TRANSPORTATION RESEARCH RECORD

No. 1258

Highway and Facility Design

Roadside Safety 1990

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1990

Transportation Research Record 1258
Price: $21.00

Subscriber Category
IIA highway and facility design

Mode
1 highway transportation

Subject Areas
21 facilities design
51 transportation safety

Printed in the United States of America

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

Roadside safety 1990.
p. cm.—(Transportation research record, ISSN 0361-1981 ; 1258)
ISBN 0-309-05007-3
1. Traffic signs and signals—Break-away supports—Testing. 2. Bridges—Railings—Testing. 3. Roads—Guard fences—Testing. 4. Median strips. I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1258
[TE228]
388 s—dc20
[625.7'95] 90-38193
CIP

Sponsorship of Transportation Research Record 1258

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
Chairman: Raymond A. Forsyth, Sacramento, California

General Design Section
Chairman: Jarvis D. Michie, Dynatech Engineering, Inc.

Committee on Roadside Safety Features
Chairman: William W. Hunter, University of North Carolina
Robert F. Baker, Maurice E. Bronstad, James E. Bryden, Ronald M. Canner, Jr., John F. Carney III, Duane O. Christensen, Julie Anna Cirillo, Arthur M. Dinitz, John C. Durkos, Michael D. Freitas, James H. Hatton, Jr., T. Heijer, John A. Hinch, Ivor B. Laker, Mark A. Marek, William G. Marley, Jr., John W. Melvin, David Pope, Edward Robert Post, Robert Quincy, James F. Roberts, Hayes E. Ross, Jr., Rudolph Kenneth Shearin, Jr., Roger L. Stoughton, Flory J. Tamanini, Harry W. Taylor, Thomas Turbell

George W. Ring III, Transportation Research Board staff

The organizational units, officers, and members are as of December 31, 1989.

Transportation Research Record 1258

Contents

Foreword

Of the 10 papers in this Record, 2 are concerned with safety aspects of sign supports, 4 with bridge rails and bridge rail transitions, 3 with traffic barriers, and 1 with a median treatment. The papers will be of interest to engineers concerned with highway safety, traffic operations, bridges, and geometrics. Brief descriptions of the scopes of the papers are as follows:

Davis examines lightweight signs mounted on work zone Type III breakaway barriers for durability under wind loads up to 60 mph and safety when subjected to vehicle crash tests at 20 and 60 mph. Vinyl roll-up signs and 0.024-in. aluminum signs were recommended for implementation.

Using impact test data, Breaux and Morgan confirm that predictions of breakaway sign performance made by using change of kinetic energy calculations (changes in velocity) are reasonably accurate and useful for estimating the margin of safety in sign support systems.

Hirsch and Romere report on 1,918- and 4,400-lb cars crash-tested into a modified Texas C202 bridge rail designed to redirect an 80,000-lb tractor-trailer striking at 50 mph at a 15-degree angle. The tests were reported to be successful.

Buth et al. describe the development and crash testing of four different types of bridge rails designed to meet Performance Level 2 requirements.

Hirsch et al. present the successful crash test results of a new type of reinforced concrete bridge rail that was selected for aesthetic attractiveness as well as structural adequacy from 22 different designs.

Mak et al. describe four crash tests using either 1,880- or 4,500-lb vehicles to evaluate the safety aspects of the Wyoming tube-type bridge rail. The tests showed that the railing meets the guidelines set forth in *NCHRP Report 230*.

Mak and Sicking made a literature study of vehicle rollovers associated with impacts with concrete safety-shaped barriers. Their findings resulted in further study consisting of computer simulations of other barrier shapes. The studies suggest that a barrier with a constant slope face will reduce rollovers and also be more adaptable to applications of overlays.

Hirsch and Mak developed and tested a portland cement–stabilized, sand-filled MK-7 barrier that contained and smoothly redirected an 80,000-lb tractor-trailer.

Glauz reports on four crash tests of a movable concrete barrier using 4,300- and 2,000-lb cars. The crash tests satisfied barrier requirements for structural adequacy and occupant risk, but did not satisfy requirements for vehicle trajectory because of large exit angles.

For possible use in work zones, a temporary asphalt median (island) was developed and evaluated for cost and effectiveness. Cottrell reports that the temporary median performed well at a traffic level between 6,000 and 30,000 vehicles per day, at less cost than movable concrete barriers, because no impact attenuators were needed. Suggestions for improvement are included in the paper.

Signs on Breakaway Barricades—Wind and Crash Tests

THOMAS D. DAVIS

Work zone informational sign panels are often mounted on portable wood or metal frames and used for changing traffic operations in New Jersey work zones. At times, these portable signs are placed close to Type III breakaway barricades used to channelize traffic. If signs could be placed on the barricades instead, some portable wood or metal sign frames would no longer be needed. This would reduce sign costs as well. Barricades with signs attached at various heights for visibility purposes were tested for durability under wind loads up to 60 mph in accordance with criteria established by AASHTO. Twenty- and 60-mph full-scale vehicle crash tests were conducted in compliance with criteria established by AASHTO and *NCHRP Report 230*. Only lightweight signs were attached to the barricades in the tests to eliminate doubt concerning the damage that standard-weight signs might cause. The 12-in.-clearance, 0.024-in.-thick aluminum sign failed the 60-mph crash test. However, vinyl roll-up signs with 21, 38, and 50 in. of clearance from the bottom of the sign to the pavement and 0.024-in.-thick aluminum signs with 29 and 41 in. of clearance passed the wind and crash tests and are recommended for implementation.

Work zone informational sign panels are often mounted on portable wood or metal frames and used for changing traffic operations in New Jersey work zones. At times, these portable diamond-shaped signs are placed close to barricades used to channelize traffic. If signs could be placed on the breakaway barricades instead, the practice of using portable wood or metal sign frames could be abandoned. This would reduce the number of hazardous objects near traffic. This subject is timely because most work zone accidents are collisions with hazardous objects (*1*). The sign expenses would be reduced as well.

The barricades used to channelize traffic are called Type III breakaway barricades. They have three lightweight horizontal panels with orange and white stripes. The barricades are made of unglued 3-in.-diameter polyvinyl chloride (PVC) pipe and have been shown to cause minimal damage to vehicles on impact (*2,3*). In order to eliminate doubt concerning causes of damage from the tests, only lightweight signs were attached to the barricades.

The *Manual on Uniform Traffic Control Devices* (*4*) allows work zone signs on barricades as long as the bottom of the sign is at least 1 ft above the pavement, although higher mounting is desirable. Efforts were made to attach the 4-ft-square sign panels as high as possible to provide good visibility to drivers, and the barricades were modified to support the signs at higher positions.

An initial search of the Transportation Research Information Service on-line computer files found no abstracts relevant to signs on Type III breakaway barricades. A second search on the broader subject of temporary or portable barricades or barriers with breakaway or frangible features found 130 abstracts. No information was found relative to testing of portable small sign supports, including signs on Type III breakaway barricades. A search of related information sources yielded only one untested prototype Type III breakaway barricade sign support, which was developed by the Ponca City, Oklahoma, Traffic Engineering Department.

The barricades with attached signs were to continue to function in a manner consistent with standard PVC Type III breakaway barricades when subjected to wind stress as specified by AASHTO (*5*) and vehicle impacts in compliance with criteria established by AASHTO (*5*) and *NCHRP Report 230* (*6*).

To select an adequate barricade-supported sign device, the following procedures were used:

1. Preliminary wind load tests using the back of a moving truck,
2. Intermediate wind load tests using a jet exhaust wind tunnel,
3. Final wind load tests using the back of a moving truck,
4. Preliminary in-house crash tests at low speeds, and
5. High-speed crash tests by a contractor.

The sign height was varied during the tests, and modifications were made as necessary. The design that passed the tests will be installed and monitored in an actual work zone.

DESCRIPTION OF BARRICADES AND SIGNS

Type III breakaway barricades were used to support the necessary sign panels. The barricades have three 9-in. × 48-in., 0.024-in.-thick aluminum horizontal panels with orange and white stripes. The panels are attached to 3-in. unglued PVC pipe, Schedule 40 ASTM 1785-74 or SDR-26 ASTM 2241-74. The initial design used two spring-tensioned wires from the top to the far bottom of the barricade to keep the barricade from vibrating apart. To restrain the top section of the barricade from striking car windshields on impact, a No. 6, 3/16-in.-diameter solid braided nylon rope was tied inside the vertical portion of the barricade. For ballast, 300 lb of sand was used.

The diamond-shaped signs were 4 ft long on each side and made of either vinyl or 0.024-in.-thick aluminum. The vinyl sign was supplied with a fiberglass cross that provided rigidity; the cross was attached to the barricade with two 1-in. No. 14 panhead screws through the horizontal fiberglass cross mem-

Bureau of Transportation Systems Research, New Jersey Department of Transportation, 1035 Parkway Avenue, Trenton, N.J. 08625.

ber. The aluminum sign was attached to the barricade with four 1-in. No. 14 panhead screws.

WIND TESTS—PHASE 1

Method

On October 2, 1986, a flatbed stake truck was used to test the effects of wind on the experimental sign on barricade devices (Figure 1). The signs were placed on the tailgate to minimize the effects of the truck cab. The signs with 1 ft of clearance from the bottom of the sign to the pavement were attached to standard Type III breakaway barricades; the 5-ft-clearance signs were attached to barricades extended upward; the 7-ft-clearance sign was attached to a barricade with an extended height and an extended base (Figure 2). To pass this preliminary screening, the signs and barricades were to keep their integrity as the truck accelerated. The goal was 60 mph, but the truck reached a maximum speed of only 50 mph because of the shortness of the test track. Each model was tested for wind stress into and behind the sign by reversing the orientation of the sign.

Results

Table 1 outlines the results of the Phase 1 truck tests. There were no problems with the 1-ft-clearance signs. However,

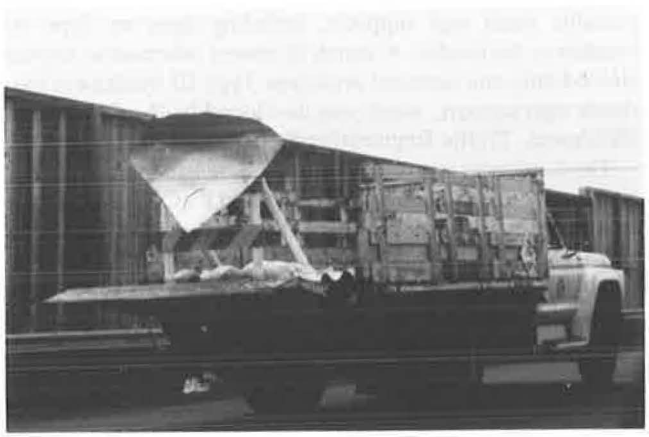


FIGURE 1 Phase 1 wind test with 1-ft-clearance aluminum sign at 50 mph.

FIGURE 2 Phase 1 wind test with 7-ft-clearance vinyl sign at 45 mph.

TABLE 1 RESULTS OF PHASE 1 WIND TESTS

Test Number	Sign Materials	Sign Clearance (ft.)	Ballast (lbs.)	Wind Direction	Results*
1	aluminum	1	300	behind sign	pass
2	aluminum	1	300	into sign	pass
3	vinyl	1	300	behind sign	pass
4	vinyl	1	300	into sign	pass
5	aluminum	5	600	behind sign	fail, 45 mph
6	aluminum	5	600	into sign	fail, 40 mph
7	vinyl	5	600	behind sign	fail, 40 mph
8	vinyl	5	600	into sign	pass
9	vinyl	7	600	behind sign	fail, 45 mph
10	vinyl	7	600	into sign	fail, 45 mph

* To pass, the sign and barricade were to keep their integrity when subject to a wind load of 50 mph.

TABLE 2 RESULTS OF PHASE 2 WIND TESTS

Test Number	Sign Materials	Sign Clearance (ft.)	Ballast (lbs.)	Wind Direction	Results*
1	vinyl	1	300	behind sign	pass
2	vinyl	1	300	into sign	pass
3	aluminum	5	300	behind sign	pass
4	aluminum	5	300	into sign	fail, 40 mph
5	vinyl	5	300	behind sign	pass
6	vinyl	5	300	into sign	pass
7	aluminum	7	300	into sign	fail, 46 mph

* To pass, the sign and barricade were to keep their integrity when subjected to a wind load of 60 mph.

both aluminum and vinyl 5-ft-clearance and the vinyl 7-ft-clearance signs failed when the top extension broke free from the barricade. The 0.024-in.-thick aluminum sign flexed in the wind, but it straightened out after the test.

At this point the 5-ft- and 7-ft-clearance sign frames were modified. The 5-ft-clearance sign frame was extended both vertically and horizontally, and the 7-ft-clearance sign frame was designed so that guy wires with springs would accept most of the wind load. These new designs were evaluated in the next wind test.

WIND TESTS—PHASE 2

Method

On October 27, 1986, the 1-ft-clearance models and the modified 5-ft- and 7-ft-clearance models were tested at the Federal Aviation Administration Technical Center jet exhaust wind tunnel facility in Atlantic City, New Jersey. According to AASHTO (5), "Roadside sign structures that are considered to have a relatively short life expectancy may be designed using wind speeds based on a 10-year mean recurrence interval." For New Jersey, the wind speed for a 10-year mean recurrence interval is 60 mph. Thus the sign and support structures would have to keep their integrity under a 60-mph wind load to pass the wind tunnel test. Whereas the 1-, 5-, and 7-ft models were tested in both directions, only the 5-ft model was tested with both the aluminum and the vinyl signs. Phase 1 showed little difference between the aluminum and vinyl signs with the same clearance.

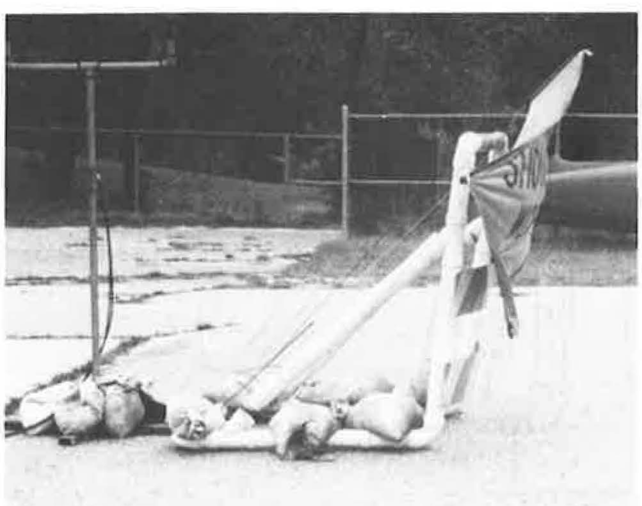


FIGURE 3 Phase 2 wind test with 1-ft-clearance vinyl sign at 69 mph.

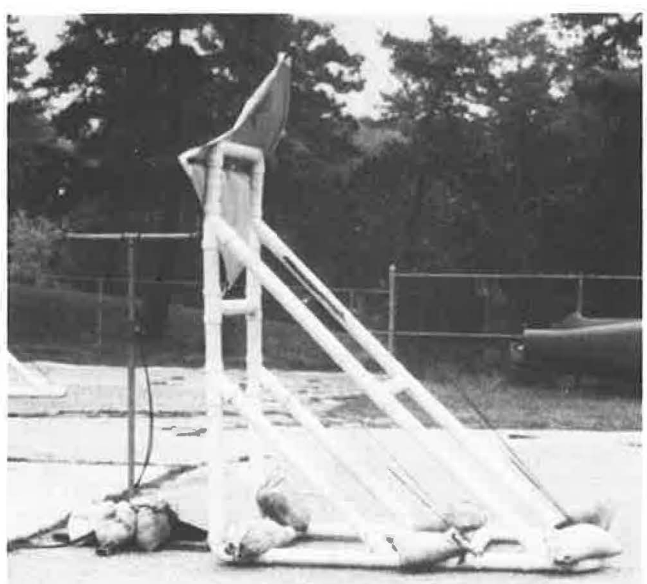


FIGURE 4 Phase 2 wind test with 5-ft-clearance vinyl sign at 63 mph.

Results

As Table 2 shows, the 1-ft-clearance vinyl sign passed this wind test as well (Figure 3). Whereas the 5-ft-clearance vinyl sign endured the test (Figure 4), the aluminum sign failed when the frame separated with the wind into the sign. The 7-ft-clearance aluminum sign frame failed when the top Y fitting fractured (Figure 5).

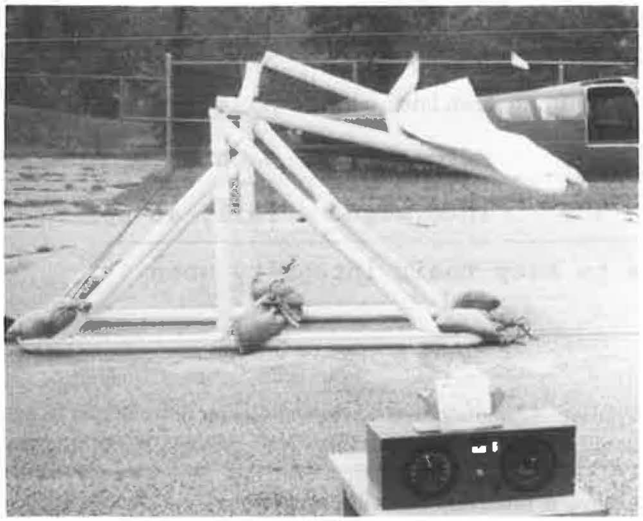


FIGURE 5 Phase 2 wind test with 7-ft-clearance aluminum sign at 46 mph.

Based on the results of the wind tunnel tests, additional changes were made to make the sign frames more durable. The springs that provided tension for the wires used on the standard barricade were removed because they allowed too much flexing under wind loads. The springs were also judged to be hazardous because they tended to come loose and become projectiles when adjusted by workers. The wires were pulled tight to provide tension. The aluminum sign clearances were reduced to 12, 29, and 41 in., and the vinyl sign clearances were reduced to 21, 38, and 50 in. The 29- and 38-in.-clearance signs were attached to barricades extended up 1 ft, and the 41- and 50-in.-clearance signs were attached to barricades extended up 2 ft. The vinyl signs were 9 in. higher than the aluminum signs because the vinyl signs had their own support systems. See Figures 6 through 11 for details.

WIND TESTS—PHASE 3

Method

The new signs on barricade structures were placed on a flatbed stake truck that reached 60 mph. As usual, the signs were tested with the wind behind the sign and into the sign.

Results

All of the redesigned signs on barricade structures passed this final wind test (Table 3). The signs were now ready for the preliminary crash tests.

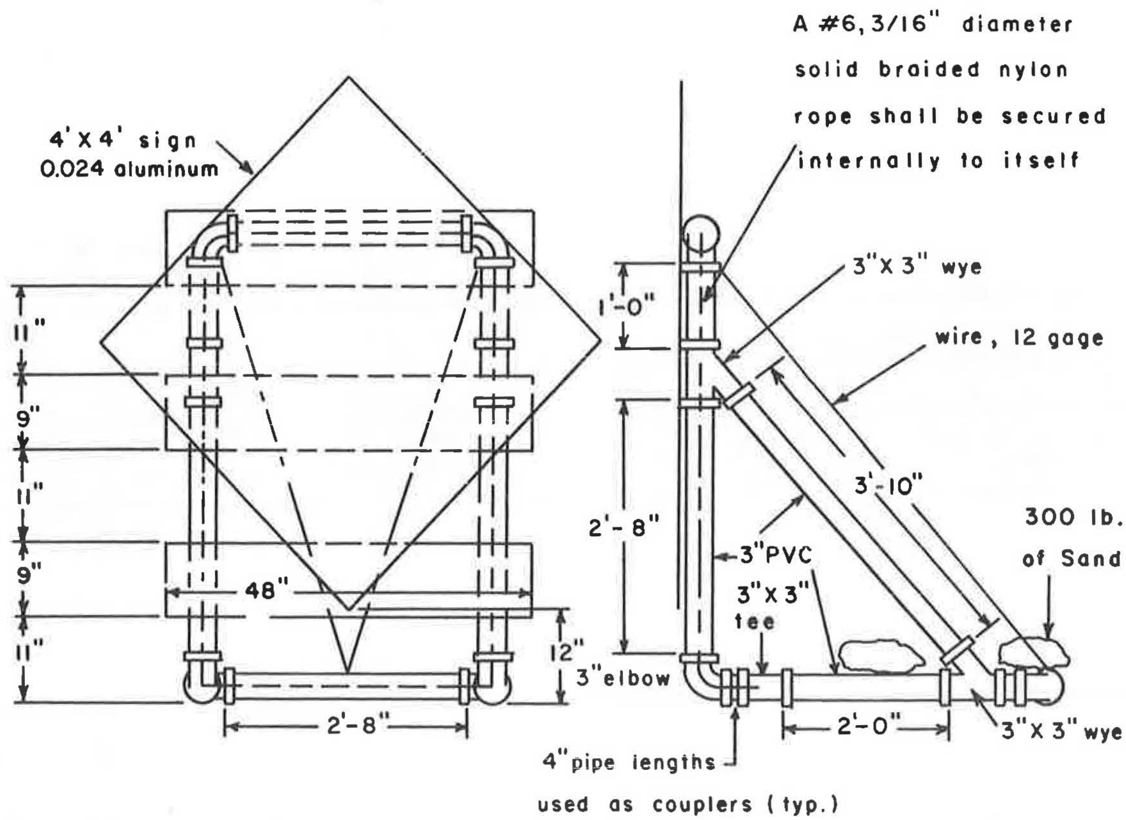


FIGURE 6 Aluminum sign on barricade with 12 in. of clearance.

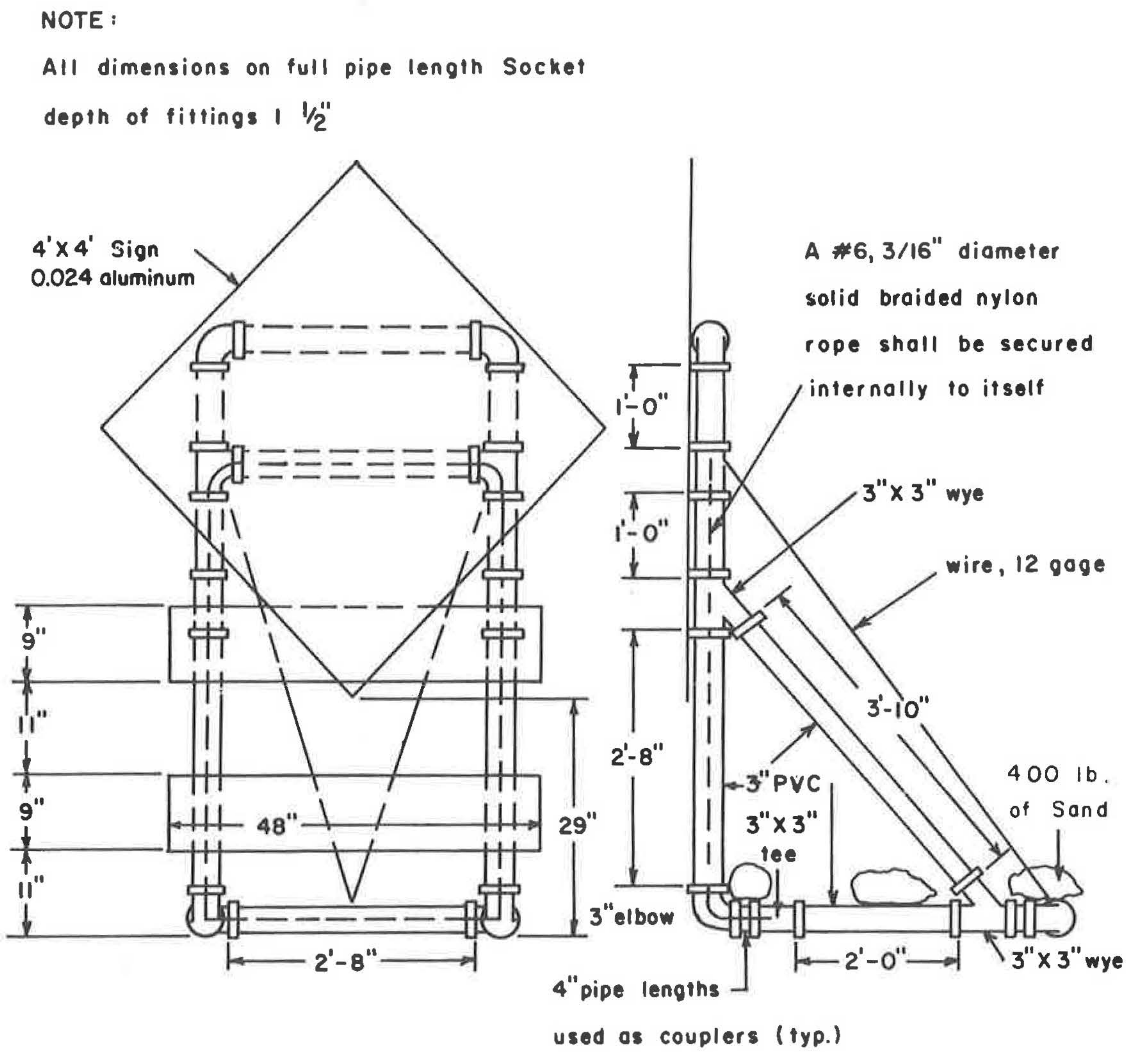


FIGURE 7 Aluminum sign on barricade with 29 in. of clearance.

PRELIMINARY CRASH TESTS

Method

On July 15, 1987, the barricades with attached signs were crash-tested to determine their effects on vehicles and nearby workers. An unoccupied Plymouth Horizon was used to crash into the signs at 25 mph, the highest speed possible under the test track conditions. A W-beam guardrail 150 ft in length was used to guide the vehicle to the sign, and a truck was used to push the car down the track (Figure 12). Once the car reached 25 mph, the truck driver braked and the car continued into the sign (Figure 13). After the collision, the car was stopped with a remotely controlled hydraulic brake. The tests were documented with two video cameras and one 35-mm camera. One video camera was positioned to view the entire site, and the other video camera recorded the impact. The preliminary test was done in-house and was designed to eliminate unsafe models before final testing by a contractor.

Results

As Table 4 shows, the six crashes, with the exception of one, did only cosmetic damage to the car (Figure 14). However, the 41-in.-clearance aluminum sign damaged the windshield (Figure 15). The right post was hit and the upper right section of the barricade structure shattered the windshield. In this case, the internal rope failed to hold the vertical portion of the barricade together, permitting the debris to hit the windshield. The internal rope was used in the standard barricade to keep the top portion of the barricade away from the windshield.

To minimize this problem, 12-gauge wire was specified and the sign panel was attached above to the top T fitting. In this way the sign and frame should separate above the T fitting and clear the top of the vehicle. The internal rope should keep the rest of the frame away from the windshield. The signs did not become missiles to harm other drivers or nearby workers in any of the tests. The sign either stayed with the vehicle or landed a few feet from the impact site. With these minor adjustments, the signs were ready for the final crash tests.

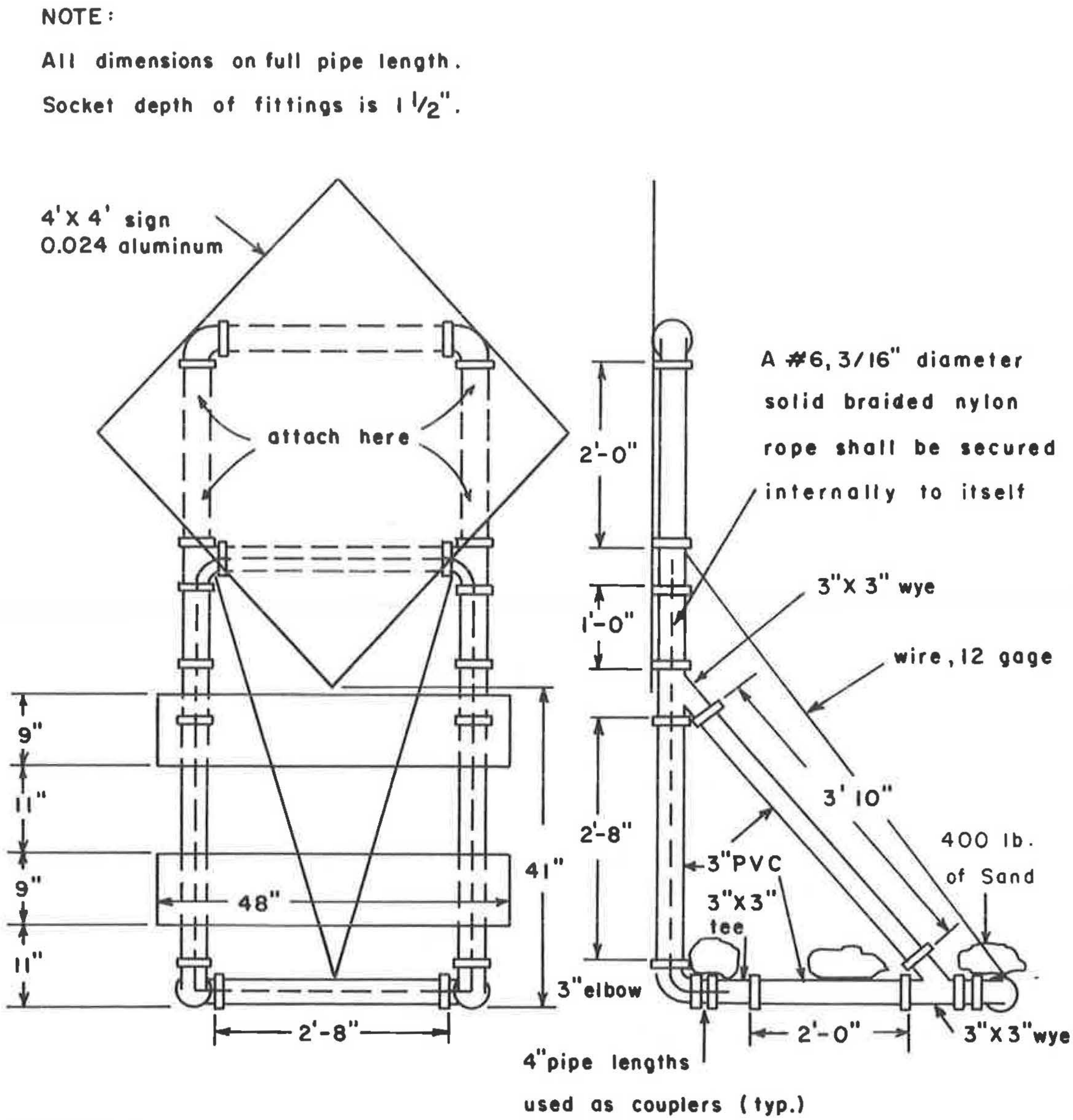


FIGURE 8 Aluminum sign on barricade with 41 in. of clearance.

FINAL CRASH TESTS

Method

In the fall of 1988, the University of Nebraska conducted 20- and 60-mph crash tests using a 1,800-lb, unoccupied 1980 Volkswagen Rabbit (J. A. Magdaleno, R. K. Faller, and E. R. Post, unpublished data, 1989). Two piezoresistive accelerometers were bolted to the car floor to measure longitudinal accelerations, two 16-mm cameras documented the collisions at 500 frames/sec, and tape pressure switches measured the speed before and after impact. The Rabbit was towed by another vehicle, guided by a suspended cable, and released before impact.

Criteria

The test performance was judged on the basis of criteria set by AASHTO (*5*) and NCHRP (*6*). The purpose of the 20-mph test was to measure the breakaway characteristics of the signs and their barricade supports. On impact the center of the bumper was planned to be midway between the two barricade posts. The purpose of the 60-mph test was to estimate vehicle stability, vehicle trajectory, occupant risk, debris intrusion into the passenger compartment, and the hazard from debris to other traffic. The right post was contacted by the quarter point of the vehicle's bumper. The 21-in.-clearance vinyl sign was not tested because it closely resembled the standard barricade that was crash-tested more than 15 years ago. On the other hand, because of the windshield cracking experienced in the preliminary tests, the 41-in.-clearance aluminum sign was also struck at 20 mph and 60 mph with the bumper centered on one post.

Results

Table 5 gives the results of the 12 full-scale crash tests into the experimental barricade-supported signs. The vehicle impact

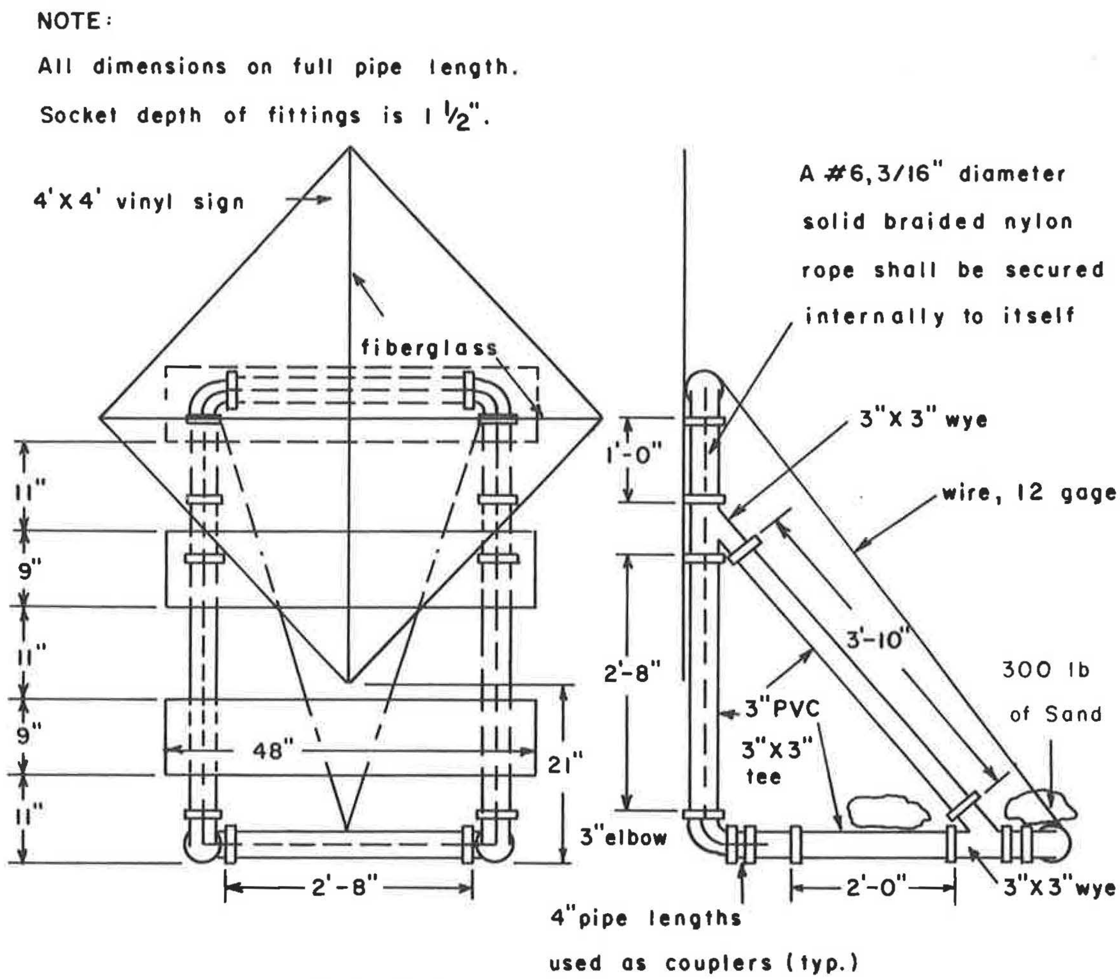


FIGURE 9 Vinyl sign on barricade with 21 in. of clearance.

velocity, the vehicle change in velocity, and the occupant impact velocity were all normalized to give values that would be more indicative of the test results had the tests been conducted at the exact target impact speed. The following conclusions can be noted:

1. Three of the devices proved satisfactory in meeting the criteria: the 41-in.-clearance aluminum sign on an extended barricade, the 38-in.-clearance vinyl sign on an extended barricade, and the 50-in.-clearance vinyl sign on an extended barricade.
2. The 29-in.-clearance aluminum sign on an extended barricade proved to be marginal because high occupant impact velocities (17.4 ft/sec) and vehicle velocity changes (16.9 ft/sec) were recorded.
3. The 12-in.-clearance aluminum sign on a standard barricade failed the criteria because the sign and support structure intruded into the passenger compartment.

SUMMARY AND CONCLUSIONS

On the basis of the results of the 60-mph wind tests and the 20-mph and 60-mph crash tests, the following meet AASHTO and NCHRP criteria for 4-ft × 4-ft vinyl and 0.024-in.-thick aluminum signs attached to Type III breakaway barricades:

1. 21-in.-clearance vinyl roll-up sign,
2. 29-in.-clearance, 0.024-in.-thick aluminum sign,
3. 38-in.-clearance vinyl roll-up sign,
4. 41-in.-clearance, 0.024-in.-thick aluminum sign, and
5. 50-in.-clearance vinyl roll-up sign.

Although the 29-in.-clearance, 0.024-in.-thick aluminum sign was marginal in passing the 60-mph crash test, the University of Nebraska said "this design has the ability to perform satisfactorily provided that it is ballasted properly" (J. A. Magdaleno, R. K. Faller, and E. R. Post, unpublished data, 1989). It is important that the sandbags be distributed evenly along the base of the barricade because they tend to pile up under the car.

The signs were reusable and the barricade frames could be repaired with available interchangeable parts. Type III breakaway barricades with attached 4-ft × 4-ft signs continued to function in a manner consistent with standard Type III breakaway barricades. This means that wood or metal frames will no longer be needed to support signs in the vicinity of existing Type III breakaway barricades. This in turn means fewer fixed objects and lower costs.

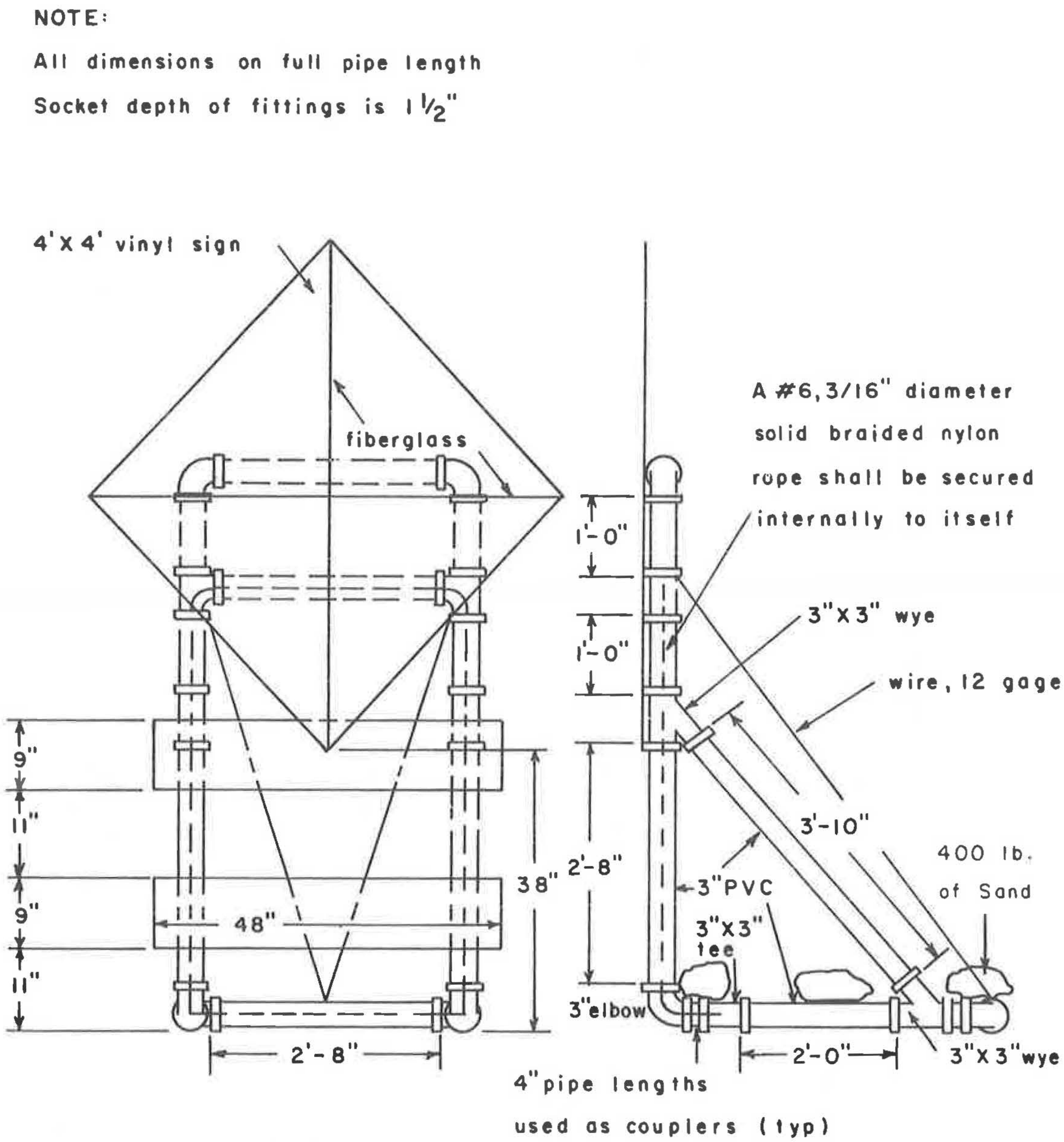


FIGURE 10 Vinyl sign on barricade with 38 in. of clearance.

RECOMMENDATIONS

Vinyl roll-up signs with 21, 38, and 50 in. of clearance and 0.024-in.-thick aluminum signs with 29 and 41 in. of clearance from the bottom of the sign to the pavement are recommended for implementation in work zones.

IMPLEMENTATION OF FINDINGS

Starting in November 1989, the 50-in.-clearance vinyl roll-up signs were used in actual work zones. The signs were monitored and functioned properly.

