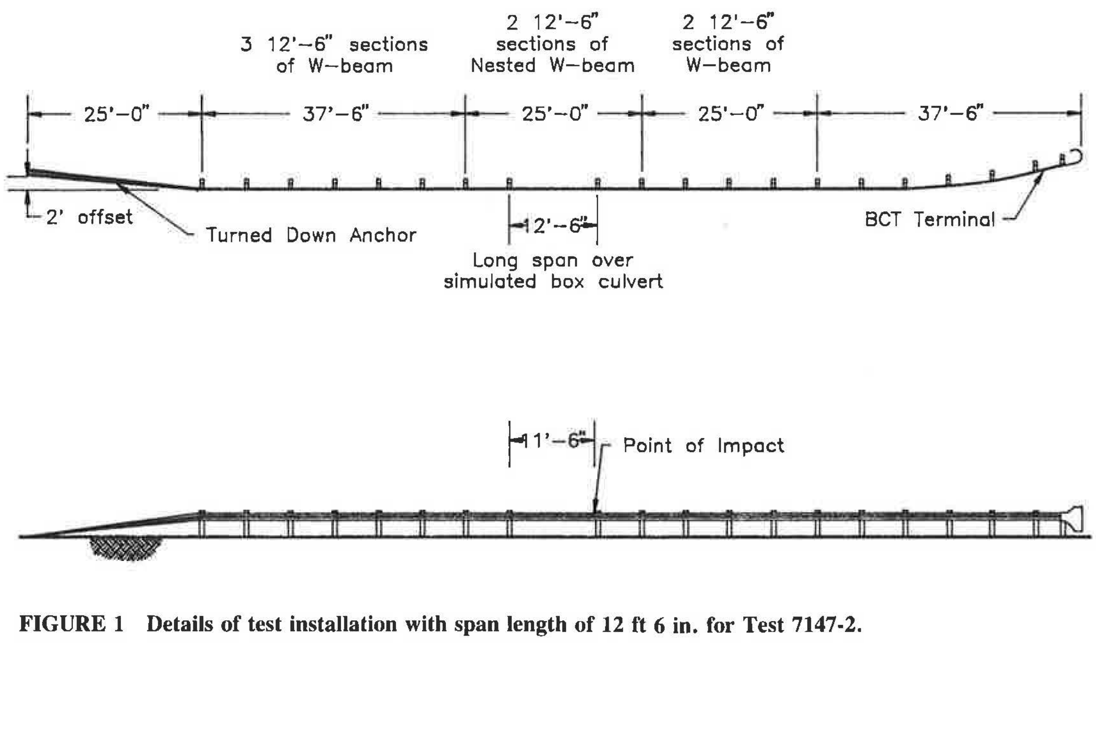


FIGURE 1 Details of test installation with span length of 12 ft 6 in. for Test 7147-2.

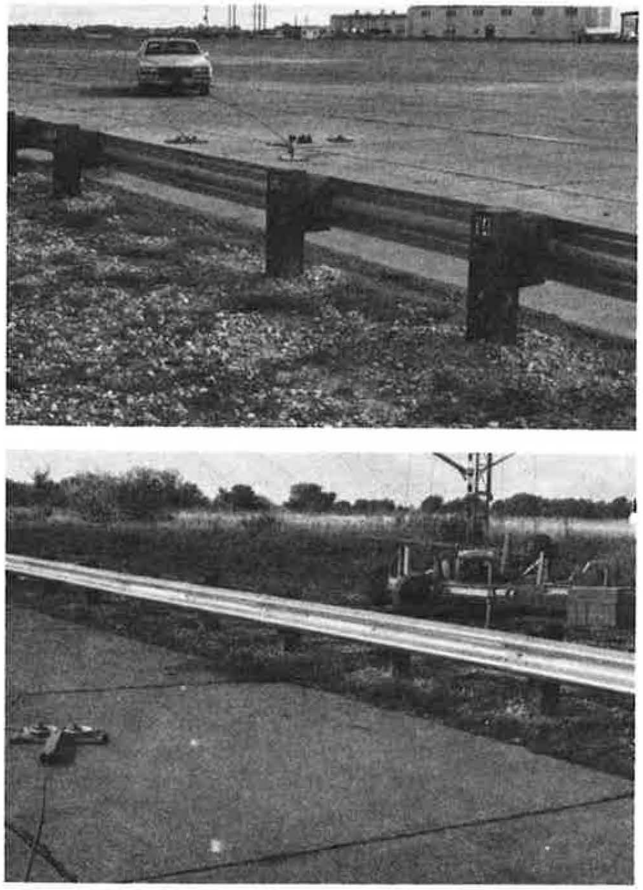


FIGURE 2 Photographs of test installation before Test 7147-2.

produce maximum dynamic deflection in the system, which was considered less critical than pocketing and snagging at the downstream post.

The long-span nested W-beam guardrail system with a 12.5-ft (3.81-m) span length performed well in the crash test. The vehicle was smoothly redirected and did not penetrate or go over the guardrail system. There were no detached elements or debris that showed potential for penetrating the occupant compartment or presenting undue hazard to other vehicles. The vehicle remained upright and stable during the impact and after exiting the test installation. Some slight pocketing and tire contact occurred at the downstream post of the long span, but their effects were minor and did not significantly affect the vehicle kinematics or trajectory. The velocity change of 20.5 mi/hr (33.0 km/hr) was higher than the limit of 15 mi/hr (24.1 km/hr) recommended in NCHRP Report 230. However, vehicle trajectory at loss of contact indicates minimal potential for intrusion into adjacent traffic lanes. Therefore, the velocity change criterion is not applicable. It should be noted that the criterion is seldom met in crash tests involving flexible barrier systems.

The guardrail system received moderate damage, as shown in Figure 3. The maximum dynamic deflection was 3.1 ft (0.9 m). The maximum permanent deformation of the W-beam rail element was 29 in. (0.74 m), located approximately 3 ft upstream of Post 13 (the downstream post of the long span). There was some flattening of the W-beam rail element at the lower corrugation upstream of Post 13 as the vehicle pocketed slightly at the post and pressed the W-beam rail element against the blockout and the post. Post 13 was pushed back 12.75 in. (32.4 cm) at ground level and 28.5 in. (0.72 m) at the center of the W-beam rail element. The blockout at Post 13 was broken and separated from the post and the head of the bolt attaching the rail to the blockout and post was pulled through the nested W-beam rail elements. The two end anchors moved slightly.

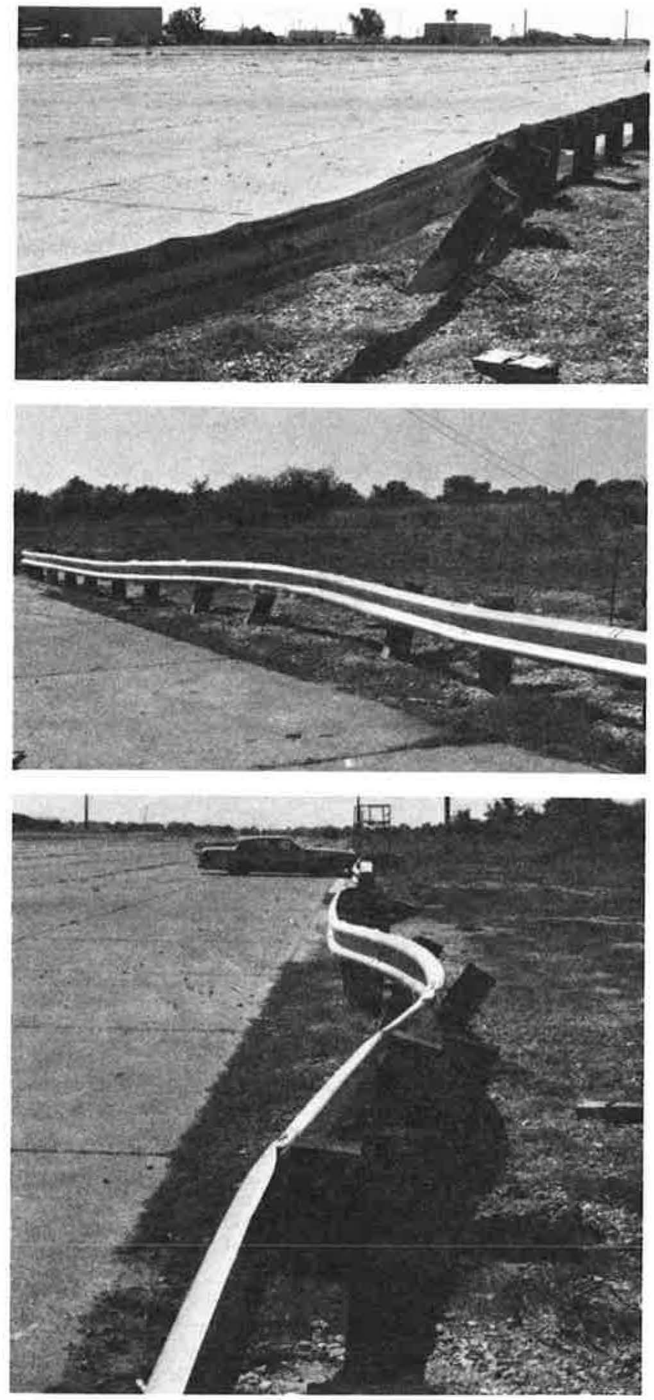


FIGURE 3 Photographs of damage to test installation after Test 7147-2.

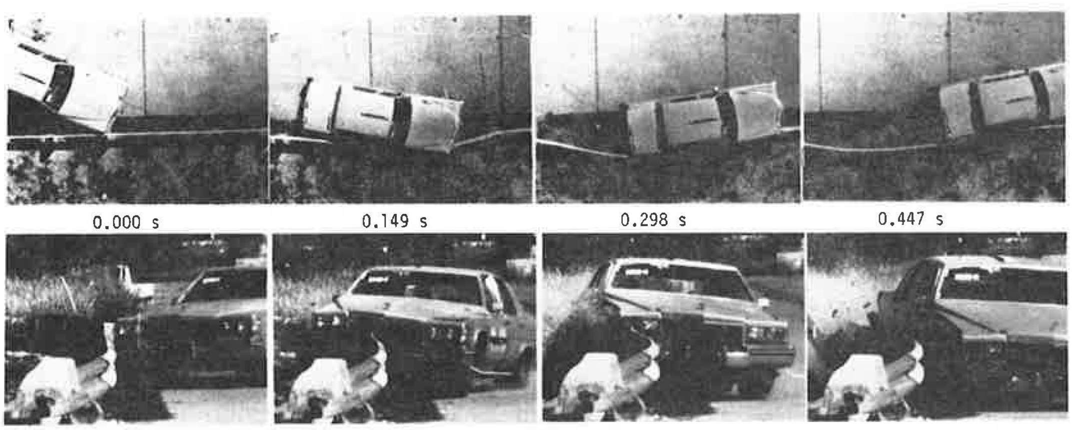
FIGURE 4 Photographs of damage to vehicle after Test 7147-2.

The vehicle sustained moderate damage to the right side, as shown in Figure 4. The tie rod was bent, and the windshield was broken. There was damage to the bumpers, hood, grill, radiator and fan, right front and rear quarter panels, and right front and rear doors. The wheelbase on the right side was shortened from 121.5 in. (3.09 m) to 116.0 in. (2.95 m). The right front and rear tires and rims were damaged from contact with the posts. Maximum crush of the vehicle was 13.0 in. (33.0 cm) at the right front corner at bumper height. However, essentially no intrusion or deformation of the passenger compartment occurred. Note that much of the damage to the front of the vehicle was the result of the vehicle impacting the end of a concrete barrier near the end of the vehicle trajectory. It should also be noted that the test vehicle had a fiberglass header panel, which made the damage to the front of the vehicle appear worse than it really was. A summary of the test results is presented in Figure 5.

Test 7147-5

The installation used for this test was similar to that used in Test 7147-2 except for the 18.75-ft (5.48-m) span constructed in the center of the test installation (between Posts 11 and 12) to simulate the long span over a low-fill culvert, as shown in Figure 6. Three 12.5-ft (3.81-m) sections of nested W-beam were used, for a total length of 37.5 ft (11.43 m), starting from Post 9, extending over the culvert span of 18.75 ft (5.72 m), and terminating at Post 13. Photographs of the completed test installation are shown in Figure 7.

A 1982 Oldsmobile Regency 98 was used for the crash test. The empty weight of the vehicle was 4,500 lb (2,043 kg), and its test weight was 4,670 lb (2,120 kg). Again, a 50th percentile




```
Test No. . . . . . . . . 7147-2                  Impact Speed  . . .  62.7 mi/h (100.9 km/h)
Date . . . . . . . . . . 09/25/90                Impact Angle  . . .  24.5 degrees
Test Installation. . . . Washington Nested       Speed at Parallel .  49.4 mi/h (79.5 km/h)
                    W-beam with wood posts       Exit Speed  . . . .  42.2 mi/h (67.9 km/h)
Installation Length. . . 150 ft (46 m)           Exit Trajectory . .  11.0 degrees
Max. Dynamic Deflection. 3.1 ft (0.9 m)          Vehicle Accelerations
Max. Perm. Deformation . 2.4 ft (0.7 m)            (Max. 0.050-sec Avg)
Vehicle  . . . . . . . . 1981 Cadillac              Longitudinal  . .  -4.5 g
Vehicle Weight                 Fleetwood            Lateral . . . . .   7.1 g
  Test Inertia . . . . . 4,500 lb (2,043 kg)     Occupant Impact Velocity
  Gross Static . . . . . 4,669 lb (2,120 kg)        Longitudinal  . .  17.8 ft/s (5.4 m/s)
Vehicle Damage Classification                       Lateral . . . . .  15.9 ft/s (4.8 m/s)
  TAD  . . . . . . . . . 01FR5 & 01RD4           Occupant Ridedown Accelerations
  CDC  . . . . . . . . . 01FREK2 & 01RDEW3          Longitudinal  . .  -6.5 g
Maximum Vehicle Crush  . 13.0 in (33.0 cm)          Lateral . . . . .  12.9 g
```

FIGURE 5 Summary of results for Test 7147-2.

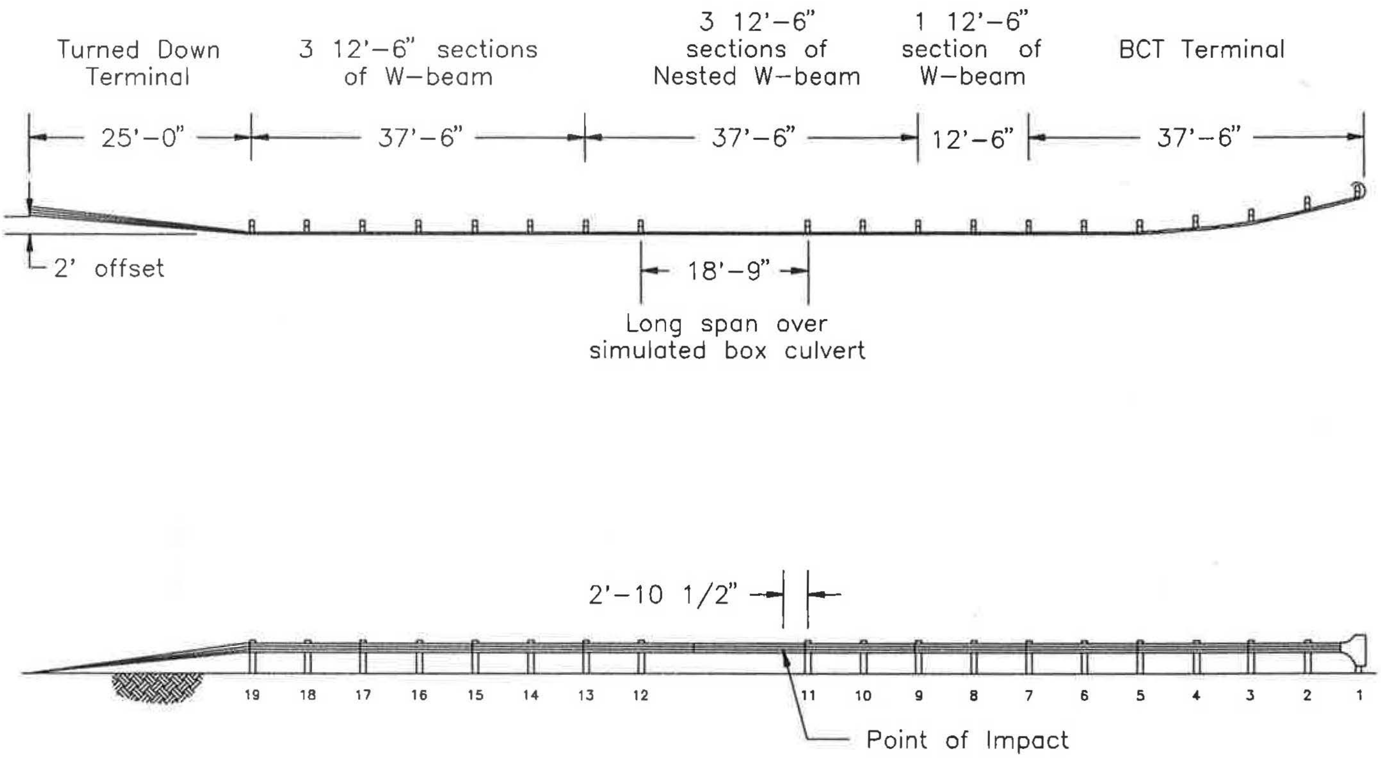


FIGURE 6 Details of test installation with span length of 18 ft 9 in. for Test 7147-5.

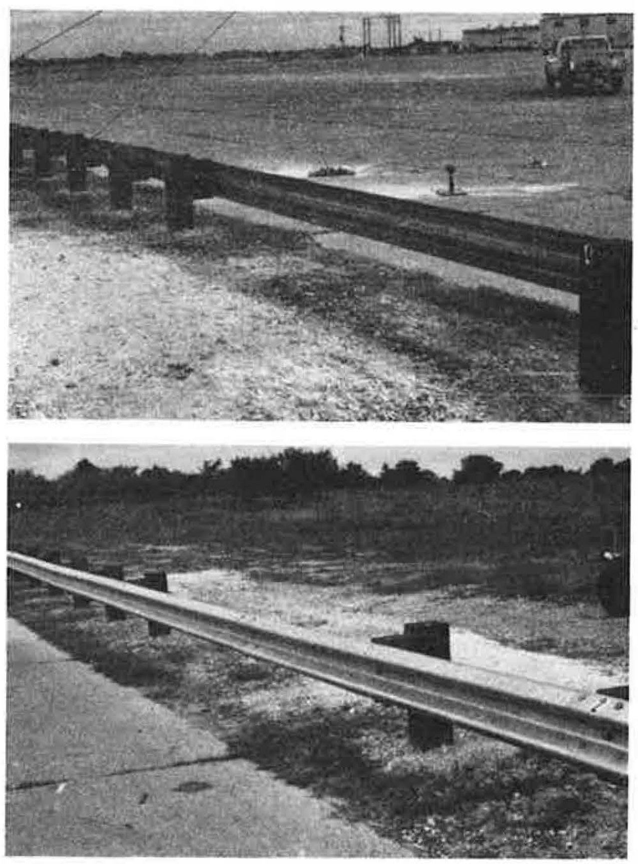


FIGURE 7 Photographs of test installation before Test 7147-5.

male anthropometric dummy was placed in the driver's seat and restrained with lap and shoulder belts. The vehicle impacted the guardrail system approximately 2.9 ft (0.9 m) downstream of Post 11 (upstream post for the long span over the simulated culvert) at a speed of 60.9 mi/hr (98.0 km/hr) and an angle of 25.1 degrees. As before, the impact point was selected to produce maximum dynamic deflection and maximum potential for pocketing and wheel snagging at the downstream post of the long span (i.e., Post 12).

The 18-ft 9-in. (5.72 m) long-span nested W-beam guardrail system performed well in the crash test. The vehicle was smoothly redirected and did not penetrate or go over the guardrail system. There were no detached elements or debris that indicated potential for penetrating the occupant compartment or presenting undue hazard to other vehicles. The vehicle remained upright and stable during the impact and after exiting the test installation. Slight pocketing and tire contact occurred at the downstream post of the long span, but their effects were minor and did not significantly affect the vehicle kinematics or trajectory. The velocity change was slightly higher than the recommended limit of 15 mi/hr (24.1 km/hr), but the vehicle trajectory at loss of contact indicates minimal potential for intrusion into adjacent traffic lanes. The velocity change criterion is therefore not applicable.

The guardrail system received moderate damage, as shown in Figure 8. The maximum dynamic deflection was 3.2 ft (0.9 m). The maximum permanent deformation of the W-beam rail element was 30.0 in. (0.76 m), located approximately in the center of the long span. Post 12 was pushed back 16.5 in. (41.9 cm) at ground level and 23.0 in. (0.58 m) at the center of the W-beam rail element. The blockout at

Post 11 was separated from the post and rail elements, and the post was split. No movement occurred at the two end anchors.

As shown in Figure 9, damage sustained by the vehicle was minor, given the severity of the impact. The upper control arm on the right side was damaged. There was damage to the bumpers, hood, grill, right front and rear quarter panels, and right front and rear doors. The wheelbase on the right side was shortened from 119.0 in. (3.02 m) to 117.0 in. (2.97 m). The right front and rear tires and rims were damaged from contact with the posts. Maximum crush of the vehicle was 8.0 in. (20.3 cm) at the right front corner at bumper height. However, there was no intrusion in or deformation to the occupant compartment. A summary of the test results is presented in Figure 10.

