TRANSPORTATION RESEARCH RECORD 868

Testing, Evaluation, and Upgrading of Safety Appurtenances

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1982

Transportation Research Record 868
Price $8.20
Edited for TRB by Scott C. Herman

mode
1 highway transportation

subject areas
21 facilities design
40 maintenance
51 transportation safety

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Testing, evaluation, and upgrading of safety appurtenances.

(Transportation research record; 868)
Reports prepared for the 61st annual meeting of the Transportation Research Board.
Includes bibliographical references.
1. Roads–Safety measures–Addresses, essays, lectures.
I. National Research Council (U.S.). Transportation Research Board. II. Series
TE7.H5 no. 868 [TE228] 380.5s [625.7'95] 82-24526
ISBN 0-309-03372-1 ISSN 0361-1981

Sponsorship of the Papers in This Transportation Research Record

GROUP 2–DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
R. V. LeClerc, consultant, Olympia, Washington, chairman

General Design Section
Lester A. Herr, T.Y. Lin and Associates, chairman

Committee on Safety Appurtenances
Jarvis D. Michie, Southwest Research Institute, chairman
Gordon A. Alison, William E. Behm, Jeffrey Alan Bloom, James E. Bryden, Arthur M. Dinitz, Malcolm D. Graham, C. William Gray, James R. Hackney, James H. Hatton, Jr., Roger W. Hove, William W. Hunter, Max N. Jensen, Frank N. Lisle, Edwin C. Lokken, Roy J. Mohler, John F. Nixon, Eric F. Nordlin, Edward Robert Post, William L. Raymond, Jr., Hayes E. Ross, Jr., F.G. Schlosser, Flory J. Tamanini, Harry W. Taylor, John G. Viner

Lawrence F. Spaine, Transportation Research Board staff

The organizational units, officers, and members are as of December 31, 1981.

Contents

Authors of the Papers in This Record

Bronstad, M.E., Southwest Research Institute, P.O. Drawer 28510, 6220 Culebra Road, San Antonio, TX 78284
Bryden, James E., New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, NY 12232
Buth, C. Eugene, Texas Transportation Institute, Texas A&M University, College Station, TX 77843
Cooner, Harold D., Texas State Department of Highways and Public Transportation, State Highway Building, 11th and Brazos, Austin, TX 78701
Damon, C.P., Federal Highway Administration, 862 Federal Office Building, Austin, TX 78701
Fox, Samuel V., Texas State Department of Highways and Public Transportation, State Highway Building, 11th and Brazos, Austin, TX 78701
Hahn, Kenneth C., New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, NY 12232
Hall, J.W., Bureau of Engineering Research, University of New Mexico, Albuquerque, NM 87131
Hirsch, T.J., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
Ivey, Don L., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
Kimball, C.E., Jr., Southwest Research Institute, P.O. Drawer 28510, 6220 Culebra Road, San Antonio, TX 78284
McDevitt, Charles F., Office of Research, HRS-12, Federal Highway Administration, 400 7th Street, S.W., Washington, DC 20590
Nixon, John F., Texas State Department of Highways and Public Transportation, State Highway Building, 11th and Brazos, Austin, TX 78701
Nordlin, Eric F., Office of Transportation Laboratory, Division of Construction, California Department of Transportation, 5900 Folsom Boulevard, P.O. Box 19128, Sacramento, CA 95819
Robertson, Richard, Texas Transportation Institute, Texas A&M University, College Station, TX 77843
Ross, Hayes E., Jr., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
Sicking, Dean, Texas Transportation Institute, Texas A&M University, College Station, TX 77843
Stocker, Arthur J., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
Stoker, J. Robert, Office of Transportation Laboratory, Division of Construction, California Department of Transportation, 5900 Folsom Boulevard, P.O. Box 19128, Sacramento, CA 95819
Stoughton, Roger L., Office of Transportation Laboratory, Division of Construction, California Department of Transportation, 5900 Folsom Boulevard, P.O. Box 19128, Sacramento, CA 95819

Safety Treatment of Roadside Drainage Structures

HAYES E. ROSS, JR., DEAN SICKING, T.J. HIRSCH, HAROLD D. COONER, JOHN F. NIXON, SAMUEL V. FOX, AND C.P. DAMON

The purpose of the research was to develop traffic-safe end treatments for cross-drainage and parallel-drainage structures that would not appreciably restrict water flow. Preliminary designs were first evaluated by computer simulation, by use of a test pit in which the clear open space and grate spacing could be varied, and by use of an earth berm similar in geometry to a driveway. Promising designs were then subjected to full-scale prototype testing by using both subcompact and full-sized automobiles. Finally, guidelines for implementation of designs were developed by using a cost/benefit analysis. Traffic-safe culvert end treatments can be achieved as follows. For cross-drainage structures, (a) all culvert ends not shielded by a traffic barrier should be made to match the existing side slope with no protrusion in excess of 4 in (10.2 cm) above grade, (b) culverts with clear openings 30 in (76.2 cm) or less need no safety treatment other than that mentioned in a, and (c) culverts with clear openings greater than 30 in can be made traffic-safe by grate members placed on 30-in centers oriented parallel to the flow and in the plane of the side slope. For parallel-drainage structures, (a) the roadway side slope (or ditch slope) should be 6:1 or flatter in the vicinity of the driveway, (b) the driveway slope should be 6:1 or flatter, (c) the transition between the side slope and the driveway slope should be rounded, and (d) safety treatment of the culvert opening should include an end section cut to match the driveway slope and have cross members (grates) spaced every 24 in (61.0 cm) perpendicular to the direction of water flow.

In designing drainage culverts, the primary objective is to properly accommodate surface runoff along the highway right-of-way. However, a second important goal should be to provide a traffic-safe design that would be traversable by an out-of-control vehicle without rollover or abrupt change in speed.

Guidelines for designing traffic-safe grates have been very limited. The National Cooperative Highway Research Program (NCHRP) published guidelines for traffic-safe drainage structures in 1969 (1). The recommendations dealt primarily with the geometry of adjoining slopes. Computer simulations have also been used to further investigate the dynamic behavior of automobiles traversing various slope and ditch configurations near driveways and median crossovers (2,3). Criteria for the structural design of inlet grates was published in 1973 (4). However, the study did not address the problem of grate design as related to safety.

Recent field reviews of drainage culverts in Texas revealed that improvements and some modifications of design details could improve both drainage and safety (5). Many of the older safety grates used to cover the open ends of culverts have small openings and the grates are easily clogged with debris, which causes water to back up and flow over the roadway, the ditch crossing, or adjacent property. In some cases safety grates do not possess enough strength to be effective or they are used on small pipe culverts that need no safety treatment.

The objective of this study was to develop guidelines for safety treatment of both cross-drainage and parallel-drainage structures that (a) can be safely traversed by an errant vehicle and (b) will exhibit desirable hydraulic behavior. Although no hydraulic analyses were made, it was assumed that hydraulic efficiency increases as the number of grate members decreases. It was therefore a goal of the research to meet the safety requirements by using as few grate members as possible.

This paper summarizes the findings of two research studies, one conducted in 1979 (6) and the other in 1980 (7). Reference should be made to the cited literature for complete details of the studies.

EVALUATION CRITERIA

A review of the literature showed that there are no nationally recognized safety performance standards for roadside drainage structures. Deceleration and stability of a vehicle during and following impact are the two primary measures of performance for safety appurtenances such as guardrails, crash cushions, etc. (8). For cross-drainage structures, performance was judged satisfactory if the vehicle smoothly traversed the culvert and the adjoining ditch slope without rollover for speeds from 20 mph (32.2 km/h) through 60 mph (96.5 km/h).

Previous research (2,6) indicated that a very flat ditch slope, a very flat driveway slope, and a very long culvert would be necessary to satisfy the above criteria for parallel-drainage structures. In view of the economic and hydraulic implications of such a design, it was concluded that trade-offs would be necessary to achieve an acceptable balance between the controlling elements. Performance of parallel-drainage structures was therefore judged acceptable if the vehicle smoothly traversed the adjoining slopes and culvert without rollover for speeds from 20 mph through 50 mph (80.5 km/h).

RESEARCH APPROACH

A four-phase approach was taken in the development of safety treatments of both cross-drainage and parallel-drainage structures. In the first two phases, computer simulations in combination with a preliminary test program were used to develop tentative design concepts. In the latter phase, prototypes were constructed by using the results of the preliminary studies and tested under representative roadside configurations. Final designs were then studied by using a cost/benefit analysis to develop guidelines for their implementation.

Cross-Drainage Structures

Simulation Studies

A computer simulation study was conducted that used the Highway-Vehicle-Object-Simulation Model (HVOSM) (9) to evaluate wheel drop into various culvert openings on flat terrain. HVOSM was also used to investigate the effect that a ramp at the leading edge of the culvert opening would have on vehicle behavior. (Figure 5, which is shown later in this paper, illustrates the ramp.) Ramps that have the following dimensions were evaluated (1 in = 2.54 cm):

Ramp	Dimension (in) Horizontal	Vertical
1	3.0	3.0
2	6.0	3.0
3	6.0	6.0
4	12.0	6.0

A 1974 Honda Civic was simulated in each of the computer runs because it was assumed that a mini-sized automobile would be more critical than a larger vehicle for the given conditions. A speed of 20 mph was used in each run, since it was deemed a critical speed. At higher speeds it was felt that it would be easier for the vehicle to clear the opening. At lower speeds, even though the vehicle would tend to drop more, velocity changes would be tolerable.

Preliminary Tests

In the second phase, a test pit was constructed on flat terrain (as shown in Figure 1) to study the behavior of a vehicle as it traversed various openings. The objectives of these tests were to determine preliminary values for (a) the maximum clear opening permissible on a nongrated culvert end and (b) the maximum spacing permissible when grates are necessary. All runs were live-driver tests at various speeds and encroachment angles. Figure 2 is a photograph of the test pit after installation. A total of 30 runs were made to determine the maximum clear opening. A test matrix for this series of tests is shown in Table 1. All tests were with a

Figure 1. Plan view of culvert test pit.

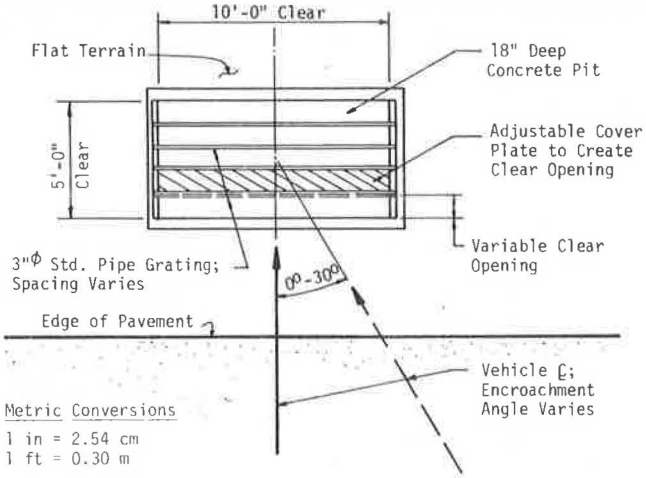


Figure 2. Test pit installation.

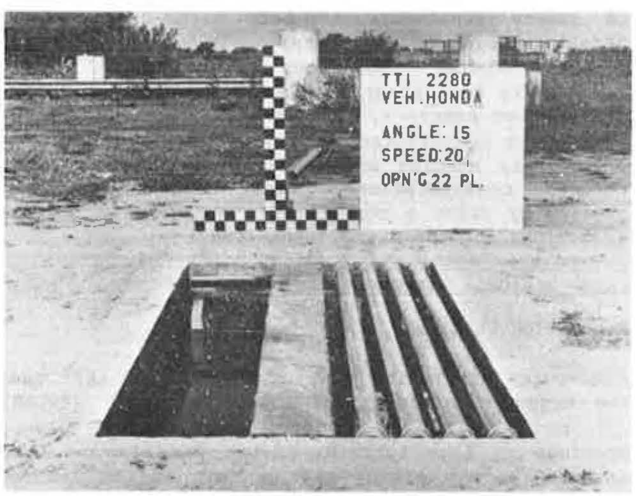


Figure 3. Sequential photographs of nongrated culvert test, 30-in clear opening, 1974 Honda Civic.

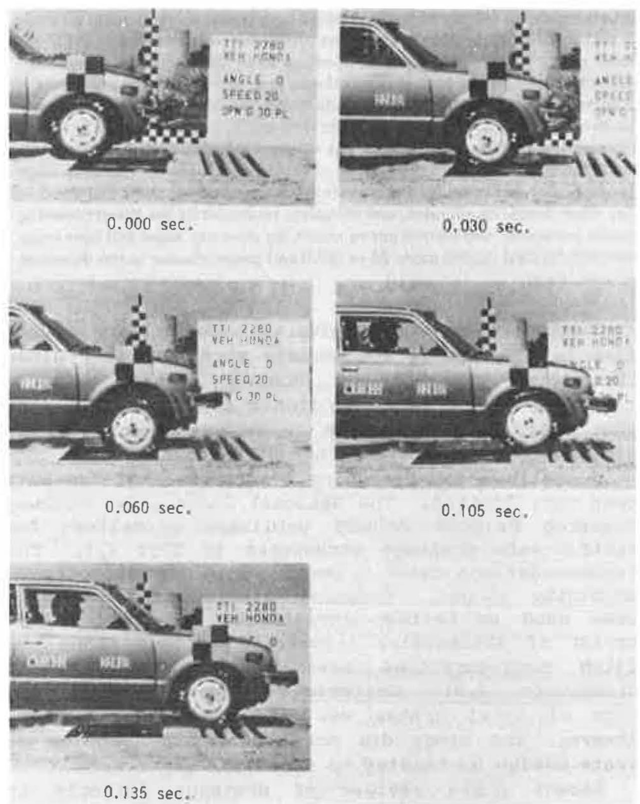


Table 1. Clear opening tests.

Test No.	Test Speed (mph)	Clear Opening (in)	Impact Angle (°)	Test No.	Test Speed (mph)	Clear Opening (in)	Impact Angle (°)
1	5	16	0	16	15	24	15
2	10	18	0	17	20	26	0
3	20	20	0	18	20	26	15
4	20	20	15	19	15	26	0
5	10	20	0	20	15	26	15
6	10	20	15	21	20	28	0
7	20	22	0	22	20	28	15
8	20	22	15	23	25	30	0
9	15	22	0	24	25	30	15
10	15	22	15	25	20	30	0
11	10	22	0	26	20	30	15
12	10	22	15	27	35	36	0
13	20	24	0	28	30	36	0
14	20	24	15	29	25	36	0
15	15	24	0	30	25	36	15

Note: 1 mph = 1.609 km/h; 1 in = 2.54 cm.

Table 2. Grate spacing tests.

Test No.	Vehicle Weight (lb)	Test Speed (mph)	Grate Spacing (in)	Encroachment Angle (°)
1	1970	10	16	0
2	1970	10	16	15
3	1970	10	16	30
4	1970	5	16	0
5	1970	5	16	15
6	1970	20	20	0
7	1970	20	20	15
8	1970	15	20	0
9	1970	15	20	15
10	1970	10	20	0
11	1970	10	20	15
12	1970	20	24	0
13	1970	20	24	15
14	1970	15	24	0
15	1970	15	24	15
16	1970	25	30	0
17	1970	25	30	15
18	4500	25	30	0
19	1970	20	30	0
20	1970	20	30	15
21	4500	20	30	0
22	4500	20	30	15

Note: 1 lb = 0.454 kg; 1 mph = 1.609 km/h; 1 in = 2.54 cm.

1974 Honda Civic that has a curb weight of approximately 1800 lb (817.2 kg). Limiting values were determined by the severity of the ride as judged by the driver. The driver was a Texas Transportation Institute (TTI) technician who was a nonprofessional driver. Sequential photographs of a 20-mph run with a 30-in (76.2-cm) clear opening are shown in Figure 3.

On completion of the clear-opening tests, the pit was used to determine the maximum permissible grate spacing. A total of 22 live-driver tests were conducted for this purpose. Table 2 gives a matrix of the grate-spacing tests conducted. The grates were 3-in (7.6-cm) schedule-40 steel pipe anchored to a steel beam that allowed adjustments of the pipe to any desired spacing. Figure 4 shows the pit setup for a 16-in (40.6-cm) grate spacing. Each grate configuration was evaluated with the 1974 Honda Civic. A 1975 Plymouth Fury that weighed about 4500 lb (2043 kg) was also used to evaluate the larger grate spacings.

As part of the second phase of the study, a limited number of live-driver tests were conducted to further evaluate the effects of a ramp at the leading edge of the culvert opening. Based on HVOSM results, a ramp that had a horizontal dimension of 12 in (30.5 cm) and a vertical dimension of 6 in (15.2 cm) was selected and constructed. HVOSM indicated that this combination would produce the greatest wheel hop of all combinations considered. The 1974 Honda Civic and the 1975 Plymouth Fury were used on the ramp test. Each test was conducted at 20 mph. Wheel hop and sprung mass center-of-gravity (cg) position for the test of the Plymouth Fury are shown in Figure 5.

Figure 4. Test pit with 16-in grate spacing.

Prototype Tests

Based on results obtained from the preliminary studies, two culvert structures were constructed for full-scale testing. They consisted of a 30-in-diameter corrugated steel-pipe culvert and a 5-ft (1.5-m) wide by 3-ft (0.92-m) high concrete box culvert that had adjoining head and wing walls. Grate members on the box culvert consisted of 3-in schedule-40 steel pipe on 30-in centers. Photographs of both installations are shown in Figure 6.

General details of the six tests conducted are shown in Figure 7. Note that the culverts were subjected to tests with both mini- and full-sized automobiles. In each test, with the exception of test 5, all four wheels of the test vehicle crossed the sloped culvert opening. In test 5 the vehicle straddled the cross member at the end of the box culvert, which allowed the left-side wheels to drop approximately 1.5 ft (0.46 m) to the ditch bottom and caused the vehicle to roll over. Sequential photographs of test 4 are shown in Figure 8.

Analysis of the strength requirements of grate members indicated that a 3-in inner diameter (ID) schedule-40 pipe was adequate for spans up to 12 ft (3.7 m). Because grate spans on many box culverts would exceed 12 ft, it was concluded that a limited test program should be undertaken to determine pipe size requirements for larger spans. To accomplish this, another test pit was constructed on flat terrain. The pit was 20 ft (6.1 m) long, 10 ft (3.1 m) wide, and 1.5 ft (0.46 m) deep. A total of four full-scale vehicle tests were conducted by using a 4500-lb (2043-kg) vehicle, each at 20 mph and each

Figure 5. Wheel hub and car displacement versus time for ramp run, 20 mph, 1975 Plymouth Fury.

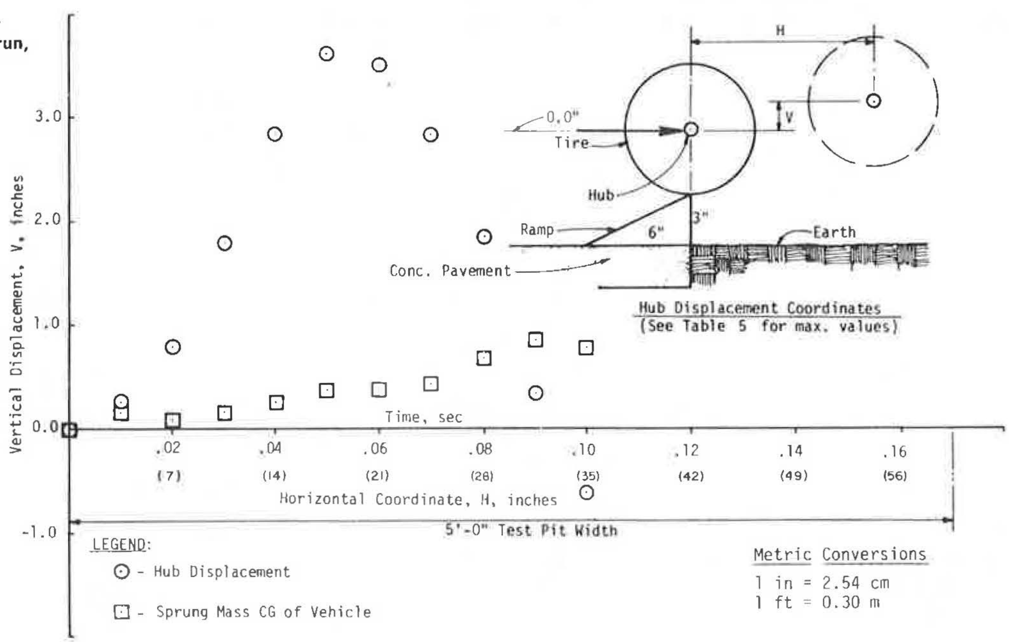
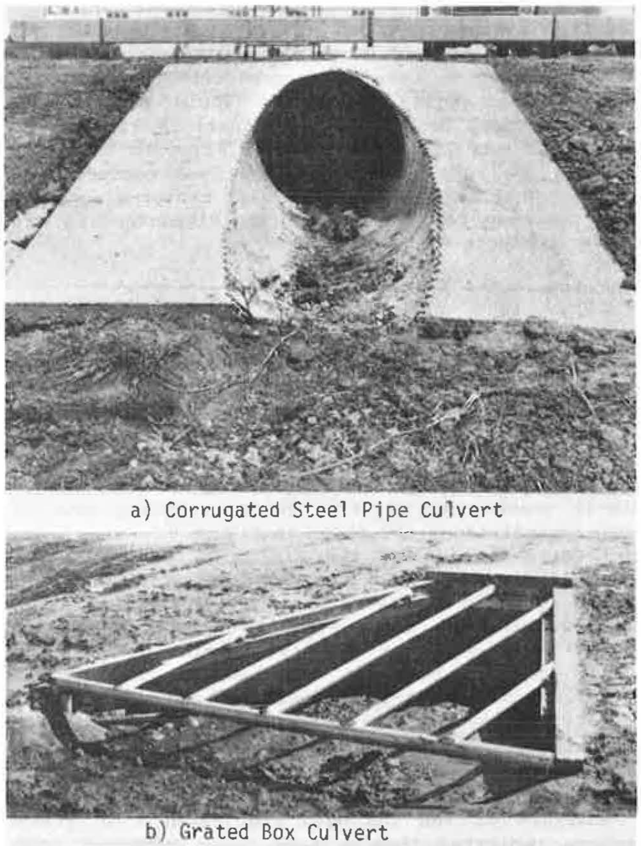


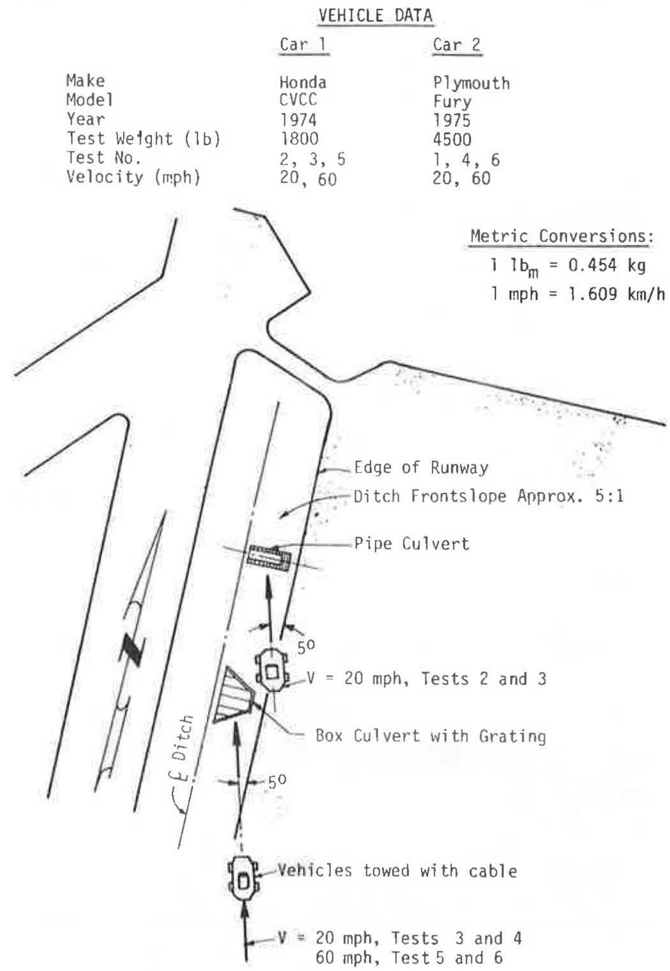
Figure 6. Prototype test installations.



a) Corrugated Steel Pipe Culvert

b) Grated Box Culvert

Figure 7. Plan view of site for prototype tests.



at a head-on approach perpendicular to the 20-ft dimension of the pit. Further details of each test are given in Table 3, including the permanent deformations noted after each test. With the exception of test 4, the grates had a 20-ft clear span. In test 4, vertical supports that consisted of 3-in ID schedule-40 pipe were placed at midspan of each of the three grate members. The grates were attached to the walls of the pit with a pin connection, which was constructed according to Texas State Department of Highways and Public Transportation (TSDHPT) standards.

Cost/Benefit Analysis

Guidelines for safety treatment of cross-drainage structures were developed in 1978 by using a cost/benefit analysis (10). Alternatives considered included (a) no treatment or baseline option [it was assumed, however, that the culvert end would be made to match the existing side slope with no protrusions greater than 4.0 in (10.2 cm) above grade for the baseline option], (b) extend the culvert end to 30 ft (9.2 m) from the edge of the travelway, (c) install guardrail, or (d) place a traffic-safe grate as recommended herein. Initial costs of grates recommended here are significantly less than similar costs for culvert grates studied by Kohutek and Ross (10). Their analysis was therefore repeated. Current cost data for recommended grates were discounted back to 1978 at a discount rate of 10 percent. Adjusted 1978 cost figures for the addition of culvert grates on six different slope and culvert combinations are shown in the table below (1 in = 2.54 cm; 1 ft = 0.30 m):

Embankment Slope	Culvert	Grate Cost ($)
2.5:1	36-in-diameter pipe	380
6:1	36-in-diameter pipe	4 600
2.5:1	4x6-ft single box	1 270
6:1	4x6-ft single box	5 100
2.5:1	4x6-ft double box	2 100
6:1	4x6-ft double box	11 800

The reader should refer to Kohutek and Ross (10) for further information on costs of other options and a description of the cost-effectiveness model used in the analysis.

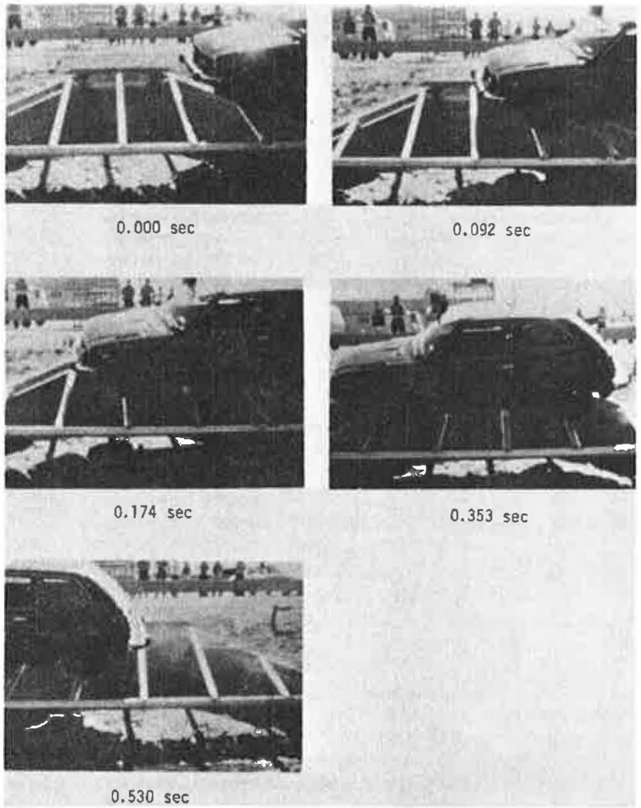
The cost/benefit analysis revealed that safety treatment beyond the baseline option of 36-in (91.4-cm) diameter or smaller cross-drainage pipe culverts is generally not warranted for traffic volumes of 20 000 vehicles/day or less. Safety treatment of larger box culverts was cost beneficial in most cases for traffic volumes greater than approximately 750 vehicles/day. Figure 9 shows warrants for a 4x6-ft single-box culvert on a 2.5:1 slope. Similar figures for other configurations are available in Ross and others (6).

Parallel-Drainage Structures

Simulation Studies

Design of a traffic-safe parallel-drainage structure not only involves the culvert itself but the adjoining slopes as well. In fact, the slopes can in many

Figure 8. Sequential photographs, test 4.



cases be a greater hazard than the culvert structure. Studies of median crossover geometry pointed to the need for relatively flat slopes to minimize vehicle rollover (2,3). To gain further insight, HVOSM was used to examine the behavior of a vehicle traversing various driveway conditions. Parameters investigated included departure angle, departure speed, and the path of vehicle encroachment; the side slopes of both the ditch and the driveway; the type of transition zone between the two slopes; depth of the ditch; and vehicle size. These parameters are illustrated in the definition sketch of Figure 10.

The following is the range of each parameter evaluated:

1. Departure angle--15° and head on;
2. Departure speed--30, 40, 50, and 60 mph (48.3, 64.4, 80.5, and 96.6 km/h);
3. Path--15° angled path across transition (path 1), 15° angled path across ditch bottom (path 2), and head-on path into driveway slope (path 3);
4. Roadway slope--4:1 and 6:1;
5. Driveway slope--4:1, 5:1, and 6:1;
6. Transition type--abrupt and rounded;
7. Ditch depth--2 and 3 ft (0.61 and 0.92 m); and
8. Vehicle size--2250 and 4500 lb (1022 and 2044 kg).

A total of 68 computer runs were made to evaluate the various parameters.

Preliminary Tests

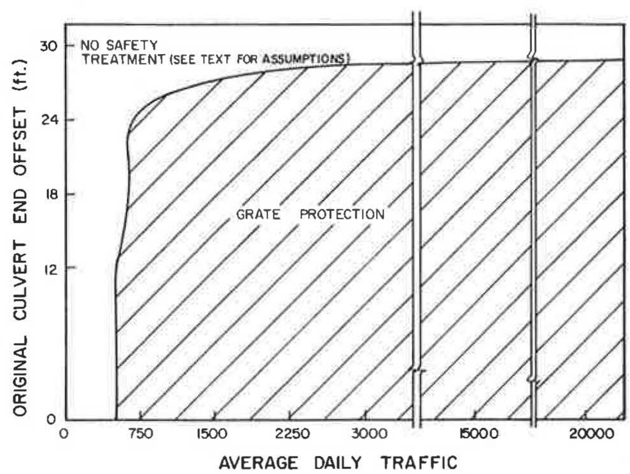
Ten full-scale vehicle tests were conducted to (a) evaluate vehicle response as a function of the driveway slope and (b) develop a tentative safety

Table 3. Cross member deflections of box culvert grating strength tests.

Test No.	Pipe ID[a] (in)	Grate Member[b]	Deflection (in) Vertical	Deflection (in) Horizontal
1	5	First	0.0	0.0
		Second	0.0	0.0
		Third	-0.94	0.4
2	4	First	-0.13	0.0
		Second	-0.50	0.3
		Third	-3.00	1.9
3	3.5	First	-1.75	2.9
		Second	-4.75	3.1
		Third	-4.13	1.4
4[c]	3	First	-0.75	1.5
		Second	+0.25	1.9
		Third	+0.13	4.8

Note: 1 in = 2.54 cm.
[a]Schedule-40 steel pipe.
[b]Grate members spaced on 30-in (76.2-cm) centers.
[c]Midspan vertical supports used on each grate.

Figure 9. Warrants for safety treatment of a 4x6-ft single box culvert on a 2.5:1 slope.



treatment for parallel-drainage structures. The test vehicles were 1974 and 1975 Chevrolet Vegas weighing approximately 2250 lb. In each test the vehicle was towed to the test site along a guidance cable, released, and then allowed to traverse the test area in a free-wheel (no-steer-input) no-braking mode. A summary of the 10 tests is given in Table 4 (tests 1-1 through 7-6). Tests 1-1 through 5-1 were designed to evaluate the relative hazard of the driveway slope. An earth berm was constructed to simulate the driveway. The berm for tests 1-1 through 1-4 had a 3.8:1 slope, was approximately 3 ft (0.92 m) high, and was approximately 20 ft (6.1 m) wide at the top. Sequential photographs of test 1-4 are shown in Figure 11.

After test 1-4, the berm slopes were flattened to the dimensions shown on the upper part of Figure 12. In this case the slope on the approach side was 6.7:1. It was obvious from test 1-3 that an automobile could traverse the 6.7:1 slope at speeds in excess of 40 mph (64.4 km/h) without rolling over. Hence, test 5-1 was conducted at 50 mph (80.5 km/h) and had the automobile approach from a head-on path. Although the vehicle was airborne for approximately 75 ft (22.9 m), it remained upright with no appreciable pitching.

The next series of tests (7-1 through 7-6) were conducted to determine if safety treatment of the

Figure 10. Definition sketch.