ACKNOWLEDGMENTS

This research was done in cooperation with the FHWA, U.S. Department of Transportation. Appreciation is expressed to the following persons: Charles A. Goessel of the New Jersey Department of Transportation's Design Division for originating the idea of using signs on breakaway barricades; Eugene F. Reilly, Richard L. Hollinger, and Arthur W. Roberts for their administrative assistance; Lad Szalaj, Bill Crowell, Bob Tomlinson, John Senyk, Zolton Zeisky, and Tom Black for their technical assistance; the Technical Committee members for their valuable input; and Judith Scymanski and Yolanda Prilo for typing the final report. In addition, appreciation is

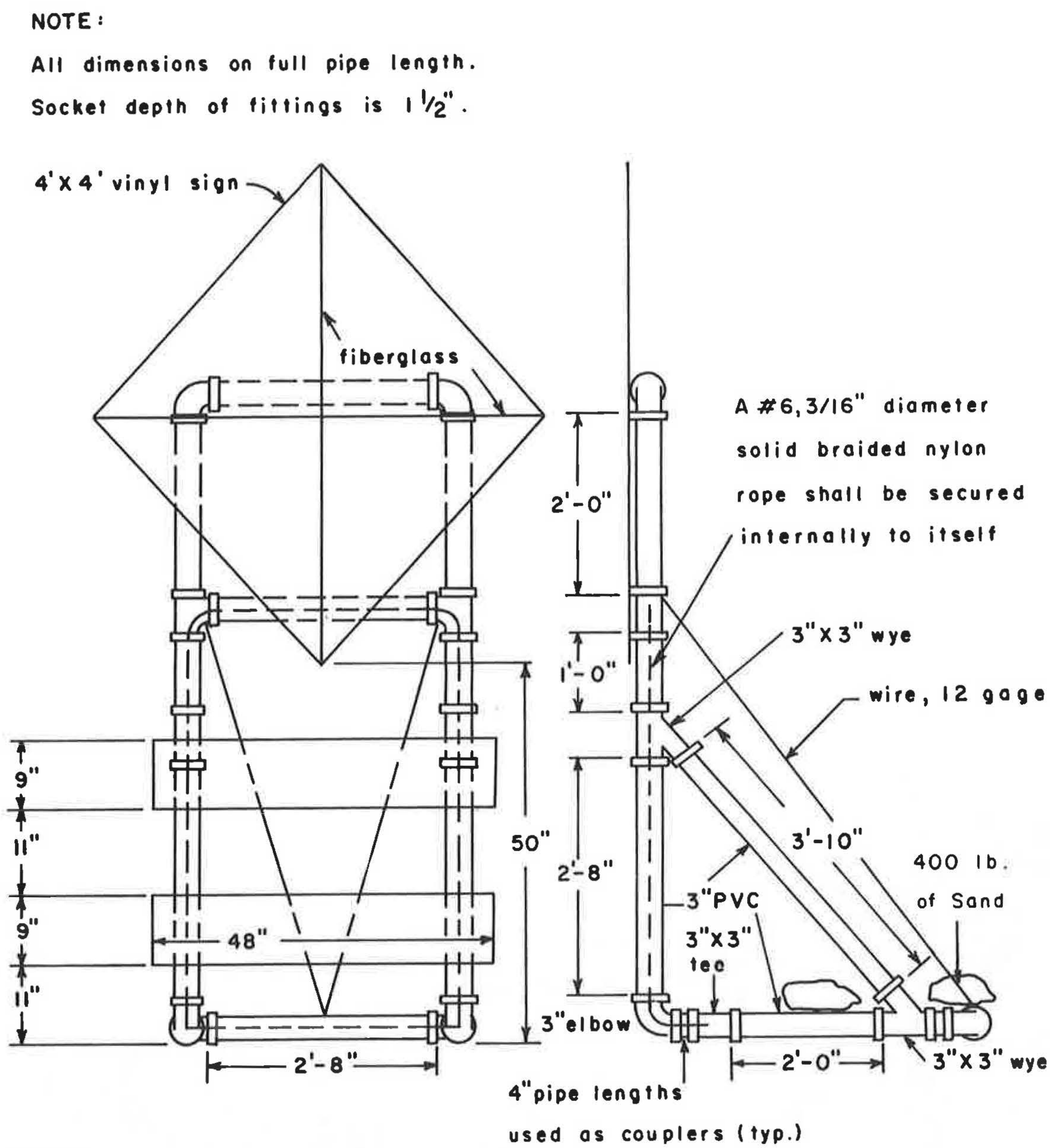


FIGURE 11 Vinyl sign on barricade with 50 in. of clearance.

TABLE 3 RESULTS OF PHASE 3 WIND TESTS

Test Number	Sign Material	Sign Clearance (in.)	Ballast (lbs.)	Wind Direction	Results*
1	aluminum	12	300	behind sign	pass
2	aluminum	12	300	into sign	pass
3	aluminum	29	400	behind sign	pass
4	aluminum	29	400	into sign	pass
5	aluminum	41	400	behind sign	pass
6	aluminum	41	400	into sign	pass
7	vinyl	21	300	behind sign	pass
8	vinyl	21	300	into sign	pass
9	vinyl	38	400	behind sign	pass
10	vinyl	38	400	into sign	pass
11	vinyl	50	400	behind sign	pass
12	vinyl	50	400	into sign	pass

* To pass, the sign and barricade were to keep their integrity when subjected to a wind load of 60 mph.

FIGURE 12 Preliminary crash test vehicle guidance system.

FIGURE 13 Preliminary crash test.

FIGURE 14 Typical preliminary crash test vehicle damage.

TABLE 4 RESULTS OF PRELIMINARY CRASH TESTS

Test Number	Sign Material	Sign Clearance(in.)	Ballast (lbs.)	Number of Posts Hit	Results
1	aluminum	12	300	1	small dent in hood
2	aluminum	29	400	2	small dent in hood
3	aluminum	41	400	1	windshield cracked
4	vinyl	21	300	2	small dent in hood
5	vinyl	38	400	2	small dent in hood
6	vinyl	50	400	2	small dent in hood

FIGURE 15 Preliminary crash test windshield damage from 41-in.-clearance aluminum sign.

TABLE 5 RESULTS OF FINAL CRASH TESTS

Test Number	Sign Material	Sign Clearance(in.)	Speed (mph)	Ballast (lb.)	Number of Posts Hit	Results*
1	aluminum	12	20	300	2	pass
2	aluminum	12	60	300	2	fail
3	aluminum	29	20	400	2	pass
4	aluminum	29	60	400	2	marginal
5	aluminum	41	20	400	2	pass
6	aluminum	41	60	400	2	pass
7	aluminum	41	20	400	1	pass
8	aluminum	41	60	400	1	pass
9	vinyl	38	20	400	2	pass
10	vinyl	38	60	400	2	pass
11	vinyl	50	20	400	2	pass
12	vinyl	50	60	400	2	pass

* To pass, the barricade supported sign structures must have met NCHRP and AASHTO criteria for:

a. debris intrusion into the passenger compartment,
b. hazard to other traffic from debris,
c. passenger compartment integrity,
d. vehicle velocity change (not to exceed 15 fps),
e. vehicle stability,
f. vehicle trajectory,
g. occupant impact velocity (not to exceed 15 fps) and
h. occupant ride down acceleration (not to exceed 15 g's).

expressed to the FAA Technical Center for the use of the jet exhaust wind tunnel; the University of Nebraska for performing the final crash tests; the Bureau of Transportation Technology Research for videotaping the wind and preliminary crash tests; the Division of Maintenance of the New Jersey Department of Transportation for providing the trucks used in the wind tests; and finally MDI Traffic Control Products, a division of Marketing Displays, Inc. for providing vinyl roll-up signs and Windmaster sign holders for testing.

REFERENCES

1. D. C. Harris. Steps Toward Stopping the Slaughter in the Work Zone. *Traffic Safety*, July–Aug. 1987.
2. A. S. Miller. Breakaway Barricades. *Public Roads*, Vol. 40, No. 1, 1976.
3. *Breakaway Barricades*. Demonstration Project No. 41, FHWA, U.S. Department of Transportation, 1978.
4. *Manual on Uniform Traffic Control Devices*. FHWA, U.S. Department of Transportation, 1978.
5. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*. American Association of State Highway and Transportation Officials, 1985.
6. J. D. Michie. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., 1981.

Evaluation of Small-Sign Systems from Existing Crash Test Data

L. Dwayne Breaux and James R. Morgan

Small signs and small-sign support systems account for a substantial investment by federal, state and local agencies. For the past 20 or more years these systems have been tested for crashworthiness. Many small-sign systems had been tested and approved on the basis of previous specifications. For the most part, these tests were conducted with different vehicles and sometimes at different impact speeds than those required by current specifications. Retesting of current systems will undoubtedly be required as new specifications are released. A rationale that can be used to predict impact performance for sign installations that have been tested previously with a different size and class of vehicles is presented. In spite of the variability in test parameters, it appears that an energy formulation will provide estimates, not only for the current standard, but also for any future vehicle weights or impact speeds. Most sign systems, breakaway or not, appear to follow a linear relationship between kinetic energy and impact velocity. Recent tests, for the most part, support this theory. The one notable exception to the linear fit is the triangular slip base. This system, because of its unique failure mechanism, is more appropriately modeled by a cubic equation of best fit. The estimated changes in velocity could be useful for recertification of existing sign systems as well as for extrapolation between single- and multiple-post systems. If additional tests are required, the estimated changes in velocity will indicate which tests are critical, thereby allowing for the possibility of fewer certification tests.

Small-sign systems include everything from Stop signs and delineator posts to signs up to about 25 ft^2. In some cases multiple small-sign support systems are used to support much larger signs (40 to 50 ft^2). Therefore, this broad class of sign is prevalent in every state, county, and municipality in the country.

To ensure the safety of vehicle occupants, specifications, guidelines, and recommendations have been written that define acceptable vehicle performance criteria (*1,2*). In 1981 *NCHRP Report 230* (*3*) became the standard for measuring crashworthiness. The NCHRP report with modifications from AASHTO (*4*) is the current standard; however, a new standard is unquestionably in the future.

Many small-sign systems had been tested and approved under the TRB specification using a 2,250-lb vehicle. However, AASHTO requires the use of an 1,800-lb vehicle tested at 20 and 60 mph. Thus, the more recent specification required additional testing for systems that had previously been tested and approved. Several recertification tests were done showing that most previously approved systems passed with the 1,800-lb cars. With these considerations, the Federal Highway Administration ruled (23 CFR 625) that it is not currently necessary to retest systems approved with 2,250-lb vehicles. Unfortunately, the need for retesting will resume at the end of the current grace period.

Texas Transportation Institute, Texas A&M University System, College Station, Tex. 77843-3135.

The primary focus of the specifications has been the changes in velocity during impact and the integrity of the occupant compartment. The current standard addresses these areas as follows. First, the change in velocity of an unrestrained occupant should not exceed 15 ft/sec (extended to 16 ft/sec by 23 CFR 625) during the impact. Second, there can be no penetration of the occupant compartment. The report includes other test specifications, but for a given sign system, it is generally these two criteria that determine the acceptability of a sign installation for crash performance.

The most significant difference between the 1,800-lb and 2,250-lb cars has been the change in velocities. Vehicle stability and occupant compartment integrity are also major considerations, but these are usually linked to the change in velocity. Unfortunately, there has been no acceptable method for comparing or predicting the crash performance of the 1,800-lb car versus the 2,250-lb car. To complicate the problem, many of the previous tests included cars of weights other than 2,250 lb, various impact speeds, different crush characteristics, and test matrices with multiple posts as well as single-post sign systems.

A rationale that can be used to predict impact performance for sign installations that have been tested previously with a different size and class of vehicles is presented. In spite of the variability in test parameters, it appears that an energy formulation will provide estimates, not only for the current standard, but also for any future vehicle weights or impact speeds. The estimated changes in velocity will be useful for recertification of existing sign systems as well as for extrapolation between single- and multiple-post systems. If additional tests are required, the estimated changes in velocity will indicate which tests are critical, thereby allowing for the possibility of fewer certification tests.

DATA COLLECTION

This study began with a compilation of recent crash test data to try to validate some of the small-sign supports currently used by the Texas State Department of Highways and Public Transportation. It soon became obvious that the data that could be classified as recent were limited in quantity. Therefore, the data search was expanded to include all previous crash tests for which the sign installation was well defined and the vehicle weight, impact speed, and change in velocity were accurately known. The data collected are given in Table 1 (*5–12*) by sign classification.

TABLE 1 CRASH DATA

Test Number	Vehicle Weight (lb)	Impact Velocity (mph)	Change Velocity (ft/s)	Change[1] Momentum (lb-sec)	Change Kin. Energy (ft-lb)
3 lb/ft U - Post Ground Splice (Rail Steel Post)					
3491-1	2250	22.7	2.7	190	6070
3491-2	2250	59.6	2.5	179	15420
3491-3	2250	17.2	5.2	368	8323
3491-4	2250	16.6	5.1	358	7807
3 lb/ft U - Post High Splice (100 ksi) -Single Post					
7024-7	1800	60.5	3.1	169	14740
7024-8	1800	19.9	6.0	339	8866
Three Posts					
7024-9	1800	59.3	10.6	197	16067
7024-10	1800	19.4	23.9	445	7348
3 lb/ft U - Post High Splice (80 ksi) Three Posts					
7024-16	1800	20.0	27.9	511	7983
7024-17	1800	62.0	18.9	353	28732
7024-18	1800	19.5	24.9	465	7496
7024-21	1800	61.5	22.9	412	32585
Two Posts					
7024-22	1800	20.0	9.4	267	6557
7024-23	1800	62.8	11.7	334	28768
3 lb/ft U - Post Ground Splice (Three High Carbon Billet Post)					
7024-26	1800	21.7	12.6	235	5998
7024-27	1800	61.6	9.1	169	14530
4 lb/ft U - Post High Splice (High Carbon Billet Post) -Two Posts					
7024-11	1800	20.2	12.5	327	7766
7024-12	1800	60.9	10.3	295	24753
-Single Post					
4 lb/ft U - Post Ground Splice (Three Rail Steel Post)					
7024-24	1800	20.6	28.0	522	8460
7024-25	1800	62.6	13.2	246	20962
8 lb/ft U - Post					
1817-4	3500	37	16.6	1810	30664
1817-25	3600	31.5	16.9	1890	71343
1817-29	3550	24	15.4	1700	46733
1817-31	3900	36	14.9	1810	82043
2466-1	4100	29.3	9.3	1180	45241
2466-2	4100	43.7	12.6	1610	93011
2466-3	4400	43.9	12.4	1700	98883
2466-4	4400	30.4	15.0	2050	76025
2466-5	3880	45.8	14.8	1780	106421
2466-6	3750	49.5	13.5	1570	103399
2466-7	3850	31.7	16.7	2000	76259
2466-8	3850	45.6	18.1	2170	125438
2466-9	3800	47.8	15.0	1770	110815
2 - 1/2 inch Pipe w/Frangible Connector					
3254-14	2250	20.3	11.4	802	19316
3254-15	2250	63.3	5.4	379	34167
3254-16	2250	19.2	9.1	638	15079
0941-3	2270	29.2	7.3	514	20139
2 x 2 inch Square Perforated Steel Tube -Single Post					
7024-3	1800	20.0	3.5	193	5328
7024-4	1800	56.8	8.5	468	37028
-Two Posts					
7024-5	1800	19.7	20.7	575	10699
7024-19	1800	18.9	14.7	413	8391
7024-20	1800	57.5	17.9	503	37860
-Three Posts					
7024-6	1800	59.3	26.3	486	35951
3 inch Pipe on Triangular Slip Base					
0941-1	2270	60.8	5.5	386	33364
0941-4	2270	45.4	3.0	209	13637
S-8	3970	46.0	1.1	136	9100
S-18	4170	31.3	1.3	168	7603

[1]NOTE: All values for Change in Momentum or Kinetic Energy are given for a single post and obtained by linear interpolation.

Each crash test supplied the following three data points:

1. M_1, the vehicle mass;
2. V_1, the impact velocity; and
3. ΔV_1, the change in velocity.

The direct comparison of the changes in velocity for a particular sign installation type showed no apparent trend. The only general tendency was a decrease in the change in velocity for a corresponding increase in impact velocity. These data confirmed the observation that the actual failure mechanism varied for different impact speeds. At this point two different methods, the conservation of energy and the principle of impulse and momentum, were incorporated to further reduce the data to find a relationship that overrides the physical differences.

DATA REDUCTION

In review, the mass (weight) of the vehicle, impact speed, and change in velocity (ΔV) during impact were all known for specific tests. However, the challenge was to predict the change in velocity for a vehicle of any mass, M_1^*, striking at any velocity, V_1^*, in a future impact.

The first approach was to use the principle of impulse and momentum, which can be expressed as

$$(M_1)V_1 + (M_2)V_2 - \int F\,dt = (M_1)V_1' + (M_2)V_2'$$

where

M_1 = mass of automobile,
M_2 = mass of sign system,
$V_\#$ = initial velocity (V_i),
$V_\#'$ = final velocity (V_f), and
$\int F\,dt$ = impulse or impact force.

Assuming that M_2 is negligible compared with M_1 gives the following:

$$(M_1)(V_i) - \int F\,dt = (M_1)(V_f)$$

or

$$\int F\,dt = (M_{CAR}) * (\Delta V) = \text{change in momentum} \quad (1)$$

This is the formulation used to calculate change in momentum from the ΔV supplied from the crash tests.

Then, for a known change in momentum with a new car mass or a new impact velocity, or both, the equation can be written:

$$(M_1^*)(V_i^*) - \int F\,dt = (M_1^*)(V_f)$$

or

$$V_f = (1/M_1^*)[(M_1^*)V_i^* - \int F\,dt] \quad (2)$$

This is the formulation that is used to predict final velocity and change in velocity for a sign system with a known change in momentum.

The next approach was to enforce conservation of energy. The total energy is expressed as the sum of the kinetic energy

(T) and the potential energy (V). Energy is conserved when the change in the total energy of a system, represented by the prefix Δ, is equal to zero. This can be stated as $\Delta T + \Delta V_g + \Delta V_e = 0$. Note that the change in potential energy is subdivided into gravitational and elastic potential, designated by the subscripts g and e, respectively.

Again, assuming that the mass of the sign system is negligible compared with the automobile's mass greatly simplifies the energy expression. The only term contributing an appreciable amount is the change in kinetic energy of the car. This term is written

$$\Delta T = 1/2\ (M_{\mathrm{CAR}})\ [V_f^2 - V_i^2] = \Delta KE \tag{3}$$

This equation is used to calculate the change in kinetic energy (ΔKE) from the crash test data.

Then, for a known change in kinetic energy with a new car mass or a new impact velocity, the equation can be written

$$V_f = [V_i^2 + 2(\Delta KE)/M_{\mathrm{CAR}}]^{1/2} \tag{4}$$

Therefore, if the change in kinetic energy is known for a particular sign system, the car's final velocity and its change in velocity can be predicted.

As noted in the footnote to Table 1, many of the tests involved multiple-post installations. Once the change in momentum or kinetic energy was calculated, the values were divided by the corresponding number of posts to obtain an extrapolated value for a single-post installation.

MOMENTUM VERSUS KINETIC ENERGY

Basic engineering mechanics provides two equations that can be used to predict the vehicle's final velocity. The questions remain as to what values for the change in either momentum or kinetic energy to use and whether either equation is appropriate.

Noting the previous trend (that the ΔV seemed to vary with impact velocity), the changes in both momentum and kinetic energy were plotted versus velocity. To find a general trend for all breakaway systems, all the data points were combined as indicated in Figures 1 and 2. The plot using momentum showed too much scatter to detect any general trend. On the other hand, the plot using kinetic energy did indicate a generally increasing trend. To qualify this trend a least-squares fit for a line was done. The corresponding equation and line are indicated in Figure 2.

The comparison between the two approaches was then narrowed to a single class of small-sign support system, the 3-lb/ft U-post that uses breakaway mechanisms. Again a least-squares fit was done (see Figure 3). The data suggest that a linear fit is reasonable, but momentum was plotted in Figure 4 as a check. After these two comparisons, it was decided that the best approach would be to use kinetic energy to predict the change in velocity. Although this model neglects many variables (vehicle crush, etc.), when limited to systems with similar strength and breakaway characteristics it shows good correlation with experimental data.

The 3-lb/ft nonbreakaway U-posts (Figure 5) and the sets of 4-lb/ft U-posts (Figures 6 and 7) exhibit similar linear behavior. It was noted that the line for the nonbreakaway systems was generally steeper than for the breakaway systems. The greater slope corresponded to the greater stiffness of the nonbreakaway systems.

During the data search, many data points were found for 8-lb/ft U-post systems. The data were plotted in Figure 8 because of the number of data points and the variety of impact speeds even though this system is no longer used. Figure 8 clearly illustrates the linear relationship between impact velocity and change in kinetic energy. Two additional graphs are included with linear relationships. The 2½-in. pipe with frangible connectors, Figure 9, and the 2-in. × 2-in. square perforated steel tube, Figure 10, exhibit good approximations to linear relationships.

One other system, a 3-in. pipe on a triangular slip base, is shown in Figure 11. This is certainly a breakaway system, but

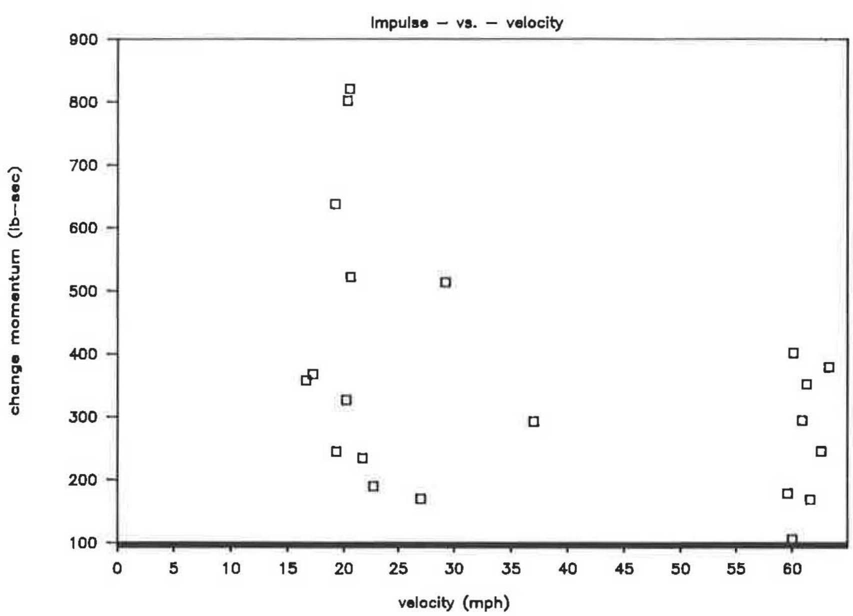


FIGURE 1 Breakaway connections—change in momentum versus velocity.

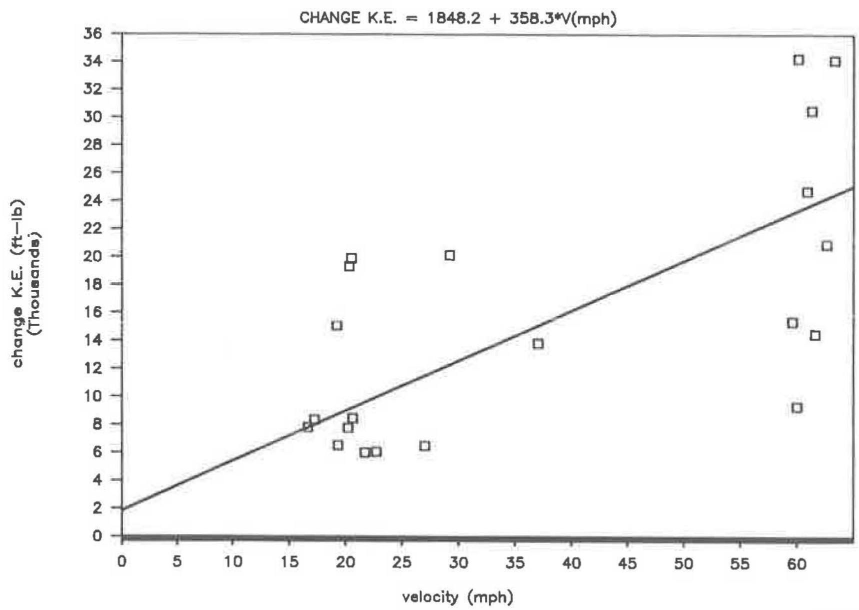


FIGURE 2 Breakaway connections—change in kinetic energy versus velocity.

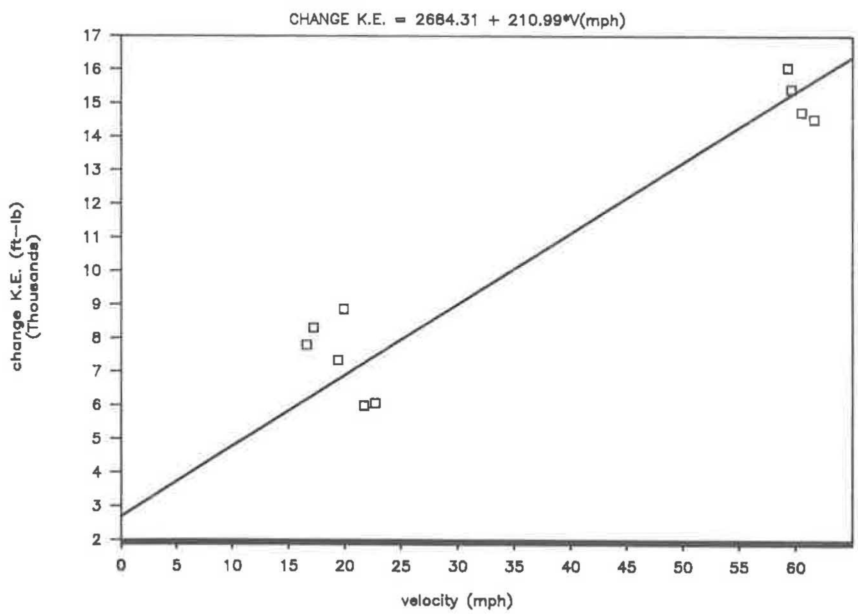


FIGURE 3 Breakaway steel U-post (3 lb/ft)—change in kinetic energy versus velocity.

it differs from all the others considered in its failure mechanism. This system uses friction to facilitate breakaway. Such a difference could mean that the relationship between velocity and change in kinetic energy is not linear but perhaps cubic, as indicated in Figure 12. Considering the limited number of data points available, it would be inappropriate to use any "recommended" best-fit curve for this system.

The diamond data points were not used in obtaining the best-fit curves shown in Figures 6, 7, 9, 11, and 12.

PREDICTING CHANGE IN VELOCITY

Although many factors (such as vehicle crush, post impact stability, size of sign, mounting height, variability in material properties, etc.) influence the behavior of breakaway sign-support systems, the significant feature is the change in kinetic energy of the vehicle. It is a great simplification to ignore all other effects, and these analyses indicate good agreement with experimental data.

The least-squares fit of the data (square data points only) shown on each of the graphs now provides a value for the change in kinetic energy for any impact velocity. One would expect the curves to tend toward zero, as is the case for all curves presented. However, these curves are valid only for systems (and impact speeds) for which a breakaway will occur. Obviously, as the impact speed decreases, at some point there will not be enough energy for a breakaway to occur. This information, taken from previous crash tests, can then be used to estimate the final velocity of a car of any mass and any

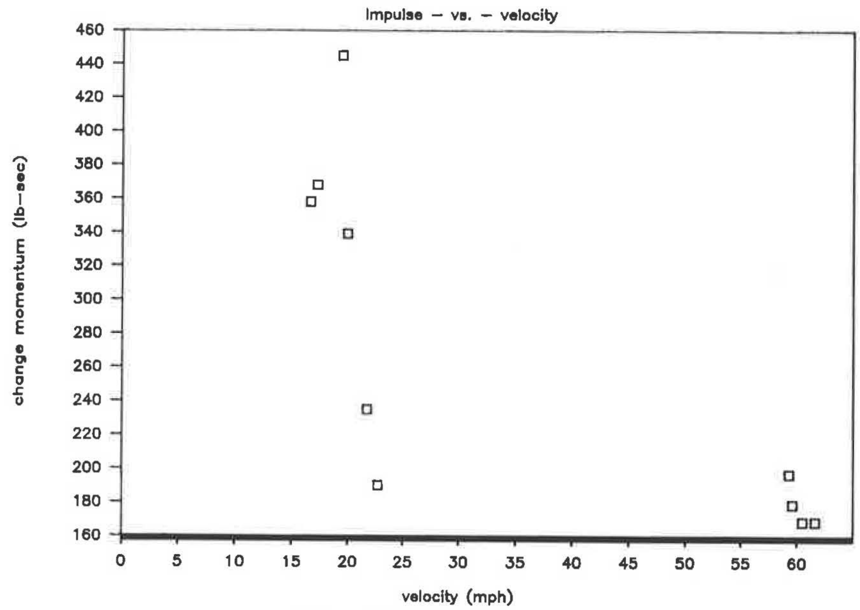


FIGURE 4 Breakaway steel U-post (3 lb/ft)—change in momentum versus velocity.

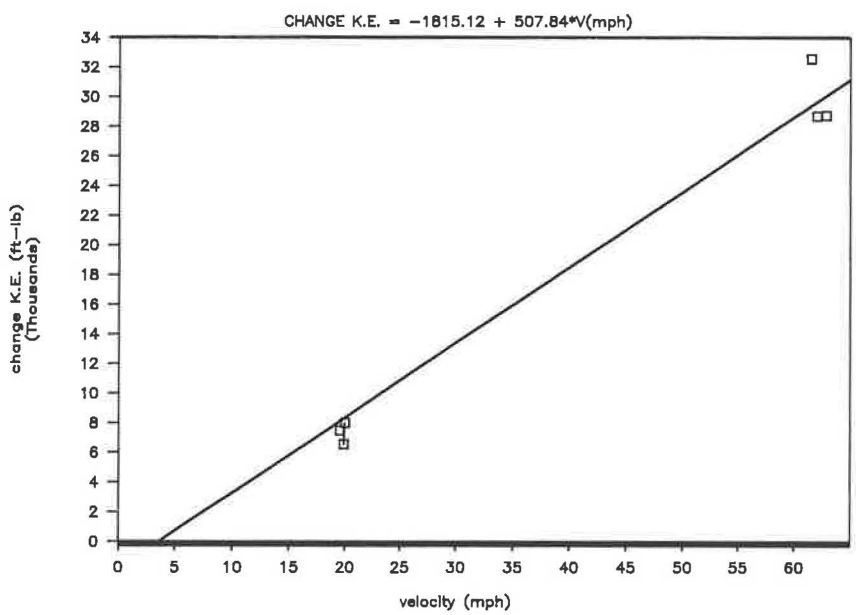


FIGURE 5 Nonbreakaway steel U-post (3 lb/ft)—change in kinetic energy versus velocity.

impact velocity from Equation 4. The difference between the final and initial velocities is the change in velocity of the vehicle during impact provided that a breakaway of the sign support does indeed occur.

This approach can be extended from a single post to multiple posts by assuming linear interpolation. That is, the ΔKE taken from the graph is simply multiplied by the number of posts. The product is then substituted into Equation 4 for ΔKE.

ACCURACY OF PREDICTIONS

This research predated the FHWA's design standards (23 CFR 625), so as part of a Texas project, additional crash tests (*12*) were required to certify several small-sign supports. This project also provided an excellent opportunity to check the validity of the assumptions on change in kinetic energy (none of these new tests were included in the curve fits). First a 40-ft^2 sign supported by three 4-lb/ft rail steel U-posts (Tests 3 and 4) was tested at 20 and 60 mph. Table 2 compares the actual changes in velocity with the values predicted using the principles presented herein. The values for ΔKE were calculated directly from the least-squares equation in Figure 7 for 4-lb nonbreakaway posts. This system was classified as nonbreakaway because large soil deformations prevented actuation of the bolted splice. Also, values for the single post in the "actual" column were extrapolated using linear interpolation.

The model was not able to predict a specific value for the change in velocity for three posts at an impact speed of 20

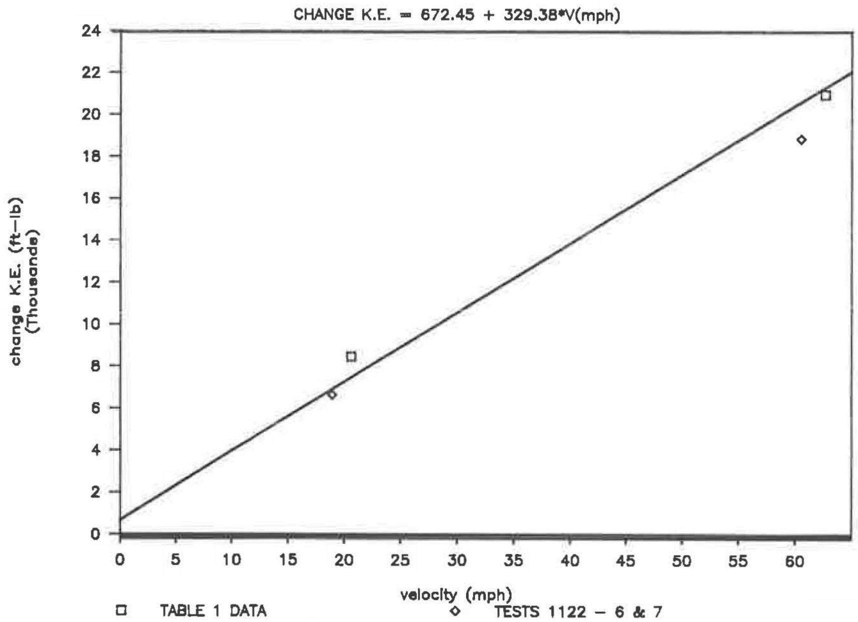


FIGURE 6 Breakaway steel U-post (4 lb/ft)—change in kinetic energy versus velocity.

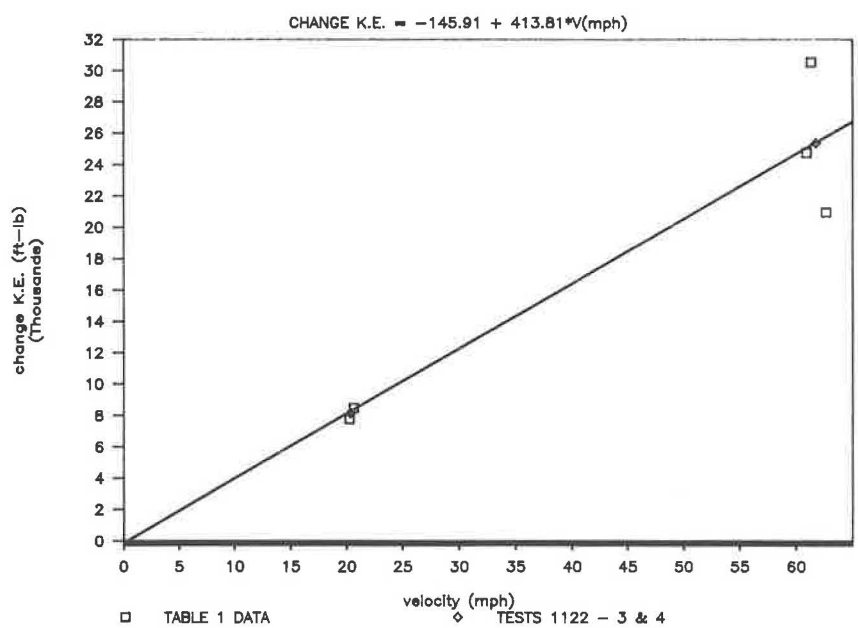


FIGURE 7 Nonbreakaway steel U-post (4 lb/ft)—change in kinetic energy versus velocity.

mph. This problem occurred because the calculated change in velocity was greater than the initial velocity. Therefore, our calculations agreed well with the first set of tests.

The next set of tests involved two 4-lb/ft U-posts with ground splices (Tests 6 and 7). The changes in kinetic energy were calculated from the line fit in Figure 6 and the changes in velocity listed in Table 2. Again there is good correlation (less than 10 percent difference) between the predictions and the actual values.

Tests 8 and 9 involved single 2½-in. standard steel pipe in a threaded coupler. Figure 9 provided the equation to predict the changes in kinetic energy. The comparison indicates that the calculated values do not agree very well with the actual values (see also Table 2). However, an upper bound estimate can be calculated from the scatter in the data. The largest vertical error between the crash test data and the "best-fit" line was used to construct a parallel offset line that provides a much better estimate.

The final set of tests, 10 and 11, involved a 3-in. pipe tree mounted on a triangular slip base. The changes in velocity were calculated using the linear and the cubic fits from Figures 11 and 12, respectively. As originally thought, the linear fit

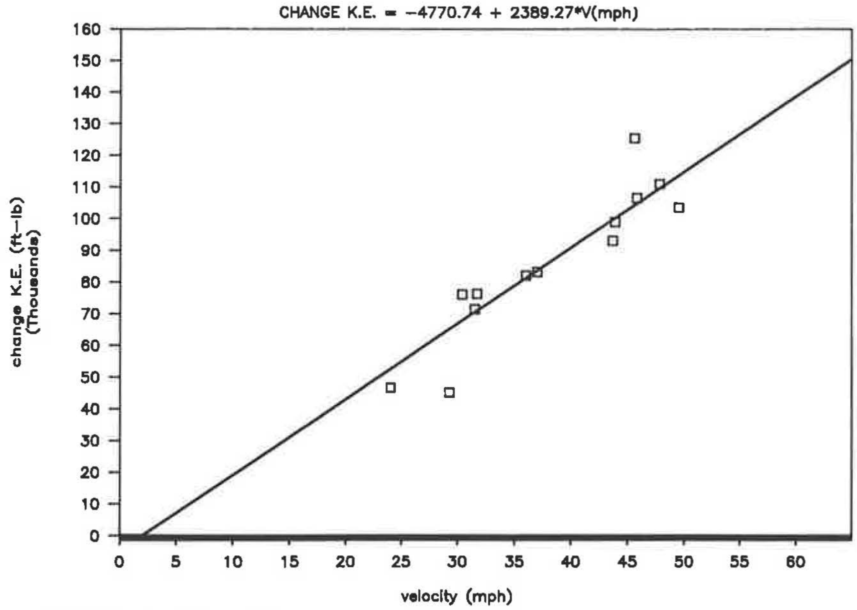


FIGURE 8 Steel U-post (8 lb/ft)—change in kinetic energy versus velocity.

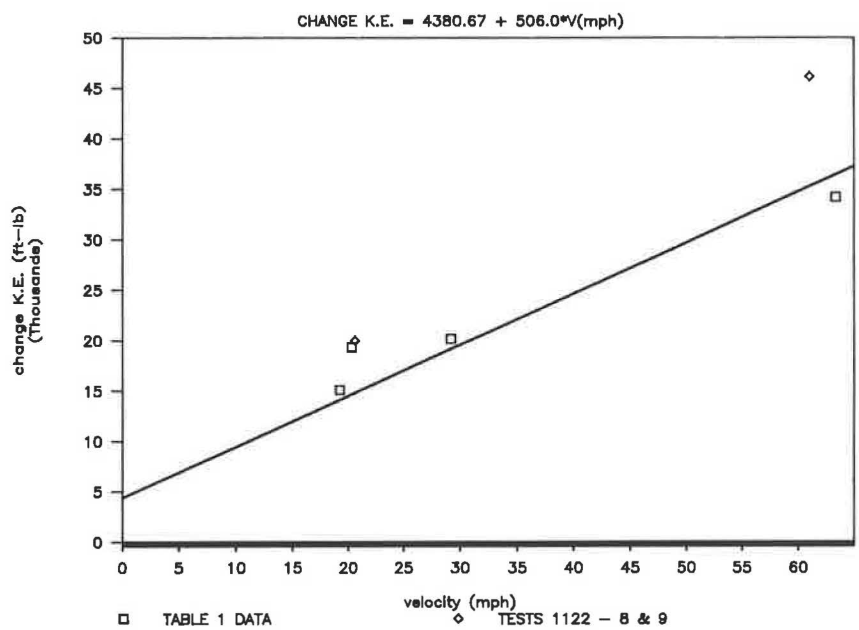


FIGURE 9 Breakaway 2½-in. standard steel pipe—change in kinetic energy versus velocity.

did not come close to predicting the car's performance even with an offset. However, the data supported the third-order fit much more closely. The low-speed prediction came within about 3 percent of the actual change in velocity. The high-speed prediction with only one previous data point estimated the change in velocity to within 19 percent.

CONCLUSIONS

The technique presented provides a method for predicting vehicle performance from existing crash data. It appears that the change in kinetic energy during impact, for specific sign systems, follows a consistent trend compared with the impact velocity regardless of vehicle size, sign mounting height, size of sign, and so forth.

The relationship between kinetic energy and impact velocity appears to be linear for most sign systems, breakaway or not. The 8-lb/ft U-post data demonstrate this trend for a wide range of intermediate impact speeds. This trend also is supported by recent tests (*12*). When there are few data or large scatter in the data, the method may not provide reasonable predictions. In these cases, use of a parallel offset line should provide adequate estimates for determining the critical tests.

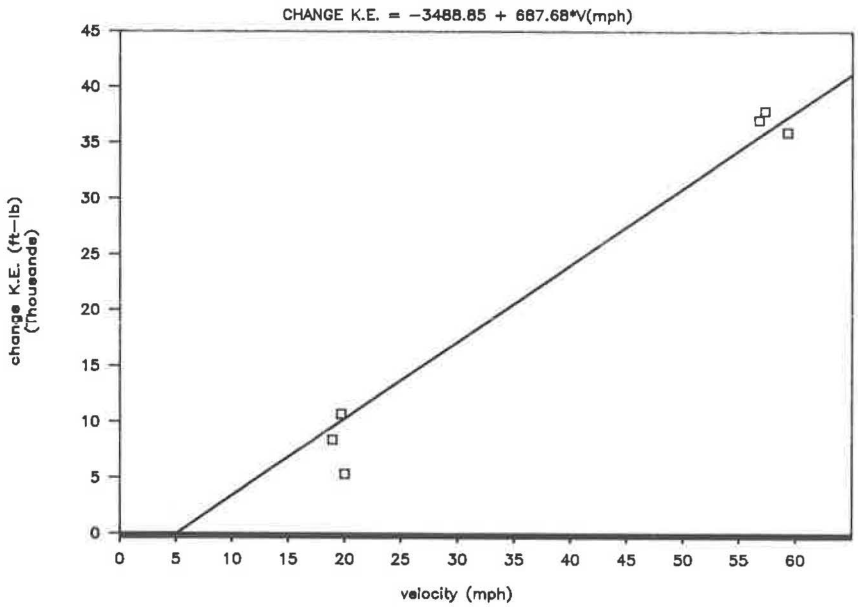


FIGURE 10 Unistrut post (2 in. × 2 in.)—change in kinetic energy versus velocity.