CONCLUSION

A design suitable for use with wood-post guardrail systems over culverts was developed and successfully crash tested. The design uses a nested W-beam rail to span the culvert and has no shallow embedment posts over the culvert. The design was

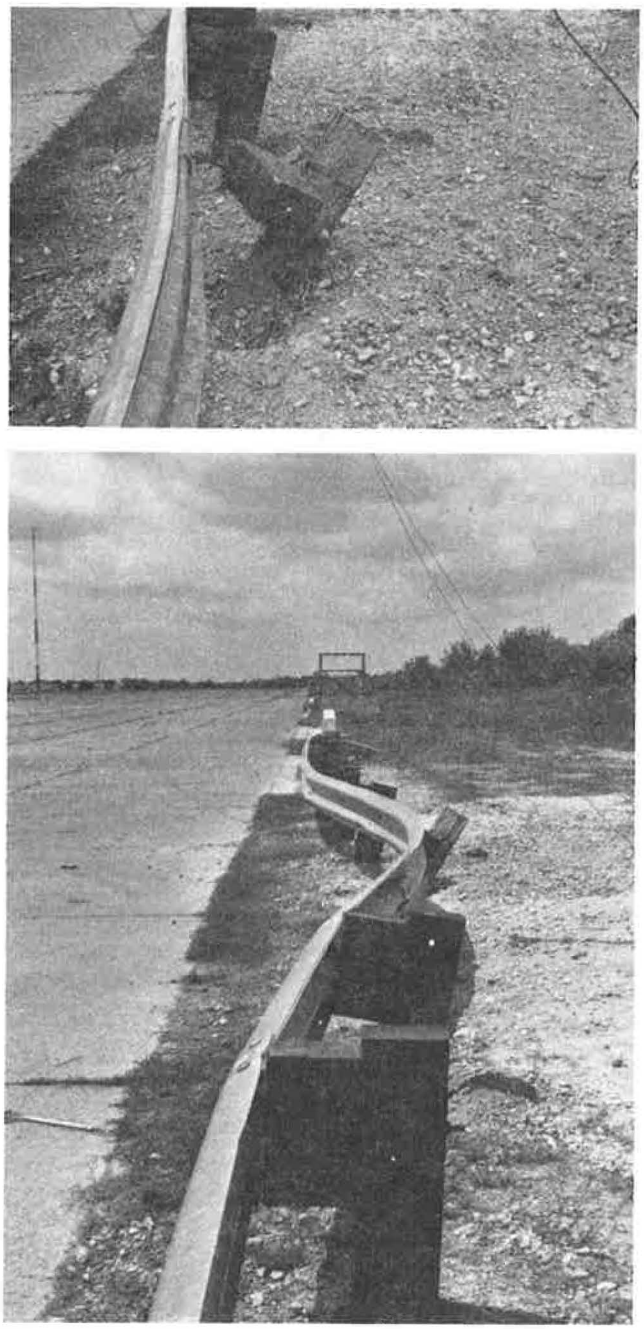


FIGURE 8 Photographs of damage to test installation after Test 7147-5.

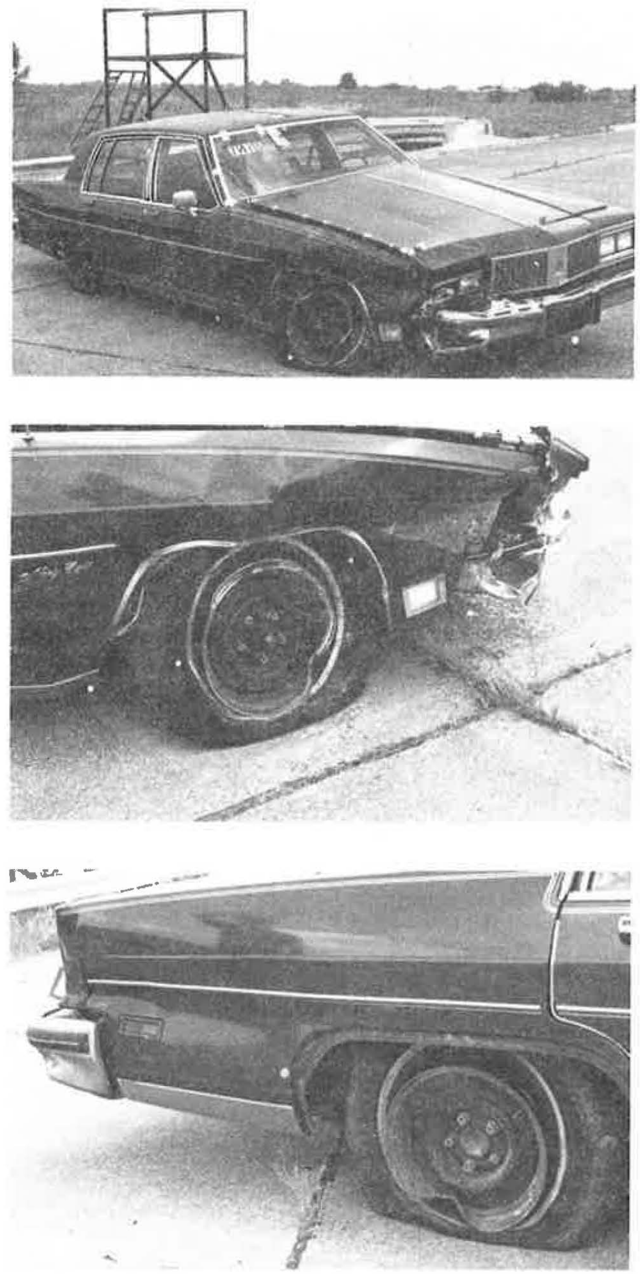
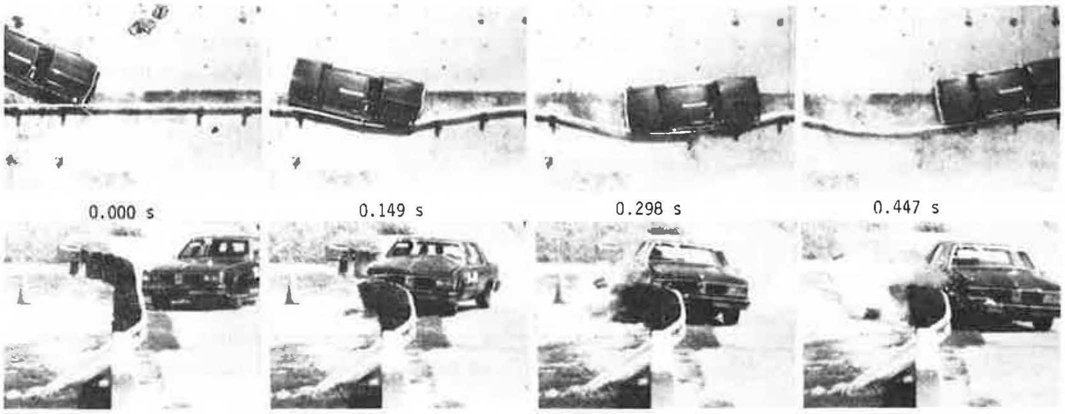


FIGURE 9 Photographs of damage to vehicle after Test 7147-5.



Test No. 7147-5	Impact Speed . . . 60.9 mi/h (98.0 km/h)
Date 05/30/91	Impact Angle . . . 25.1 degrees
Test Installation. . . . Washington Nested W-beam with wood posts	Speed at Parallel . 48.6 mi/h (78.2 km/h)
Installation Length. . . 150 ft (46 m)	Exit Speed . . . . 44.2 mi/h (71.1 km/h)
Max. Dynamic Deflection. 3.2 ft (1.0 m)	Exit Trajectory . . 10.4 degrees
Max. Perm. Deformation . 2.5 ft (0.8 m)	Vehicle Accelerations (Max. 0.050-sec Avg)
Vehicle . . . . 1982 Oldsmobile Regency 98	Longitudinal . . -2.7 g
Vehicle Weight	Lateral . . . . . 6.8 g
Test Inertia . . . . . 4,500 lb (2,043 kg)	Occupant Impact Velocity
Gross Static . . . . . 4,670 lb (2,120 kg)	Longitudinal . . 14.7 ft/s (4.5 m/s)
Vehicle Damage Classification	Lateral . . . . . 14.2 ft/s (4.3 m/s)
TAD . . . . 01FR4 & 01RD3	Occupant Ridedown Accelerations
CDC . . . . 01FREK2 & 01RDEW2	Longitudinal . . -3.5 g
Maximum Vehicle Crush . 8.0 in (20.3 cm)	Lateral . . . . . 9.7 g

FIGURE 10 Summary of results for Test 7147-5.

successfully crash tested for span lengths of 12 ft 6 in. (3.81 m) and 18 ft 9 in. (5.72 m). This design resolves problems associated with poor guardrail performance caused by shallow post embedment depths over low-fill culverts. The additional costs associated with implementing this design are relatively low.

Although this long-span nested W-beam guardrail design was developed for wood-post guardrail systems (the sponsoring agencies use wood-post guardrail systems), it is believed that it would also work for steel-post guardrail systems. The strengths of wood and steel posts are generally compatible, and there is no reason to believe that this design would behave differently for a steel-post system. This would be a simpler and less expensive alternative to the design of connecting the bottoms of the posts to the top of the culvert. Thus, this design is recommended for use with both wood-post and steel-post guardrail systems.

This design has been approved by FHWA and adopted by the Washington State Department of Transportation and the State of Idaho Transportation Department for field implementation. It is also being considered for situations in which a span length greater than the standard 6-ft 3-in. (1.91 m) post spacing is required at isolated locations (e.g., ditch lines or concrete drainage aprons).

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Guardrail End Treatments in the 1990s

DON L. IVEY, M. E. BRONSTAD, AND LINDSAY I. GRIFFIN III

An attempt is made to objectively review the most important characteristics of most commercially available and widely implemented terminals or end treatments for guardrail installations. These characteristics include collision performance in the testing and roadside environments, maintenance characteristics, and costs. Field experience with these devices is reviewed. An effort is made to compare performance, use, and costs to aid interested parties in selecting the most cost-effective terminals to meet specific needs.

Roadside safety has improved spectacularly since the mid-1960s. Functional life-saving structures have been developed rapidly in response to readily perceived needs. An exception to these achievements is end treatments or terminals for W-section guardrails.

When it was recognized in the mid-1960s that unprotected or unmodified guardrail ends were lethal roadside hazards (Figure 1), the highway community moved toward what appeared at the time to be a good, economical solution—the turned-down end. Turning the first section of the W-beam down and anchoring it at ground level certainly solved the spearing problem. Turndowns were considered good practice to enhance "the forgiving roadside" and were widely implemented throughout the United States (*1,2*).

With the momentum of AASHTO safety publications, the inertia of the research community, and the lack of good alternatives working to its advantage, turndowns continued to be implemented in many states. Concern for the ramping problem resulted in the development of a variation that was included in the 1977 barrier guide as an experimental design (*3,4*). It was designed to prevent severe ramping by collapsing when stuck head-on. It represents another example of good performance when struck head-on by a full-size vehicle, but marginal performance when struck by a small car. Efforts to improve performance by Hirsch and Buth (*4*), Hinch (*5*), and FHWA had limited success and neither GEET1 nor controlled releasing terminals (CRTs) have been used in significant numbers.

By the 1970s, the problem of ramping and capsizing was recognized (*6*). After Southwest Research Institute (SwRI) demonstrated this problem, it began development of the breakaway cable terminal (BCT) (*7*). BCT showed great promise in early tests with full-sized vehicles (*8*). The head-on 15-in. offset test with a 1,800-lb vehicle at 60 mph, which became a required test in 1981, was a problem (*9*). This was demonstrated in the early 1980s by FHWA (*10*).

The 4-ft offset, 37.5-ft parabolic flare was a prominent and important feature in the development of BCT. If there had been better alternatives to the BCT at that time, it might not have received such wide acceptance. It has several virtues: it was low cost, was relatively simple to install, and was the only operational terminal in the 1977 barrier guide (*3*).

It is estimated that 45 states have installed approximately 450,000 BCTs since 1972. Over time two problems began to emerge. Some state departments of transportation (DOTs) were not installing BCTs according to the recommended design drawings and the 1977 barrier guide. In some cases, state standards allowed installation with only 1 ft of flare, and in others, BCTs were installed with no flare at all. Furthermore, vehicles were not impacting the terminal in the same way in which the crash tests were conducted. The result was collisions in which BCTs did not perform well (*11–13*).

Continued testing of BCTs after initial implementation was conducted to (a) reduce costs, and (b) develop the steel post, slipbase alternative. Only after the FHWA program in the early 1980s was there a definite need recognized to change the basic BCT design. BCT-type devices such as the eccentric loader terminal (ELT) and the modified eccentric loader terminal (MELT) (Figure 2) are products of those efforts. The 18-in. offset test has not yet been tested.

In a memorandum of June 28, 1990, FHWA declined participation in any new installation of turndowns in high-speed, high-volume facilities (*14*). Turndowns have been used almost exclusively in a number of states. Texas recently completed a study of statewide accident data that may illustrate the shortcomings of the turndown. Texas has now changed the policy of constructing turndowns on high-speed, high-volume roadways, and Ohio is entering a new rehabilitation phase of replacing many turndowns. California (*15*) has recently evalu-

FIGURE 1 Unprotected or unmodified guardrail ends are lethal roadside hazards.

D. L. Ivey and L. I. Griffin III, Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843. M. E. Bronstad, Dynatech Engineering, Inc., 8023 Vantage, Suite 900, San Antonio, Tex. 78230.

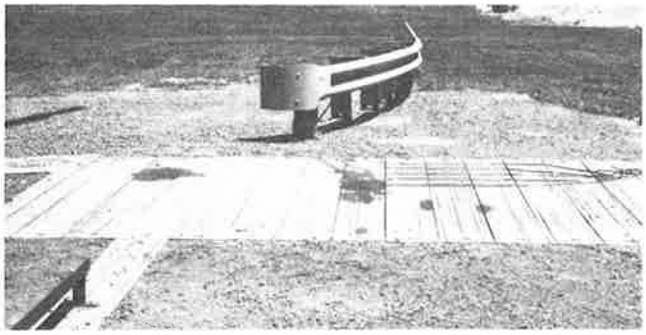


FIGURE 2 MELT (*29*).

FIGURE 3 BRAKEMASTER.

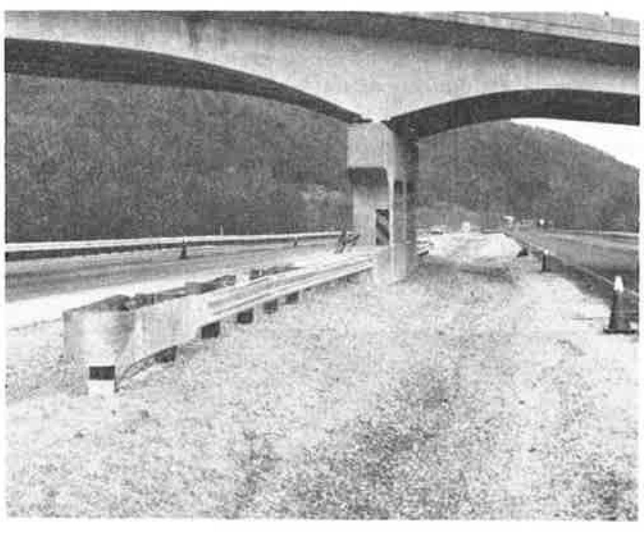


FIGURE 4 CAT installation.

ated eight terminal devices, including BCT. California noted the limitations of BCTs, but has not restricted their use as long as there is space for the full 4-ft flare. California has also approved use of ELT with a 4-ft flare and has declined its use with a 1.5-ft flare.

A design called Sentre (safety barrier end treatment) has been available for several years, but its cost has remained at

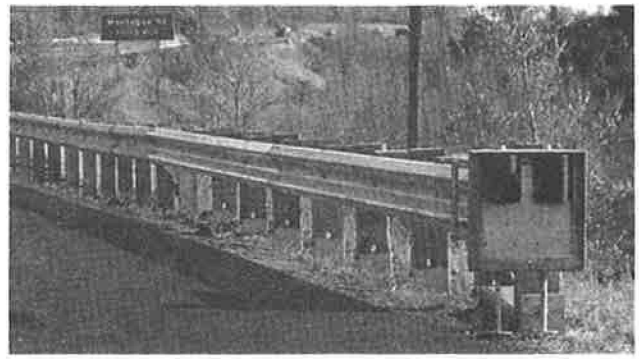


FIGURE 5 ET-2000, guardrail extruder terminal.

a level considered prohibitive in many states in all but the most accident-prone locations.

With significant questions regarding the adequacy of the lower-cost and most widely used terminals, three new devices have recently reached the market. They are BRAKEMASTER, CAT (crash-cushion attenuating terminal), and ET-2000, shown in Figures 3–5. These devices, described in more detail in the following section, meet safety requirements, but are more expensive than BCTs and turndowns.

OPERATIONAL EXPERIENCE

Figure 6 shows the periods during which various designs were used. The blunt end has been in use from the beginning and for decades has been recognized as an extremely hazardous roadside object. However, it has never been replaced on low-volume highways and is still being constructed in some counties and municipalities.

The turndown has been widely applied since the late 1960s but is now slowly being replaced in some states. Many states are still satisfied with BCTs. This satisfaction may be because care has been taken to ensure compliance with the 4-ft flare requirement or because the accident experience has not been documented and evaluated. The other five terminals are still

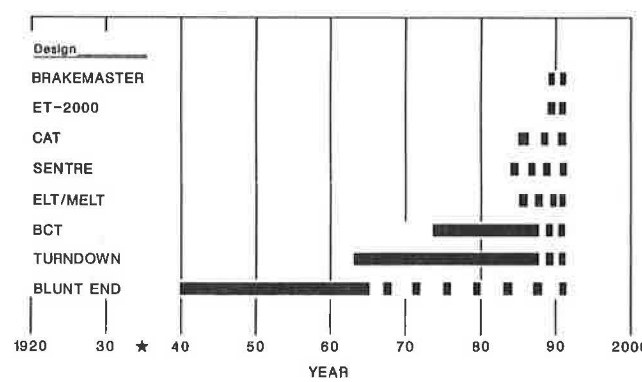


FIGURE 6 Periods of use for various end treatments.

relatively new, but significant positive operational experience should be acknowledged for Sentre and CAT. ET-2000 is rapidly gaining that experience.

BCT Design and Installation

Since the original BCT design drawing was introduced in 1972, a number of changes or alternatives to this design have been developed by NCHRP (*8,16,17*):

- An end-post size change from 8 × 8 in. to 6 × 8 in.,
- A steel, slip-base post alternative,
- Recommended omission of the steel nose diaphragms, and
- Steel tube foundation alternative to concrete footings.

Other known changes to the design have been incorporated into state standards and are not considered advisable:

- Reduction or omission of the 4-ft lateral offset, use of a straight taper in lieu of a parabolic flare at the end, or both;
- Use of a 10-gauge W-beam; and
- Use of a rub rail within the 37.5-ft flare length, sometimes with increased beam height.

Other observed installation errors include the following:

- Not building 4-ft offset, 37.5-ft parabolic flare as shown on standard,
- Installing end on steep slope or near slope break,
- Lack of consideration of run-out for vehicles impacting or narrowly missing terminal (too short),
- Installing beam too high or too low,
- Inadequate foundation for end posts (could be due to poor geometrics, concrete material, weak soil, etc.), and
- Use of a square washer not in compliance with the plans.

Examples of proper and improper installations are shown in Figure 7.

BCT Accident Experience

New Jersey (*11*) and Indiana (*12*) both reported adverse accident experience with straight or moderately flared BCTs. Both reported more satisfactory results when the full 4-ft offset flare was constructed. The latest reports from Kentucky (*13,18*) recommend using the BCTs where the full 4-ft offset can be obtained. Other states have related satisfactory experience with the BCT.

Among the notable problem areas in addition to those attributed to improper installation are impacts with the side of a vehicle and impacts in which the beam enters the wheel well area. BCTs, like all terminals, were developed using frontal impact and do not perform well in side impacts. In addition, when the beam goes between the wheel and the engine, an area of minimal resistance is encountered. This has resulted in penetration into the passenger compartment.

IMPLEMENTATION PROBLEMS

The most significant problem for the BCT is the 4-ft offset, 37.5-ft long parabolic flare. In many observed cases, there was adequate space to accommodate this critical geometry, but it was not done. This failure is often associated with sloping terrain problems. It is sometimes judged that it is safer to reduce the offset distance than to carry the flare down sloping terrain, but this is probably incorrect in most cases.

The foregoing paragraphs lead to the conclusion that the BCT is not always dependable. The FHWA memorandum of March 27, 1991, called for multistate financing of research to improve BCTs (*19*). This is an ongoing study.

TURNDOWNS

Griffin (*20*) sought to determine if, and to what degree, turned-down guardrail ends constitute a safety problem. He attempted to estimate the number of vehicles that overturned on turned-down guardrail ends in Texas in one year and the number of people who were seriously injured in accidents involving turned-down guardrail ends.

These accidents were drawn from the 190,512 accidents reported to have occurred on the Texas highway system in 1989. Of these 190,512 accidents, 4,047 (2.1 percent) were alleged to involve an impact with a guardrail. Of these, 100 were fatal accidents, although the guardrails (or their turned-down ends) may have had little or nothing to do with the fatalities.