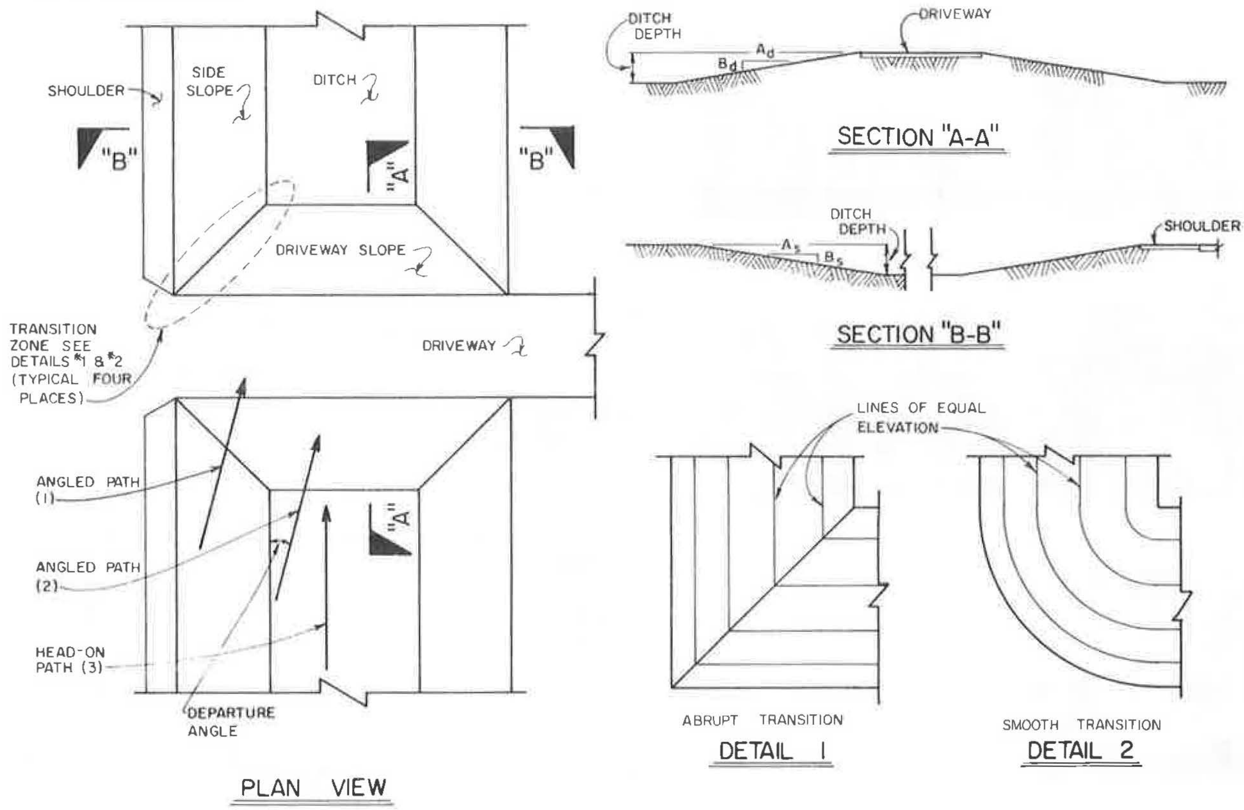


Table 4. Summary of full-scale test results.

Test No.	Vehicle Speed (mph)	Vehicle Path[a]	Driveway Slope	Ditch Slope	Culvert Configuration	Results
1-1	30	3	3.8:1	NA	No culvert	Satisfactory, no rollover
1-2	35	3	3.8:1	NA	No culvert	Satisfactory, no rollover
1-3	40	3	3.8:1	NA	No culvert	Satisfactory, no rollover
1-4	50	3	3.8:1	NA	No culvert	Unsatisfactory, vehicle pitched over
5-1	50	3	6.7:1	NA	No culvert	Satisfactory, no rollover
7-1	50	3	6.7:1	NA	See Figure 10	Unsatisfactory, vehicle rolled over
7-2	50	3	6.7:1	NA	See Figure 10	Unsatisfactory, vehicle rolled over
7-4	20	3	6.7:1	NA	See Figure 10	Satisfactory, no rollover
7-5	50	3	6.7:1	NA	See Figure 10	Unsatisfactory, vehicle rolled over
7-6	50	3	6.7:1	NA	See Figure 10	Satisfactory, no rollover
9-1	40	2	6.5:1	6.8:1	See Figure 13	Satisfactory, no rollover
9-2	50	2	6.5:1	6.8:1	See Figure 13	Satisfactory, no rollover

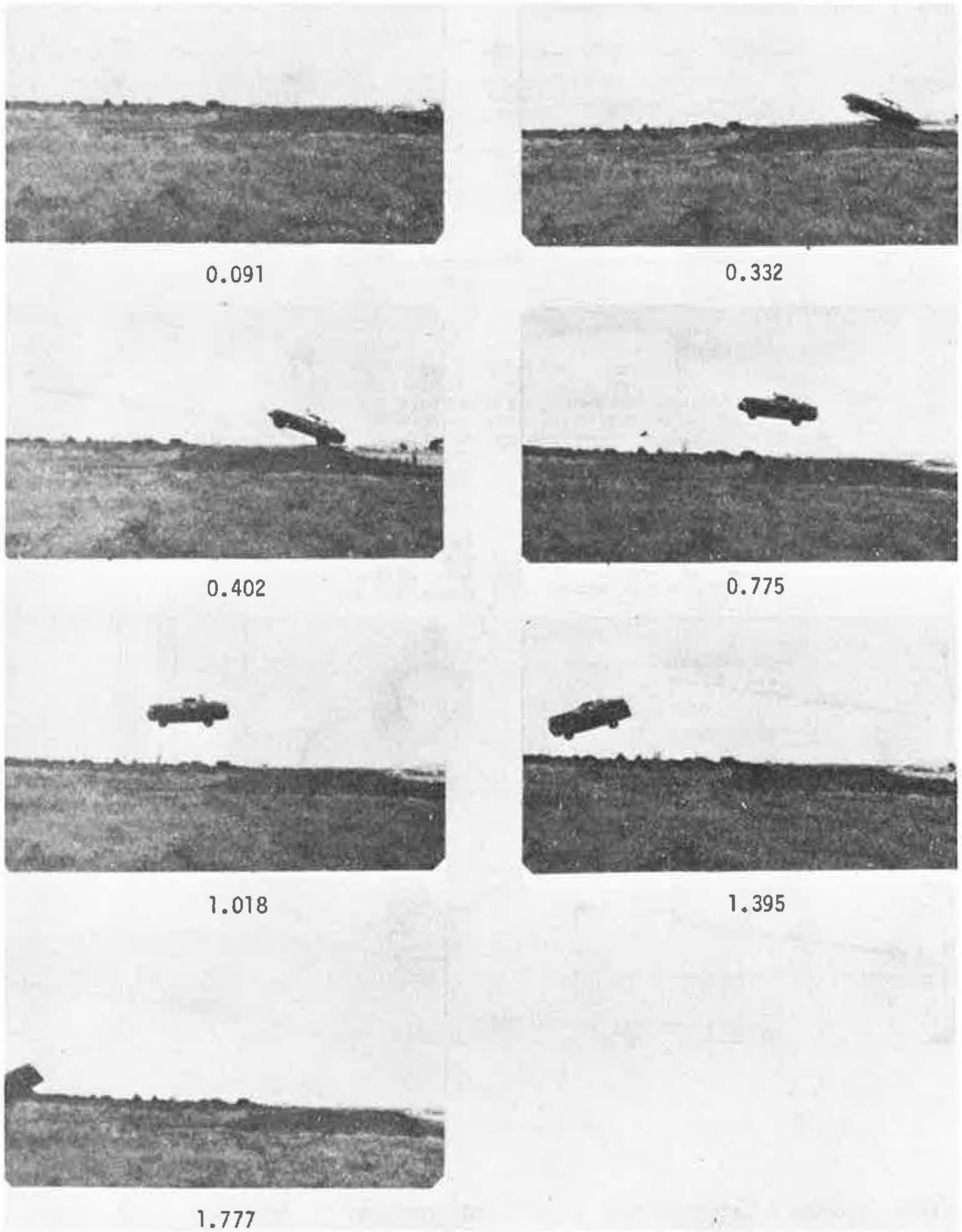
Note: 1 mph = 1.609 km/h; NA = not applicable.
[a]See Figure 1.

culvert end was needed in addition to the sloped end treatment. The 6.7:1 driveway slope was used in each test. It was assumed that a head-on path into the driveway culvert would be as critical as, or more critical than, any other path regarding the culvert itself. Based on this assumption, a 24-in (61.0-cm) diameter corrugated steel-pipe culvert with a sloped end was installed in the earth berm as shown on the upper part of Figure 12. This culvert size was selected because the diameter of most driveway culverts in Texas are equal to or less than 24 in. The vehicle impact point for this series of tests was selected such that the right-side wheels of the test vehicle traversed the center of the culvert end.

Details of the culvert configuration for each of the culvert tests are shown in Figure 12. Test 7-1 was conducted at 50 mph with an open culvert, i.e., no grate members. Photographs of the installation are given in Figure 13 and sequential photographs of the test are given in Figure 14. Large pitch and roll rates occurred after impact with the culvert, and the vehicle rolled over. In test 7-2 a single grate member was placed across the culvert as shown in details 3 and 4 of Figure 12. Very little improvement in vehicle behavior was realized and rollover again occurred.

Analysis of test 7-2 showed that grates spaced approximately on 2-ft (0.61-m) centers were needed to avoid excessive wheel drop and wheel snagging. The next treatment therefore incorporated this feature, as shown in details 5 and 6 of Figure 12. Grate members consisted of 2-lb/ft (2.98-kg/m) steel flanged channel sections. The channel section was chosen because it is widely used as a delineator post by TSDHPT and would therefore be readily avail-

Figure 11. Sequential photographs, test 1-4.



able. The first test on this treatment (test 7-4) was conducted at 20 mph and the results were acceptable. Test 7-5 was conducted at 50 mph and rollover occurred due to structural failure of the grates.

In test 7-6, 2.5-in (6.35-cm) ID standard steel pipe (schedule 40) was used as a grate member. Details 7 through 10 of Figure 12 show how the pipe was attached to the culvert. Although the vehicle was airborne approximately 65 ft (19.8 m), it remained upright and the test was deemed acceptable. The culvert was only slightly damaged.

Prototype Tests

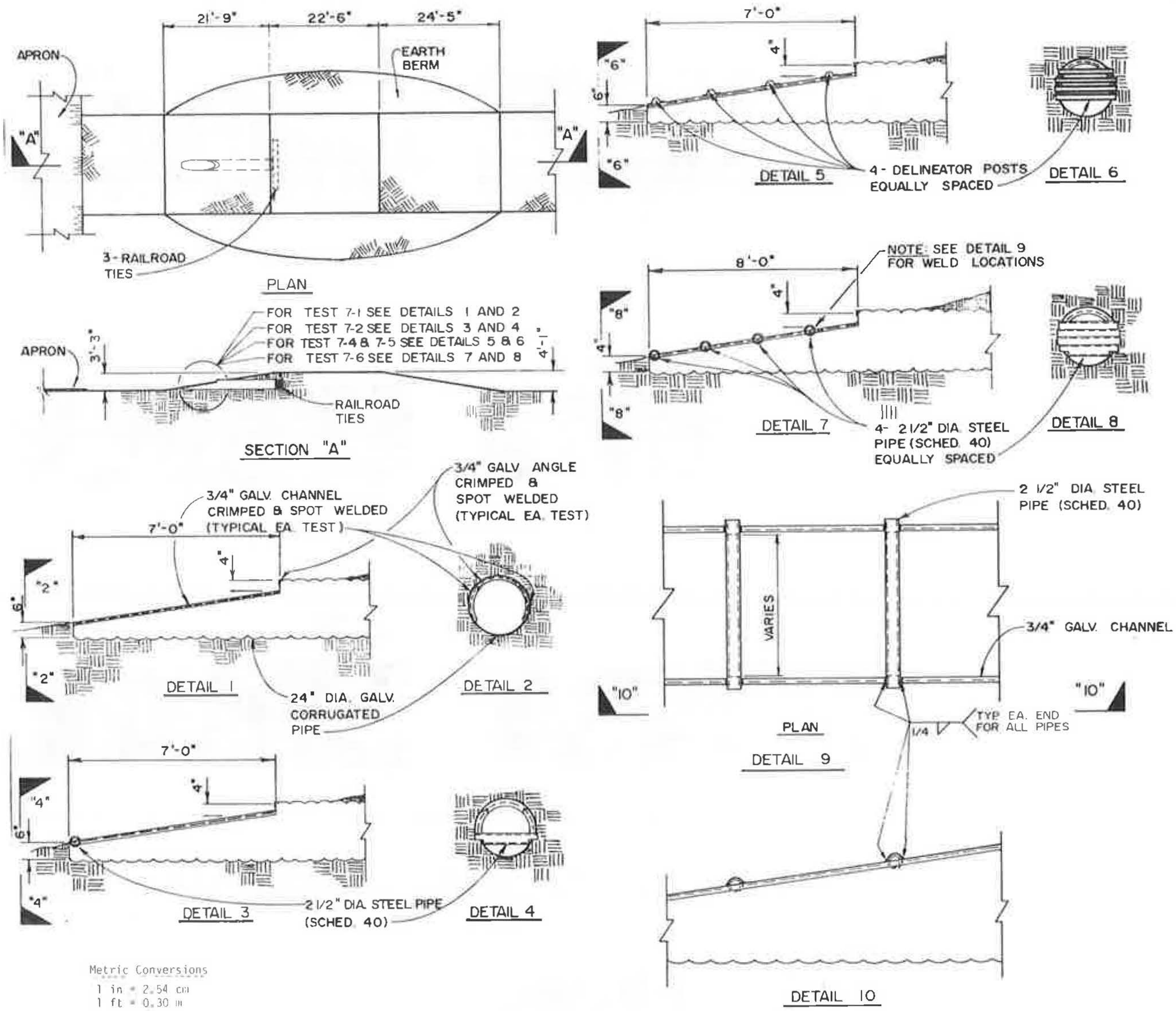
The final two tests (tests 9-1 and 9-2) were selected to verify the tentative conclusions reached as a result of the simulation work and the full-scale slope and culvert testing. A full-scale prototype of a ditch and driveway configuration was constructed as shown in Figure 15 and the photographs of Figure 16. Test 9-1 was conducted at 40 mph (64.4 km/h) and the approach path into the driveway was as shown in Figure 15, such that the left-side wheels crossed the culvert. No adverse vehicle behavior occurred during the test and the results were considered acceptable.

Test 9-2 was identical to test 9-1 except that the speed was increased to 50 mph. Sequential photographs of the test are shown in Figure 17. The vehicle remained upright and sustained only minor damage. The culvert was only slightly damaged and could have been used without repair.

Cost/Benefit Analysis

A cost/benefit analysis was made to develop warrants for safety treatment of parallel-drainage structures and adjoining roadside slopes. The analysis was conducted by assuming that (a) the roadway side slope was 6:1, (b) the roadway had a 12-ft (3.66-m) shoulder, and (c) the centerline of the culvert was 25 ft (7.62 m) from the edge of the travelway.

Figure 12. Berm and culvert details, tests 5-1 through 7-6.



Safety treatment alternatives considered included (a) 1.5:1 driveway slope and no culvert safety treatment (this is considered the untreated condition), (b) 6:1 driveway slope and culvert end cut to match the 6:1 slope, and (c) 6:1 driveway slope, culvert end cut to match slope, and a safety grate treatment as recommended herein.

With the three options above and the assumed roadside geometry, a cost/benefit analysis was conducted. A description of the cost/benefit analysis procedure used is given in Kohutek and Ross (10). Input required to perform the analysis included cost of treatment, accident or societal cost, traffic volume, hazard size and location, discount rate, and severity index of the hazard being evaluated.

Costs of safety treatment of each culvert are given in Table 5. Severity indices and construction costs were estimated by TTI and TSDHPT engineers.

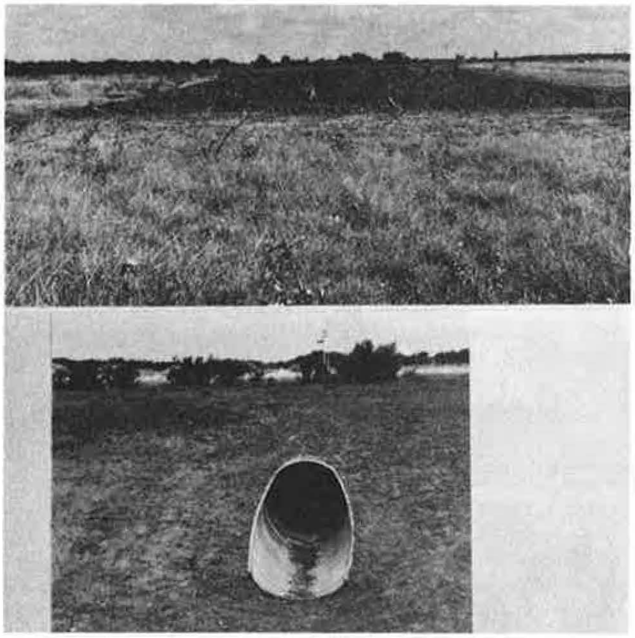
Figure 18 shows the warrants developed for parallel-drainage culverts. Because the warrants shown in this figure were based in part on judgment and the analysis was conducted for only one highway cross section, discretion must be used in their application.

FINDINGS

Cross-Drainage Structures

Based on the computer simulations and the preliminary test program, it was shown that clear openings of at least 30 in (76.2 cm) could easily be traversed at a speed of 20 mph (32.2 km/h). A 36-in (91.4-cm) spacing was easily traversed at 25 mph (40.2 km/h). For clear openings in excess of 30-36 in, it was shown that grates spaced on 30-in centers would provide satisfactory safety treatment. These findings were in fact borne out through six full-scale prototype tests. Tests of a 30-in-diameter corrugated steel-pipe culvert end, cut to match a 5:1 side slope, were successfully conducted. The culvert opening was readily traversed by both a full- and mini-sized automobile at 20 mph. Tests of a relatively large box culvert constructed to match the existing 5:1 side slope also verified that grates spaced on 30-in centers provide a satisfactory safety treatment. Tests of this treatment at 20 mph and 60 mph (96.5 km/h) by both full- and mini-sized automobiles were conducted. It was also shown that

Figure 13. Test installation before test 7-1.



the grates should be extended and anchored at the flow line to avoid any appreciable drop-off at the end of the culvert treatment. In one test, vehicle rollover occurred when the left-side wheels dropped off an 18-in (45.7-cm) opening at the end of the culvert.

Preliminary tests and the prototype tests showed that 3-in (7.6-cm) ID schedule-40 steel-pipe grates were of sufficient strength to support a full-sized automobile for simple-supported spans up to approximately 12 ft (3.7 m). Additional full-scale tests were conducted with a test pit to determine pipe size requirements for larger spans. Results of these tests provided the following guidelines (1 ft = 0.30 m; 1 in = 2.54 cm):

Span Length (ft)	Suggested Standard Schedule-40 Pipe Size ID (in)
Up to 12	3.0
12-16	3.5
16-20	4.0

If midspan vertical supports are used, 3.0-in ID standard schedule-40 pipe can be used for spans up to 20 ft (6.1 m). Other sections that have equivalent strengths could of course be used. Reference may also be made to a Federal Highway Administration

Figure 14. Sequential photographs, test 7-1.

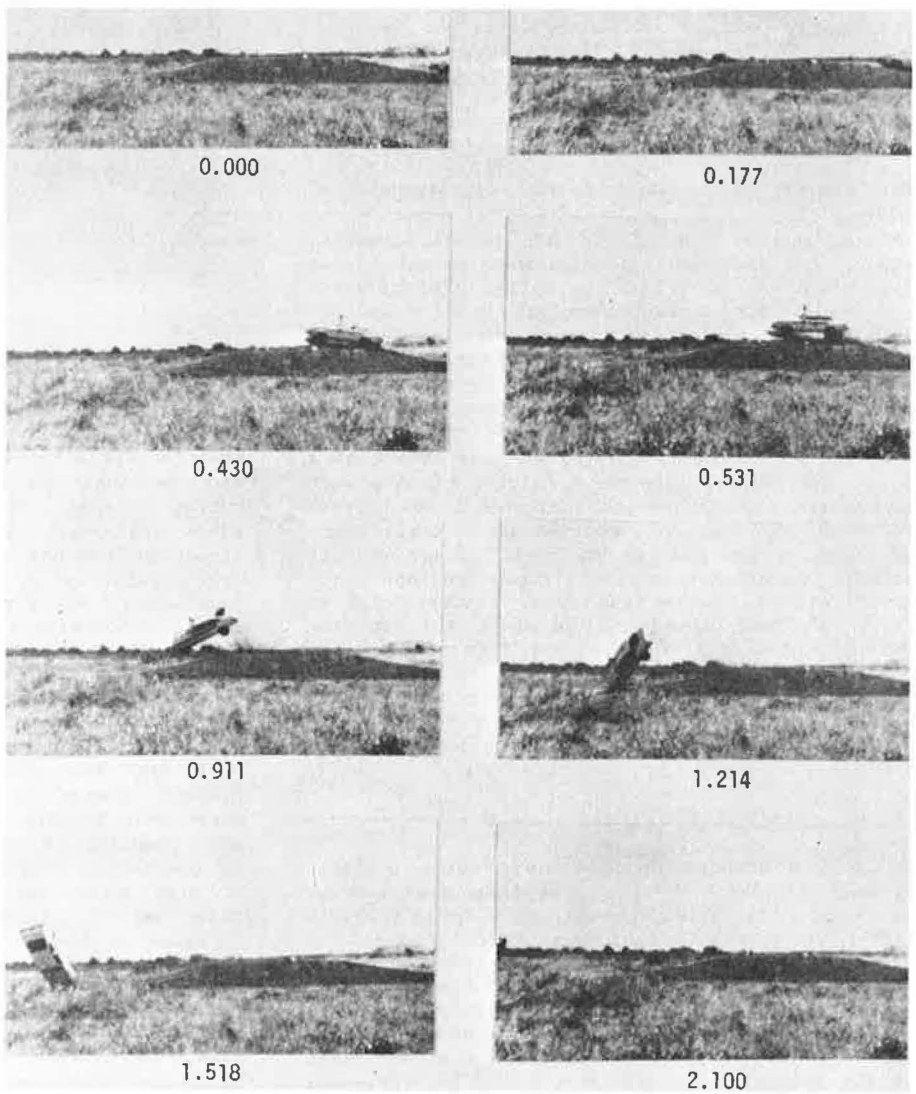


Figure 15. Test site conditions, tests 9-1 and 9-2.

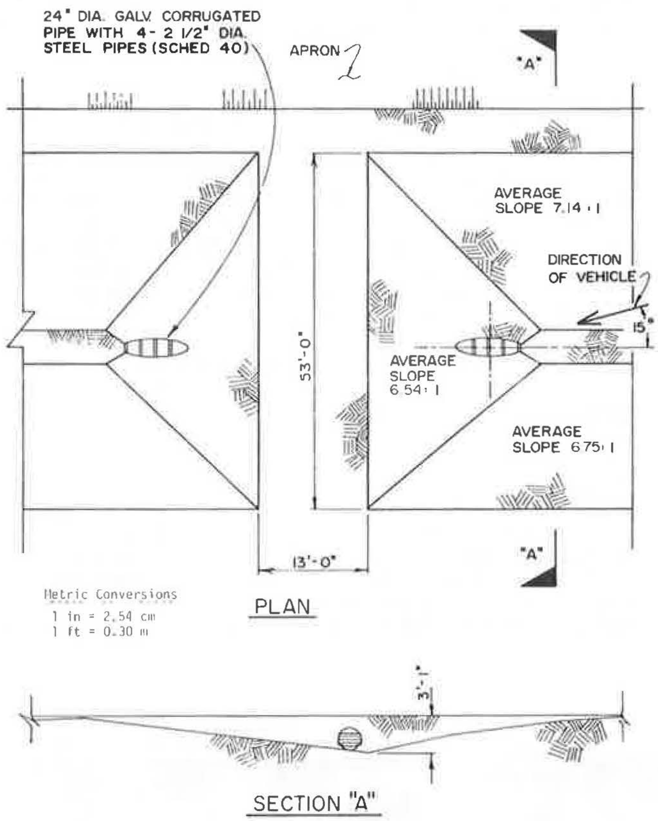


Figure 16. Test site, tests 9-1 and 9-2.

(FHWA) report (4) for strength requirements of grates.

A cost/benefit analysis of six typical culvert, roadway, and side-slope combinations revealed that safety treatment of 36 in or smaller pipe culverts is generally not warranted unless traffic volumes exceed 20 000 vehicles/day. Treatment of larger box culverts is generally warranted for traffic volumes greater than approximately 750 vehicles/day. More specific guidelines for safety treatment of culverts are available in Ross and others (6).

Results of the study to evaluate the effect of a ramp at the leading edge of a culvert opening were inconclusive. HVOSM results indicated that appreciable wheel hop could be achieved by a small ramp, thus enabling the vehicle to clear larger culvert openings. An attempt to verify these findings via a full-scale test program was made. However, due in part to the test procedure, the tests did not provide sufficient data to reach any firm conclusions. To minimize damage to test vehicles, the area behind the ramp was not excavated and, as a consequence, the total wheel drop that would have occurred otherwise was unobtainable. Further evaluation and testing of ramp treatments appear warranted.

Parallel-Drainage Structures

Based on the computer simulations and the preliminary test program, it was shown that the driveway slope should be 6:1 or flatter to avoid vehicle rollover for speeds up to 50 mph (80.5 km/h). The computer simulations indicated that the ditch side slope should also be 6:1 or flatter. Even at these relatively flat slopes, a vehicle traveling at 50 mph will become airborne for approximately 65 ft (19.8 m). The computer simulations also indicated that the potential for rollover could be minimized by a smooth rounded transition zone between the ditch side slope and the driveway slope.

Preliminary tests were conducted on various degrees of safety treatment of the culvert end. The vehicle approached the culvert head on in each test, and the right-side wheels crossed the center of the culvert opening. The baseline test involved an open 24-in (61.0-cm) diameter corrugated steel pipe sloped at the end to match the 6:1 driveway slope. Considerable wheel drop occurred, especially the rear wheel, which caused large vertical and longitudinal forces on the vehicle and subsequently produced rollover. The initial treatment involved a single grate member placed at the end of the culvert. This provided little improvement. Results indicated that grates would have to be placed approximately every 2 ft (0.61 m) to prevent significant wheel drop. The next two tests evaluated steel flanged channel grate members on 2-ft centers. Structural failure of the grates during the 50-mph test resulted in vehicle rollover. The final test in the series involved 2.5-in (6.4-cm) ID schedule-40 pipe grates on 2-ft centers. The test vehicle traversed the treatment at 50 mph without rollover.

Based on the preliminary studies, a prototype of a typical ditch, driveway, and culvert configuration was constructed and tested. Slopes of the ditch and the driveway were approximately 6:1 and the culvert end was safety treated with 2.5-in ID schedule-40 pipe. Tests at 40 mph (64.4 km/h) and 50 mph verified the tentative conclusions reached in the pre-

Figure 17. Sequential photographs, test 9-2.

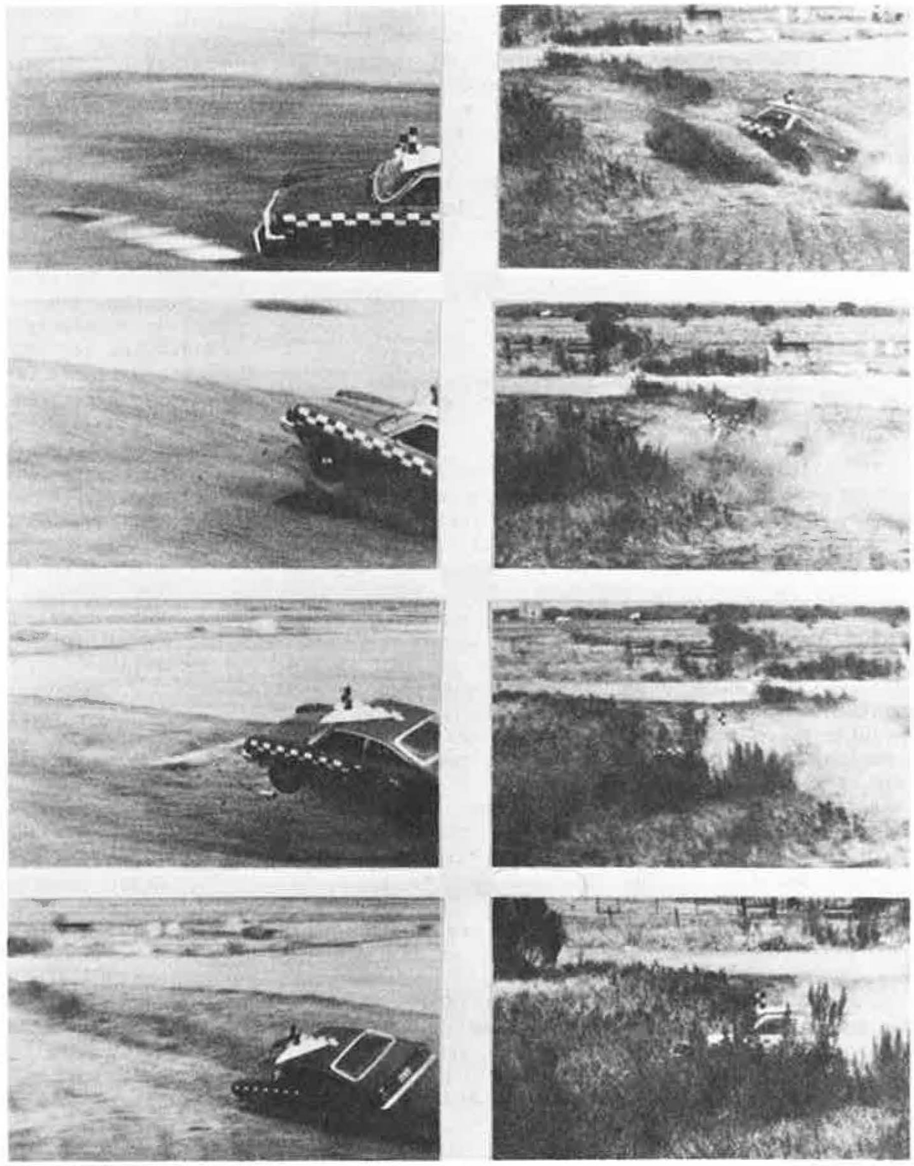


Table 5. Incremental cost of treatments.

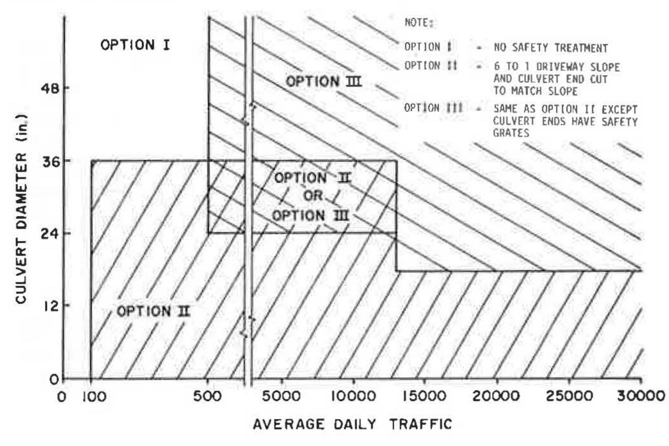
Culvert Diameter (in)	Cost to Upgrade from Option I to II ($)	Cost to Upgrade from Option II to III	
		Construction ($)	Maintenance ($/year)
18	375	225	150
24	378	300	150
36	475	600	150
48	835	900	150

Note: 1 in = 2.54 cm.

liminary studies. Results of the 12 full-scale tests are summarized in Table 4.

Analysis of the crash tests and the computer simulations showed that the dynamic wheel load on a driveway grate member is about 10 000 lbf (44 480 N) when impacted by a 4500-lb (2043-kg) automobile at 50 mph, assuming the culvert is on a 6:1 slope. It is therefore suggested that a 10 000-lbf concentrated load applied at midspan be used in designing a driveway cross member, its attachment to the culvert and/or riprap, and any reinforcing that may be necessary to the culvert and/or riprap. It is noted

Figure 18. Warrants for safety treatment of parallel-drainage culverts.



that the 2.5-in schedule-40 steel pipe used in the test program, while structurally adequate for a 2250-lb (1022-kg) automobile and a 24-in-diameter culvert, would probably not have supported a 4500-lb automobile. Calculations show that a 3-in (7.6-cm) ID schedule-40 pipe would have been needed for the larger automobile.

Warrants for recommended safety treatments of parallel drainage culverts were developed by using cost/benefit techniques, and they are shown in Figure 18.

CONCLUSIONS

Cross-Drainage Structures

The conclusions for traffic-safe cross-drainage structures are as follows:

1. All culvert ends not shielded by a traffic barrier should be made to match the existing side slope if they terminate within the clear zone. Protrusions of the culvert and adjoining wing walls and head wall above the terrain in excess of 3-4 in (7.6-10.3 cm) should be avoided.
2. Round culverts with diameters of 30 in (76.2 cm) or less need no end treatment other than what was mentioned in 1 above. Elliptic or oval-shaped culverts with major axes 30 in or less need no end treatment other than as mentioned in 1 above. Rectangular-shaped culverts with a horizontal clear distance 30 in or less need no end treatment other than as mentioned in 1 above.
3. Culverts that have dimensions greater than those given in 2 above can be made traffic-safe by grate members placed on 30-in centers that are oriented parallel to the flow and in the plane of the surface of the side slope.
4. Grate members should extend to and be anchored at the flow line. Drop-offs at the end of the culvert should be avoided.
5. Necessary grate member sizes will depend on the span of the grates, the manner in which the grates are supported, and the design vehicle weight. To support a full-sized automobile, the following sizes or their equivalent are adequate (1 ft = 0.3 m, 1 in = 2.54 cm):

Span Length (ft)	Suggested Standard Schedule-40 Pipe Size ID (in)
Up to 12	3.0
12-16	3.5
16-20	4.0

A 3.0-in ID standard schedule-40 pipe can be used for spans up to 20 ft (6.1 m) if a midspan vertical support is used.

6. Safety treatment of large cross-drainage structures is warranted on most highways that have traffic volumes in excess of 750 vehicles/day. Guidelines for application of the cross-drainage culvert safety treatments are available in Ross and others (6).

Parallel-Drainage Structures

The conclusions for traffic-safe parallel-drainage structures (for driveways, median crossovers, ramps, etc.) are as follows:

1. The roadway side slope (or ditch slope) in the vicinity of the driveway slope should be 6:1 or flatter.
2. The driveway slope should be 6:1 or flatter.
3. The transition area between the roadway side slope and the driveway slope should be rounded or smoothed as opposed to an abrupt transition.
4. Safety treatment of the culvert opening should include an end section cut to match the driveway slope with cross members (grates) spaced approximately every 24 in (61.0 cm) perpendicular to the direction of flow.
5. The cross members should be designed to support a concentrated wheel load of approximately 10 000 lbf (44 480 N) applied at midspan.
6. Warrants for safety treatment of parallel-drainage structures are shown in Figure 18.