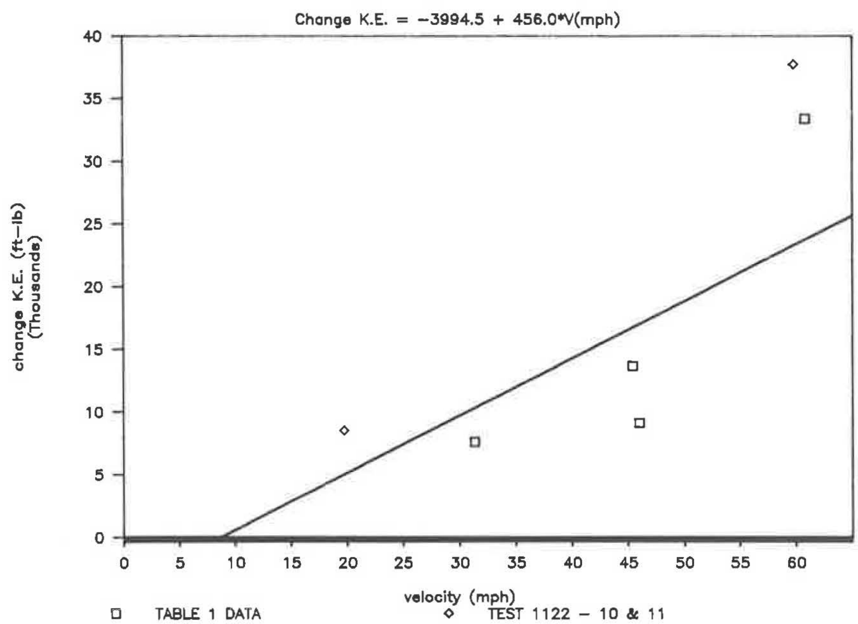


FIGURE 11 Change in kinetic energy versus velocity for 3-in. pipe on triangular slip base—linear relationship.

In one such case, the predicted changes in velocity were high, and in fact when the system was tested it proved to be marginal.

The one notable exception to the linear fit was the triangular slip base. This system, because of its unique failure mechanism, is more appropriately modeled by a cubic equation of best fit. Including the new test data would certainly improve the predictions; however, use of the current cubic equation is not recommended.

One key observation from the new tests is that breakaway systems that do not actuate should be included as nonbreakaway systems for analysis. Examples of this type of behavior may result from improper installation, excessive material strength, or large soil deformation. As data become available, this method of analysis could be extended to weak-soil applications and systems with characteristically large soil deformation. For now, it only applies to the existing crash test data base which, until recently, only included strong-soil tests.

More tests would increase confidence in the estimates provided using these energy calculations. However, a good deal of information already exists for many types of sign-support systems. The calculations of change in kinetic energy indicate that many systems have a large margin of safety (so that further testing should not be needed). For the systems that are borderline, or for extending the allowable number of posts,

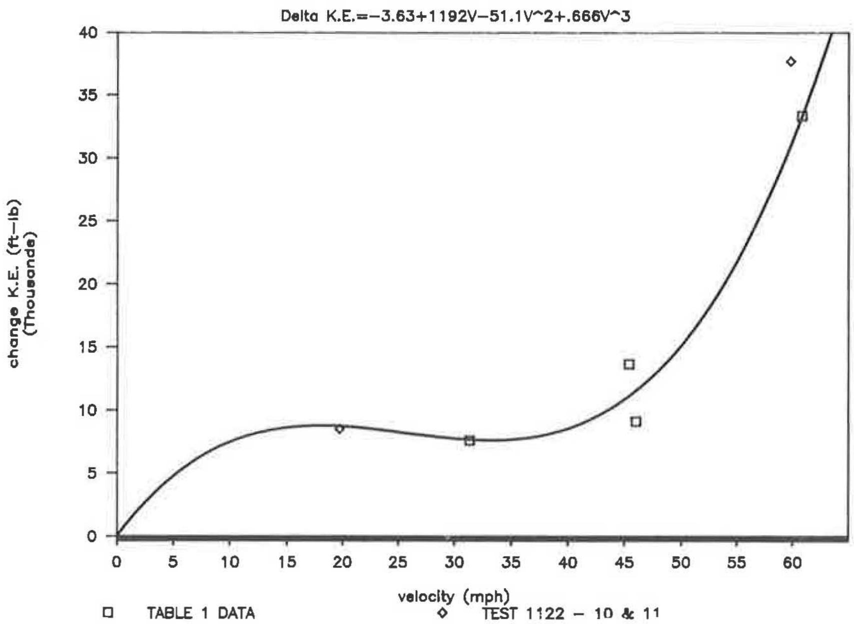


FIGURE 12 Change in kinetic energy versus velocity for 3-in. pipe on triangular slip base—cubic relationship.

TABLE 2 COMPARISON OF CHANGES IN VELOCITY

V_i (mph)	No. of Posts	ΔV (ft/sec) Actual	Estimated
Tests 3 and 4[a]			
20.27	3	33.25	>29.73
20.27	1	5.37	5.46
61.67	3	16.56	16.56
61.67	1	5.16	5.16
Tests 6 and 7[b]			
18.89	2	10.60	11.14
18.89	1	4.68	4.88
60.46	2	7.96	8.74
60.46	1	3.89	4.25
Tests 8 and 9[c]			
20.58	1	16.16	10.65
20.58	1	16.16	15.52[d]
61.03	1	9.75	7.63
61.03	1	9.75	8.37[d]
Tests 10 and 11[e]			
19.67	1	5.87	3.27[f]
19.67	1	5.87	6.06[g]
59.77	1	8.07	4.88[f]
59.77	1	8.07	6.55[g]

[a]Three nonbreakaway 4-lb/ft posts.
[b]Two breakaway 4-lb/ft posts.
[c]Two and one-half in. pipe with threaded coupler, offset = 4,660 ft-lb.
[d]With offset.
[e]Three-in. pipe on triangular slip base.
[f]Linear.
[g]Cubic.

this method can at least identify the critical tests and possibly reduce the number of tests required.

ACKNOWLEDGMENTS

Much of the research included herein was conducted under a cooperative program between the Texas Transportation Institute, the Texas State Department of Highways and Public Transportation, and the Federal Highway Administration. Hayes E. Ross, Jr., and Lewis R. Rhodes, Jr., worked closely with the researchers and provided valuable input to this study. The authors appreciate their comments and suggestions.

REFERENCES

1. M. E. Bronstad and J. D. Michie. *NCHRP Report 153: Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., 1974.
2. *Transportation Research Circular 191: Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., Feb. 1978.
3. J. D. Michie. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., 1981.
4. *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals.* American Association of State Highway and Transportation Officials, Washington, D.C., 1985.
5. J. R. Morgan, H. E. Ross, and R. E. Schuler. *Small Sign Support Analysis: Static, Pendulum and Full-Scale Crash Test Programs.* Draft Report 7024-2. Texas Transportation Institute, Texas A&M University, College Station, Aug. 1987.

6. H. E. Ross, K. C. Walker, and M. J. Effenberger. *Crash Tests of Small Highway Sign Supports*. Report FHWA/RD-80/502. FHWA, U.S. Department of Transportation, May 1980.
7. H. E. Ross, T. J. Dolf, and K. C. Walker. *Crash Test Evaluation of Small Signs Impacted from the Side*. Research Report 0941-1. Texas Transportation Institute, Texas A&M University, College Station, Oct. 1979.
8. J. A. Bloom and J. A. Hinch. *Laboratory Evaluation of Existing Breakaway Structures, Volume II Technical Results*. Report FHWA/RD-79/140. ENSCO, June 1980.
9. H. E. Ross, J. L. Buffington, G. D. Weaver, and D. L. Schafer. *State of the Practice in Supports for Small Highway Signs*. Research Report 3254-1. Texas Transportation Institute, Texas A&M University, College Station, June 1977.
10. H. E. Ross, J. L. Buffington, G. D. Weaver, and D. L. Schafer. *State of the Practice in Supports for Small Highway Signs*. Technology Sharing Report 80-222. FHWA, U.S. Department of Transportation, April 1980.
11. H. E. Ross, Jr., H. S. Perera, D. L. Sicking, and R. P. Bligh. *Roadside Safety Design for Small Vehicles*. Preliminary draft final report on Project 7043. Texas Transportation Institute, Texas A&M University, College Station, April 1988.
12. J. R. Morgan and D. L. Bullard, Jr. *Crash Test Reports for Project 1122, Generic Small Sign Support System and Validation of Existing Support Performance*. Texas Transportation Institute, Texas A&M University, College Station, Sept. 1987–Aug. 1989.

The contents of this paper reflect the views of the authors, who are responsible for the opinions, findings, and conclusions presented herein. This paper does not necessarily reflect the official views or policies of the Texas Transportation Institute, the Texas State Department of Highways and Public Transportation, or the Federal Highway Administration.

Crash Test of Modified Texas C202 Bridge Rail

T. J. HIRSCH AND PERRY ROMERE

In 1980 a standard Texas traffic rail, C202, was modified to increase its height and strength to restrain and redirect an 80,000-lb (36 300-kg) van-type tractor-trailer under 50-mph (80.5-km/hr), 15-degree-angle impacts. The concrete parapet height was increased to 36 in. (91 cm), and an elliptical steel rail was mounted on steel posts to increase the rail height to 54 in. (137 cm). In 1980 one crash test was conducted on the bridge rail. The truck was restrained and smoothly redirected. This promising high-performance bridge rail was not tested at that time with passenger cars. The results of two successful crash tests with a 1,918-lb (871-kg) car traveling at 61.3 mph (98.6 km/hr) striking at a 21-degree angle and with a 4,400-lb (1998-kg) car traveling at 59.4 mph (95.6 km/hr) striking at a 25.9-degree angle are presented.

The bridge rail tested was selected and designed to restrain and redirect an 80,000-lb (36 287-kg) van-type tractor-trailer in 1980 (*1,2*). The design was based on procedures and test data presented by Hirsch (*3*) and Buth (*4*).

The rail was a modification of the concrete parapet, Texas traffic rail type C202. The modified C202 rail consisted of a concrete beam element 13 in. (33 cm) wide and 23 in. (58 cm) deep, mounted 36 in. (91 cm) high on concrete posts located at 10-ft (3-m) center-to-center spacing. The posts were concrete walls 7 in. (19 cm) thick × 5 ft (1.5 m) long with 5-ft (1.5-m) openings. The beam element contained considerable reinforcing steel and provides flexibility, thus minimizing cracking of the concrete when struck by heavy vehicles. The modified C202 concrete parapet can be placed in lengths that give good structural continuity and strength.

To increase the effective height of this bridge rail, another standard Texas steel rail, designated C4, was mounted on top of the concrete rail. The bridge deck strength was also increased to minimize cracking or damage when the bridge rail is struck by a heavy vehicle.

Research Report 230-4F (*1*) and Hirsch (*2*) presented the results of a crash test on this bridge rail that successfully redirected an 80,000-lb (36 287-kg) tractor-trailer traveling at nominally 50 mph (80 km/hr) and striking at a 15-degree angle. In addition to successfully redirecting the tractor-trailer, the modified C202 bridge rail with the C4 metal rail on top must also redirect a 1,800-lb (810-kg) automobile and a 4,500-lb (2025-kg) automobile in order to meet all of the requirements set forth in *NCHRP Report 230* (*5*).

Texas Transportation Institute, Texas A&M University System, College Station, Tex. 77843-3135.

DESCRIPTION OF BRIDGE RAIL AND DECK MODIFICATIONS

Drawings of this rail are shown in Figures 1 and 2. Figure 3 contains photographs comparing the size of the combination bridge rail with the truck used in previous crash tests (*1,2*).

The strength of the standard Texas bridge deck 7.5 in. (19 cm) thick was increased by the addition of welded wire fabric centered under each post and along the deck steel to within 1 in. (2.5 cm) of the edge of the slab. A drawing of the welded wire fabric is shown in Figure 4. The deformed wire has a minimum yield strength of 70 ksi (48.3 kN/cm^2), and the smooth wire has a minimum yield strength of 65 ksi (44.9 kN/cm^2).

The concrete post was 13 in. (33 cm) high × 7 in. (17.8 cm) thick × 60 in. (152 cm) long with a 60-in. (152-cm) open space between each post. Each concrete post was anchored to the bridge deck by means of 13 No. 4 bars (traffic side) and 5 No. 4 bars (field side). The 13 No. 4 bars contained an 8-in. (20-cm) lap splice on top of the bridge deck that was intended as a breakaway connection.

The concrete rail on top of the post was 13 in. (33 cm) thick × 23 in. (58 cm) high for the entire length of the rail. It contained two sections of square spiral, as shown, with 10 No. 8 bars along the length of the rail. The twin spirals were used instead of a single spiral because the square spiral was available from a producer of Texas standard prestressed square piling that requires this type of spiral.

The steel rail on top of the modified C202 concrete rail was the Texas standard C4 steel rail. It was made from standard steel pipe 6 in. (15 cm) in diameter (ASTM A53 Grade B) shaped into an 8-in. × 4⅞-in. (20-cm × 12.4-cm) ellipse and welded to a post and base plate made of 1-in. (2.54-cm) steel plates. This post was anchored to the concrete rail by means of four A325 bolts ¾ in. in diameter and 15 in. (38 cm) long. A high-cast steel conical washer was installed under each bolt nut. These washers were evidently the standard being supplied by the fabricator for this type of Texas bridge rail. The standard drawing indicates that only washers are to be supplied.

All steel bars in the concrete post and rail were grade 60, including the bent bars that anchor the post to the deck. The deck steel bars were grade 40. The concrete for the deck, post, and rail was such that its strength was 3,000 psi (2.068 kN/cm^2) at the time of the test.

HONDA CRASH TEST (TEST 1179-1)

This bridge rail was crash-tested with a 1979 Honda Civic weighing 1,750 lb (795 kg) but with a gross weight of 1,918

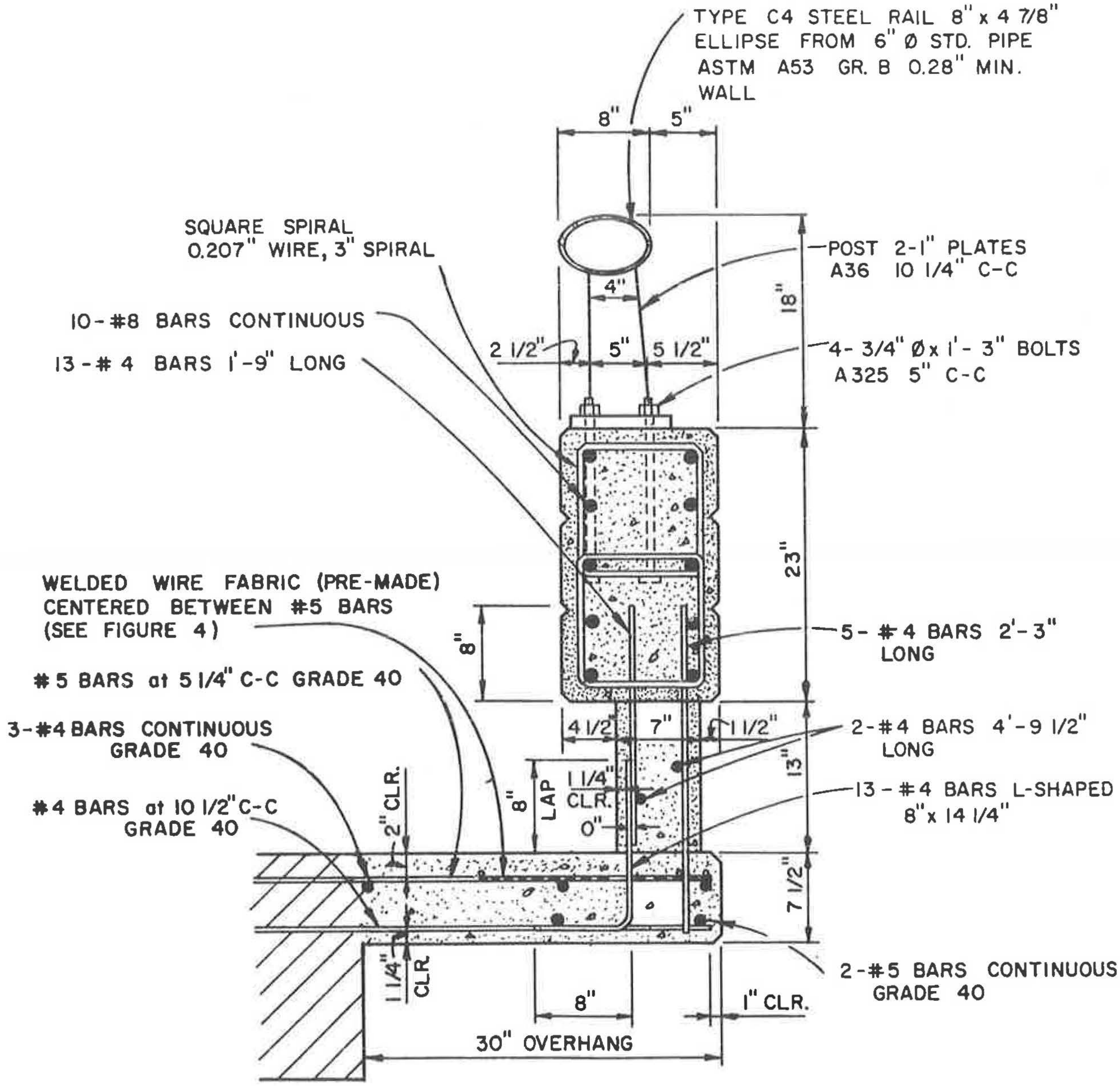


FIGURE 1 Cross section of modified C202 bridge rail.

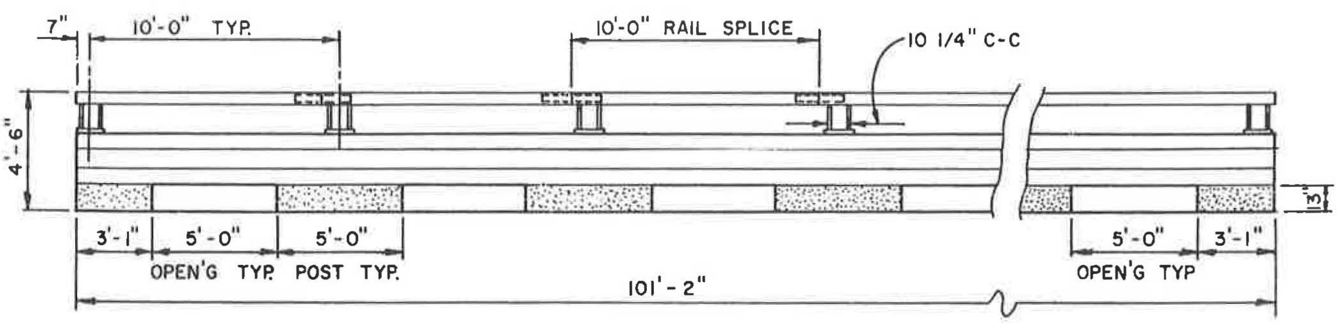


FIGURE 2 Elevation of modified C202 bridge rail.

FIGURE 3 Comparison of 80,000-lb truck with modified combination rail.

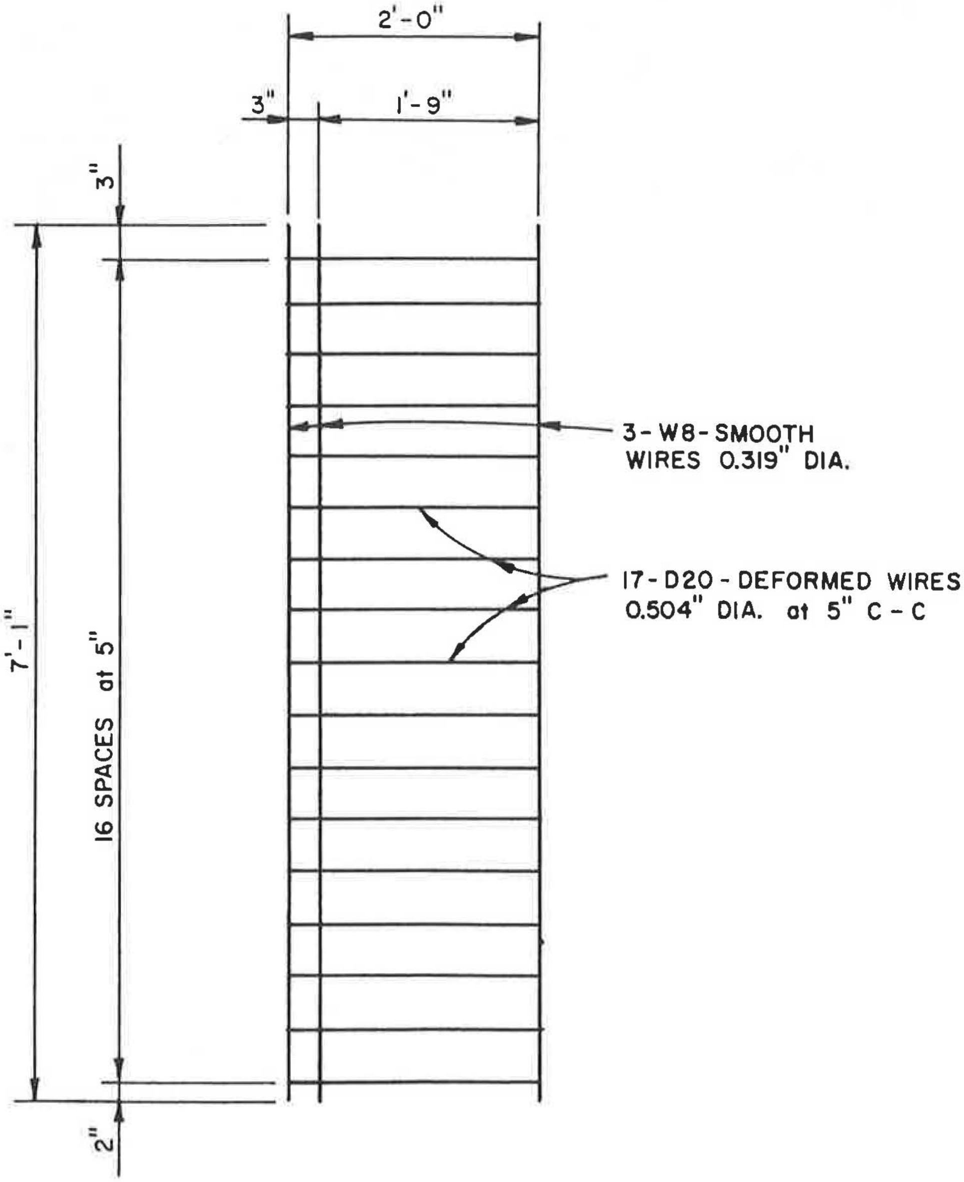


FIGURE 4 Detail of special slab reinforcement used under each concrete post.

FIGURE 5 Vehicle before and after Test 1179-1.

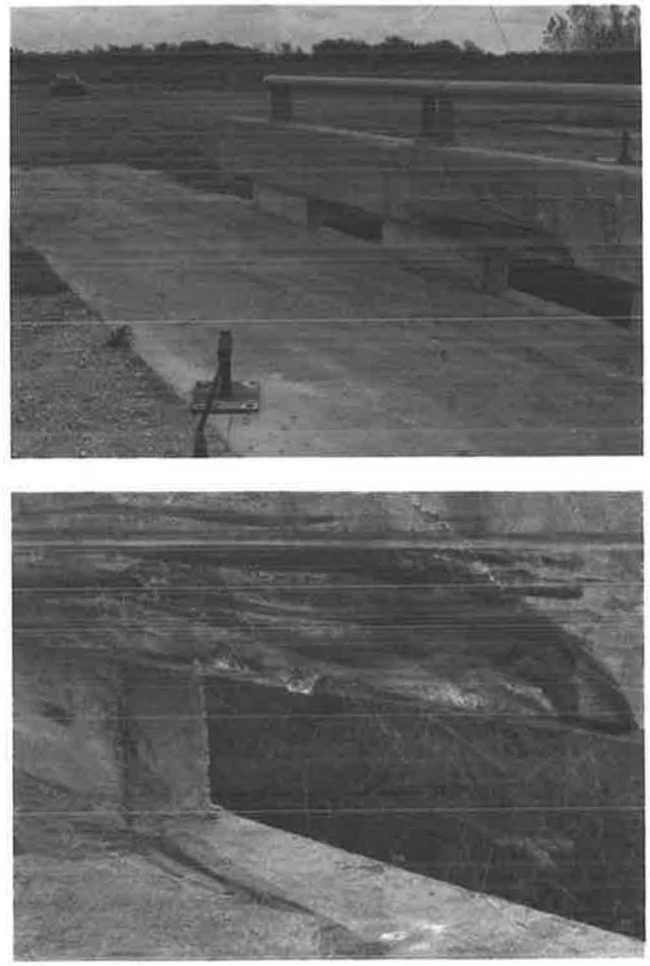


FIGURE 6 Bridge rail after Test 1179-1.

lb (871 kg) including a dummy. Photographs of the Honda before and after the test are presented in Figure 5.

The Honda struck the rail at 61.3 mph (98.6 km/hr) at a 21-degree angle. The impact occurred 7.0 ft upstream of Post 11 and was smoothly redirected. The exit angle of the Honda was only 0.6 degrees, and the car would have remained on the right-hand shoulder and not reentered the traffic lanes. Figure 6 shows the bridge rail and test site immediately after Test 1179-1. The Honda sustained damage to the right front and right side. The right front tire came in contact with Post 11, which can be seen in Figure 6. This contact caused some damage to the front right wheel and suspension; however, the wheel was still rolling after impact. An anthropomorphic dummy was placed in the driver's seat for this test. A summary of the crash test data is shown in Figure 7.

The Honda was equipped with roll, pitch, and yaw rate gyros, an x, y, and z accelerometer group on the floorboard 14.2 in. in front of the center of gravity, and an x and y accelerometer group 50.8 in. behind the center of gravity. Graphs of the filtered data from this instrumentation are presented in Figure 8, which shows a plot of the maximum 0.050-sec average accelerations along the vehicle length at 0.050 sec after impact. This is when the maximum lateral vehicle acceleration at the center of gravity occurred.

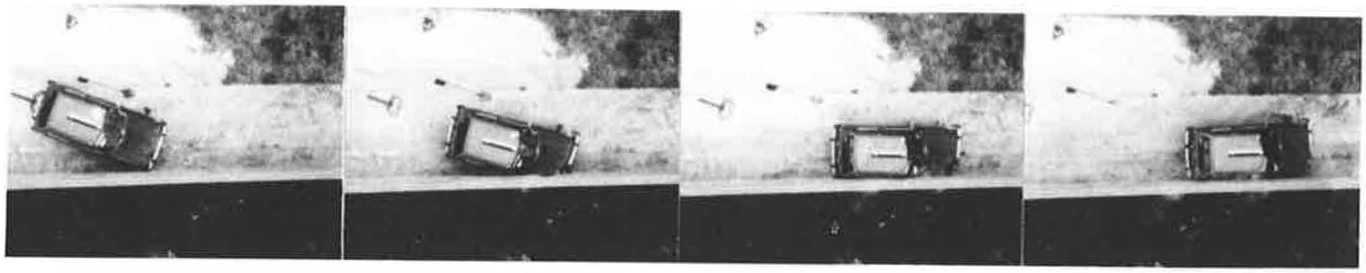
The vehicle and barrier met all of the evaluation criteria required by *NCHRP Report 230* (*5*) and the *Guide Specifications for Bridge Railings* (*6*).

CADILLAC CRASH TEST (TEST 1179-2)

This bridge rail was crash-tested with a 1979 Cadillac weighing 4,400 lb (1998 kg). Photographs of the Cadillac before and after the test are presented in Figures 9 and 10.

The Cadillac struck the rail at 59.4 mph (95.6 km/hr) and at a 25.9-degree angle. Impact occurred 7.5 ft upstream of Post 11 and was smoothly redirected. Figure 11 shows the bridge rail and test site immediately after Test 1179-2. The Cadillac sustained damage to the right front and right side. The right front tire made light contact with concrete Post 11 and the hood came in contact with the metal post directly above concrete Post 11, as shown in Figure 12. This contact caused slight damage to the front right tire and suspension; however, the wheel was still rolling after contact. Severe damage to the hood resulted when it struck the steel post. The impact cracked the right front windshield, which is shown in Figure 10. The hood pushed the windshield inward several inches but did not penetrate the passenger compartment. A summary of the crash data is shown in Figure 13.

The Cadillac was equipped with roll, pitch, and yaw rate gyros, an x, y, and z accelerometer group on the floorboard 16.2 in. in front of the center of gravity, and an x and y accelerometer group 104.8 in. behind the center of gravity. Graphs of the filtered data from this instrumentation are pre-



0.000 s 0.064 s 0.131 s 0.200 s

Test No	1179-1
Date	11/24/87
Test Installation	C202 Bridge Rail with C4 Steel Rail
Length of Installation	101 ft (31 m)
Vehicle	1979 Honda Civic
Vehicle Weight	
Test Inertia	1,750 lb (795 kg)
Gross Static	1,918 lb (871 kg)
Vehicle Damage Classification	
TAD	01RFQ5
CDC	01RYAS4

Impact Speed	61.3 mi/h (98.6 km/h)
Impact Angle	21.0 deg
Exit Speed	44.5 mi/h (71.6 km/h)
Exit Angle	0.6 deg
Vehicle Accelerations at C.G. (Max. 0.050-sec Avg)	
Longitudinal	-10.2 g
Lateral	+14.0 g
Occupant Impact Velocity	
Longitudinal	23.3 ft/s (7.1 m/s)
Lateral	25.7 ft/s (7.8 m/s)
Occupant Ridedown Accelerations	
Longitudinal	2.0 g
Lateral	-9.3 g

FIGURE 7 Summary of results for Test 1179-1.

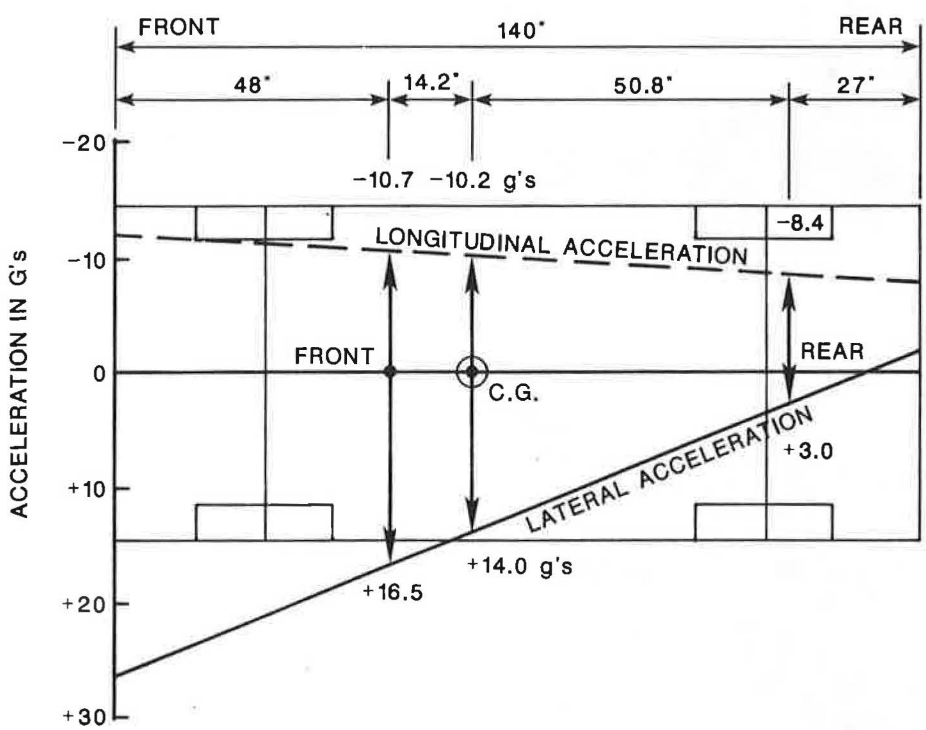


FIGURE 8 Test 1179-1—graph of maximum 0.050-sec average acceleration along vehicle length at 0.050 sec after impact.

FIGURE 9 Vehicle before Test 1179-2.

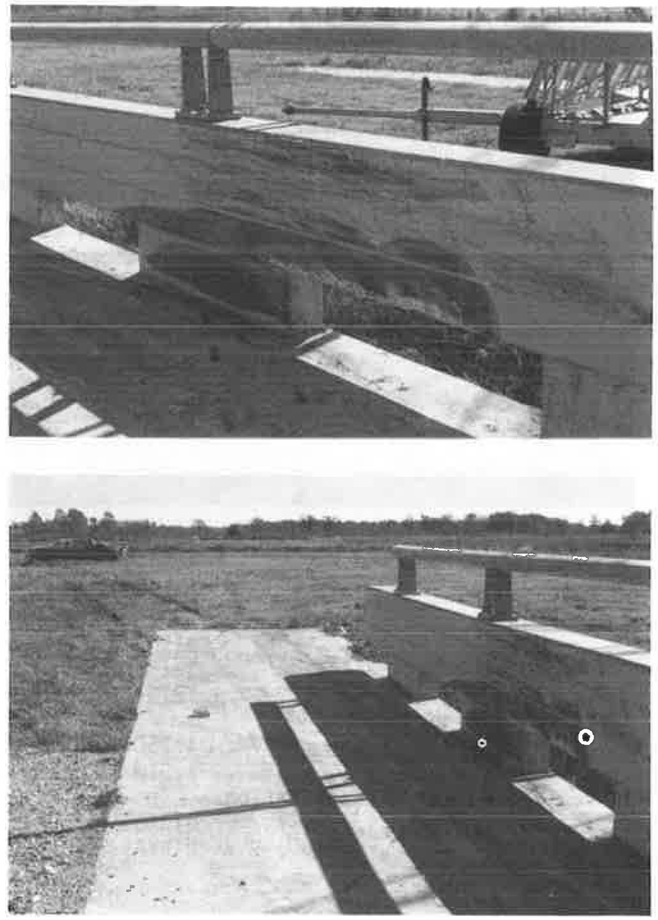


FIGURE 11 Bridge rail after Test 1179-2.

FIGURE 10 Vehicle after Test 1179-2.

sented in Figure 14, which shows a plot of the maximum 0.050-sec average accelerations along the vehicle length at 0.075 sec after impact. This is when the maximum lateral vehicle acceleration at the center of gravity occurred.

The vehicle and barrier met all of the safety evaluation criteria required by *NCHRP Report 230* (*5*) and the *Guide Specifications for Bridge Railings* (*6*).

DISCUSSION OF RESULTS

The Honda Civic test was *NCHRP Report 230* Test S13 and the Cadillac test was Test 10. For a beam-and-post system, *NCHRP Report 230* calls for the impact point to be at mid-span for both tests. However, to determine if the front wheel or hood will contact the posts, *NCHRP Report 230* suggests using a more vulnerable impact location. This was done in the two tests. The impact point was moved 2.0 ft and 2.5 ft, respectively, further upstream of the midspan location and the critical post.

The Honda Civic struck 4.5 ft upstream of the leading edge of the concrete post, and the wheel did contact the post. The damage to the wheel and suspension was moderate, but the wheel was still rolling after impact. The vehicle trajectory was excellent with a departure angle of only 0.6 degree, and the vehicle would not have returned to the traffic lanes. This test

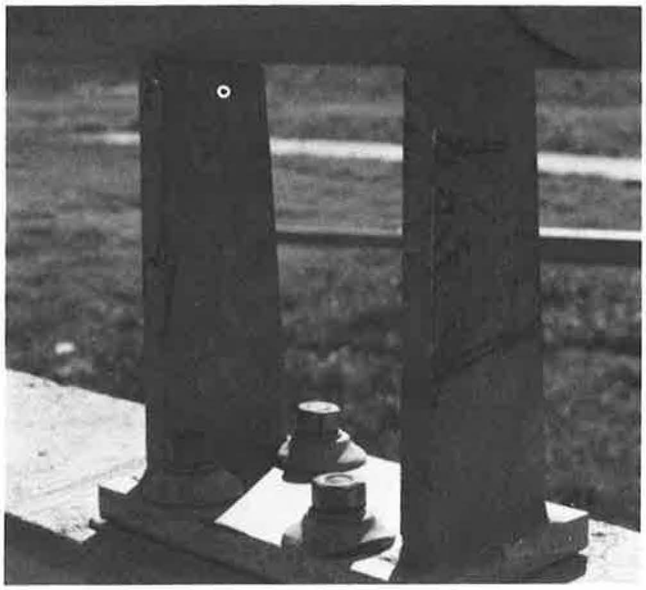


FIGURE 12 Damage to upper and lower Post 11 after Test 1179-2.

was successful and met the evaluation criteria of *NCHRP Report 230*.

The Cadillac struck 5 ft upstream of the leading edge of the concrete post and 7 ft upstream of the leading edge of the steel post. The Cadillac wheel did not contact the concrete post, and the damage to the wheel and suspension was moderate. The wheel was still rolling after impact, and the vehicle trajectory was good with a departure angle of only 2.0 degrees. The hood contacted the steel post and was severely damaged. The hood pushed the right front windshield inward several inches but it did not intrude into the passenger compartment. Consequently, the Cadillac test was judged successful.

Late-model vehicles in the 4,500-lb class are difficult to obtain. The car used was a 1979 model with a large hood that protruded 16 in. over the top of the concrete parapet. Similar vehicles (1977 Plymouths) used in tests reported elsewhere (*7*) had hoods that protruded 14 in. (Test OBR-2) and 12 in. (Test NCBR-2) over the bridge rails. Such vehicles are not representative of modern passenger cars, which have much smaller and differently shaped hoods. The older passenger car hoods extended to within 1 or 2 in. of the outside edge of the car. Modern, smaller hoods terminate 6 to 8 in. inside the outside car edge and are usually shielded by the fenders. A classic example of this is the 1,800-lb Honda Civic used in *NCHRP Report 230* Test S13 (see Figure 6 of *NCHRP Report 230*). Contact between hood and posts has never been observed in tests with this vehicle.

NCHRP Report 230 recommends that the impact position be midway between the posts for longitudinal barriers. In this study the impact positions were selected to be as severe as possible. This was done in order to provide test data on railing geometrics that would help refine the geometrics design guidelines presented by AASHTO (*6*).

Other crash test agencies have almost never moved the impact point far enough upstream of the leading edge of the posts to permit maximum underride of the wheel or override of the hood to achieve this level of interaction (*7*). The vehicle and barrier met the evaluation criteria required by *NCHRP Report 230*.

SUMMARY AND CONCLUSIONS

A standard Texas traffic rail, C202, was modified by increasing its height and strengthened so that it could restrain and redirect an 80,000-lb truck. The modified C202 rail consisted of a concrete beam element 13 in. (33 cm) wide and 23 in. (58 cm) deep, mounted 36 in. (91 cm) high on concrete posts located at 10-ft (3.0-m) center-to-center spacing. The posts were concrete walls 7 in. (19 cm) thick × 5 ft (1.5 m) long with 5-ft (1.5-m) openings between each post. To increase the effective height of the bridge rail, a standard C4 steel rail was mounted on top of the concrete rail.

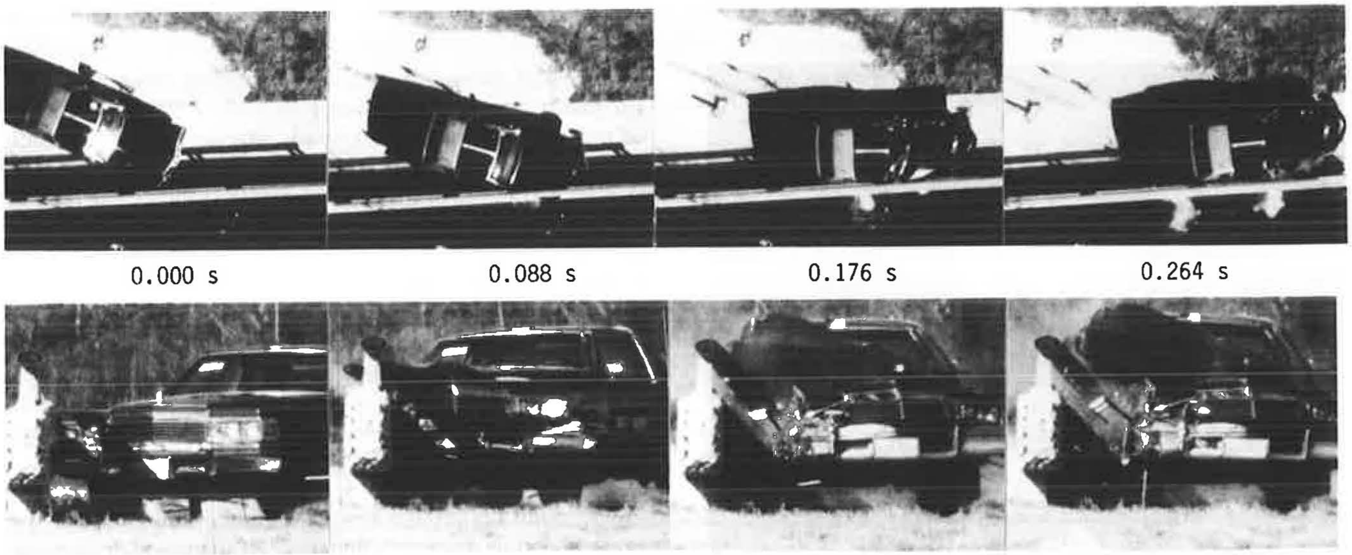
As reported in Research Report 230-4F (*1*) and Hirsch (*2*), a crash test was conducted on this bridge rail with a 79,770-lb (36 184-kg) tractor-trailer striking the rail at 49.1 mph (79.0 km/hr) at a 15-degree angle. The vehicle was smoothly redirected. Damage to the truck and rail was moderate.

This high-performance bridge rail has now been successfully crash-tested with a 1,918-lb car traveling at 61.3 mph and striking at a 21-degree angle and also with a 4,400-lb car traveling at 59.4 mph and striking at a 25.9-degree angle. The results of both tests met the evaluation criteria in *NCHRP Report 230*. The test with the Cadillac sedan was more critical than a test with a 5,400-lb pickup truck traveling at 60 mph and striking at an angle of 20 degrees. Therefore, the barrier is also considered to meet the requirements for Performance Level 3 in the new AASHTO *Guide Specifications for Bridge Railings* (*6*).

For new construction, consideration should be given to forming a 2-in. chamfer on the traffic side edge of the post. This will further reduce the potential for wheels snagging on the posts.

ACKNOWLEDGMENTS

This research was conducted under a cooperative program between the Texas Transportation Institute, the Texas State Department of Highways and Public Transportation (SDHPT),



Test No	1179-2
Date.	12/01/87
Test Installation	C202 Bridge Rail with C4 Steel Rail
Length of Installation. . . .	101 ft (31 m)
Vehicle	1979 Cadillac
Vehicle Weight	
Test Inertia.	4,400 lb (1,998 kg)
Vehicle Damage Classification	
TAD	01RFQ6
CDC	01RYAS4

Impact Speed	59.4 mi/h (95.6 km/h)
Impact Angle	25.9 deg
Exit Speed	44.5 mi/h (71.6 km/h)
Exit Angle	2.0 deg
Vehicle Accelerations at C.G. (Max. 0.050-sec Avg)	
Longitudinal	-9.7 g
Lateral.	+14.3 g
Occupant Impact Velocity	
Longitudinal	23.9 ft/s (7.3 m/s)
Lateral.	27.3 ft/s (8.3 m/s)
Occupant Ridedown Accelerations	
Longitudinal	-4.9 g
Lateral.	-16.7 g

FIGURE 13 Summary of results for Test 1179-2.