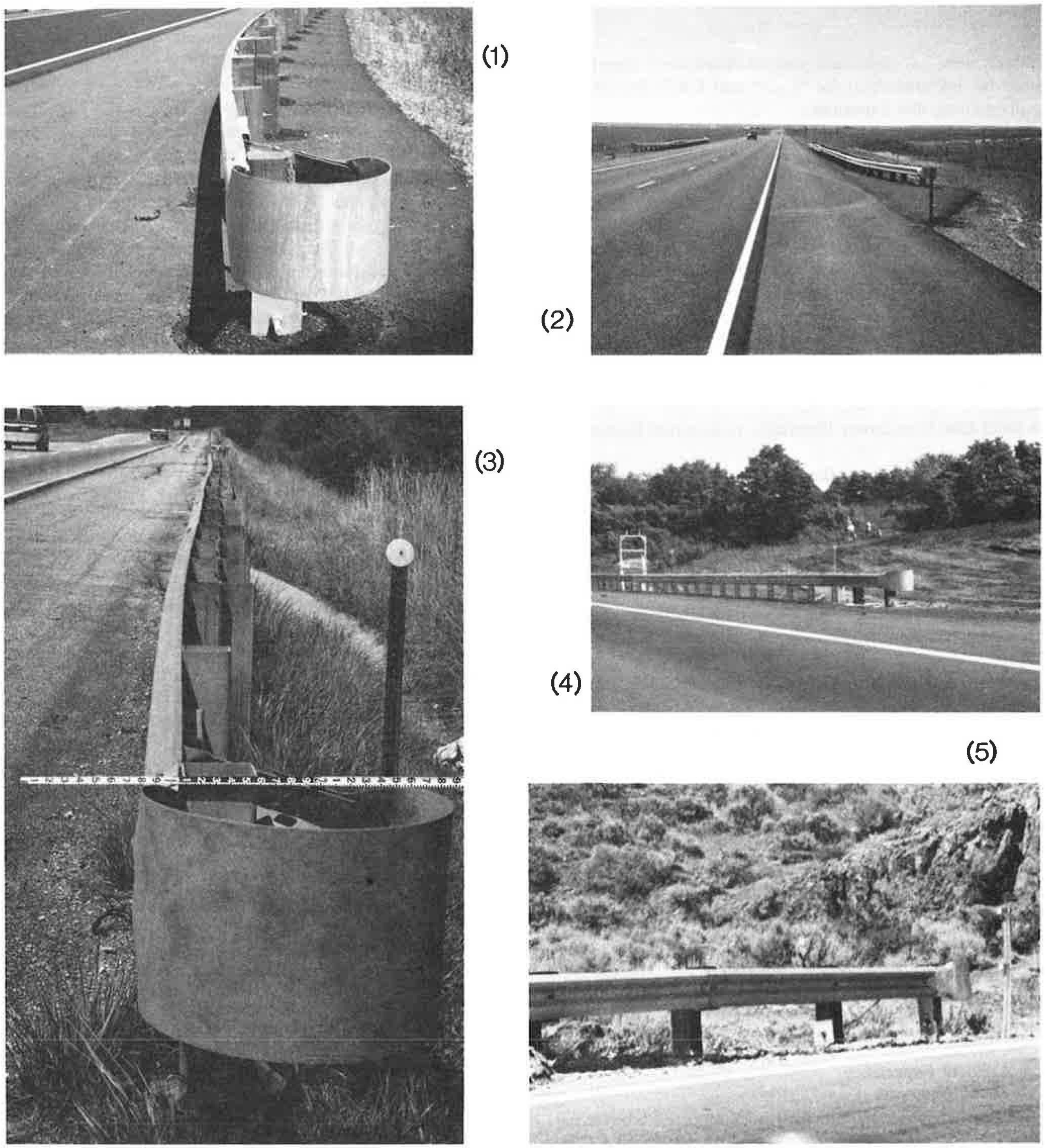
The police accident reports for all 100 fatal accidents and a 25 percent sample of the remaining 3,947 nonfatal accidents were reviewed to answer two questions: Was the point of impact in the accident with the end of the rail (i.e., on a turned-down end) or somewhere else on the rail? and Did the vehicle overturn?

The answers led to the following conclusion (*20*).

> It is estimated on the Texas state-maintained highway system in a typical year some 736 accidents occur on turned-down guardrail ends. 278 of these vehicles overturn, 43 individuals are killed and another 85 sustain incapacitating (A-Level) injuries. These are considered unsatisfactory statistics.
>
> It should also be understood that the degree to which vehicle overturns and driver/occupant deaths and injuries could be reduced by replacing turned-down guardrail ends with other end treatments (e.g., breakaway cable terminals) is unknown. The analyses contained in this report suggest that fatal accidents on turned-down guardrail ends tend to be associated with high speeds, drunk driving, darkness, sleeping/fatigued drivers, etc.

IMPROVED TERMINALS

Throughout the 1970s and early 1980s BCTs and turndowns were rapidly deployed. Without good field information these end treatments seemed to be adequate. Their advantages over the blunt end were clear. Widespread implementation of the turndown and BCT also occurred before there was a recognized test matrix for terminals. Few terminal tests were conducted according to NCHRP Report 153 (*21*) before 1981 when NCHRP Report 230 (*9*) was published. SwRI demon-



(1) Parabolic flared BCT. PROPER
(2) Parabolic flared BCT, local shoulder widening*.
(3) Straight steel slip base post BCT (offset < 9"). IMPROPER
(4) Straight wood post BCT (0 offset). IMPROPER
(5) Cable tension member across the wrong diagonal of the first opening. IMPROPER

* Since the grade of the soil is not at the same grade of the ACP shoulder addition the vehicle trajectory just before impact may be adversely affected.

FIGURE 7 Properly and improperly installed BCTs.

strated the poor characteristics of turndowns as early as 1969 and in other tests in 1982 showed that BCTs did not perform well when struck by small cars (*10*). Attempts to modify these popular treatments resulted in the eccentric loader BCT, GEET1 and CRT, but in terms of implementation, success was limited.

Sentre

Leading developments in the early 1980s was Energy Absorption Systems, Inc., with Sentre. Sentre consisted of overlapping segments of Thrie beam guardrail mounted on blocked out steel posts with slipbases. "Sandbox" inertia elements absorbed some energy, and a redirection cable moved a collapsing Sentre laterally. Sentre performed well in tests required in NCHRP Report 230 (*22*) and has also performed well in the field. Through 1987, 31 collisions with Sentre were documented by the manufacturer. Performance was judged to be good in 29 cases and marginal in 2 cases. As of July 1991, there were 475 Sentre installations in 19 states. A fact summary sheet is shown in Figure 8.

CAT

The race was on for a high-performance end treatment at reasonable cost as problems with BCT and turndown became better understood. The next entry was developed by FHWA, SwRI, Syro Steel Company. The design was originally called

GUARDRAIL END TREATMENT
FACT SUMMARY

NAME SENTRE

MANUFACTURER(S) Energy Absorption Systems, Inc.

DEVELOPER(S) Energy Absorption Systems, Inc.

DEVELOPMENT PERIOD 1981 **to** 1983 (Basic System)

NUMBER OF DEVELOPMENT CRASH TESTS 22 basic, and 11 supplimental performance tests.

DATE OF FIRST FIELD INSTALLATION December, 1983.

STATES USING THE DEVICE Oklahoma, Texas, Indiana, Illinois, Delaware, Maryland, Michigan, District of Columbia, Utah, New Mexico, Arizona, Connecticut, New Hampshire, Pennsylvania, and Vermont; also Nevada, Hawaii, Iowa, and Washington.

APPROXIMATE NUMBER OF DEVICES NOW IN USE 475

FIELD COLLISION EXPERIENCE See attached. The impact data has not been collected by the state DOTs since the SENTRE system was accepted as operational by FHWA on April 7, 1989.

THIS DEVICE IS A STANDARD IN THE FOLLOWING STATES (see addendum sheet attached)

*** COST OF HARDWARE (a)** (see addendum sheet attached)

*** COST OF INSTALLING (b)** (see addendum sheet attached)

*** COST OF DEVICE INSTALLED (c)** (see addendum sheet attached)

COST OF RESTORATION (see addendum sheet attached)
(Subsequent to a major collision)

INDIVIDUAL PROVIDING INFORMATION J. M. Essex, Vice President, Sales

***** **It is understood that (a), (b) and (c) are not independent. In some cases only (c) may be available.**

FIGURE 8 Fact summary sheet for Sentre (***continued on next page***).

THIS DEVICE IS A STANDARD IN THE FOLLOWING STATES:

Accepted by FHWA as "operational highway hardware" on April 7, 1989. Several states have this system as one of their operational end treatments and it remains to the designer's decision as to which end terminal he specifies for a site.

COST OF HARDWARE (a):

Range depends upon anchorage option chosen. From $1700 to $4850.

COST OF INSTALLING (b):

Range depends upon anchorage option chosen and contractor capability. From $500 to $2500.

COST OF DEVICE INSTALLED (c):

Range depends on amount of preliminary site work required by specification. Early bid prices were non-typical due to installations being "experimental".

COST OF RESTORATION (Subsequent to a major collision):

Range estimated from $100 to $1700. Based on severity of impact up to design limits.

FIGURE 8 *(continued).*

The Shredder, but evolved through the designation of vehicle attenuating terminal to CAT. CAT met NCHRP Report 230 requirements and was a good step toward reasonable cost. CAT has been evaluated using 4,500- and 1,800-lb automobiles and a 5,400-lb pickup (*23–26*). CAT can be installed parallel to the road without flaring.

Projected cost is about $3,700. The 42 collisions now reported indicate good field performance. An installation is shown in Figure 4, and a fact summary sheet is shown in Figure 9.

ELT

FHWA has continued efforts to make variations of the BCT acceptable. ELT was the first stage of BCT evolution (*27*). ELT has also been evaluated for an end-on impact with a 5,400-lb pickup (*28*). Because of the problems in implementing the 4-ft flare, both 4-ft and 1.5-ft flare offset designs were tested. The results of these tests fundamentally meet the NCHRP Report 230 criteria, but the 1.5-ft flare offset design was considered marginal. A fact summary sheet for ELT is shown in Figure 10.

MELT

FHWA has recently designated MELT as operational (*29*). MELT is an FHWA design that differs from ELT in the nose piece. MELT functions reasonably well with a 4-ft flare, but head-on performance remains a concern when the flare is reduced to 1.5-ft, which has not been tested (*29*). FHWA officials believe that MELT should perform as well as ELT. The main advantage of MELT in comparison with high-performance terminals is its cost, projected to be about $1,000, excluding earthwork. The main disadvantages of MELT are possibly the same shortcomings of all BCT designs.

ET-2000

ET-2000 was developed progressively by the Texas Transportation Institute, Texas DOT, and SYRO. ET-2000 meets the criteria in NCHRP Report 230 (*30*). This device works in a unique way. A die at the end of the rail acts as an extruder in a vehicle collision. The die bends the W-section 90 degrees, flattens it, and projects it out away from the vehicle. The cost of installation is about $2,300. A fact summary sheet is shown in Figure 11.

BRAKEMASTER

BRAKEMASTER, from Energy, is shown in Figure 3 and functions in the following way. The forward structural elements of the terminal include a unique braking mechanism on a heavy longitudinal cable. When a vehicle strikes BRAKEMASTER head-on, the braking mechanism is pushed

GUARDRAIL END TREATMENT
FACT SUMMARY

NAME CAT (For use as a crash cushion, median terminal, shoulder terminal)

MANUFACTURER(S) Syro Steel Company - Girard, Ohio & Centerville, Utah

DEVELOPER(S) Southwest Research Institute (SwRI)

DEVELOPMENT PERIOD January 1983 **to** January 1988

NUMBER OF DEVELOPMENT CRASH TESTS 32

DATE OF FIRST FIELD INSTALLATION November 1986

STATES USING THE DEVICE Alaska, Arizona, California, Colorado, Connecticut, Delaware, Illinois, Indiana, Kentucky, Maine, Maryland, Michigan, Minnesota, Missouri, Nebraska, New Hampshire, New Mexico, North Carolina, Ohio, Pennsylvania, South Carolina, Tennessee, Texas, Utah, Virginia, Washington, West Virginia, Wyoming. Also Canada

APPROXIMATE NUMBER OF DEVICES NOW IN USE 800 (576)

(42)

FIELD COLLISION EXPERIENCE 59 impacts reported to date with no fatalities resulting from impacting the C-A-T. Accident data was compiled and submitted to the FHWA. On June 4, 1990 FHWA moved the C-A-T from experimental to operational. Numbers in parentheses are those associated with CATs used as terminals.

THIS DEVICE IS A STANDARD IN THE FOLLOWING STATES About 40% of the above states bid the C-A-T regularly.

* **COST OF HARDWARE (a)** $3300 terminal - $4700 crash cushion

* **COST OF INSTALLING (b)** $400 terminal - $600 crash cushion

* **COST OF DEVICE INSTALLED (c)** $3700 terminal - $5300 crash cushion

COST OF RESTORATION $3000.00
(Subsequent to a major collision)

INDIVIDUAL PROVIDING INFORMATION John C. Durkos, Syro Steel Company

* **It is understood that (a), (b) and (c) are not independent. In some cases only (c) may be available.**

FIGURE 9 Fact summary sheet for CAT.

down the cable and the side W-beam sections telescope. BRAKEMASTER has performed well in NCHRP 230 testing (*31*). An average installation costs about $5,000. A fact summary sheet is shown in Figure 12.

TRENDS IN TERMINAL USE

A comparison of the various terminal designs now in use is presented in Table 1. It is based on data from the manufacturers and FHWA. Of devices with good performance, Sentre has been used the longest, more than 7 years. The installation rate of Sentre is mid-range at 63 per year. Next in longevity, ELT and MELT, have the lowest installation rate at seven. CAT, in use for 5 years, has the highest installation rate at 123. BRAKEMASTER, in use for 2 years, has an installation rate averaging 29. Finally, ET-200, in use for 1 year, has an installation rate of 88. In field experience, only Sentre and CAT could be called field-proven devices. ELT and MELT, BRAKEMASTER, and ET-2000 all need additional exposure before they can be so categorized. For an independent evaluation of the performance of these terminals, the reader may refer to work by Jewel et al. (*15*).

CONCLUSION

In the highway safety field, the engineer responsible for "forgiving roadsides" is in an unaccustomed position relative to guardrail end treatments. After decades of confronting the

GUARDRAIL END TREATMENT
FACT SUMMARY

NAME ELT

MANUFACTURER(S) Not proprietary (Syro, Trinity, Mission, etc.)

DEVELOPER(S) Southwest Research Institute and FHWA

DEVELOPMENT PERIOD ______ to ______

NUMBER OF DEVELOPMENT CRASH TESTS ______

DATE OF FIRST FIELD INSTALLATION 1986

STATES USING THE DEVICE South Dakata, Utah, Washington, Michigan and New Jersey.

APPROXIMATE NUMBER OF DEVICES NOW IN USE 35 (50)*

FIELD COLLISION EXPERIENCE One hit in South Dakota a few weeks after installation. Results were good.

* As per a meeting of January 14, 1992 with FHWA engineers there may be fifty of these installations in the U.S. and up to 300 in Canada

THIS DEVICE IS A STANDARD IN THE FOLLOWING STATES None

* COST OF HARDWARE (a) (Syro)

* COST OF INSTALLING (b) Varies

* COST OF DEVICE INSTALLED (c) $1000

COST OF RESTORATION $1200
(Subsequent to a major collision)

INDIVIDUAL PROVIDING INFORMATION Richard Powers, FHWA, (202) 366-1320

* It is understood that (a), (b) and (c) are not independent. In some cases only (c) may be available.

FIGURE 10 Fact summary sheet for ELT.

necessity of choosing between marginally performing systems, the engineer is now confronted with an array of choices. These choices are systems with vastly improved performance characteristics. This conclusion assumes that proving-ground testing will relate well to field experience. A few of the new designs have significant field exposure, but others are young in application.

In an effort to summarize the performance of current designs, the following categories are proposed:

- I Unacceptable performance,
- II Improved performance based on comparisons with Category I with questionable field experience,
- III Marginal performance based on compliance testing with questionable field experience or lack of field experience,
- IV Acceptable performance based on compliance testing but without significant field experience, and
- V Acceptable performance based on compliance testing and field experience.

The various competing systems were categorized on the basis of compliance crash testing and field experience. The costs were supplied by FHWA, Energy, and Syro. These data are shown in Figure 13. The figure shows the trade-off between cost and performance. At this time, as costs per system increase, the field-verified performance level increases. The

GUARDRAIL END TREATMENT
FACT SUMMARY

NAME ET-2000 (For use as a shoulder terminal for guardrail)

MANUFACTURER(S) Syro Steel Company - Girard, Ohio and Centerville, Utah

DEVELOPER(S) Texas Transportation Institute (TTI)

DEVELOPMENT PERIOD September 1985 **to** June 1989

NUMBER OF DEVELOPMENT CRASH TESTS 14

DATE OF FIRST FIELD INSTALLATION June 1990

STATES USING THE DEVICE Illinois, Minnesota, Missouri, Texas and Utah.

APPROXIMATE NUMBER OF DEVICES NOW IN USE 105

FIELD COLLISION EXPERIENCE Only one hit has been reported to date. A 1984 Mazda pickup truck impacted the ET-2000 end-on. The estimated speed was 60 mph and the driver was not injured.

THIS DEVICE IS A STANDARD IN THE FOLLOWING STATES Texas and Utah now, but many in addition to the above plan to incorporate.

* **COST OF HARDWARE (a)** $1,900.00

* **COST OF INSTALLING (b)** $400.00

* **COST OF DEVICE INSTALLED (c)** $2,300.00

COST OF RESTORATION $500.00
(Subsequent to a major collision)

INDIVIDUAL PROVIDING INFORMATION John C. Durkos, Syro Steel Company

* **It is understood that (a), (b) and (c) are not independent. In some cases only (c) may be available.**

FIGURE 11 Fact summary sheet for ET-2000.

placement of various terminals in the proposed categories is somewhat subjective.

The following are additional observations from Figure 13:

- If field-verified performance is most important, the choice is probably between Sentre and CAT. They may be economically justified in areas in which many collisions occur.
- If MELT moves into Category V, the cost advantages would be considerable.
- Any end condition in Category I should be replaced or modified as quickly as is economically feasible.
- Any terminal in Category II (turndowns and nonflared BCTs) should be gradually phased out, with emphasis placed on those sites where collisions are most likely. The exception to this may be low-volume rural roads in low-exposure locations.
- There will probably soon be four systems in Category V, contingent on continued good field experience with Sentre and CAT, and with developing good field experience with BRAKEMASTER and ET-2000. This should result in a brisk competition resulting in design improvements and cost reductions.

There are still problems in accurately predicting terminal performance, and costs will vary widely and change often. Known performance levels and costs are now approaching the

GUARDRAIL END TREATMENT
FACT SUMMARY

NAME BRAKEMASTER

MANUFACTURER(S) Energy Absorption Systems, Inc.

DEVELOPER(S) Energy Absorption Systems, Inc.

DEVELOPMENT PERIOD 1987 **to** 1989

NUMBER OF DEVELOPMENT CRASH TESTS 62

DATE OF FIRST FIELD INSTALLATION November, 1989

STATES USING THE DEVICE South Carolina, Colorado, Kentucky, Wisconsin, Minnesota, Tennessee, and Pennsylvania; also Oregon and Alabama.

APPROXIMATE NUMBER OF DEVICES NOW IN USE 50

FIELD COLLISION EXPERIENCE (see attached summary)

THIS DEVICE IS A OPERATIONAL **~~STANDARD~~ IN THE FOLLOWING STATES** Accepted as "experimental" by FHWA on October 30, 1989.

*** COST OF HARDWARE (a)** (see addendum sheet attached)

*** COST OF INSTALLING (b)** (see addendum sheet attached)

*** COST OF DEVICE INSTALLED (c)** (see addendum sheet attached)

COST OF RESTORATION (see addendum sheet attached)
(Subsequent to a major collision)

INDIVIDUAL PROVIDING INFORMATION J. M. Essex, Vice President, Sales

* **It is understood that (a), (b) and (c) are not independent. In some cases only (c) may be available.**

ADDENDUM SHEET:

COST OF HARDWARE (a):

Range based on anchorage option chosen; from $3000 to $5000.

COST OF INSTALLING (b):

Range based on anchorage option chosen; from $500 to $1500.

COST OF DEVICE INSTALLED (c):

Range based on site work specified and contractor capability. Not available from manufacturer. Number of systems specified to be bid will influence this price also.

COST OF RESTORATION:

From $200 to $2800.

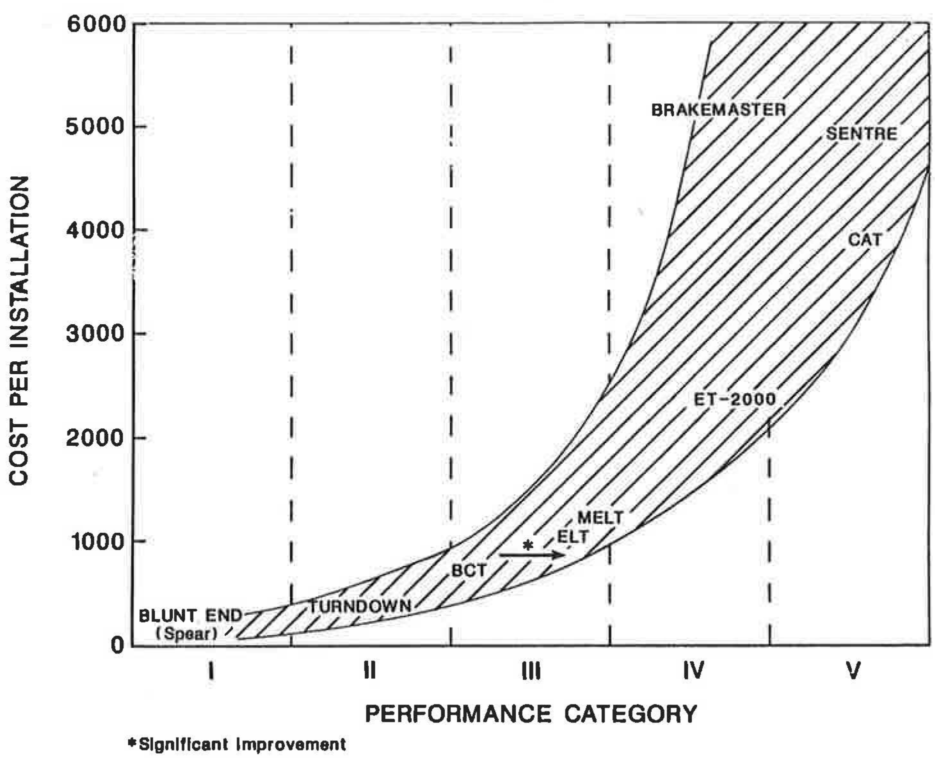
FIGURE 12 Fact summary sheet for BRAKEMASTER.

TABLE 1 Terminal Installations in Use in the United States

	Number of Terminals Installed	First Installation (Year)	Time in use (Years)	Average Installation Rate in U.S. (Inst. per yr.)
TURNDOWN	More than 450,000	1963	28	---
BCT	More than 450,000	1973	18	---
ELT	50	1986	5	10
MELT	0	---	---	---
BRAKEMASTER	50	Nov., 1989	1.7	29
CAT	576	Nov., 1986	4.7	123
ET-2000	105	June, 1990	1.2	88
SENTRE	475	Dec., 1983	7.6	63

point, however, at which benefit-cost analysis can be used to determine which systems are most appropriate for specific sites or classes of sites (*32,33*). That should be the next step.

The highway engineer is now blessed by good choices in the selection of guardrail terminals. Since there has been much said about meeting NCHRP 230 experimental requirements and in gaining field experience for the terminals under consideration, perhaps both writers and readers might consider the following statement by Leonardo da Vinci "Experience does not ever err, it is only your judgment that errs in promising itself results which are not caused by your experiments."