REFERENCES

1. Traffic-Safe and Hydraulically Efficient Drainage Practice. NCHRP, Synthesis of Highway Practice 3, 1969.
2. H.E. Ross, Jr., and E.R. Post. Criteria for the Design of Safe Sloping Culvert Grates. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 140-3, Aug. 1971.
3. N.J. DeLeys. Safety Aspects of Roadside Cross Section Design. FHWA, Rept. FHWA-RD-75-41, Feb. 1975.
4. C.A. Ballinger and R.H. Gade. Evaluation of the Structural Behavior of Typical Highway Inlet Grates, with Recommended Structural Design Criteria. FHWA, Rept. FHWA-RD-73-90, Dec. 1973.
5. Improving Safety of Drainage Facilities. Texas State Department of Highways and Public Transportation, Austin, Administrative Circular 8-79, Jan. 1979.
6. H.E. Ross, Jr., T.J. Hirsch, B. Jackson, Jr., and D. Sicking. Safety Treatment of Roadside Cross-Drainage Structures. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 280-1, March 1981.
7. H.E. Ross, Jr., T.J. Hirsch, and D. Sicking. Safety Treatment of Roadside Parallel-Drainage Structures. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 280-2F, June 1981.
8. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. TRB, Transportation Research Circular 191, Feb. 1978.
9. D.J. Segal. Highway-Vehicle-Object-Simulation Model--1976. FHWA, Rept. FHWA-RD-76-164, Feb. 1976.
10. T.L. Kohutek and H.E. Ross, Jr. Safety Treatment of Roadside Culverts on Low Volume Roads. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 225-1, March 1978.

Crash Tests of Box-Beam Upgradings for Discontinuous-Panel Bridge Railing

JAMES E. BRYDEN AND KENNETH C. HAHN

A 6- by 6- by 3/16-in box-beam guiderail upgrading for discontinuous-panel bridge railings was tested to develop a system for safe redirection of 4500-lb cars impacting at 60 mph and 25°. After several design changes and seven crash tests, the system consists of a single box beam blocked out from the existing railing on the bridge and a double box-beam approach guiderail that has the upper rail blocked out from the S3x5.7 posts. This system will provide a safe, economical, and relatively easy-to-maintain upgrading for discontinuous-panel bridge rails.

Through the early 1960s, New York State's standard bridge rail consisted of short panels (up to about 20 ft long) that did not have connections between adjacent panels. These railings, designed to meet American Association of State Highway Officials (AASHO) specifications (1), included three or four thin-wall steel-tube rails supported by three posts connected to the bridge deck by heavy anchor plates and bolts. However, impact tests conducted in the mid-1960s (2) resulted in high decelerations and dangerous vehicle reactions. When subjected to a severe impact, a railing panel could deflect, which allowed the vehicle to snag on the end of the adjacent panel. The highest 50-ms average deceleration recorded was 22 g. As a result of that research, discontinuous-panel bridge railings were eliminated from state design standards for future installations.

Although these railings have not been erected for more than 13 years, many remain in service throughout the state. The Structures Design and Construction Division of the New York State Department of Transportation is now upgrading structures where these rails were installed. Because complete statewide replacement of these railing systems is not economically feasible, other less-costly solutions were needed. Efforts thus were directed toward modifications to improve the existing railings.

One suggested design to upgrade performance was to attach a continuous 6- by 6- by 3/16-in box-beam guiderail to the existing bridge rail and splice it to the approach guiderail at either end of the bridge. Blocked out from the face of the bridge rail, the box beam is intended to limit deflections of the existing rail by distributing the load over more than one panel and to equalize deflections across the joints, thus preventing vehicles from snagging on the ends of panels. Such a system would make use of existing approach box-beam guiderail without any special transitions or anchorages. It would require a minimum of new hardware, which is a substantial benefit from the standpoint of both initial cost and maintenance inventory requirements. More important, a successful upgrading system would save the cost of replacing much of the discontinuous-panel bridge rail now in service in New York State.

METHODOLOGY AND DESCRIPTION OF BARRIERS

This study consisted of seven full-scale crash tests to determine the performance of box-beam guiderail upgradings for discontinuous-panel bridge rail. [More information about these tests is presented elsewhere (3).] Testing details were taken from National Cooperative Highway Research Program (NCHRP) Report 153 (4) and its successor, Transportation Research Board (TRB) Research Circular 191 (5). All seven tests were standard strength tests and used target impact conditions of 4500-lb vehicles at 60 mph and 25°. Because of test site limitations, the inclusion of 15° tests with 2250-lb vehicles would have required construction of a second simulated bridge deck at considerable additional cost and long delays in the test program. Based on the excellent results achieved in the large-vehicle tests, it was decided that the delay and cost of performing the 15°, 2250-lb tests were not justified. Two additional factors supported that decision. First, about 75 installations of this upgrading system have been completed, and no unsatisfactory collisions by small vehicles have been recorded. Second, the final configurations of the railing system provided a 12-in blockout from the bridge-rail posts and a dual rail in the transition to eliminate any potential for snagging or wheel entrapments of small vehicles. Sufficient clearance from the posts and an absence of vertical projections or rail faces that may be climbed by the front wheel have both been shown to be important to prevent wheel snag and high roll potential (6,7).

The box-beam upgrading consists of a 6- by 6- by 3/16-in box-beam guiderail mounted in front of the existing railing at a height of 27 in above the pavement. Tubular steel blockouts (6x8x0.25 in), which vary in depth from 6.75 to 11 in, were used at each bridge-rail post. A 3-ft-deep, 3-ft-wide concrete footing was used to anchor the bridge rail for these tests, which protruded above grade 10 in for the first test and 6 in for the others, to simulate a curb and safety walk.

Because field experience with discontinuous bridge rail had shown the anchor bolt and deck details on actual bridges to be adequate for severe impacts, it was not necessary to duplicate an actual deck for these tests. Instead, an asphalt pavement was placed adjacent to the curb to simulate the deck. A firmly anchored timber curb, which was the same height as the concrete curb, was used to simulate the granite curb normally used on bridge approaches. The approach guiderail was a 6- by 6- by 3/16-in box beam mounted 30 in high on S3x5.7 posts driven into compacted granular fill on 6-ft centers. The last 18-ft section of the box beam upstream of the bridge was tapered down to 27 in in height to meet the upgrading elevation.

The first design, shown in Figure 1, was impacted on the bridge. Upstream of the bridge the 6-ft post spacing was closed to 3 ft (8 spaces) and 2 ft (4 spaces), and the last S3x5.7 post was 4 ft from the first bridge-rail post. On the bridge the box beam was connected to each 5x5x0.75x8-in-long support angle with one 3/4- by 8-in long bolt. A support angle was welded to each 11-in-high blockout, which was then bolted to the bridge-rail post by using four 3/4- by 7-in long bolts. Two 4x8x0.625-in backup plates were used at each post. The approach guiderail was a standard box beam that had standard post-to-rail connections: 3/8- by 7-in long A325 bolts. Post-to-rail connections were provided every 6 ft, starting 6 ft from the bridge, and the remaining posts in the transition were unconnected back-up posts.

A second design with a modified blockout and support-angle configuration is also shown in Figure 1. It was tested three times--twice with impact on the bridge and once with impact on the approach rail. The first two of these tests were standard strength tests, and a low impact speed on the first required retest. The third test, which impacted 10 ft upstream of the first bridge-rail post, was a standard strength test of the transition.

Following unsatisfactory performance in the transition test, the design was revised as shown in Figure 1. Five W6x8.5 posts with 8-in-wide, 6-in-high blockouts were set on 3-ft centers upstream of the bridge. The 6- by 6- by 3/16-in box-beam guiderail was bolted to the blockouts by using two 3/4- by 7-in long A325 carriage bolts. The blockouts were connected to the W6x8.5 posts by using two 3/4- by 1-1/2-in long A325 bolts. Because of a snag that occurred in the first transition test, the support angle was removed from the first bridge-rail blockout and replaced by two 3/4- by 7-in-long A307 carriage bolts and a 0.75-in spacer plate. All of the remaining blockouts on the bridge rail remained unchanged from the previous tests.

After the successful performance of this design with impact at the center of the W6x8.5 post configuration, the system was impacted upstream of the first W6x8.5 post to determine the redirective characteristics of the secondary transition from light to heavy posts. Because that transition performed poorly, a third and final transition design was prepared for the final test. That design, shown in Figure 2, includes a second 6- by 6- by 3/16-in box beam installed below the primary rail. The latter is blocked out from the bridge posts and the S3x5.7 approach posts for the entire length of the second rail. The second rail is connected to the posts by using standard guiderail connections, and the primary rail is fastened to the 6x8x0.25-in blockouts by using one 3/4- by 7-in-long A307 carriage bolt. The blockouts are connected to the posts by using one 5/16- by 1-1/2-in-long A307 bolt. Upstream of the beginning of the lower rail, the primary rail is mounted by using the standard guiderail connection.

Seven full-scale crash tests of the box-beam upgrading system are summarized in Table 1. For all seven tests, target impact conditions were 4500 lb,

Figure 1. Details of bridge-rail upgrading and guiderail evaluated in tests 19-21B.

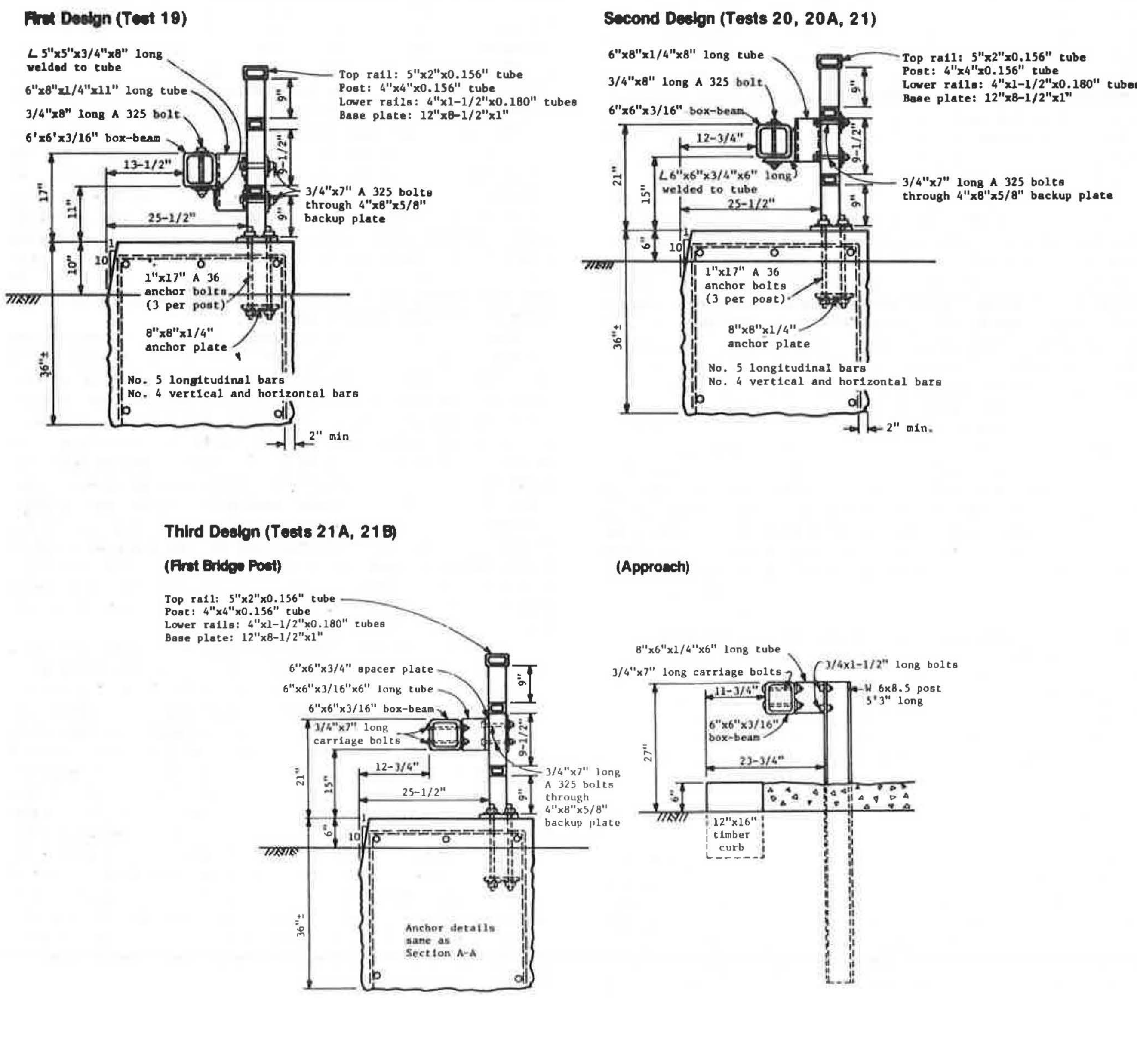
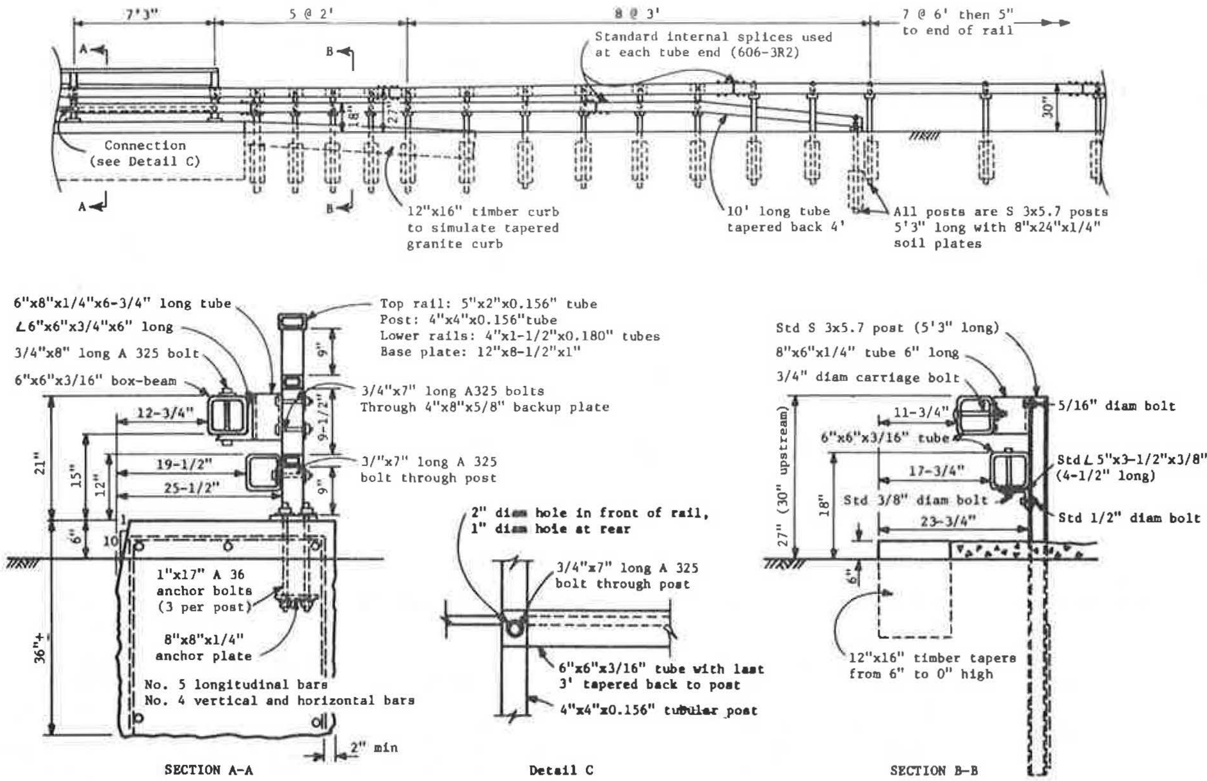


Figure 2. Final design (test 28).



60 mph, and 25°, although actual impact conditions varied somewhat.

IMPACTS ON BRIDGE

For the first test (test 19), a 4010-lb sedan impacted the upgrading at 48.7 mph and 25°, 5 ft downstream from the first bridge-rail post. Impact occurred on the right front wheel and fender. The car was in contact with the 10-in-high curb for 22 ft and the rail assembly for 7 ft and had a maximum dynamic barrier deflection of 0.1 ft. The car traveled about 125 ft along an exit trajectory of 11° before stopping. The highest 50-ms longitudinal deceleration was 2.4 g, but the lateral deceleration was lost due to equipment malfunction.

Vehicle damage was limited to the front bumper, fender, hood, right-side front door, and the right front tire and wheel. There was no permanent rail deflection and no structural damage to either the curb or the rail. Only minor scrapes and paint marks were observed on the rail. Vehicle redirection was accomplished primarily by impact of the wheel and front frame assembly on the curb. Inspection of the crashed car showed that sheet-metal damage, which occurred during contact with the rail, was superficial and none was driven back into the structural members.

Several design changes were made before the next test. As described previously, the blockout and support-angle configurations and sizes were changed, and the 10-in curb height was reduced to 6 in. The latter is more representative of existing installations (where resurfacing has resulted in a similar height reduction) and provides a more severe test of the railing because less of the impact is absorbed by the lowered curb.

For the second test (test 20), a 4540-lb station wagon impacted the upgrading at 48.7 mph and 27°, 2 ft downstream of the first bridge-rail post. Impact was on the right front fender and wheel. The car was in contact with the 6-in curb for 30 ft and with the rail for 18 ft. Maximum dynamic deflection was 1.1 ft. On impact, the car was redirected smoothly and did not begin to roll or pitch until it was exiting the rail. After leaving the curb, the car traveled about 100 ft along a 12° exit trajectory before stopping. The highest 50-ms decelerations were 7.0 g longitudinal and 4.2 g lateral. Vehicle damage included a bent bumper, grill, right-side sheet metal, sprung hood, broken radiator, and flattened right-side tires. Two sections of the box beam were bent and the first bridge-rail section was deflected back 0.4 ft at the top because all three posts separated from their base plates at the welds. The blockouts on the first and third bridge-rail posts were bent and slightly deformed, and the one on the second post was twisted and partly crushed. Maximum permanent deflection was 0.5 ft.

Because impact speed in test 20 was significantly below 60 mph, it was repeated. For test 20A, a 4420-lb sedan impacted the upgrading at 56.8 mph and 25°, 5 ft downstream of the first bridge-rail post. Impact was on the right front bumper, fender, and wheel. The car was in contact with the curb for 20 ft and the rail for 12 ft. Maximum dynamic barrier deflection was 0.5 ft. After leaving the barrier, the vehicle traveled along a 12° exit trajectory for about 125 ft. The highest 50-ms decelerations were 8.7 g longitudinal and 3.8 g lateral.

Vehicle damage was similar to that incurred in the two previous tests: bent bumper, grill, and right-side sheet metal; flattened right-side tires; and a sprung hood. Two box-beam sections were damaged and the first three bridge-rail posts were deflected back 2.5 in because the 1-in-thick base

Table 1. Results of full-scale crash tests.

Item	Test 19: Single Rail on 11-in Blockouts	Test 20: Single Rail on 6.75-in Blockouts	Test 20A: Single Rail on 6.75-in Blockouts	Test 21: Single Rail on S3x5.7 Posts	Test 21A: Single Rail on W6x8.5 Posts	Test 21B: Single Rail on W6x8.5 and S3x5.7 Posts	Test 28: Double Rail on S3x5.7 Posts
Point of impact	5 ft onto bridge	2 ft onto bridge	5 ft onto bridge	10 ft before bridge	10 ft before bridge	10 ft before first W6x8.5	10 ft before bridge
Vehicle weight (lb)	4010	4540	4420	4500	4540	4500	4700
Vehicle speed (mph)	48.7	48.7	56.8	60.9	58.8	55.0	56.8
Impact angle (°)	25	27	27	25	25	25	25
Exit angle (°)	11	12	12	12	6	3	10
Maximum roll (°)	-9	-10	-5	-14	-5	-18	+2
Maximum pitch (°)	+11	+5	+3	+10	0	+8	+3
Maximum yaw (°)	+10	0	-6	-10	-6	-45	0
Contact distance[a] (ft)	22/7	30/18	20/12	29/22	29/24	13[b]	20/20
Contact time (ms)	389	304	214	476	340	170	260
Deflection (ft)							
Dynamic	0.1	1.1	0.5	2.0	0.5	2.0	1.5
Permanent	0.0	0.5	0.3	1.3	0.4	1.3	0.8
Deceleration (*g*)							
50-ms avg							
Longitudinal	2.4	7.0	8.7	NA	NA	7.0	6.0
Lateral	NA	4.2	3.8	NA	NA	5.6	9.0
Maximum peak							
Longitudinal	10.0	21.0	21.0	NA	NA	26.0	10.0
Lateral	NA	7.8	7.4	NA	NA	16.9	14.1
Avg continuous							
Longitudinal	0.4	3.0	3.8	NA	NA	5.0	2.5
Lateral	NA	1.3	0.9	NA	NA	2.0	3.9
Vehicle	1974 Matador sedan	1973 Plymouth wagon	1970 Dodge sedan	1968 Buick sedan	1968 Dodge sedan	1970 Mercury sedan	1969 Cadillac sedan
Damage							
TAD	RFQ-4	RFQ-4	RFQ-6	RFQ-7	RFQ-4	RFQ-6	RFQ-5
SAE	01RYEW6	01RDEW9	01RDEW9	01RDEW9	01RDEW9	01RDAW9	01RYAW6
Results and comments	11-in blockouts and 10-in curb; good redirection, speed too low	Same as test 19 with modified blockouts and lower curb; good redirection, speed too low	Same as test 20 at higher speed; good redirection, good decelerations	Transition test on light-post approach rail; vehicle snagged on first rail post	Transition test on heavy-post approach rail; good redirection even through transition	Transition test on heavy- and light-post approach rail; vehicle snagged on first two W6x8.5 posts	Transition test on light-post approach rail; good redirection, good deceleration

Note: NA = not available, TAD = Traffic Accident Data Project, and SAE = Society of Automotive Engineers.

[a] First distance is on curb, second on rail. [b] No curb, rail only.

plates were bowed upward. The first four blockouts were bent from 0.25 to 0.75 in and the maximum permanent barrier deflection at the face of the box beam was 0.3 ft.

Based on this test, it appears that the box-beam upgrading has adequate strength to withstand standard strength test impacts (4500 lb, 60 mph, and 25°) on the bridge rail.

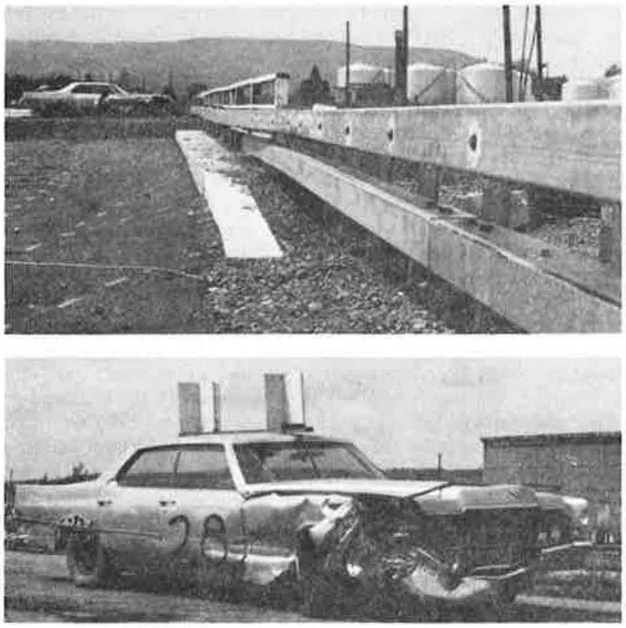
TRANSITION TESTS

The guiderail approach transition was tested next. In the first of these tests (test 21), a 4500-lb car impacted at 60.9 mph and 25°, 10 ft upstream of the first bridge-rail post. Impact was on the right front fender, bumper, and wheel. The car was in contact with the curb for 29 ft and the rail for 22 ft. The maximum dynamic deflection for both the guiderail and the upgrading was 2.0 ft at the first bridge-rail post. Vehicle redirection was smooth until 5 ft after impact when the 3/4-in vertical bolt at the first blockout broke, which allowed the box beam to rise as the car rolled -14°. As the front of the vehicle left the upgrading, the right rear wheel caught the first bridge-rail post and 0.75-in support angle and spun out to the left. Maximum permanent rail deflection was 1.3 ft at the first bridge-rail post. After losing contact with the barrier, the car traveled along a 12° trajectory about 100 ft more, spinning sharply to the right because of severe damage to the right front suspension and sheet metal. Decelerations were not available because of equipment malfunction, but this loss of data is not significant here because the snag and poor redirection after leaving the rail make this design unacceptable.

The vehicle suffered extensive sheet-metal and structural damage to the entire front end and right side. The right rear wheel was torn from the frame, and the hood tore loose and broke the windshield but did not penetrate into the passenger compartment. Approach-rail damage included two bent box-beam sections, six S3x5.7 posts bent over from 4 in to nearly flat to the ground, and three posts pushed through the soil 2-4 in. The first bridge-rail section was bent and twisted, and the first two bridge-rail posts failed at the base-plate welds after the plates bowed. The first blockout was crushed and the support angles on the second and third were bent. The maximum permanent deflection of the box-beam guiderail was 1.3 ft about 5 ft upstream of the bridge rail. The maximum permanent deflection, measured to the top of the bridge rail at the first post, was 0.9 ft.

Because of the snagging and subsequent poor redirection experienced in test 21, the approach guiderail system was stiffened, as described previously, by adding heavy posts just upstream of the bridge. For test 21A, a 4540-lb sedan impacted the approach rail at 58.8 mph and 25°, 10 ft upstream of the first bridge-rail post. Impact was on the right front fender and wheel. The car was in contact with the curb for 29 ft and the rail for 24 ft. Maximum dynamic deflection was 0.5 ft on the guiderail, 6 ft upstream of the first bridge-rail post. Vehicle redirection was very smooth and vehicle reactions during impact were very slight. After losing contact with the rail, the car continued another 100 ft

Figure 3. Barrier and vehicle damage resulting from test 28.



along a 6° exit trajectory. Deceleration data were again lost due to equipment malfunction, but the observed vehicle reactions indicate that this was a gentle redirection.

Vehicle damage was moderate and typical. It included bent bumper and grill, a crumpled right front fender, right-side sheet-metal damage, and a flattened right front tire. Damage to the upgrading was also moderate. One section of the 6x6-in box beam was bent, and all five W6x8.5 posts were pushed through the soil from 3 to 6 in, but none were bent. The first three bridge-rail posts were deflected from 0.50 to 3.50 in, and the modified blockout on the first bridge-rail post was crushed 0.25 ft. The maximum permanent deflections were 0.4 ft on the guiderail (about 6 ft upstream of the first bridge-rail post) and 0.3 ft at the top of the bridge rail at the first post.

Based on the previous tests, it appears that both the upgrading and stiffened approach rail have adequate strength to withstand standard impacts and smoothly redirect impacting vehicles with acceptable decelerations. Inclusion of the W6x8.5 posts in the transition design, however, introduces a secondary transition upstream of the bridge where the post type changes from S3x5.7. This transition area was tested next.

For test 21B, impact was to occur upstream of the first W6x8.5 post. It was therefore necessary to locate the rail in front of the existing bridge-rail footing and simulate a bridge rail by stiffening the box beam downstream of the approach rail. The test was performed with neither bridge rail nor footing because those areas were outside the impact zone.

A 4500-lb sedan impacted the guiderail at 55.0 mph and 25°, 10 ft upstream of the first W6x8.5 post, i.e., 24 ft upstream of what should have been the first bridge-rail post. Impact occurred on the right front fender and front bumper. The car was in contact with the rail for 13 ft and had a maximum dynamic deflection of 2.0 ft. Vehicle redirection was quick but smooth for the first 10 ft, but on contacting the first W6x8.5 post, the right front suspension and wheel snagged and the rear of the car spun sharply to the left. On initial impact, the 6x6-in box beam began to slice into the sheet metal of the right front fender and, by the time the car reached the heavy post, the fender was twisted and hooked over the top of the box beam.

Because of this intrusion of the rail into the car, the 6-in blockout on the heavy post was not wide enough to prevent snagging. A second snag, which was less severe than the first, occurred at the second W6x8.5 post, and the vehicle was wrenched free of the box beam. The car slid free of the rail as it yawed to the right. It recontacted the rail and came to rest 48 ft after leaving the rail.

The maximum 50-ms average decelerations were 7.0 g longitudinal and 5.6 g lateral, but deceleration spikes of 26.0 g longitudinal and 16.9 g lateral were observed as the car impacted the W6x8.5 posts. Vehicle damage was severe and extensive. The hood, front end, and right-side sheet metal were crumpled; the engine compartment was deeply penetrated; and the frame was bent. Also, the right front suspension was broken and twisted and the right-side tires flattened. On the barrier, three sections of rail were badly bent--two bent both back and up, six S3x5.7 posts were bent and/or twisted at ground level, and two W6x8.5 posts were bent over and their blockouts crushed. The maximum permanent deflection was 1.3 ft, 5 ft downstream from the impact.

After analysis of the results of the previous six tests, the entire approach-rail segment of the upgrading was redesigned by the Structures Design and Construction Division. This new design added a second 6- by 6- by 3/16-in box-beam rail in the transition to strengthen the rail upstream of the bridge and to prevent contact with the S3x5.7 guiderail posts and the first bridge posts. By doubling the rail strength, it was possible to eliminate the stronger W6x8.5 posts.

For the final test (test 28), the double-rail system was impacted by a 4700-lb car at 56.8 mph and 25°, 10 ft upstream of the first bridge-rail post. Impact occurred on the right front fender and right edge of the front bumper, and the car remained in contact with the rail for 20 ft and had a maximum dynamic deflection of 1.5 ft. The vehicle redirected quickly and smoothly, the transition onto the bridge was without any adverse reaction, and the car exited along a 10° trajectory. Maximum roll was only +2°, maximum pitch was +3°, and there was no yaw until after loss of contact when the right front suspension damage caused the car to turn to the right as it came to a stop some 125 ft after impact. The maximum 50-ms average decelerations were 6.0 g longitudinal and 9.0 g lateral.

Vehicle damage was moderate; the bumper, grill, and right front fender and suspension were crushed and bent and there were dents in the right-side doors and right rear fender. The right front suspension was broken and the tire flattened; the vehicle could not have been driven from the scene. Barrier damage was limited to eight displaced posts (only one was bent 0.25 in) and two bent rail sections at the upper rail splice in the vicinity of impact (one blockout was crushed 2 in and one base plate was bowed 0.50 in), both on the first bridge-rail post. The maximum permanent deflection was 0.8 ft about 7 ft downstream of the impact point. Vehicle and barrier damage resulting from this test is shown in Figure 3.

DISCUSSION AND FINDINGS

The seven tests performed in this study were standard strength tests for longitudinal barriers with target impact conditions of 4500 lb, 60 mph, and 25°. Impact speeds in the first two tests on the bridge were low (49 mph in each test) but, in the third test, the higher speed (57 mph), the very smooth vehicle redirection, and the very moderate rail damage confirm that this upgrading satisfies the standard strength test criteria. Vehicle trajectory hazards were minimal in all three tests and had exit angles between 11° and 12°. Vehicle decelerations (50-ms average) were all below the values specified for 15° impacts.

The first guiderail and bridge-rail transition that used S3x5.7 posts and one box-beam rail performed poorly and had two specific problems. First, the lateral strength of the approach guiderail was significantly less than that of the bridge rail, which resulted in partial pocketing as the vehicle approached the first bridge-rail post. Second, the weak post-to-rail connection on the guiderail, which is designed to fail on impact, permitted the rail to raise more than 2 ft when the vehicle pocketed and decelerated abruptly upstream of the bridge. This led to a failure of the rail connection at the first bridge-rail post by exposing that blockout, which then snagged the vehicle's rear wheel.

To eliminate these problems, the transition was redesigned for the next test. To increase the lateral strength of the guiderail, W6x8.5 posts were added upstream from the bridge. To prevent wheel contact on these heavier posts, 6-in-deep blockouts were added. Two 3/4-in carriage bolts were used to connect the rail to the first bridge post and each of the W6x8.5 posts. The standard strength test on this transition resulted in very good performance and confirmed the adequacy of this design. However, by adding the heavy posts in the transition area, a secondary transition was introduced at the change in post sizes. Post spacing for the first five S3x5.7 posts was reduced to 2 ft in an attempt to equalize the lateral strengths as closely as possible on both sides of this transition point. However, test 21B demonstrated that this design was not adequate. On initial impact, the box-beam rail cut sharply into the vehicle sheet metal, probably aggravated by the added stiffness achieved in the transition zone by adding the extra posts. This penetration of the rail element into the side of the car permitted the front suspension and wheel to intrude behind the rail face. This presented no problem in the area of the S3x5.7 posts, which yielded on impact with the bumper. However, when the vehicle reached the heavy posts, the combined effects of barrier deflection plus intrusion of the rail into the car resulted in a solid impact of the suspension, wheel, and frame assembly on the first two heavy posts, and a violent snag and spin-out occurred.

To eliminate this undesirable performance, the transition was completely redesigned for the final test. The W6x8.5 posts were eliminated, and S3x5.7 posts were used throughout. A second 6- by 6- by 3/16-in box-beam rail was added in the transition zone to increase lateral strength of the guiderail. By doubling the rail face width from 6 to 12 in, penetration of the rail into the car would be reduced, and contact with both the guiderail posts and the first bridge-rail post would be eliminated. Both ends of the lower rail were safely terminated, i.e., flush with a bridge post on the downstream end and tapered behind the posts and down to the ground on the upstream end.