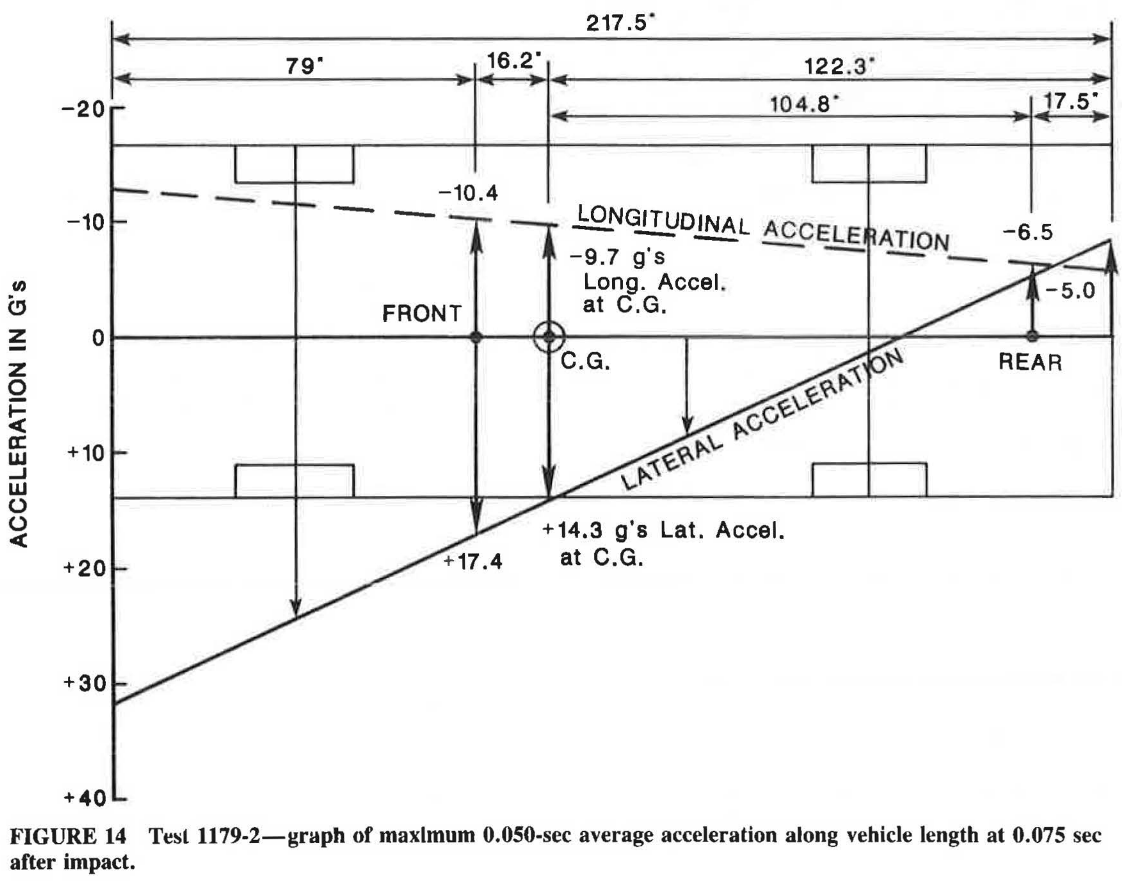


FIGURE 14 Test 1179-2—graph of maximum 0.050-sec average acceleration along vehicle length at 0.075 sec after impact.

and the Federal Highway Administration. Dean Van Landuyt and John J. Panak, both of the SDHPT, were closely involved in all phases of this study.

REFERENCES

1. T. J. Hirsch and A. Arnold. *Bridge Rail to Restrain and Redirect 80,000 lb Trucks*. Research Report 230-4F. Texas Transportation Institute, Texas A&M University, College Station, Aug. 1981.
2. T. J. Hirsch and A. Arnold. Bridge Rail to Restrain and Redirect 80,000-lb Trucks. In *Transportation Research Record 942*, TRB, National Research Council, Washington, D.C., 1983, pp. 10–16.
3. T. J. Hirsch. *Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks*. Research Report 230-2. Texas Transportation Institute, Texas A&M University, College Station, Aug. 1978.
4. C. E. Buth. *Safer Bridge Railings*. Vols. 1, 2, 3, and 4, Report FHWA/RD-82-072. Texas Transportation Institute, Texas A&M University, College Station, June 1984.
5. J. D. Michie. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., 1981.
6. *Guide Specifications for Bridge Railings—An Alternative Bridge Railing Specification in the AASHTO Standard Specifications for Highway Bridges*. AASHTO, Washington, D.C., 1988.
7. M. E. Bronstad et al. *Bridge Rail Designs and Performance Standards, Vol. I: Research Report*. Report FHWA/RD-87/049. FHWA, U.S. Department of Transportation, Feb. 1987.

Performance Level 2 Bridge Railings

C. E. Buth, T. J. Hirsch, and C. F. McDevitt

The highway profession is in the process of upgrading performance of bridge railing systems. In 1989 the American Association of State Highway and Transportation Officials adopted the *Guide Specifications for Bridge Railings*. That document addresses bridge railing systems for three levels of performance. Proof of performance should be demonstrated by full-scale crash tests set forth in that guide, and there is a general trend toward full-scale crash testing of new highway safety hardware in the highway industry. Performance level selection procedures included in the guide indicate that a performance level 2 (PL 2) railing is needed on many new bridge structures. This level has a strength test with an 18,000-lb single-unit truck striking the railing at 50 mph and at a 15-degree angle. The specified height of the center of gravity of the test truck is 49 in. Other tests with smaller vehicles are also required of a PL 2 railing. Four railing designs have been tested in a continuing pooled-funds study involving 23 states, the District of Columbia, and the Federal Highway Administration. The railings included one steel beam-and-post design and three concrete parapet designs. Performance of these railings in full-scale tests indicates that they are all acceptable for PL 2 of the 1989 guide specifications. All railings were sufficiently strong that no structural distress was observed except in the bolted rail-to-post connections in the metal railing. In all tests except one, vehicles were contained and redirected with reasonably good stability in roll and tracking with small exit angles and acceptable collision severity values. The exception was the 18,000-lb truck test on the New Jersey safety shape concrete parapet. In this test the vehicle finally rolled onto its side (away from the railing). This is considered acceptable behavior.

The Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), the National Cooperative Highway Research Program (NCHRP), and individual states have had a continuing research program on bridge railing systems including warrants, designs, testing, and evaluation of performance. In 1989 AASHTO adopted the *Guide Specifications for Bridge Railings* (*1*). This document brought together many results of recently completed and continuing research studies in a form ready for implementation by practicing highway designers.

A major pooled-funds project to study bridge railings and transitions was begun in August 1986. The project is sponsored by FHWA, the District of Columbia, and 23 states. The purpose of the study is to develop and prove, through full-scale crash tests, a collection of railing designs that would meet the needs of many of the states. Railings of different styles and various materials are to be developed so that the needs in the various climates will be best served by selections from the collection of satisfactory designs. Also, railing designs are to be developed for the various performance levels that are needed for different facilities and traffic conditions.

C. E. Buth and T. J. Hirsch, Texas Transportation Institute, Texas A&M University System, College Station, Tex. 77843-3135. C. F. McDevitt, Safety Design Division, Federal Highway Administration, 6300 Georgetown Pike, McLean, Va. 22101.

The recently adopted *Guide Specifications for Bridge Railings* includes three performance levels. These levels are defined by full-scale crash test conditions and performance evaluation criteria. The guide also recommends a procedure for determining which performance level is appropriate for a given facility and traffic condition. This procedure appears to indicate that a performance level 2 (PL 2) railing would be needed on many new and replacement bridges. As seen in Table 1, PL 2 requires a strength test with an 18,000-lb single-unit truck striking at 50 mph and at a 15-degree angle.

This paper presents the results of work performed to develop and test four railing designs to meet PL 2 requirements.

DESIGN CONSIDERATIONS

Some of the early work performed under the pooled-funds bridge rail study was devoted to consideration of test vehicles and impact conditions that would be appropriate for performance levels for bridge railings. This involved study of the collision forces generated by the various vehicles at differing impact speeds and angles and the required railing heights to provide acceptable containment and redirection of the vehicles. Much of the input information for this task was taken from two earlier studies wherein full-scale collisions were performed on an instrumented concrete wall (*2,3*). This and other FHWA in-house work finally resulted in definition of the performance levels shown in Table 1.

Data from these earlier studies indicated that the longtime standard test with a 4,500-lb automobile striking at 60 mph and at a 25-degree angle generated a maximum 0.050-sec average impact force of approximately 56 to 60 kips (two separate tests) at a height above the surface of approximately 20 in. This height was measured on the flat-faced, vertical, rigid wall, and it is not necessary to provide a resisting force at that height in order to prevent rollover of the vehicle. A resisting force at a somewhat lower height is adequate because the weight of the vehicle itself resists rollover. Tests with a 20,000-lb school bus striking a 42-in.-high instrumented wall at 58 mph and 16 degrees produced a maximum 0.050-sec average impact force of approximately 74 kips at a height of approximately 23 in. Tests with an 18,000-lb single-unit truck striking a 90-in.-high instrumented wall at 51.6 mph and 16.8 degrees produced a maximum 0.050-sec average impact force of approximately 90 kips at a height of 47 in. above the road surface. If this PL 2 truck had struck a 42-in.-high wall, the estimated impact force would be reduced to about 62 kips. Because the truck cargo box had a 50-in.-high clearance above the roadway, the impact force would only be distributed over the 42-in. tire diameter. This force with a load factor of 1.0 has been used in designing bridge railings for vehicle impacts.

TABLE 1 PERFORMANCE LEVELS FOR BRIDGE RAILINGS

PERFORMANCE LEVELS		TEST SPEEDS—mph[1,2] TEST VEHICLE DESCRIPTIONS AND IMPACT ANGLES: Small Automobile W = 1.8 Kips A = 5.4' ± 0.1' B = 5.5' H_{cg} = 20" ± 1" θ = 20 deg.	Pickup Truck W = 5.4 Kips A = 8.5' ± 0.1' B = 6.5' H_{cg} = 27" ± 1" θ = 20 deg.	Medium Single-Unit Truck W = 18.0 Kips A = 12.8' ± 0.2' B = 7.5' H_{cg} = 49" ± 1" θ = 15 deg.	Van-Type Tractor-Trailer[4] W = 50.0 Kips A = 12.5' ± 0.5' B = 8.0' H_{cg} = See Note 4 R = 0.61 ± 0.01 θ = 15 deg.
PL-1		50	45		
PL-2		60	60	50	
PL-3		60	60		50
CRASH TEST EVALUATION CRITERIA[3]	Required	a, b, c, d, g	a, b, c, d	a, b, c	a, b, c
	Desirable[5]	e, f, h	e, f, g, h	d, e, f, h	d, e, f, h

Notes:

1. Except as noted, all full-scale tests shall be conducted and reported in accordance with the requirements in NCHRP Report N0. 230. In addition, the maximum loads that can be transmitted from the bridge railing to the bridge deck are to be determined from static force measurements or ultimate strength analysis and reported.
2. Permissible tolerances on the test speeds and angles are as follows:

Speed	-1.0 mph	+2.5 mph
Angle	-1.0 deg.	+2.5 deg.

   Tests that indicate acceptable railing performance but that exceed the allowable upper tolerances will be accepted.
3. Criteria for evaluating bridge railing crash test results are as follows:
   a. The test article shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the test article is acceptable.
   b. Detached elements, fragments, or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
   c. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.
   d. The vehicle shall remain upright during and after collision.
   e. The test article shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle or, in the case of a combination vehicle, the rear of the tractor or trailer does not yaw more than 5 degrees away from the railing from time of impact until the vehicle separates from the railing.
   f. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction, μ:

μ	Assessment
0-0.25	Good
0.26-0.35	Fair
>0.35	Marginal

where $\mu = (\cos\theta - V_p/V)/\sin\theta$

   g. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 2.0-ft. longitudinal and 1.0-ft. lateral displacements, shall be less than:

Occupant Impact Velocity--fps	
Longitudinal	Lateral
30	25

   and the vehicle highest 10-ms average accelerations subsequent to the instant of hypothetical passenger impact should be less than:

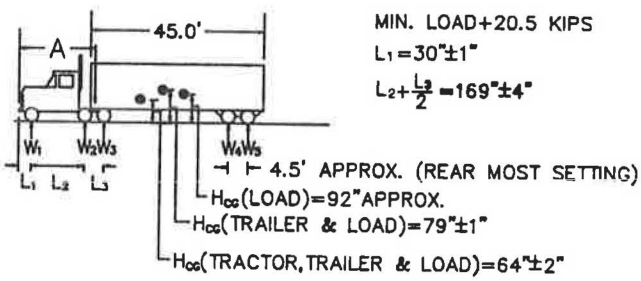
Occupant Ridedown Acceleration--g's	
Longitudinal	Lateral
15	15

   h. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 100 ft. plus the length of the test vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 20-ft. from the line of the traffic face of the railing. The brakes shall not be applied until the vehicle has traveled at least 100-ft. plus the length of the test vehicle from the point of initial impact.

(continued on next page)

TABLE 1 (*continued*)

4. Values A and R are estimated values describing the test vehicle and its loading. Values A and R are described in the figure below and calculated as follows:



$$A = L_1 + \frac{W_2L_2 + W_3(L_2+L_3)}{W_1+W_2+W_3}$$

$$R = \frac{W_1+W_2+W_3}{W}$$

$$W = W_1 + W_2 + W_3 + W_4 + W_5$$

= TOTAL VEHICLE WEIGHT

5. Test articles that do not meet the desirable evaluation criteria shall have their performance evaluated by a designated authority that will decide whether the test article is likely to meet its intended use requirements.

For metal beam-and-post railing systems, plastic mechanism analysis and design procedures with yield strengths of the materials were used. The applied load was assumed to be two line loads, each uniformly distributed along rail elements over a length of 42 in. The portion of load applied to each rail element was in the same ratio as its respective bending strength. Plastic hinges were assumed at the centers of the loads and at the ends of the rail element failure mechanisms. Plastic hinges were also assumed at the bases of all posts within the length of the failure mechanism.

For concrete parapet railings, yield line theory with unreduced ultimate strength bending moment capacities was used. The applied load was assumed to be a line load uniformly distributed along the top edge of the parapet over a 42-in. length of parapet. The failure pattern consisted of three yield lines extending from a point centered directly below the load and at the base of the parapet. One yield line extended vertically and the other two extended diagonally to the top edge of the parapet.

FULL-SCALE CRASH TESTS

Four railing designs for PL 2 have been tested and evaluated. They include

- Illinois 2399-1 metal railing,
- 32-in. vertical concrete parapet,
- 32-in. F-shaped concrete parapet, and
- 32-in. New Jersey safety shape concrete parapet.

A summary of the tests performed is presented in Table 2.

Illinois 2399-1 Metal Railing

This railing design was adapted from an existing design used by Illinois as a retrofit railing. It could also be used in new construction. The design load used for this railing was a 56,000-lb line load uniformly distributed over a 42-in. length of railing at 29 in. above the road surface. The posts used were W 6 × 25 rolled shapes spaced at 6 ft 3 in. W 6 × 15 posts would have had sufficient strength, but Illinois Department of Transportation engineers chose to retain the W 6 × 25 shape for other considerations. Total geometric height of the railing on the 7-in. curb is 32 in. A cross section of this railing is shown in Figure 1, and the prototype test installation is shown in Figure 2. After final selection of member sizes, a strength analysis based on a plastic mechanism and yield strengths of the materials indicated an ultimate load for the expected failure mechanism of approximately 80 kips, suggesting that the railing was somewhat overdesigned for strength. However, its height was marginal.

Three full-scale crash tests were performed on a prototype railing: (a) an 1,800-lb automobile striking at 60 mph and 20 degrees, (b) a 5,400-lb pickup truck striking at 65 mph and 20 degrees, and (c) an 18,000-lb single-unit truck striking at 50 mph and 15 degrees. The railing performed acceptably in all three tests.

Tests 7069-1 (1795-lb Automobile, 58.7 mph, 20.0 degrees)

The vehicle struck the railing midway between the sixth and seventh posts from the upstream end and was smoothly redirected. It was in contact with the railing for a distance of 9.7 ft and exited 0.226 sec after impact at an angle of 5.2 degrees. The vehicle was stable throughout the collision and was tracking on loss of contact with the railing.

Damage to the vehicle is shown in Figure 3. Maximum crush of the right front corner at bumper height was 8 in. There was no measurable movement or deformation of the railing.

Data and other pertinent information from this test are summarized in Figure 4. The effective coefficient of friction was calculated to be 0.28. Occupant impact velocity was 16.9 ft/sec in the longitudinal direction and 25.1 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were -1.4 *g* (longitudinal) and 8.5 *g* (lateral). The maximum 0.050-sec average accelerations of the vehicle were -6.4 *g* (longitudinal) and 14.2 *g* (lateral).

The barrier contained and smoothly redirected the vehicle with no lateral movement of the barrier. There were no detached

TABLE 2 FULL-SCALE CRASH TESTS

Test Number	VEHICLE Model	VEHICLE Test Inertia Wt (lbs)	VEHICLE Gross Static Wt (lbs)	IMPACT CONDITIONS Speed (mph)	IMPACT CONDITIONS Angle (deg)	Railing Design
1	1980 Honda Civic W.B. = 88 in.	1,975	1,961	58.7	20.0	Ill. 2399-1
2	1981 Chevrolet Pickup C-20 W.B. = 132 in.	5,450	5,797	63.6	19.2	Ill. 2399-1
15	1980 Ford 7000 SU Truck W.B. = 205 in.	12,320	18,000	50.8	15.1	Ill. 2399-1
5	1981 Honda Civic W.B. = 88.7 in.	1,800	1,965	60.5	21.0	32 in. Vertical Parapet
6	1982 Chevrolet Pickup C-20 W.B. = 132 in.	5,420	5,759	59.7	20.2	32 in. Vertical Parapet
16	1982 Ford 7000 SU Truck W.B. = 205 in.	13,820	18,000	50.0	14.0	32 in. Vertical Parapet
3	1980 Honda Civic W.B. = 88 in.	1,800	1,966	60.1	21.4	32 in. F-shape
4	1981 Chevrolet Pickup C-20 W.B. = 132 in.	5,440	5,780	65.4	20.4	32 in. F-shape
11	1982 Ford 7000 SU Truck W.B. = 220 in.	11,000	18,000	52.1	14.8	32 in. F-shape
14	1981 Chevrolet Pickup C-20 W.B. = 132 in.	5,390	5,724	57.7	20.6	32 in. New Jersey
12	1982 GMC 7000 SU Truck W.B. = 203 in.	10,900	18,000	51.6	15.5	32 in. New Jersey

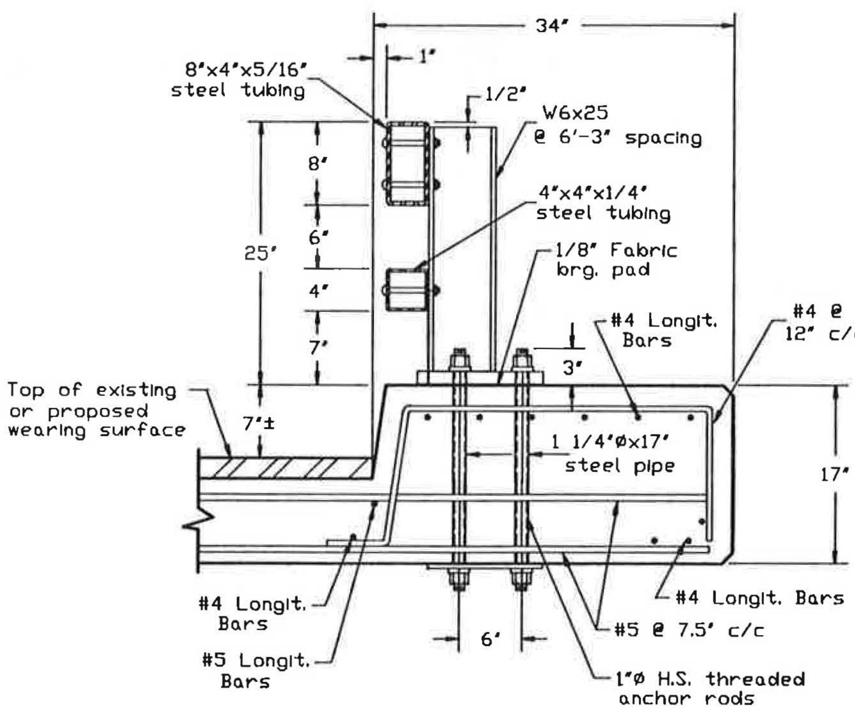
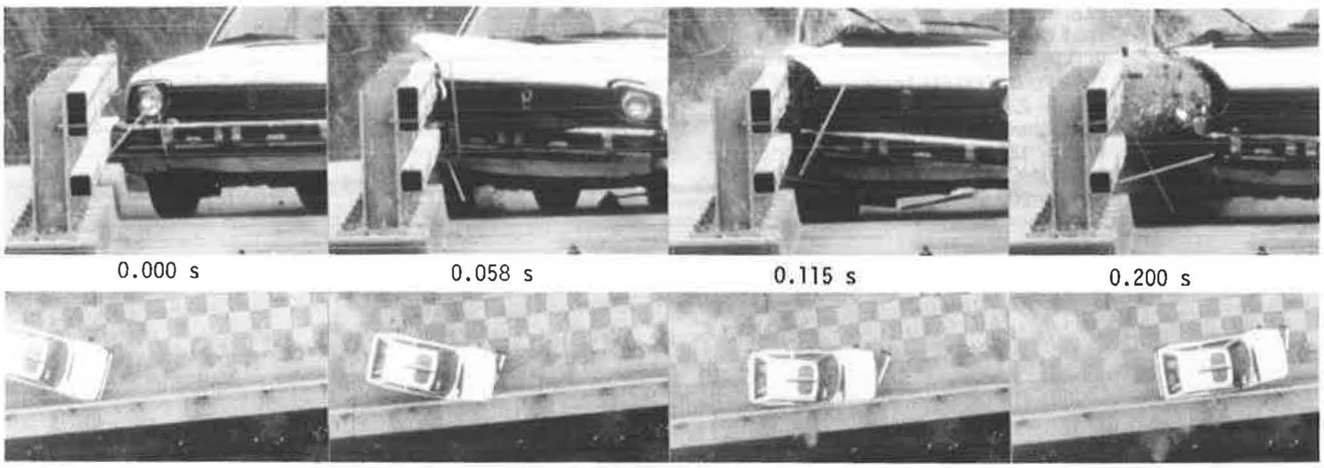
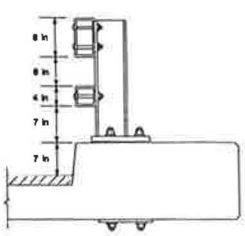


FIGURE 1 Illinois 2399-1 railing.

FIGURE 2 Prototype test installation of Illinois 2399-1 railing.

FIGURE 3 Vehicle after Test 7069-1 (1,795 lb, 58.7 mph, 20.0 degrees).





Illinois 2399
Bridge Rail

Test No. 7069-1
Date 7/14/87
Test Installation . . Illinois 2399 Bridge Rail
Installation Length . 100 ft (30 m)
Vehicle . . . . 1980 Honda
Vehicle Weight
  Test Inertia . . . . 1,795 lb (815 kg)
  Gross Static . . . . 1,961 lb (890 kg)
Vehicle Damage Classification
  TAD 01RFQ5
  CDC . . . .
Maximum Vehicle Crush 8.0 in (20.3 cm)
Maximum Dynamic Rail
  Deflection . . . . . Nil
Maximum Permanent Rail
  Deformation . . . . None

Impact Speed. . . 58.7 mi/h (94.4 km/h)
Impact Angle. . . 20.0 deg
Exit Speed. . . . 48.5 mi/h (78.0 km/h)
Exit Angle. . . . 5.2 deg
Vehicle Accelerations
  (Max. 0.050-sec Avg)
  Longitudinal. . -6.4 g
  Lateral . . . . 14.2 g
Occupant Impact Velocity
  Longitudinal. . 16.9 ft/s (5.2 m/s)
  Lateral . . . . 25.1 ft/s (7.7 m/s)
Occupant Ridedown Accelerations
  Longitudinal. . -1.4 g
  Lateral . . . . 8.5 g

FIGURE 4 Summary of results for Test 7069-1.

elements or debris. There was no intrusion into the occupant compartment. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes. The vehicle remained upright and stable during the entire collision. Performance of the railing was considered acceptable.

Test 7069-2 (5,450-lb Pickup Truck, 63.6 mph, 19.2 degrees)

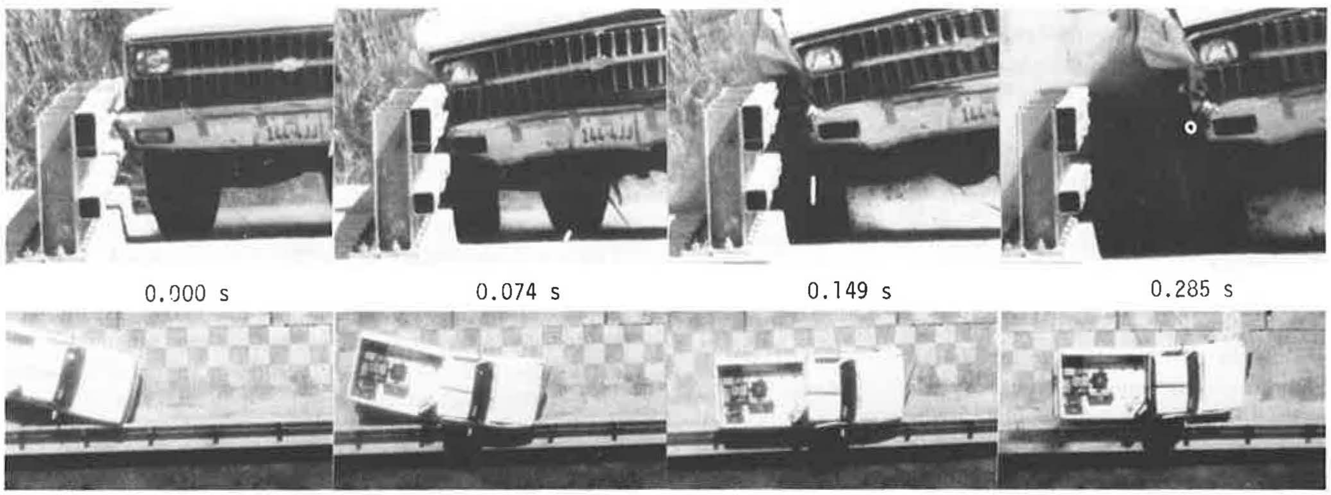
The vehicle struck the railing midway between the sixth and seventh posts from the upstream end and was smoothly redirected. It was in contact with the railing for a distance of 14.5 ft and contact ended 0.234 sec after impact. On loss of contact, the vehicle yaw angle was 1.0 degree and its trajectory was 5.8 degrees relative to the railing.

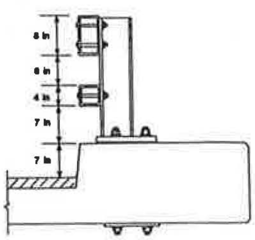
Exterior damage to the vehicle is shown in Figure 5. Both right side wheels and the front suspension were damaged. Also, the cab was twisted and the frame was permanently deformed.

Damage to the railing is shown in Figure 6. Maximum dynamic deflection of the railing was 2.4 in. and maximum permanent deflection was 0.5 in. The front of the baseplate

FIGURE 5 Damage to vehicle in Test 7069-2 (5,450 lb, 63.6 mph, 19.2 degrees).

FIGURE 6 After-test photograph at railing in Test 7069-2.





Illinois 2399
Bridge Rail

Test No. 7069-2
Date 7/24/87
Test Installation . . Illinois 2399 Bridge Rail
Installation Length . 100 ft (30 m)
Vehicle . . . . 1981 Chevrolet Pickup
Vehicle Weight
Test Inertia . . . . 5,450 lb (2,474 kg)
Gross Static . . . . 5,797 lb (2,632 kg)
Vehicle Damage Classification
TAD 01RD4
CDC . . . .
Maximum Vehicle Crush 5.0 in (12.7 cm)
Maximum Dynamic Rail
Deflection . . . . . 2.4 in (6.1 cm)
Maximum Permanent Rail
Deformation . . . . 0.5 in (1.3 cm)

Impact Speed. . . 63.6 mi/h (102.3 km/h)
Impact Angle. . . 19.2 deg
Exit Speed. . . . 57.6 mi/h (92.7 km/h)
Exit Trajectory . 5.8 deg
Vehicle Accelerations
(Max. 0.050-sec Avg)
Longitudinal. . -3.8 g
Lateral . . . . 14.3 g
Occupant Impact Velocity
Longitudinal. . 8.5 ft/s (2.6 m/s)
Lateral . . . . 24.6 ft/s (7.5 m/s)
Occupant Ridedown Accelerations
Longitudinal. . -1.1 g
Lateral . . . . 12.8 g

FIGURE 7 Summary of results for Test 7069-2.

on Post 6 was pulled up slightly and the concrete was chipped around the bolts at the rear of the baseplate.

The effective coefficient of friction was calculated to be 0.03. Occupant impact velocity was 8.5 ft/sec in the longitudinal direction and 24.6 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were -1.1 g (longitudinal) and 12.8 g (lateral). The maximum 0.050-sec averages were -3.8 g (longitudinal) and 14.3 g (lateral). These data and other pertinent information from the test are summarized in Figure 7.

The barrier contained and smoothly redirected the vehicle with minimal lateral movement of the barrier. There were no detached elements or debris. There was no intrusion into the occupant compartment. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes. The vehicle remained upright and stable during the entire test period. Performance of the railing in this test was judged acceptable.

Test 7069-15 (18,000-lb Single-Unit Truck, 50.8 mph, 15.1 degrees)

The vehicle struck the rail approximately 26 ft from the upstream end between Posts 4 and 5. Shortly after impact, the right front tire made contact with the lower rail element and began to ride the curb. As the vehicle continued its forward motion into the rail, the right front tire pushed the lower rail element down. Before becoming parallel to the railing, the left side of the vehicle became airborne. The wheels returned to the pavement just before the vehicle lost contact with the railing. Maximum roll angle of the vehicle was approximately 23 degrees. The vehicle was in contact with the railing all the way to the downstream end (approximately 74 ft).

Damage to the vehicle is shown in Figure 8. The frame of the truck was permanently deformed. The cargo box was torn during the test; as the vehicle left the rail, the load shifted and tore open the right side of the cargo box.

Damage to the railing in the vicinity of Post 6 is shown in Figure 9. The bolts connecting the lower rail element to the post were sheared on Posts 3 through 7, apparently because of vertical downward load. At Post 5, the bolt on the upper rail element was sheared and the face of the rail element itself was gouged. The flange on Post 6 was bent and the concrete curb was cracked at Posts 6 through 9. The top of Post 8 was deformed by the edge of the cargo box on the truck.

The exit angle was 0 degree. The effective coefficient of friction was calculated to be 0.11. Occupant impact velocity was 9.8 ft/sec in the longitudinal direction and 12.4 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were $-2.5\ g$ (longitudinal) and $7.4\ g$ (lateral). These data and other pertinent information from the test are summarized in Figure 10.

The bridge rail contained and smoothly redirected the vehicle with minimal lateral movement of the bridge rail. There was no intrusion into the occupant compartment and very little deformation of the compartment. The vehicle trajectory at loss of contact indicated no intrusion into adjacent traffic lanes and the vehicle remained stable during the collision. Performance of the railing was judged acceptable.

FIGURE 8 Photograph of vehicle after Test 7069-15 (18,000 lb, 50.8 mph, 15.1 degrees).

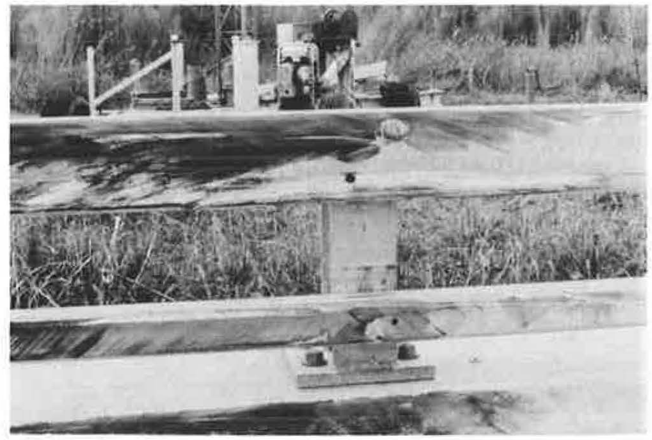


FIGURE 9 Damage to railing at Post 6.

Thirty-two-in. Vertical Concrete Parapet

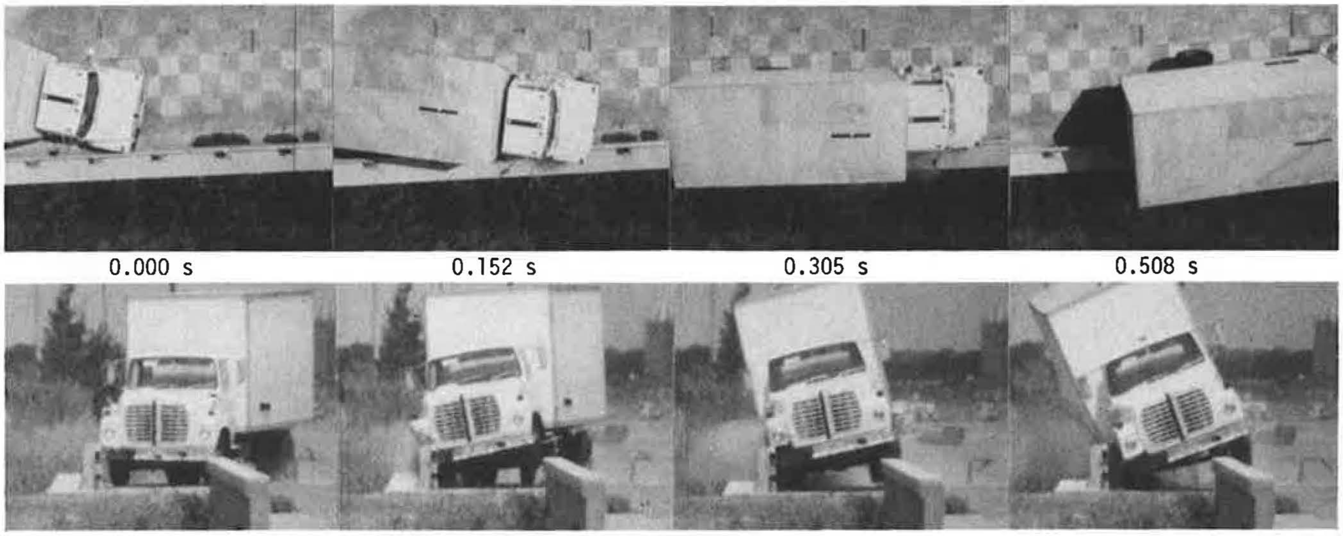
This railing was designed with a thickened section at the top of the parapet to provide additional strength and stiffness along the top edge (Figure 11). This produces a greater length of failure mechanism, which allows a greater length of parapet to carry and distribute the applied load to the deck. This railing was originally designed before the final test matrix in the 1989 guide specifications was established. When the railing was designed, the strength test requirement for a PL 2 railing was a 5,400-lb pickup truck striking at 65 mph and 20 degrees. The design force used for this test was 56 kips distributed over 42 in. and applied 29 in. above the road surface. A strength analysis of the final design showed it would resist 57 kips applied near the top (approximately 30 in.). The yield line failure mechanism, if it occurred, would be expected to extend over a 7- to 10-ft length of railing, and the railing load transferred into the deck would be expected to extend over approximately 15 ft.

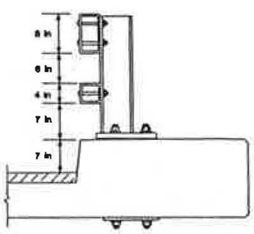
Three full-scale crash tests were performed on a prototype railing: (a) an 1,800-lb automobile striking at 60 mph and 20 degrees, (b) a 5,400-lb pickup truck striking at 65 mph and 20 degrees, and (c) an 18,000-lb single-unit truck striking at 50 mph and 15 degrees. The railing performed acceptably in all three tests.

Test 7069-5 (1,800-lb Automobile, 60.5 mph, 21.0 degrees)

The impact point for this test was at midlength of the railing. The vehicle was smoothly redirected and was stable throughout the collision. It was in contact with the railing for a distance of 10.3 ft. The vehicle lost contact with the railing 0.236 sec after impact and exited with a yaw angle of 3.5 degrees and a trajectory of 6.2 degrees.

Damage to the vehicle is shown in Figure 12. Maximum crush of the right front corner at bumper height was 5 in. Damage to the railing was cosmetic only and is shown in Figure 13.





Illinois 2399 Bridge Rail

Test No. 7069-15
Date 9/13/88
Test Installation . . Illinois 2399 Bridge Rail
Installation Length. . 100 ft (30 m)
Vehicle . . . . 1980 Ford 7000 Single-Unit Truck
Vehicle Weight
  Test Inertia . . . . 12,320 lb (5,593 kg)
  Gross Static . . . . 18,000 lb (8,172 kg)
Maximum Vehicle Crush. 10.0 in (25.4 cm)

Impact Speed. . . 50.8 mi/h (81.7 km/h)
Impact Angle. . . 15.1 deg
Exit Speed. . . . N/A
Exit Trajectory . 0 deg
Vehicle Accelerations
  (Max. 0.050-sec Avg)
  Longitudinal. . -1.9 g
  Lateral . . . . 4.9 g
Occupant Impact Velocity
  Longitudinal. . 9.8 ft/s (3.0 m/s)
  Lateral . . . . 12.4 ft/s (3.8 m/s)
Occupant Ridedown Accelerations
  Longitudinal. . -2.5 g
  Lateral . . . . 7.4 g

FIGURE 10 Summary of results for Test 7069-15.

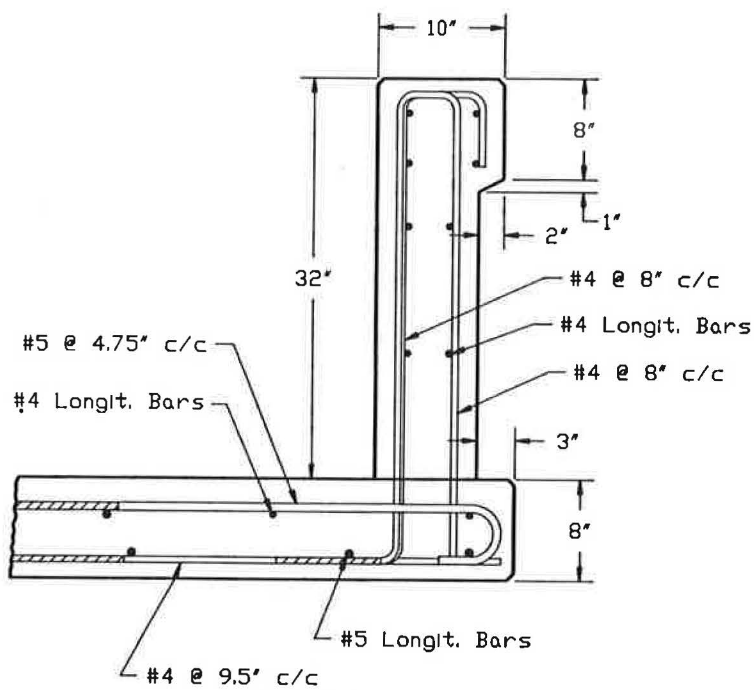


FIGURE 11 Thirty-two-in. vertical concrete parapet.

FIGURE 12 Vehicle after Test 7069-5 (1,800 lb, 60.5 mph, 21.0 degrees).

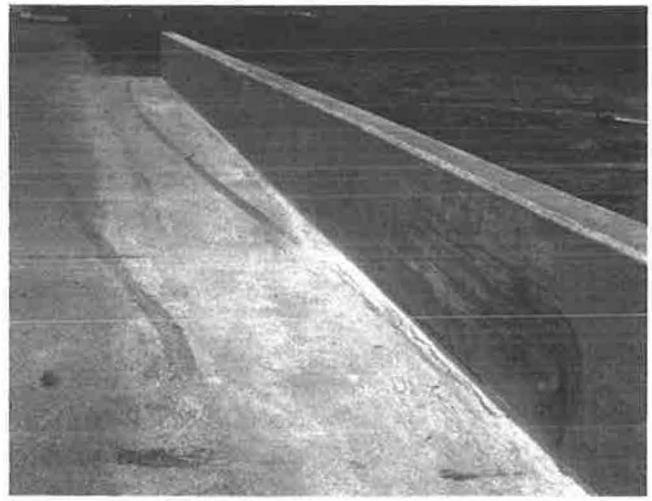
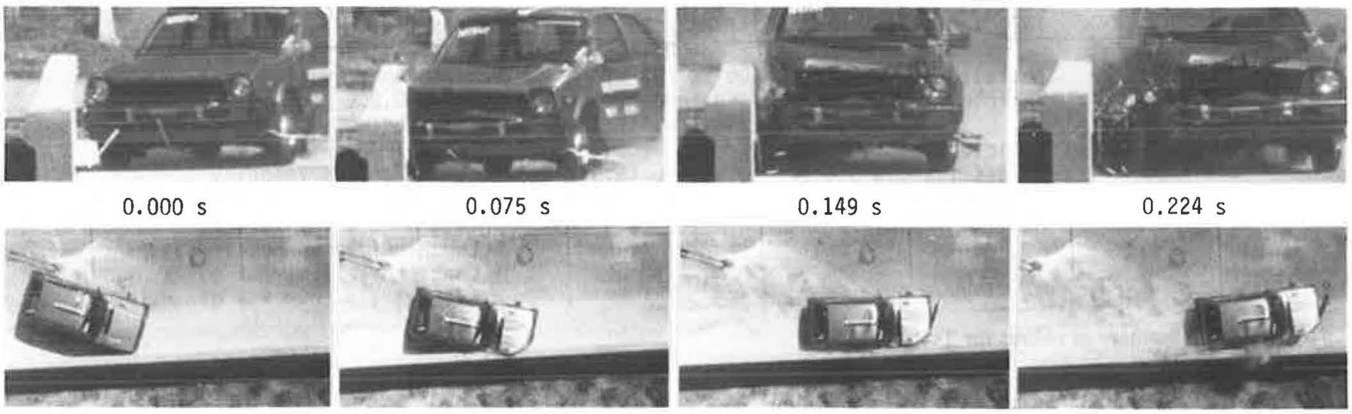
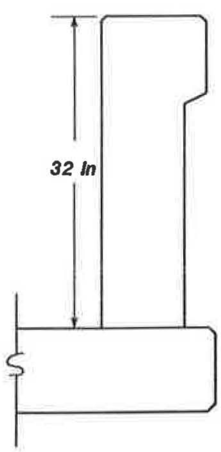


FIGURE 13 Vertical concrete parapet after Test 7069-5.