Note: Median costs are plotted for Sentre and BRAKEMASTER. Depending on site characteristics, Sentre may vary ± 50%, while BRAKEMASTER may vary ± 30%.

FIGURE 13 Current costs and performance categories.

ACKNOWLEDGMENT

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DISCUSSION

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Although the authors of this paper present an accurate chronology of guardrail terminal evolution, they fail to discuss specific reasons for alleged dissatisfaction with some of the commonly used generic appurtenances. Thus, the paper leads one to the conclusion that proprietary terminals should be used regardless of their cost. Although terminals such as turndowns and BCTs do not perform well under all circumstances, many years of accumulated experience have revealed that they perform satisfactorily most of the time, particularly when installed and maintained properly and when careful attention is given to site selection and grading. Site considerations should include such exposure and risk factors as traffic volumes and speeds and the selection of an appropriate level of service.

The newer proprietary terminals in general do have superior energy-absorbing capabilities and have exhibited good in-service performance in their limited exposure to date. It is important to note, however, that all terminals have inherent limitations and that none will perform satisfactorily over the entire spectrum of possible impacts. Highway agencies would be well advised to keep current on the latest developments in the barrier terminal field, to become familiar with the advantages and disadvantages of each system, and to select the most cost-effective terminal for each specific site. A continuing performance evaluation of all in-service terminals will provide invaluable information in the on-going selection process.

DISCUSSION

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In the abstract of this paper, the authors indicate that the paper provides an objective review of guardrail terminals and aid for selecting cost-effective terminals to meet specific needs.

The criteria upon which the objective review is based are highly subjective in that there is no demonstrated correlation between the review criteria and actual field performance. Presumably, the authors will argue that crash test results are objective. I would suggest that the current test procedures, with tests conducted on flat, level ground with tracking vehicles impacting over a narrow range in speed and angle, basically provide rough go, no-go screening and provide little basis for discriminating between various terminal types because they fall far short of examining the full range of service conditions. Thus, until laboratory practices are changed, the only valid basis I see for rating terminals would be cogent, comparative statistical analyses of their field performance.

Information presented in the paper on field performance is primarily anecdotal, except for work by Griffin on the "Texas twist" terminal. Griffin deserves recognition for his work; it should provide guidance and encouragement to others. However, his results neither support nor argue against continued use of the Texas twist because there is no information presented on how well the alternatives might work. What his results do show is that striking a Texas twist terminal can be hazardous. Work by others, notably that by Agent and Pigman, show hazardous results from striking other types of terminals. Several of the terminals cited in the paper have not been in service long enough to have demonstrated their safety performance. Therefore, from the information presented, there is no basis on which an objective assessment of the relative safety performance of the various terminals can be made. Nevertheless, the authors are correct in suggesting the need for objective guidance in the selection of guardrail terminals. The problem is that much more field evaluation of terminal performance is needed to form a basis for such guidance.

This paper does not provide aid in selecting cost-effective terminals to meet specific needs, even though this needed aid is promised in the abstract.

My expectation is that even the terminals the authors suggest as superior performers, if subjected to field evaluation, would be shown to represent significant hazards, though, possibly, they would not be shown to be as hazardous as some of the existing alternatives.

Readers who are in a position to do so should institute field evaluation programs of terminals and use the evaluation results to develop procedures for selecting cost-effective guardrail terminals for given site conditions. I also submit that further improvement in guardrail terminals is needed and that properly designed field evaluations of existing terminals would reveal their shortcomings and provide bases for performance goals for new terminals.

I further suggest that if such evaluations are undertaken that they be extensive, detailed, and include the following considerations:

- Terrain geometries at terminal sites;
- The fact that terminals are impacted by many types of vehicles traveling at various combinations of speeds, angles, orientations, and yaw rates (side-on impacts are probably important);
- Unreported contacts, which will be essential for the analysis of field performance; and
- Site traffic speed, mix, offset, and approach alignment.

AUTHORS' CLOSURE

The authors are grateful to Powers and Hatton for their insightful discussions, their help in clarifying several areas during the writing and review process, and their direct contributions to this paper.

Although the authors are not in total agreement with every point Powers and Hatton make, the areas of agreement are certainly dominant, and readers are advised to consider all the reviewers' points carefully when deciding what weight to give the conclusions and opinions presented by the authors.

Concerning both reviewers' suggestions that field evaluations of terminals be continued or initiated to provide the data for benefit-cost comparisons, the authors could not be more in agreement. To this end, comparisons of the newer devices with the older turndown and BCT devices will only be possible if these new devices are installed in sufficient numbers to obtain meaningful accident data. Field performance is the ultimate evaluation. Only through careful evaluation of performance can the indications of testing be confirmed or rejected and can the relative effectiveness of safety systems be accurately determined.

Minnesota Bridge Rail–Guardrail Transition Systems

KING K. MAK, GLENN R. KORFHAGE, AND J. A. KINDOM

Summarized are the results of crash testing and evaluation of two W-beam guardrail-to-concrete safety-shaped bridge-rail transition designs developed by the Minnesota Department of Transportation. The first, integral end-post design, is intended for use with new construction and reconstruction in which replacement of bridge railings is required. The second, separate end-post design, is intended for use as retrofits to existing bridge railings on 3R and 4R projects. Three crash tests were used to evaluate the integral end-post design. Results of the crash tests indicated that the integral end-post design met all impact performance evaluation criteria according to guidelines outlined in NCHRP Report 230. Two versions of the separate end-post transition design were evaluated, each with one crash test. Results of the first crash test indicated that the initial separate end-post design did not meet the impact performance evaluation criteria according to guidelines outlined in NCHRP Report 230. The vehicle pocketed and impacted the end of the concrete bridge end post, resulting in an unacceptable level of longitudinal occupant ridedown acceleration. The design was then modified and the improved transition design was successfully crash tested.

The primary functions of a bridge rail are to prevent errant vehicles from going over the side of the bridge and to prevent the wheels of an impacting vehicle from falling between the bridge rail and the edge of the bridge deck. Thus, bridge rails must be either rigid or semirigid in construction. The most common types of bridge rails are reinforced concrete walls or metal rails on concrete parapets. If improperly treated, the exposed ends of these railings can present a serious safety hazard to errant vehicles.

In most instances, an approach guardrail is used to shield the exposed end of the bridge railing and to prevent errant vehicles from getting behind the railing and encountering underlying hazards. These approach guardrails are typically much more flexible than the bridge rails to which they are attached and thus have the potential for deflecting sufficiently to allow an errant vehicle to impact the end of the rigid or semi-rigid bridge railing. A transition section is therefore used whenever there is a significant change in lateral strength from the approach guardrail to the bridge railing.

The purpose of a transition section is to provide continuity of protection where an approach guardrail joins a bridge rail. In order to achieve this continuity of protection, the lateral stiffness of the transition section should be increased smoothly and continuously from the more flexible to the less flexible system. This required increase in lateral barrier strength can be achieved by varying one or more key design parameters, including increasing guardrail beam strength, reducing post spacing, and increasing post size or embedment depth. An effective transition design is one that limits dynamic deflection and minimizes vehicle pocketing or snagging on the end of the bridge railing.

Two W-beam guardrail-to-concrete safety-shaped bridge-rail transition designs were developed by the Minnesota Department of Transportation (MnDOT). The designs were the product of an MnDOT committee and were based on review of literature and FHWA-approved transition designs and on existing field conditions, including bridge railing, curb, and approach guardrail designs. Another consideration in the design was the use of only standard in-stock components to minimize maintenance and inventory problems.

MnDOT contracted with the Texas Transportation Institute to crash test and evaluate the impact performance of these two Minnesota transition designs (*1*,*2*). These two transition designs are referred to herein as integral end-post design and separate end-post design. The integral end-post design is intended for use with new construction and reconstruction in which replacement of bridge railings is required. The separate end-post design is intended for use as retrofits to existing bridge railings on 3R and 4R projects on roadways with speed limits above 40 mph and average daily traffic (ADT) volume of more than 1,500 vehicles. The impact performance of these two transition designs is summarized and presented here.

INTEGRAL END-POST TRANSITION DESIGN

The integral end-post transition design incorporates special steel reinforcements near the end of the standard concrete safety-shaped bridge rail so that the W-beam guardrail transition can be attached directly to the bridge rail, and thus the term "integral end post". The transition from the standard concrete safety-shaped bridge rail to the standard G4(2W) W-beam guardrail spans a length of 25.0 ft (7.6 m). The major features of the transition design are as follows:

- The first 12.5-ft- (3.8-m-) long section of W-beam is nested (i.e., one W-beam is placed on another). The nested W-beams are attached to a standard W-beam terminal connector and anchored to the concrete parapet with four ⅞-in. × 12.0-in. (2.22-cm × 30.5-cm) high-strength bolts and nuts and 2-in. × 3-in. × ¼-in. (5.08-cm × 7.62-cm × 0.64-cm) plate wash-

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ers. A single W-beam is used for the second 12.5-ft (3.8-m) guardrail section.

- In addition to the nested W-beams, a reduced post spacing is used to transition the lateral stiffness between the systems. The first post is 8½ in. (21.6 cm) from the end of the concrete parapet. The post spacings are 1 ft 6¾ in. (0.48 m) center to center (3 spaces) for Posts 1 through 4, 3 ft 1½ in. (0.95 m) center to center (4 spaces) for Posts 5 through 8, and 6 ft 3 in. (1.91 m) for Post 9 and beyond. All posts used in the transition are standard 6-in. × 8-in. × 6-ft (15.2-cm × 20.3-cm × 1.83-m) timber posts. The W-beam sections are attached to the posts with ⅝-in button head bolts and recess nuts with rectangular washers.
- A 7-ft- (2.1-m-) long curb transition is used from the top of the lower slope of the concrete safety-shaped barrier at 13 in. (33.0 cm) above ground to a 4-in. (10.2-cm) high roll curb, which then continues throughout the remainder of the 25-ft (7.6-m) transition area. The curb section facilitates drainage and reduces the potential of wheel snagging by an impacting vehicle on the posts.
- The upper corner of the concrete parapet is tapered from a height of 32 in. (81.3 cm) to 27 in. (68.6 cm) so that it is level with the top of the W-beam to minimize the potential of snagging on the exposed corner.

The test installation in this study consisted of a simulated concrete bridge parapet that incorporated both the integral end-post and the separate end-post designs. To simplify construction, a single foundation, 14.1 ft (4.3 m) in length, was designed and constructed for use with both transition designs. A 12.0-ft- (3.7-m-) long section of the integral end-post bridge railing and a 2.0-ft- (0.61-m-) long separate end post, separated by a construction joint, were attached to the foundation. The guardrail installation consisted of the 25-ft (7.6-m) transition area, 25 ft (7.6 m) of standard G4(2W) W-beam guardrail, and a 25-ft (7.6-m) W-beam turndown terminal anchor for a total length of 75 ft (22.9 m). Photographs of the completed test installation are shown in Figure 1.

Three crash tests were conducted for the integral end-post transition design:

- A 4,500-lb (2,041-kg) passenger car impacting the transition at a speed of 60 mph (96.5 km/hr) and an angle of 25 degrees. The point of impact was midspan between guardrail Posts 6 and 7, 13 ft 5½ in. (4.1 m) upstream from the end of the bridge parapet. This is the required test for a transition installation, according to NCHRP Report 230 (*3*).
- A 1,800-lb (817-kg) passenger car impacting the transition at a speed of 60 mph (96.5 km/hr) and an angle of 20 degrees. The point of impact was just downstream from guardrail Post 5, 8 ft 6¼ in. (2.6 m) from the end of the bridge parapet. This test is intended to assess the potential for wheel snagging on the posts or for the tire to wedge in the area between the W-beam and the curb transition section.
- A 4,500-lb (2,041-kg) passenger car impacting the transition at a speed of 60 mph (96.5 km/hr) and an angle of 25 degrees. The point of impact was 1 ft (0.3 m) downstream from guardrail Post 9, 23 ft 4¾ in. (7.1 m) from the end of the bridge parapet. This test was intended to assess the effect of the curb section on impact performance and the potential for snagging or pocketing at the end of the nested W-beam section.

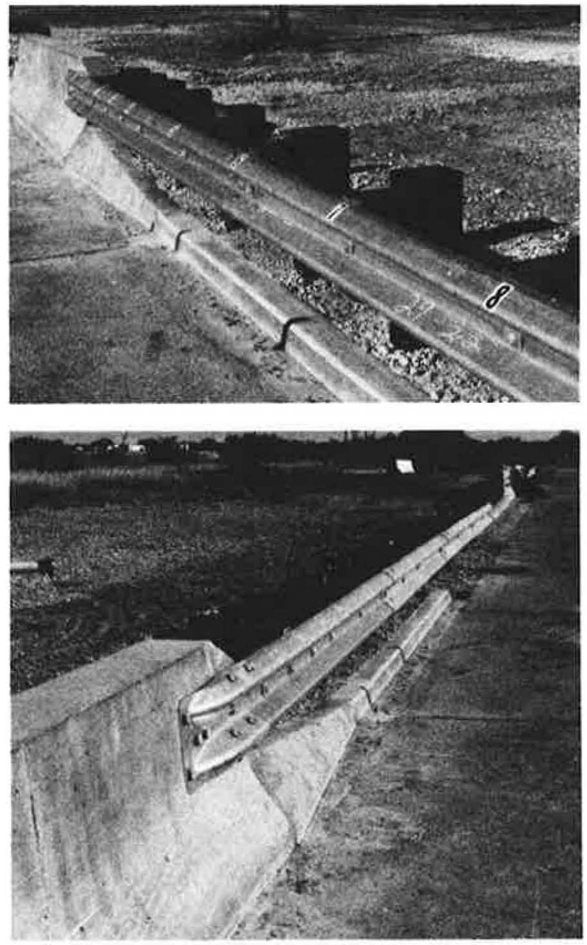

FIGURE 1 Photographs of integral end-post transition design test installation.

The crash test procedures and evaluation of the impact performance were in accordance with the guidelines in NCHRP Report 230. A summary of the test results is presented in Table 1. Following are brief descriptions of the tests and discussions of the results.

Test 1

A 1982 Oldsmobile Ninety-Eight hit the midspan between guardrail Posts 6 and 7, 13 ft 5½ in. (4.1 m) upstream from the end of the bridge parapet, at 59.8 mph (96.2 km/hr) and an angle of 25.4 degrees. The test weight of the vehicle was 4,500 lb (2041 kg). The transition successfully contained and redirected the vehicle. The vehicle remained upright and stable during the initial test period and after leaving the installation.

The transition installation received moderate damage, as shown in Figure 2. There was residual deformation to the nested rail in the area of the first seven posts. In addition,

TABLE 1 Summary of Test Results, Integral End-Post Transition Design

Description	Test 1 (7163-1)	Test 2 (7163-3)	Test 3 (7163-4)
Test Vehicle	1982 Oldsmobile Ninety-Eight	1987 Chevrolet Sprint	1981 Pontiac Bonneville
Test Weight, lb (kg)	4500 (2041)	1969 (894)	4500 (2041)
Impact Speed, mi/h (km/h)	59.8 (96.2)	61.5 (99.0)	61.8 (99.4)
Impact Angle, deg.	25.4	20.5	25.2
Exit Speed, mi/h (km/h)	40.0 (64.4)	43.4 (69.8)	39.7 (63.9)
Exit Angle, deg.	10.2	3.0	7.5
Velocity Change[1], mi/h (km/h)	19.8 (31.8)	18.1 (29.2)	22.1 (35.5)
Occupant Impact Velocity[2]			
Longitudinal, ft/s (m/s)	29.5 (9.0)	28.6 (8.7)	27.5 (8.4)
Lateral, ft/s (m/s)	-25.0 (7.6)	-26.5 (8.1)	-21.3 (6.5)
Occupant Ridedown Acceleration[2]			
Longitudinal, g	-11.4	1.6	-12.4
Lateral, g	12.5	13.2	12.4
Length of Rail Contact, ft (m)	16.3 (5.0)	13.5 (4.1)	19.2 (5.9)
Maximum Dynamic Rail Deflection, in (cm)	13.7 (34.8)	5.2 (13.1)	21.6 (54.9)
Maximum Vehicle Crush, in (cm)	16.0 (40.6)	8.0 (20.3)	16.0 (40.6)

Notes. [1] The velocity change was higher than the recommended value of 15 mi/h (24.1 km/h) in all three tests, but the vehicle was judged not to be a hazard to adjacent traffic lanes.

[2] According to NCHRP Report 230 guidelines, the occupant risk criteria are applicable only for the 1,800-lb passenger car crash test (test 2) and not applicable for the 4,500-lb passenger car crash test (tests 1 and 3).

vehicle tire marks were observed on Posts 2–5 and on the first 4.0 ft (1.22 m) of the concrete bridge parapet. However, there was no apparent structural damage to the concrete bridge parapet. Also, there was no debris or detached elements to show potential for penetration of the occupant compartment or to present undue hazard to other vehicles.

The vehicle received considerable damage to the right front quarter. The right front wheel and control arm were severely bent and pushed rearward 10.0 in. (25.4 cm). The entire front end of the vehicle was shifted 7.0 in. (17.8 cm) to the left. The roof, floor pan in the rear passenger area, hood, and bumper were also damaged. However, there was minimal deformation and intrusion into the occupant compartment.

Test 2

A 1987 Chevrolet Sprint impacted the transition just downstream from guardrail Post 5, 8 ft 6¼ in. (2.6 m) from the end of the bridge parapet, at 61.5 mph (99.0 km/hr) and an angle of 20.5 degrees. The empty weight of the vehicle was 1,800 lb (817 kg); the test weight was 1,969 lb (894 kg). The additional weight was the weight of an unrestrained, uninstrumented, 50th percentile male anthropometric dummy in the driver's seat.

The transition successfully contained and redirected the vehicle. The vehicle remained upright and stable during the initial test period and after leaving the installation. The lateral occupant impact velocity of −26.5 ft/sec (8.1 m/sec) was higher than the design value of 20 ft/sec (6.1 m/sec), but below the limit of 30 ft/sec (9.1 m/sec). The longitudinal occupant impact velocity and the ridedown accelerations were all below the design values outlined in NCHRP Report 230.

The installation received minor damage, as shown in Figure 3. There was residual deformation to the rail in the area of the first six posts, but no apparent structural damage to the concrete bridge parapet. It appeared that the right front tire of the vehicle was momentarily wedged between the W-beam and the top of the curb transition, resulting in the bottom of the W-beam being pushed and deformed slightly upward. However, there is no evidence to indicate that this action had any adverse effect on the vehicle kinematics or trajectory. Also, there was no debris or detached elements.

The vehicle received considerable damage to the right front quarter. The right front wheel and strut assembly/control arm were severely bent and pushed rearward 10.0 in. (25.4 cm). The floor pan, hood, and bumper were also damaged. The passenger door was ajar and the window was broken from the impact by the dummy. However, there was minimal deformation and intrusion into the occupant compartment.

Test 3

A 1981 Pontiac Bonneville impacted the transition 1.0 ft (0.3 m) downstream from Post 9, 23 ft 4¾ in. (7.1 m) from the end of the bridge parapet at 61.8 mph (99.4 km/hr) and an angle of 25.2 degrees. The test weight of the vehicle was 4,500 lb (2,041 kg). The transition successfully contained and redirected the vehicle. The vehicle remained upright and stable during the initial test period and after leaving the installation.

The installation received moderate damage, as shown in Figure 4. There was residual deformation to the rail from Posts 2 through 10, but no apparent structural damage to the concrete bridge parapet. Also, there was no debris or detached elements. The vehicle received severe damage, the majority of which occurred to the right front quarter of the vehicle. The right front wheel and control arm were severely bent and pushed 7.0 in. (17.8 cm) rearward. In addition, the entire front end was shifted 4.0 in. (10.2 cm) to the left. The floor pan and drive shaft were also damaged.

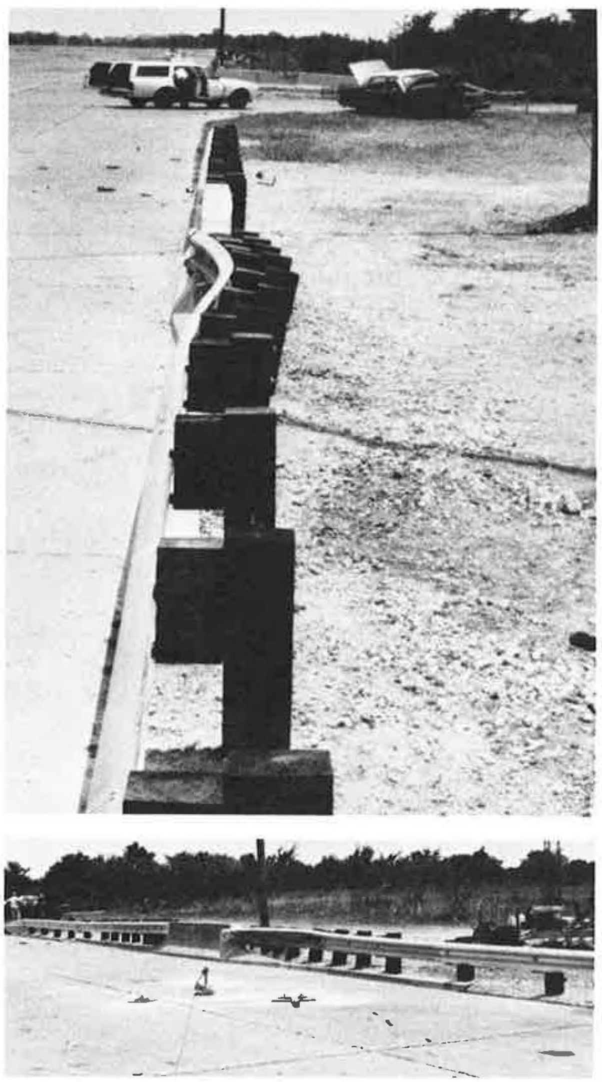

FIGURE 2 Integral end-post transition after Test 1.