The success of the final design was demonstrated in tests 20A and 28. The vehicle decelerations experienced were comparable with those reported for other tests of very stiff bridge-railing systems and were near or below acceptable decelerations for 15° impacts (5). Vehicle redirection was good, roll angles were low (-5° and +2°), and potential pocketing and snagging points were eliminated by the balanced stiffness of the transitions from one to two tubes and from two tubes to the bridge rail. Vehicle damage was moderate, considering the severity of the impacts, and compared favorably with damage reported in tests of other bridge-rail upgrading systems (6). Although no tests were run with 2250-lb vehicles at 15°, this system appears to be capable of providing smooth redirection for those impacts. The two large-vehicle tests discussed earlier resulted in smooth redirection and low roll angles, and the final design includes no potential snag points or areas to trap a small-vehicle wheel. In addition, about 75 similar upgradings are now in service throughout the state and there have been no known adverse reactions with small vehicles. Both snagging and high roll angles have been problems in tests with small-vehicle impacts at 15° conducted elsewhere (6,7). However, these problems can be attributed to two conditions that were eliminated in this design: (a) insufficient clearance to the posts and a narrow rail face that permitted wheel-post contact and (b) a high curb that could be easily climbed by the front wheel and result in high roll angles. Based on these tests, the following conclusions appear warranted:

1. Performance of the discontinuous-panel bridge rail was raised to current standards by the addition of a single 6- by 6- by 3/16-in box-beam upgrading,
2. Stiffening the approach guiderail with W6x8.5 posts eliminated pocketing at the end of the bridge but created a snag point at the transition from S3x5.7,
3. The double-rail transition design provided smooth vehicle redirection through the transition onto the upgraded bridge rail, and
4. The final upgrading design appears capable of safely redirecting 4500-lb vehicles impacting at 60 mph and 25° at any point on the bridge or approach rails.

ACKNOWLEDGMENT

The work described in this paper was conducted in cooperation with the Federal Highway Administration, U.S. Department of Transportation.

REFERENCES

1. Standard Specifications for Highway Bridges, 7th ed. AASHO, Washington, DC, 1957, pp. 195-198.
2. M.D. Graham and others. New Highway Barriers: The Practical Application of Theoretical Design. HRB, Highway Research Record 174, 1967, pp. 88-183.
3. J.E. Bryden and K.C. Hahn. Crash Tests of Box-Beam Upgradings for Discontinuous-Panel Bridge Railings. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Res. Rept. 92, Sept. 1981.
4. M.E. Bronstad and J.D. Michie. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. NCHRP, Rept. 153, 1974.
5. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. TRB, Transportation Research Circular 191, Feb. 1978.
6. J.D. Michie and M.E. Bronstad. Upgrading Safety Performance in Retrofitting Traffic Railing Systems. Southwest Research Institute, Rept. FHWA-RD-77-40, Sept. 1978.
7. Monthly Progress Report on Development of Safer Bridge Railing Design. Highway Research Institute, Texas Transportation Institute, Texas A&M Univ., College Station, Dec. 1980-Jan. 1981.

Crash-Test Evaluation of Barriers Installed on a Curved Off Ramp

M.E. BRONSTAD, C.E. KIMBALL, JR., AND C.F. McDEVITT

Although much has been learned about a relatively large number and variety of barrier systems installed on straight and level alignments, there has been a total lack of information on vehicle and barrier behavior and curved-superelevated-sloped alignments. Some recent catastrophic accidents on freeway off ramps have suggested that a better understanding of barriers mounted on these types of alignments was in order. Accordingly, a test program was designed to evaluate the performance of three barrier configurations mounted on a curved, superelevated structure with a downgrade. The objective of this project was to evaluate the performance of the three barrier configurations by using three vehicle types for comparison. The project included full-scale tests of three basic barrier installations: (a) concrete safety shape with vertical orientation, (b) concrete safety shape installed perpendicular to the superelevated roadway, and (c) tubular Thrie-beam and collapsing tube retrofit. Crash tests were conducted by using three vehicle types impacting a 40 mph (65 km/h) and a 15° angle (as measured from curve tangent). The three vehicle types were (a) 1800-lb (820-kg) mini-compact car (Honda Civic), (b) 2250-lb (1020-kg) subcompact (Vega), and (c) 20 000-lb (9070-kg) school bus (1970 66-passenger Ford/Wayne). All three barrier systems contained and redirected the full range of test vehicles. In terms of vehicle stability and acceleration, the tubular Thrie-beam retrofit was superior. However, there was some barrier damage in the bus test of this system.

For the past two decades, extensive crash-test evaluations have been conducted on longitudinal traffic barriers (i.e., guardrails, median barriers, and bridge-railing systems). In addition, many investigations have also included the use of computer simulations to predict vehicle and barrier behavior during the collision event. Although much is known about the performance of a relatively large number and variety of barrier systems installed on straight and level alignments, there has been a total lack of information on vehicle and barrier behavior on curved-superelevated-sloped alignments. Some recent catastrophic accidents on freeway off ramps have suggested that a better understanding of barriers mounted on these types of alignments was in order. Accordingly, a test program was designed to evaluate the performance of three barrier configurations mounted on a curved, superelevated structure with a downgrade.

The objective of this project was to evaluate the performance of three barrier configurations by using three vehicle types for comparison. In addition, two indirectly related tasks were also structured to provide information on vehicle mass and crush properties.

The project included full-scale tests of three basic barrier installations:

1. Concrete safety shape with vertical orientation,
2. Concrete safety shape installed perpendicular to the superelevated roadway, and
3. Tubular Thrie-beam and collapsing tube retrofit.

These barriers are shown in Figure 1.

Crash tests were conducted by using three vehicle types impacting at 40 mph (65 km/h) and a 15° angle (as measured from curve tangent). The three vehicle types were as follows:

1. 1800-lb (820-kg) mini-compact car (Honda Civic),
2. 2250-lb (1020-kg) subcompact (Vega), and
3. 20 000-lb (9070-kg) school bus (1970 66-passenger Ford/Wayne).

Each of the test vehicles contained two uninstrumented part 572 anthropometric dummies (50th percentile males). The dummies were positioned in the driver (restrained) and right front seat (unrestrained) occupant positions for the car tests. In the bus tests, the dummies were positioned to represent a restrained driver (lap belt) and an unrestrained passenger. The remaining payload of the bus was composed of three loose 100-lb (45-kg) sandbags per seat. An on-board camera recorded the motion of the dummies during the tests.

FINDINGS

In order to conduct the full-scale tests, a test installation that had the selected off-ramp geometry was excavated at the end of a paved airport runway. This excavation was paved with asphalt to simulate an off-ramp deck. The installation as shown in Figure 2 is essentially a curved ramp with the following characteristics:

1. 160-ft (48.8-m) outside radius,
2. 25-ft (7.6-m) roadway width,
3. 4.5 percent downgrade, and
4. 12 percent superelevation.

The crash tests were conducted on the three different barrier configurations by using the same test conditions. Sequential test photographs are arranged by vehicle type in Figures 3, 4, and 5. Figures 6, 7, and 8 contain after-test photographs arranged by vehicle type. The results of the crash tests are summarized in Table 1.

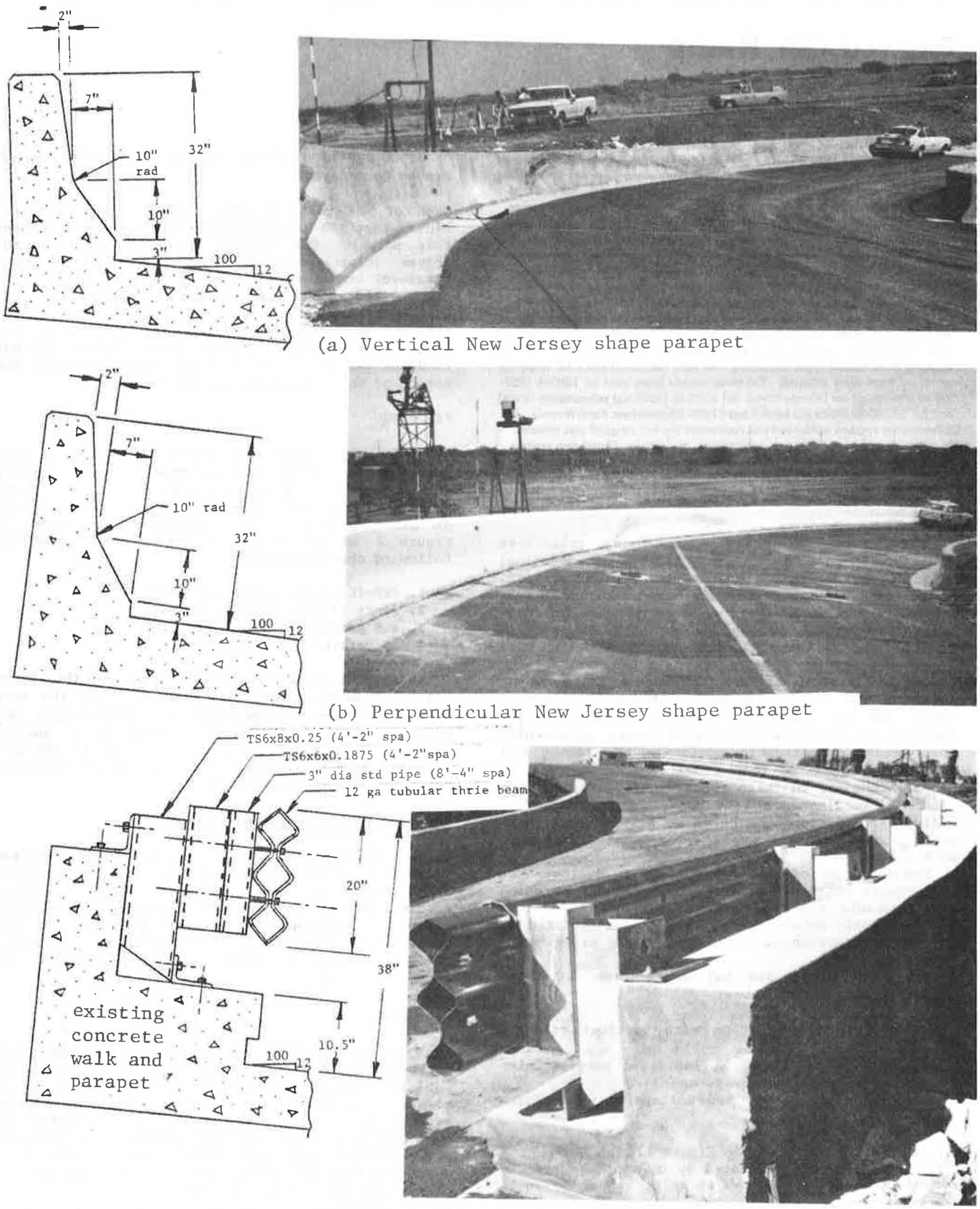
Vertical Safety Shape Test Series

A New Jersey-shape bridge parapet was installed vertically as the outside bridge rail on the simulated deck. The cross-section dimensions and reinforcing of the barrier were selected from a state standard, although the barrier strength was not expected to be critical for the 40-mph (65-km/h), 15° angle impacts. Findings from the tests are described below.

Test CB-1

A 1974 Vega impacted the barrier at 37.2 mph (59.8 km/h) and an 18.7° angle. As shown in Figure 3, the vehicle front wheels turned into the barrier as it climbed the lower sloped face. Rolling of the vehicle away from the barrier, which is typical of the interaction between New Jersey-shape barriers and vehicles, continued until the vehicle front wheel was near the top of the barrier. The maximum tire climb was 1.6 ft (0.5 m). The vehicle wheels then returned to grade with a continuous cyclic scrubbing of the outside barrier (with less climb at each cycle) until the vehicle left the barrier. The vehicle came to rest 4.5 ft (1.6 m) from the downstream end.

Figure 1. Barrier test installations.

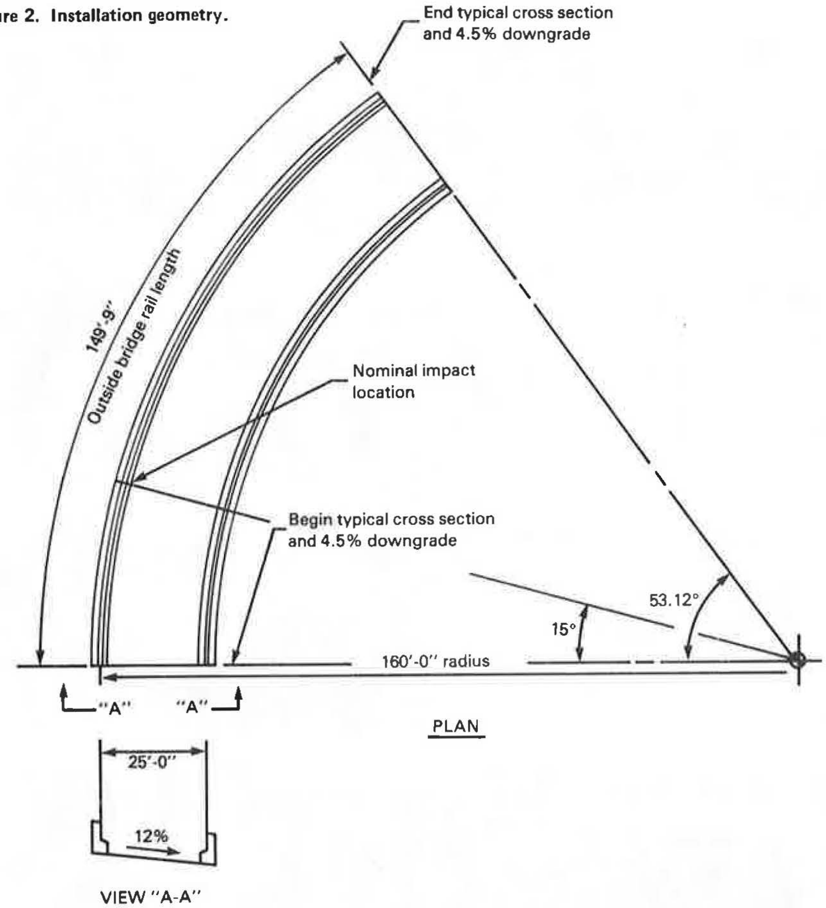


(a) Vertical New Jersey shape parapet

(b) Perpendicular New Jersey shape parapet

(c) Tubular thrie beam retrofit

Figure 2. Installation geometry.



Test CB-2

The school bus impacted the barrier at 41.8 mph (67.3 km/h) and a 15.5° angle. As shown in Figure 4, the bus rolled slightly away from the barrier as the left front tire climbed up the barrier face a maximum of 1.9 ft (0.6 m) and the front of the bus pitched upward. As the front moved downward from the maximum climb, the bus rolled toward the barrier before returning to a stable position near the installation end. After barrier contact was terminated, the bus turned to the right during braking and stopped about 100 ft (30 m) from the end.

Barrier damage consisted of minor scraping. Bus damage included a bent bumper and fender, two of five lug nuts sheared, and two shattered windows, as shown in Figure 7. The window damage was due to driver head intrusion and loose sandbag contact (near the rear end).

Test CB-3

A 1976 Honda Civic impacted the barrier at 40.0 mph (64.4 km/h) and a 13.9° angle. As shown in Figure 5, the vehicle rolled away from the barrier as the left front tire climbed the barrier face a maximum of 1.7 ft (0.5 m) and the vehicle front pitched upward. At 0.3 s after impact, the entire vehicle was airborne (tire contact with upper portion of barrier existed) and remained so for about 0.3 s. At this time the right wheels returned to grade, and the left wheels remained in barrier contact until the vehicle came to rest 8 ft (2.4 m) past the end of the barrier.

Perpendicular Safety Shape Test Series

Findings from the series of tests conducted on the New Jersey safety shape parapet, which was oriented perpendicular to the superelevation, are described below.

Test CB-4

A 1976 Honda Civic impacted the barrier at 38.9 mph (62.6 km/h) and a 13.4° angle. As shown in Figure 5, the vehicle rolled away from the barrier as the left front tire climbed up to a maximum of 1.7 ft (0.5 m) and the vehicle front pitched upward. At 0.3 s after impact, the entire vehicle was airborne, although left tire contact with the upper barrier was maintained. After the right tires returned to grade 0.3 s later, the vehicle remained in contact with the barrier until coming to rest 1 ft (0.3 m) from the barrier end.

Insignificant barrier damage occurred, and vehicle damage consisted of sheet-metal and left front wheel damage. Although the wheel was bent, there was no indication of air leakage.

Test CB-5

A 1975 Vega impacted the barrier at 38.9 mph (62.6 km/h) and a 14.9° angle. As shown in Figure 3, the vehicle rolled away from the barrier as the left wheels climbed the barrier up to a maximum of 1.5 ft (0.5 m). The left tires returned to grade, and then a second and third climb occurred before the vehicle reached the end of the barrier. After losing contact with the barrier, the vehicle went 42 ft (13 m) past the barrier end before coming to rest.

Figure 3. Vega sequential photographs.

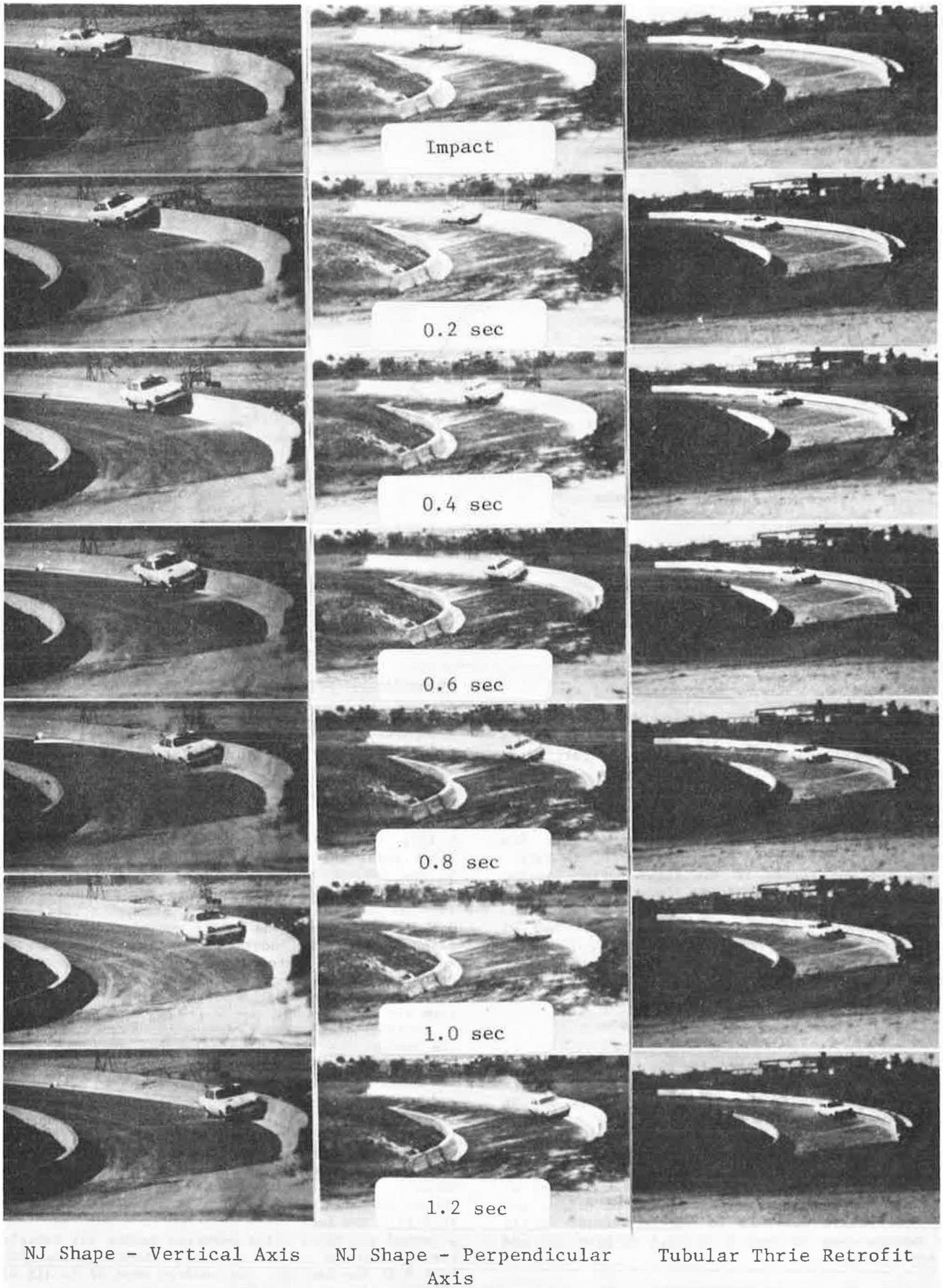


Figure 4. School bus sequential photographs.

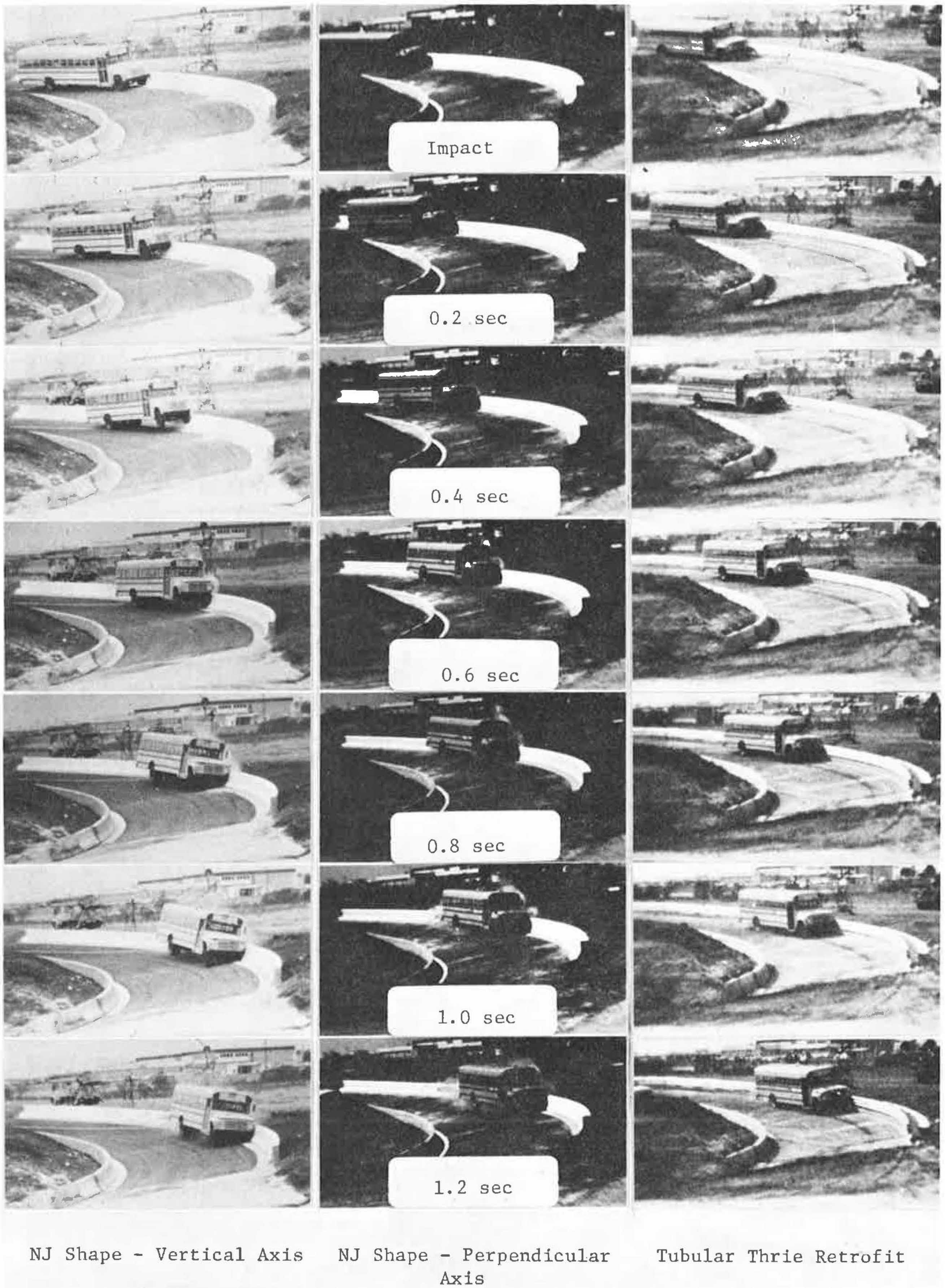


Figure 5. Honda Civic sequential photographs.

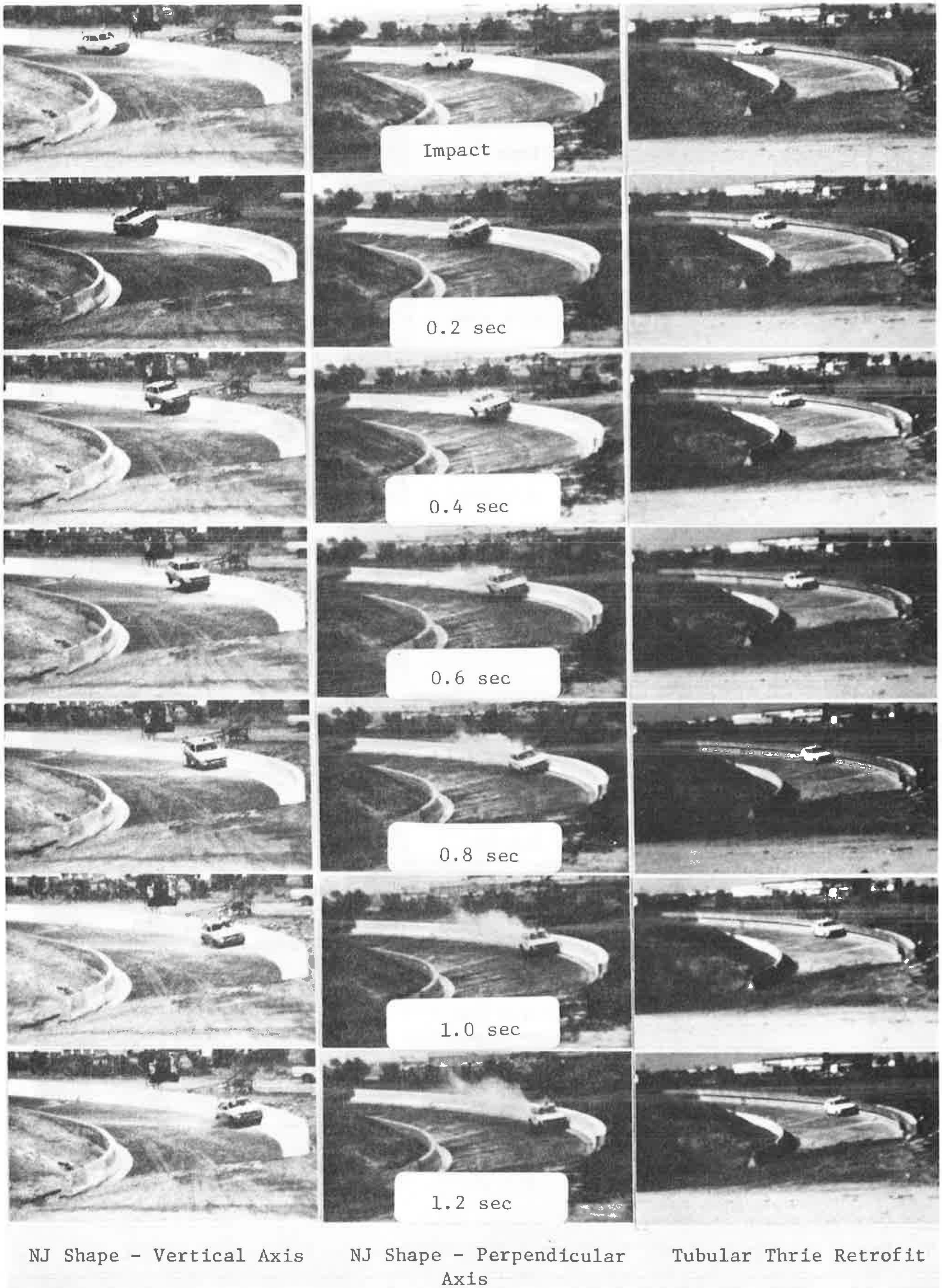


Figure 6. Photographs after Vega tests.

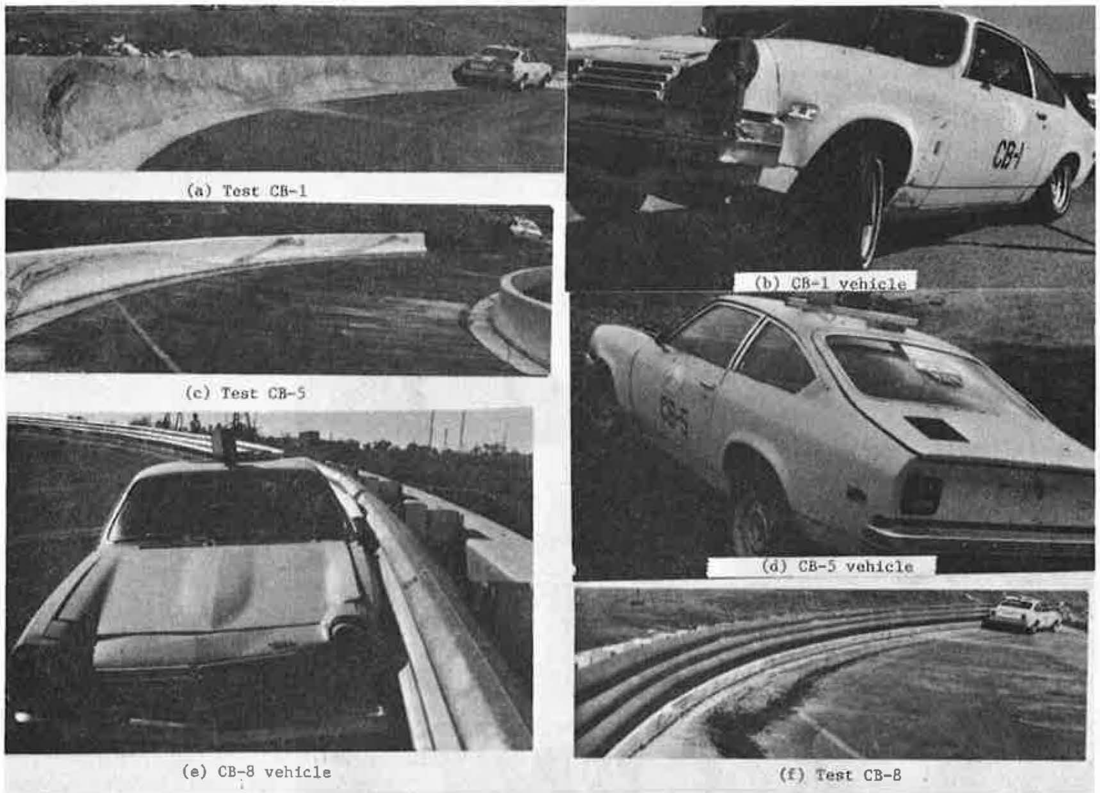
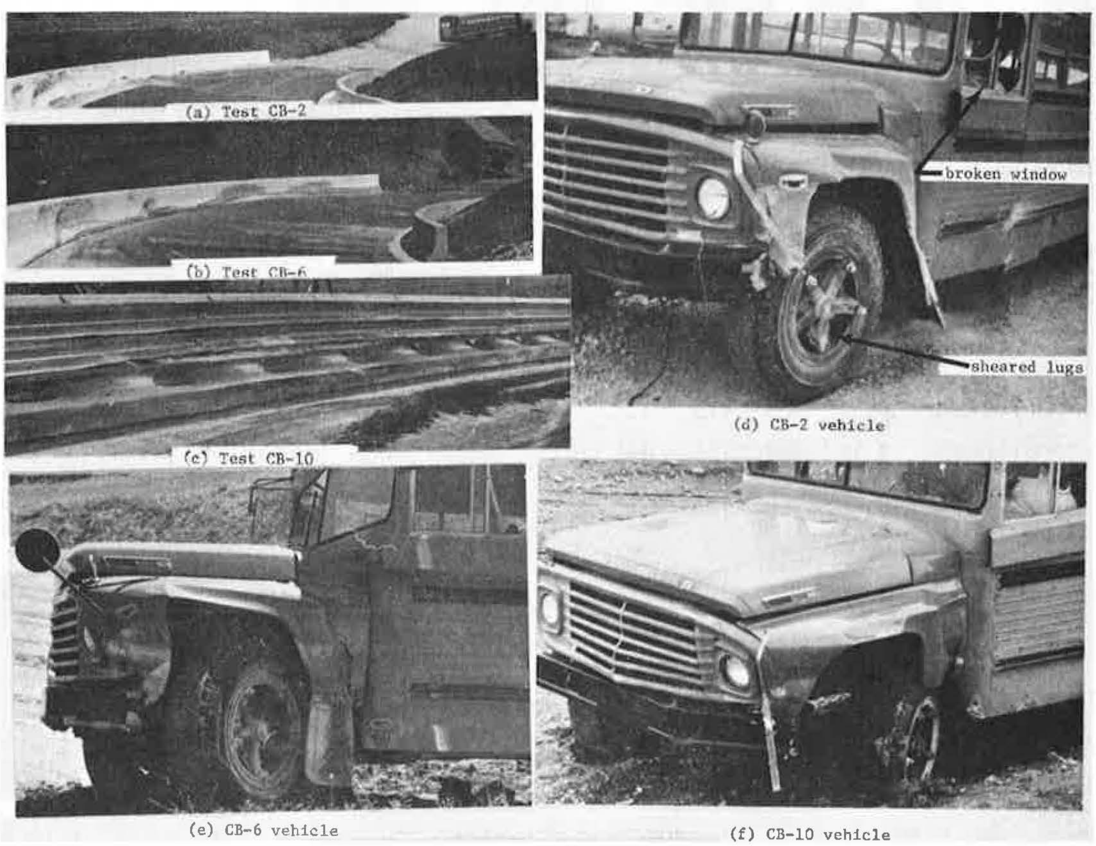


Figure 7. Photographs after bus tests.