Test No.	7069-5
Date	9/24/87
Test Installation . .	32 in Vertical Wall
Installation Length .	100 ft (30 m)
Vehicle	1981 Honda Civic
Vehicle Weight	
Test Inertia	1,800 lb (817 kg)
Gross Static	1,965 lb (892 kg)
Vehicle Damage Classification	
TAD	01RFQ4
CDC	
Maximum Vehicle Crush	5.0 in (12.7 cm)

Impact Speed. . .	60.5 mi/h (97.3 km/h)
Impact Angle. . .	21.0 deg
Exit Speed. . . .	48.6 mi/h (78.2 km/h)
Exit Trajectory .	6.2 deg
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal. .	-8.0 g
Lateral	14.0 g
Occupant Impact Velocity	
Longitudinal. .	20.1 ft/s (6.1 m/s)
Lateral	26.0 ft/s (7.9 m/s)
Occupant Ridedown Accelerations	
Longitudinal. .	-1.6 g
Lateral	9.4 g

FIGURE 14 Summary of results for Test 7069-5.

Exit speed at time of contact (0.236 sec) was 48.6 mph and the vehicle trajectory was 6.2 degrees with a vehicle yaw angle of 3.5 degrees. The effective coefficient of friction was calculated to be 0.22. Occupant impact velocity was 20.1 ft/sec in the longitudinal direction and 26.0 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were -1.6 *g* (longitudinal) and 9.4 *g* (lateral). These data and other pertinent information from the test are summarized in Figure 14.

The railing contained and smoothly redirected the vehicle with no lateral movement. There were no detached elements or debris. There was no intrusion into the occupant compartment although some deformation of the compartment occurred. The vehicle trajectory indicated no intrusion into adjacent traffic lanes. The vehicle remained upright and stable during the entire collision. Performance of this railing was judged acceptable.

Test 7069-6 (5,400-lb Pickup Truck, 59.7 mph, 20.2 degrees)

The vehicle struck the railing at midlength and was smoothly redirected. It was in contact with the railing for a length of 10.5 ft. Loss of contact between the railing and the vehicle occurred at 0.418 sec. The vehicle exited with a yaw angle of 5.6 degrees and a trajectory of 6.4 degrees relative to the railing.

Damage to the vehicle is shown in Figure 15. Note that the right front wheel was separated at the welds connecting the outer and inner portion, allowing the outer portion of the wheel and tire to separate from the vehicle. The front suspension was damaged. The cab was twisted and the frame was permanently deformed. No structural distress was noted in the parapet (Figure 16).

The effective coefficient of friction was calculated to be 0.32. Occupant impact velocity was 18.6 ft/sec in the longitudinal direction and 21.1 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were -5.5 *g* (longitudinal) and 8.6 *g* (lateral). These data and other pertinent information from the test are summarized in Figure 17.

The barrier contained and smoothly redirected the vehicle with minimal lateral movement of the barrier. There were no detached elements or debris. There was no intrusion into the occupant compartment although some deformation of the right door occurred. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes. The vehicle remained upright and stable during the entire test period. Performance of the barrier was acceptable in this test.

Test 7069-16 (18,000-lb Single-Unit Truck, 50 mph, 14.0 degrees)

The impact point for this test was approximately 20 ft from the upstream end of the railing. Shortly before the vehicle became parallel to the railing, its left side became airborne.

FIGURE 15 Vehicle after Test 7069-6 (5,400 lb, 59.7 mph, 20.2 degrees).

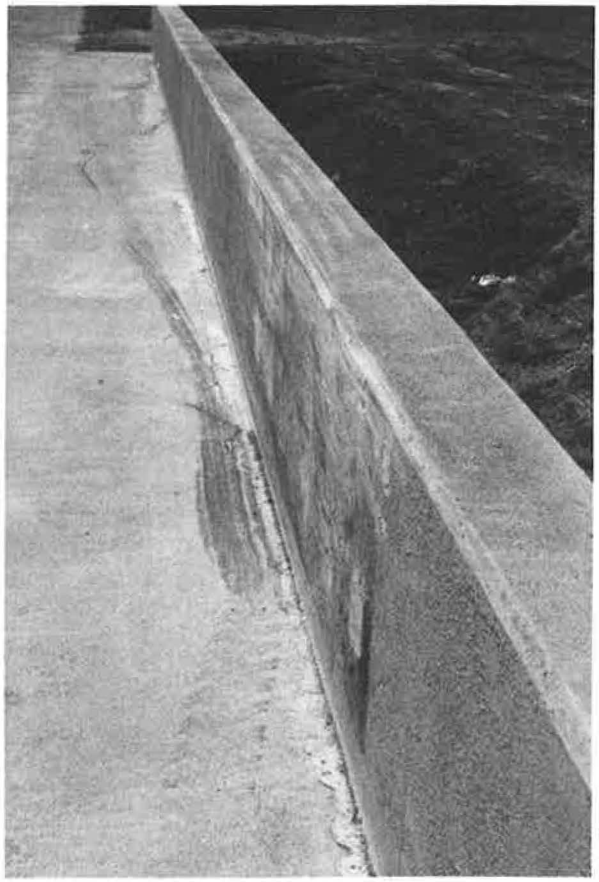


FIGURE 16 Vertical concrete parapet after Test 7069-6.

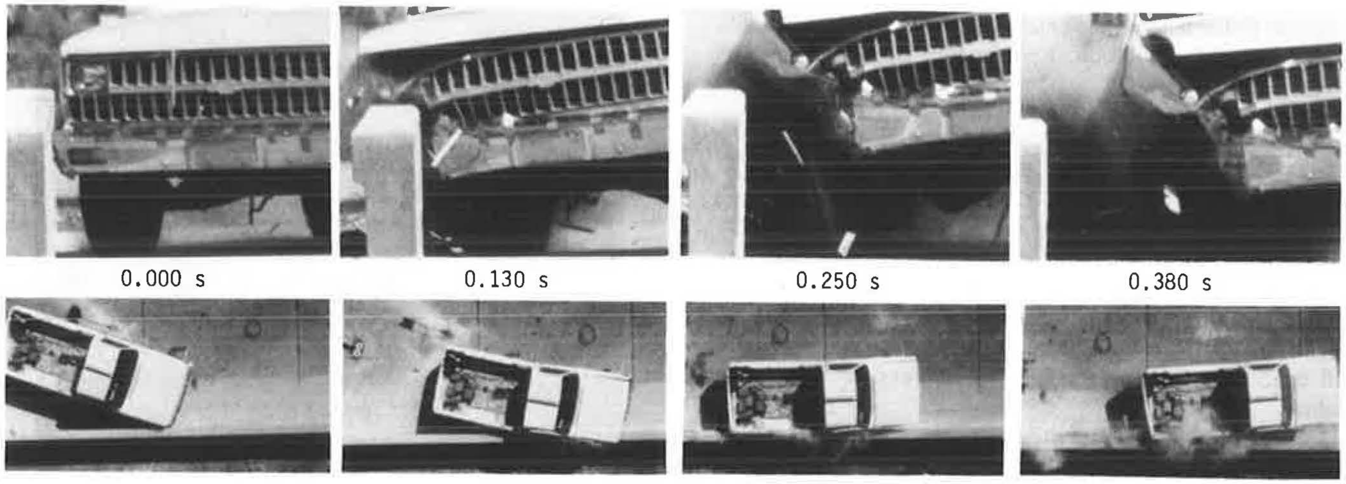
As the vehicle continued along the railing, it also continued to roll toward the railing and attained a maximum roll angle of approximately 17.6 degrees. During the collision, the lower edge of the cargo box was bearing on and sliding along the top surface of the railing. This undoubtedly helped stabilize the vehicle and may or may not occur in other railing designs. On passing the downstream end of the railing, the vehicle was steered to the right and followed a curved path, finally rolling onto its left side.

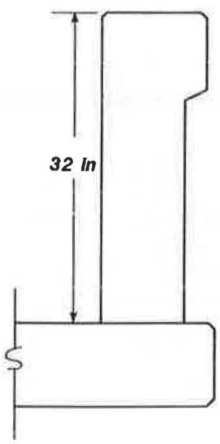
The vehicle sustained damage to its right side during interaction with the railing, as indicated in Figure 18. Maximum crush at the right front corner at bumper height was 10.0 in.

As can be seen in Figure 19, the bridge rail sustained cosmetic damage. Tire marks on the face extended to the top edge for about 30 ft. The box of the vehicle scraped the top of the bridge rail for another 15 ft. The vehicle was in contact with the bridge rail for about 45 ft.

The effective coefficient of friction was calculated to be 0.41. The vehicle left the bridge rail traveling at 34.2 mph. Occupant impact velocity was 10.9 ft/sec in the longitudinal direction and 11.8 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were -2.3 *g* (longitudinal) and 8.4 *g* (lateral). These data and other pertinent information from the test are summarized in Figure 20.

The bridge rail contained and smoothly redirected the vehicle with no lateral movement of the bridge rail. There was no intrusion into the occupant compartment and very little deformation of the compartment. The vehicle trajectory at loss of contact indicated no intrusion into adjacent traffic





Test No.	7069-6
Date	10/08/87
Test Installation . .	32 in Vertical Wall
Installation Length .	100 ft (30 m)
Vehicle	1982 Chevrolet Pickup
Vehicle Weight	
Test Inertia	5,420 lb (2,461 kg)
Gross Static	5,759 lb (2,615 kg)
Vehicle Damage Classification	
TAD	01RD4
CDC	
Maximum Vehicle Crush	9.0 in (22.9 cm)

Impact Speed. . .	59.7 mi/h (96.1 km/h)
Impact Angle. . .	20.2 deg
Exit Speed. . . .	47.0 mi/h (75.6 km/h)
Exit Trajectory .	6.4 deg
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal. .	-5.7 g
Lateral	13.1 g
Occupant Impact Velocity	
Longitudinal. .	18.6 ft/s (5.7 m/s)
Lateral	21.1 ft/s (6.4 m/s)
Occupant Ridedown Accelerations	
Longitudinal. .	-5.5 g
Lateral	8.6 g

FIGURE 17 Summary of results for Test 7069-6.

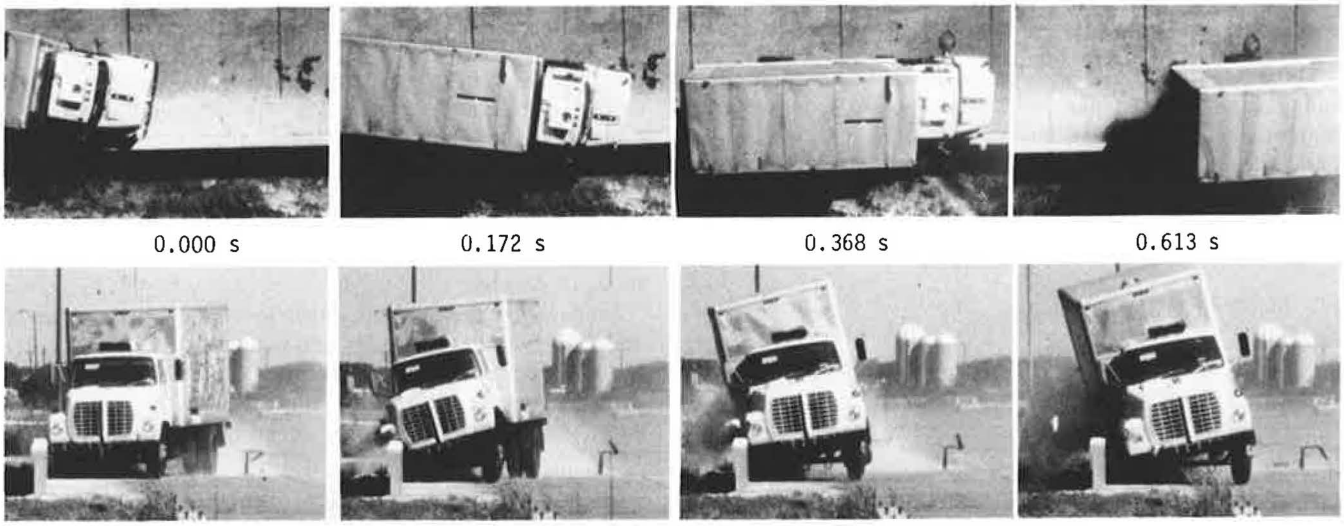
FIGURE 18 Damage to vehicle in Test 7069-16 (18,000 lb, 50 mph, 14 degrees) after being uprighted.

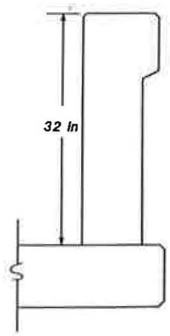
FIGURE 19 Thirty-two-in. vertical parapet after Test 7069-16.

lanes; however, the vehicle did not remain upright after the collision. Performance of the railing was judged acceptable.

Thirty-two-in. F-Shaped Concrete Parapet

The median barrier version of the F-shape was developed by Southwest Research Institute as an alternative to the New Jersey safety shape. The same F-shaped traffic face was used in the bridge parapet railing evaluated in the study reported herein. This railing was designed for an impact by a 5,400-lb pickup truck traveling 65 mph and striking at an angle of 20 degrees. The design load was 56 kips of line load uniformly distributed over a longitudinal distance of 42 in. and applied 29 in. above the road surface. A cross section of the prototype design is shown in Figure 21.





Test No. 7069-16
Date 10/13/88
Test Installation . . 32-in Vertical Wall Bridge Rail
Installation Length. . 100 ft (30.5 m)
Vehicle . . . . 1982 Ford 7000 Single-Unit Truck
Vehicle Weight
Test Inertia . . . . 13,820 lb (6,274 kg)
Gross Static . . . . 18,000 lb (8,172 kg)
Maximum Vehicle Crush. 10.0 in (25.4 cm)

Impact Speed. . . 50.0 mi/h (80.5 km/h)
Impact Angle. . . 14.0 deg
Exit Speed. . . . 34.2 (55.0 km/h)
Exit Trajectory . 5 deg
Vehicle Accelerations
(Max. 0.050-sec Avg)
Longitudinal. . -1.7 g
Lateral . . . . 4.6 g
Occupant Impact Velocity
Longitudinal. . 10.9 ft/s (3.3 m/s)
Lateral . . . . 11.8 ft/s (3.6 m/s)
Occupant Ridedown Accelerations
Longitudinal. . -2.3 g
Lateral . . . . 8.4 g

FIGURE 20 Summary of results for Test 7069-16.

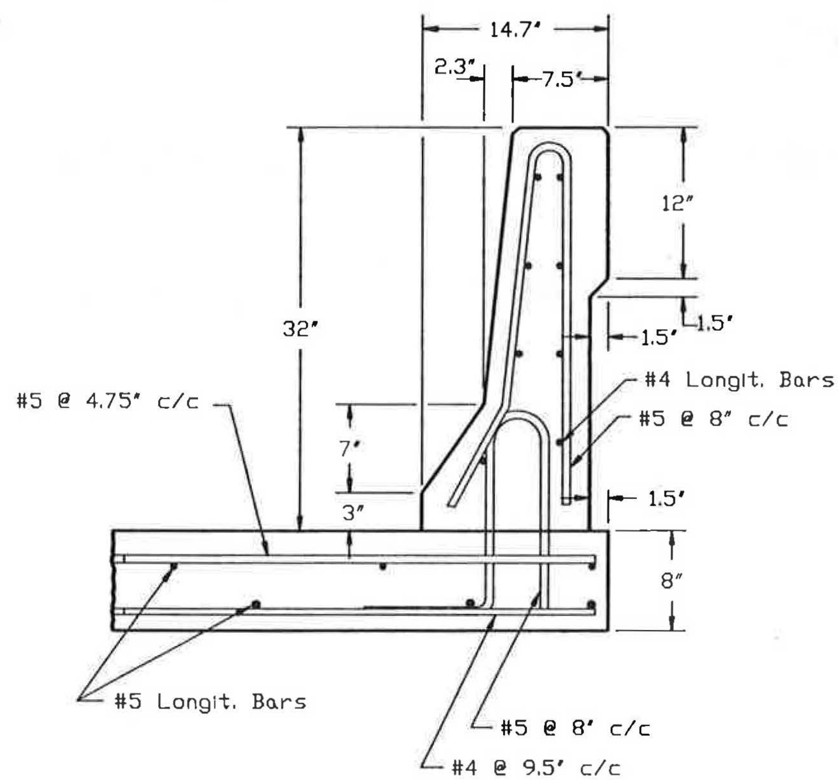


FIGURE 21 Thirty-two-in. F-shaped concrete parapet.

Three full-scale crash tests were performed on a prototype railing: (a) an 1,800-lb automobile striking at 60 mph and 20 degrees, (b) a 5,400-lb pickup truck striking at 65 mph and 20 degrees, and (c) an 18,000-lb single-unit truck striking at 50 mph and 15 degrees. The railing performed acceptably in all three tests.

Test 7069-3 (1,800-lb Automobile, 60.1 mph, 21.4 degrees)

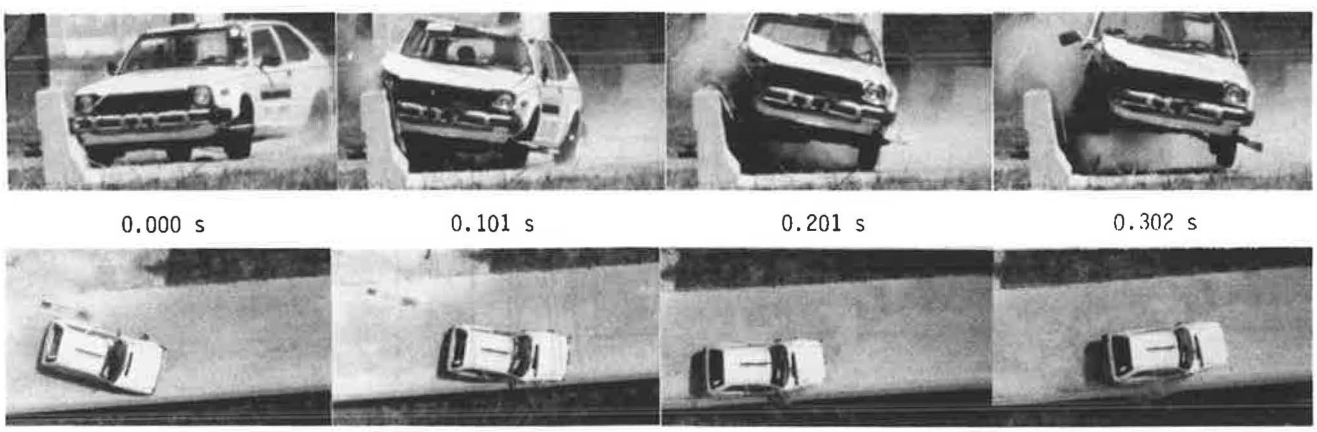
The impact point for this test was at midlength of the railing. The vehicle was smoothly redirected and lost contact with the railing 0.276 sec after impact. The exit yaw angle of the vehicle was 0.9 degree and its trajectory was 6.2 degree relative to the railing. The vehicle was in contact with the railing for 10.3 ft. During redirection, the right side of the vehicle was lifted by the sloping face of the railing. Tire marks on the railing indicate that the right side of the vehicle was lifted about 17 in. The vehicle was banked with a maximum roll angle of about 11 degrees.

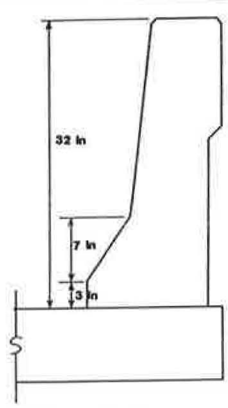
Damage to the vehicle is shown in Figure 22. Damage to the railing was cosmetic only and is shown in Figure 23.

The effective coefficient of friction was calculated to be 0.33. Occupant impact velocity was 19.0 ft/sec in the longitudinal direction and 23.7 ft/sec in the lateral direction. The highest

FIGURE 22 Damage to vehicle in Test 7069-3 (1,800 lb, 60.1 mph, 21.4 degrees).

FIGURE 23 Thirty-two-in. F-shape after Test 7069-3.





32 in F-Shape
Bridge Rail

```
Test No. . . . . . . . 7069-3
Date . . . . . . . . . 7/28/87
Test Installation  . . 32 in F-Shape
                       Bridge Rail
Installation Length  . 100 ft (30 m)
Vehicle  . . . . . . . 1980 Honda
                       1300 DX
Vehicle Weight
  Test Inertia . . . . 1,800 lb (817 kg)
  Gross Static . . . . 1,966 lb (893 kg)
Vehicle Damage Classification
  TAD  . . . . . . . . 01RFQ4
  CDC  . . . . . . . .
Maximum Vehicle Crush  9.0 in (22.9 cm)

Impact Speed. . . 60.1 mi/h (96.7 km/h)
Impact Angle. . . 21.4 deg
Exit Speed. . . . 53.0 mi/h (85.3 km/h)
Exit Trajectory . 6.2 deg
Vehicle Accelerations
  (Max. 0.050-sec Avg)
  Longitudinal. . -8.0 g
  Lateral . . . . 12.8 g
Occupant Impact Velocity
  Longitudinal. . 19.0 ft/s (5.8 m/s)
  Lateral . . . . 23.7 ft/s (7.2 m/s)
Occupant Ridedown Accelerations
  Longitudinal. . -2.1 g
  Lateral . . . .  4.9 g
```

FIGURE 24 Summary of results for Test 7069-3.

0.010-sec occupant ridedown accelerations were $-2.1\ g$ (longitudinal) and $4.9\ g$ (lateral). These data and other pertinent information from the test are summarized in Figure 24.

The railing contained and smoothly redirected the vehicle with no lateral movement of the barrier. There were no detached elements or debris. There was no intrusion into the occupant compartment although some deformation of the compartment occurred. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes. The vehicle remained upright and reasonably stable during the entire collision. Performance of the railing was judged acceptable.

Test 7069-4 (5,440-lb Pickup Truck, 65.4 mph, 20.4 degrees)

The railing contained and smoothly redirected the pickup truck in this test. The vehicle began to ride up the barrier face immediately after initial contact, and the right front tire was deflated during interaction with the railing. Just before becoming parallel with the railing, the vehicle became airborne and rose approximately 1 ft above the pavement surface. On exiting, the vehicle returned to the pavement surface in a stable condition. The exit yaw angle was 0.4 degree and the exit trajectory was 7.4 degrees.

After-test photographs of the vehicle and barrier are shown in Figures 25 and 26, respectively.

The effective coefficient of friction was calculated to be 0.31. Occupant impact velocity was 12.5 ft/sec in the longitudinal direction and 24.1 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were $-1.2\ g$ (longitudinal) and $5.9\ g$ (lateral). These data and other pertinent information from the test are summarized in Figure 27.

The barrier contained and smoothly redirected the vehicle with minimal lateral movement of the barrier. There were no detached elements or debris. There was no intrusion into the occupant compartment although some deformation of the right door occurred. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes. The vehicle remained upright and stable during the entire test period. Performance of the railing was judged acceptable.

FIGURE 25 Damage to vehicle in Test 7069-4 (5,440 lb, 65.4 mph, 20.4 degrees).

FIGURE 26 Thirty-two-in. F-shape after Test 7069-4.

Test 7069-11 (18,000-lb Single-Unit Truck, 52.1 mph, 14.8 degrees)

The impact point was approximately midlength of the railing. On contact the right front wheel began to ride up the face of the railing, and subsequently the left front tire came off the pavement surface. As the vehicle yawed to become parallel to the railing, the left rear wheels came off the pavement surface and the vehicle continued to roll, reaching a maximum roll angle of 31 degrees. The lower edge of the cargo box contacted and slid along the top surface of the railing.

The vehicle sustained extensive damage to the right side, as shown in Figure 28. Maximum crush at the right front corner at bumper height was 20.0 in. The front axle was torn loose, which caused damage to the springs, shackles, U-bolts, and tie rods. The steering arm and cylinder were damaged and the oil pan was dented. The fuel tank broke loose from the truck.

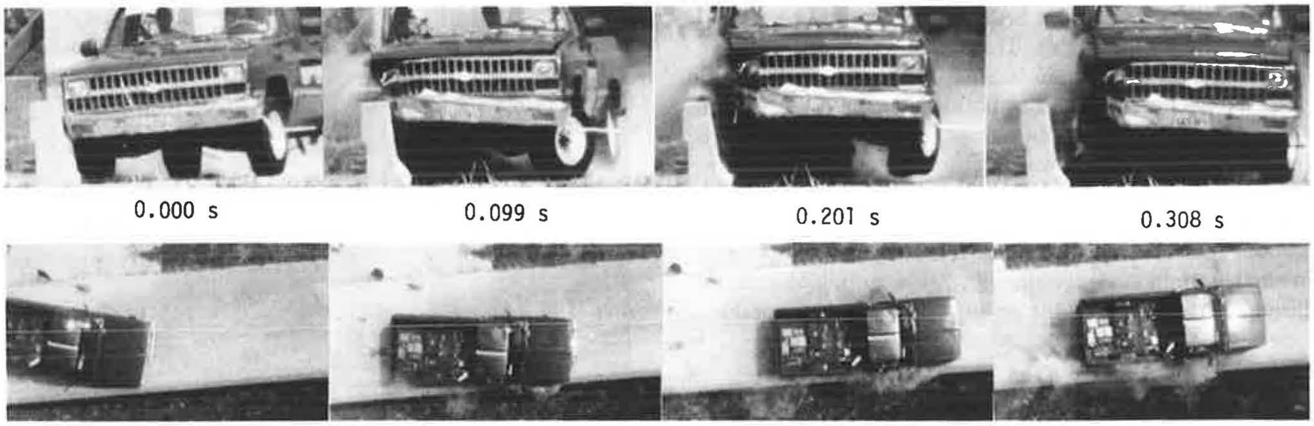
As can be seen in Figure 29, the rail sustained cosmetic damage. There were tire marks on the face of the bridge rail and along the top. The top of the bridge rail was scraped along the remaining length from the lower edge of the cargo box of the truck. The vehicle was in contact with the bridge railing for 39 ft.

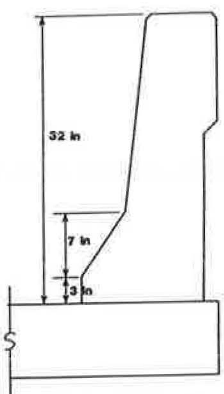
The exit speed was not available. Exit angle was about 0 degree. The effective coefficient of friction was calculated to be 0.12. Occupant impact velocity was 5.7 ft/sec in the longitudinal direction and 8.2 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were $1.3\ g$ (longitudinal) and $5.4\ g$ (lateral). These data and other pertinent information from the test are summarized in Figure 30.

The barrier contained and smoothly redirected the vehicle with no lateral movement of the barrier. There were no detached elements or debris. There was no intrusion into the occupant compartment. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes. The vehicle remained upright and stable during the entire test period. Performance of the railing was judged acceptable.

New Jersey Safety Shape Concrete Parapet

The New Jersey safety shape median barrier has been in use for many years and is currently used for some applications by





32 in F-Shape Bridge Rail

Test No.	7069-4
Date	7/30/87
Test Installation . .	32 in F-Shape Bridge Rail
Installation Length .	100 ft (30 m)
Vehicle	1981 Chevrolet Pickup
Vehicle Weight	
Test Inertia	5,440 lb (2,470 kg)
Gross Static	5,780 lb (2,624 kg)
Vehicle Damage Classification	
TAD	01RD4
CDC	
Maximum Vehicle Crush	5.0 in (12.7 cm)
Impact Speed. . .	65.4 mi/h (105.2 km/h)
Impact Angle. . .	20.4 deg
Exit Speed. . . .	56.9 mi/h (91.6 km/h)
Exit Trajectory .	7.4 deg
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal. .	-4.7 g
Lateral	13.1 g
Occupant Impact Velocity	
Longitudinal. .	12.5 ft/s (3.8 m/s)
Lateral	24.1 ft/s (7.3 m/s)
Occupant Ridedown Accelerations	
Longitudinal. .	-1.2 g
Lateral	5.9 g

FIGURE 27 Summary of results for Test 7069-4.

FIGURE 28 Vehicle after Test 7069-11 (18,000 lb, 52.1 mph, 14.8 degrees).

virtually every state. Many states use a concrete parapet type bridge with the safety shape on the traffic face. Such a railing with a 6-in. top width was tested and evaluated. A cross section of this railing with steel reinforcement is shown in Figure 31. A strength analysis of the final prototype design indicated that its ultimate strength by yield line theory was about 52 kips.

Two full-scale crash tests were performed on the prototype installation: (a) a 5,390-lb vehicle striking at 57.7 mph and 20.6 degrees and (b) an 18,000-lb vehicle striking at 51.6 mph and 15.5 degrees. The railing performed satisfactorily in both tests.

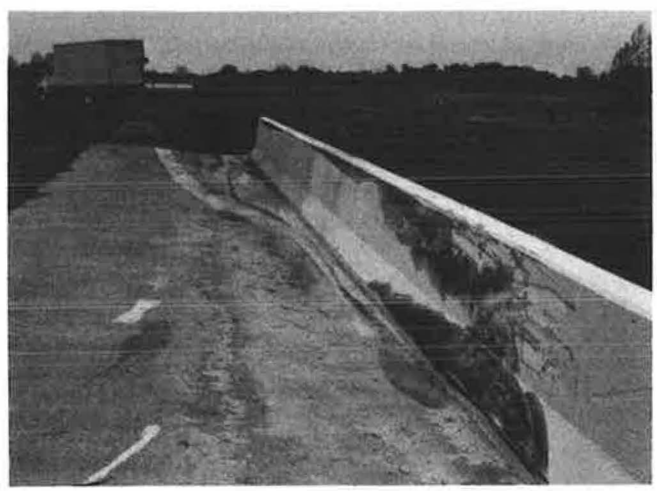
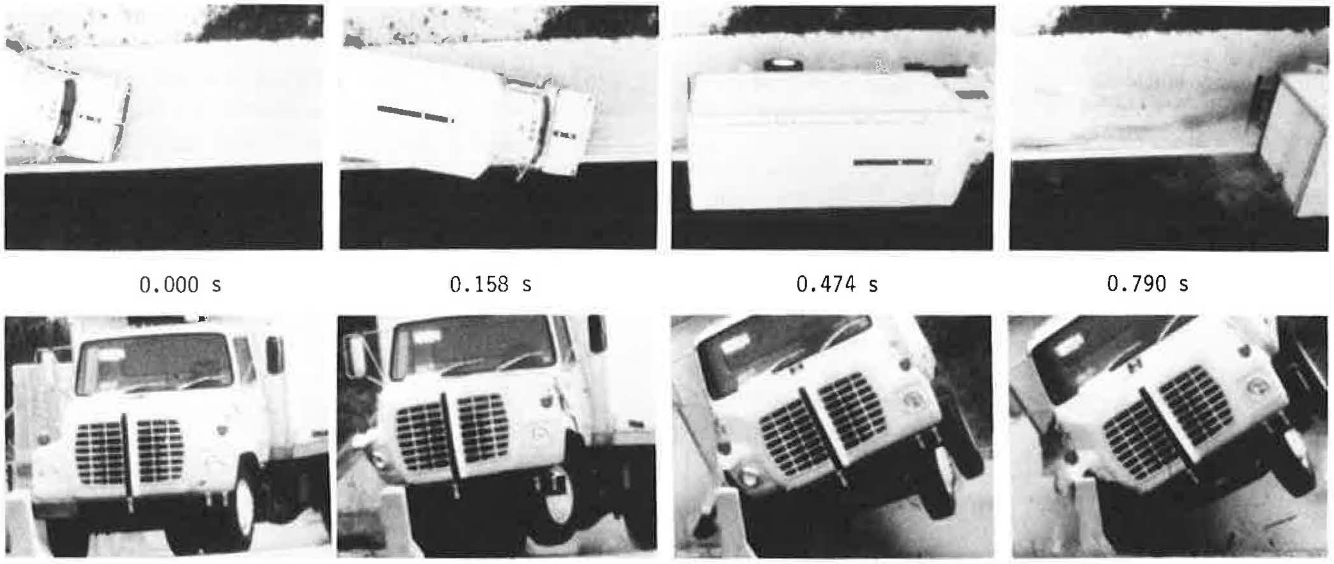
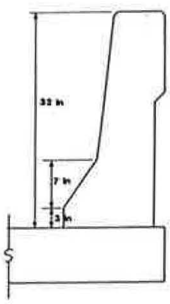


FIGURE 29 Thirty-two-in. F-shape after Test 7069-11.

Test 7069-14 (5,390-lb Vehicle, 57.7 mph, 20.6 degrees)

The vehicle began to ride up the face of the railing shortly after contact. Just after becoming parallel with the railing, the vehicle became airborne and reached a maximum height of approximately 23 in. above the deck. While still airborne





Test No. 7069-11
Date 3/30/88
Test Installation . . 32 in F-Shape Bridge Rail
Installation Length. . 100 ft (30 m)
Vehicle . . . . 1982 Ford 7000 Single-Unit Truck
Vehicle Weight
Test Inertia . . . . 18,000 lb (8,172 kg)
Gross Static . . . . 18,000 lb (8,172 kg)
Maximum Vehicle Crush. 20.0 in (50.8 cm)

Impact Speed. . . 52.1 mi/h (83.8 km/h)
Impact Angle. . . 14.8 deg
Exit Speed. . . . Not Available
Exit Trajectory . 0 deg
Vehicle Accelerations
(Max. 0.050-sec Avg)
Longitudinal. . -1.4 g
Lateral . . . . 3.9 g
Occupant Impact Velocity
Longitudinal. . 5.7 ft/s (1.7 m/s)
Lateral . . . . 8.2 ft/s (2.5 m/s)
Occupant Ridedown Accelerations
Longitudinal. . 1.3 g
Lateral . . . . 5.4 g

FIGURE 30 Summary of results for Test 7069-11.

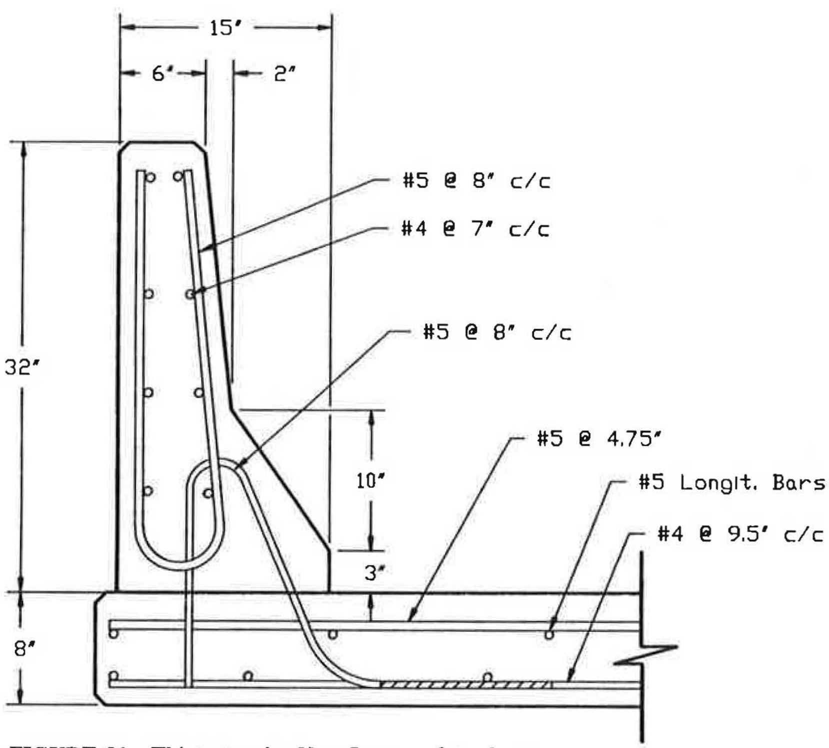


FIGURE 31 Thirty-two-in. New Jersey safety shape.

and traveling at 35.8 mph with a heading of 0.9 degree and a trajectory of 0.9 degrees, the vehicle lost contact with the railing. The vehicle rose approximately 23 in. above the pavement. Approximately 15 ft of railing was in contact with the vehicle (Figure 32).

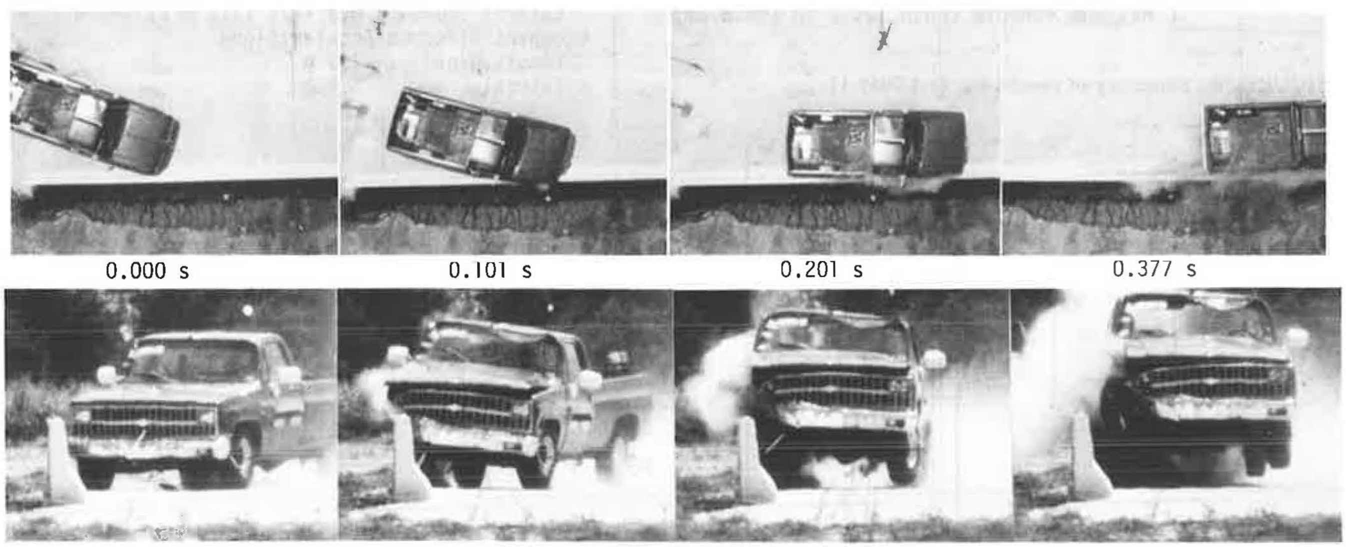
Damage to the vehicle is shown in Figure 33. Maximum crush at the right front corner at bumper height was 12 in.

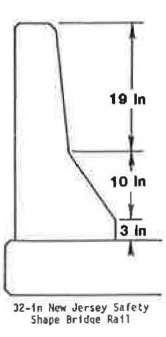
The effective coefficient of friction was calculated to be 0.83. Occupant impact velocity was 17.8 ft/sec in the longitudinal direction and 18.7 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were −5.1 *g* (longitudinal) and 9.2 *g* (lateral). These data and other pertinent information from the test are summarized in Figure 34.

The bridge rail contained and smoothly redirected the vehicle with no lateral movement of the bridge rail. There was no intrusion into the occupant compartment and minimal deformation of the compartment. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes. The vehicle remained upright and stable during the collision. Performance of the railing was judged acceptable.

FIGURE 32 Thirty-two-in. safety shape railing after Test 7069-14.

FIGURE 33 Damage to vehicle in Test 7069-14 (5,390 lb, 57.7 mph, 20.6 degrees).





Test No. 7069-14
Date 8/11/88
Test Installation . . 32-in New Jersey Safety Shape Bridge Rail
Installation Length. . 100 ft (30 m)
Vehicle . . . . 1981 Chevrolet Custom Deluxe C-20 truck
Vehicle Weight
Test Inertia . . . . 5,390 lb (2,447 kg)
Gross Static . . . . 5,724 lb (2,599 kg)
Maximum Vehicle Crush. 12.0 in (30.7 cm)

Impact Speed. . . 57.7 mi/h (92.8 km/h)
Impact Angle. . . 20.6 deg
Exit Speed. . . . 35.8 mi/h (57.6 km/h)
Exit Trajectory . 0.9 deg
Vehicle Accelerations
(Max. 0.050-sec Avg)
Longitudinal. . -6.6 g
Lateral . . . . 7.3 g
Occupant Impact Velocity
Longitudinal. . 17.8 ft/s (5.4 m/s)
Lateral . . . . 18.7 ft/s (5.7 m/s)
Occupant Ridedown Accelerations
Longitudinal. . -5.1 g
Lateral . . . . 9.2 g

FIGURE 34 Summary of results for Test 7069-14.

Test 7069-12 (18,000-lb Single-Unit Truck, 51.6 mph, 15.5 degrees)

Shortly after contact the right front wheel began to ride up the face of the railing, and the axle broke loose from the vehicle. The left front wheel became airborne, and the front of the vehicle continued to ride up as the vehicle began to yaw to become parallel with the railing. The front of the vehicle reached a maximum height of about 44 in. above the pavement surface. The vehicle continued to roll toward the railing, reaching a maximum angle of 44 degrees. When the vehicle slid off the end of the railing, it rolled back away from the railing and came to rest on its left side.

As can be seen in Figure 35, the rail sustained cosmetic damage. There were tire marks on the face of the bridge rail and along the top. The top of the bridge rail was scraped along the remaining length from the undercarriage of the truck. The vehicle was in contact with the bridge rail for 77 ft.

The vehicle sustained damage, as shown in Figure 36. Maximum crush at the right front corner at bumper height was 8.0 in. The front axle was torn off the vehicle and the undercarriage was damaged. There was damage to the U-bolts, Pittman arm rod, steering arm, brake lines, and leaf spring bolts. The outer right rear wheel rim was bent and the tire was damaged. The fuel tank was also damaged.

The exit speed and the effective coefficient of friction were not attainable. The vehicle did not become parallel while in contact with the bridge rail. Occupant impact velocity was 13.4 ft/sec in the longitudinal direction and 10.2 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were $-3.0\ g$ (longitudinal) and $4.9\ g$ (lateral). These data and other pertinent information from the test are summarized in Figure 37.