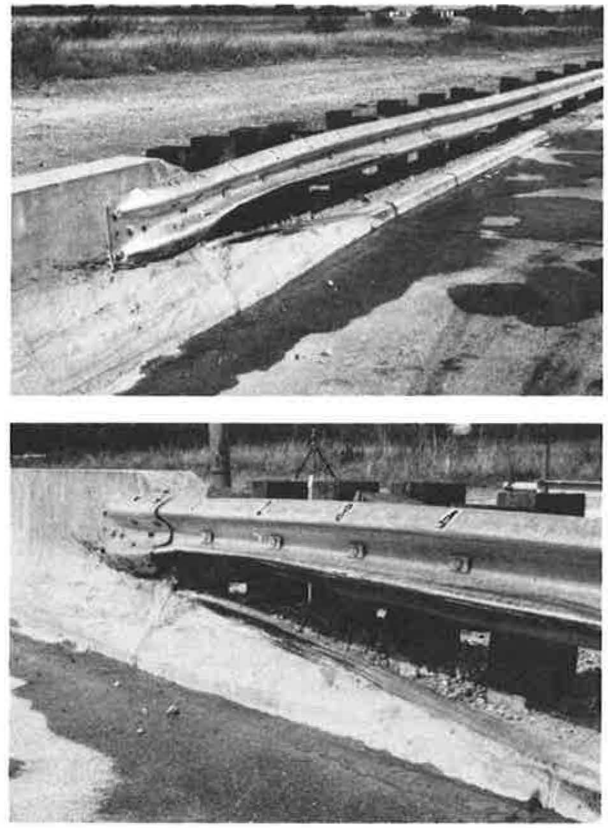

FIGURE 3 Integral end-post transition after Test 2.

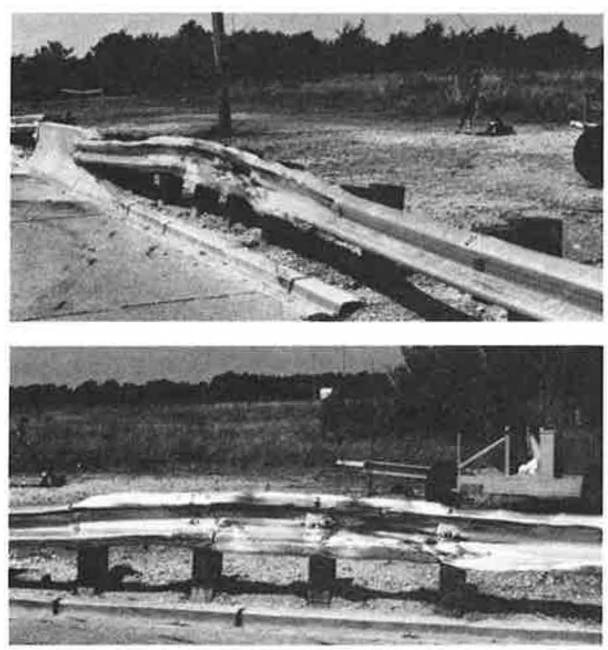

FIGURE 4 Integral end-post transition after Test 3.

Summary

Three crash tests were used to evaluate the integral end-post transition design. Results of the crash tests indicated that the design met all impact performance evaluation criteria according to the guidelines outlined in NCHRP Report 230.

In all three crash tests, the transition successfully contained and redirected the vehicle. There was severe damage to the vehicle, but only moderate damage to the guardrail and no apparent structural damage to the concrete bridge parapet. There was minimal deformation and intrusion into the occupant compartment. The vehicle remained upright and stable during the initial test period and after leaving the installation. The velocity change was higher than the recommended velocity change of 15 mph (24.1 km/hr), but the vehicle was judged to not be a hazard to vehicles in adjacent traffic lanes. It should be noted that this velocity change criterion is seldom met in crash tests involving transition designs.

SEPARATE END-POST TRANSITION DESIGN

The separate end-post transition design, as implied by its name, incorporates an end post 2 ft (0.61 m) in length, separate from the existing bridge railing, for attachment of the W-beam guardrail transition. Two versions of the transition designs were evaluated. The initial design did not perform satisfactorily in the crash testing and was subsequently modified to improve its impact performance. The improved design was then crash tested and found to perform satisfactorily.

One crash test (Test Designation 30 in NCHRP Report 230) was conducted for each of the two versions of the separate end-post transition design, which involved a 4,500-lb (2,041-kg) passenger car impacting the transition at a speed of 60 mph (96.5 km/hr) and an angle of 25 degrees. The point of impact was the midspan of guardrail Posts 7 and 8, 14 ft 8½ in. (4.5 m) from the end of the bridge-rail end post. The crash test procedures and evaluation of the impact performance were in accordance with guidelines presented in NCHRP Report 230. A summary of the test results is presented in Table 2. Following are brief descriptions of the tests and discussions of the results.

Initial Design

The basic design of the initial separate end-post transition design is similar to that of the integral end-post design with the following exceptions.

TABLE 2 Summary of Test Results, Separate End-Post Transition Design

Description	Initial Design (7163-2)	Improved Design (7182-1)
Test Vehicle	1981 Buick Electra	1980 Cadillac Coupe deVille
Test Weight, lb (kg)	4500 (2041)	4500 (2041)
Impact Speed, mi/h (km/h)	57.6 (92.7)	62.2 (100.1)
Impact Angle, deg.	27.3	26.2
Exit Speed, mi/h (km/h)	N/A	44.0 (70.8)
Exit Angle, deg.	N/A	14.4
Velocity Change[1], mi/h (km/h)	57.6 (92.7)	18.2 (29.3)
Occupant Impact Velocity[2]		
Longitudinal, ft/s (m/s)	29.8 (9.1)	24.3 (7.4)
Lateral, ft/s (m/s)	20.8 (6.4)	22.5 (6.9)
Occupant Ridedown Acceleration[2]		
Longitudinal, g	-24.1	- 4.0
Lateral, g	- 9.7	-11.1
Length of Rail Contact, ft (m)	16.9 (5.2)	17.3 (5.3)
Maximum Dynamic Rail Deflection, in (cm)	15.2 (38.7)	20.4 (51.8)
Maximum Vehicle Crush, in (cm)	32.0 (81.3)	10.0 (25.4)

Notes. [1] For the initial design, the vehicle was practically stopped when it exited from the transition and the velocity change was thus the same as the impact speed of 57.6 mi/h (92.7 km/h). For the improved design, the velocity change was higher than the recommended value of 15 mi/h (24.1 km/h), but the vehicle was judged not to be a hazard to adjacent traffic lanes.

[2] According to NCHRP Report 230 guidelines, the occupant risk criteria are not applicable for 4,500-lb passenger car crash tests.

• The height from the top of the W-beam guardrail at its anchor to the concrete parapet is 32 in. (81.3 cm) instead of the standard 27 in. (68.6 cm) height. The height of the top of the W-beam is then gradually reduced to the standard 27 in. (68.6 cm) over the transition length of 25 ft (7.6 m).

• The first post is located 7¾ in. (19.7 cm) from the end of the concrete parapet. The post spacings are 1 ft 6¾ in. (0.48 m) center to center (4 spaces) for Posts 1 through 5, 3 ft 1½ in. (0.95 m) center to center (3 spaces) for Posts 6 through 9, and 6 ft 3 in. (1.91 m) for Post 10 and beyond.

• The curb transition section is only 3 ft (0.91 m) long. Its height changes from 9 in. (22.9 cm) at the concrete parapet to a 6-in. (15.2-cm) curb. The face of the curb is about 4 in. (10.2 cm) in front of the face of the W-beam. In comparison, the curb transition for the integral end-post design is 7 ft (2.1 m) long. Its height changes from 13 in. (33.0 cm) at the concrete parapet (the same height as the breakpoint between the lower and upper slopes of the safety shape) to a 4-in. (10.2-cm) curb. The face of the curb aligns with the face of the W-beam for the integral end-post design.

These differences in the designs of the curb transition sections are reflective of the different applications of the two transition designs. The separate end-post design is intended for retrofit of existing bridge railings where curbs are already in place. The curb height and location of the curb face reflect what is already in the field. The integral end-post design is intended for new construction and reconstruction in which there are no existing restrictions on the curb height and the location of the curb face.

The test installation for the initial separate end-post transition design is shown in Figure 5.

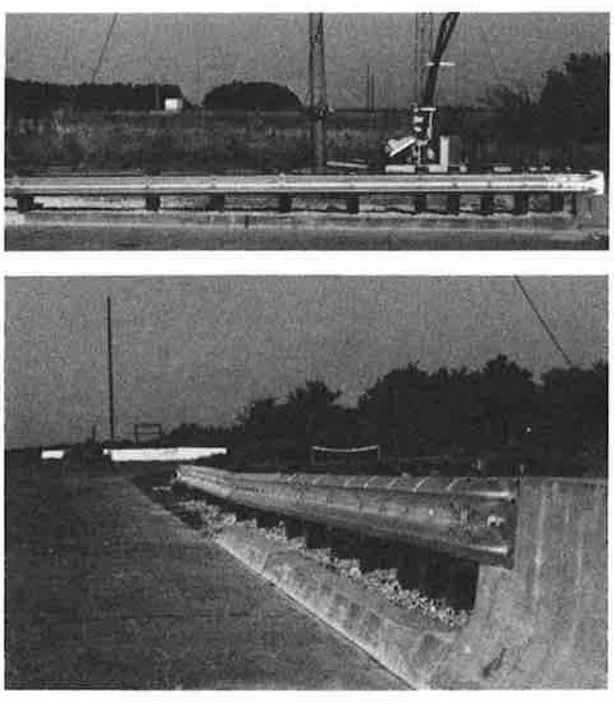

FIGURE 5 Photographs of initial separate end-post transition design test installation.

A 1981 Buick Electra impacted the midspan of Posts 7 and 8, 14 ft 8½ in. (4.5 m) from the end of the concrete end post, at 57.6 mph (92.7 km/hr) and an angle of 27.3 degrees. The test weight of the vehicle was 4,500 lb (2,041 kg). The front bumper of the vehicle slid below the W-beam shortly after impact. The left front wheel contacted Post 7, and the bumper contacted Post 6. As the vehicle proceeded along the transition, the bumper remained below the W-beam and hit the remaining posts (1 through 5) directly, splintering or breaking Posts 2 – 5. This resulted in excessive deflection to the rail, but the vehicle was not significantly redirected. The vehicle pocketed at the end of the bridge parapet, traveling at 35.1 mph (56.5 km/hr). The vehicle snagged on the end of the bridge rail and was displaced almost perpendicularly away from the end of the bridge parapet. The vehicle was practically stopped as it exited from the bridge parapet, and the exit angle was unobtainable.

The installation received severe damage, as shown in Figure 6. There was residual deformation to the rail in the area of the first eight posts. Posts 2 – 5 were splintered and broken, and the W-beam was pushed upward. There was no apparent structural damage to the concrete end post. Also, there was minimal debris or detached elements.

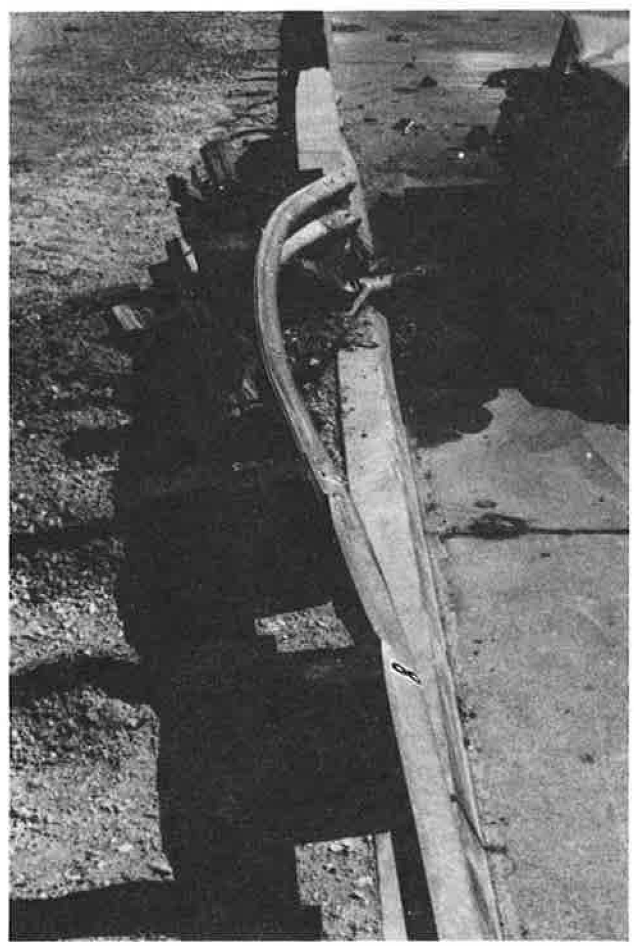

FIGURE 6 Initial separate end-post transition after initial design test.

The vehicle was severely damaged. The maximum crush was 32.0 in. (81.3 cm) at the left front corner of the vehicle. The left front wheel and control arm were severely bent and pushed rearward 22.0 in. (55.9 cm). The entire front end of the vehicle was shifted to the left 6.5 in. (16.5 cm). The subframe, roof, floor pan, hood, and bumper were among the many damaged components. In addition, the drive shaft and the steering column were bent. There was substantial deformation and intrusion into the occupant compartment. Although the occupant risk criteria are not applicable to this test, a review of the longitudinal accelerometer trace and the resulting occupant impact velocity and occupant ridedown acceleration is revealing. The deceleration levels during impact with the transition guardrail was acceptable, as reflected in a longitudinal occupant impact velocity of 29.8 ft/sec (9.1 m/sec), which was below the design value of 30.0 ft/sec (9.1 m/sec). However, as the vehicle pocketed and impacted the end of the concrete end post, high deceleration levels were experienced by the vehicle and reflected in a ridedown acceleration of $-24.1\ g$, which was above the limit of $20\ g$.

In summary, the installation contained and redirected the vehicle. The damage to the vehicle was severe, and there was deformation and intrusion into the occupant compartment. The guardrail transition sustained severe damage, but the concrete end post sustained no apparent structural damage. The amount of debris and detached elements was minimal. The vehicle pocketed and impacted the end of the concrete bridge parapet and was practically stopped as it exited from the test installation. The velocity change for the vehicle was thus the same as the impact speed of 57.6 mph (92.7 km/hr).

The poor impact performance of the separate end-post design was partially attributed to the mounting height of the guardrail. The height of the top of the W-beam at the point of impact was approximately 30 in. (76.2 cm), 3 in. (7.6 cm) higher than the standard 27 in. (68.6 cm). This resulted in the front bumper of the vehicle underriding the W-beam, thus allowing the bumper to come into direct contact with the posts. The posts were splintered or broken, resulting in excessive deflection of the W-beam. The vehicle was not significantly redirected.

Improved Design

To eliminate the potential for the vehicle to underride the W-beam, the separate end-post design was improved by incorporating a C6x8.2 rub rail into the transition design. The rub rail was mounted directly beneath and parallel to the W-beam.

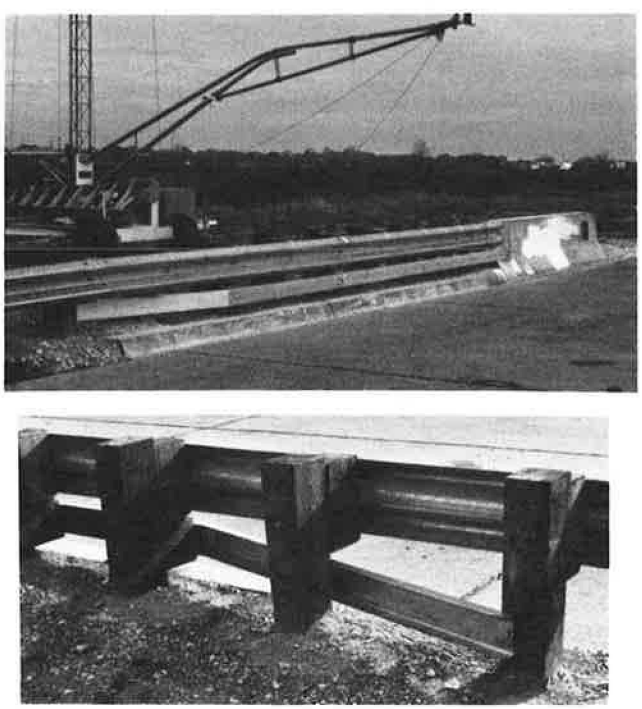

FIGURE 7 Photographs of improved separate end-post transition design test installation.

FIGURE 8 Improved separate end-post transition after improved design test.

The center of the rub rail was 11 in. (27.9 cm) below that of the W-beam. The rub rail was blocked out with 6- × 8- × 8-in. (15.2- × 20.3- × 20.3-cm) wooden blockouts so that the face of the rub rail aligned with that of the W-beam. The total length of the rub rail was 25 ft (7.62 m), spanning from the concrete end post to Post 9, after which the rub rail was bent slightly backward for termination at the back of Post 11. At its connection with the concrete end post, the lower flange of the channel rub rail was cut to accommodate the lower sloped surface of the concrete safety shape so that the face of the rub rail would remain vertical. A special end shoe was fabricated to cover the exposed end of the rub rail. Photographs of the completed installation are shown in Figure 7.

A 1980 Cadillac Coupe de Ville hit the transition midspan of Posts 7 and 8, 14 ft 8½ in. (4.5 m) from the end of the concrete end post at a speed of 62.2 mi/hr (100.1 km/hr) and an angle of 26.2 degrees. The test weight of the vehicle was 4,500 lb (2,041 kg). The vehicle was successfully redirected and did not penetrate or go over the transition system. The vehicle remained upright and stable during the impact with the transition and after exiting the test installation.

The transition system received moderate damage, as shown in Figure 8. There were tire marks all along the rub rail, and some of the bolts were damaged from contact with the vehicle wheel rims. The maximum permanent deformation of the W-beam rail element was 1.3 ft (0.4 m) between Posts 5 and 6. Maximum permanent deformation of the rub rail was 1.25 ft (0.38 m), also between Posts 5 and 6. There were no detached elements or debris.

The vehicle sustained extensive damage to the left side. There was damage to the front and rear bumpers, hood, grill, right and left front quarter panels, left front and rear doors, and left rear quarter panel. The tie-rods, left upper and lower control arms, and left front and rear rims and tires were damaged. A small strip of sheet metal was torn from the left side of the vehicle, evidently by the exposed end of the terminal connector (end shoe) lapped in the direction of impact (the end shoe had to be lapped in this manner in order for the bolt holes to fit). There was no deformation or intrusion into the occupant compartment.

The velocity change of 18.2 mi/hr (29.3 km/hr) was slightly higher than the recommended limit of 15 mi/hr (24.1 km/hr) according to NCHRP Report 230 guidelines, but the vehicle was judged to not be a hazard to adjacent traffic lanes. This higher-than-recommended velocity change could partially be attributed to the interaction between the blocked-out rub rail and the tires of the vehicle.

FINDINGS

Two Minnesota W-beam guardrail-to-concrete safety-shaped bridge-rail transition designs were crash tested and evaluated. The integral end-post design was evaluated by means of three crash tests. Results of the crash tests indicated that the integral end-post design met all impact performance evaluation criteria according to NCHRP Report 230 guidelines.

Two versions of the separate end-post transition design were evaluated, each with one crash test. Results of the crash tests indicated that the initial separate end-post design did not meet the impact performance evaluation criteria outlined in NCHRP Report 230. The vehicle pocketed and impacted the end of the concrete bridge end post, resulting in substantial deformation and intrusion into the passenger compartment and an unacceptable level of longitudinal occupant ride-down acceleration. The vehicle was practically stopped by the impact with the concrete bridge end post and was not significantly redirected. The design was then modified by MnDOT, and the improved separate end-post transition design was crash tested. The improved design successfully met the impact performance evaluation criteria outlined in NCHRP Report 230.

These two transition designs have since been approved by FHWA and adopted as standard designs by MnDOT for field implementation. As mentioned previously, the integral end-post transition design is used with new construction and reconstruction in which existing bridge railings are to be replaced. The separate end-post transition design is used for retrofit of existing bridge railings in 3R and 4R projects where speed limits are greater than 40 mph and the traffic volume is greater than 1,500 ADT.

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Site-Specific Issues: Application or Misapplication of Highway Safety Appurtenances

JAMES D. CROWLEY AND OWEN S. DENMAN

The misapplication of safety appurtenances sometimes occurs because designers and specifiers do not have a thorough understanding of appurtenance characteristics or how they should be applied to hazards. The primary reasons include the following. First, highway appurtenance application and use guidelines are limited. Second, the background knowledge and experience of designers and specifiers is declining as a result of turnover and retirement. Third, appurtenance characteristics are not always published in a consistent and useful form. Fourth, the number of alternative appurtenances has increased. Fifth, the existing appurtenance approval procedures group devices into broad categories that do not adequately describe the intended use or limitations of any specific system. Providing better information and training to designers and specifiers can result in better application of safety appurtenances. Designers and specifiers should understand critical site considerations and system characteristics and have access to guidelines for selecting the appropriate system characteristics to address each specific site. Researchers, industry associations, manufacturers, FHWA, AASHTO, and others should work together to improve the information provided to designers and specifiers. A document could be published in which the testing, critical site considerations, system characterization, and application information are combined. Designers and specifiers need this information to make proper applications of safety appurtenances. This paper is focused on why safety appurtenances are misapplied, and steps to improve the overall situation are recommended. Detailed descriptions of key site considerations, system characteristics, and system testing and evaluation requirements are discussed.

The safety of the motoring public on the nation's highway system has significantly improved during the past 2 decades. The identification and removal of hazards and the development and application of safety appurtenances have been a major part of this improvement (*1*). However, even today, many serious injuries and fatalities could be avoided through continued focus on safety and the proper application of existing technology. The purpose of this paper is to address an element of the improvements that can be made to upgrade the level of safety for the motoring public.

Professionals in this industry can easily identify the problem by driving down the roadway and seeing a good safety appurtenance incorrectly applied (*2*). The appurtenance may be applied in such a way that it cannot work properly, or another system could have been applied that would have been less costly and would perform more acceptably for that specific site. In that case, an acceptable safety appurtenance, a valid system, has been misapplied.

The primary focus of this paper is on why safety appurtenances are not applied properly to hazardous sites and to suggest what can be done to improve the situation.

PROBLEM DEFINITION

The reasons that safety appurtenances are applied improperly must be identified before solutions can be formulated. It is suggested that the primary reason is that designers and specifiers do not have a thorough understanding of appurtenance characteristics or how they should be applied to hazards. Designers and specifiers work for departments of transportation, consultants, and contractors, and they should know what is being specified. The primary reasons for misapplication include the following.