Test CB-6

The school bus impacted the barrier at 40.0 mph (64.6 km/h) and a 14.8° angle. As shown in Figure 4, the vehicle front pitched upward as the left front wheel climbed the lower barrier slope. The bus rolled toward the barrier as the front wheels turned left. The bus then returned to a stable attitude and remained in constant barrier contact before leaving the barrier with a leftward turn imposed by the direction of the front wheels.

Vehicle damage was confined to the left front fender and bumper. Although the left front wheel and tire contacted the barrier, only tire scuffing was observed and no wheel lug damage was noted.

Figure 8. Photographs after Honda tests.

Table 1. Summary of full-scale crash test.

Test No.	Barrier	Vehicle	Vehicle Weight[a] (lb)	Impact Speed (mph)	Impact Angle (°)	Avg 50-ms Maximum Acceleration (g) Lateral	Avg 50-ms Maximum Acceleration (g) Longitudinal	Maximum Vehicle Climb (ft)	Maximum Roll Angle (°)	Vehicle Damage
CB-1	New Jersey shape, vertical axis	1974 Chevrolet Vega	2 686	37.2	18.7	-5.4[b]	-2.7[b]	1.6	11	Left front fender sheet-metal deformation, bumper displaced, bent left front wheel
CB-2	New Jersey shape, vertical axis	1970 Ford school bus	20 000	41.8	15.5	-5.6[b]	-1.9[b]	1.7	12	Left front fender sheet-metal scraping, damaged left front tire
CB-3	New Jersey shape, vertical axis	1976 Honda Civic	2 170	40.0	13.9	-1.3[b]	-4.5[b]	1.9	11	Left front fender sheet-metal deformation, bumper bent, two lugs sheared on left front wheel
CB-4	New Jersey shape, perpendicular axis	1976 Honda Civic	2 170	38.9	13.4	-2.6[b]	-1.3[b]	1.7	12	Left front fender sheet-metal deformation, damaged left front tire, bent left front wheel
CB-5	New Jersey shape, perpendicular axis	1974 Chevrolet Vega	2 611	38.9	14.9	-5.3[b]	-3.6[b]	1.5	15	Left front fender sheet-metal deformation, bumper displaced
CB-6	New Jersey shape, perpendicular axis	1970 Ford school bus	20 000	40.0	14.8	-2.9[b]	-1.1[b]	0.9	9	Left front fender sheet-metal deformation, bumper bent, left front tire scuffing
CB-7	Tubular Thrie-beam retrofit	1975 Honda Civic	2 170	38.8	15.1	-4.7[b]	-2.8[b]	0	0	Left front fender, scraping of left side
CB-8	Tubular Thrie-beam retrofit	1974 Chevrolet Vega	2 580	39.4	16.8	-3.8[b]	-2.6[b]	0	0	Left front fender, scraping of left side, front wheel suspension
CB-10	Tubular Thrie-beam retrofit	1970 Ford school bus	20 000	39.4	13.9	-1.3[c]	-0.9[c]	0	0	Sheet metal and front suspension

Note: 1 lb = 0.45 kg; 1 mph = 1.609 km/h; 1 ft = 0.30 m.

[a]Includes ballast and instrumentation. [b]Electronic transducer data. [c]Film analysis; electronic data unavailable.

Tubular Thrie-Beam Retrofit Series

The tubular Thrie-beam retrofit system tested in this series was developed in a previous Federal Highway Administration (FHWA) contract (1), where it successfully contained and redirected both a 40 000-lb (18 100-kg) intercity bus and an 1800-lb (800-kg) Honda Civic at 60 mph (95 km/h) and a 15° angle. As shown in Figure 1, it was developed for upgrading concrete parapets with or without metal railing on top and was used at narrow safety walks. The retrofit railing is shown attached to the parapet and walk system in Figure 1.

Test CB-7

A 1975 Honda Civic impacted the barrier at 38.8 mph (62.6 km/h) and a 15.1° angle. As shown in Figure 5, the vehicle was smoothly redirected with no measurable roll or wheel climb.

There was no significant barrier damage or deformation. Vehicle damage was limited to the sheet metal at the left fender and along the left side.

Test CB-8

A 1974 Vega impacted the barrier at 39.4 mph (63.4 km/h) and a 16.8° angle. As shown in Figure 3, the vehicle was smoothly redirected with excellent vehicle stability.

No barrier damage or deformation occurred. Damage to the test vehicle included sheet-metal deformation of the left front fender and some suspension damage.

Test CB-10

The school bus impacted the barrier at 39.4 mph (63.4 km/h) and a 13.9° angle. As shown in Figure 4, the bus rolled slightly toward the barrier as the front wheels turned left. The bus remained in a stable attitude throughout contact with the barrier until coming to rest 24 ft (7.3 m) past the downstream end.

Damage to the test vehicle was moderate. The left front fender and bumper were deformed, and the left front wheel was pushed rearward by the impact, fracturing the shaft from the steering box to the pitman arm as well as the u-bolts that connect the spring to the axle on the right side.

Vehicle and Barrier Damage

Figures 6, 7, and 8 contain damage photographs after the Vega, school bus, and Honda tests. Installation damage was significant only in test CB-10, where local crushing of the tubular Thrie-beam and some permanent deflection occurred.

CONCLUSIONS

General Performance

All three barrier systems contained and redirected the full range of test vehicles. In terms of vehicle stability and acceleration, the tubular Thrie-beam retrofit was superior. However, there was some barrier damage in the bus test of this system.

Safety Shape Orientation

There was not a dramatic difference in performance for the two barrier orientations. The preferred orientation from the concrete median barrier research program (2) was perpendicular to the superelevation when the vehicle approach is up the superelevation. Vehicle climb was reduced by this preferred orientation in the car tests, although only in the bus test was this significant. The school bus test was noticeably less severe in terms of vehicle redirection with the preferred perpendicular orientation.

Observations of the Honda test on the vertical barrier (CB-3) indicated that the vehicle was near the threshold of riding on top of the barrier. A slightly larger angle or speed could have produced this performance limit.

Tubular Thrie-Beam Retrofit

The installed Thrie-beam system was clearly more than adequate for the range of impacts tested. The system that was developed to redirect much larger vehicles at 60 mph (95 km/h) and 15° could be substantially reduced in cost by eliminating the intermediate posts that were not needed for the test conditions of this program.

The installed Thrie-beam retrofit was oriented perpendicular to the superelevation, which is preferred. Shimming of the spacers in the field may be required to orient the barrier in this manner.

Vehicle Factors

The shearing of two lugs from the bus wheel during the vertical safety shape test (CB-2) is cause for some concern. This wheel was tracking erratically after leaving the barrier; a loss of this wheel could have dramatically changed the test results.

The unexplained loss of a spindle nut during test CB-5 on the perpendicular safety shape made after-test photographs of the Vega sedan (Figure 6) look much worse than warranted. This spindle nut is a special one used to hold the guide wire flag to the wheel and cannot be considered part of the standard vehicle equipment.

REFERENCES

1. C.E. Kimball, Jr., and others. Heavy Vehicle Tests of Tubular Thrie-Beam Retrofit Bridge Railing. FHWA, Final Rept., Aug. 1980.
2. M.E. Bronstad and others. Concrete Median Barrier Research. FHWA, Rept. FHWA-RD-77-4, March 1976.

Vehicle Impact Tests of Breakaway Wood Supports for Dual-Support Roadside Signs

ROGER L. STOUGHTON, J. ROBERT STOKER, AND ERIC F. NORDLIN

Since the late 1960s, the California Department of Transportation has used 6 x 8-in (nominal) or smaller wood posts and timber poles (classes 1-6) that have drilled holes near the bases as breakaway supports for dual-support roadside signs. Due to the recent rapid increase in the lightweight-car population, crash tests were conducted with 2205-lb cars on these designs to determine whether they met performance criteria recommended in Transportation Research Circular 191 [now superseded by National Cooperative Highway Research Program Report 230, which recommends tests with even lighter-weight cars (1800 lb)]. When impacted by 2205-lb vehicles at 19.8 and 57.7 mph, the 6 x 8-in wood posts met all the criteria. A 9.25-in-diameter timber pole impacted by a 2205-lb vehicle at 19.2 mph did not break away. A modified timber-pole design was similarly tested; it broke away but was still too stiff. Consequently, timber-pole supports are no longer used on new construction in California. A 7.875 x 14.875-in laminated wood veneer box-section post that had saw cuts in the webs was impacted with a 2205-lb vehicle at 19.2 and 58.4 mph and met all test criteria. The design was adopted as a standard in California. A number of full-scale pendulum and static-bend tests on various breakaway support designs was conducted during this project.

For a number of years, roadside signs on California state highways have used breakaway wood-post or timber-pole supports. They have holes drilled near the base to make them break away when impacted by a vehicle. This design was based on three vehicle impact tests conducted by the California Department of Transportation (Caltrans) in 1966 and 1967 (1). This design has proved quite successful in California.

In July 1976, the Federal Highway Administration (FHWA) distributed FHWA Notice N5040.20 (2), which stated that all new federal-aid projects should comply with the FHWA suggested guidelines for application of breakaway requirements of the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (3), which were attached to the notice. These guidelines stated that in an 8-ft path, single wood posts should be no larger than 4x6 in and double posts no larger than 3x6 in or 4x5 in (full dimension). Hence, the timber poles and the 6x6-in and 6x8-in wood posts used by Caltrans would no longer be acceptable unless they were successfully tested by vehicle impacts in accordance with the FHWA guidelines.

Between 1972 and 1975, there were 11 fatal accidents on California state highways that involved wood sign supports. However, most of these accidents included vehicle rollovers, occupant ejections, motorcycle impacts, or multiple fixed-object impacts. Hence, the record looked good, but there was concern for the future, where many more lightweight cars would be on the highways. The FHWA guidelines recognized this trend.

The objective of this research was to conduct crash tests by using a lightweight (2250-lb) vehicle on the largest wood-post size used by Caltrans (6x8 in) and the largest size timber pole expected to meet the new FHWA guidelines. If these sizes met the criteria, all smaller sizes would qualify automatically.

If these tests were unsuccessful, the support designs would be modified or new types of wood supports would be developed and tested. Midway through the project it was decided to include full-scale static-bend tests to check wind load designs and full-scale pendulum tests to screen out new breakaway designs.

The crash tests were conducted in accordance with Transportation Research Circular (TRC) 191 (4). These procedures encompassed the requirements in the FHWA guidelines (2) and AASHTO specifications (3) and included detailed test procedures.

Table 1 summarizes all the known crash tests on dual-legged breakaway wood supports other than a lightly documented series in Pennsylvania in 1968 (7).

TEST CONDITIONS

Test Facility

All crash tests and some static-bend tests took place at Caltrans' Dynamic Test Facility. All pendulum tests and some static-bend tests were performed under contract with Southwest Research Institute (SwRI). For all crash tests and pendulum tests, the breakaway supports were embedded in standard soil pits in accordance with TRC 191 (4).

Test Vehicles

The test vehicles used for the six crash tests were 1976 Toyota Corolla 2-door sedans. The test inertial mass of these vehicles (excluding the part 572 dummy weight) was 2205 lb.

Test Sign Construction

The dimensions of the test signs are given in Table 1 and Figure 1. All posts and poles were made of Douglas fir; the posts were No. 1 grade. The sign panels were aluminum with a paper honeycomb core, either 1.125 or 2.625 in thick. A truck-mounted auger or bucket auger was used to drill holes in the ground for the supports. The sign panels were attached to the supports on the ground and then the entire sign was set in the holes. The holes were backfilled and tamped. Finally, breakaway holes and sawcuts were cut in the supports. Asphalt concrete pavement was removed around the supports.

Wood Post Properties

Caltrans uses wood posts in sizes from 4x4 to 6x8 in as single and dual supports for roadside signs. The largest wood-post size of 6x8-in Douglas fir could support a sign panel area up to 90 ft². The 6x8-in posts that have 2.5-in-diameter holes near the base were used in tests 351 and 352.

Timber Pole Properties

Test 353

Caltrans formerly used timber poles to support sign panels with areas from 85 to 265 ft². The poles were classes 1 to 6 and had average diameters from approximately 6.5 to 12.5 in near the ground line (8). There could be considerable variation in the average pole diameter for any given class because the diameter varied with the length of the pole and, in addition, the Caltrans specification (9) allowed the minimum circumference to be exceeded by as much as 5 in.

After examining the results of previous crash and pendulum tests on wood supports, it was concluded that some of the larger-sized poles might not break away. It was decided to crash test 9.25-in-diameter poles that had 3-in-diameter breakaway holes and a net shear area of 40 in².

SwRI Pendulum Tests

After the pole design used in crash test 353 stopped the 2205-lb test vehicle impacting at 19.2 mph without breaking, it was decided to evaluate some poles that had other hole patterns with pendulum tests.

One hole pattern that was pendulum tested at SwRI looked promising and was used for crash test 354. The timber-pole supports in test 353 had been virtually undamaged; therefore, they were reused for test 354, except that the opposite pole was impacted. The 4-in-diameter holes and connecting sawcut shown in Figure 1 were added to the existing 3-in-diameter holes.

Caltrans Static-Bend Tests

Following test 354, which was unsuccessful, Caltrans conducted static-bend tests on three pole specimens that had hole and sawcut patterns similar to those in test 354. These tests were to check the wind load design and to determine if larger holes could be cut in the poles.

Laminated Wood Veneer Box and I-Section Properties

SwRI Pendulum and Static-Bend Tests

After the timber-pole designs in tests 353 and 354 proved inadequate by crash testing, it was decided to try built-up wood-post sections by using high-strength laminated wood veneer lumber. Pendulum and static-bend tests were conducted at SwRI on box- and H-section posts. The 1-in-diameter holes and connecting sawcuts were used when rectangular web cutouts reduced the static-bend strength too much.

Studies of parallel-laminated wood veneer lumber have been conducted by the Forest Products Laboratory of the U.S. Forest Service (10,11). Some of the benefits of this lumber, when compared with solid sawn lumber, include the following:

1. Higher yield of material from logs,
2. Improvement in grade quality due to dispersal of knots and minimization of knot volumes,
3. Consequent higher average strength with less variation in strength, and
4. Longer lengths of material that are more dimensionally stable.

The laminated wood veneer lumber was built up from 0.125- or 0.1-in-thick C and D grade plywood-type Douglas fir veneers that had been ultrasonically graded and combined to obtain a specific bending strength. The veneers were all oriented with the grain of the wood parallel to the length of the member in order to maximize the bending strength in that direction. The lumber was manufactured in "billets" 2 ft wide and up to 80 ft long.

An exterior type glue (phenol-formaldehyde) was used to join the veneers. The flange and web elements of the built-up posts were joined with a phenol-resorcinol adhesive.

The lumber was available in allowable bending stress grades of 2500 to 3150 psi; a 2650-psi grade was used and applied both to billet material and the whole box section. The ultimate bending strength was 7400 psi; the modulus of elasticity was 2.0×10^6 psi. Allowable shear stress was the same for all bending stress grades. For shear perpendicular to the glue lines (neutral axis of box section), the allowable stress used was 285 psi and the average ultimate stress was 855 psi. For shear parallel to the glue lines (joint between flange and web), the allowable stress was 190 psi and the ultimate stress was 570 psi. Allowable stress adjustment factors were used for wind loading, wet condition of use, and shape factor. It was assumed the holes and sawcuts would reduce the ultimate bending strength of the box section by 20 percent, which was accurate.

The penetration and retention of preservatives in parallel-laminated wood veneer lumber is good. This is due to the lathe checks formed when the veneers are peeled from the logs and flattened (10,11). Waterborne preservatives require a strength reduction, but oil-borne preservatives do not. Built-up sections should be treated after gluing the joints. The glues and preservatives used are durable and not deleterious to each other (12-15).

Caltrans Static-Bend Test

The static-bend strengths for the SwRI post tests were low, probably due to a short clamping length with resultant high shear stresses. Hence, Caltrans performed one test on a box section fully embedded in the ground. The final hole and sawcut pattern was used. The ultimate moment of the post at ground level was an acceptable 79.8 kip•ft.

Two box-section posts can support 200-ft² sign panels that have midpanel heights of 21 ft. The design wind loading was 18.7 lb/ft² from a 60-mph wind at 15-30 ft heights, which is the maximum wind speed in California over a 10-year mean recurrence interval (except in local high wind areas).

The box section, which weighes 12 lb/ft, was selected over the H-section. It should be less susceptible to handling damage, more resistant to wind loads without increased impact resistance, and higher in torsional and lateral load resistance.

The box-section posts used for the tests cost approximately $6.00/linear ft. A verbal quote in August 1981 for small quantities, free on board (FOB) in Oregon, was $6.90/lineal ft for type M and $8.80/lineal ft for type L (see Figure 2).

Bolts that extended completely through the sign panel and box section were used for test 355. Because both posts were sheared off in this test, lag screws were used in test 356 to connect the sign panel to the adjacent box-section flange only.

A 12-ft-long wire rope choker that had swagged looped ends was buried 2-3 ft below ground around the post for east in extracting the post stub after a crash test.

TEST RESULTS

The results of the crash tests are summarized in Table 1.

Test 351

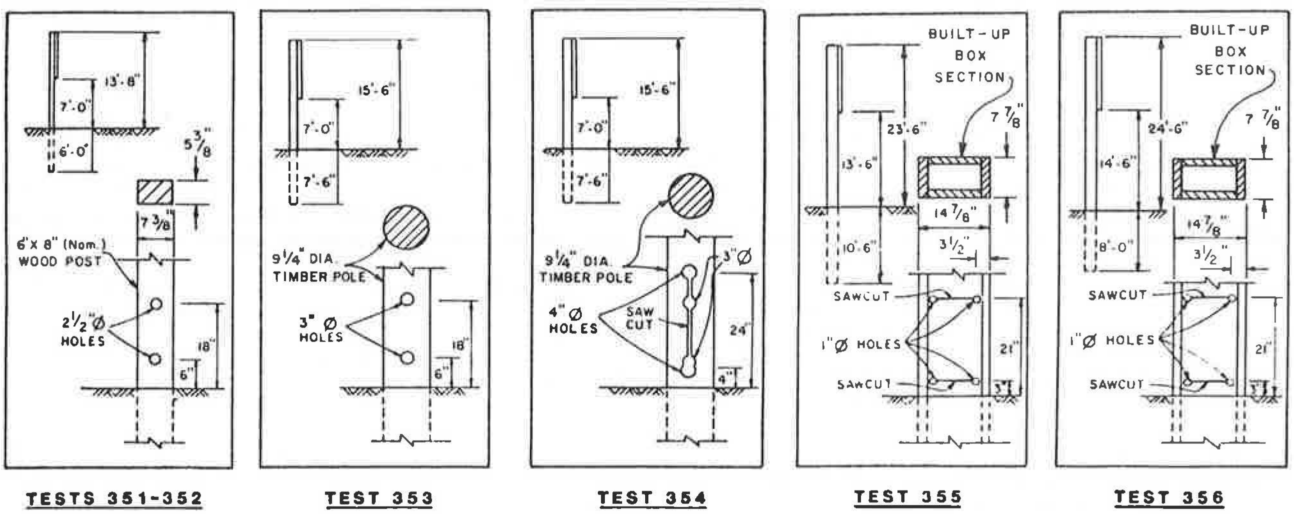
During impact, the post first split between the 2.5-in-diameter breakaway holes, and the split continued down below ground (see Figures 3 and 4). The post was torn off at the upper hole and the split stub below it was loaded because of two independent cantilevers that failed 10 in below ground. These two cantilevered post segments, which were 29.5 in long, stayed together and lodged beneath the

Table 1. Summary of crash tests.

Test Identification			Breakaway Support					Sign Panel		
Reference No.	Test No.	Test Date	Type	Modification	Net Shear Area (in^2)	Spacing (ft)	Embedment (ft)	Connection to Support	Size[a]	Ground Clearance (ft)
1	151	11/66	6 x 6-in Douglas fir posts	None	43	9.0	6.0	3/8-in bolts	5 ft by 14 ft by 1 in	7.0
1	152	5/67	11-in Douglas fir poles	None	95	12.0	9.5	3/8-in lag screws	10 ft by 20 ft by 2.5 in	7.0
1	153	5/67	11-in Douglas fir poles	3- and 4-in holes at 4, 10, and 16 in aboveground	52	12.0	9.5	3/8-in lag screws	10 ft by 20 ft by 2.5 in	7.0
5	351	4/78	6 x 8-in Douglas fir posts	2- and 2.5-in holes at 6 and 8 in aboveground	25	8.0	6.0	Eight 3/8-in threaded rods	6 ft 8 in by 13 ft by 1.125 in	7.0
5	352	8/78	6 x 8-in Douglas fir posts	2- and 2.5-in holes at 6 and 8 in aboveground	25	8.0	6.0	Eight 3/8-in threaded rods	6 ft 8 in by 13 ft by 1.125 in	7.0
5	353	1/79	9.25-in Douglas fir poles	2- and 3-in holes at 6 and 8 in aboveground	40	8.0	7.5	Eight 3/8-in lag screws	8 ft 6 in by 13 ft by 1.125 in	7.0
5	354	5/80	9.25-in Douglas fir poles	Same as test 353 plus two 4-in holes at 4 and 24 in with sawcut between	31	8.0	7.5	Eight 3/8-in lag screws	8 ft 6 in by 13 ft by 1.125 in	7.0
5	355	1/81	7.875 x 14.875-in box section	1-in holes 3.5 in from each edge and sawcut between at 3 and 21 in aboveground	30	12.0	10.5	Eight 3/8-in threaded rods	10 ft by 21 ft by 2.625 in	13.5
5	356	3/81	7.875 x 14.875-in box section	Same as test 355	30	13.0	8.0	Sixteen 1/2- by 5-1/2-in lag screws	10 ft by 21 ft by 2.625 in	14.5
6	902	1/79	6 x 8-in Douglas fir posts	Two 2.5-in holes at 6 and 18 in aboveground	28	8.0	6.0	3/8-in threaded rods	6 ft 8 in by 13 ft by 1.125 in	7.0

[a]Material used = aluminum.

Figure 1. Hole patterns for breakaway supports: crash tests 351-356.



vehicle. The upper section of the post and sign panel were pushed back by the vehicle as it yawed 35°.

Test 352

The impacted post failed the same way as the one in test 351 (see Figures 5 and 6). Again, a 28-in post segment separated from the post, lodged under the vehicle, and was dragged by the vehicle until it stopped. The upper part of the post, which was connected to the sign panel, was thrust up in the air while the vehicle passed underneath it and continued straight downstream.

Test 353

The timber-pole support was virtually undamaged by the vehicle except for scuff marks. The ground-line movement of the pole was 0.75 in (see Figures 7 and 8).

Timber-Pole Pendulum Tests: SwRI

One of four hole patterns met the change-of-momentum requirements. This pattern was used in the timber poles for test 354. In test 354, the vehicle sheared off the pole and pushed it ahead 5.5 ft before stopping and rebounding to a point 1.75 ft beyond the original pole location (see Figures 9 and 10). The segment of the pole between the bored holes separated from the main pole and split into several pieces.

Vehicle				Test Results								
Test Inertia Mass (lb)	Impact Velocity (ft/s)	Impact Velocity (mph)	Impact Angle (°)	Occupant-Compartment Impact Velocity (ft/s)		Initial Momentum, MV (lb·s)	Change of Momentum, ΔMV (lb·s)		High 50-ms Avg Longitudinal Vehical Acceleration (g)	Initial Vehicle Kinetic Energy, KE (kips·ft)	Change in Kinetic Energy, ΔKE (kips·ft)	Maximum Front Vehicle Crush (in)
				Film	Acceleration		Film	Acceleration				
4540	55.7	38.0	0	2.93		7858	414			219	22	
4540	58.7	40.0	0	5.87		8272	827			243	46	
2000	57.2	39.0	0	17.6		3553	1093			102	53	
2205	29.0	19.8	0	14.0	10.0	1989	958	685	-3.7	29	16	7
2205	84.7	57.7	0	10.0	3.82	5797	685	262	-1.9	245	22	8
2205	28.2	19.2	0	33.2	29.4	1930	1930	1930	-11.2	27	27	16
2205	29.2	19.9	0	17.0	18.0	1999	1160	1230	-7.5	29	25	13
2205	28.2	19.2	0	10.3	10.5	1928	706	721	-4.3	27	16	10
2205	85.7	58.4	0	3.2	3.74	5865	219	256	-4.0	251	21	10
2250	28.9	19.7	0	28.9	9.17	2021	2021	1380		29	16	

Timber-Pole Static-Bend Tests: Caltrans

The failure mode was different in each of the three static-bend tests. The ultimate bending moments varied from 1.5 to 2.4 times the design wind load bending moment.

Box- and H-Section Posts

Both pendulum and static-bend tests were conducted by SwRI and Caltrans. The final tests in this set showed the box-section posts had good static-bend strength and good impact performance (see Figure 11).

Test 355

After impact in test 355, the post split between the upper and lower 1-in holes on the upstream side and split to the ground from the lower downstream 1-in hole (see Figures 12-15). Then most of the post sheared off through the lower holes and sawcut. The vehicle moved in a circular path, pushing the box-section post in front of it. When this post was 15° off vertical, cracks appeared in the nonimpacted post. Eventually this post was twisted off at its base through the lower sawcuts. The sign panel remained attached to the posts but buckled in the middle. The vehicle went under the impacted post and stopped beyond the fallen sign.

Test 356

In test 356, the post was torn off through the lower sawcuts (see Figures 16-19). The upstream flange split away from the box section starting at its midpoint; however, the flange stayed attached to the sign panel with lag screws. The separated flange and the rest of the box section were thrust into the air by the vehicle, which passed underneath with no contact. When the partial box section struck the ground, it split into three pieces--two webs and the downstream flange. Meanwhile, the upstream flange of the nonimpacted post split off the box section from the top of the post to the bottom of the sign panel, which rotated and tore off the post. The upper upstream flange piece remained attached to the sign panel by the eight lag screws. The sign panel dropped flat on the ground.

DISCUSSION OF TEST RESULTS

The crash test results were compared with the three appraisal factors recommended in TRC 191 (4).

Structural Adequacy

In tests 353 and 354, the timber poles stopped the test vehicles too abruptly; thus, they did not meet the structural adequacy requirements. In test 355, the fallen sign projected 11 ft laterally beyond the original post location, thereby posing a possible traffic hazard. The switch from through bolts to lag screws for the sign-panel-to-post connection in test 356 prevented pull down of the nonimpacted post. The post pieces in test 356 projected out laterally 15 ft. The 1.5-in-thick pieces were flat on the ground. They would pose a psychological hazard more than a physical hazard.

In tests 351 and 355, which had impact speeds of 19-20 mph, the vehicles stayed in contact with the posts while stopping. Despite this, there was no apparent danger of passenger-compartment penetration.

Occupant Risk (Impact Severity)

The test results were compared against maximum recommended change-of-momentum values in TRC 191 (4) of 1100 lb•s (absolute) and 750 lb•s (preferred). Tests 351 and 352 on 6x8-in posts and tests 355 and 356 on box-section posts had values less than 750 lb•s and satisfied the criterion. Tests 353 and 354 on timber poles had values more than 1100 lb•s and failed the criterion.

Figure 2. Caltrans standard plan for laminated wood box post for roadside signs.

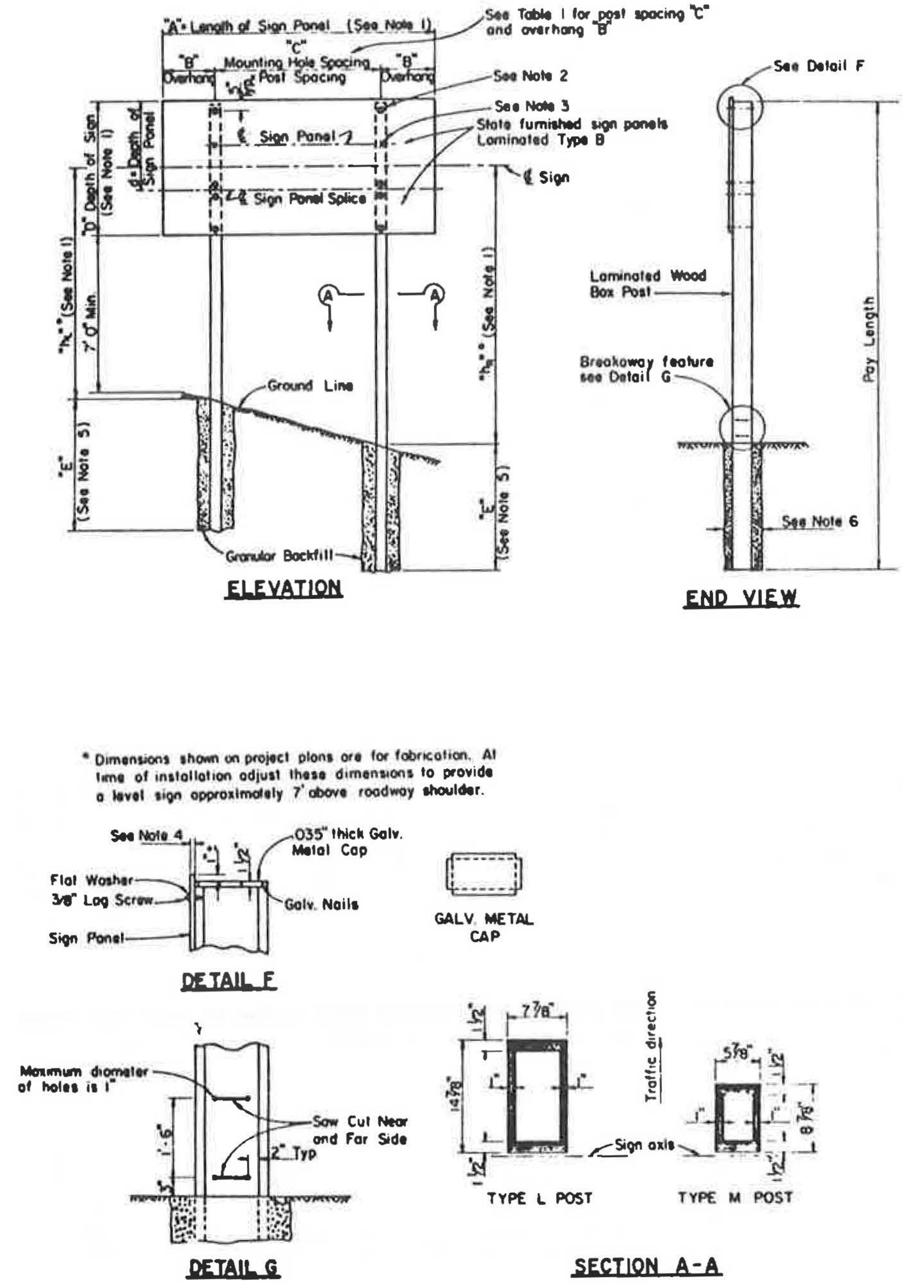


TABLE 1

SIGN PANEL LENGTH (See Note 1)	SIGN PANEL OVERHANG	MOUNTING HOLE SPACING
"A"	"B"	"C"
8'-0"	18"	5'-0"
9'-0"	22"	5'-4"
10'-0"	24"	6'-0"
11'-0"	24"	7'-0"
12'-0"	30"	7'-0"
13'-0"	30"	8'-0"
14'-0"	30"	9'-0"
15'-0"	36"	9'-0"
16'-0"	39"	9'-6"
17'-0"	39"	10'-6"
18'-0"	42"	11'-0"
19'-0"	45"	11'-6"
20'-0"	48"	12'-0"
21'-0"	51"	12'-6"
22'-0"	51"	13'-6"
23'-0"	54"	14'-0"
24'-0"	57"	14'-6"

TABLE 2

MINIMUM POST EMBEDMENT "E" FOR TYPE L POST

"h_L" or "h_R" (in feet)	TOTAL SIGN AREA (sq ft) 40 to 90	90+ to 140	140+ to 190	190+ to 240	240+ to 290
9 to 13	6'	6.5'	7.5'	8.5'	9'
13+ to 17	6'	7'	8'	9'	10'
17+ to 21	6'	7.5'	9'	9'	
21+ to 26	7'	8'	9'		

NOTES:

1. See Project Plans for:
   Location of each sign
   Length of sign panel "A".
   Depth of sign "D".
   Height "h_L" and "h_R" of centerline of sign above ground line at each post
   Type of post, L or M.
   See Standard Plan S41-3 for other details
2. "●" indicates location of 3/8" log screws and existing holes in panels. Log screws are to be embedded at least 1 1/2" into post.
3. "x" indicates location of additional 3/8" log screws required when the depth of sign panel (d) and the length of sign panel (A) are as follows:

d	A
60"	17'-0" to 24'-0"
54"	19'-0" to 24'-0"
48"	21'-0" to 24'-0"
42"	24'-0"

4. State-furnished Type B laminated sign panels are 1 1/8" thick for sign lengths of 15 feet and less. Panels over 15 feet in length are 2 5/8" thick.
5. Embedment "E" for Type L posts shall conform to the requirements in Table 2. Embedment for Type M posts shall be 6 feet minimum.
6. Diameter of post holes for Type L posts shall be at least 30 inches. Diameter of post holes for Type M posts shall be at least 24 inches.

Figure 3. Test 351: impact sequence.