The bridge rail contained and smoothly redirected the test vehicle with no lateral movement of the bridge rail. There was no intrusion into the occupant compartment and very little deformation of the compartment. The vehicle trajectory at loss of contact indicated minimum intrusion into adjacent traffic lanes; however, the vehicle did not remain upright after collision. Performance of the railing was judged acceptable.

FIGURE 35 Thirty-two-in. New Jersey safety shape railing after Test 7069-12.

FIGURE 36 Damage to vehicle in Test 7069-12 (18,000 lb, 51.6 mph, 15.5 degrees).

SUMMARY

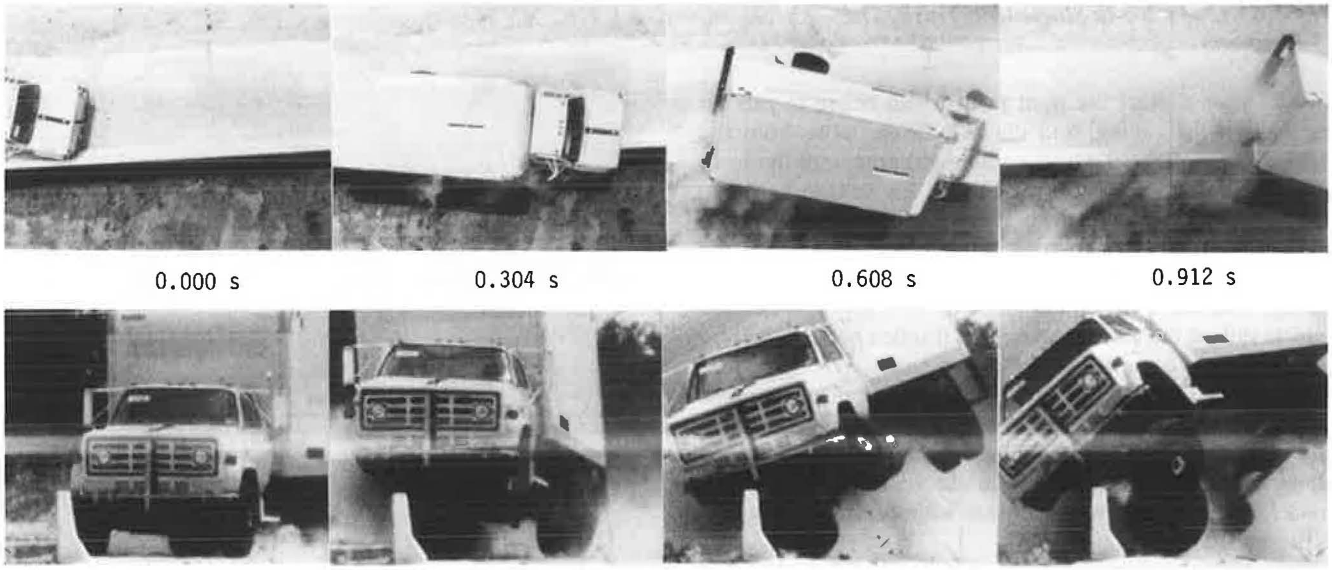
It is generally thought that the AASHTO *Guide Specifications for Bridge Railings*, if followed, will produce a general improvement of the performance of bridge railing systems. The performance level selection criteria given in that guide appear to indicate that a PL 2 bridge railing design should be used on much of the nation's highway system.

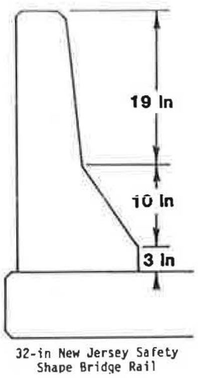
Four PL 2 railing designs have been developed and proven through full-scale crash tests. All the railings had a total geometric height of 32 in. Test results indicate that this is probably the minimum height for a PL 2 railing, at least for the types of railings tested. Some innovative designs of lesser height might be made to function suitably, but they should be subjected to full-scale testing to prove their performance.

Of the railing designs reported herein, one was a steel beam-and-post system with tubular rail elements mounted on wide flange posts mounted on a curb. The other three were concrete parapets: a vertical face, an F-shape, and the standard New Jersey safety shape. All had suitable height and geometric features as indicated by full-scale tests evaluated in accordance with the 1989 guide specification. The strengths of the railing systems were adequate and possibly on the conservative side. Extensive structural distress of the railings was not experienced in the tests. Virtually no cracking occurred in the concrete railings during full-scale tests, which indicates that the forces applied to the railings were significantly less than their ultimate strengths.

Some differences in performance of the three concrete parapet railings should be observed. The two parapets with sloped faces, the New Jersey safety shape and the F-shape, both caused the automobile and pickup test vehicles to ride up the face and become airborne. The vertical parapet did not produce this effect. However, the forces generated on these vehicles by the vertical parapet were generally slightly more severe. In all cases, stability of the vehicle was considered acceptable.

In tests with 18,000-lb single-unit trucks on the 32-in. vertical parapet and the 32-in. F-shape, the vehicles remained generally stable during interaction with the railing, although roll displacements were significant. The vehicle did finally roll onto its left side in the test on the 32-in. vertical parapet





Test No.	7069-12
Date	6/22/88
Test Installation . .	32-in N.J. Safety Shape Bridge Rail
Installation Length. .	100 ft (30 m)
Vehicle	1982 GMC Single-Unit Truck
Vehicle Weight	
Test Inertia	10,900 lb (4,949 kg)
Gross Static	18,000 lb (8,172 kg)
Maximum Vehicle Crush.	8.0 in (20.3 cm)

Impact Speed. . .	51.6 mi/h (83.0 km/h)
Impact Angle. . .	15.5 deg
Exit Speed. . . .	N/A
Exit Trajectory .	2.0 deg
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal. .	-3.2 g
Lateral	2.5 g
Occupant Impact Velocity	
Longitudinal. .	13.4 ft/s (4.1 m/s)
Lateral	10.2 ft/s (3.1 m/s)
Occupant Ridedown Accelerations	
Longitudinal. .	-3.0 g
Lateral	4.9 g

FIGURE 37 Summary of results for Test 7069-12.

because of its curved path. In the test on the 32-in. New Jersey safety shape, the vehicle rode up the barrier more and rolled onto its side. This difference in behavior is thought to be the result of the geometry of the face of the railing. However, the make of the vehicle used on the New Jersey safety shape was different from the others, and differences in the vehicle may have had some influence. All vehicles met the test vehicle specifications in the AASHTO *Guide Specifications for Bridge Railings*.

REFERENCES

1. *Guide Specifications for Bridge Railings*. AASHTO, Washington, D.C., 1989.
2. E. Buth. *Safer Bridge Railings*, Vol. 1—*Summary Report*. Report FHWA/RD-82/072. FHWA, U.S. Department of Transportation, Washington, D.C., June 1984.
3. W. L. Beason and T. J. Hirsch. *Measurement of Heavy Vehicle Impact Forces and Inertia Properties*. Texas Transportation Institute, The Texas A&M University System, College Station, May 1989.

Aesthetically Pleasing Concrete Beam-and-Post Bridge Rail

T. J. HIRSCH, C. E. BUTH, AND DARRELL KADERKA

Research has developed railing to withstand impact loads from vehicles of ever-increasing size; however, aesthetic considerations have been overshadowed by safety and structural requirements. The objective of this research study was to develop aesthetically pleasing, structurally sound railings that can serve as alternative railings in urban areas. A new type of open concrete bridge rail—Texas Type T411—is presented. This bridge rail is constructed of reinforced concrete 32 in. high by 12 in. thick and contains 8-in.-wide by 18-in.-high openings at 18-in. center-to-center longitudinal spacing. The bridge rail was crash-tested and evaluated in accordance with *NCHRP Report 230* for Service Level 2. Two crash tests were required—a 4,500-lb passenger car striking at 60 mph and a 25-degree impact angle and an 1,800-lb passenger car striking at 60 mph and a 20-degree impact angle. In both tests the bridge rail contained and redirected the test vehicle. There were no detached elements or debris to present undue hazard to other traffic. The vehicle remained upright and relatively stable during the collision. The occupant impact velocities and 10-msec occupant ridedown accelerations were within the limits specified in *NCHRP Report 230*. The vehicle trajectory at loss of contact indicates no intrusion into adjacent traffic lanes (exit angles of 0 degree and 5.9 degrees). These test data also met the occupancy safety evaluation guidelines in the 1989 AASHTO *Guide Specifications for Bridge Railings*.

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

An attempt is being made to develop one or more new concrete, steel, and aluminum railings or combination railings, some with curb and sidewalk.

A new type of open concrete bridge rail—Texas Type T411—is presented. The research study advisory committee reviewed design sketches of 22 different bridge rail designs before selecting the new Texas Type T411 as its top priority. The advisory committee was composed of two architects (private consultants from Dallas), two research engineers from Texas Transportation Institute, two highway design engineers from the Dallas District, one bridge design engineer from the Dallas District, and three bridge design engineers from Austin headquarters.

Texas Transportation Institute, Texas A&M University System, College Station, Tex. 77843-3135.

DESCRIPTION OF TEXAS TYPE T411 BRIDGE RAIL

Texas Type T411 bridge rail is constructed of reinforced concrete 32 in. high by 12 in. thick and contains 8-in.-wide by 18-in.-high openings at 18-in. center-to-center longitudinal spacing. Figures 1 and 2 present a plan view, elevation, and cross section of the T411 rail. The bridge deck is an 8-in.-thick typical Texas bridge slab design in accordance with AASHTO specifications (*1*).

Figure 3 shows a photograph of the bridge rail installation before crash testing. The installation is 75 ft 10 in. long. The three pilasters are not super-strong posts, as they appear to be. They contain styrofoam blocks 10.5 in. by 13 in. by 21 in. (void), which means that the pilasters are similar to the 8-in. by 18-in. openings. The use of the pilasters is optional because they did not contribute to the bridge rail strength as built and crash-tested.

This bridge rail was designed using a failure mechanism (or yield line) method of analysis (*2*). The design strengh of the concrete was f_c = 3,600 psi and the yield strength of reinforcing steel was f_y = 60,000 psi. The top beam was nominally 7 in. wide and 11 in. thick (b = 7 in. and d = 8.25 in.), yielding an ultimate moment capacity of 20.0 kip-ft. The posts are 10 in. wide and 10 in. thick (b = 10 in. and d = 8 in.), yielding an ultimate moment capacity of 20.6 kip-ft. With a moment arm of 2.2 ft, each post could resist a lateral load of about 9.5 kips. Figures 4 and 5 present a summary of the failure mechanism analysis of the strength of the T411 bridge rail. The failure load would be about 65.9 kips or more. Five posts would crack, and a 9-ft. length of bridge rail would be involved.

Concrete specimens taken from the simulated bridge deck yielded a compressive strength of 4,880 psi at 28 days of age. The compressive strength of the concrete rail was 5,110 psi at 28 days of age.

CRASH TESTS

In order to qualify this bridge rail for use on Federal-Aid highways, it was crash-tested and evaluated in accordance with *NCHRP Report 230* (*3*) for Service Level 2. Two crash tests were required—Test 10 with a 4,500-lb passenger car striking at 60 mph and a 25-degree impact angle and Test S13 with an 1,800-lb passenger car striking at 60 mph and a 20-degree impact angle.

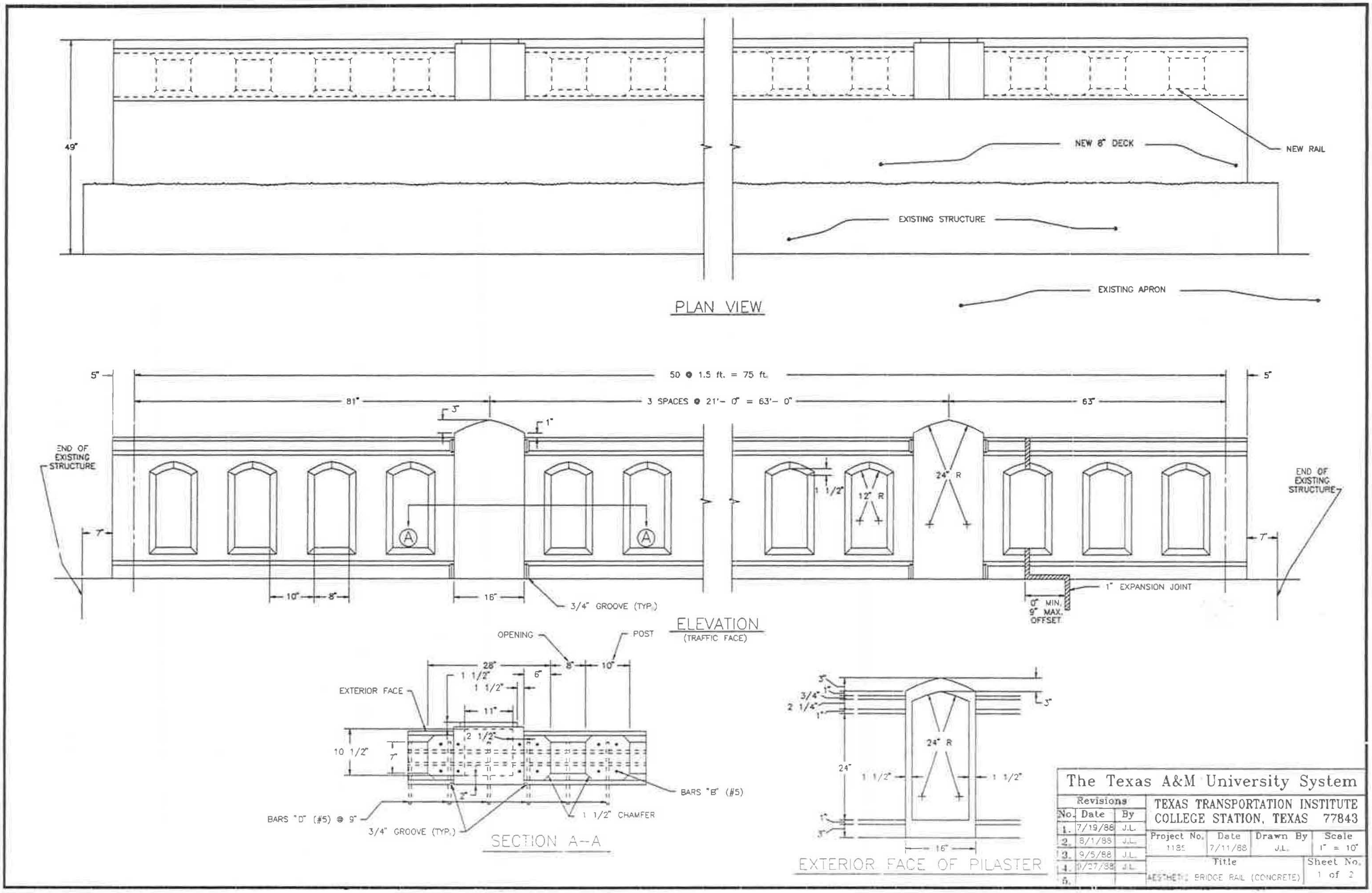
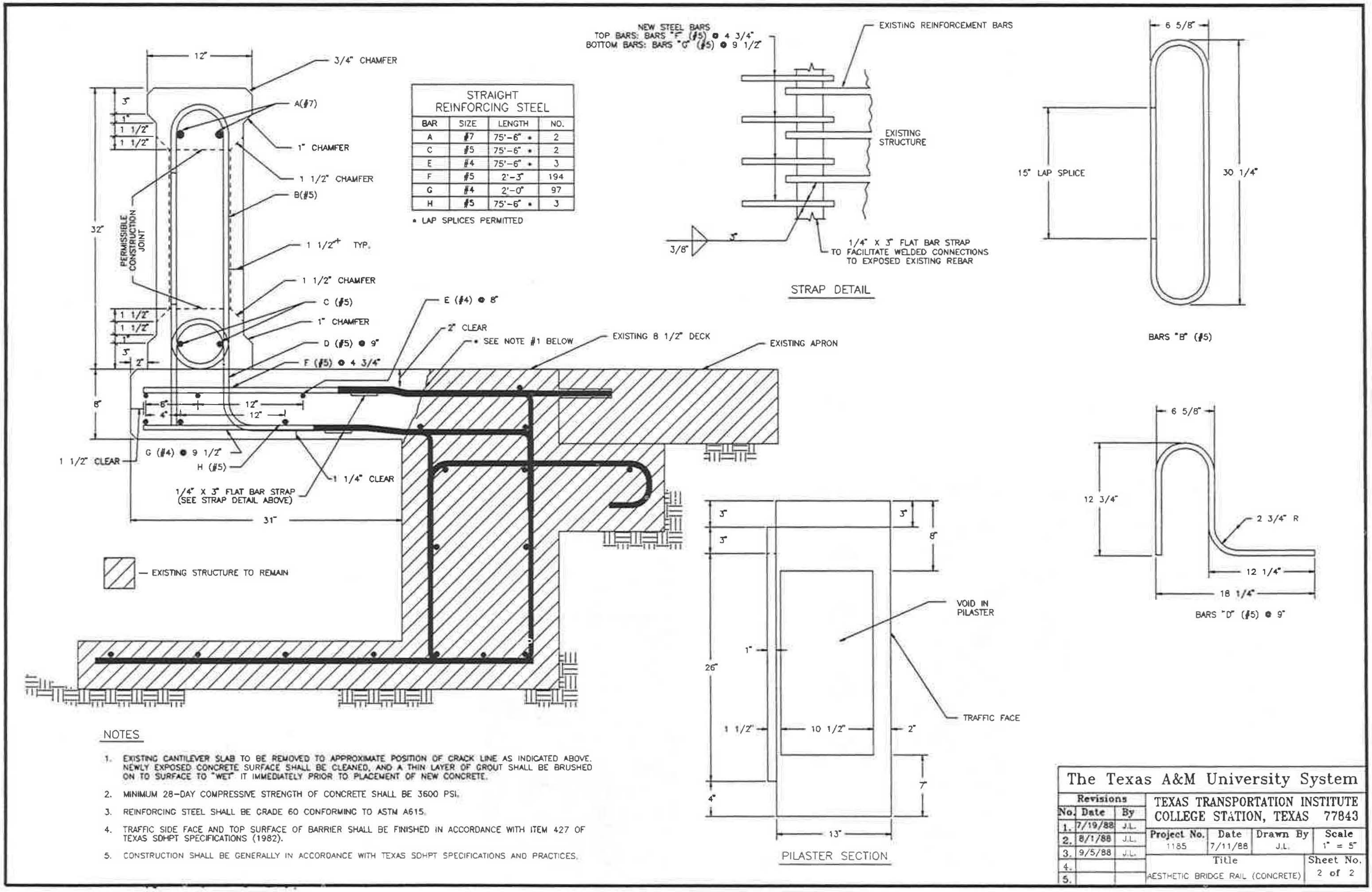


FIGURE 1 Texas Type T411 bridge rail—plan and elevation.



STRAIGHT REINFORCING STEEL

BAR	SIZE	LENGTH	NO.
A	#7	75'-6" *	2
C	#5	75'-6" *	2
E	#4	75'-6" *	3
F	#5	2'-3"	194
G	#4	2'-0"	97
H	#5	75'-6" *	3

* LAP SPLICES PERMITTED

NOTES

1. EXISTING CANTILEVER SLAB TO BE REMOVED TO APPROXIMATE POSITION OF CRACK LINE AS INDICATED ABOVE. NEWLY EXPOSED CONCRETE SURFACE SHALL BE CLEANED, AND A THIN LAYER OF GROUT SHALL BE BRUSHED ON TO SURFACE TO "WET" IT IMMEDIATELY PRIOR TO PLACEMENT OF NEW CONCRETE.
2. MINIMUM 28-DAY COMPRESSIVE STRENGTH OF CONCRETE SHALL BE 3600 PSI.
3. REINFORCING STEEL SHALL BE GRADE 60 CONFORMING TO ASTM A615.
4. TRAFFIC SIDE FACE AND TOP SURFACE OF BARRIER SHALL BE FINISHED IN ACCORDANCE WITH ITEM 427 OF TEXAS SDHPT SPECIFICATIONS (1982).
5. CONSTRUCTION SHALL BE GENERALLY IN ACCORDANCE WITH TEXAS SDHPT SPECIFICATIONS AND PRACTICES.

The Texas A&M University System

TEXAS TRANSPORTATION INSTITUTE
COLLEGE STATION, TEXAS 77843

Revisions No.	Date	By
1.	7/19/88	J.L.
2.	8/1/88	J.L.
3.	9/5/88	J.L.
4.		
5.		

Project No.	Date	Drawn By	Scale
1185	7/11/88	J.L.	1" = 5"

Title: AESTHETIC BRIDGE RAIL (CONCRETE)

Sheet No. 2 of 2

FIGURE 2 Texas Type T411 bridge rail—cross section.

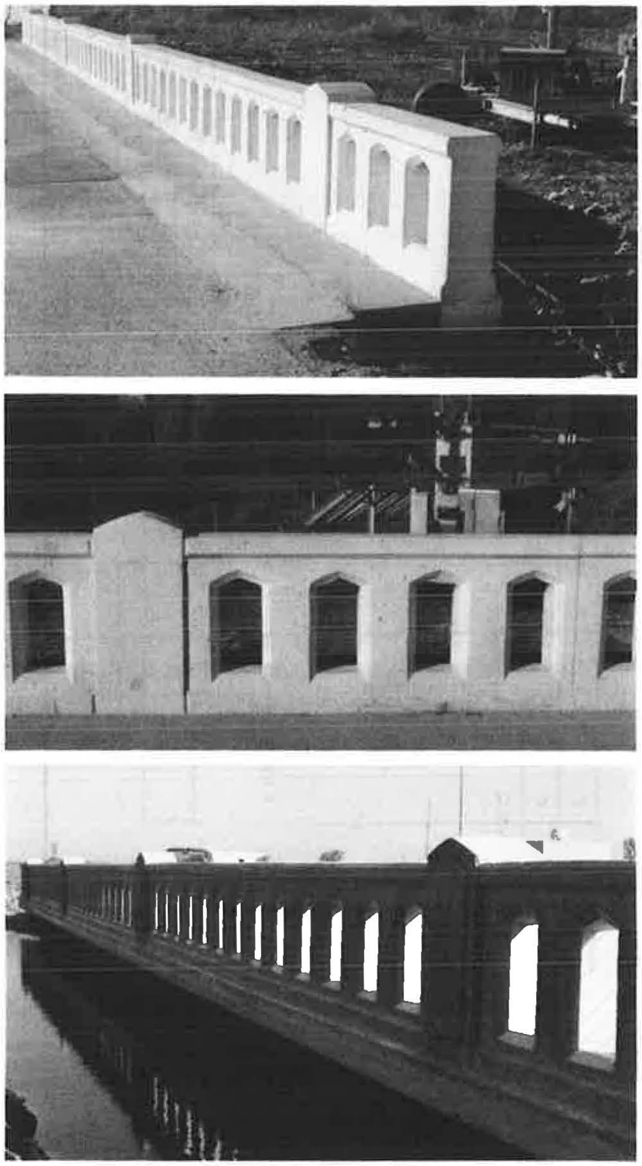


FIGURE 3 Installation before Test 1185-1.

Honda Crash Test (Test 1185-1)

The 1980 Honda Civic (Figure 6) was directed into the bridge rail using a reverse tow and guidance system. Test inertia mass of the vehicle was 1,800 lb. The lower edge of the vehicle bumper was 14.25 in. high, and the top of the bumper was 19.25 in. high. The vehicle was freewheeling and unrestrained just before impact.

The speed of the vehicle at impact was 60.2 mph and the angle of impact was 21.2 degrees. The vehicle struck the bridge rail approximately 22 ft from the end. The right front wheel made contact with the bridge rail shortly after impact. The vehicle began to redirect at 0.039 sec. By 0.052 sec the vehicle had deformed to the A-pillar, which allowed the windshield to begin to pop out, and at 0.075 sec the windshield broke. At 0.378 sec the vehicle was traveling almost parallel with the bridge rail and its speed was about 39.3 mph. The front of the vehicle remained in contact with the bridge rail until it rode off the end at 0.974 sec at a speed of 30.2 mph. When the brakes were applied, the vehicle yawed clockwise and subsequently came to rest 100 ft from the point of impact.

As can be seen in Figures 7 and 8, the rail sustained minimal cosmetic damage. Tire marks on the face of the bridge rail extended from the point of impact to the end of the rail. Some scraping and gouging along the edges of the portholes and of the first pilaster beyond impact occurred. The vehicle was in contact with the bridge rail for 53 ft. The vehicle sustained severe damage to the right side as shown in Figure 9. Maximum crush at the right front corner at bumper height was 11.0 in. The drive axle universal joint and right strut were damaged. The instrument panel in the passenger compartment was bent as well as the floor pan and roof, and the windshield was broken. The right front rim was bent and the tire was damaged. There was damage to the hood, grill, bumper, right front quarter panel, the right door and glass, the right rear quarter panel, and the rear bumper.

Test Results

Impact speed was 60.2 mph and the angle of impact was 21.2 degrees. Occupant impact velocity was 28.6 ft/sec in the longitudinal direction and 16.6 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were -2.0 *g* (longitudinal) and 3.6 *g* (lateral). These data and other pertinent information from the test are summarized in Figure 10.

These data were further analyzed to obtain 0.050-sec average accelerations versus time. The maximum 0.050-sec averages measured at the center of gravity were -13.5 *g* (longitudinal) and 11.3 *g* (lateral).

Conclusions

The bridge rail contained and smoothly redirected the test vehicle with no lateral movement of the bridge rail. There were no detached elements or debris to present undue hazard to other traffic. The vehicle remained upright and relatively stable during the collision. The occupant impact velocities and 10-msec occupant ridedown accelerations were within the limits specified in *NCHRP Report 230*. The vehicle trajectory at loss of contact indicates no intrusion into adjacent traffic lanes (exit angle 0 degree).

These test data were also evaluated using the occupant safety evaluation guidelines in the 1989 AASHTO *Guide Specifications for Bridge Railings* (*4*). The effective coefficient of friction *u* was found to be 0.54, or marginal for this test.

Cadillac Crash Test (Test 1185-2)

The 1980 Cadillac Sedan DeVille (Figure 11) was directed into the bridge rail using a reverse tow and guidance system. Test inertia mass of the vehicle was 4,500 lb. The lower edge of the vehicle bumper was 12.5 in. high, and the top of the bumper was 21.0 in. high. The vehicle was freewheeling and unrestrained just before impact.

The speed of the vehicle at impact was 62.2 mph and the angle of impact was 26.0 degrees. The vehicle struck the bridge

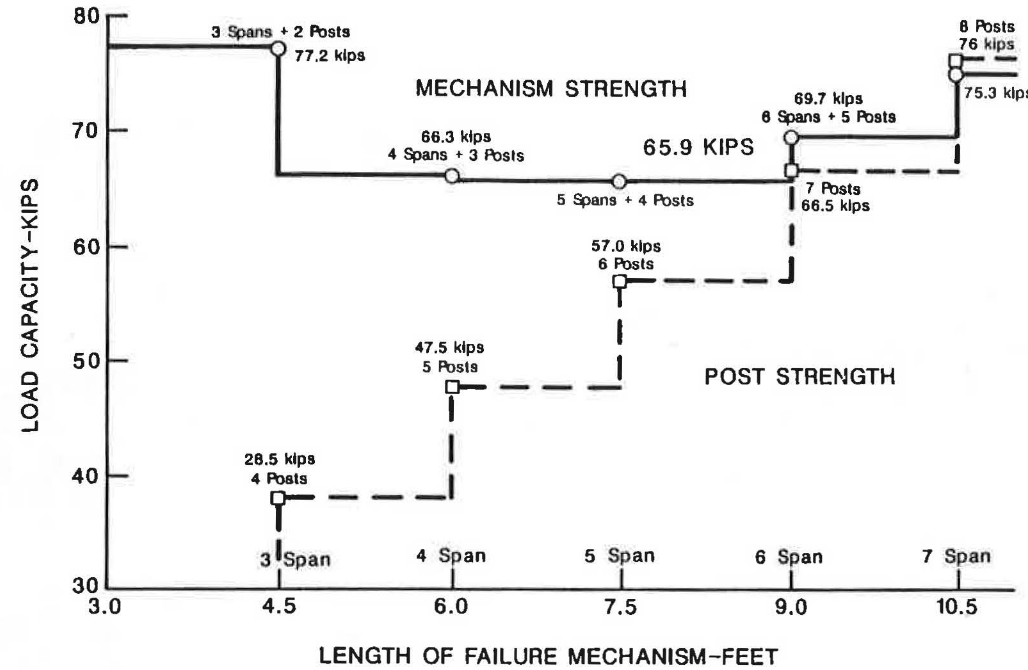
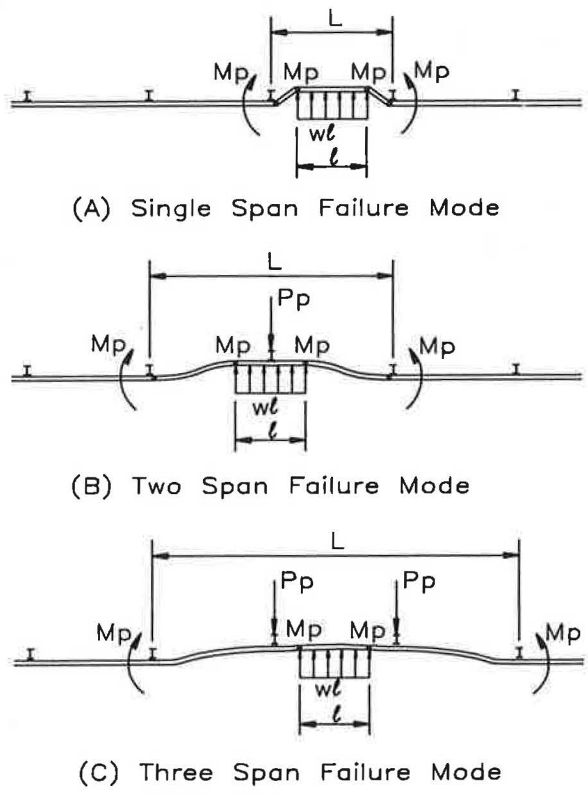


FIGURE 4 Failure mechanism analysis for Texas Type T411 bridge rail.



Mp = plastic moment capacity of rail = Mult.

Pp = ultimate load capacity of a single post

$w\ell$ = total ultimate vehicle impact load $= \dfrac{8\ Mp}{L - \ell/2} + \Sigma Pp$

ℓ = 3.5 ft.

PLAN VIEW

FIGURE 5 Possible failure modes for rails.

FIGURE 6 Vehicle–bridge rail geometrics for Test 1185-1.

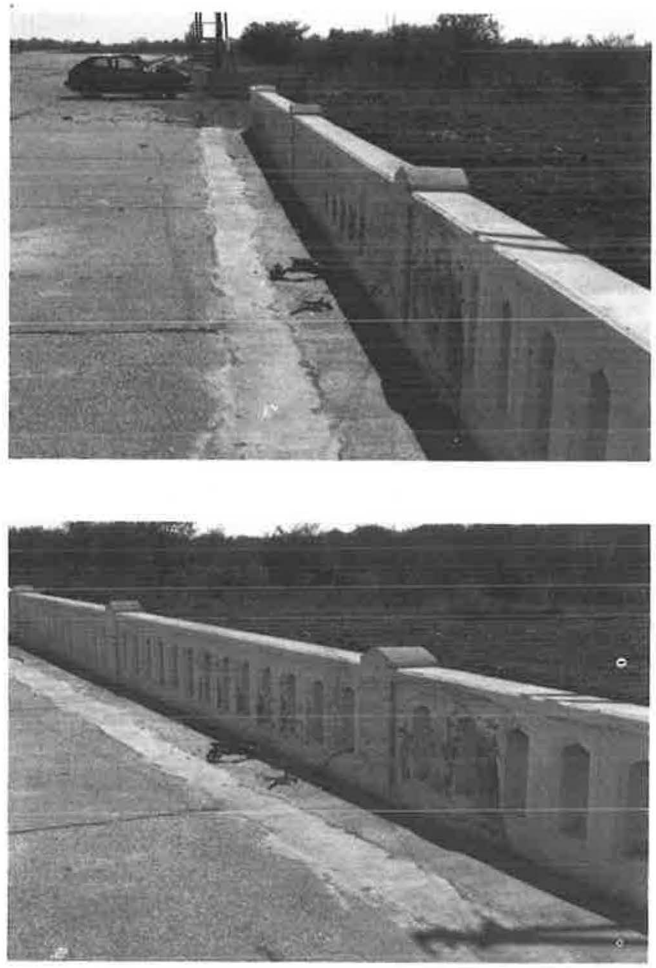


FIGURE 7 Test installation after Test 1185-1.

rail approximately 38 ft from the end. The right front wheel made contact with the bridge rail shortly after impact. The vehicle began to redirect at 0.064 sec. By 0.085 sec the vehicle had deformed to the A-pillar and the windshield broke. At 0.240 sec the vehicle began to move parallel with the bridge rail, traveling at a speed of 41.7 mph. The rear of the vehicle struck the bridge rail at 0.264 sec. The vehicle lost contact with the bridge rail at 0.379 sec, traveling at 38.9 mph and 5.9 degrees. The brakes were then applied; the vehicle yawed clockwise and subsequently came to rest against a safety barrier 125 ft from the point of impact.

As can be seen in Figure 12, the rail sustained minimal cosmetic damage. Tire marks on the face of the bridge rail extended from the point of impact to the end of the rail. Some scraping and gouging along the edges of the portholes and of the first pilaster beyond impact occurred. The vehicle was in contact with the bridge rail for 12 ft.

The vehicle sustained moderate damage to the right side, as shown in Figure 13. Maximum crush at the right front corner at bumper height was 16.0 in. The right A-arm, the tie rod, and the upper and lower ball joints were damaged, and the subframe was bent. The instrument panel in the passenger compartment was bent as well as the floor pan and roof, and the windshield was broken. The right front and rear rims were bent and the tires were damaged. There was damage to the hood, grill, bumper, right front quarter panel, the right front and rear doors, the right rear quarter panel, and the rear bumper.

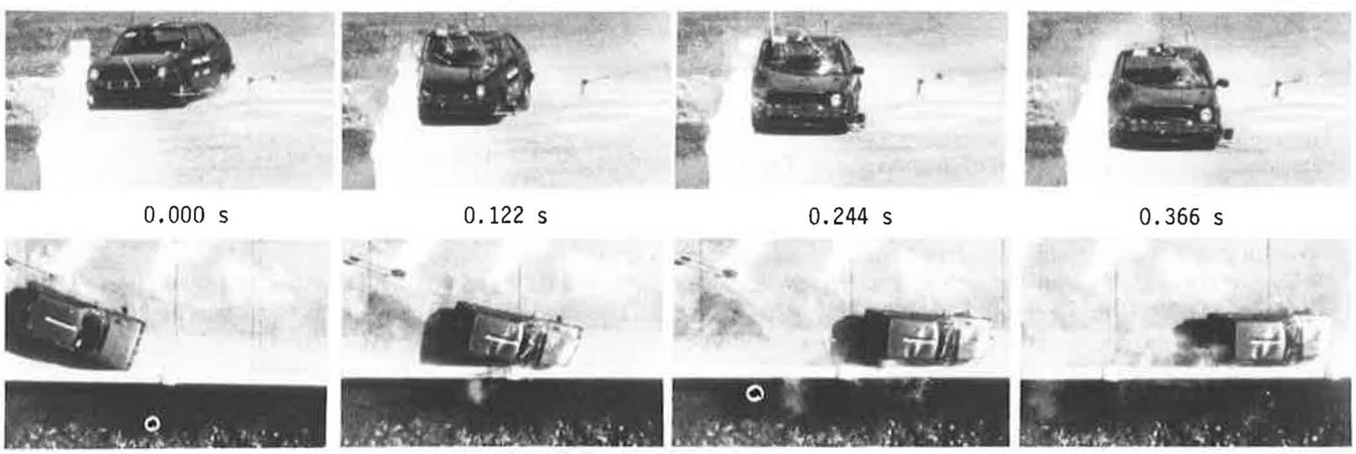
Test Results

Impact speed was 62.2 mph and the angle of impact was 26.0 degrees. The vehicle exited the rail at 38.9 mph and 5.9 degrees. *NCHRP Report 230* describes occupant risk evaluation criteria and places limits on these for acceptable performance for tests conducted at 15-degree impact angles. These limits do not apply to tests conducted at 25-degree impact angles but were computed and reported for information only. Occupant impact velocity was 28.7 ft/sec in the longitudinal direction and 23.0 ft/sec in the lateral direction. The highest 0.010-sec occupant ridedown accelerations were $-12.4\ g$ (longitudinal) and $10.5\ g$ (lateral). These data and other pertinent information from the test are summarized in Figure 14.

These data were further analyzed to obtain 0.050-sec average accelerations versus time. The maximum 0.050-sec averages at the center of gravity were $-12.8\ g$ (longitudinal) and $16.5\ g$ (lateral).

FIGURE 8 Damage to rail at point of impact.

FIGURE 9 Vehicle after Test 1185-1.



Test No. 1185-1
Date 11/29/88

Test Installation . . . T411 Bridge Rail
Installation length . . 75 ft

Vehicle . . . . 1980 Honda Civic
Vehicle Weight
  Test Inertia . . . . . 1,780 lb
Vehicle Damage Classification
  TAD . . . . 01FR5 & 01RFQ5
  CDC . . . . 01FREK2 & 01RYAW3
Maximum Vehicle Crush . 11.0 in

Impact Speed . . . 60.2 mi/h
Impact Angle . . . 21.2 degrees
Speed at Parallel . 39.3 mi/h
Exit Speed . . . . 30.2 mi/h
Exit Trajectory . . 0 degrees
Vehicle Accelerations
  (Max. 0.050-sec Avg)
  Longitudinal . . -13.5 g
  Lateral . . . . . 11.3 g
Occupant Impact Velocity
  Longitudinal . . 28.6 ft/s
  Lateral . . . . . 16.6 ft/s
Occupant Ridedown Accelerations
  Longitudinal . . -2.0 g
  Lateral . . . . . 3.6 g

FIGURE 10 Summary of results for Test 1185-1.

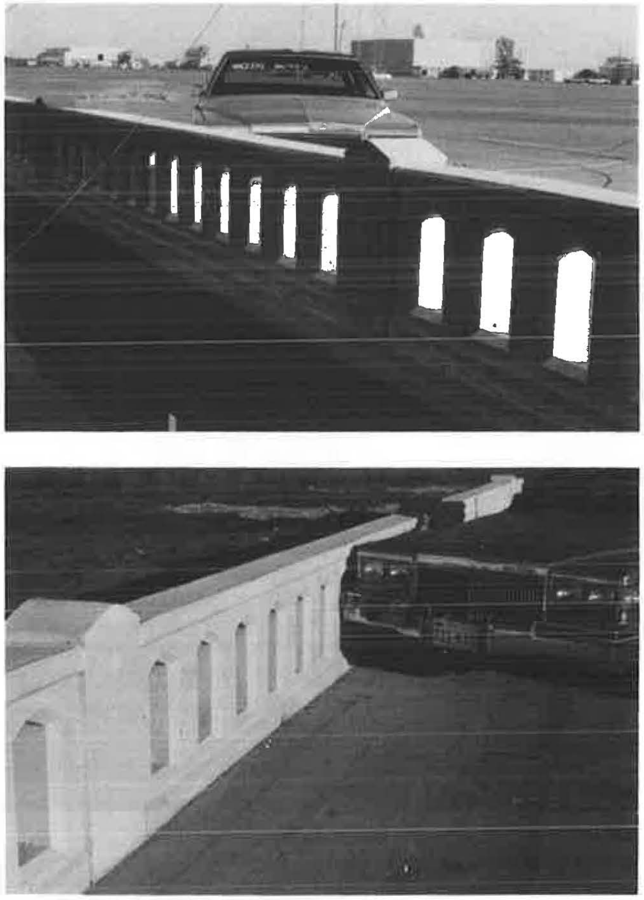


FIGURE 11 Vehicle and bridge rail geometrics for Test 1185-2.

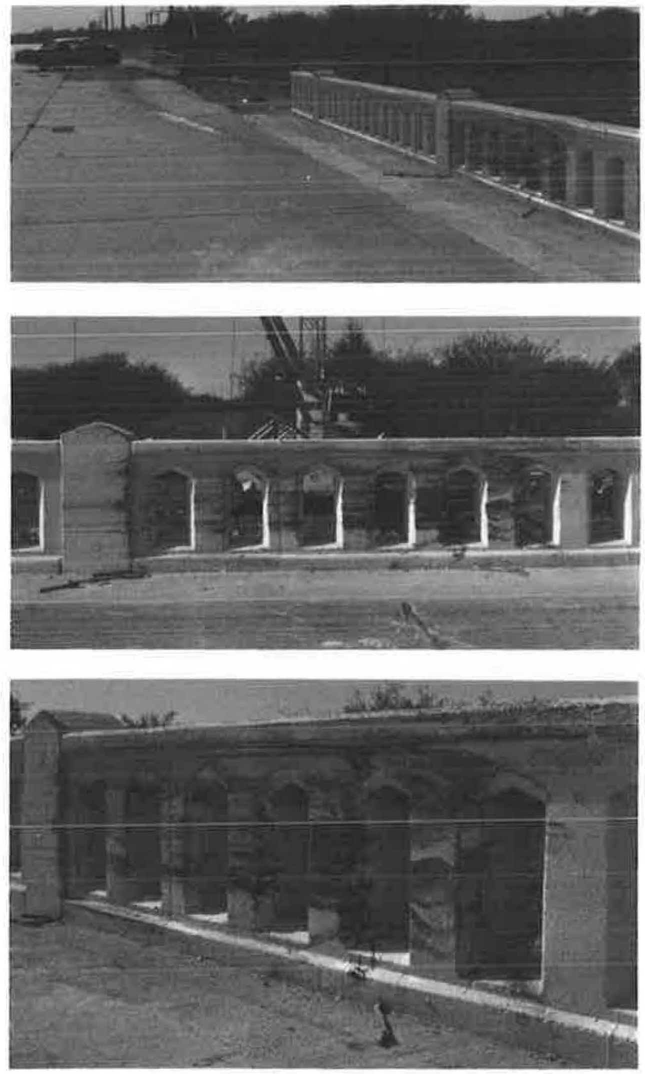


FIGURE 12 Test installation after Test 1185-2.

Conclusions

The bridge rail contained and smoothly redirected the test vehicle with no lateral movement of the bridge rail. The vehicle remained upright and relatively stable during the collision. The vehicle trajectory at loss of contact indicates minimum intrusion into adjacent traffic lanes (exit angle 5.9 degrees). These test data satisfied all the occupant safety evaluation criteria of *NCHRP Report 230* and those in the 1989 AASHTO *Guide Specifications for Bridge Railings*. The effective coefficient of friction u for this test was 0.77, or marginal.