First, guidelines for the proper application and use of highway appurtenances are significantly lacking.

Second, the background knowledge and experience of designers and specifiers is declining as a result of turnover and retirement. Many people who were involved in the field of highway safety in the 1950s, who started developing safety programs and hardware in the 1960s, and who started implementing these systems in the 1970s and 1980s are retiring. Their successors may know what products are available, but they do not always know the characteristics of the products and why one is preferred over the alternatives. Significant differences in products are often not well documented or understood.

Third, appurtenance characteristics are not always publicized in a consistent and useful form. It is confusing for designers and specifiers to try to evaluate and compare alternatives when the available information is not consistent. The people who design and specify highway appurtenances receive most of their information from product developers, manufactures, and promoters, and the product literature from various manufacturers is not consistent in characteristic descriptions.

Fourth, the number of alternative appurtenances has increased. In the past, a designer had only one or two solutions to choose from. New solutions have been developed over time to address specific sites. Each new solution has brought a new set of characteristics best suited to a particular application. Experience has shown that a single, generic solution is no longer the best choice for all sites.

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Fifth, under existing appurtenance approval procedures, devices are grouped into broad categories that do not adequately describe the intended use or limitations of any specific system. Approvals are granted in such general categories as longitudinal barriers, crash cushions, end terminals, transitions, and truck-mounted attenuators. These categories may appear specific because they have differentiated between longitudinal barriers and crash cushions and between end terminals and crash cushions. The problem is that systems within the categories may be quite different. There is a significant difference between a cable guardrail and a concrete safety-shaped barrier in both performance and situations in which each should be used, but both are longitudinal barriers. Many types of longitudinal barriers have widely varying characteristics. Crash cushions are another example of specific devices that have different performance characteristics that affect application parameters. However, not enough descriptive information is included in the approval process. Approvals are granted in broad categories, and thus one cannot expect designers and specifiers to understand where and how specific devices should be used.

Regulating bodies are not filling the gap by supplying information that is technically descriptive enough to guide the designer or specifier. As an example, National Cooperative Highway Research Program Report 230 (*3*) describes how to test a crash cushion and how to evaluate its performance. Not explained, however, is where crash cushions should be applied. Guides published by AASHTO (*4,5*) describe the G-R-E-A-T system, Hex-Foam Sandwich System, Energite barriers, and other safety hardware as acceptable appurtenances, but do not provide sufficient information on how they should be used, the systems' strengths and limitations, or where they should be applied.

State specifications are another source of information, but they sometimes only identify the location (the mile marker) where a crash cushion is required. Specifiers refer to the AASHTO guides (*4,5*) or the state's approved list of highway hardware to select a product. The lists provide only limited system information or applications guidance, and thus the specifier makes a selection based on inadequate parameters.

GENERAL SOLUTION

The problems can be substantially resolved by providing better information and training for designers and specifiers (*6*). Critical site considerations that should be addressed by designers and specifiers to ensure that the hazard and the site conditions are properly identified should be developed.

In addition, approved systems could be described by characteristics that relate to the critical site considerations. An appurtenance that best meets the required site characteristics could then be applied. Specific recommendations include the following.

First, designers and specifiers should be trained to evaluate the hazard and describe it in terms that ensure that the hazard condition can be properly identified.

Second, testing, evaluating, and reporting procedures should be developed that describe safety appurtenances' performance characteristics and limitations relative to a comprehensive listing of critical site considerations. Broad categories (e.g., crash cushions) should be replaced with more specific classifications (e.g., redirective, bidirectional, nongating crash cushions with limited lateral deflection). This change would not require a whole series of new tests but rather a more thorough evaluation and description of existing tests. Information known from the testing being done on safety appurtenances should be made available to the people who design and specify their use.

Third, guidelines or recommendations matching each type of safety appurtenance (by specific characteristic category) to specific site conditions should be developed and published (*6*) to help designers and specifiers develop a better level of understanding of how appurtenance technology can be applied to the roadside to reduce injuries and fatalities.

The solution to the problem statement presented here is by no means the only way to solve the problem. Refinements would improve the suggested solution, and entirely different methods could be used to solve the problem. The solutions presented here are only one way to improve the siting and selection process for highway safety appurtenances.

RECOMMENDED APPROACH

The recommended approach is to develop a method to better ensure that highway safety appurtenances are properly applied to hazardous sites to improve motorist safety. The first step should always be to remove the hazard. However, if the hazard cannot be removed, it should be treated with the proper safety appurtenance (*4,5*).

The treatment of highway hazards should start with a description of the hazard and the key site considerations. Once this description is complete, safety appurtenances can be reviewed and the appropriate characteristics matched to the needs of the site (*6*). The recommendations must also include information on how to obtain the safety appurtenance characteristics. These issues are addressed in the following sections.

Key Site Considerations

The description of the hazard and key site considerations must be addressed by the designer or specifier. The following list is a starting point and is not all-inclusive. A group of experts in highway safety can add to and modify the list to ensure that the hazardous site characteristics are properly defined to facilitate the proper appurtenance selection.

The items on the list are not in order of relative importance. Although there may be general agreement that some of the factors consistently demand a higher value, the relative weight of other factors is specific to particular hazardous sites.

- *Available longitudinal space*. The available longitudinal space should be described. Longitudinal space is frequently limited when other site considerations must be addressed. Examples of these situations include bifurcations (gore areas) in which encroachment into the driver decision area cannot be allowed, longitudinal barrier ends near a crossover or turnaround area, and wherever geometric characteristics of the site limit the available longitudinal space. The longitudinal space requirements of safety appurtenances vary significantly, so site constraints must be understood.

• *Hazard width and height.* The width and height of the hazard or the object that is being shielded should be identified. This information is valuable during consideration of attachments and transition options.

• *Available lateral space.* Restriction of available lateral space is a common site characteristic. Sharp drop-offs, drainage features, or other hazards may be located close to a hazard. If a solution is chosen that allows a vehicle trajectory behind the system, one problem has been solved but another created. There could be an object directly behind the guardrail that does not match the longitudinal barrier's deflection characteristics (e.g., a cable rail that may deflect up to 12 ft and a hazard that is only 2 ft behind the rail).

• *Hazard site width.* The overall site width should be described and considered. The available site width restricts potential solutions in the same way that limited longitudinal space does. The treatment options for a 30-ft-wide median are different from those for a 6-ft-wide median with a concrete median barrier (CMB) in the center.

• *Hazard proximity to traffic.* The distances from the lane line to the hazard and to opposing traffic lanes are additional restrictions on available lateral space that should be known for proper evaluation of a site.

• *Available maintenance space.* The available maintenance space affects overall site safety and system maintenance costs. If the available space is restricted, maintenance personnel will be subjected to personal risk, costly traffic controls will have to be applied to maintain the system, and traffic congestion will become an issue. Since there are significant differences between appurtenance maintenance characteristics, the available maintenance space should be described.

• *Surface conditions and anchoring options.* The surface conditions in the area around the hazard should be described such that the various anchoring options for appurtenances can be properly applied. The soil characteristics, type of subbase, thickness and strength of portland cement concrete or asphaltic concrete, and cross-slopes should be included in the description. The presence and location of drainage features, expansion joints, and other surface features should also be described.

• *Anticipated impact speed.* The anticipated impact speed should be defined to ensure that the appurtenance selected has sufficient capacity. Guidelines (*4,5*) and other information is available (*7*) to help designers and specifiers estimate the anticipated impact speed for a specific site. This estimate can be improved by consulting local traffic engineering professionals.

• *Average traffic volume.* The average traffic volume at a site, along with the various site geometry factors, has a significant influence on impact frequency and maintenance requirements. This factor should be described and taken into account during appurtenance selection.

• *Impact frequency.* The impact frequency is not always known for new installations, but a prediction can be made based on similar sites. Existing sites that are being renovated should have an accident history. Whatever information is available should be described and used during selection of the appurtenance.

• *Unidirectional or bidirectional traffic.* The direction of traffic in the vicinity of the hazard should be described. If the site has unidirectional or bidirectional traffic in the vicinity of the hazard, the appurtenance characteristics should match the site conditions. This factor, along with other site geometry issues, will help determine such key system requirements as redirection and gating characteristics.

The key considerations for the site can now be defined in terms that pertain to a specific hazard that is to be protected. The next step is to describe safety appurtenance system characteristics such that the proper system can be selected for that hazard.

Definitions of System Characteristics

The system characteristics for each approved safety appurtenance should be described in terms that give the designer or specifier enough information to determine if the appurtenance is applicable to a specific site. Those system characteristics are discussed in this section (*5,6*).

Redirection Capability

The basic definition of a redirective system is a system that, when impacted along the side at an angle, will redirect the impacting vehicle away from a fixed object. If the vehicle hits a nonredirective system at an angle, it will continue in nearly the same direction until it interacts with another highway fixture or vehicle, or stops (Figure 1).

The difference between redirective and nonredirective systems is not subtle. As an example, some crash cushions are redirective for a portion of the system and nonredirective for a portion of the system (CIAS) (*8*). Other systems are clearly either redirective (G-R-E-A-T, BRAKEMASTER, HFSS, etc.) (*9–11*) or nonredirective (inertial barriers) (*12*). Some systems are redirective in both directions (bidirectional), and some can only redirect in one direction (unidirectional).

The site conditions will determine whether a system with redirective or nonredirective characteristics in a uni- or bidirectional mode is required. Once that decision is made, a selection can be made from the systems possessing that characteristic.

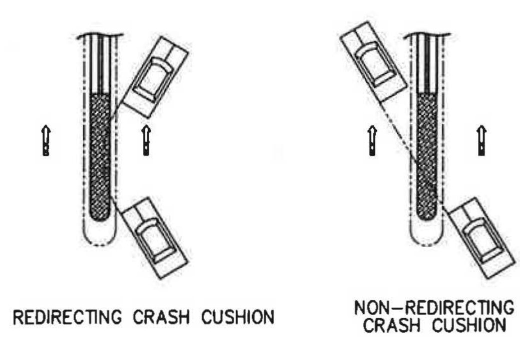

FIGURE 1 Redirective and nonredirective systems.

Capacity

Capacity is the ability of the appurtenance to absorb the kinetic energy of the impacting vehicle in a safe and controlled manner. The anticipated impact speed, weight of vehicles, and impact angle are the key variables that must be considered (*5*). If a safety appurtenance that has a capacity of 45 mph is used for a roadway with a anticipated impact speed of 65 mph, the appurtenance is mismatched to the hazard; it does not have enough capacity. Thus, the designer or specifier must know the anticipated impact speed. Unfortunately, this simple characteristic is one of the least understood and most abused by some appurtenance suppliers. Frequently, systems with a design capacity of 60 mph or less are promoted as having the capacity for hazards where anticipated impact speeds are well over 60 mph. What may appear to be only a slight mismatch (e.g., 5 to 10 mph) can result in a serious injury or fatality. If a 4,500 lb vehicle impacts the appurtenance at 65 mph or more, it will bottom out with serious consequences. In order to ensure that the capacity of the system being considered matches the capacity needed at the site, test results should prove that the appurtenance has the required design capacity.

Gating

The gating characteristic is another term that is not well understood by most designers and specifiers. The basic definition is a system that, when impacted at an angle on the front (nose), allows the vehicle to pass through in the same general direction of travel (Figure 2). The system opens like a gate. If the impacting vehicle is brought to a controlled stop by the safety appurtenance, the system is nongating.

The gating issue is somewhat controversial. The controversy centers around applications of an end terminal or crash cushion that is attached to the end of a longitudinal barrier. The decision to allow gating is based on whether the proper length-of-need (LON) has been used for the longitudinal barrier. If the proper LON has been established (*5*), the end treatment could allow gating without hazardous consequences. However, frequently, the LON has not been established, and thus the gating issue is important. Safety appurtenances such as end terminals and crash cushions are frequently applied to longitudinal barriers where the LON has not been properly established. An example of this situation occurs when there are emergency access breaks in a section of CMB. The ends of the CMB can be protected with a crash cushion, but the system should not gate.

Crash cushions are also frequently applied to other hazards where there is no longitudinal barrier. If the site characteristics show that an impacting vehicle would be subjected to a higher level of risk or that there is a high probability of either a secondary impact or encroachment into opposing traffic lanes if the appurtenance gated, a nongating appurtenance should be used.

The approved safety appurtenances should be grouped into gating and nongating devices. This descriptor should be decided for each system such that the designer or specifier understands the characteristics of the systems being considered. This will help ensure that the proper appurtenance is selected.

Pocketing

The pocketing characteristic occurs when the lateral stiffness of two redirective devices is so different that the impacting vehicle "pockets" into the softer barrier and is brought to an abrupt stop or redirects at an angle that is too high when the stiffer barrier is contacted.

The lateral stiffness and deflection characteristics of the safety appurtenance should be described to ensure that the designer or specifier can match the appurtenance to the adjoining barrier with the proper type of transitioning device to reduce the potential of pocketing.

Intrusion of Vehicle into Traffic Lane

The trajectory and final stopping position of an impacting vehicle subsequent to an impact with the appurtenance is critical. This intrusion characteristic is not well described for existing appurtenances and leads to the application of systems that result in high probabilities of secondary impacts, intrusion into adjacent traffic lanes, or both.

The intrusion characteristics of systems vary significantly. Under a specific set of impact conditions, some systems bring the vehicle to a controlled stop within the confines of the appurtenance. Other systems allow the vehicle to exit the appurtenance and cross several lanes of traffic (Figure 3), roll back in an opposing direction, or exit at a steep angle into adjacent traffic. The designer or specifier must understand the large variation in the characteristics of existing systems to better apply the proper system to a specific hazard.

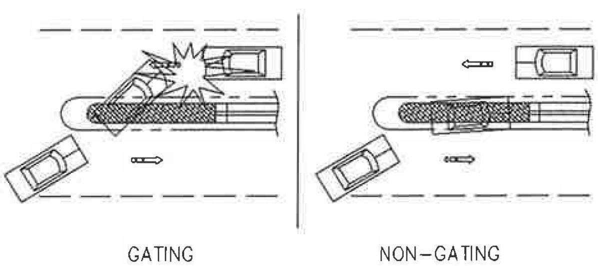

FIGURE 2 Gating and nongating systems.

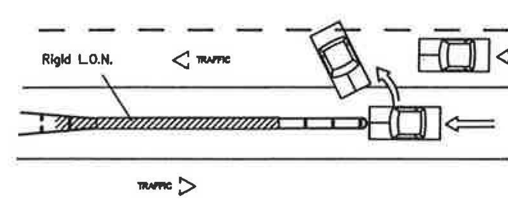

FIGURE 3 Intrusion of vehicle into traffic lane.

Intrusion in the Median or Roadside

The lateral deflection characteristics of safety appurtenances vary significantly and are not readily available. Whether the appurtenance is a longitudinal barrier, an end terminal, or a crash cushion, the lateral deflection characteristics must be known by the designer or specifier so that the appurtenance with proper deflection is applied. Otherwise, a system may be applied that can deflect into opposing lanes of traffic or into the hazard.

Other System Characteristics

Several other system characteristics could be published that would aid the designer or specifier in selecting the best appurtenance for a specific site. These characteristics include the following:

- System width and width options,
- System lengths for specific capacities,
- System anchoring requirements and options,
- System maintenance requirements,
- Level of reusability of components from design impacts,
- Refurbishment requirements from design impacts, and
- Environmental considerations.

These characteristic descriptions should be controlled and published during the approval process. Changes that occur during the product life that are critical to performance should also be published. The designers and specifiers must have accurate information to be able to select the proper system.

Recommendations for Testing, Evaluating, and Reporting System Characteristics

The system characteristics that have been described are currently not available or published in a form that is valuable or useful to designers and specifiers. This information should be collected, evaluated, approved, and published in a form that provides designers and specifiers formal guidelines to apply the site-specific criteria.

The system characteristics should be developed in the testing and evaluation phase of a new product. The development of these characteristics will not require additional testing as compared with that proposed in the NCHRP 230 Rewrite Document (*13*). However, the evaluation and documentation requirements for the tests that are conducted should be modified to record some new parameters.

The new parameters that need to be recorded are those that focus on system characteristics that will provide the designer or specifier the information needed to apply the appurtenance to a hazardous site (*6*). These proposed parameters are discussed next.

Redirective Characteristics

The redirective characteristics of the appurtenance should be reported in a form that is useful to the designer or specifier. The standard characteristics such as redirective capacity (e.g., weight of impacting vehicle, speed, center of mass height, entrance angle, exit angle, and speed change during the impact) should be recorded. In addition, the amount of deflection of the appurtenance and specific siting issues particular to the appurtenance should be reported. A summary of these characteristics can be condensed into a form that is useful to the designer or specifier. Appurtenances such as crash cushions and end terminals can be easily categorized as redirective or nonredirective. Within these groups, specific characteristics of an appurtenance can be listed in tabular form.

System Capacity and Length Options

System capacity of appurtenances is being reported in a generally acceptable form to be useful to designers and specifiers. Testing documentation and the approval process ensure that an appurtenance has the capability to absorb or redirect at least some specified level of kinetic energy (e.g., that of a 4,500-lb vehicle at 60 mph and 25 degrees). What seems to be missing is a statement that clearly defines the design limits for the particular appurtenance being evaluated.

The appurtenance design limits relative to capacity should be described in a clear and consistent format (*14,15*). An appurtenance may be tested and pass criteria for a 60-mph impact, but its limiting capacity may allow it to be used for impacts up to 65 mph. On the other hand, the appurtenance may have a design limit of no more than 60 mph. With posted speeds for the highways changing from 55 to 65 mph or 70 mph, this can be an important characteristic to be reported.

The appurtenance may be available in various lengths and widths that have specific capacities. These capacities should be evaluated and described in a clear and consistent form. The appurtenance capacity may be based on specific attachment or anchoring options. Again, this effect on the system's capacity should be clearly and consistently stated. The reporting of these capacity characteristics will provide the designer or specifier with clear and concise information that is critical to proper siting of the appurtenance. Otherwise, the designer or specifier can only rely on the appurtenance approval letter, a test report (if available), or sales literature to estimate the capacity of the device.

Gating Characteristics

The gating characteristic is primarily applicable to such appurtenances as crash cushions and end terminals. Again, the proposed testing (*5*) is totally adequate to evaluate these systems for gating. All that is needed is to evaluate and report the characteristics in a form useful to designers and specifiers.

The evaluation for gating should be done on all frontal impacts into crash cushions and end terminals. These impacts include the zero degree impacts with and without the offset of vehicle and appurtenance centerlines as well as the angled impact on the nose of the device. Although the angled nose impact is the most pertinent of the frontal tests for evaluating gating, some appurtenance designs gate during all frontal impacts.

The reporting should categorize the appurtenance being evaluated as gating or nongating. If the appurtenance gates the impacting vehicle (i.e., the vehicle is not brought to a controlled stop within the confines of the appurtenance) from any of the frontal impacts, the system should be classified as gating. If the system is classified as gating, the additional characteristics of vehicle speed change, post impact trajectory, and siting limitations in addition to the standard occupant risk factors should be reported. If the system is classified as nongating, only the occupant risk factors need to be reported.

Pocketing (Transition) Characteristics

The appurtenances that are evaluated for redirective characteristics also need to be evaluated for pocketing. Pocketing can occur from redirective impacts and is most likely to occur when there is a change in the lateral stiffness of the systems is involved. Thus, areas within the length of crash cushions, end terminals, or longitudinal barriers or those where one system transitions to another should be evaluated for potential pocketing.

The pocketing characteristics or potential could be described for the appurtenance being evaluated and for that appurtenance attached or transitioned to other appurtenances for which approval is being sought. The options and characteristics for each option could be summarized in tabular form for reference by designers and specifiers.

Intrusion Characteristics

The intrusion characteristics need to be reported for all appurtenances tested. These characteristics include the intrusion of the impacting vehicle into adjacent and opposing traffic lanes and the intrusion of the vehicle into the confines of the appurtenance to evaluate the potential of the vehicle interacting with a hazard.

The intrusion of the impacting vehicle into adjacent traffic lanes is partially covered in existing evaluating procedures. However, there are no specific criteria that define acceptable or unacceptable intrusion, and the reporting of this element is not provided to the designer or specifier in a form that is informative or useful. The designer or specifier needs to know where, how, and how much the vehicle intrudes into both adjacent and opposing traffic lanes to be able to understand this characteristic. The appurtenance can then be applied to minimize the hazard presented by the impacting vehicles intrusion into all potential lanes of traffic.

The intrusion of the vehicle into the confines of the appurtenance being evaluated is currently not well documented. The designer or specifier needs to understand this characteristic to ensure that the appurtenance is located properly such that the impacting vehicle will not impact a hazard in the vicinity of the appurtenance. This will help avoid the situation where an appurtenance with excessive lateral deflection is used and allows an impacting vehicle to deflect the barrier and contact a fixed hazard.

The evaluation and reporting could address the specific appurtenance being considered as well as specified or anticipated attachment and transitioning options.

System Width Options

The current evaluation and reporting procedures would allow an appurtenance such as a crash cushion to be tested at one length and width and receive approval for the entire product line. The product line could be composed of several length and width combinations (*8,14,15*). The designer or specifier is unable to assess whether lengths or widths other than the one evaluated were considered in the approval process.

The testing that is currently being done can be evaluated, and the expected performance of other lengths or widths of systems can be reported. The conclusions reported could be in a clear and consistent format that gives the designer or specifier proper guidance.

System Anchoring Requirements and Options

The current testing guidelines allow testing in either strong (S-1) or weak (S-2) soils (*3*). This does not address anchoring on concrete or asphalt, or on other types of soil conditions where the appurtenance may be applied. Some appurtenances require specific anchoring conditions to function properly. These appurtenances may require strong reinforced concrete footings or foundations, strong soils with custom-designed driven anchors, or other conditions that must be described to the designers and specifiers.

The appurtenance being tested could be evaluated with respect to the function and operation of anchoring devices for which approval is being considered. Enough design information or testing data could be supplied in the approval process such that the specific requirements and acceptable options can be described for the designer or specifier. If limitations on the other system performance characteristics can be seen if the proper anchoring conditions cannot be met, these limitations could also be clearly reported.