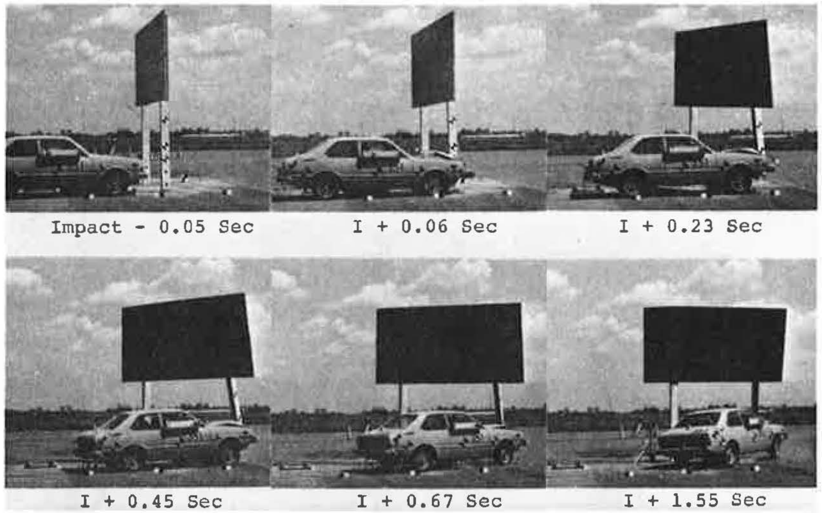


Figure 4. Test 351: final locations of test sign and vehicle after impact.

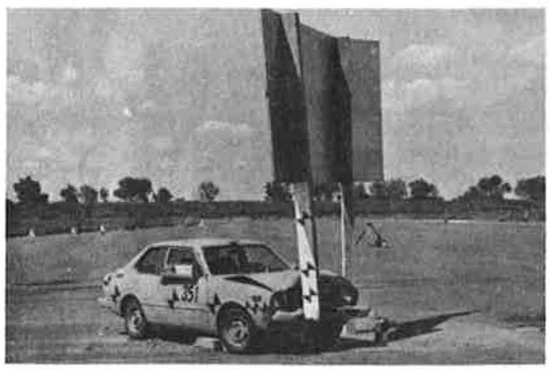


Figure 5. Test 352: impact sequence.

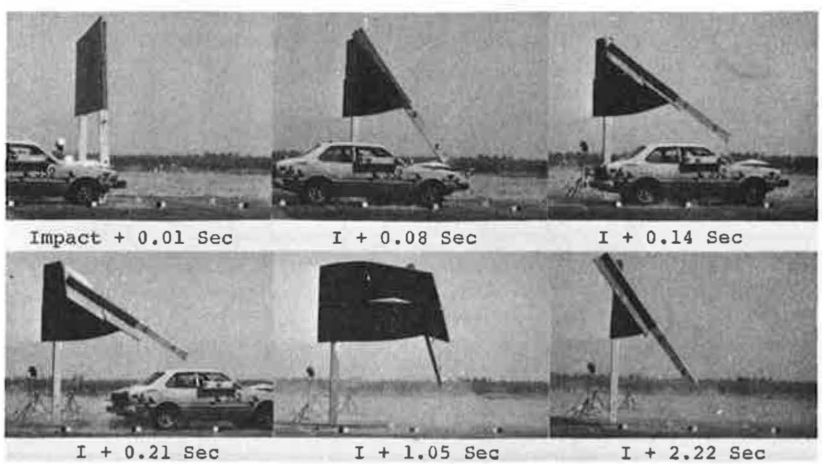


Figure 6. Test 352: crush at front of vehicle.

The test results were also compared against the new criterion in National Cooperative Highway Research Program (NCHRP) Report 230 (16) for maximum occupant and compartment impact velocities in the longitudinal direction that have a 2-ft flail space of 15 ft/s and a maximum ridedown acceleration of -15 *g* over any 10-ms period thereafter. Again, tests 351, 352, 355, and 356 met the criterion and tests 353 and 354 did not.

Figure 7. Test 353: impact sequence.

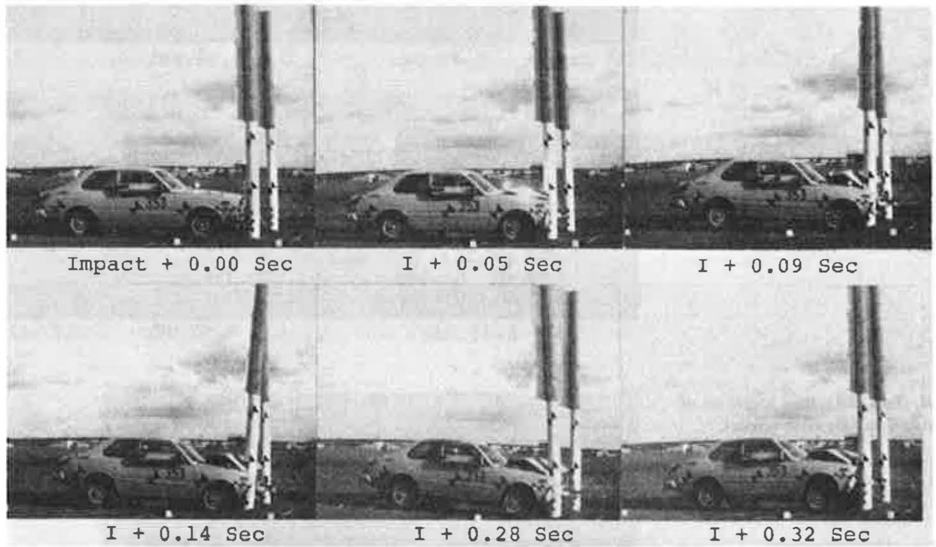


Figure 8. Test 353: final locations of test sign and vehicle after impact.

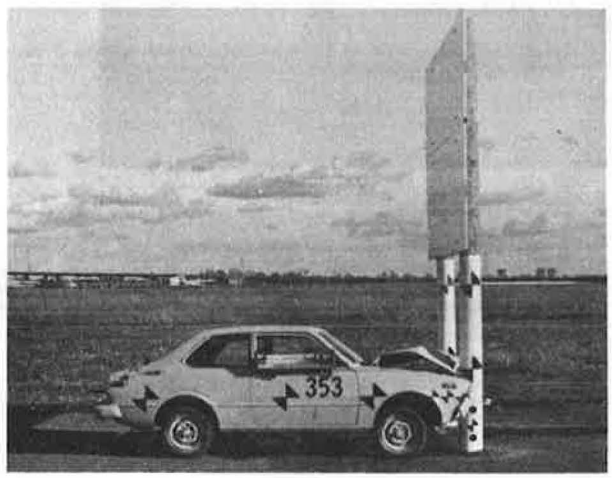


Figure 9. Test 354: impact sequence.

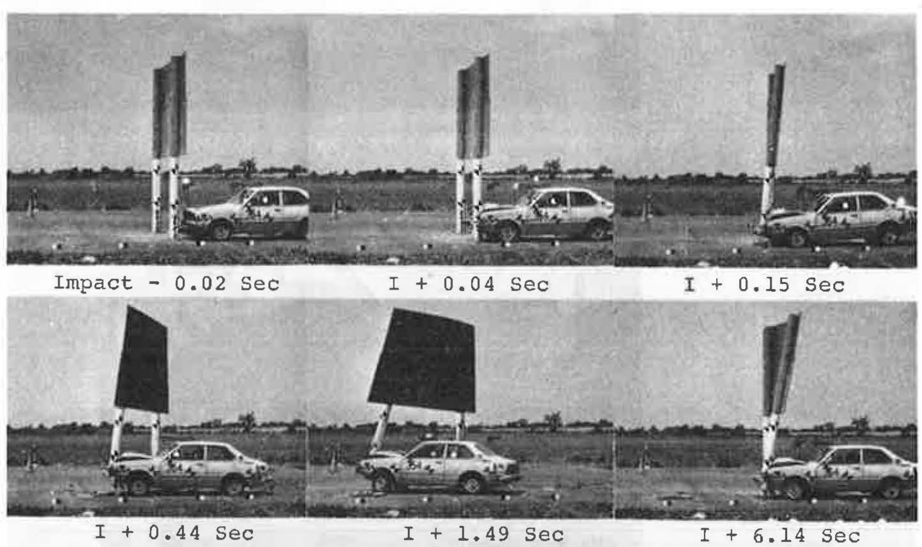


Figure 10. Test 354: crush at front of vehicle.

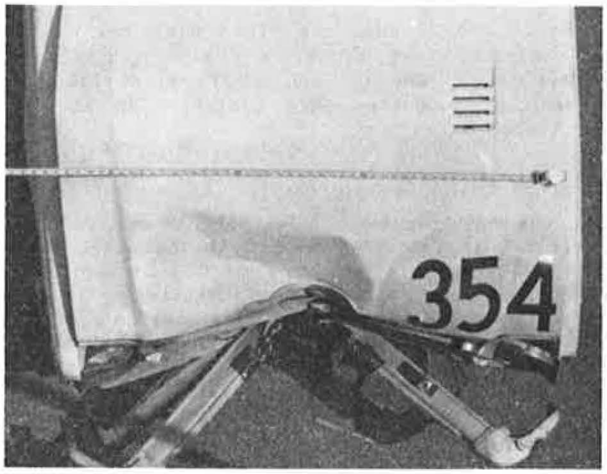


Figure 11. Static-bend test of laminated wood box-section post—broken stub.

Figure 12. Test 355: impact sequence.

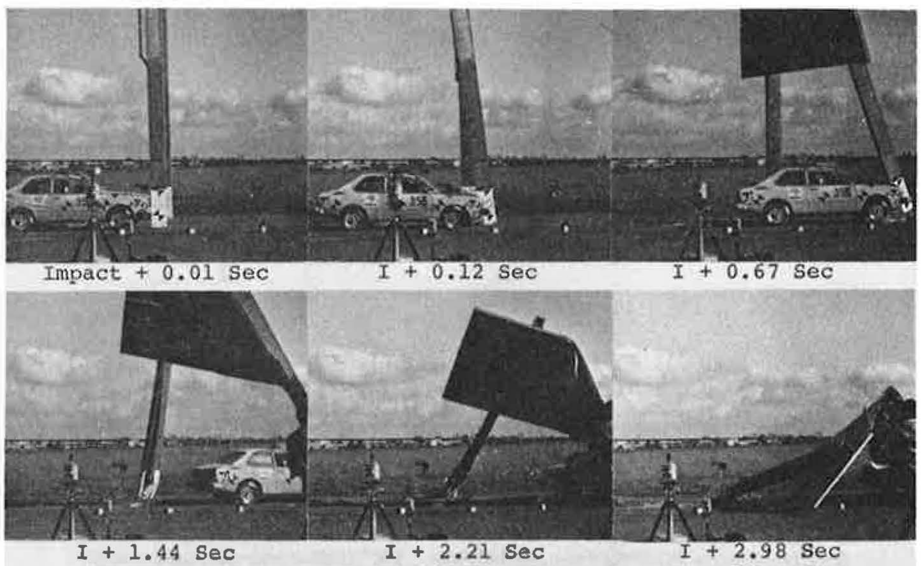


Figure 13. Test 355: test vehicle and box-section post before impact.

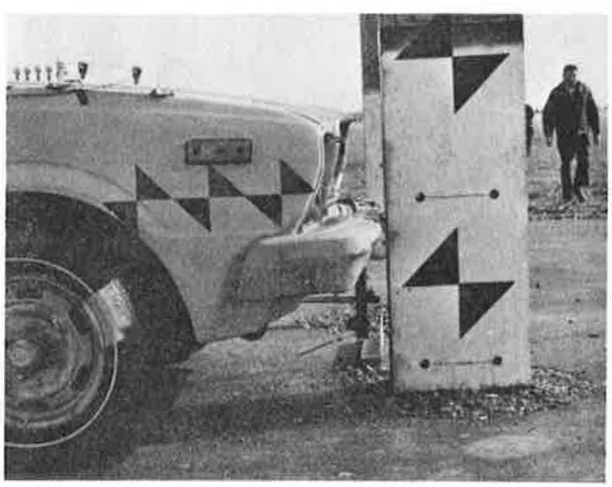


Figure 14. Test 355: final location of test sign and vehicle, looking downstream.

Figure 15. Test 355: stub of impacted box-section post.

Vehicle Trajectory

Figures 20 and 21 show the final positions of the test vehicles, test signs, and sign debris. The criteria in TRC 191 (4) and NCHRP Report 230 (16) for vehicle trajectory were satisfied in all six crash tests.

Implementation

After the unsuccessful timber-pole tests, Caltrans substituted a standard design by using steel posts and a slip base in February 1980. In mid-1981, box-section posts were added as an alternative (see Figure 2). The standard plan for roadside signs by using 6x8-in or smaller wood posts was unchanged. Although the steel post and slip base designs have functioned well, Caltrans has preferred wood support designs for the following reasons: (a) The wood supports have generally been less expensive in California, (b) their service life has proved to be sufficient, (c) they are easier for maintenance

Figure 16. Test 356: impact sequence.

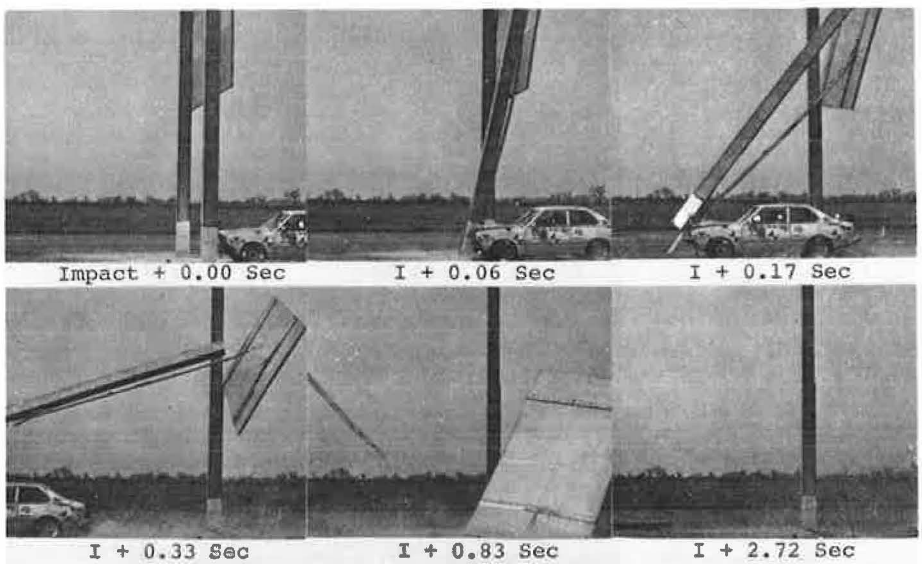


Figure 17. Test 356: crush at front of vehicle.

Figure 18. Test 356: final location of sign panel and pieces of impacted box-section post, looking upstream.

Figure 19. Test 356: nonimpacted post that had upper 10 ft of flange torn off.

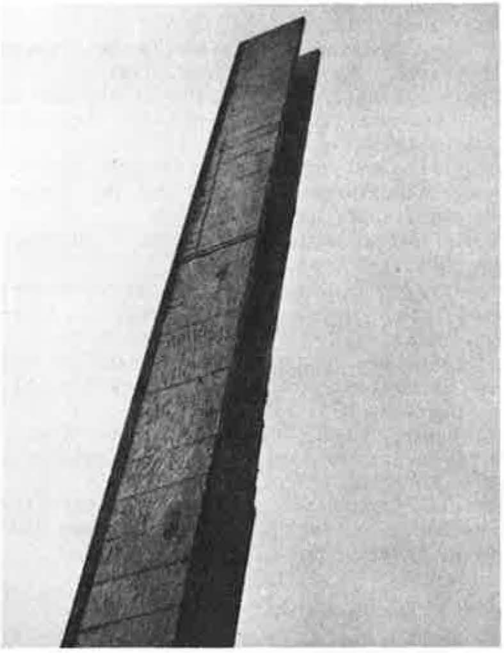


Figure 20. Final position of test vehicle and sign: tests 351-353.

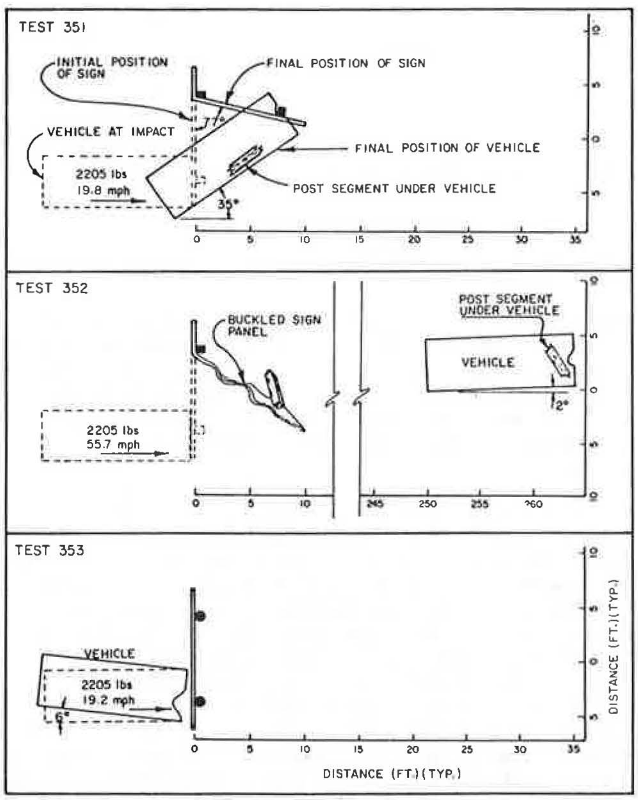
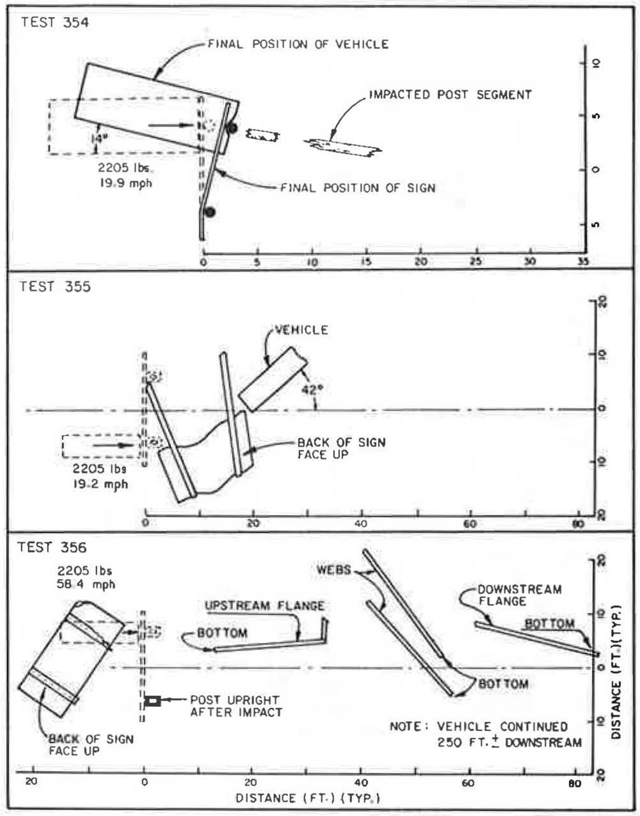


Figure 21. Final position of test vehicle and sign: tests 354-356.



personnel to erect, and (d) they can be stocked in a small number of standard sizes and easily sawed to the correct length.

Future Work

Additional crash tests should be conducted by using 1800-lb vehicles on 6x8-in and box-section breakaway wood sign supports as recommended in NCHRP Report 230 (16), the new crash-test guidelines.

CONCLUSIONS

Crash tests, pendulum tests, and static-bend tests were conducted on three general types of breakaway wood supports for dual-legged roadside signs. The tests were judged against criteria in the AASHTO guidelines (3), in TRC 191 (4), and, to some extent, in NCHRP Report 230 (16). Wood posts 6x8 in and smaller that have holes drilled according to Caltrans standard plans, and 7.875x14.875-in laminated wood veneer box-section posts that have 1-in drilled holes connected by horizontal sawcuts in the webs reasonably met the above criteria. Timber poles 9.25 in in diameter with 3-in-diameter holes or 4-in-diameter holes with a connecting sawcut did not meet the above criteria and are not recommended for new construction. It is recommended that new sign installations that use the box-section posts be subjected to an in-service evaluation equal or similar to the one recommended in NCHRP Report 230 (16). Complete details of this research project are contained in a report by Stoughton and others (5).

ACKNOWLEDGMENT

This research was sponsored by FHWA, U.S. Department of Transportation. The findings and conclusions in this paper represent our views and not necessarily the official views or policies of the State of California or the U.S. government. This paper is not an endorsement of products or manufacturers.

The excellent work (pendulum tests, static-bend tests, and resulting reports) done at SwRI by Ed Kimball, Glen Deel, and Maurice Bronstad is gratefully acknowledged.

REFERENCES

1. E.F. Nordlin, W.H. Ames, and R.N. Field. Dynamic Tests of Wood Post and Timber Pole Supports for Roadside Signs, Series XV. Materials and Research Department, California Department of Transportation, Sacramento, Rept. 635398, Dec. 1967.
2. AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals. FHWA, FHWA Notice N5040.20, July 4, 1976.
3. Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals. Subcommittee on Bridges and Structures, AASHTO, Washington, DC, 1975.
4. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. TRB, Transportation Research Circular 191, Feb. 1978.
5. R.L. Stoughton, J.R. Stoker, and E.F. Nordlin. Vehicular Impact Tests of Breakaway Wood Supports for Dual-Support Roadside Signs. Office of Transportation Laboratory, California Department of Transportation, Sacramento, Rept. FHWA/CA/TL-81/14, July 1981.
6. J.H. Bloom and J.H. Hinch. Laboratory Evaluation of Existing Breakaway Structures, Volumes I-III. FHWA, Rept. FHWA-RD-79-139, 140, and 141, June 1980, pp. 193-247.
7. F.C. Sankey. Dynamic Field Test of Wooden Signposts. In Highway Safety, HRB, Special Rept. 107, 1970, pp. 158-168.
8. Sign Reference Sheets. Office of Structures Design, California Department of Transportation, Sacramento, June 1973.
9. Standard Specifications. California Department of Transportation, Sacramento, Jan. 1981.
10. G.B. Harpole. Assessing a Continuous Process to Produce Press-Lam Lumber. Forest Products Journal, Aug. 1976.
11. J.A. Youngquist and others. Press-Lam Timbers for Exposed Structures. Journal of the Structural Division, ASCE, July 1979.
12. M.L. Selbo. Glue Joints Durable. Southern Lumberman, Dec. 15, 1958.
13. R.H. Gillespie. Accelerated Aging of Adhesives in Plywood-Type Joints. Forest Products Journal, Sept. 1965.
14. R.H. Gillespie and B.H. River. Durability of Adhesives in Plywood. Forest Products Journal, Technical Section, Oct. 1976.
15. Synthetic Resin Glues. Forest Products Laboratory, U.S. Forest Service, Res. Note FPL-0141, June 1966.
16. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP, Rept. 230, March 1981.

Notice: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names appear in this paper because they are considered essential to its object.

Thrie-Beam Guardrails for School and Intercity Buses

DON L. IVEY, CHARLES F. McDEVITT, RICHARD ROBERTSON, C. EUGENE BUTH, AND ARTHUR J. STOCKER

The results of full-scale tests that were conducted to establish the upper performance limits of conventional W-beam guardrail and Thrie-beam guardrail systems are described. The tests showed that these conventional guardrail systems cannot safely redirect a 9070-kg (20 000-lb) school bus in a 15° angle impact at 96.5 km/h (60 mph). The development and evaluation of a modified Thrie-beam guardrail are also described. A series of full-scale tests has demonstrated that the unique feature of this guardrail system, a special 0.36-m (14-in) deep blockout, not only prevents the wheels of mini-compact cars from snagging on the posts but also raises the rail during impact to stably redirect heavier vehicles such as school and intercity buses.

In order to provide safer highway appurtenances for the public, there is an increasing emphasis on designing traffic barriers such as guardrails and bridge rails for a wider spectrum of highway vehicles. Witness the growing emphasis on designing guardrail terminals for mini-compact cars as they become a more significant part of the vehicle fleet and also recent efforts to design bridge rails for both school and intercity buses (1,2).

This report describes work that was aimed at investigating the feasibility of enlarging the spectrum of vehicles considered in the guardrail design process. Until recently, guardrails have been designed to accommodate a 2041-kg (4500-lb) automobile at 96.5 km/h (60 mph) and 25° as the most critical test. The goal of this study was to determine if a relatively conventional guardrail design is suitable to safely redirect a 9072-kg (20 000-lb) school bus moving at 96.5 km/h and at an impact angle of 15°. If this proved not to be the case, the objective was to see if reasonably economical guardrails can be designed to accomplish this task.

To reach these objectives, the tests described in Table 1 were conducted. The cross sections of the guardrail for each test are shown in Figures 1, 2, and 3.

The tests were conducted in the order given in Table 1. The Thrie-beam guardrail shown in Figure 1 was selected for the first test. Because it was a choice between the conventional W-beam guardrail and the conventional Thrie-beam guardrail, the following reasoning dictated the choice of the Thrie-beam. If the Thrie-beam (G9) guardrail failed to redirect a school bus, there was no reason to test the W-beam, since it would certainly be of lower capacity. This might save one test that could be used to evaluate a modified Thrie-beam rail. If the Thrie-beam functioned reasonably well, there was a chance that the W-beam (G4-1S) guardrail would also perform adequately. The W-beam guardrail then would be selected for the second test. The testing program would prove the latter situation to be the one encountered. Although detailed accounts of these individual tests are given in subsequent parts of this report, a brief description of each test is presented here.

DISCUSSION OF TEST RESULTS

In the first test, which was conducted on the Thrie-beam guardrail shown in Figure 1, the 9081-kg (20 020-lb) bus at 89.5 km/h (55.6 mph) and 13.5° was contained and redirected; the bus then went through a slow 90° counterclockwise roll before falling onto its left side and sliding to a stop. Although the 90° roll was not an ideal reaction, it was a fairly smooth roll, which should not be extremely hazardous to passengers if the integrity of the left-side windows is maintained. The performance of the rail was therefore considered marginal. The guardrail exhibited enough strength and maintained continuity so that the bus was contained and redirected. Accelerations on the bus during the event were low, while permanent deflection of the rail was about 0.41 m (1.33 ft).

Based on the results of the first test, it was decided that the conventional W-beam guardrail has a reasonable chance of containing and redirecting a school bus. The W-beam had about as much post support as the Thrie-beam. After impact deflection, it has about the same point of resistance height as the Thrie-beam. This is true as the rail begins to deflect, at least up to the time that the bus rolls enough to make contact with the top part of the deflecting and rotating W-beam or Thrie-beam. To counter the argument that the W-beam guardrail had a chance of containing and redirecting a bus were the facts that the barrier height would be reduced 13.3 cm (5.25 in) and the bending stiffness of the W-beam would be much lower than the Thrie-beam, a factor that results in the transmission of lateral load to fewer support posts during an impact. The full-scale test resolved this question by demonstrating that the factors against a successful containment were dominant.

In the second test, conducted on the W-beam guardrail shown in Figure 2, the bus was not contained. At a speed slightly higher than in the first test [96.0 km/h (59.6 mph) compared with 89.5 km/h], the bus started to redirect as the left front corner made contact. However, as it rolled left and yawed clockwise, the rear of the bus went over the barrier, penetrating into the zone behind the rail. At one point the bus was sliding upside down along the guardrail, which resulted in a shredding of the bus top. This reaction was obviously unacceptable because it would have resulted in many severe passenger injuries.

Table 1. Description of tests.

Test No.	Vehicle	Impact Velocity[a] (km/h)	Impact Angle[a] (°)	Point of Impact	Rail Type
1	9072-kg school bus	96.5	15	Midstream	Thrie-beam
2	9072-kg school bus	96.5	15	Midstream	W-beam
3	9072-kg school bus	96.5	15	Midstream	Modified Thrie-beam
4	1032-kg 1976 Honda sedan	96.5	15	Midstream	Modified Thrie-beam
5	956-kg 1975 Honda sedan	96.5	20	Midstream	Modified Thrie-beam
6	14 515-kg intercity bus	96.5	15	Midstream	Modified Thrie-beam

Note: 1 kg = 2.24 lb; 1 km/h = 0.62 mph.
[a]Values shown here are the planned test values; actual observed values differed slightly, as shown in Table 2.

Figure 1. Conventional Thrie-beam guardrail (test 1).

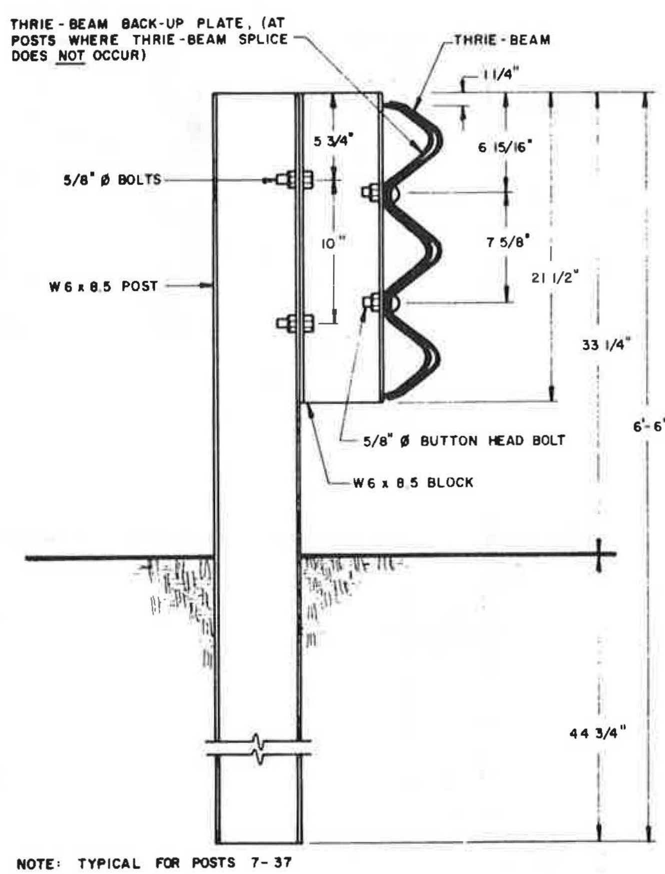


Figure 2. Conventional W-beam guardrail (test 2).

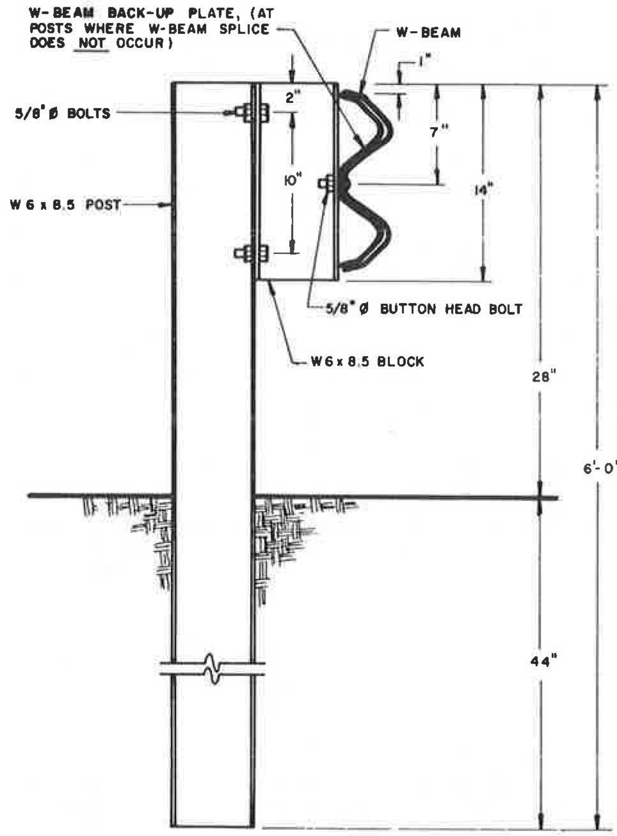
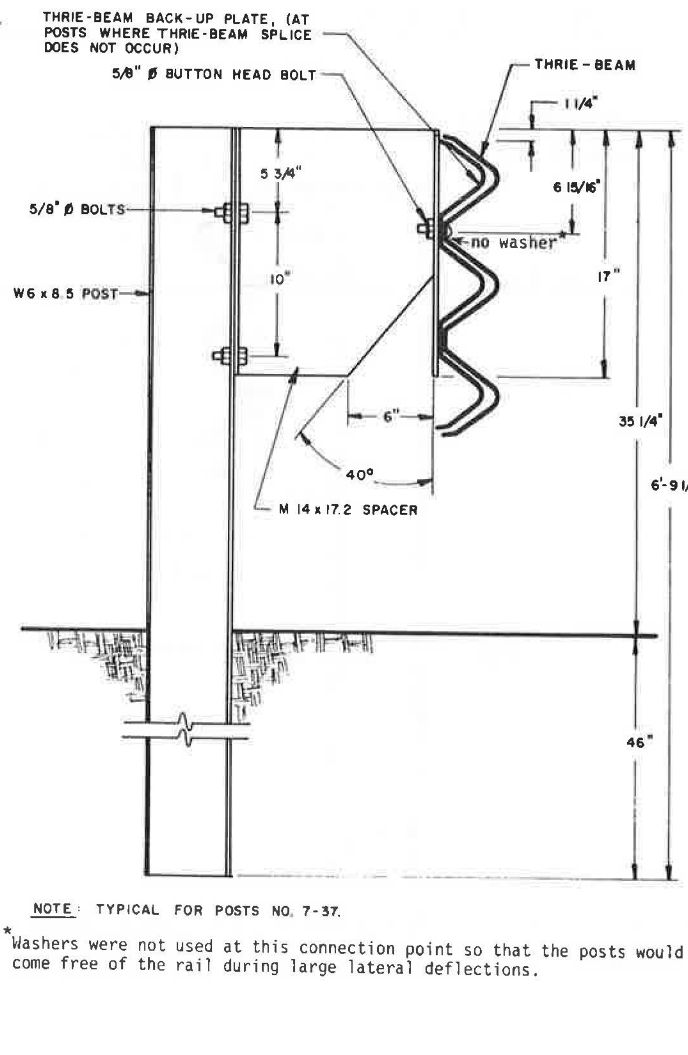


Figure 3. Modified Thrie-beam guardrail (tests 3-6).



NOTE: TYPICAL FOR POSTS NO. 7-37.

*Washers were not used at this connection point so that the posts would easily come free of the rail during large lateral deflections.