SUMMARY AND CONCLUSIONS

Table 1 compares the vehicle impact behavior of the aesthetic bridge rail, T411, with vehicle impact behavior obtained from several other rigid longitudinal traffic barriers. It can be seen that the change in speeds of the vehicles during impact (23.3 mph and 30.0 mph) were larger than those obtained from the others, but the exit angles (0 degrees and 5.9 degrees) were smaller than those obtained from the others. Because the vehicles did not return to the traffic lanes but stayed against the rail, the larger change in speed is not important.

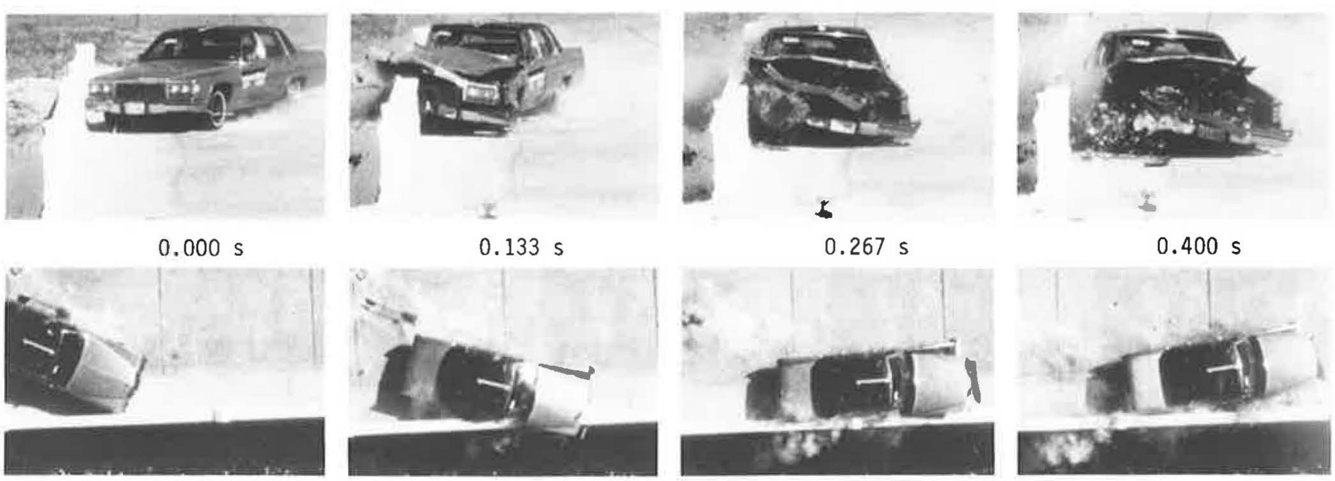
The longitudinal accelerations ($-12.8\,g$ and $-13.5\,g$) were larger than those obtained from the other rails but were acceptable. These larger longitudinal accelerations were expected because the vehicle grinds into the vertical openings. The larger effective coefficients of friction u of 0.54 and 0.77 were also expected and attributed to the vertical openings in the T411 rail. The transverse accelerations of 11.3 g and 16.5 g were about the same as those obtained from the other barriers.

The longitudinal occupant impact velocities of 28.6 ft/sec and 28.7 ft/sec were larger than those obtained from the other rails but were less than the limit of 30.0 ft/sec (*4*). The transverse occupant impact velocities of 16.6 ft/sec and 23.0 ft/sec were less than those obtained from the other rails and smaller than the limit of 25.0 ft/sec (*4*).

The longitudinal ridedown accelerations of $-2.0\,g$ and $-12.4\,g$ were larger than those obtained from the other rails but less than the proposed limit of $-15.0\,g$. The transverse ridedown accelerations of 3.6 g and 10.5 g were smaller than those obtained from the other rails and smaller than the proposed limit of 15.0 g.

It is therefore concluded that the new Texas T411 bridge rail has successfully met the crash test requirements of *NCHRP Report 230*.

FIGURE 13 Vehicle after Test 1185-2.



Test No. 1185-2
Date 12/01/88

Test Installation . . . T411 Bridge Rail
Installation Length . . 75 ft

Vehicle . . . . 1980 Cadillac
Vehicle Weight
  Test Inertia . . . . . 4,500 lb
Vehicle Damage Classification
  TAD . . . . 01FR6 & 01RFQ7
  CDC . . . . 01FZEK2 & 01RYAW4
Maximum Vehicle Crush . 16.0 in

Impact Speed . . . 62.2 mi/h
Impact Angle . . . 26.0 degrees
Speed at Parallel . 41.7 mi/h
Exit Speed . . . . 38.9 mi/h
Exit Trajectory . . 5.9 degrees
Vehicle Accelerations
  (Max. 0.050-sec Avg)
  Longitudinal . . -12.8 g
  Lateral . . . . . 16.5 g
Occupant Impact Velocity
  Longitudinal . . 28.7 ft/s
  Lateral . . . . . 23.0 ft/s
Occupant Ridedown Accelerations
  Longitudinal . . -12.4 g
  Lateral . . . . . 10.5 g

FIGURE 14 Summary of results for Test 1185-2.

TABLE 1 COMPARISON OF VEHICLE IMPACTS INTO THE AESTHETIC TYPE T411 BRIDGE RAIL WITH VEHICLE IMPACTS INTO OTHER RIGID LONGITUDINAL TRAFFIC BARRIERS

(1) Test No.	(2) Change in Speed mph	(3) Exit Angle degrees	(4) Long. Accel. g's	(5) Trans. Accel. g's	(6) Long. Occupant Impact Vel. fps	(7) Trans. Occupant Impact Vel. g's	(8) Long. Ridedown Accel. g's	(9) Trans. Ridedown Accel. g's	(10) Type Rail
NCHRP 230 Test 10 - 4,500 lb, 60 mph, 25°									
1179-2	14.5	2.0	- 9.7	14.3	23.9	27.3	- 4.9	16.7	Conc. C202
7046-1	15.9	17.5	- 4.8	14.0	19.4	28.2	- 5.4	14.4	Conc. Wall
3451-7	18.5	13.5	- 5.2	6.9	11.9	15.4	-	-	T101
7091-10	12.9	6.3	- 6.3	12.5	18.6	27.0	- 5.9	10.8	IBC
3451-36	17.4	6.3	- 9.1	15.4	10.9	23.0	-	-	Conc. Wall
7091-11	13.4	7.3	- 6.4	11.6	23.2	26.6	- 3.8	10.6	IBC
Avg.	15.4	8.8	- 6.9	12.5	18.0	24.6	- 5.0	13.1	
1185-1	23.3	5.9	-12.8	16.5	28.7	23.0	-12.4	10.5	T411
NCHRP 230 Test 13 - 1,800 lb, 60 mph, 20°									
1179-1	16.8	0.6	-11.2	14.0	23.3	25.7	- 2.0	9.3	Conc. C202
3451-27	13.2	1.0	- 9.2	10.3	18.6	19.5	-	-	Indiana Alum.
3451-28	19.9	3.5	-13.6	10.2	20.1	20.3	-	-	Indiana Alum.
7069-3	7.1	6.2	- 8.0	12.8	19.0	23.7	- 2.1	4.9	Conc. F Shape
7069-5	11.9	6.2	- 8.0	14.0	20.1	26.0	- 1.6	9.4	Conc. Wall
7069-10	10.2	5.2	- 6.4	14.2	16.9	25.1	- 1.4	8.5	Ill. Steel
Avg.	13.2	3.8	- 9.4	12.6	19.7	23.4	- 1.7	8.0	
1185-2	30.0	0	-13.5	11.3	28.6	16.6	- 2.0	3.6	

ACKNOWLEDGMENTS

This research study was conducted under a cooperative program between the Texas Transportation Institute, the State Department of Highways and Public Transportation (SDHPT), and the Federal Highway Administration. Dean Van Landuyt, John J. Panak, and Van M. McElroy (all of the SDHPT) were closely involved in all phases of this study.

REFERENCES

1. *Standard Specifications for Highway Bridges*. Twelfth ed. AASHTO, Washington, D.C., 1977.
2. T. J. Hirsch. *Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks*. Research Report 230-2. Texas Transportation Institute, Texas A&M University, College Station, Aug. 1978.
3. J. D. Michie. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., 1981.
4. *Guide Specifications for Bridge Railings—An Alternative Bridge Railing Specification in the AASHTO Standard Specifications for Highway Bridges*. AASHTO, Washington, D.C., 1988.

Wyoming Tube-Type Bridge Rail and Box-Beam Guardrail Transition

KING K. MAK, ROGER P. BLIGH, AND DAVID H. POPE

The results of testing and evaluation of the current Wyoming steel tube-type bridge railing system and the development, testing, and evaluation of a box-beam guardrail transition for use with this bridge railing system are presented. The crash test results indicate that both the bridge railing and the box-beam guardrail transition retrofit design satisfy the guidelines set forth in *NCHRP Report 230*. The steel tube-type bridge railing is currently on the approved list of bridge railings for use on Federal-Aid projects. The box-beam guardrail transition is currently under review by FHWA for approval. The key advantages of this steel tube-type bridge railing are that it is aesthetically pleasing and that it allows the traveling public views of surrounding areas from the bridge deck. Its initial and maintenance costs are competitive. This type of railing does not present problems with drifting snow or clearing snow from roadways, commonly associated with concrete barriers. These problems are what prompted the development of the transition treatment so that the box-beam guardrail could be used in conjunction with the steel tube-type bridge rail.

The Wyoming steel tube-type bridge railing system has been used in the state since the early 1960s with only minor changes over the years. It is a low-profile, streamlined rail that is aesthetically pleasing and allows the traveling public views of surrounding areas from the bridge deck. The rail is versatile and has minimal maintenance costs. Replacement rail posts, rails, and hardware can be stockpiled, both by fabricators and in highway department maintenance yards, to expedite repairs to damaged rails. Experience indicates that the rail has performed well in the field. There has never been any penetration or vaulting over the rail, even when struck by tractor-semitrailers.

The rail's installed cost is competitive with the concrete alternatives installed on a limited basis throughout the state. One major problem encountered with concrete-type bridge railings, because of their closed nature, is that of drifting snow and clearing snow from roadways. The open nature of the structural steel, tube-type bridge railing does not present this problem. This bridge railing remains popular throughout the state.

The Wyoming State Highway Department contracted with Texas Transportation Institute (TTI) to crash test and evaluate this steel tube-type bridge railing (*1*) and, in a follow-up study (*2*), to develop a transition treatment from a box-beam guardrail to the steel tube-type bridge railing. The results of these two studies are presented.

K. K. Mak and R. P. Bligh, Texas Transportation Institute, Texas A&M University System, College Station, Tex. 77843. D. H. Pope, Wyoming State Highway Department, Cheyenne, Wyo. 82002.

WYOMING TUBE-TYPE BRIDGE RAIL

Description of Bridge Rail and Installation

The Wyoming bridge rail consists of fabricated posts spaced 9 ft 3 in. apart with two TS 6 × 2 × 0.25 tube-type beams. The structural steel components of the bridge rail conform to the requirements of ASTM A 500 or ASTM A 501. The metal rail sits on top of a 6-in.-high curb for a total height of 29 in. above the pavement surface. The face of the curb was flush with the traffic face of the rails. The 77-ft bridge rail was installed on a simulated bridge deck of the same length, which was designed and constructed in accordance with standard bridge specifications used by the Wyoming State Highway Department. Photographs of the installation are shown in Figure 1.

Crash Testing and Evaluation

Two crash tests were conducted to evaluate the Wyoming bridge rail system:

1. Test S13—1,800-lb vehicle striking the bridge rail at 60 mph and 20 degrees.
2. Test 10—4,500-lb vehicle striking the bridge rail at 60 mph and 25 degrees.

A decision was made by the Wyoming Highway Department, after consultation with FHWA, to use Test S13 instead of Test 12 (1,800-lb passenger car striking the bridge rail at 60 mph and 15 degrees) as the small-car test. The rationale was that the 20-degree impact angle is a more severe test and provides a better assessment in terms of wheel snagging.

The crash test and data analysis procedures were generally in accordance with guidelines presented in *NCHRP Report 230* (*3*). The test vehicles were instrumented with three rate transducers to measure roll, pitch, and yaw rates and with a triaxial accelerometer near the vehicle center of gravity to measure acceleration levels. An uninstrumented 50-percentile male dummy was placed in the driver's seat for the 1,800-lb car test, but not for the 4,500-lb car test.

Test 1 (0368-1)

A 1979 Honda Civic struck the railing at 61.1 mph and 20.0 degrees. The point of impact was the center of the splice for the top rail, approximately 38 ft downstream from the begin-

FIGURE 1 Wyoming tube-type bridge rail.

FIGURE 2 Barrier and vehicle damage after Test 0368-1.

ning of the bridge railing. The vehicle was smoothly redirected and exited from the rail at a speed of 49.7 mph and at an exit angle of 7.1 degrees. The vehicle was in contact with the rail for a total of 8.1 ft.

The rail sustained minor damage, as shown in Figure 2. The permanent residual deformation was 0.25 in. vertically and laterally for both the top and bottom rails. The only repair necessary after the test was to loosen the bolts attaching the rail elements to two posts and realign the rail elements.

The vehicle sustained moderate damage, considering the severity of the impact. As shown in Figure 2, the damage consisted primarily of sheet metal crushing along the front left side of the vehicle. Maximum crush was 7.0 in. at the left front corner of the vehicle. There was also damage to the left front strut assembly and tire rim. In addition, the left door became ajar and the window glass was broken.

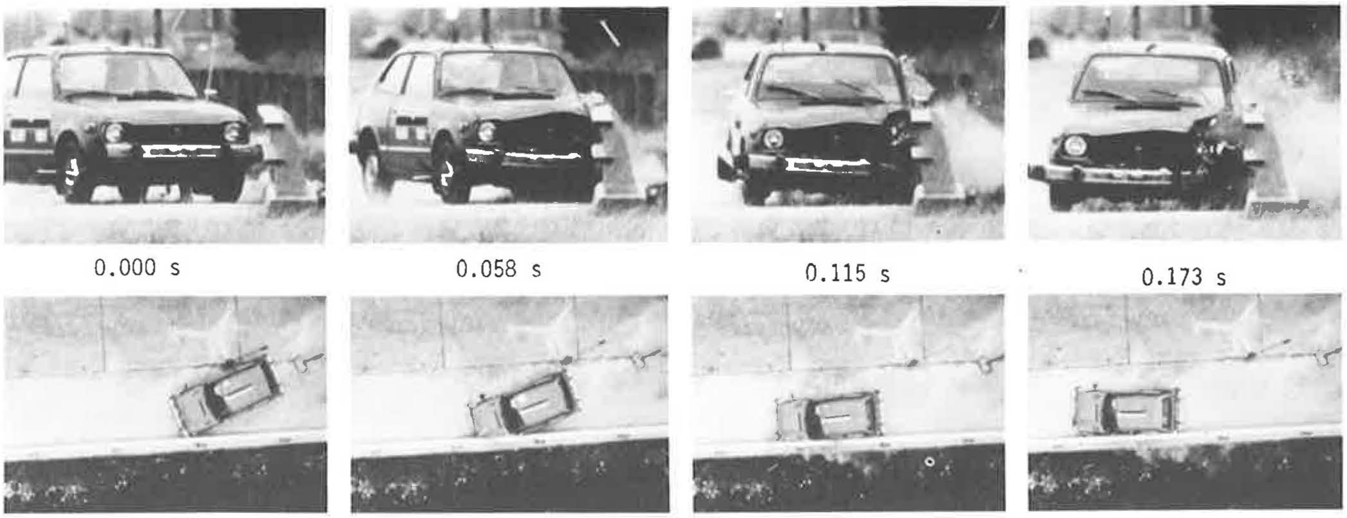
Sequential photographs and a summary of the test results and other information pertinent to this test are given in Figure 3. The maximum 0.050-sec average acceleration experienced by the vehicle was -8.0 *g* in the longitudinal direction and -16.5 *g* in the lateral direction. Occupant impact velocities were 20.9 ft/sec and 30.9 ft/sec in the longitudinal and lateral directions, respectively. The highest 0.010-sec occupant ride-down accelerations were -2.7 *g* (longitudinal) and -10.1 *g* (lateral).

The lateral occupant impact velocity of 30.9 ft/sec was marginally higher than the limit of 30 ft/sec according to the guidelines on occupant risk criteria in *NCHRP Report 230*. However, once the occupant impact velocity is adjusted to account for the higher vehicle impact speed of 61.1 mph, it falls within the limit at 29.8 ft/sec. A comparison was made with other bridge rails recently crash tested at TTI. For tests involving 1,800-lb passenger cars striking the bridge rails at 60 mph and 20 degrees, the lateral occupant impact velocities ranged from 23.7 to 30.3 ft/sec. Although the Wyoming tube-type bridge rail is at the high end of the spectrum, its performance is not considered to be significantly different from that of the other bridge rails. Given the rigid nature of bridge rails and the severe impact angle of 20 degrees, a relatively high lateral occupant impact velocity is to be expected.

The occupant risk criteria are not applicable to any of the four crash tests reported in this paper, in accordance with *NCHRP Report 230* requirements. The results are reported for information purposes only.

Test 2 (0368-2)

A 1979 Cadillac Sedan de Ville struck the railing at 63.3 mph and 25.0 degrees. The point of impact was midway between the posts for the span containing the splice for the top rail, approximately 40 ft downstream from the beginning of the rail. Although the deflated front tire and deformed sheet metal of the vehicle contacted the first post downstream from the impact point, the vehicle was smoothly redirected. The vehicle exited from the rail at a speed of 45.9 mph and an exit angle of 4.6 degrees. The vehicle was in contact with the rail for a total of 10.6 ft.



Test No	0368-1
Date.	02/09/88
Test Installation	Wyoming Tube-Type Bridge Rail
Length of Installation. . . .	77 ft (23.5 m)
Vehicle	1979 Honda Civic
Vehicle Weight	
Test Inertia.	1,800 lb (816 kg)
Gross Static.	1,968 lb (893 kg)
Vehicle Damage Classification	
TAD	11LFQ5
CDC	11FLES2 & 11LFES3
Maximum Vehicle Crush	7.0 in (17.8 cm)
Max. Perm. Rail Deformation .	0.25 (0.64 cm)

Impact Speed	61.1 mi/h (98.3 km/h)
Impact Angle	20.0 degrees
Exit Speed	49.7 mi/h (80.0 km/h)
Exit Angle	7.1 degrees
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal	- 8.0 g
Lateral	-16.5 g
Occupant Impact Velocity	
Longitudinal	20.89 ft/s (6.4 m/s)
Lateral	30.89 ft/s (9.4 m/s)
Occupant Ridedown Accelerations	
Longitudinal	- 2.7 g
Lateral	-10.1 g

FIGURE 3 Summary of results for Test 0368-1.

The rail sustained minor damage, as shown in Figure 4. The permanent residual deformation was 1.25 in. vertically and 0.75 in. laterally for the bottom rail and approximately 0.50 in. both vertically and laterally for the top rail. Diagonal stress cracks were found on the concrete bridge deck around the post immediately downstream from the point of impact, and a small piece of concrete was broken off behind the post.

The vehicle sustained light to moderate damage, as shown in Figure 4. The front end of the car was shifted to the right and the subframe was bent. Maximum crush was 18.0 in. at the left front corner of the vehicle. The primary and secondary hood latches of the vehicle were disengaged by the impact, and part of the hood slid across the top of the top rail element.

Sequential photographs and a summary of the test results and other information pertinent to this test are given in Figure 5. The maximum 0.050-sec average accelerations experienced by the vehicle were -9.6 *g* in the longitudinal direction and -14.7 *g* in the lateral direction. Occupant impact velocities were 25.1 ft/sec and 29.5 ft/sec in the longitudinal and lateral directions, respectively. The highest 0.010-sec occupant ridedown accelerations were -5.8 *g* (longitudinal) and -12.2 *g* (lateral). Although not required for the evaluation of a transition test, the occupant impact velocities and ridedown accelerations were all within the acceptable limits.

The vehicle velocity change of 17.4 mph was higher than the limit of 15 mph recommended in *NCHRP Report 230*. However, because the exit angle of 4.6 degrees was substantially less than 60 percent of the impact angle and the vehicle trajectory indicated a minimal potential for intrusion into the adjacent traffic lanes, the 15 mph criterion was not considered applicable.

Summary

Results of the two crash tests indicate that the Wyoming steel tube-type bridge railing generally meets the guidelines set forth in *NCHRP Report 230*. The rail contained and smoothly redirected the vehicles with little lateral movement of the barrier. The vehicles sustained light to moderate damage with minimal deformation and intrusion into the occupant compartment. The vehicle trajectories at loss of contact with the rail indicate minimum potential for intrusion into the adjacent traffic lanes. The vehicles remained upright and stable during the initial test periods and after leaving the rail. This bridge railing is approved for use on Federal-Aid projects.

BOX-BEAM GUARDRAIL TRANSITION

As discussed in the previous section, the tube-type bridge rail was found to be in compliance with guidelines set forth in *NCHRP Report 230*. However, the exposed end of this bridge railing, like any rigid bridge railing, can present a serious

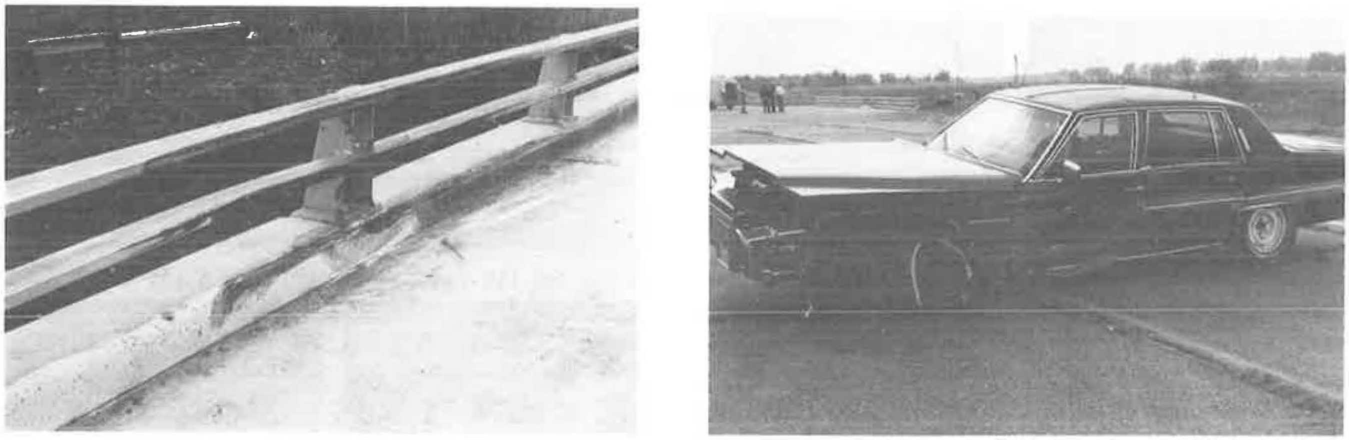
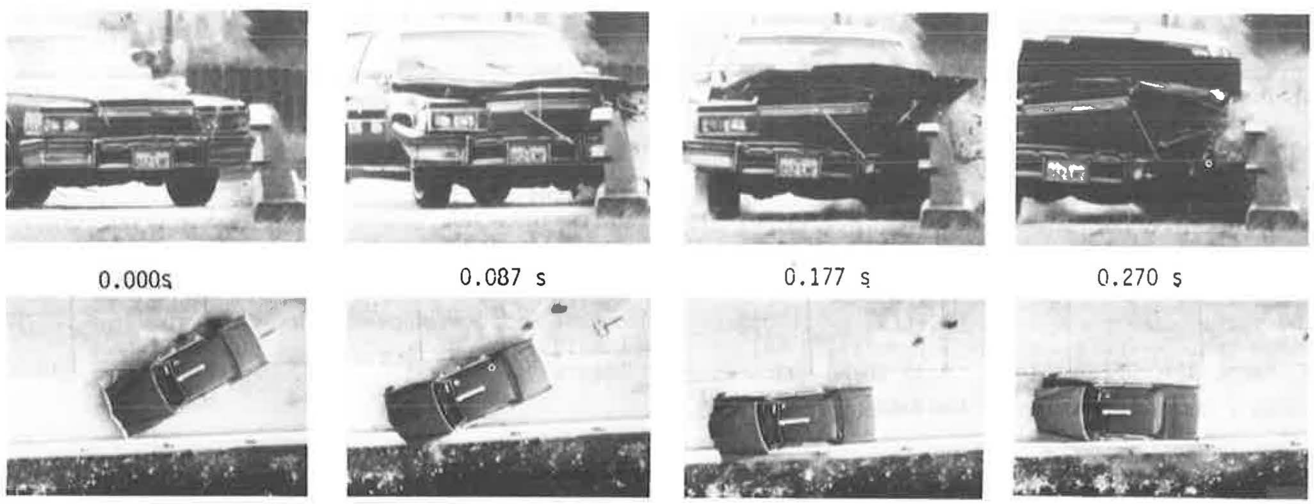


FIGURE 4 Barrier and vehicle damage after Test 0368-2.



Test No	0368-2
Date.	02/10/88
Test Installation	Wyoming Tube-Type Bridge Rail
Length of Installation. . . .	77 ft (23.5 m)
Vehicle	1979 Cadillac Sedan Deville
Vehicle Weight	
Test Inertia.	4,510 lb (2,045.7 kg)
Vehicle Damage Classification	
TAD	11LFQ6
CDC	11FLEK3 & 11LYES4
Maximum Vehicle Crush	18.0 in (45.7 cm)
Max. Perm. Rail Deformation .	1/25 (3.2 cm)

Impact Speed	63.3 mi/h (101.9 km/h)
Impact Angle	25.0 degrees
Exit Speed	45.9 mi/h (73.9 km/h)
Exit Angle	4.6 degrees
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal	- 9.6 g
Lateral	-14.7 g
Occupant Impact Velocity	
Longitudinal	25.1 ft/s (7.7 m/s)
Lateral	29.5 ft/s (9.0 m/s)
Occupant Ridedown Accelerations	
Longitudinal	- 5.8 g
Lateral	-12.2 g

FIGURE 5 Summary of results for Test 0368-2.

safety hazard if improperly treated. In most instances, an approach roadside barrier is used to shield the exposed bridge railing end and to prevent errant vehicles from getting behind the railing and encountering underlying hazards. These approach guardrails are typically much more flexible than the bridge railings and can deflect sufficiently to allow an errant vehicle to strike or snag on the end of the rigid bridge railing. A transition section is therefore warranted whenever there is a significant change in lateral strength from the approach guardrail to the bridge rail.

A limited number of studies have addressed the transition problem and, consequently, few standards exist. In recent years, however, several acceptable guardrail-to-bridge-railing transition designs have been developed and tested (*4–6*). Although these designs have exhibited good impact performance, most of this research has focused on developing a transition from a strong-post W-beam guardrail to a rigid concrete parapet. Little, if any, analysis has been conducted on developing a transition from a weak-post box-beam guardrail to a steel tube-type bridge railing such as that used by Wyoming and other states.

Transition Development

The relatively high degree of flexibility of the box-beam guardrail (deflections of 5 to 6 ft are not uncommon for severe impacts) makes significant modifications necessary in developing a transition to a rigid bridge railing. The required increase in lateral barrier strength can be achieved by varying several key design parameters. These parameters include guardrail beam strength, post size or strength, and post spacing.

The major features of the basic transition design included a continuation of the lower TS 6 × 2 × 0.25 steel tube from the bridge rail onto the transition treatment and the use of stronger W 6 × 9 steel posts at a reduced post spacing near the bridge rail end. Computer simulation techniques were used to model this basic design and to evaluate various design alternatives, such as the number and spacing of the heavier posts.

Computer Simulation

The Barrier VII computer simulation model (*7*) was chosen for use in developing the new transition design. Despite its two-dimensional nature, the Barrier VII program has been successfully used to simulate impacts with a variety of flexible barriers, including transitions from flexible to rigid barriers (*4–6*). The program has been shown to be capable of accurately predicting barrier response under severe impact conditions. Further, for impacts into barriers on flat terrain, such as that found on the approach to a bridge, vehicle vaulting and underride are of little concern.

All simulations for this study involved impacts with a 4,500-lb vehicle traveling 60 mph at an angle of 25 degrees. This impact condition simulated Test 30 of *NCHRP Report 230*, which is the recommended test for evaluating the performance of a transition. This test examines the structural adequacy of the transition as well as the propensity for the more flexible barrier to deflect and allow a vehicle to snag on the end of the stiffer barrier.

The purpose of the computer simulations was to evaluate the effect of post size and spacing on barrier performance. The systems were compared on the basis of maximum dynamic barrier deflection and the extent of wheel contact estimated on the guardrail posts and bridge rail end. Because of the large deflections associated with their use, the weak S 3 × 5.7 posts had to be replaced in the transition region. Use of a stronger W 6 × 9 post was investigated for two different post spacings, 4 ft and 2 ft, near the bridge end. These post spacings were selected because Wyoming's current transition to the steel tube-type bridge railing uses a 4-ft post spacing. Thus, both of these spacings would provide for a simplified retrofit operation, which was considered an important factor in the transition development.

Table 1 summarizes the simulation results obtained when using W 6 × 9 steel posts with different post spacing. As indicated in Table 1, Barrier VII predicted various degrees of wheel contact for the two alternatives. Computer simulation models, such as Barrier VII, cannot simulate tire-post interactions, but they can be used to predict when wheel contact might occur. The extent of wheel contact is inferred from post deflections and wheel positions during the impact event.

Because tire-post interactions cannot be accurately simulated, it is difficult to determine how a vehicle's wheel will behave after such contact has occurred. Depending on the type and degree of contact, the tire may simply roll around or over the post, be pushed back into the wheel well, or rotate about the ball joint. The behavior of the wheel after initial contact will determine the extent of contact on subsequent posts. Wheel contact with the steel guardrail post, in itself, does not necessarily represent a severe safety hazard. By design, the box-beam guardrail readily detaches from its supporting posts. This leaves the steel posts unrestrained at the top and allows them to deform more readily on wheel contact. Furthermore, some wheel contact can be viewed as beneficial to vehicle stability and trajectory. When a wheel is damaged, the vehicle tends to remain adjacent to the barrier, thereby lending stability to the vehicle and preventing it from exiting into adjacent traffic lanes at a high angle. Thus, the design alternatives were evaluated not solely on whether or not post contact occurred, but also on the amount of contact predicted.

Numerous simulations were also made to analyze the behavior of the secondary transition from the standard box-beam guardrail to the transition section with the lower rail extended. The extension of the lower rail aids in the smooth transition of lateral stiffness from a weak-post box-beam guardrail to the strong-post transition treatment. Simulations also indicated that a 9-in. spacer or blockout should be used behind the guardrail post where the lower rail was terminated. This

TABLE 1 BARRIER VII RESULTS FOR TRANSITION USING W 6 × 9 STEEL POSTS WITH DIFFERENT POST SPACINGS

Post Spacing (ft)	Maximum Barrier Deflection (in)	Extent of Wheel Contact: Post 2* Contact (in)	Post 2* Rotation (deg)	Post 1 Contact (in)	Post 1 Rotation (deg)	Bridge Rail End Contact (in)	Bridge Rail End Rotation (deg)
4	12.3	NA	NA	4.0	12	-2.0	0
2	9.3	2.9	7	3.5	8.5	-4.5	0

* Intermediate post for 2-foot post spacing design

reduces the potential for snagging on the end of the lower tube when the barrier is struck upstream of the transition. Details of the final design are described below.

Final Design Selection

As indicated in Table 1, use of the 2-ft post spacing increases the lateral strength of the transition and thereby reduces the dynamic deflection of the rail by approximately 3 in. By decreasing the deflection, the probability of wheel contact on the rigid bridge rail post is reduced. On the other hand, use of the 2-ft post spacing introduces more posts into the vehicle's path and, therefore, the amount of significant wheel contact on the W 6 × 9 steel posts is increased. Post rotations are reduced because of the decrease in rail deflection, so the predicted snagging for this design has the potential for being more severe than that predicted for the 4-ft post spacing design. In addition, the 2-ft post spacing has the potential for collecting drifting snow and hindering snow-clearing operations. Taking into account the simulation results and the considerations mentioned, it did not appear that the closer post spacing was warranted. Thus, the 4-ft post spacing was selected for testing in the final design.

The final transition design uses two different rail elements. The upper TS 6 × 6 × 3⁄16 box beam is mounted at a height of 29 in. and is attached to the upper bridge rail element with a special tapered sleeve. The lower TS 6 × 2 × 0.25 steel tube is mounted at a height of 17 in. and is carried off the bridge a distance of 36 ft, at which point it is flared away from the roadway behind a guardrail post. A C 9 × 13.5 spacer is used to block out the lower rail from the post when it is terminated. Three standard S 3 × 5.7 posts extend into the transition at the standard post spacing of 6 ft 0 in. before switching to the heavier W 6 × 9 posts. The first space with the heavier posts remains at 6 ft 0 in., after which the spacing is reduced to 4 ft 0 in. near the bridge end. The end of the curb on the bridge deck is tapered back away from the roadway to help reduce the potential for wheel snagging.

Photographs of the test installation are shown in Figure 6.

Crash Testing and Evaluation

According to *NCHRP Report 230* guidelines, one crash test (Test 30) is recommended for the evaluation of a transition installation. The test involves a 4,500-lb full-size automobile striking 15 ft (or at the most critical point) upstream of the second and more laterally stiff system at a speed of 60 mph and at an angle of 25 degrees. However, because of the design of this transition treatment, there are two transition points, one from the flexible weak-post box-beam guardrail to the semirigid transition treatment and the second from the semirigid transition treatment to the rigid bridge railing. Two full-scale crash tests were thus conducted, one for each of the two transition points.

Simulation runs using the Barrier VII program were conducted to determine the most critical point of impact for each of the two transition points. For the transition from the transition treatment to the bridge railing, the most critical impact point was determined to be approximately 9½ ft upstream from the first bridge rail post, or midspan of Posts 2 and 3 of the transition treatment. For the transition from the box-beam to the transition treatment, a distance of 15 ft upstream from the beginning of the transition treatment was found to be

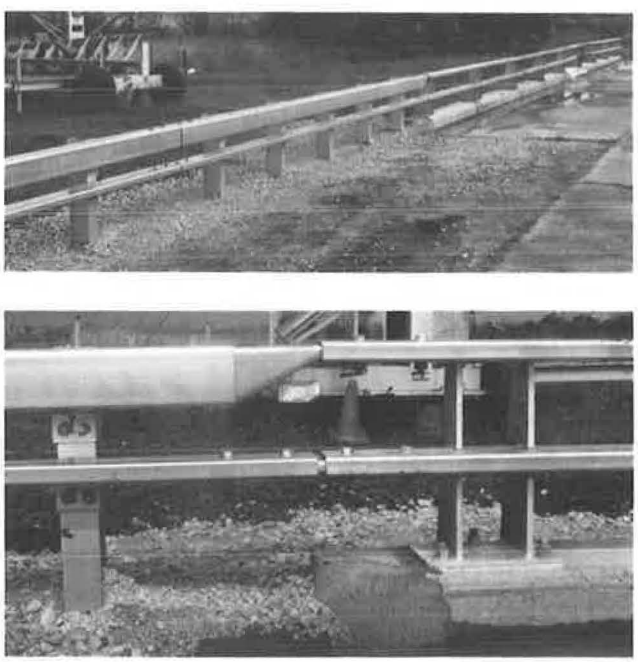
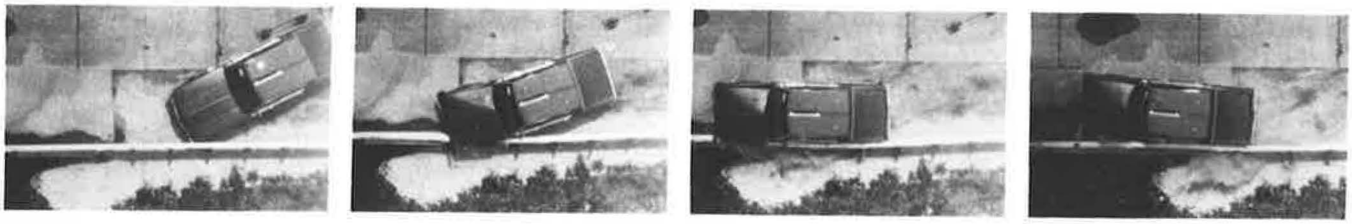


FIGURE 6 Wyoming bridge rail transition.

FIGURE 7 Barrier and vehicle damage after Test 0382-1.



0.000 s 0.099 s 0.199 s 0.298 s

Test No. 0382-1
Date 07/11/89
Test Installation. . . . . Wyoming Bridge Rail Transition
Installation Length . . . 168.3 ft (51.3 m)
Vehicle. . . . . 1980 Oldsmobile Ninety-Eight
Vehicle Weight
Test Inertia . . . . . . 4,500 lb (2,043 kg)
Vehicle damage Classification
TAD. . . . . 11LFQ-4 & 11LD-2
CDC. . . . . 11FLEK2 & 11LFES
Maximum Vehicle Crush. . . 12.0 in (30.5 cm)
Maximum Dynamic
Rail Deflection. . . . . 12.0 in (30.5 cm)

Impact Speed . . . . . . 62.8 mi/h (101.1 km/h)
Impact Angle. . . . . . . 25.5 deg
Exit Speed. . . . . 44.4 (71.4 km/h)
Exit Trajectory . . . . . 9.7 deg
Vehicle Accelerations
(Max. 0.050-sec Avg)
Longitudinal. . . . . . -8.4 g
Lateral . . . . -12.6 g
Occupant Impact Velocity
Longitudinal. . . . . . 28.0 ft/s (8.5 m/s)
Lateral . . . . 27.7 ft/s (8.4 m/s)
Occupant Ridedown Accelerations
Longitudinal. . . . . . -6.1 g
Lateral . . . . -14.2 g

FIGURE 8 Summary of results for Test 0382-1.

most critical. As it turned out, this point corresponded to a rail splice connection for the box beam.

Test 3 (0382-1)

A 1980 Oldsmobile Ninety-Eight struck the transition installation 9½ ft upstream of the first bridge rail post at 62.8 mph and 25.5 degrees. The vehicle was smoothly redirected, although the front of the vehicle did partially strike the first bridge rail post (i.e., the post immediately downstream of the transition). The vehicle exited the installation at a speed of 44.4 mph and an exit angle of 9.7 degrees.

The transition installation and bridge railing sustained moderate damage, as shown in Figure 7. The first post in the transition (i.e., Post 1 immediately upstream of the bridge railing end) became completely detached from the upper and lower rails and was displaced a maximum of 4.0 in. rearward and 7.0 in. laterally. Maximum permanent rail deformation was 5.0 in. for the upper rail at the first transition post and 4.8 in. for the lower rail at the second post in the transition. The maximum dynamic rail deflection was 12 in. Maximum displacement at the first bridge rail post was 3.8 in.

Damage to the vehicle is shown in Figure 7. The damaged areas included the left side of the vehicle and the left front tire and rim. The left front wheel was pushed rearward a total of 9.5 in., causing the floor of the passenger compartment to be deformed slightly. The maximum crush was 12.0 in. at the front left corner of the vehicle.

Sequential photographs and a summary of the test results and other information pertinent to this test are given in Figure 8. The maximum 0.050-sec average accelerations experienced by the vehicle were -8.4 *g* in the longitudinal direction and -12.6 *g* in the lateral direction. Occupant impact velocity was 28.0 ft/sec in the longitudinal direction and 27.7 ft/sec in lateral direction. The maximum 0.010-sec occupant ridedown accelerations were -6.1 *g* (longitudinal) and -14.2 *g* (lateral). Although not required for the evaluation of a transition test, the occupant impact velocities and ridedown accelerations were all within the maximum acceptable limits.

The vehicle velocity change of 18.4 mph was higher than the limit of 15 mph recommended in *NCHRP Report 230*. However, because the exit angle of 9.7 degrees was substantially less than 60 percent of the impact angle and the vehicle trajectory indicated a minimal potential for intrusion into the adjacent traffic lanes, the 15-mph criterion was not considered applicable.

Test 4 (0382-2)

A 1981 Oldsmobile Ninety-Eight struck the box-beam guardrail 15 ft upstream from the beginning of the transition treatment at 61.3 mph and 27.2 degrees. The vehicle was smoothly redirected. As is typical of a flexible weak-post barrier system, the top box-beam rail separated from the posts at the clip angles on impact, and the vehicle contacted the posts and pushed them down. As the front of the vehicle approached the end of the lower rail attached behind the ninth post in the transition, the lower rail detached from its posts, allowing the vehicle to push it down and ride over it. As the vehicle proceeded down the rail, the lower rail continued to separate from the posts at the clip angles. The vehicle exited the rail traveling at 40.5 mph at a shallow angle.

The box-beam guardrail and transition treatment sustained only minor damage, as shown in Figure 9. The first seven posts downstream from the point of impact (Posts 5 through 9 of the transition treatment and Posts 10 and 11 of the box-beam guardrail) were bent over and separated from the upper and lower rails. The next four posts (Posts 1 through 4 of the

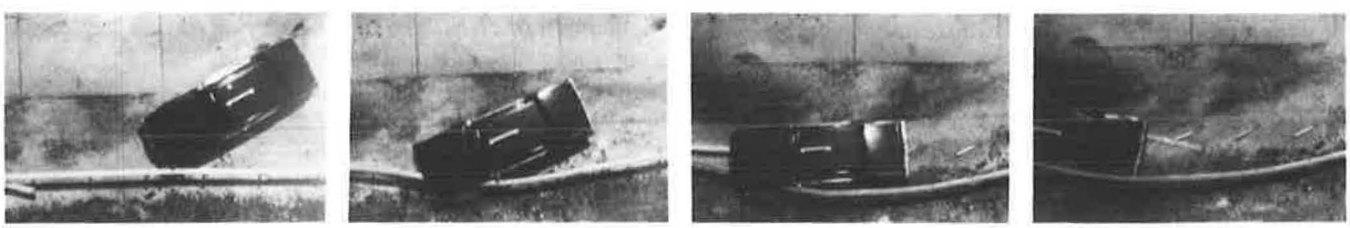
FIGURE 9 Barrier and vehicle damage after Test 0382-2.

transition) remained upright, and the upper rail remained attached to these posts. Post 6 of the transition was completely pulled out from the soil. Maximum dynamic rail deflection was 5.8 ft at the eighth post of the transition. The vehicle remained in contact with the guardrail and transition treatment for a distance of approximately 62 ft.

The vehicle sustained only minor damage, as indicated in Figure 9. The damage was confined to the left side of the vehicle and the left front tire and rim. The maximum crush was 12.5 in. at the front left corner of the vehicle. Although slightly damaged, the left front wheel was not displaced rearward.