System Maintenance Requirements

The normal (pre-impact) maintenance requirements for an appurtenance could be clearly defined in the approval process and reevaluated at the end of the in-service evaluation period. Some appurtenances can be installed and remain in an acceptable performance condition for more than 10 years with minimal or no maintenance. Others may require periodic maintenance, which could be significant, to ensure that the system is crashworthy.

The components and function of all appurtenances being evaluated could be analyzed for this characteristic during the approval process, and specific recommendations could be reported. This information could be updated as product experience is gained through the in-service evaluation period. Otherwise, the designer or specifier may apply an appurtenance that will not function properly after a few years, and motorists will not be protected.

Reusability of System Components and Refurbishment Requirements

The evaluation of an appurtenance could include an assessment of the reusability of system components and the refurbishment requirements after design impacts. This information can be developed during the testing phase and could be presented in a form that will give needed guidance to designers, specifiers, and state maintenance personnel.

The components of an appurtenance that has been impacted can either be replaced, reworked, or used again, depending on the type of system being tested, the type of impact, and the type of specific damage observed on key components. The current lack of guidance results in improper maintenance of systems and thus undue exposure to risk for motorists.

The documentation of these key system characteristics will allow designers and specifiers to select an appurtenance that better meets the siting requirements and the abilities of state maintenance operations.

Environmental Characteristics

The environmental characteristics that can affect the performance of an appurtenance are not currently considered in the evaluation process. The materials that are used in an appurtenance and the specific system design can be affected by severe environmental conditions. This information is currently not being supplied to designers and specifiers.

The effect of environmental conditions on the performance of an appurtenance could be analyzed, evaluated, and reported in a form that provides guidance to designers and specifiers. These effects include high and low temperatures (−20°F to +120°F), moisture, ice on system components, snow or soil build-up around the system, exposure to ozone and ultraviolet radiation, corrosion, vibration, and other factors that could affect or impede the operation of the system. The effect of not including these items in the evaluation process will result in truck-mounted attenuators that fall off the shadow trucks, inertial barriers that freeze into solid blocks of moisture-laden sand (*15*), slip bases that loosen or corrode and do not function, energy-absorbing materials used in hot or cold areas that do not function properly, and a multitude of other problems.

The inclusion of these characteristics in the evaluation and reporting process will again provide needed guidance to designers and specifiers.

Guidelines for Matching System Characteristics to Applications

The guidelines to direct a designer or specifier to a specific type of system characteristic to address specific site considerations are inadequate or nonexistent. Guidelines that show the designer or specifier when redirection, gating, and the like are allowable should be developed. The specific site characteristics that influence these decisions are fairly well known and can be documented to form a set of guidelines (*6,16*).

The guidelines can be defined in terms such as distance from the edge of lane, distance to opposing traffic, proximity of the hazard to traffic (either direction), divergence angle of the lane near the hazard, and others. Recommendations can be made as to what critical system characteristics should be used for specific (critical) site considerations. The designer or specifier can then compare the hazardous site considerations to the guidelines and then to the approved system characteristics to best match the appurtenance to the hazardous site. The qualified systems could then be analyzed for cost-benefit considerations as described in the ROADSIDE software program (*17*).

TABLE 1 Considerations for Safety Appurtenance Selection and Application

CRASH CUSHION – END TERMINAL	
REDIRECTIVE	
UNIDIRECTIONAL	
HAZARD PROXIMITY TO TRAFFIC	< 10'
DISTANCE TO OPPOSING TRAFFIC	> 30'
TRAFFIC DIVERGENCE ANGLE	> 5°
BIDIRECTIONAL	
HAZARD PROXIMITY TO TRAFFIC	< 10'
DISTANCE TO OPPOSING TRAFFIC	< 30'
TRAFFIC DIVERGENCE ANGLE	> 5°
NON-REDIRECTIVE	
HAZARD PROXIMITY TO TRAFFIC	> 10'
DISTANCE TO OPPOSING TRAFFIC	> 30'
TRAFFIC DIVERGENCE ANGLE	< 5°

The Certified Lifesaver Program (*6*) and the SNAP software package (*16*) have been developed to assist the training needs of designers and specifiers. These packages focus on the generic aspects of both site characteristics and system characteristics.

An example relative to redirection is shown in Table 1. In this example, a hazard that is relatively close to adjacent traffic (e.g., less than 10 ft) with a significant divergence angle of the lane (e.g., greater than 5 degrees) should require an appurtenance that is redirective. Further, if the distance to opposing traffic is relatively close (e.g., less than 30 ft), the appurtenance should have redirective capacity from both directions or be bidirectionally redirective. With the vast knowledge and experience of professionals in the area of highway safety, other guidelines can be established.

CONCLUSION

A problem was identified and a solution proposed in this paper. The key site considerations have always existed. Designers and specifiers should understand the site considerations when selecting a treatment. The information needed to categorize appurtenances by characteristics related to the key site considerations is available from existing test procedures. Designers and specifiers need to make better use of the available information by evaluating and classifying systems in terms that will allow a proper match to a specific application.

A lot has been learned about highway safety during the last 25 years. A methodology was presented here to better apply this knowledge. It is an opportunity to further improve highway safety today without more research dollars, new technology, or new product development. All that is required is to use what has been learned to make better applications of existing hardware. It is the responsibility of highway professionals to continue improving safety. To begin this effort, a training program (*6*) and a site-specific software program (*16*) have been developed to provide better guidance to designers and specifiers. This is only a start; input from highway safety professionals can further improve the process.

Finally, implementation of these solutions requires a group effort. Researchers, industry associations, manufacturers, FHWA, AASHTO, and others must work together. No one group covers everything from product testing and evaluation to application analysis and specifications. Ideally, a document could be published in which are combined the testing, product characterization, site considerations, and application information. At a minimum, more information should be included in existing documents such as NCHRP Report 230, the updated version of NCHRP Report 230, the Barrier Guide, the Manual on Uniform Traffic Control Devices, and so on. Designers and specifiers are looking for help. This information is needed to better apply existing technology, save lives, and reduce injuries on the nation's highways.

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ADIEM: Low-Cost Terminal for Concrete Barriers

DON L. IVEY AND MARK A. MAREK

Presented here are the results of a research and compliance testing program to develop a low-cost, high-performance terminal for portable concrete barriers (PCBs) and permanent concrete median barriers (CMBs). The result is the Advanced Dynamic Impact Extension Module (ADIEM). The energy absorption elements of this terminal are lightly reinforced, ultra-low-strength Perlite concrete modules. The redirection element of this terminal is a heavily reinforced conventional concrete variable height curb with automobile hub-height pipe rail. ADIEM meets the requirements in *NCHRP Report 230* at a cost of approximately $100/ft. This translates into a projected cost for a 60-mph class, 30-ft barrier of $3,000. This appears to represent a major cost reduction for high-performance PCB and CMB terminals. ADIEM is also expected to find wide application in protecting vehicle occupants from such other roadside obstacles as utility poles.

In the field of roadside safety, transportation entities have always been handicapped by severe limitations in the amount of public funds available for improvements. Although the public demand for mobility has always been strong, the demand for greater levels of safety has been both limited and sporadic. This is the underlying reason for normally severe funding limitations for roadside safety improvements. Because of these economic constraints, the achievement of cost-effectiveness has been and continues to be of critical importance.

The ends of concrete median barriers (CMBs) and portable concrete barriers (PCBs) are a troublesome safety problem. Some solutions, such as the sloping concrete wedge, have been low cost, but effectiveness in reducing injuries is questionable. Sand-filled barrels and the steel barrel cushions are fairly low cost, but maintenance is difficult. Further, they require a wide median or roadside, which is often not available, especially in constrained construction areas, and they do not have side redirection characteristics. The excessive width of these two cushions greatly increases the target size of the protective device, resulting in more collisions than would result from a narrow cushion. Finally, there are narrow cushions for end treatments in narrow zones that perform well in collisions. These cushions, however, are costly. The motivator for this work is the fact that no low-cost, high-performance, easily maintained end treatments for CMBs and PCBs existed. The development and final performance verification of such a terminal are described here. Figure 1 and Table 1 show the final results of this development.

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CHRONOLOGICAL DEVELOPMENT

The concept of using low-strength, lightweight concrete in the end treatment of PCBs and CMBs emerged in 1986. The Texas Transportation Institute (TTI), in an internal program in 1986, developed a design called Advanced Dynamic Impact Extension Module (ADIEM), a low-strength concrete terminal. In 1987, TTI staff approached engineers of the Texas State Department of Highways and Public Transportation (SDHPT) [now the Texas Department of Transportation (TxDOT)] with the ADIEM design and asked SDHPT to consider it for further development. Development under SDHPT sponsorship was carried out in three phases.

In Phase 1, the original design was modified significantly to improve installation and maintenance characteristics. Material strength testing was conducted, and individual modules of reinforced Perlite were tested at low speed using a 5,000-lb ram. From these tests a module was selected for vehicle crash testing. The complete ADIEM consists of a structural concrete carrier base and a number of low-strength concrete modules. The carrier base was tapered and attached to a conventional PCB by a standard lapped channel beam connection. Into the carrier base were keyed low-strength concrete modules. Each module was 3 ft long, 2 ft tall, and 11.5 in. wide. Each module weighed about 200 lb. At the completion of Phase 1, SDHPT engineers decided that the potential of the prototype was such that full-scale crash testing was warranted.

In Phase 2, five crash tests were conducted. These tests are summarized in Table 2 and are presented in detail in an interim report (*1*, Vol. 2). In this phase, results of the redirec-

FIGURE 1 ADIEM terminal for CMBs, PCBs, and toll-road collection zones. (First ADIEM installed by Ohio Turnpike Authority in Cleveland, December 19, 1991.)

TABLE 1 Results of Development of PCC Terminal for CMBs and PCBs

PROJECTED* COST	$3000.00
INSTALLATION TIME	< 1 hour
EFFECTIVE DIMENSIONS	Length - 30 ft. Width - 2 ft.
AFTER A MAJOR COLLISION	
• Cost of replacement modules	$1500.00
• Time to clear crushed modules	< 20 min.
• Time to install new modules	< 20 min.
NCHRP 230 COMPLIANCE	Exceeds requirements of this guide by significant margins. (See Table 3.)

* Includes 50% profit for the manufacturer. This does not include a profit estimate for the contractor.

TABLE 2 Summary of Developmental Crash Tests

Test Type	Test No.	NCHRP* 230 No.	Test Date	Results	Comments:
Developmental (4500 lb./43.1 mph head on)	1	NA	03/03/89	Poor	Excessive deceleration, poor module failure pattern, vehicle ramped and rolled over. Redesign of modules was necessary.
Compliance (1800 lb./15°, mid-side)	2	44	03/03/89	Excellent	Passed 230-Vehicle was appropriately redirected. All aspects of 230 were met. Barrier performance was ideal. No maintenance would have been necessary. Barrier totally undamaged.
Developmental (4500 lb., 37.1 mph, head on)	3	NA	05/25/89	Good	Vehicle was smoothly decelerated. Deceleration rates were very low indicating module crushing strength was ideal. Vehicle damage was slight. All modules would need to be replaced.
Developmental (1800 lb./58.4 mph, head on, 15 inches off center)	4	45	08/01/89	Marginal	Did not pass 230. Deceleration rates were too high. Vehicle stability was good, but damage severe. Concrete strength determined to be 60% too high. Some failure in module reinforcement noted. Small change in module reinforcement was necessary.
Developmental (4500 lb., 57.6 mph, head on)	5	41	09/28/89	Good	Passed 230. Deceleration rates excellent. All aspects of 230 were met. Vehicle damage reasonable. Some modules did not clear as preferred resulting in modest vehicle ramping at end of interaction with barrier and after speed had been reduced to below 20 mph. Modest changes in module reinforcement should improve interaction.

tion test (NCHRP 230 No. 44—1,800 lb, 60 mph, and 15 degrees) were excellent, whereas the results of head-on tests were not ideal. Overall, the results were encouraging, and the final compliance test phase was initiated.

NCHRP Report 230 (*2*) presents crash tests appropriate for barrier end treatments (or barrier terminals) in high-speed areas. The three applicable tests are 41, 44, and 45 under "terminal" tests. Tests 42 and 43 are not needed because Tests 44 and 45, in which a smaller automobile is used, are more critical in terms of vehicle stability and acceleration. Test 40 is not needed since the ADIEM terminal joins a conventional PCB at the beginning of length of need. The 14 36- × 1⅛-in. steel dowels or 1⅛-in. bolts that secure the ADIEM carrier beam to the ground, asphaltic concrete pavement, or portland cement concrete, along with the formidable structural connection from ADIEM to a PCB, ensure that the impacted end of the PCB is laterally and longitudinally stable. Thus, conducting this test would simply be testing a well-secured PCB, which has been done many times. Note also the standard Texas connection of lapped channel sections had been tested previously and found to be one of the strongest structural connections. The test conducted in Phase 2 are described in detail in the final report (*3*).

At the conclusion of Phase 2 it was determined that significant changes to improve performance should be made and that the final phase be initiated (Phase 3, NCHRP 230 compliance testing).

PHASE 3: REDESIGN AND FINAL COMPLIANCE TESTING

A complete analysis of the tests performed in Phase 2 was performed. Changes were made to the carrier beam and to the modules on the basis of this analysis. Those changes are described in detail in the final report (*3*).

The final three compliance tests are summarized in Table 3 and by Figures 2–4. In addition, Test D is shown to provide verification of improvement resulting from the modification of the side rail pipe taper. These tests are documented by test reports A, B, C, and D (Test 2 from development tests), which are in the appendix to the final report (*3*).

These tests are described in the following paragraphs, and the single change that was required to achieve ideal performance and unqualified compliance with NCHRP 230 is discussed.

Test A

In Test A, a 1979 Lincoln Continental impacted the ADIEM terminal at 60.3 mph (97.1 km/hr). The vehicle weight was 4,500 lb (2,041 kg).

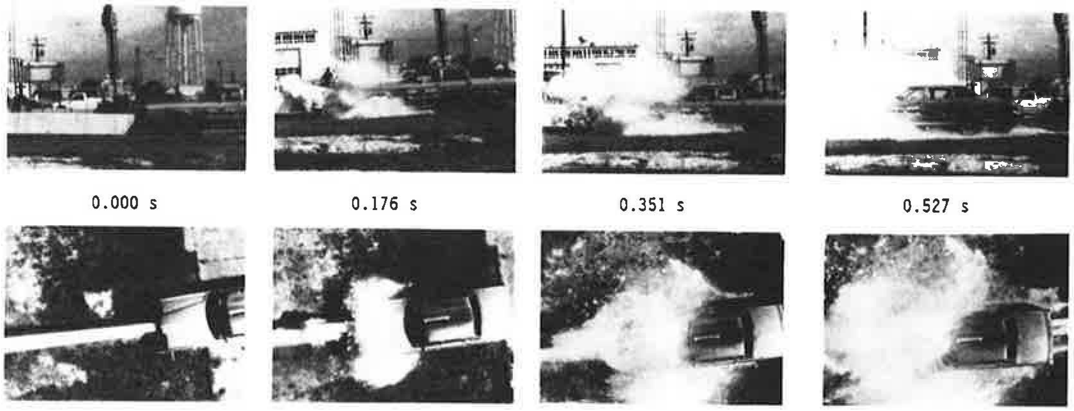
On impact, the modules began to crush at the design level of resistance. The vehicle remained extremely stable and level

TABLE 3 Summary of Compliance Test Data and NCHRP Report 230 Requirements

Test No. (Wt., Angle, Position, Speed)	NCHRP* 230 No.	Change in Velocity (longitudinal/lateral)	Acceleration (longitudinal/lateral)	Remarks:
A (4500 lb./0°/head on, 60.3 mph)	41	29.8 fps / NA (30)*	-6.3 g's / No Contact (15)	Performance good.
B (1800 lb./0°/15" offset, 58.6 mph)	45	37.4 fps / 8.9 fps (40)**	-10.6 g's / -1.6 g's (15)	Performance good.
C (1800 lb./15°/Side, 58.8 mph)	44	11.8 fps / -26.3 fps (30)	-4.9 g's / -7.3 g's (15)	Performance fair. Pitch larger than preferred. (Rail modification to correct problem verified by test D.)
D (1800 lb./15°/Side, 61.2 mph)	44	16.6 fps / 24.7 fps (30)	-1.8 g's / -5.0 g's (15)	Test verifies performance of rail modification

* Numbers in parentheses are NCHRP 230 Requirements (2, Table 8).

** Concerning the use of 40 fps as the value of ΔV for comparison in the 1800 lb head-on test (Test 42) it is noted that almost all terminal devices, certainly including the primary commercially supplied devices, do not conform to the 30 fps preferred value. Thus it has been the pragmatic approach to compare this small car test characteristic to the 40 fps limit. To do otherwise would require increasing the length of most barrier terminals by at least 10 feet, a step that is not seen as practical or cost effective. It might also be noted that some widely used guardrail terminals do not even meet the 40 fps limit.



Test No.	9901E-1
Date	10/16/90
Test Installation.	Adiem Impact Attenuator
Length of Installation. .	33.0 ft (10.1 m)
Vehicle	1979 Lincoln
Vehicle Weight	
Test Inerita	4,500 lb (2,041 kg)
Vehicle Damage Classification	
TAD.	12FC3
CDC.	12FCEN1

Impact Speed	60.3 mi/h (97.1 km/h)
Impact Angle.	0 deg - center
Exit Speed.	Not Applicable
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal.	-7.9 g
Lateral	-0.8 g
Occupant Impact Velocity	
Longitudinal.	29.8 ft/s (9.1 m/s)
Lateral	No Contact
Occupant Ridedown Accelerations	
Longitudinal.	-6.3 g
Lateral	N/A

FIGURE 2 Summary of results for Test 9901E-1.

as it penetrated the modules. The vehicle penetrated 25.6 ft (7.8 m) into the terminal.

The modules were all crushed to varying degrees. The carrier beam was not damaged. Minimal amounts of debris and detached pieces of soft concrete remained around the installation after the collision. The debris was confined to an area of about 10 ft on either side of the terminal extending a distance about 30 ft downstream from the beginning of the PCB. The carrier beam remained firmly attached to the ground and the PCB.

The vehicle received minimal damage. Maximum permanent deformation was 10 in. (25.4 cm) at the center of the front end of the vehicle. In addition, the vehicle sustained damage to the bumper, grill, and radiator. No intrusion into the occupant compartment occurred.

A summary of the test results and other information pertinent to this test is presented in Figure 2, along with sequential photographs of the collision. The maximum 0.050 sec average acceleration imposed on the vehicle was $-7.9\ g$ in the longitudinal direction. Occupant impact velocity in the longitudinal direction was 29.8 fps (9.1 m/sec). The highest 0.010 sec occupant ridedown acceleration was $-6.3\ g$ (longitudinal).

In summary, the terminal smoothly arrested the forward motion of the vehicle. The vehicle sustained minimal damage and did not present a significant hazard to other traffic. Occupant impact velocities and ridedown accelerations were within the limits recommended in *NCHRP Report 230* (i.e., 30 fps) (*2*). These test results meet the evaluation criteria recommended in *NCHRP Report 230*.

Test B

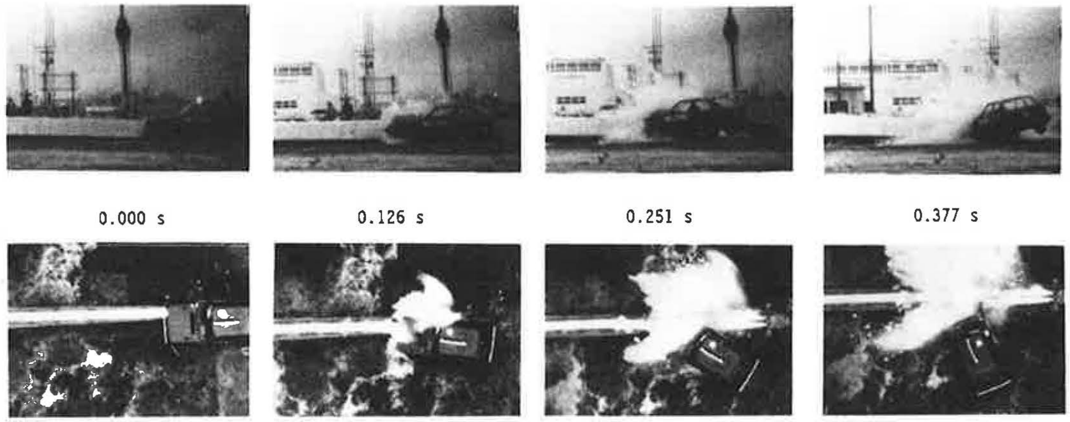
In Test B, a 1981 Honda Civic impacted the ADIEM terminal at 58.6 mph (94.3 km/hr). The vehicle weight was 1,800 lb (816 kg).

On impact, the modules began to crush as designed. The vehicle remained stable and level as it penetrated the first module. As the vehicle penetrated the second module, it began to yaw clockwise. The vehicle continued to yaw clockwise as module crush continued. The vehicle yawed to about 90 degrees as loss of contact between the Honda and the crushed modules occurred. The vehicle penetrated 9.9 ft (3.0 m) into the terminal.

All terminal modules were crushed to varying degrees. No damage occurred to the terminal carrier beam, the base structure. Minimal amounts of debris and small pieces of soft concrete were distributed around the installation. The modules yielded appropriately and the carrier beam remained firmly attached to the ground and the PCB.

Maximum permanent deformation was 9 in. (22.9 cm) at the right front corner of the vehicle. In addition, the vehicle sustained damage to the bumper, grill, radiator, front fenders, and right front strut assembly. No intrusion into the occupant compartment occurred.

The test results and other information pertinent to this test are summarized in Figure 3. The maximum 0.050 sec average acceleration experienced by the vehicle was $-11.7\ g$ in the longitudinal direction. Occupant impact velocity in the longitudinal direction was 37.4 fps (11.4 m/sec). Although this is above the recommended level of 30 fps, it is generally



Test No. 9901E-2	Impact Speed . . . . . . 58.6 mi/h (94.3 km/h)
Date 01/29/91	Impact Angle. . . . . . . 0 deg (15 in. right side offset
Test Installation. . . . . Adiem Impact Attenuator	Exit Speed. . . . . Not Applicable
Length of Installation. . 33.0 ft (10.1 m)	Vehicle Accelerations (Max. 0.050-sec Avg)
Vehicle . . . . 1981 Honda Civic	Longitudinal. . . . . . -11.7 g
Vehicle Weight	Lateral . . . . -3.1 g
Test Inerita . . . . . . 1,800 lb (816 kg)	Occupant Impact Velocity
Vehicle Damage Classification	Longitudinal. . . . . . 37.4 ft/s (11.4 m/s)
TAD. 12FR4	Lateral . . . . 8.9 ft/s (2.7 m/s)
CDC. 12FREN2	Occupant Ridedown Accelerations
	Longitudinal. . . . . . -10.6 g
	Lateral . . . . -1.6 g

FIGURE 3 Summary of results for Test 9901E-2.

observed that few terminals do better than meet the 40 fps limit for small car head-on tests (*2*, Table 8). The highest 0.010 sec occupant ridedown acceleration was -10.6 *g* (longitudinal).