By using the experience gained from the first two tests, it was apparent that significant design changes would have to be made if a guardrail was to safely contain and redirect a bus after a 96.5-km/h (60-mph) collision. The Thrie-beam guardrail used in test 1 proved strong enough, but it exerted its resisting force at a point too low to prevent the bus from rolling. It was considered the prime candidate for redesign. The emphasis would be to make design changes that would elevate the point of resistance during a collision. The guardrail shown in Figure 3 is the result of those efforts. The following design changes were established during design meetings between Texas Transportation Institute (TTI) and Federal Highway Administration (FHWA) engineers:

1. The overall height of the barrier was increased by 0.05 m (2 in), from 0.84 m (33.25 in) (Figure 1) to 0.90 m (35.25 in) (Figure 3).
2. The blockout depth was increased by 0.20 m (8 in), from 0.15 m (6 in) to 0.36 m (14 in). This results in the rail moving upwards as the support post rotates.
3. A triangular-shaped segment was cut from the web of the M14x17.2 spacer as shown in Figure 3. This notch allows the lower portion of the Thrie-beam and the adjacent spacer block flange to bend in during a collision. This keeps the rail face vertical in the impact zone. It also reduces the contact forces between an impacting vehicle and the lower part of the Thrie-beam, thereby requiring the centroid of the resisting loads to move up onto the fully supported part of the rail. The net effect is that the resultant resisting force of the rail is raised to a higher position, which produces a smaller roll moment on the vehicle.
4. Embedment length of the guardrail posts was incresased slightly from 1.14 m (44.75 in) to 1.17 m (46 in). Consideration was given to welding bearing plates on the support posts to significantly increase post capacity. This option was not taken, since it was not determined that additional post capacity was necessary and the addition of the plates would significantly increase fabrication costs.

The modifications described above proved adequate. The third test of a school bus at 89.8 km/h (55.8 mph) and 15° produced a bus reaction that was acceptable. The bus was contained and smoothly redirected and remained upright throughout the event. During the rail contact period, there was approximately 25° of counterclockwise bus roll when viewed from the rear. Overall, it was interpreted as a stable rail collision. Table 2 summarizes data from all of the tests. Sequential photographs from test 3 appear in Figure 4.

Next, two tests of the same modified Thrie-beam guardrail were conducted with Honda Civic sedans in order to see if raising the Thrie-beam rail by 0.05 m (2 in) had compromised its performance for small vehicles. There was concern that the front wheels might get under the rail and snag on the blockout or post. No snagging was observed in either test. In test 4, a 1976 Honda Civic sedan weighing 1032 kg (2276 lb) was redirected with a shallow exit angle and remained upright after a 100.6-km/h (62.5-mph) and 15° impact. The dummy driver's head impacted and broke the side door window. However, the dummy accelerations meet the flail-space criteria in National Cooperative Highway Research Program (NCHRP) Report 230 (3), and the test results are considered satisfactory. Similar results were obtained in test 5, which was conducted with a 1975 Honda Civic sedan at 99.1 km/h (61.6 mph) and an 18° angle. Table 2 summarizes these tests. After tests 4 and 5 had been conducted with the Honda Civic sedans, the bent flange tabs and Thrie-beam rails in the impact zones were restored with a bumper jack and a hammer, as shown in Figure 5.

The final question to be answered was whether the modified Thrie-beam could redirect a 14 515-kg (32 000-lb) intercity bus at 96.5 km/h (60 mph) and 15°. This question was addressed by using several analytical approaches and finally with a full-scale crash test. The analytical approaches attempted were a simple energy balance, a comparative structural analysis, and the Barrier VII program. They all predicted marginal performance of the modified Thrie-beam in an intercity bus test. Barrier VII predicted a deflection of 2.3 m (7.3 ft), but it was noted that this program has on occasion predicted deflections that were somewhat high. We believed that redirection could be achieved if the dynamic deflection could be held under 1.8 m (6 ft).

When the intercity bus test was conducted, the results were excellent. This is evident from Figure 6 and from the test summary given in Table 2. The impact angle was 14.0°. The speed just prior to impact was 95.9 km/h (59.6 mph). Vehicle stability was good, and there was a maximum counterclockwise roll angle of approximately 15° (i.e., roll into the barrier). The dynamic deflection was approximately

Table 2. Summary of data: tests 1-6.

Item	Test 4098-1	Test 4098-2	Test 4098-3	Test 4098-4	Test 4098-5	Test 4098-6
Rail	Thrie-beam	W-beam	Modified Thrie-beam	Modified Thrie-beam	Modified Thrie-beam	Modified Thrie-beam
Blockout	W6x8.5	W6x8.5	M14x17.2	M14x17.2	M14x17.2	M14x17.2
Rail deflection (m)						
Permanent	0.41	1.0	0.71	0.03	0.07	0.9
Dynamic	NA	NA	0.87	0.24	0.31	1.4
Vehicle	1971 school bus	1971 school bus	1971 school bus	1976 Honda Civic	1975 Honda Civic	1962 GMC coach bus
Vehicle weight (kg)	9081	9095	9081	1032	956	14 515
Impact speed (km/h)	89.5	96.0	89.79	100.6	99.1	95.9
Impact angle (°)	13.5	15.0	15.0	15.0	18.0	14.0
Exit speed (km/h)	--[a]	--[a]	--[b]	89.0	79.8	--[b]
Exit angle (°)	--[a]	--[a]	--[b]	2.7	1.0	--[b]
Vehicle acceleration, maximum 0.050-s avg (g)						
Longitudinal	−1.13	−1.84	−1.13	−2.50	−3.10	−0.8
Transverse	−2.95	−2.45	−2.49	−7.35	−7.04	−2.4
Vertical	−1.35	−3.04	−0.85	2.43	1.74	

Notes: 1 m = 3.28 ft, 1 kg = 2.24 lb, 1 km/h = 0.62 mph, NA = not available.
Post = W6x8.5 steel, post spacing = 1.91 m (6.25 ft), and length of installation = 76.2 m (250 ft).
[a]Vehicle rolls.
[b]Undetermined.

Figure 4. Interaction of school bus and barrier at progressive stages of test 3.

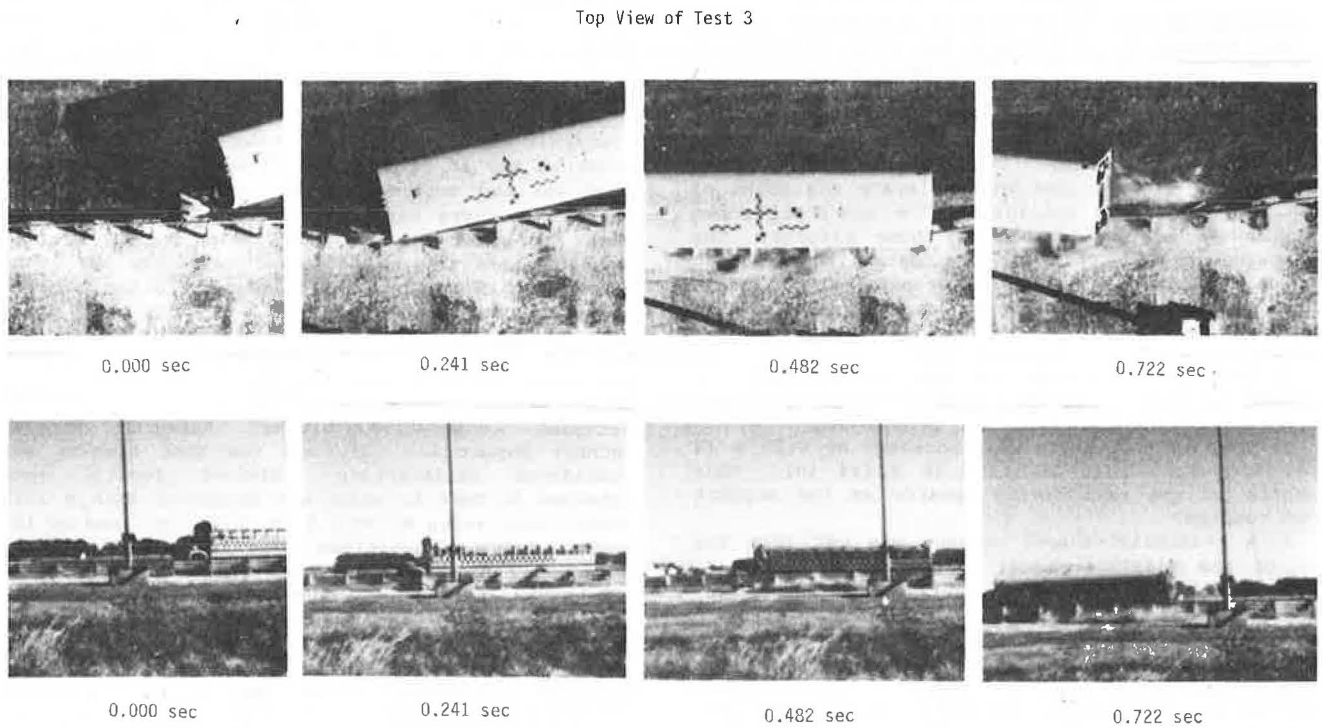
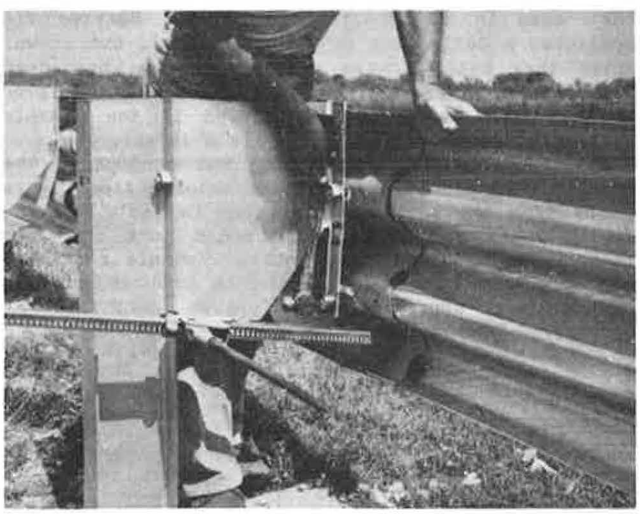


Figure 5. Restoring modified Thrie-beam guardrail after tests 4 and 5 with Honda Civic sedans.



1.4 m (4.6 ft). Eight posts were deformed by the left front wheel, but the rail remained intact and at a level suitable for redirection. The peak 0.050-s average lateral acceleration was 2.5 g. The corresponding longitudinal acceleration was only 0.8 g, which shows the relatively low forces exerted by the support posts on the left front wheel. Damage to the bus was modest; light sheet-metal damage occurred at the left front and left rear corners.

Even though the performance of the modified Thrie-beam guardrail proved to be a major advance in the performance of conventional rails, cost is always a critical factor when new systems are considered. At this stage, detailed cost-effectiveness analyses have not been conducted, but cost analyses of the three rail systems show a rather modest increase in cost for the modified Thrie-beam guardrail.

Table 3 gives cost estimates for three rail systems (conventional W-section, conventional Thrie-beam, and modified Thrie-beam) for three different installation lengths [less than 304.88 m (1000 ft), between 4573.17 and 9146.34 m (15 000 and 30 000 ft), and between 9146.34 and 30 487.8 m (30 000 and 100 000 ft)]. This comparison, which was based on costs from several prominent suppliers, fabricators, and contractors, shows a 25 percent increase from conventional W-section to modified Thrie-beam [$43.95-$54.78/m ($13.40-$16.70/ft)] and only a 3.4 percent increase from conventional Thrie-beam to modified Thrie-beam [$52.97-$54.78/m ($16.15-$16.70/ft)]. This is for placement of more than 9146.34 m of rail. The comparisons in Table 3 are not as good for smaller jobs but, considering the increased performance spectrum that results from including school and intercity buses, cost-effectiveness is considered likely. It should certainly be cost effective to step up from the conventional Thrie-beam system to the modified.

CONCLUSIONS

Conventional guardrail designs that use standard W-beam rails are not adequate to safely redirect school buses. The W-beam guardrail shown in Figure 2 and subjected to test 2 is representative of the best W-beam systems. Similar rails that have longer post spacings, shorter post-embedment lengths, lower rail heights, or are without blockouts would be expected to perform in an even less-acceptable manner.

The conventional Thrie-beam guardrail will perform marginally to contain and redirect school buses, but it is not likely to keep the bus upright during a collision. Although the 90° roll docu-

Figure 6. Interaction of intercity bus and barrier at progressive stages of test.

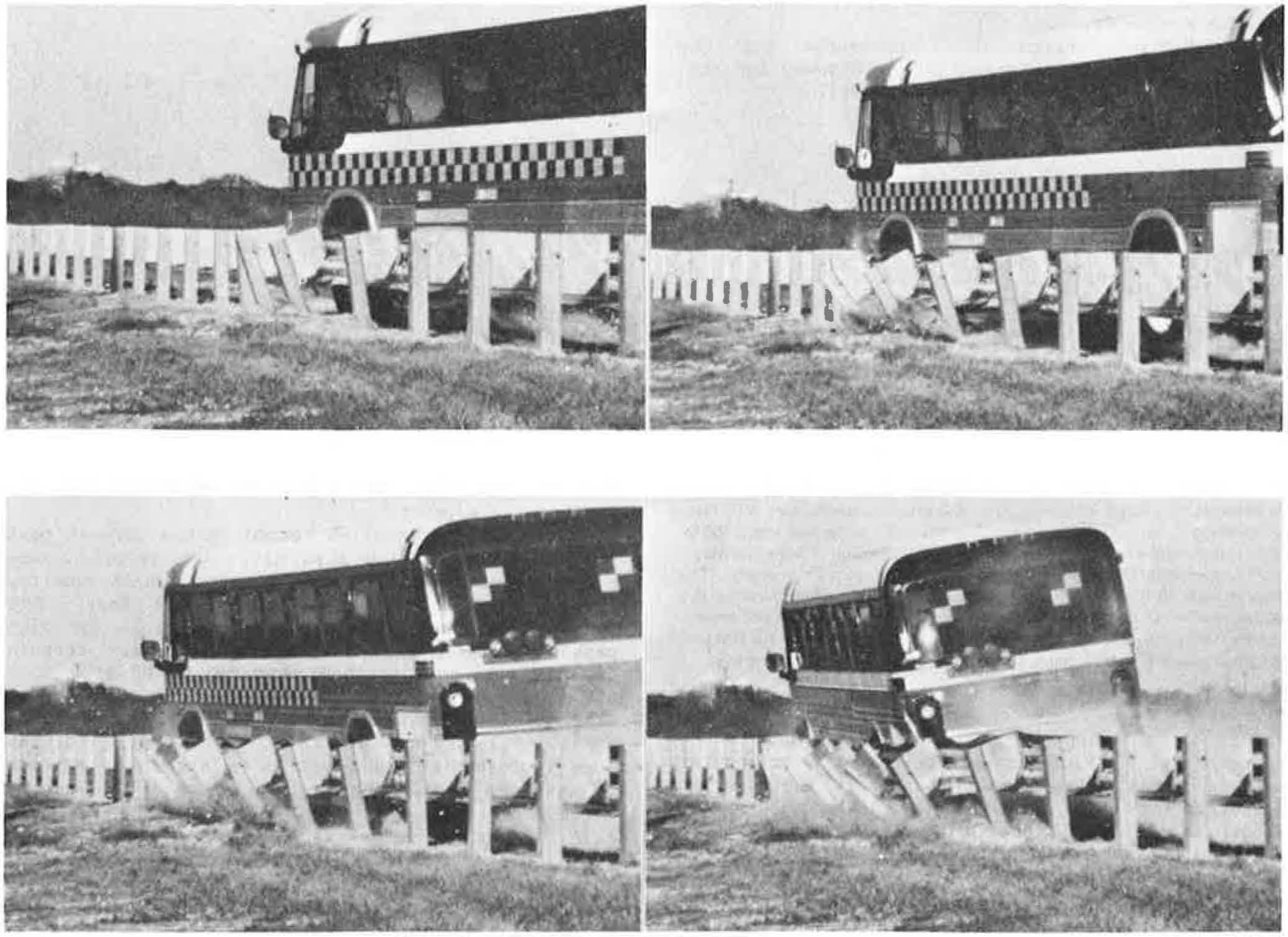


Table 3. Cost analysis for construction of Thrie-beam and W-beam guardrail systems.

Guardrail Type	Cost by Length of Installation ($/m)		
	Less than 304.88 m	4573.17-9146.34 m	9146.34-30 487.8 m
Conventional W-section[a]	54.61	48.88	43.95
Conventional Thrie-beam[a]	63.63	57.73	52.97
Modified Thrie-beam[b]	65.44	59.86	54.78

Note: 1 m = 3.28 ft.
[a]Performance good for automobiles only.
[b]Performance good for automobiles and school and intercity buses.

mented by test 2 was fairly slow and reasonably smooth, any roll that results in the bus ending up on its side is potentially hazardous. The conventional Thrie-beam guardrail does seem to be a significant improvement in performance over the conventional W-beam. If the redirection of heavier vehicles such as school buses becomes an accepted performance criterion, significant modifications of current guardrail systems will be necessary to ensure safe performance.

The modified Thrie-beam guardrail shown in Figure 3 performed well in test 3, the only school bus test to which it has been subjected. The 96.5-km/h (60-mph) tests with Honda Civic sedans at 15° and 18° have demonstrated that the increased rail height and the blockout modification, which allows the lower part of the Thrie-beam to bend inward, will not compromise the rail performance for mini-compact automobiles. No wheel or bumper snagging was observed during these tests.

The fact that the modified Thrie-beam rail functioned well in redirecting a 14 515-kg (32 000-lb) intercity bus illustrates the fact that Thrie-beam guardrails can be designed to accommodate a class of vehicles much larger than automobiles. Although cost-effectiveness has not been demonstrated for the usual highway situation that warrants guardrail, just as in the case of bridge rail there may be special situations where higher-performance guardrails such as the modified Thrie-beam could be justified. The development of warranting criteria for the use of higher-performance guardrail could produce improved highway safety.

ACKNOWLEDGMENT

The work discussed in this paper was sponsored by the Office of Research, FHWA. The opinions, findings, and conclusions expressed in this paper are ours and not necessarily those of the sponsor or other agencies with which we are affiliated.

REFERENCES

1. E. Asmussen and others. Roadside Safety Structures--A Description of the Crash Barriers Developed in the Netherlands. SWOV Institute for Road Safety, The Netherlands, Res. Rept. 1970-6, 1970.

2. M.E. Bronstad and others. Crash Testing Evaluation of Thrie-Beam Traffic Barriers. FHWA, Rept. FHWA-RD-75-509, Jan. 1975.
3. J.D. Michie. Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. NCHRP, Rept. 230, March 1981.

Notice: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names appear in this paper because they are considered essential to its object.

Abridgment

Crash Tests of Omnidirectional Slip-Base Sign Supports

KENNETH C. HAHN AND JAMES E. BRYDEN

Omnidirectional sign supports with triangular slip bases, which are similar to those successfully tested elsewhere on single-support appurtenances, were tested on multilegged sign installations. Four tests that were performed with 2150-lb vehicles determined compliance with American Association of State Highway and Transportation Officials specifications for vehicle momentum change. The supports were hit from two directions at two speeds, and each test resulted in a momentum change below 750 lb·s. In all the tests, vehicle damage and impact severity were light. The omnidirectional hinge design cannot hold the sign panel upright after one support is removed, but the entire design performs safely.

This study consisted of four full-scale crash tests to determine the impact performance of a triangular omnidirectional slip-base sign support that has an all-direction upper post hinge. [More information about these tests is provided elsewhere (1).] Testing details were taken from Transportation Research Circular 191 (2).

The support design (Figure 1) included base posts set in concrete, intermediate posts bolted to the base, and upper posts spliced to the intermediate posts (all W6x12 sections). The base posts, each topped by a triangular 1.5-in-thick plate, were set in 2-ft-diameter, 4-ft 9-in deep concrete foundations and had the plate top set flush with the ground line. Intermediate 8-ft-long posts that had matching triangular plates were attached to the bases, and three 6-in-long 1-1/8-in-diameter bolts were torqued to 110 lbf•ft. To permit the sign to be erected at 90° and 30° to the direction of vehicle travel, the left base plate was made circular rather than triangular and had two sets of three bolt slots offset by 60°. Two right bases were installed, also offset 60° from each other, so that the sign could thus be erected in either position. The 7-ft 6-in long upper posts were spliced to the intermediate posts with two 0.375-in-thick hinge plates. These plates were bolted to the drilled upper posts through holes and to the drilled intermediate posts through slots in the plates with 5/8-in bolts torqued to 170 lbf•ft for tests 29 and 30 and 190 lbf•ft for tests 31 and 32. An 8.5x16.5-ft (140 ft²) aluminum sign panel, which had three 2-3/8- by 1-1/4- by 3/16-in Z-bars, was mounted on the upper posts above the splice plates. The bottom of the panel was 7 ft above the ground. The Z-bars were attached to the sign panel with 1/4-in bolts on 16-in centers and to each post with two 1/4-in bolts.

During impact, the triangular plate on the intermediate post slips free of the base and, as the post rotates back, the splice plates bend to form a hinge. As bending continues, the bolts holding the slotted splice plate to the intermediate post pull free and the intermediate post is separated from the rest of the support.

The W6x12 post section tested is the largest post size to be used with this slip-base design. Successful tests of the W6x12 post would qualify smaller post sizes for use with this base. The two-support installation tested is typical for sign panels of up to 147 ft² erected on flat terrain and designed to withstand winds up to 80 mph (zone B). All of the bolt torques used initially were determined to be sufficient to withstand the loads developed by 80-mph winds. The hinge-bolt torques were increased for the last two tests in an attempt to keep the sign panel upright on a single support after impact.

All test vehicles were 1973 Chevrolet Vegas weighing approximately 2150 lb and speeds were near the 20- and 60-mph requirements. Vehicle test weights were reduced about 100 lb from the usual 2250 lb, recognizing that future test-weight requirements will be reduced. The actual test weights achieved could not be further reduced by using the vehicles available without extensive alterations. The impact angles were 90° and 30° to the sign face, which corresponds to a car traveling parallel to and at 60° to the pavement, respectively. Based on previous tests of triangular slip bases, these impact angles would produce the maximum vehicle velocity change and a reasonably expected impact condition for the roadway situations previously described.

RESULTS

Results of four full-scale crash tests of the omnidirectional slip-base sign support are summarized in Table 1.

In the first test (test 29), impact was perpendicular to the sign face at 27.7 mph and resulted in a 726-lb•s vehicle momentum. The slip-base bolts, torqued to 110 lbf•ft, separated on impact as designed, but the upper hinge bolts, torqued to 170 lbf•ft, remained in place and pulled the sign panel downward and backward and pitched the car -3° (upward) before the hinge released. The car traveled 11 ft during that period before the hinge released and traveled another 5 ft until the post flew free of the car.

The displaced sign panel then contacted the car roof. This secondary impact, which was directly over the front seat and about 1 ft to the right of center, resulted in a dent about 4 ft long, 3-7 in wide, and less than 1 in deep. This impact was not severe and presented no apparent hazard to vehicle

Figure 1. Omnidirectional slip-base sign support.

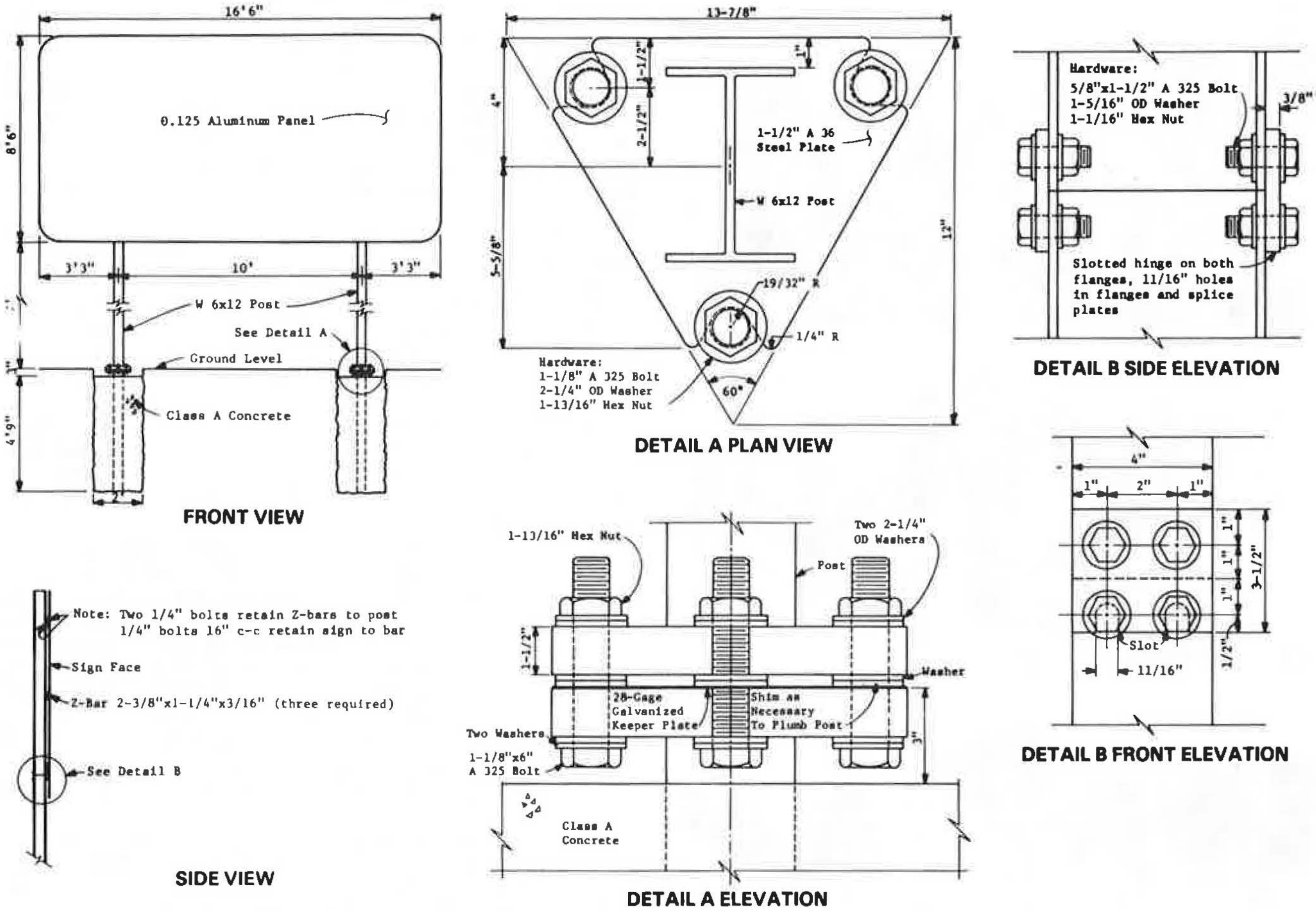


Table 1. Test results.

Measurement	Test 29	Test 30	Test 31	Test 32
Impact condition				
Speed (mph)	27.7	21.0	64.9	59.4
Angle (°)	90	30	30	90
Weight (lb)	2155	2130	2200	2160
Contact				
Time (ms)	489	257	78	85
Distance (ft)	16.0	8.5	6.8	6.8
Exit speed (mph)	20.3/19.8[a]	15.4	59.9	55.3
Exit angle (°)	90	30	30	90
Maximum yaw	0	0	0	0
Maximum roll	0	0	0	0
Maximum pitch	-3	-2	-2	-2
Momentum change (lb·s)	726/775[a]	543	501	403
Decelerations (*g*)				
50-ms avg				
Longitudinal	NA	NA	4.02	3.45
Lateral	NA	NA	1.89	1.26
Maximum peak				
Longitudinal	NA	NA	7.42	7.20
Lateral	NA	NA	9.24	6.72
Avg continuous				
Longitudinal	NA	NA	2.40	1.31
Lateral	NA	NA	0.42	0.42
Vehicle damage				
TAD	FC-3	FC-3	FC-4	FC-4
SAE	12FCEN9	12FCEN1	12FCEN2	12FCEN2

Note: TAD = Traffic Accident Data Project, SAE = Society of Automotive Engineers, and NA = not available.

[a]Result of secondary impact of sign panel and car roof after loss of contact with post.

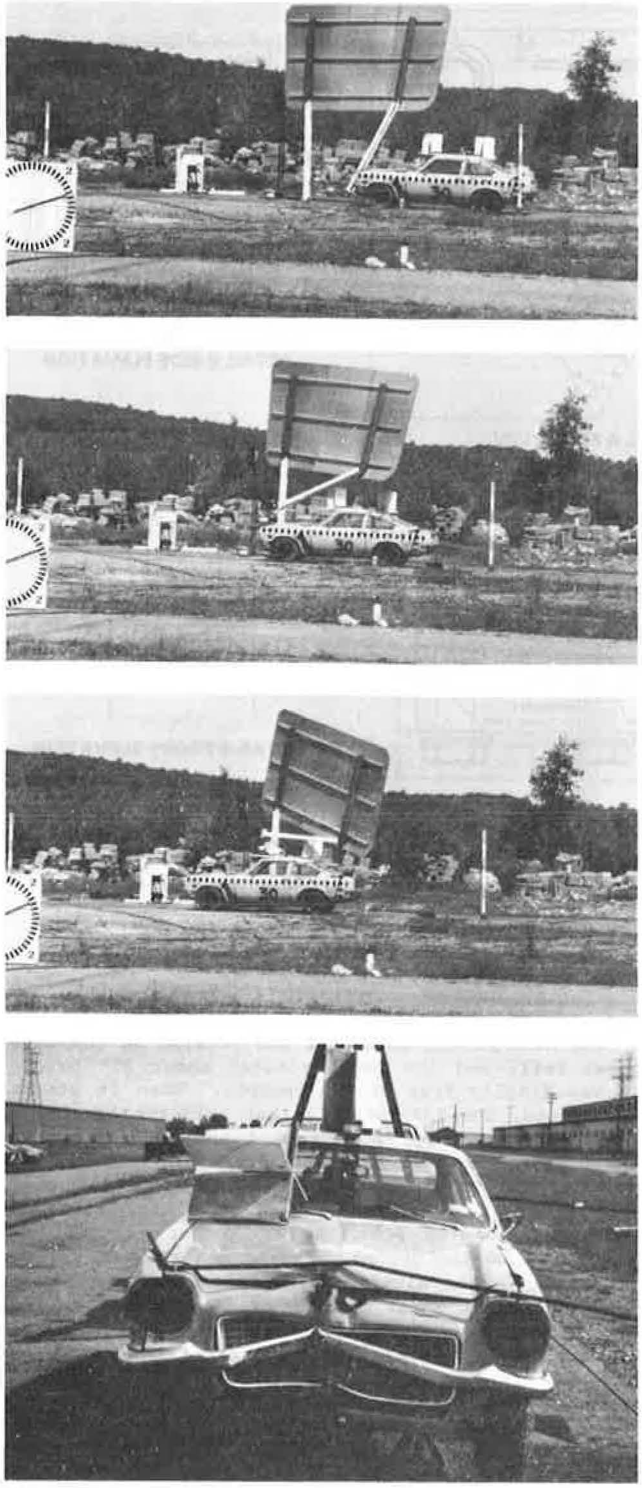
occupants, but an additional 49-lb·s momentum was lost.

The nonimpacted post bent and twisted as the sign panel fell, and the panel rotated about 80° before it was finally free of the vehicle. When it struck the ground, the 1/4-in bolts that held the lower two Z-bars to the upper post failed and left only the two 1/4-in bolts at the top Z-bar to support the panel. Both hinge plates on the nonimpacted post pulled free of the outboard bolts (those farthest from the impacted post) as the other end of the panel fell to the ground. Two of the riveted vertical seams in the aluminum sign panel were torn open below the first Z-bar because of the panel's twisting. These were later repaired with 1/4-in bolts for subsequent tests. The sign panel came to rest face down and rotated about 80° from its original position, and the nonimpacted support was still attached at both the base and the hinge.

The impacted post, which sustained a small dent at bumper height on the upstream flange, was thrown 45 ft from its base and about 8 ft to the right (away from traffic). The upper post remained undamaged and attached to the Z-bars. Both slotted splice plates pulled free from the intermediate post and remained attached to the upper post, although they were bent and not reusable.

Vehicle damage was limited to a 9-in-deep, 21-in-wide dent in the bumper, grill, and hood. The car's trajectory was unaffected by the impact and it exited along the same path on which it entered. Vehicle decelerations were not available because of equipment malfunction.

Figure 2. Typical impact sequence and vehicle damage resulting from test 30.



For the second test (test 30), impact was at 30° (nearly parallel to the sign panel) at 21.0 mph, which resulted in a 543-lb•s vehicle momentum change. Both base bolts were torqued to 110 lbf•ft and hinge bolts to 170 lbf•ft; they released as designed, i.e., the base bolts on impact and the hinge bolts after about 1.0 ft of vehicle travel. During that time, the sign bottom dropped only about 3 in and rotated back about 5° (see Figure 2).

After the hinge released, the intermediate post remained attached to the upper post by the right-hand bolt on the downstream splice plate. The sign rotated about 40° more after hinge release and came to rest still partly attached to the nonimpacted post; it was turned approximately 45° from its original position. The sign panel was undamaged except for the lower left corner being bent when it hit the ground. Both upper posts remained fully attached to the Z-bars. The nonimpacted intermediate post twisted slightly but was reusable for another test.

The impacted intermediate post sustained a small dent in the upstream flange at bumper height and bent flanges at the top of the intermediate section where the splice plates remained almost straight. None of the nonimpacted components sustained permanent damage, and the hinge release resulted from the slotted splice plates simply pulling free of the 5/8-in bolts; there was no bending of the plates.

Vehicle damage was limited to a 9-in-deep, 19-in-wide dent in the bumper, grill, and hood. The car did not deviate from its path and exited along the impact trajectory. Again, vehicle decelerations were not available.

For the third test (test 31), impact was again at 30° to the sign panel, and impact speed was 64.9 mph, which resulted in a 501-lb•s vehicle momentum change. The base bolts, torqued to 110 lbf•ft, released on impact. The hinge bolts were torqued to 190 lbf•ft in an attempt to keep the sign panel upright after one support was removed, but this increased torque did not appear to have an adverse effect on hinge release, which occurred about 7 ms after impact, before the sign panel could either rotate or drop, and after the car traveled about 0.6 ft.

After hinge release, the intermediate post remained partly attached to the upper post, then slipped free of the splice plates, and then flew end-over-end 125 ft downstream and 15 ft to the right. The sign dropped to the ground, bending one corner while rotating back about 25° from its original position. As in the previous lower-speed test, the upper posts remained fully attached to the Z-bars and there was no permanent damage to any of the nonimpacted components. Again, the impacted post sustained only a dent from the impact and bends in the flanges at the splice-plate bolt locations.

Vehicle damage was again limited to a large front-end dent. This time it was 12 in deep and 28 in across in the bumper, grill, and hood. However, damage to the fan and radiator prevented this car from being driven away. As in the previous tests, the impact resulted in no change in the path of the vehicle.

For the final test (test 32), a 0.25-in-diameter cable tether was attached to the top of the intermediate post and the bottom of the upper post to eliminate the flying post experienced in the previous high-speed test. The post was impacted at 90° (perpendicular to the sign panel) at 59.4 mph and had a 403-lb•s vehicle momentum change. The base bolts (torqued to 110 lbf•ft) released on impact, and the hinge bolts (torqued to 190 lbf•ft) released after about 0.6 ft of vehicle travel.

After the hinge released, the intermediate post remained partly attached to the upper post by the downstream splice plate. This attachment held until after loss of contact between the post and car, when the posts separated and the only remaining connection was the 0.25-in tether cable. The additional moving mass of the tethered intermediate post caused the sign to rotate about 110° from its original position, which was significantly more than if the intermediate post had been allowed to fly free after complete separation of the splice plates. The intermediate post remained tethered until both it and the sign panel hit the ground. The tether then snapped and the post bounced about 5 ft away to the right.

As in the previous tests, the upper posts remained fully attached to the Z-bars. The nonimpacted post was twisted 90° clockwise and bent about 18 in above the base plate. The sign panel's lower left corner was bent when it hit the ground and the lower right edge bent as it was folded against its support by the extreme rotation. The impacted support sustained a dent on the upstream flange where it was struck by the bumper and sustained bent flanges where it separated from the downstream splice plate.

Vehicle damage was again limited to a large dent in the front of the car, 12 in deep and 24 in wide in the bumper, grill, and hood. As in the previous high-speed test, it precluded driving the car from the scene after impact.

FINDINGS

All four impacts with the posts resulted in changes in vehicle momentum below the preferred 750 lb•s. Decelerations were tolerable in the two tests measured, and no violent vehicle reactions or abrupt changes of vehicle direction occurred. Impact was deliberately off center in the two high-speed impacts, but even then the vehicles exited on the same trajectories along which they entered. In all cases, the slip bases released as designed, but the lack of downstream flange continuity across the hinge (as in the one-direction design) prevented the sign panel from remaining upright on the nonimpacted support.

Based on these four tests, the following findings can be stated:

1. The omnidirectional sign support tested meets American Association of State Highway and Transportation Officials (AASHTO) criteria for momentum transfer, and all of the resulting momentum changes were below 750 lb•s;
2. Vehicle damage was light in all cases, and the lower-speed tests resulted in slightly lighter damage than the high-speed tests;
3. Off-center impact in the high-speed tests did not adversely affect vehicle trajectory or appurtenance performance;
4. The impacted posts were dented by the vehicle bumper, and the flange ends were bent at the hinge;
5. The nonimpacted posts sustained greater damage than the impacted ones because they were bent and twisted when the sign panels fell;
6. The sign panel sustained a bent lower left corner in each test when it hit the ground; during the first test, one of the riveted vertical seams separated due to twisting of the panel; and
7. The slotted splice plates on the nonimpacted post did not develop enough resistance to maintain the sign in an upright position.

ACKNOWLEDGMENT

The research reported in this paper was conducted in cooperation with the Federal Highway Administration, U.S. Department of Transportation.

REFERENCES

1. K.C. Hahn and J.E. Bryden. Crash Tests of Omnidirectional Slip-Base Sign Supports. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Res. Rept. 94, Oct. 1981.
2. Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances. TRB, Transportation Research Circular 191, Feb. 1978.

Guardrail Installation and Improvement Priorities

J. W. HALL

The methodology and findings of a detailed study of New Mexico traffic crashes involving impacts with guardrails, selected fixed objects, or overturning are described. Analysis of computerized accident records for 1978 and 1979 found that guardrail accidents were more often characterized by rural conditions, unfamiliar drivers, and snow-covered roads. Guardrail accidents tend to be less severe than other single-vehicle crashes. Field studies were conducted at the sites of 113 pairs of guardrail and nearby run-off-the-road crashes. Roadway geometrics were similar at both types of sites; both had significant downgrades and curvature to the left. Roadside slopes behind the guardrail did not differ significantly from front slopes at the run-off-the-road sites. Highly significant correlations were found among certain crash-site parameters. Average values of roadway and roadside characteristics at both types of crash sites were more adverse than for the roadway system in general. The research has developed a severity-reduction model that can be used to prioritize sites that warrant guardrail installation or upgrading.

The intent of guardrail is to reduce the severity of impact for motorists who have left the roadway. Guardrails should be designed to lessen the injury to occupants of vehicles that strike it and to safely redirect vehicles back to the roadway.

It is possible to learn something of guardrail use from accident records. Data from 1978 Fatal Accident Reporting System (FARS) records (1) indicate that more than half of the fatal accidents in the United States involve a single vehicle. Approximately 27 percent of all fatal accidents involve fixed objects, and of these approximately one-ninth involved vehicles that have struck guardrail. Another substantial component of the fatal-accident experience in this country involves noncollision accidents (primarily overturning), which account for approximately 11 percent of the fatal accidents nationwide.

In an attempt to determine the nature of the guardrail accident problem in New Mexico, an analysis was conducted of 1978 and 1979 New Mexico accident data. Of the 100 000 reported accidents during this two-year period, 22 percent of all accidents involved either fixed objects or overturning, and these accidents accounted for 42 percent of all fatal accidents. Guardrail was involved in 1.8 percent of the fatal accidents. The fixed-object and overturning accidents were grouped into three classes:

1. Guardrail--574,
2. Overturning--8037, and
3. Other fixed object--13 894.

The severity index for these accidents was highest for overturning accidents (0.53), an intermediate value for guardrail (0.37), and lowest for other fixed objects (0.28). The severity index for the other-fixed-objects class is misleadingly low because the category includes objects that are known to cause little or no injury, such as fences, fire hydrants, etc. On the other hand, those fixed objects that are more likely to be shielded by guardrail, including abutments, bridges, and culverts, all have severity indices higher than that reported for the guardrail accidents in New Mexico.

Contingency-table techniques were used to make comparisons among the 22 000 accidents that involved overturning and fixed objects in New Mexico. An analysis was conducted of various characteristics of these accidents as reported in the New Mexico accident record system. On the basis of the accident statistics, it was possible to draw a few conclusions about guardrail use and effectiveness in New Mexico. Guardrail accidents occur to a lesser extent in New Mexico than is reported nationwide. The guardrail accidents that do occur in New Mexico have moderate severity, which is consistent with that reported in other states. The accidents tend to be less severe than both overturning accidents and impacts with those fixed objects that are often shielded by guardrail. The guardrail accidents in New Mexico tend to be rural in nature, as suggested by the higher speeds and the dark, unlighted conditions under which many of them occur. With respect to the other single-vehicle, nonpedestrian accidents in New Mexico, the guardrail accidents show more involvement in snow and with unfamiliar drivers and less with use of alcohol. Horizontal curves were overrepresented at the overturning sites, whereas guardrail and other fixed objects showed no specific difference in this regard.

The findings from this analysis of the computerized accident record system are not conducive to engineering action. There is a suggestion that more guardrail needs to be used in New Mexico. It is also possible that the guardrail that is used could be improved to reduce the severity of accidents that occur. With these thoughts in mind, this study was designed to evaluate the potential for guardrail improvement in New Mexico. The primary objective of this study was to determine the relative priority of upgrading existing guardrail that does not meet current standards versus the installation of new guardrail at locations where it is not currently used. Clearly, both items are important because New Mexico has older guardrail as well as a number of locations where guardrail is warranted but not installed.

STUDY METHODOLOGY

In accord with the objectives of this research, a study procedure was developed that would evaluate the effect of current guardrail installations and that would also evaluate the merits of installing guardrail at additional locations. The study was restricted to the state highway system.

The study plan for this research called for a paired comparison of guardrail accidents with those accidents susceptible to severity reduction through guardrail use. To ensure the comparability of traffic volume as well as climatic and topographic conditions, a pair of nearby accident sites were selected on state-administered routes. Each pair consisted of a guardrail accident site and a nonguardrail accident site. The latter were the sites of overturning accidents or accidents with those fixed objects that might be susceptible to corrective action through proper guardrail use.

The 1978 and 1979 New Mexico accident record system was used together with a detailed sampling scheme to select pairs of nearby guardrail and certain types of run-off-the-road accidents. The 125 pairs of sites identified through this process were subsequently reduced to 113 when supposedly valid sites could not be located on the photologs or in field site investigations. Sites were located in 22 of New Mexico's counties on 34 different highway routes. With the exception of severity, which was used as a partial criterion for choosing sites, this set of accidents exhibits characteristics similar to those for all guardrail, overturning, and fixed-object accidents. The similarity of the characteristics of the sample and the population suggests (but does not prove) the absence of bias in the site-selection process. Among the accident sites with guardrail, 28 percent involved guardrail at bridges. Accidents involving culverts and embankments each accounted for 17 percent of the nonguardrail sites, while the remaining sites involved overturning or accidents with abutments, bridges, and ditches.

The objectives of this study suggest that several types of data should be collected and analyzed. With respect to guardrail crashes, it is necessary to know the characteristics of the guardrail--specifically, its height, type, terminal treatment, and the effectiveness with which it performed in the crash. Other data requirements for both guardrail and nonguardrail sites include roadway geometries and the nature of the roadside. A description of the specific measurement procedures is contained in the project report (2).

The extensive data sets collected in the field were supplemented with data from the investigating officer's report and from New Mexico State Highway Department files. The data were processed by using standard computer programs.

ANALYSIS OF DATA

The mean values of the geometric characteristics in the vicinities of both the guardrail and nonguardrail crash sites are summarized in Table 1. The maximum degree of curvature at the guardrail sites exceeds the corresponding value at the nonguardrail sites. On the other hand, the minimum degree of curvature is slightly less at the guardrail sites. The mean degree of curvature, which is the average value at each site of the curvature at the 10 measurement positions, is essentially the same at both the guardrail and nonguardrail sites. A similar condition exists for the approach degree of curvature, which is the mean value of the degree of curvature in the area immediately upstream of the crash site.

A further analysis of the data made use of a signed degree of curvature. For the purpose of this analysis, curves to the left were assigned a positive sign, while those to the right were assigned a negative sign. Although the signing convention is arbitrary, it does serve to distinguish between the direction of curvature. As shown in Table 1, the nonguardrail sites have a more pronounced curvature to the left. This is true for both the average of all 10 curvature readings and for the curvature in the area immediately upstream of the crash sites. Although it may be obvious, it is worth noting that

Table 1. Summary of mean values of geometric characteristics: guardrail and nonguardrail crash sites.

Item	Mean Values	
	Guardrail	Nonguardrail
Degree of curvature		
Maximum	3.61	2.75
Minimum	0.15	0.22
Mean[a]	1.41	1.44
Approach[b]	1.58	1.51
Mean signed[a,c]	0.18	0.42
Approach signed[b,c]	0.27	0.42
Superelevation (%)		
Maximum	3.70	3.87
Minimum	0.91	0.84
Mean[d]	2.31	2.42
Approach[b]	2.27	2.43
Gradient (%)		
Maximum	0.71	0.66
Minimum	−1.82	−1.42
Mean[e]	−0.58	−0.40
Approach[b]	−0.74	−0.47

[a]Mean of 10 curvature values.
[b]Mean value in the area immediately upstream of the site.
[c]Sign convention: + for left curves, - for right curves.
[d]Mean of 10 superelevation values.
[e]Mean of 11 gradient values.

Table 2. Site characteristics.

Item	Mean Values	
	Guardrail	Nonguardrail
Pavement width (ft)	24.6	24.6
No. of lanes	3.0	3.0
Shoulder width (ft)	5.8	6.8
Continuous downhill distance (ft)	1100	1400
Speed limit (mph)	52.0	52.4
Traffic volume[a]	5400	5400
Pavement friction	0.75	0.75
No. of driveways	0.57	0.57
No. of intersections	0.10	0.03
No. of spot-fixed objects	2.5	2.2
Length of continuous-fixed objects (ft)	400	330
Shoulder slope (%)	1.59	2.04
Front slope (%)	4.2	18.3[b]
Back slope (%)	21.0	0.8[b]
Embankment depth (ft)	35.8	23.5[b]

[a]Volume of average daily traffic (ADT).
[b]Statistically significant difference at $\alpha = 0.05$.

a driver has less recovery room on a curve to the left than one to the right.

Analysis of the superelevation data showed no differences between the guardrail and nonguardrail sites. Superelevation was assigned an algebraic sign of + if it was proper for the particular location, i.e., a normal crown section on a tangent and proper superelevation on the curve. The gradient data given in Table 1 indicate that the guardrail sites tend to be on steeper downgrades than the nonguardrail sites. The minimum, mean, and approach gradients are all less in the case of the guardrail crash sites.

As might be expected, all of the parameters shown in Table 1 have rather high standard deviations, i.e., the curvature values had an extreme range and included sharp curvature as well as many tangent sections. Because of the high standard deviation of the data, none of the differences suggested in the summary characteristics are statistically significant. Additional characteristics at the study sites are indicated in Table 2. These characteristics relate to the width of the roadways, roadside slope information, and the presence of driveways and intersections. The roadway width factors are of obvious importance, while the last two would logically be limited by the installation of guardrail. The pavement width, which was the total pavement width for undivided highways and the one-directional roadway width for divided highways, was virtually the same at both sites. Shoulder widths were somewhat higher at the nonguardrail sites. The length of the continuous downhill distance in advance of the crash site was higher at the nonguardrail sites.

Most of the crashes occurred on high-speed rural highways that have a median speed limit of 55 mph. Because of the proximity of the paired guardrail and run-off-the-road crash sites, it was not possible to distinguish the traffic volumes at the two types of sites, and therefore the average volumes at both were the same. Pavement friction, which was measured with a 70-lb drag tester, had identical values at guardrail and nonguardrail sites.

As shown in Table 2, there is no difference in the number of driveways between the guardrail and nonguardrail sites. There are approximately three times as many intersections at the guardrail sites as at the nonguardrail sites, but the number of intersections in total is quite small, and the apparent difference is not statistically significant. The number of spot-fixed objects, which included trees, poles, and large rocks, is virtually the same at both the guardrail and nonguardrail sites. The length of continuous-fixed objects was approximately 400 ft at the guardrail sites and 330 ft at the nonguardrail sites. The continuous-fixed objects within this grouping included the length of guardrail that, on average, was five times longer at the guardrail sites.

The slope of the shoulder at the nonguardrail sites averaged 2 percent, while at the guardrail sites the slope was approximately 1.5 percent. The difference is not statistically significant. A principal factor in distinguishing between the guardrail and nonguardrail sites was the roadside slope characteristics. The front slope, which was measured immediately beyond the shoulder, is significantly higher at the nonguardrail sites than at the guardrail sites. The positive sign associated with the slopes indicates a fill or embankment type of slope. The back slope was measured in the area behind the guardrail in the case of guardrail sites and at the point where the slope changed to a cut slope or leveled out in the case of the nonguardrail site. Not surprisingly, the back slope was significantly higher at the guardrail sites, although it does not differ from the front slope at the nonguardrail sites. The depth of embankment was 35 ft at the guardrail sites versus only 23 ft at the nonguardrail sites.

The general characteristics of the guardrail at the crash sites are summarized in Table 3. The principal type of guardrail, which accounted for nearly two-thirds of all the guardrail crash sites, was the blocked-out W-beam, while a quarter of the guardrail was of the non-blocked-out W-beam type. The principal terminal type at the guardrail crash sites was the buried-end type of terminal, which is normally accompanied by a guardrail flare. The breakaway cable terminal type and the old style nonburied terminal type each accounted for approximately a quarter of the sample.

The field crew attempted to identify the reason for the installation of the guardrail at the crash sites. In some cases there were multiple reasons for guardrail installation. Bridge approaches and embankments each accounted for 31 percent of the guardrail installations. Rivers, culverts, and other purposes each accounted for approximately 10 percent. The average height of guardrail was 2.23 ft above ground level. However, a significant range in heights was found; some rails were as low as 1.2 ft and others as high as 3.2 ft. In general, older installations tended to have lower heights.

An analysis was made of the severity of accidents involving different types of guardrail. The results, presented in Table 4, are shown for the type of guardrail, the terminal type, and the purpose for which the guardrail was installed. Twelve of the accidents that involved guardrail resulted in fatalities, 75 resulted in injuries, and 26 resulted in property-damage-only (PDO) accidents. Contingency-table analysis showed that crash severity was independent of both guardrail type and purpose, although crash severity for guardrail at bridge approaches was slightly higher than expected. Severity was also found to be independent of terminal type, a finding that needs to be interpreted carefully, since most crashes did not involve a direct hit on the terminal.

The independence of guardrail type and crash severity is surprising, especially since laboratory tests have shown that certain types of guardrail perform better than others. It must be noted, however, that guardrail tests are typically performed under extreme conditions with respect to speed and impact angle and these conditions are often not met in real-world guardrail crashes. Although there is good reason to believe that the blocked-out W-beam design provides a safer environment for impacting vehicles, the independence of crash severity and guardrail type indicates that there may be other factors that are of equal or greater importance than guardrail type alone in projecting guardrail crash severity.

Correlation analyses were conducted among the parameters at both the guardrail and nonguardrail sites. Several interesting findings from these analyses include the following:

1. At the guardrail sites, locations with sharper curvature to the left tend to have flatter side slopes, while the opposite was true at the nonguardrail sites;
2. At both types of sites, embankment heights were greater on steeper downgrades;
3. As might be expected, both types of sites showed traffic volume to be positively correlated with pavement width, shoulder width, and speed limit;
4. Higher guardrail heights were negatively correlated with crash severity; and
5. At both types of sites, front slopes were positively correlated with severity.

Because other research (3) has suggested a relation between vertical and horizontal alignment and crash occurrence, an analysis was conducted to examine what relation, if any, existed between the roadway curvature and gradient at the crash sites. The curvature used in this analysis was the approach curvature from 250 through 50 ft before the site. The gradient was the average value of the gradient from 200 ft before the site to the site itself. Average values of curvature in excess of 2 degrees were categorized as sharp, while those between 0.1 and 2 degrees were categorized as gentle.

The results of the contingency-table analysis of the combined horizontal and vertical curvature for guardrail and nonguardrail crash sites are presented in Table 5. Statistical testing indicated that, at both guardrail and nonguardrail sites, curvature and gradient are not independent. The small number of observations in certain cells of the table detract from the statistical significance of this finding. However, the table clearly indicates the excess number of crashes at both the guardrail and nonguardrail sites that occur on curves to the left. Testing did indicate that, at both types of sites, crashes occurred on downgrades at curves at approximately twice the statistically expected level.

An attempt was made to relate observed crash severity to roadway and roadside parameters by using multiple regression techniques. The equations that were developed explained only a small portion of the observed variation. The discrete nature of the severity scale (fatal, injury, and PDO), coupled with the fact that many parameters not measured in this study can contribute to crash severity, led to the abandonment of this approach.

Discriminant analysis was used to determine if a

Table 3. Guardrail characteristics at study sites.

Item	Value
Type of guardrail (%)	
Blocked-out W-beam	64.9
Regular W-beam	23.4
Thrie beam	6.3
Cable	2.7
U-channel	2.7
Terminal type (%)	
Buried	39.0
Breakaway	26.5
Old style	24.8
Not applicable	9.7
Purpose for installing guardrail (%)	
Bridge approach	31.0
Embankment	31.0
River	10.6
Culvert	9.7
Other	8.8
Overpass or underpass	7.1
Ditch	1.8
Avg guardrail height (ft)	2.23
Range of heights (ft)	1.2-3.2

Table 4. Crash severity versus guardrail characteristics.

	No. of Accidents		
Item	Fatal	Injury	PDO
Type of guardrail			
Blocked-out W-beam	7	52	13
Regular W-beam	4	14	8
Thrie beam	1	4	2
Other	0	5	3
Terminal type			
Buried	4	29	11
Breakaway	2	22	6
Old style	3	17	8
Not applicable	3	7	1
Purpose			
Bridge approach	5	26	4
Embankment	3	19	13
River	1	7	4
Culvert	0	10	3
Other	3	13	2

Table 5. Combined effect of approach curvature and approach gradient.

	Curvature[a]				
Gradient (%)	Sharp Right	Gentle Right	Nearly Tangent	Gentle Left	Sharp Left
Guardrail sites					
>-2	3	6	71	6	8
<-2	2	3	6	4	4
Run-off-the-road sites					
>-2	2	7	61	9	8
<-2	3	0	11	6	6

[a]Tangents have a curvature of less than 0.1 degree, gentle curves between 0.1 and 2.0 degrees, and sharp curves greater than 2.0 degrees.

set of independent variables could be used to establish the classification of the dependent variable--crash severity. The independent variables used in the discriminant analysis to predict the severity of guardrail crashes were approach degree of curvature and gradient, roadside slope, guardrail height, traffic volume, pavement width and friction, shoulder width, and posted speed limit. All of these parameters except guardrail height and posted speed limit were also used in the discriminant analysis at the nonguardrail crash sites.

The results, summarized in the table below, show the actual severity of the accidents versus the severity based on the discriminant analysis:

	Classified Severity		
Actual Severity	Fatal	Injury	PDO
Guardrail crash sites			
Fatal	10	2	0
Injury	3	66	2
PDO	0	10	12
Nonguardrail crash sites			
Fatal	12	6	1
Injury	7	53	15
PDO	0	0	13

It is not possible to use the entire set of study sites because one or more data items might be missing at a particular site. In the case of the guardrail sites, the discriminant analysis properly classified 84 percent of the accidents according to their actual severity. The analysis for the nonguardrail sites properly classified 73 percent of the crash severities. Recognizing that the discriminant analysis does not give direct consideration to many nonhighway factors (such as vehicle type and actual speed at time of collision), the discriminant analysis does a good job of distinguishing crash severity on the basis of the selected roadway and roadside parameters. A subsequent analysis found that two factors that can influence crash severity--vehicle occupancy and safety belt use--reportedly were not significantly different for the guardrail and run-off-the-road crashes.

The discriminant analysis was also applied to the entire data set to determine the feasibility of distinguishing the type of site on the basis of selected crash-site characteristics. The characteristics used for this purpose were the approach degree of curvature and gradient, the roadside slope, and the shoulder width. These variables were employed to create a model that would classify a site on the basis of these characteristics as either a guardrail or a run-off-the-road crash. Good success was obtained with the data from the guardrail sites, where 90 percent of the sites were properly classified on the basis of these four variables. On the other hand, only 62 of the nonguardrail sites were properly classified, while the remainder were erroneously classified as having characteristics more similar to those of the guardrail sites. This finding is important because it indicates that, despite the differences among guardrail sites, the sites are generally similar enough to be properly classified. On the other hand, nearly half of the nonguardrail sites exhibit roadway and roadside characteristics that are more similar to those at the guardrail sites. On a statistical basis, as opposed to an engineering design basis, nearly half of the nonguardrail sites should have had guardrail installed. The results of the discriminant analysis of sites are summarized in the table below:

	Classified Site	
Actual Site	Guardrail	Run-off-the-Road
Guardrail	102	11
Run-off-the-road	51	62

Several differences between the characteristics at guardrail and nonguardrail sites have been cited. It is also important to note that both sites have different characteristics than the roadway system in general. Although an extensive sampling procedure would be necessary to thoroughly describe the system characteristics, it is possible to reach some general conclusions through logic and limited sampling. Intuition suggests that there are as many curves to the right as to the left, and if the directions of curvature are assigned opposite algebraic signs, then the average curvature on the road is zero. Also, for every upgrade there is a corresponding downgrade, and the result is that the average gradient is zero. A recent study (4) supports both of these conclusions, as well as finding that the average roadside slope is approximately 14 percent.

Data from this study show, however, that the sites of both types of crashes have significant curvature to the left. It was also found that the average gradient at the guardrail sites was significantly less than zero. The roadside slopes, which had average values of 18 percent at the nonguardrail sites and 21 percent behind the guardrail, are also steeper than for the roadway system in general. In other words, the roadways and roadsides at these sites are significantly different than the typical roadway. The relevance of this to the engineer is that the crashes are considerably more frequent at locations with adverse geometrics and that the engineer has a basis for selecting sites for corrective action.

PRIORITIZING GUARDRAIL IMPROVEMENTS

The numerous findings reported in this paper, which for the sake of brevity will not be recounted, form a partial basis for developing a guardrail improvement program that would assign the proper weights to the upgrading of existing guardrail versus the installation of new guardrail. Recognition must be given, however, to the following facts:

1. When normalized for vehicle travel or highway mileage, New Mexico currently uses less guardrail than most other states, and
2. For the most part, existing guardrail in New Mexico appears to be performing satisfactorily.

As those familiar with guardrail use are aware, the installation of guardrail is warranted by the engineer's inability, due to economic or physical constraints, to provide a clear, traversible roadside. Guardrail is a fixed object and will not prevent accidents. Its only proven benefit is that, when properly installed, it can reduce severity. Priorities must therefore be based on the severity-reduction potential of upgrading versus new installation.

From a practical viewpoint, severity reduction actually embodies two concepts. In one instance it refers to the decrease in crash severity from fatal or injury to PDO. Recognizing that the difference between fatal and injury accidents is primarily one of degree and luck rather than substance, and that a PDO accident is the best result that guardrail can produce, the proper perspective for judging this component of severity reduction is the potential to change accidents from injury to PDO crashes. The second aspect of severity reduction deals with the potential for an improvement at a specific location to reduce the statewide number of injury accidents involving guardrail. Superficially, this second component would appear to be directly related to traffic volume, since higher volume moving past a particular site would seem to increase the likelihood of impact with a guardrail. However, data from this and other studies clearly show that the probability of a vehicle departing from the traveled roadway is related to roadway characteristics--specifically, curvature and gradient. This is also supported by accident rate data, which are not the same for all road systems. The fact is that geometrics are more likely to be worse on non-Interstate facilities, thus increasing the probability of a vehicle running off the road. This is indicated by the data base of more than 3000 guardrail, fatal, injury overturning, and selected fixed-object accidents from which the study sites for this project were selected, which shows that the rates for these types of accidents are 27 percent higher on the federal-aid primary system and 40 percent higher on the federal-aid secondary system than on the Interstate system. The data also show that the severity indices for these crashes are identical on the three types of roadway systems. Although it is therefore appropriate to consider traffic volume in setting priorities, it is necessary to make an adjustment on the basis of the roadway system.

Numerous studies support the concept that speed at impact is related to crash severity. At the same time, the engineer has negligible control over speed at impact and, for the rural state highways examined in this study, there is little evidence that the vehicular speed at impact is related to the posted speed limit. The complete absence of a significant correlation between posted speed limit and crash severity at both the guardrail and nonguardrail sites, which can be attributed partly to the 55-mph speed limit at most of the study sites, supports this concept. Thus, while corrective action involving the upgrading or installation of guardrail may be concentrated on higher-speed facilities, the justification for such action is not that these facilities have higher speed limits.

A literal interpretation of the data from this study suggests that the top priority for improvement should be given to the highest rate-adjusted volume location on the steepest downgrade that has the sharpest curve to the left and the steepest side slope. At this location, wherever it may be, the existing guardrail should meet or exceed American Association of State Highway and Transportation Officials (AASHTO) guardrail standards (5) or new guardrail should be installed. The chances are that if this location exists, it is already adequately shielded. The more likely situation is that this condition does not exist, but rather that there are separate locations with highest volume, steepest downgrade, etc. The problem is further complicated by the fact that certain parameters that the engineer is not likely to measure, such as the extent of roadway use by nonfamiliar drivers, affect the probability of a vehicle leaving the roadway and thus influence the potential merits of an improvement.

A factor that increases the difficulty of establishing priorities is that 62 percent of the guardrail crashes and 54 percent of the nonguardrail crash sites are at tangent locations that are characterized by upgrades or minor downgrades. However, this set of design characteristics exists on more than 75 percent of the rural New Mexico state highway system. Thus, while these good design characteristics are found at a substantial number of crash sites, they are actually underrepresented with respect to their share of the roadway system. It is difficult to conceive of a priority scheme that would emphasize these locations for either upgrading or the installation of new guardrail.

The New Mexico State Highway Department, through its inventory, can identify all of its existing guardrail installations, classified by design adequacy. The Highway Department can identify locations that meet current AASHTO standards for the installation of new guardrail. There is good reason to assume that the existing warrants, as they relate to embankment depth and slope, are conservative, in that they do not require the use of guardrail at locations with obviously hazardous conditions. A more realistic set of warrants would clearly increase the number of potential sites to be prioritized.

The priority for selecting sites for improvement will be established by using a severity-reduction index calculated on the basis of three parameters: adjusted volume, potential for severity reduction, and likelihood of vehicle departure. The product of these three factors is a measure of the increased safety provided at a location through the installation or upgrading of guardrail to current standards.

The importance of traffic volume, adjusted for the rate of run-off-the-road types of accidents on the various types of facilities, has been previously noted. The roadway system factor (R) in the index is established from the average daily traffic (V) at the site, as follows:

$R = V$ for Interstate highways,
$R = 1.27V$ for federal-aid primary highways, and
$R = 1.40V$ for federal-aid secondary highways.

The severity reduction potential of a guardrail installation is established on the basis that a properly designed and installed guardrail might achieve a severity index of 0.30, which is the lower limit of values reported in the technical literature. Analysis of New Mexico's accident data showed that guardrail of the older design currently has a severity index of approximately 0.4, while guardrail at bridges has a severity index of 0.45. Similar analyses determined the severity indices for accidents involving ditches (0.40), culverts (0.50), embankments (0.51), and abutments (0.60). This information was used to establish the severity-reduction factor (S), as follows:

Type of Improvement	S
Upgrade deficient guardrail	0.10
New guardrail at ditch	0.10
Upgrade guardrail at bridge	0.15
New guardrail at culvert	0.20
New guardrail at embankment	0.21
New guardrail at abutment or bridge	0.30

The third factor in the index is based on geometrics and reflects the relative probability of running off the road under various conditions of roadway alignment. The factor was established on the basis of the ratio of curvature and alignment conditions found at the study sites to the estimated extent of similar characteristics on the remainder of the roadway system. The ratios were normalized to a base value of unity. The alignment factor (A) is as follows:

	Curvature			
Gradient (%)	2°R-0.1°L	0.1°L-2°L	>2°R	>2°L
>-2	1.0	1.3	2.2	7.1
<-2	1.0	1.8	7.5	15.0

These three factors are used to calculate a severity-reduction index. The index is given by $SRI = RSA/250$, where SRI is the severity-reduction index (the other parameters are described above), and 250 normalizes the index to a range of 0 to approximately 100.

The index would be used in the following manner.

Initially, a set of locations would be identified where a guardrail is warranted or where existing guardrail is not in accord with current design standards. The limited data requirements for the index calculation can be met with existing records systems. The index could be calculated for each site, and highest priority would be given to sites with the highest values.

The procedure was applied to data collected in this study. This approach is not completely valid, since conditions at some of the nonguardrail crash sites do not meet AASHTO guardrail warrants. The resulting hazard indices ranged from 0 to 89. Although the median index value is 3.2, 25 percent of the sites have index values in excess of 7, and these are the locations needing the most immediate attention. Although the actual severity of individual crashes is not used directly in the model, the correlation between the calculated index and actual severity is positive, which suggests that the locations of more serious accidents tend to have higher indices. As a group, the nonguardrail sites had a significantly higher mean index (9.0) than the guardrail sites (3.3). This implies that more attention would initially be given to the nonguardrail sites. Among the sites with indices in excess of 7, only 27 percent were the locations that currently have guardrail. Bias in the sample used for this application of the model, which was due to the inclusion of some run-off-the-road sites that do not warrant guardrail, may be responsible for this result. The common characteristic of most of the guardrail sites with high indices is poor terminal treatment. W-beams without blockouts also tend to have higher indices. Cable guardrail sites tended to have low index values, principally because these locations had low traffic volumes. Sites with the highest indices are distributed proportionately among Interstate, primary, and secondary roadway systems, which indicates that concentration of improvements on one type of system would not be appropriate.

CONCLUSION

The research discussed in this paper has developed a rational and justifiable methodology for distributing funds for improvement between guardrail installation and upgrading. The merit of this approach is that it has the potential to achieve a high severity reduction for guardrail and selected run-off-the-road crashes under the constraint of a moderate funding level. It is recognized, however, that the optimal solution to the specific issue of new versus upgraded guardrail is not necessarily a component of a plan for the most cost-effective expenditure of limited highway monies. A well-conceived priority scheme for all highway improvements clearly needs to address issues and project types that were outside the scope of this project.

ACKNOWLEDGMENT

The research described in this paper was supported by the New Mexico Traffic Safety Bureau.

REFERENCES

1. Fatal Accident Reporting System, Annual Report. National Highway Traffic Safety Administration, U.S. Department of Transportation, 1978.
2. J.W. Hall. Priorities for Guardrail Installation and Improvement. FHWA, Final Rept. FHWA-NMDOT-80-1, Nov. 1980.
3. Roy Jorgenson Associates, Inc. Cost and Safety Effectiveness of Highway Design Elements. NCHRP, Rept. 197, 1978.
4. J.W. Hall and P. Zador. A Survey of Single-Vehicle Fatal Rollover Crash Sites in New Mexico. New Mexico State Highway Department, Santa Fe, Nov. 1980.
5. Guide for Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, DC, 1977.