In summary, the terminal functioned precisely as designed. The vehicle sustained significant damage, but no intrusion into the occupant compartment occurred. Occupant impact velocities and ridedown accelerations were within the limits recommended in *NCHRP Report 230*. These test results meet the evaluation criteria recommended in the report.

Test C

In Test C, a 1985 Dodge Colt impacted the ADIEM terminal at 58.8 mph (94.6 km/hr) at an angle of 15 degrees. The vehicle weight was 1,800 lbs (816 kg).

On impact, the vehicle began to redirect. As the vehicle redirected, the left wheels lost contact with the roadway. At approximately 0.140 sec, at a vehicle speed of 55.9 mph (89.9 km/hr), the rear of the vehicle came into contact with the terminal. The vehicle began to yaw counterclockwise and pitch as it became parallel to the terminal. The vehicle lost contact with the rail at approximately 0.245 sec, traveling 53.9 mph at an angle of 2.4 degrees. The brakes were applied as the vehicle exited the installation. The vehicle came to rest in a stable and upright condition 140 ft downstream from the point of impact.

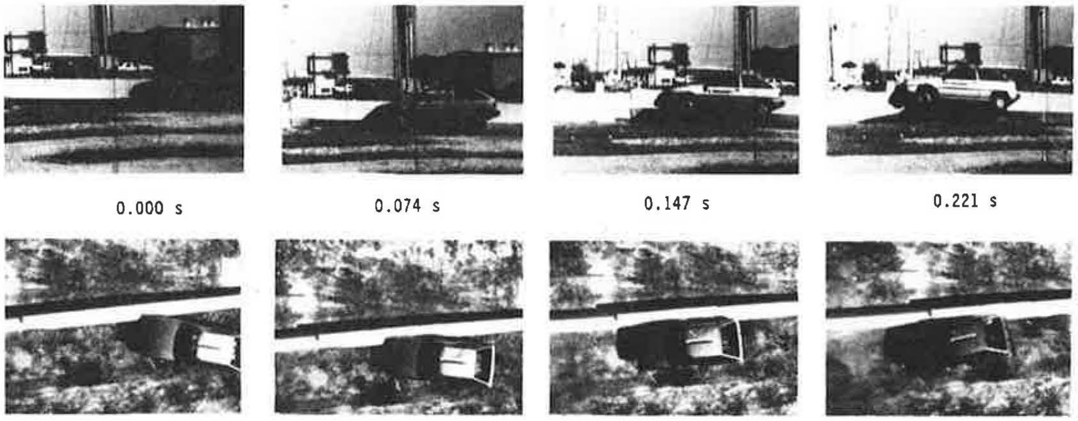
The soft concrete modules were scraped, but did not sustain any structural damage. The terminal carrier beam was not damaged. There was no debris or detached elements around the installation. The base structure remained firmly attached to the roadway and PCB.

The vehicle received modest damage, primarily to the right front control arm assembly, and wheel. The subframe and floorpan were bent. No intrusion into the occupant compartment occurred.

The test results and other information pertinent to this test are summarized in Figure 4. The maximum 0.050 sec average acceleration experienced by the vehicle was -5.4 *g* in the longitudinal direction and 15.7 g in the lateral direction. Occupant impact velocity in the longitudinal direction was 11.8 fps (3.6 m/sec) and 26.3 fps (8.0 m/sec) in the lateral direction. The highest 0.010 sec occupant ridedown accelerations were -4.9 *g* (longitudinal) and 7.3 *g* (lateral).

In summary, the terminal safely redirected the vehicle. Occupant impact velocities and ridedown accelerations were within the limits recommended in *NCHRP Report 230*. These test results fundamentally meet the evaluation criteria recommended in *NCHRP Report 230*, but did not meet the expectations of the designers. More vehicle pitch than expected occurred. A careful examination of the terminal and the vehicle and comparison of this test to Test D yielded the reason.

In Test D the 1,800-lb vehicle impacted a similar side rail on an earlier ADIEM terminal at a speed of 60 mph and an angle of 15 degrees. The result was an extremely smooth and safe redirection [*3*, appendix (Test Report 9429G-2)]. A quick comparison of the acceleration traces in these two tests showed that the vehicle in Test D lost only 5 mph during the first 100 msec, whereas the Test C vehicle lost about 12 mph. Clearly,



Test No.	9901E-3	Impact Speed	58.8 mi/h (94.6 km/h)
Date	02/08/91	Impact Angle.	15.9 deg
Test Installation.	Adiem Impact Attenuator	Exit Speed.	57.2 mi/h (92.1 km/h)
Length of Installation. .	33.0 ft (10.1 m)	Vehicle Accelerations (Max. 0.050-sec Avg)	
		Longitudinal.	-5.4 g
Vehicle	1985 Dodge Colt	Lateral	15.7 g
Vehicle Weight		Occupant Impact Velocity	
Test Inerita	1,800 lb (816 kg)	Longitudinal.	11.8 ft/s (3.6 m/s)
Vehicle Damage Classification		Lateral	-26.3 ft/s (8.0 m/s)
TAD.	01RFQ2	Occupant Ridedown Accelerations	
CDC.	01RFEW1	Longitudinal.	-4.9 g
		Lateral	-7.3 g

FIGURE 4 Summary of results for Test 9901E-3.

there was much more retarding force in Test C on the front wheel than in Test D. Inspection of the right front wheel rim and the point on the ADIEM side rail where the major redirective load was applied yielded the answer: in Test C, the wheel rim impacted on the 3-ft tapered part of the side rail. The way the taper was produced was by simply slicing away a portion of the pipe and replacing it with a flat plate. The pipe was then welded to the angle section with the flat part of the taper out, or facing the impacting wheel. At the bottom of the taper section, replacing the section of pipe with a flat plate results in an edge with a blunt radius of about 1/8 in. facing down and another edge facing up. As the wheel rim applied force to the tapered section during initial impact, the lower edge of the taper cut into the rim on the trailing side of the rim. The rotation of the wheel and friction with the ground forced the wheel down about the pivot point at the place the side-rail edge cut into the rim. The result was that the tire was forced downward almost to the rim; the resulting vertical force translated into a friction (retarding) force on the right front tire that was at least ten times what could normally be produced by braking the tire on the same surface. Thus, the right front was forced down by the edge, and a large force to the rear occurred at the tire-ground interface. The result was the unexpected pitch that occurred in Test C. The solution to this minor problem was obvious: in Test D, the wheel impacted a curved pipe surface, and an ideal redirection occurred. Thus, the only necessary change in the design was to put the flat surface of the pipe taper flush with the carrier beam side and have the curved surface of the taper facing out to accommodate the impact of the wheel. With this small design modification, it is clear the ADIEM terminal will perform well under all required tests in *NCHRP Report 230* (*2*). In the more extensive TTI/TxDOT report (*3*), the adequacy of this design change is discussed in great detail. There is not a conceivable way in which changing the surface of the pipe taper can also affect the head-on tests. It is clear that ADIEM will gate if struck by a large car at a significant angle in the first one-third of the barrier. Gating, however, is a nonissue as long as the length of need is properly accommodated. This simply requires that the length of need be set at the end of the PCB or CMB and not be accommodated by the full length of an ADIEM.

FINAL DESIGN

The final design functions well for vehicle speeds up to 60 mph and for vehicle weights up to 4,500 lb. It is composed of a 30-ft carrier beam or base structure that accommodates 10 Perlite concrete crushable modules. Details of this design are available on full size plan sheets 2/4, 3/4, and 4/4 (*3*).

The carrier base of ADIEM is composed of standard Class A five sack concrete. Longitudinal reinforcement is predominantly No. 5 bars. Transverse reinforcement is all No. 4 bars.

Ten modules are required for an installation. Details are shown in Figure 5. These modules are cast in three layers of varying strength, shown in Figure 6. The lowest 3 in. is Concrete T, 120 psi compressive strength. The next 14 in. is Concrete M, 40 psi compressive strength. The final top 7 in. is Concrete T, 120 psi compressive strength. The constituents of these three levels of Perlite concrete are shown in Figure 6. Perlite is an expanded inert mineral soil filler normally used for soil aeration. It weighs only about 7.5 lb/ft^3 in bulk form, and single particles are not usually more than ⅛ in. in diameter. When concrete is made of Perlite, white portland cement, water, and an air-entraining agent, it is extremely lightweight and has a white color. Wet unit weights are given between 25 and 40 lb/ft^3, but these unit weights decrease as the concrete hydrates and dries, approaching 80 percent of the wet unit weights. The average dry weight of the module concrete is only about 30 lb/ft^3. A complete module after curing weighs about 190 lb and can be installed by two people (see Figure 7).

Both the strength and durability of the Perlite crushable modules are of great importance. If the strength levels are not controlled during the precasting phase within reasonable boundaries, the resisting forces during collisions, and thus accelerations on impacting vehicles, could vary significantly from those observed in the compliance testing. Unit weight of wet Perlite is one indicator of final strength, but water/cement ratio and Perlite aggregate content are also important. The need for control of the strength level of the module is important and must be verified by postcuring testing, not simply implied by wet concrete batch characteristics. A penetrometer, developed for this test program, is an appropriate way to determine strength after curing. These levels are from 30 to 60 psi for the low-strength concrete and 100 to 150 psi for the higher-strength concrete. During the 3 years of development and construction, it was found these ranges were both appropriate from a performance standpoint and practically achievable in the batching process. These observations can be made at any time after 21 days of curing. The average of six penetrometer tests should be compared with these limits. Note that if the penetrometer is placed directly over an element of wire reinforcement, the reading will be invalid. It will also be arbitrarily high. With a little practice, the individual conducting the penetrometer test can tell immediately if a wire element interferes with a reading. The difference is normally great.

The batching procedure and quality control necessary to achieve reasonable control of this ultra-lightweight Perlite concrete could be the subject of an entire TRB publication. For this reason, the writers recommend against fabrication of the soft modules by other than experienced precasters and

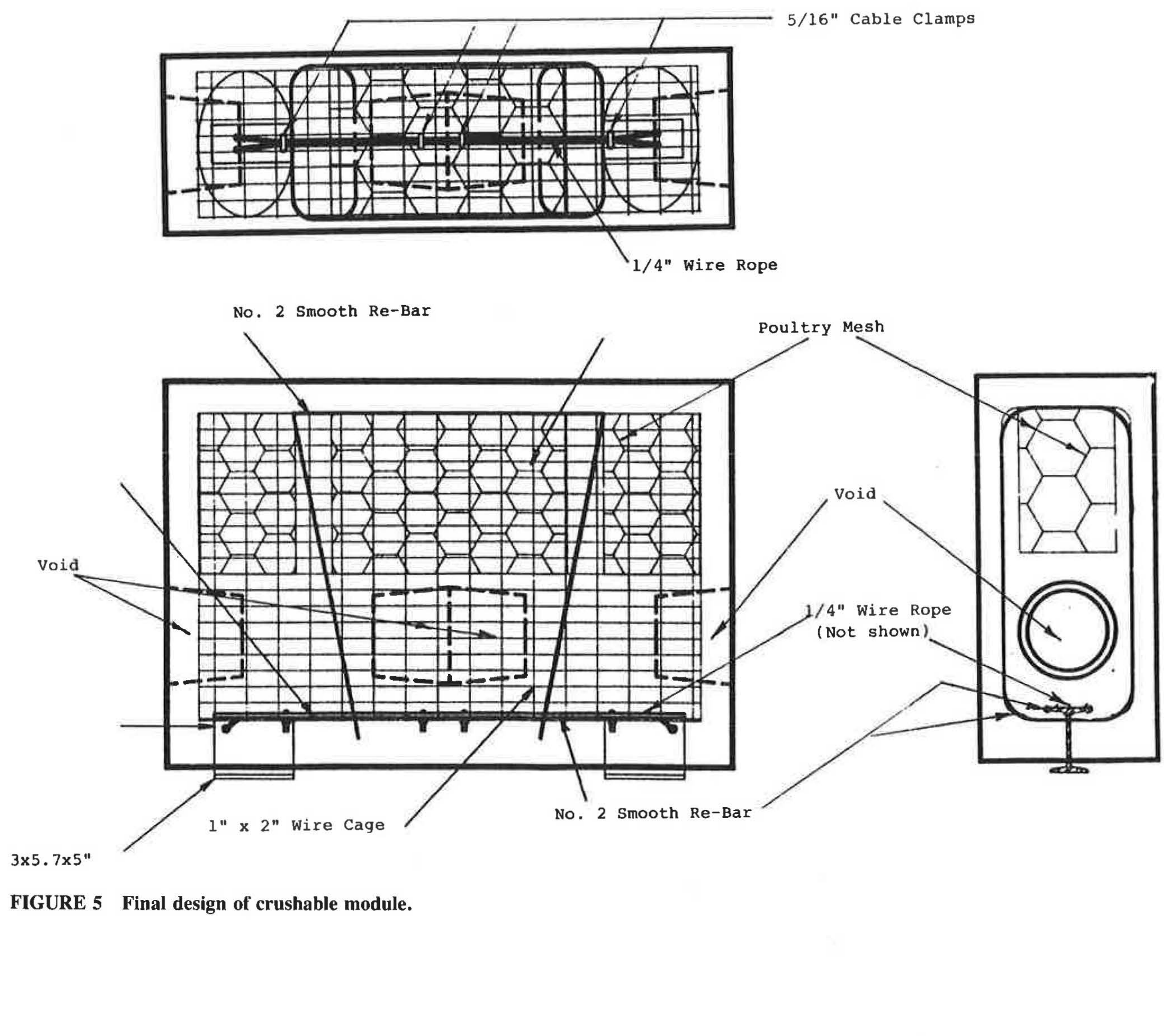
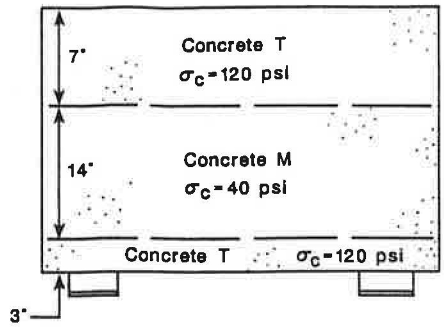


FIGURE 5 Final design of crushable module.



Note: Reinforcement is not shown.

ELEVATION SINGLE MODULE

	Concrete T	Concrete M
Cement	340 lbs.	180 lbs.
Water	425 lbs.	350 lbs.
Perlite	205 lbs.	225 lbs.
Air Agent	1000 cc	1300 cc
Unit Weight	36 lbs./ft.^{3}	28 lbs./ft.^{3}
Compressive Strength	120 psi	40 psi

Note: These batch designs are applicable for the brand of Perlite used in this program. Trial batch designs to verify appropriate strength will be necessary when other brands are used and possibly when the Perlite provided by a particular supplier varies from shipment to shipment. Unit weight is a good early warning of product variability.

FIGURE 6 Final concrete placement recommended for modules.

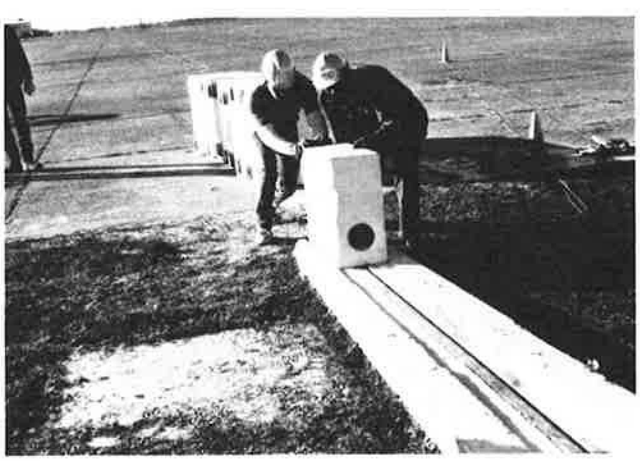


FIGURE 7 Installation of ADIEM terminal modules.

only then when subject to the counsel of the developers. TTI expects to exercise effective control over all manufacturers to ensure appropriate quality control.

Durability of a low-strength concrete, especially the 40 psi portion of the modules, is required. The problem is obvious. The uncoated concrete will absorb water. It is highly porous. If that water freezes, the 40 psi material will gradually deteriorate. The solution is to coat the modules to keep their surfaces impermeable. Two products have been found to perform well in the laboratory. They are two coats of Alkyd Traffic Marking Paint (in white or yellow) and Plasti-Dip #11602 (PDI, Inc.), which is an elastomeric rubber. Freeze-thaw testing of these coatings on samples of low-strength Perlite showed this approach to be effective. During the manufacturing process the coating should be applied only after the individual modules have passed the penetrometer test. The coatings should also be applied so that the surface is fully

TABLE 4 Cost Estimates

(Based on invoice costs of small quantities during construction of one barrier.)

BASE (1) (Carrier beam for modules and redirection rails.)		
Re-Bar	#4 & #5	$800.00
Concrete	2.5 yds. @ $46.00	115.00
3" S Beams	(70' @ $1.65/ft.)	115.00
3" Pipe	(30' @ $1.80/ft.)	54.00
	Sub-total	$1084.00
MODULES (10)		
2" x 4" welded wire (60' @ $0.30/ft.)		$ 18.00
Poultry Wire (44' @ $0.40/ft.)		18.00
Re-Bar, No. 2	250 ft.	25.00
Perlite	(25 bags @ $9.50/bag)	238.00
White Cement	(10 bags @ $10.40/bag)	104.00
¼" Wire Rope and cable clamps		80.00
	Sub-total	$ 483.00
	Total of Materials	$1567.00
BASE (1) (Does not include cost of form.)		
Assembly of forms	5 man-hours	
Placing and tying reinforcement	14 man-hours	
Placing Concrete (Redi-Mix Truck)	1 man-hour	
Breaking out base	2 man-hours	
	Sub-total 22 man-hours @ $15.00/hr. =	$ 330.00
MODULES (10) (Does not include cost of forms.)		
Assembly of forms	8 man-hours	
Fabrication of reinforcement	36 man-hours	
Placing concrete	12 man-hours	
Breaking out modules	5 man-hours	
	Sub-total 61 man-hours @ $15.00/hr. =	$ 915.00
	Total Labor	$1245.00
	Grand Total* Labor and Material	$2812.00

* In a research oriented non-production environment.

covered, leaving no avenue for water intrusion. If unprotected Perlite concrete is allowed to absorb water and a hard freeze then occurs, the concrete will become unstable and compressive strength will be rapidly compromised. It is the view of the researchers that these modules will remain effective under all weather conditions indefinitely as long as the coating is effective in preventing water intrusion. Side angle hits may or may not require module replacement. If the modules are structurally intact with no significant fractures from a visual inspection, the module can be reused. Otherwise, the module should be replaced. When partial impacts or scuffing occurs and damages the coating, the affected areas should be recoated to avoid long-term deterioration. The modules can be damaged during handling, but the potential for cosmetic damage affecting performance is not significant. The protective coatings will mask cosmetic damage to some extent, but will not mask significant structural damage.

The cost of an ADIEM terminal is presented in Table 4. These costs were based on construction of three carrier bases and some 70 modules in a prototype development environment. Table 4 shows material costs of $1,567, labor costs of $1,245.00, and a total cost of $2,812.00. It is likely that complete cushions could be fabricated in a production environment for two-thirds of this cost. This would yield a production cost per barrier of $2,000. Allowing 50 percent for profit margins, it is estimated that this cushion could be placed in the field for $3,000, plus a reasonable cost of installation. All significantly damaged modules should be replaced in a reasonable time following a collision. Low-speed head-on collisions will probably require the replacement of only a few modules. Speeds up to 45 and higher will probably require all new modules. Many side angle hits may require no module replacement. In construction zones, due to the completely precast portable construction, it is estimated the complete end treatment can be installed in less than 1 hr. A two-person crew was timed to determine the time necessary to clear a terminal that had been completely crushed. The time was 17 min. Extraordinary efforts to do the job quickly were not made. The same crew then retrieved 10 modules from a truck bed and replaced those in the carrier beam in 15 min. In most cases it is estimated that a collision site could be restored in about 30 min by a two-person crew with the use of a straight or dump truck. It is also advisable to sweep the site because small elements of debris will be distributed about the collapsed modules.

CONCLUSION

ADIEM, the low-cost end treatment for PCBs and CMBs, has been subjected to eight full-scale crash tests. Four of these tests were developmental; four were the compliance tests suggested in *NCHRP Report 230* (*2*). The results of the four compliance tests are presented in Table 5. These results show that the final terminal design clearly meets the requirements of *NCHRP Report 230* (*2*). The design has been approved by FHWA for terminal applications. This terminal is by far the most economical of the terminals now in use that have *NCHRP Report 230* performance characteristics. It is believed that the cost-effectiveness of this design will be demonstrated as field experience is gained. ADIEM is now ready for field appli-

TABLE 5 Results of Compliance Crash Tests

Test Type	Test No.	NCHRP* 230 No.	Results	Comments:
Compliance	A	41	Excellent	Met all requirements of NCHRP 230. Barrier performance ideal.
Compliance	B	45	Excellent	Met all requirements of NCHRP 230. Barrier performance ideal.
Compliance	C	44	Fair*	Met all requirements of NCHRP 230 except that vehicle pitch was more than would be preferred. (See footnote *.)
Compliance	D	44	Excellent	Met all requirements of NCHRP 230. Barrier performance ideal.

* Simple rail modification required to produce excellent performance verified by test D.

cation as a portable terminal for construction zones and as a permanent terminal for concrete barriers.

ACKNOWLEDGMENT

The authors are grateful for the cooperation and support of Harold D. Cooner, Frank D. Holzmann, and William A. Lancaster, of TxDOT.

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