Sequential photographs and a summary of the test results and other information pertinent to this test are given in Figure 10. The maximum 0.050-sec average accelerations experienced by the vehicle were $-4.4\ g$ in the longitudinal direction and $-4.8\ g$ in the lateral direction. Occupant impact velocity was 21.1 ft/sec in the longitudinal direction and 16.3 ft/sec in the lateral direction. The maximum 0.010-sec occupant ridedown accelerations were $-8.0\ g$ (longitudinal) and $-8.1\ g$ (lateral). Although not required for the evaluation of a transition test, the occupant impact velocities and ridedown accelerations were all within the maximum acceptable limits.

The vehicle velocity change of 20.8 mph was higher than the limit of 15 mph recommended in *NCHRP Report 230*. However, the exit angle was very shallow and substantially less than 60 percent of the impact angle, and the vehicle trajectory indicated a minimal potential for intrusion into the adjacent traffic lanes. The 15 mph criterion was therefore not considered applicable.



0.000 s   0.170 s   0.340 s   0.510 s

Test No.	0382-2
Date	07/12/89
Test Installation.	Wyoming Bridge Rail Transition
Installation Length. . . .	168.3 ft (51.3 m)
Vehicle.	1981 Oldsmobile Ninety-Eight
Vehicle Weight	
Test Inertia	4,500 lb (2,043 kg)
Vehicle damage Classification	
TAD.	11LFQ-4 & 11LD-2
CDC.	11FLEK2 & 11LFES1
Maximum Vehicle Crush. . .	12.5 in (31.8 cm)
Maximum Dynamic Rail Deflection.	5.8 ft (1.8 m)

Impact Speed	61.3 mi/h (98.6 km/h)
Impact Angle.	27.2 deg
Exit Speed.	N/A
Exit Trajectory	N/A
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal.	-4.4 g
Lateral	-4.8 g
Occupant Impact Velocity	
Longitudinal.	21.1 ft/s (6.4 m/s)
Lateral	16.3 ft/s (5.0 m/s)
Occupant Ridedown Accelerations	
Longitudinal.	-8.0 g
Lateral	-8.1 g

FIGURE 10 Summary of results for Test 0382-2.

TABLE 2 SUMMARY OF CRASH TEST RESULTS AND EVALUATION CRITERIA

Evaluation Factors		Evaluation Criteria	Bridge Rail Test 1	Bridge Rail Test 2	Transition Test 3	Transition Test 4
Structural Adequacy	A.	Test article shall smoothly redirect the vehicle; the vehicle shall not penetrate or go over the installation although controlled lateral deflection of the test article is acceptable.	Yes	Yes	Yes	Yes
	D.	Detached elements, fragments or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.	Yes	Yes	Yes	Yes
Occupant Risk	E.	The vehicle shall remain upright during and after collision although moderate roll, pitching and yawing are acceptable. Integrity of the passenger compartment must be maintained with essentially no deformation or intrusion.	Yes	Yes	Yes	Yes
	F.	Impact velocity of hypothetical front seat passenger against vehicle interior, calculated from vehicle accelerations and 24 in. forward and 12 in. lateral displacements, shall be less than:				
		Occupant Impact Velocity:				
		Longitudinal: Limit - 40 fps, Desirable - 30 fps	20.9	25.1	28.0	21.1
		Lateral: Limit - 30 fps, Desirable - 20 fps	30.9*	29.5	27.7	16.3
		and vehicle highest 10 ms average accelerations subsequent to instant of hypothetical passenger impact should be less than:				
		Occupant Ridedown Accelerations:				
		Longitudinal: Limit - 20 g's, Desirable - 15 g's	- 2.7	- 5.8	- 6.1	- 8.0
		Lateral Limit - 20 g's, Desirable - 15 g's	-10.1	-12.2	-14.2	- 8.1
Vehicle Trajectory	H.	After collision, the vehicle trajectory and final stopping distance shall intrude a minimum distance, if at all, into adjacent traffic lanes.	Yes	Yes	Yes	Yes
	I.	In test where the vehicle is judged to be redirected into or stopped while in adjacent traffic lanes, vehicle speed change during test article collision should be less than 15 mph and the exit angle from the test article should be less than 60 percent of test impact angle, both measured at time of vehicle loss of contact with test device.				
		Vehicle Speed Change: Limit - 15 mph	11.4	17.4**	18.4**	20.8**
		Exit Angle: Less than 60 Percent of Impact Angle (12° for 20° impact angle and 15° for 25° impact angle)	7.1	4.6	9.7	N/A***

* The 30.9 fps lateral occupant impact velocity was marginally higher than the limit of 30 fps. However, once the occupant impact velocity is adjusted to account for the higher vehicle impact speed of 61.1 mph, it would fall within the limit at 29.8 fps.

** The limit of 15 mph speed change is considered as not applicable if the vehicle exit angle is less than 60 percent of the impact angle and the vehicle trajectory does not pose any potential hazard to vehicles in adjacent traffic lanes.

*** The vehicle exit angle was not available since the vehicle was out of the overhead camera's view at the point of exit. However, review of other camera angles indicated that the exit angle would be less than 60 percent of the impact angle.

Summary of Results of Crash Tests

Results of the two crash tests indicate that the Wyoming transition treatment from a standard box-beam guardrail to the steel tube-type bridge rail generally meets with the guidelines set forth in *NCHRP Report 230*. The rail contained and smoothly redirected the vehicle in both crash tests. There was minimal deformation or intrusion into the vehicle occupant compartment. The vehicle exit angle and trajectory indicated minimal potential for intrusion into adjacent traffic lanes. In addition, the vehicle remained upright and stable during the initial test period and after exiting the rail installation.

SUMMARY

The results of testing and evaluation of the current Wyoming steel tube-type bridge railing system and the development, testing, and evaluation of a box-beam guardrail transition for use with this bridge railing system were presented. The crash test results, as summarized in Table 2, indicate that both the bridge railing and the box-beam transition generally satisfy the guidelines set forth in *NCHRP Report 230*. The steel tube-type bridge railing is currently on the approved list of bridge railings for use on Federal-Aid projects. The box-beam guardrail transition is currently under review by FHWA for approval.

REFERENCES

1. K. K. Mak and D. L. Bullard. *Testing and Evaluation of Wyoming Tube-Type Bridge Rail*. Report on Project 0368. Texas Transportation Institute, Texas A&M University, College Station, March 1988.
2. K. K. Mak, R. P. Bligh, C. E. Buth, and D. L. Bullard. *Development of Box-Beam Guardrail Transition to Wyoming Steel Tube-Type Bridge Railing*. Report on Project 0382. Texas Transportation Institute, Texas A&M University, College Station, Aug. 1989.
3. J. D. Michie. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., 1981.
4. R. P. Bligh, D. L. Sicking, and H. E. Ross, Jr. Development of a Strong Beam Guardrail-to-Bridge-Rail Transition. In *Transportation Research Record 1198*. TRB, National Research Council, Washington, D.C., 1988, pp. 105–116.
5. M. E. Bronstad, L. R. Calcote, M. H. Ray, and J. B. Mayer. *Guardrail-Bridge Rail Transition Designs*. Report FHWA-RD-86-178, FHWA, U.S. Department of Transportation, 1986.
6. E. R. Post, R. J. Duby, D. F. Ronnan, and M. D. Cress. Full-Scale Vehicle Crash Tests on Guardrail-Bridgerail Transition Designs with Special Post Spacing. In *Transportation Research Record 1198*. TRB, National Research Council, Washington, D.C., 1988, pp. 11–30.
7. G. H. Powell. *A Computer Program for Evaluation of Automobile Barrier Systems*. Report FHWA-RD-73-51. FHWA, U.S. Department of Transportation, 1973.

Rollover Caused by Concrete Safety-Shaped Barrier

KING K. MAK AND DEAN L. SICKING

The results of a study sponsored by the Federal Highway Administration and conducted at the Texas Transportation Institute that examined the issue of rollovers caused by concrete safety-shaped barriers are presented. The study objectives were to determine the extent and severity of overturn collisions with concrete safety-shaped barriers, identify the causes of rollover of vehicles in impacts with concrete safety-shaped barriers, and identify potential countermeasures to reduce concrete safety-shaped barrier rollovers. The study approach consisted of a critical review of the literature, clinical and statistical analysis of accident data files, and computer simulation. The extent of the rollover problem on concrete safety-shaped barriers was found to be less than reported in previous literature. A number of impact conditions were identified from accident studies and confirmed by simulation as potential contributory factors to rollovers. Three alternative barrier shapes were evaluated as potential countermeasures: F-shape, constant slope, and vertical wall. Results of the evaluation indicate that the F-shaped barrier offers little performance improvement over the existing safety shape. The vertical wall barrier offers the greatest reduction in rollover potential, but with the greatest increase in lateral accelerations. The constant sloped barrier may provide the best compromise solution.

The concrete safety-shaped barrier has been one of the most popular types of barrier since its introduction in the early 1960s, and hundreds of miles of such barriers are in use on the nation's highways. Although the degree to which the concrete safety-shaped barrier has been successful in reducing deaths and serious injuries is unknown, results from various full-scale crash tests suggest that the benefits are substantial. Hundreds, perhaps thousands, of lives may be saved each year because of the deployment of these barriers.

The original research on and development of the concrete safety-shaped barrier began in the 1950s at the General Motors Proving Grounds in Milford, Michigan. In the intervening years, further research sponsored by the Federal Highway Administration (FHWA) continued the development and improvement of this barrier. Advantages of the concrete safety-shaped barrier are several:

- The design of this barrier, with its inclined lower surface, is intended to minimize or prevent damage to vehicles during low-angle impacts.
- The concrete safety-shaped barrier is a rigid barrier that does not deflect to any appreciable degree, even under severe impact conditions.
- Compared with flexible longitudinal barriers (e.g., W-beam guardrails), the maintenance costs for the concrete safety-shaped barrier are negligible.

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

Although the concrete safety-shaped barrier is an important development in the continuing efforts to safely restrain and redirect errant vehicles on the highways, it is not a panacea. One concern regarding the performance of concrete safety-shaped barriers is the increased likelihood of vehicle rollover on impact with this barrier, especially for small cars (i.e., cars weighing less than 2,250 lb) and vehicles with high centers of gravity (e.g., pickup trucks and vans), not to mention large trucks, intercity buses, and school buses.

Past research has provided some insights into the various aspects of the rollover problem in general and with regard to concrete safety-shaped barriers in particular.

- Smaller passenger cars, with reduced roll and yaw moments of inertia, are more prone to overturn than larger passenger cars.
- The relative severity of single-vehicle rollover accidents is much higher than that of nonrollover single-vehicle accidents.
- The potential for overturning during concrete safety-shaped barrier impacts is affected by seemingly small variations in the profile of the barrier. The approach geometrics of the roadside and the friction coefficient of the barrier may also play important roles in the propensity for rollover.
- The concrete safety-shaped barrier was not designed for impacts involving large trucks, intercity buses, or school buses; such impacts frequently result in rollovers.

This paper presents the results of a study sponsored by FHWA and conducted at the Texas Transportation Institute (TTI) that examined the issue of rollovers caused by concrete safety-shaped barriers (*1*). The study objectives were to determine the extent and severity of overturn collisions with concrete safety-shaped barriers, identify the causes of rollover of vehicles in impacts with concrete safety-shaped barriers, and identify potential countermeasures to reduce concrete safety-shaped barrier rollovers.

RESEARCH APPROACH

The research approach for the study consisted of three major activities: literature review, accident studies, and simulation studies.

Literature Review

Available literature relating to rollover accidents on concrete safety-shaped barriers as well as rollover and small car safety in general was critically reviewed to obtain insights into the problem being studied. In general, a relatively large number of potential information sources relating to concrete safety-shaped barriers and rollover accidents were identified through the literature search. However, many of the references reviewed were found to contain little information useful to this study.

Accident Studies

A number of available accident data files were considered for use in the accident studies. The following three data files were eventually selected for use in the analyses:

- The Texas barrier accident data file,
- The Texas concrete median barrier (CMB) accident data file, and
- The National Accident Sampling System (NASS) Longitudinal Barrier Special Study (LBSS) data file.

Brief descriptions of these accident data files are provided below.

Texas Barrier Accident Data File

This data file contained all police-reported longitudinal barrier accidents on urban Interstates and freeways in Texas for the 3-year period 1982 to 1984 (more than 16,331 barrier accidents, 6,728 of which involved median barriers). This data file was used in the preliminary analysis and limited to general descriptive statistics.

The limited use of this data file was the result of a number of problems identified in the preliminary analysis and a manual check using printed copies of police accident reports for a sample of highway sections. First, concrete safety-shaped barriers were not specifically identified in the accident reports, nor were the locations of these barriers available from any computerized data file. The manual check found that less than half of the CMB accidents were correctly identified in the computerized data file. Second, rollover was not specifically identified in the accident reports. Damage to the top of the vehicle was initially used as a surrogate for rollover, but the manual check found that less than half of the rollover accidents were correctly identified using this approach.

Texas CMB Accident Data File

Because of the problems with computerized accident data files discussed, a second data file was created using a manual process. First, the locations of concrete median barriers were identified through contacts with the major urban districts of the Texas State Department of Highways and Public Transportation (SDHPT). The location information on the CMBs was then computerized and merged with the Texas barrier accident data file. Of the total 6,870 median barrier accidents on urban Interstates and freeways, 1,964 were identified as involving CMBs through this location-matching process.

Printed copies of police accident reports on these CMB accidents were obtained from the Texas Department of Public Safety. The police accident reports were reviewed manually to verify barrier type and rollover involvement. Also, supplemental data that were not available in the computerized accident data file, but that might be gleaned from manual review of the police accident reports, were coded from the reports. The supplemental data included indications as to whether the impact was with or near the end of the median barrier, the impact sequence, and whether the vehicle was spinning or skidding sideways before impact with the concrete median barrier.

The supplemental data were then entered into the computer and merged with the accident data file. Of the 1,964 accidents in the data file, 125 were eliminated for various reasons, such as accidents not involving concrete median barriers or other incorrect codes. The usable number of accidents in the Texas CMB accident data file was therefore 1,839.

The Texas CMB data file was based on police level accident data supplemented by manual review and coding of the accident reports. It did not contain any detailed information on impact conditions. The quality of the data was limited to that of the police accident reports. The Texas CMB data file was therefore used mainly for determining the extent of the rollover problem and for some limited analysis on the causative or contributory factors associated with rollover involvement.

NASS LBSS Data File

The NASS program is a continuing crash data collection effort sponsored by the National Highway Traffic Safety Administration (NHTSA). Teams of trained investigators, under contract to NHTSA, collected data on a statistical sample of accidents at selected locations throughout the nation. The LBSS study was sponsored by FHWA and conducted as a special study under the NASS program. NASS investigators were specifically trained for this data collection effort. The data collection forms and protocol were specifically designed for impacts involving longitudinal barriers. For these reasons, detailed information on impact conditions was collected.

A total of 130 NASS LBSS cases involving concrete safety-shaped barriers were identified for the years 1982 to 1984. The sample size is clearly too small for any form of statistical analysis. Thus, the analysis of the LBSS data file was mainly clinical in nature. Printed copies of these 130 LBSS cases were first reviewed for accuracy and corrected, as appropriate. The accidents were then reconstructed to estimate impact speed using a simplified reconstruction procedure developed specifically for impacts involving concrete safety-shaped barriers.

A total of 31 rollover accident cases were identified from the 130 NASS LBSS cases. After further review, 9 of the 31 cases were excluded from the analysis, including 6 cases in which the rollovers were not related to the barriers and 3 cases involving tractor-trailers. The remaining 22 cases were then clinically analyzed to determine potential causative factors and conditions contributing to the vehicle rollovers.

Simulation Studies

A version of the HVOSM-RD2 program modified specifically for use with rigid barrier impacts was used for the simulation study. Most of the original modifications were accomplished under NCHRP Project 22-6, whereas some of the refinements to handle unusual impact conditions were accomplished under this study (*1,2*). Modifications to the simulation program included improvements to the sheet metal–barrier interaction model, the suspension damping model, and the tire normal force model. The modified program was validated extensively using data from available crash test results. Because of limitations associated with the program's thin disk tire model, HVOSM could not be adequately validated for very low angle impacts (i.e., 5 degrees or less). Although this limitation restricted the use of the program for simulating some impacts of interest to this study, HVOSM is believed to be the best available tool for analyzing rigid barrier impacts.

The modified simulation model was used to evaluate the potential for concrete safety-shaped barriers to cause vehicle rollovers and to assess potential barrier improvements to minimize the identified rollover problems. The simulation effort was divided into three parts: a baseline evaluation of the concrete safety-shaped barrier, an evaluation of contributory factors identified in the accident analysis, and a study of potential countermeasures to minimize the rollover problem.

Baseline Simulations

The first step in the simulation effort involved simulation of 27 impact conditions that were believed to be representative of a majority of concrete barrier impacts. Results of these simulations provided a basis for comparing the existing shape with any potential modifications.

Simulation of Contributory Factors

Factors identified from accident analysis as causative or contributory to vehicle rollover during impacts with concrete safety-shaped barriers were verified with simulation. The factors evaluated included impact conditions that might increase the propensity for vehicle rollovers, such as impact speed and angle and vehicle orientation. These impact conditions were simulated for a variety of vehicle sizes to better understand the nature of concrete barrier impacts, especially those impact conditions resulting in rollovers.

Simulation of Potential Countermeasures

After analyzing the accident data and the simulation efforts, countermeasures to reduce the significance of the rollover problem were identified. This phase of the simulation effort evaluated the effectiveness of each of these potential countermeasures. All impact conditions identified as potential contributors to vehicle rollover under the second phase of the simulation effort were simulated with each proposed countermeasure.

The effectiveness of each countermeasure was then evaluated by the proportion of rollover conditions that were eliminated. All baseline simulation runs were then conducted for the best countermeasure. Comparisons between the baseline runs on the standard concrete safety-shaped barrier and the best countermeasure were then conducted to assess changes, if any, in measures of the potential for occupant injury and vehicle damage, such as lateral acceleration levels and extent of vehicle crush.

CONCLUSIONS AND RECOMMENDATIONS

Highlights of the major findings and conclusions of the study are summarized and discussed together with recommendations.

Findings and Conclusions

Extent of Rollover Problem

Analysis of the Texas CMB file indicated that rollover occurred in 8.5 percent of the accidents involoving concrete safety-shaped barriers. This is somewhat lower than the rollover rate reported previously [e.g., California reported a rollover rate of 9.9 percent (*3*; K. Sides, unpublished data)]. However, much of the difference could be attributed to the difference in the proportion of smaller cars between Texas and California.

The severity of rollover accidents was much higher than that of nonrollover accidents, as indicated in Table 1, based on the Texas CMB data file. The percentage of drivers sustaining some form of injury in rollover CMB accidents was 68.8 percent compared with only 40.5 percent for nonrollover CMB accidents. Differences were more pronounced for more severe injuries. For incapacitating injuries, the percentages were 11.5 percent for rollover CMB accidents and only 6.0 percent for nonrollover CMB accidents. The driver fatality rate for nonrollover CMB accidents was only 0.1 percent; that for rollover CMB accidents was 1.3 percent. Similar results were found when the highest injury sustained in an accident was considered instead of driver injury.

TABLE 1 INJURY SEVERITY BY ROLLOVER INVOLVEMENT (TEXAS CMB DATA FILE)

	Driver Injury				Highest Injury			
	Nonrollover		Rollover		Nonrollover		Rollover	
Injury Severity	No.	%	No.	%	No.	%	No.	%
No Injury	988	59.5	49	31.2	890	53.0	44	28.0
Possible Injury	182	10.8	18	11.5	209	12.5	19	12.1
Nonincapacitating Injury	406	24.2	70	44.6	456	27.2	71	45.2
Incapacitating Injury	100	6.0	18	11.5	115	6.9	21	13.4
Fatal	2	0.1	2	1.3	8	0.5	2	1.3
Total	1678	100.0	157	100.0	1678	100.0	157	100.0

In the analysis of NASS LBSS accident cases, it was found that 6 of the 31 rollover accidents (19.4 percent) were not related to the barrier itself and would have occurred regardless of the barrier type. Because the LBSS accident cases were not sampled on a representative basis, it is not possible to determine the proportion of rollovers involving concrete safety-shaped barriers that are not attributable to the barrier. However, it is evident that the proportion of the rollover problem for concrete safety-shaped barriers treatable by countermeasures is less than the 8.5 percent indicated.

Though the extent of the rollover problem was found to be less than previously reported, this does not mean that rollover is not a problem for concrete safety-shaped barriers, but only that the problem is less extensive than anticipated. Given the severe nature of rollover accidents, efforts to identify potential improvements to the concrete safety-shaped barrier to reduce the propensity for rollover should continue.

Causative or Contributory Factors

Police level accident data, even with manual review of printed copies of the police accident reports, are not detailed enough to identify factors that cause or contribute to rollovers on concrete safety-shaped barriers. Nonetheless, analysis of the Texas CMB data file identified several factors that are correlated with rollover involvement.

- The rollover rate was found to be lower under adverse weather and surface conditions, as indicated in Table 2. This might be attributed to the lower coeffcient of friction, which would reduce the buildup of large side forces for tripping vehicles, under wet or snowy and icy surface conditions. Reduced operating speeds associated with adverse weather conditions could also contribute to the lower rollover rate.
- The rollover rate was found to be lower for vehicles that skid or rotate before impact with the barrier, as indicated in Table 3. Review of the NASS LBSS accident cases confirmed this finding.
- There is a definite relationship between vehicle size and weight and rollover involvement, as illustrated in Figure 1. The rollover rate of lighter vehicles was much higher than that of their heavier counterparts. This problem is inherent in the nature of small vehicles because of their narrow track width and low roll and yaw moments of inertia. However, these basic problems with small vehicles could be further aggravated by the shape of the concrete safety-shaped barrier.

Analysis of the NASS LBSS accident cases provided much more information and insight into potential causative or contributory factors for rollover despite the small sample size. Three impact conditions were identified as potential factors. The descriptors used to define the impact conditions are in accordance with vehicle simulation conventions and are as follows:

1. A vehicle is *tracking* when the vehicle heading and the velocity vector of the vehicle are the same.
2. A vehicle is *yawing* when the vehicle heading is different from that of the velocity vector.
3. The angle between the vehicle heading and the barrier, expressed in degrees, is the *yaw angle*.
4. The rate at which the vehicle heading angle is changing, expressed in degrees per second, is the *yaw rate*.
5. The angle between the vehicle heading angle and its velocity vector, expressed in degrees, is the *slip angle*.
6. The angle between a vehicle's velocity vector and the longitudinal axis of the barrier at the point of initial contact with the barrier, expressed in degrees, is the *impact angle*.
7. The velocity of the vehicle at the point of initial contact with the barrier is the *impact speed*.

TABLE 2 ROLLOVER INVOLVEMENT BY SURFACE CONDITION (TEXAS CMB DATA FILE)

Surface Condition	Total Accidents No.	Total Accidents %	Rollover Involvement No.	Rollover Involvement %
Dry	1226	66.7	139	11.3
Wet	573	31.2	17	3.0
Snowy/Icy	40	2.2	1	2.5
Total	1839	100.0	157	8.5

TABLE 3 ROLLOVER INVOLVEMENT BY VEHICLE ATTITUDE (TEXAS CMB DATA FILE)

Vehicle Attitude	Total Accidents No.	Total Accidents %	Rollover Involvement No.	Rollover Involvement %
Skidding Sideways/ Rotating	683	37.1	37	5.4
Tracking	965	52.5	101	10.5
Unknown/Unsure	191	10.4	19	10.0
Total	1839	100.0	157	8.5

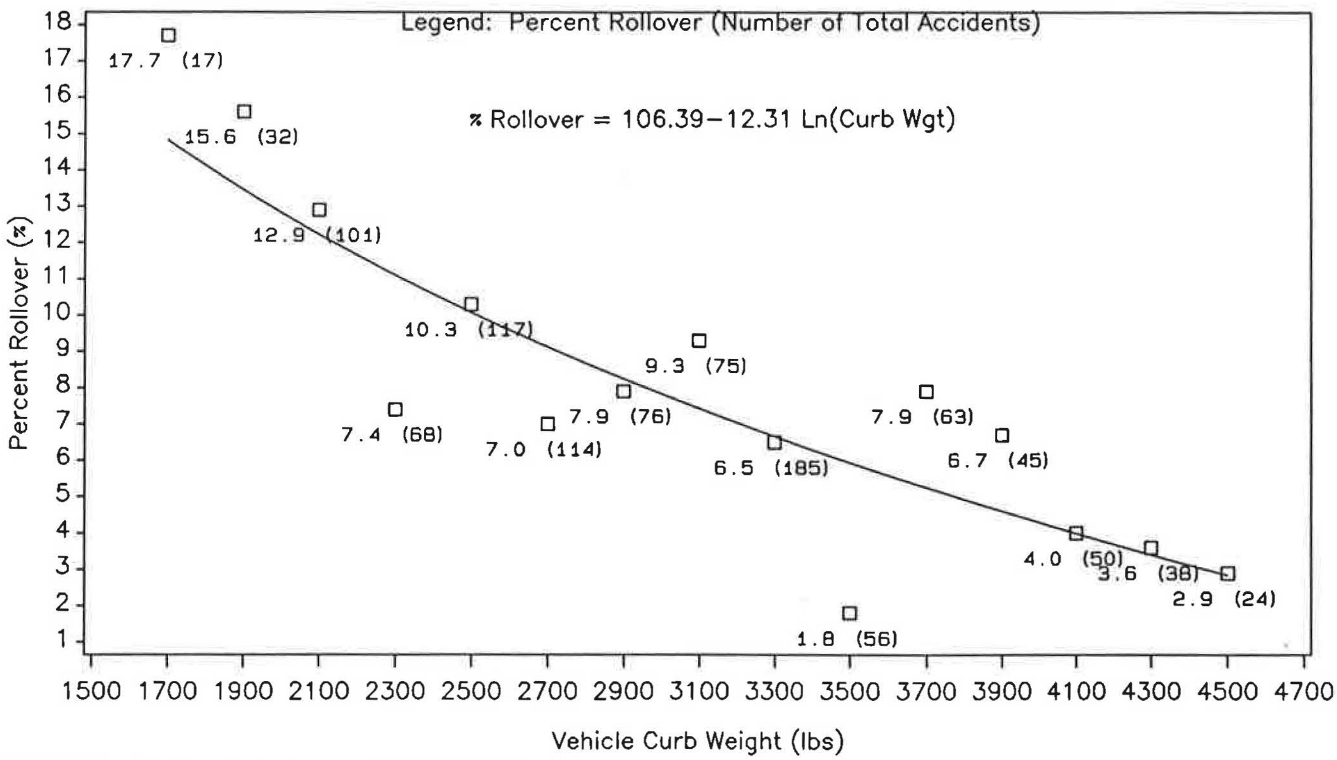


FIGURE 1 Relationship between vehicle curb weight and rollover rate.

The three impact conditions were as follows, where "moderate" impact speed means 25 to 50 mph and "high" impact speed means more than 50 mph:

- High impact angle (at least 25 degrees) and moderate to high impact speed;
- High slip angle (at least 30 degrees), low to moderate yaw rate, and moderate to high impact speed [vehicles that were rotating at impact (i.e., with a high yaw rate) were found to be less likely to result in rollovers]; and
- High impact speed and low impact angle (not more than 10 degrees) for vehicles in a tracking mode (i.e., slip angle not more than 15 degrees).

Table 4 shows a comparison between rollover and nonrollover accidents on these three impact conditions. Eight (36.3 percent) of the 22 rollover accidents involved high impact angles compared with only 10.3 percent for nonrollover accidents. The vehicle would typically climb the lower sloped face of the barrier and continue to climb the upper sloped face of the safety shape without any significant redirection. This would cause the vehicle to attain a high roll angle away from the barrier as the vehicle began to redirect and separate from the barrier, leading to rollover.

This finding is consistent with the results of a full-scale crash test of an 1,800-lb Honda Civic that struck a safety-shaped barrier at 27 mph and 52 degrees and subsequently rolled over (*4*). However, another test with a 3,600-lb full-size passenger car impacting the barrier at 40 mph and 45 degrees did not result in rollover (*5*). These are the only two crash tests available with such high impact angles. The normal impact angles used for crash testing are 15 to 25 degrees, substantially lower than some of the impact angles observed in these accidents.

Four (18.2 percent) of the 22 rollover accidents involved vehicles yawing into the barriers with high slip angles at moderate to high impact speeds. In comparison, 20 (34.5 percent) of the 58 nonrollover accidents had similar impact conditions but did not result in rollovers. The major difference observed between the rollover and the nonrollover accidents under these impact conditions pertained to the yaw rate or the rate at which the vehicle was rotating or spinning.

For the rollover accidents, the yaw rates were usually low to moderate and the vehicles principally skidded sideways. The vehicle would roll slightly into the skid as it struck the barrier. The roll angle would continue to increase as the vehicle crashed into the barrier, leading to rollover. On the other hand, review of nonrollover accidents indicated that most of the vehicles principally rotated with high yaw rates as the vehicles struck the barriers. The vehicle would typically continue to rotate after the initial impact with the barrier and then strike the barrier a second time with the rear corner. The roll angle of the vehicle was usually fairly small and the second impact would generally stabilize the trajectory of the vehicle as it separated from the barrier, thus preventing rollovers.

As discussed previously, results from the analysis of the Texas CMB accident data file indicated that the vehicle skidding sideways or rotating prior to impact with the barrier was a fairly common impact condition, composing 37 percent of the accidents involving concrete safety-shaped barriers. Further, vehicles skidding or rotating at impact were found to have lower rollover rates than tracking vehicles. This suggests

TABLE 4 ROLLOVER AND NONROLLOVER ACCIDENTS BY IMPACT CONDITION (NASS LBSS DATA FILE)

Impact Condition	Rollover No.	Rollover %	Nonrollover* No.	Nonrollover* %
1	8	36.3	6	10.3
2	4	18.2	20	34.5
1 and 2	1	4.5	5	8.6
3	5	22.7	1	1.7
Other	4	18.2	26	44.8
Total	22	100.0	58	100.0

* Only 58 of the 99 nonrollover accident cases have all three data elements (i.e., impact speed, impact angle, and slip angle) available.

Impact Condition	Description
1	High impact angle (>= 25 degrees) and moderate (25-50 mph) to high (> 50 mph) impact speed.
2	High slip angle (>= 30 degrees), low to moderate yaw rate and moderate (25-50 mph) to high (> 50 mph) impact speed.
3	High impact speed (> 50 mph) and low impact angle (<= 10 degrees) for vehicles in a tracking mode (i.e., slip angle <= 15 degrees).

that only a small proportion of the vehicles were skidding sideways at impact (i.e., had high slip angles and low yaw rates) and that most of the vehicles were rotating at impact (i.e., had high yaw rates).

Five (22.7 percent) of the 22 rollover accidents involved vehicles striking the barriers in a tracking mode at high impact speeds and low impact angles, compared with only 1.7 percent of the nonrollover accidents. The vehicle would typically quickly climb to the top of the lower sloped face of the safety shape and then slowly climb the upper sloped face. Because of the high impact speeds, the vehicle would climb higher and stay on the barrier longer than normal. The vehicle would eventually roll away from the barrier as it separated from the barrier.

Concrete glare screens were found on top of the concrete safety-shaped barrier in two of the high-speed, low-angle rollover accidents. It appeared that the glare screen would act as an extension to the top of the safety-shaped barrier, thereby causing the vehicle to climb higher on the barrier than without the glare screen. This allowed the roll angle on the vehicle to go higher than normal, leading to rollover.

In some of the rollover accidents, the vehicles separated from the barriers in a relatively stable fashion and then began to rotate after separation and subsequently rolled over. These rotations were probably the result of driver braking and steering inputs or damage to the front suspension from impact with the barrier or a combination of these factors. It is arguable whether the subsequent rollover was related to the shape of the barrier.

Lateral displacement of the barrier segments was found in one rollover accident. Crash tests have shown that lateral barrier displacement during impact increases the time that a vehicle is in contact with the lower curb surface and reduces the slopes of all surfaces as the barrier leans away from the vehicle. As a result, the vehicle climbs higher on the barrier and the propensity for rollover is increased. Lateral displacement of the barrier is usually not a problem for permanent barrier installations, but is certainly an area of concern for temporary installations, such as construction zones.

The majority of the rollover accidents in the NASS-LBSS file occurred under dry surface conditions. This is consistent with accident analysis results, which indicated that the propensity for rollover after impact with a concrete safety-shaped barrier was actually lower under a wet or snowy and icy surface condition than under a dry surface condition. The reduced coefficient of friction under a wet or snowy and icy surface condition might have prevented critical side forces from building up and tripping the vehicle. Lower operating speeds typical of adverse surface conditions might also have contributed to the reduced incidence of rollover.

Figure 2 compares impact speed in rollover and nonrollover accidents. It is evident from the figure that rollover accidents were associated with much higher impact speeds than nonrollover accidents. None of the rollover accidents had an impact speed of less than 25 mph, compared with 30 percent of the nonrollover accidents. On the other hand, 73 percent of the rollover accidents had impact speeds of over 50 mph compared with only 14 percent of the nonrollover accidents.

Smaller and lighter vehicles were found to be disproportionately involved in rollovers, as illustrated in Figure 3, where the cumulative distributions of vehicle curb weights for rollover and nonrollover accidents are shown. The median (50th percentile) vehicle curb weight for rollover accidents was 2,500 lb, whereas that for nonrollover accidents was 3,150 lb. It is interesting to note that the weight of the vehicle appears to have less of an effect on rollovers in high-angle impacts with a higher median vehicle curb weight of 2,700 lb.

Some of the characteristics identified in previous studies as affecting the propensity for rollover (e.g., height of reveal and lower curb face, slope and offset of upper face, barrier

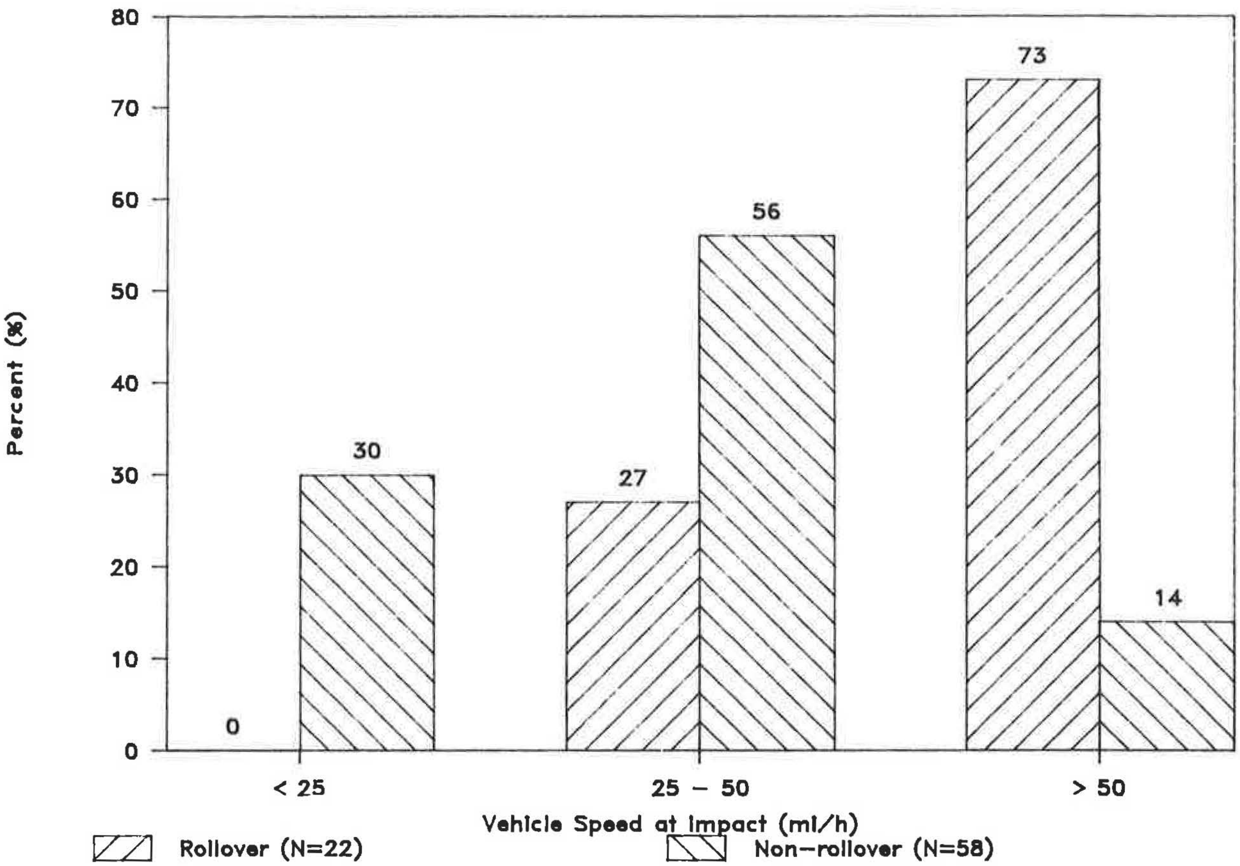


FIGURE 2 Comparison of impact speed for rollover and nonrollover accidents.

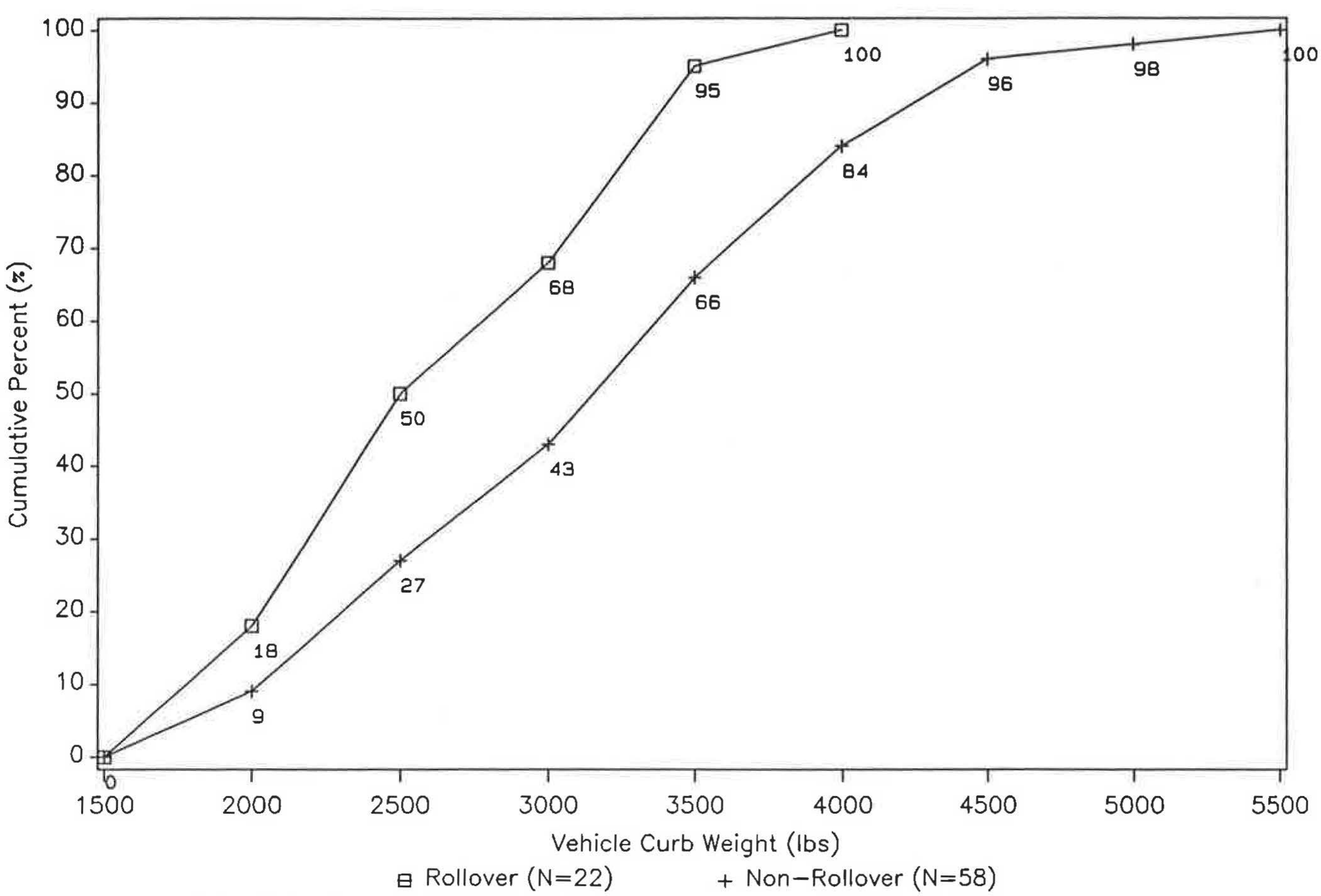


FIGURE 3 Cumulative distributions of vehicle curb weights for rollover and nonrollover accidents.

surface friction, and approach terrain) did not appear to play a part in any of the rollover accident cases studied. This finding apparently reflects the lack of variation in barrier shape and dimensions that would allow their effects to be assessed. Further, all barriers involved in rollover accidents had flat approach terrain and only one had an unpaved shoulder. Consequently, the effects of approach terrain on the propensity for rollover could not be properly assessed.

On the basis of the results of the clinical analysis, the following four factors, or conditions, were selected for further evaluation in the simulation studies:

- High-angle impacts with moderate to high impact speeds;
- Impacts with high slip angles, low yaw rates, and moderate to high impact speeds;
- Impacts with safety-shaped concrete barriers with glare screens; and
- Low-angle impacts with high impact speeds.

As discussed previously, HVOSM was not well validated for very low impact angles. Thus, the final impact condition selected for evaluation in this study, low-angle and high-speed impacts, could not be included in the simulation effort. These limitations notwithstanding, the simulation results generally supported findings from the accident studies, as described below.

The significance of vehicle rollover during high-angle impacts was investigated by conducting 12 HVOSM simulations with each of three classes of vehicles—1,800 lb, 3,800 lb, and 4,500 lb. The 12 combinations of impact speed and impact angle are listed in the first two columns of Table 5. The simulation results indicated that only small cars were significantly susceptible to rollover during high-angle impacts. Rollovers for mini-size vehicles were predicted even for some moderate-speed impacts.

Impacts with high slip angles and low yaw rates were evaluated through the simulation of barrier accidents involving yaw angles ranging from 45 to 75 degrees with a yaw rate of 15 degrees/sec. The 18 combinations of impact speed, impact angle, and yaw angle are listed in the first three columns of Table 6. HVOSM simulations of run-off-road accidents has indicated that most automobiles can attain yaw rates in excess of 45 degrees/sec during steering maneuvers. Thus, the 15-degree/sec yaw rate was chosen as representative of a relatively low yaw rate for a nontracking vehicle.

HVOSM simulation of impacts with safety-shaped barriers with glare screens was limited to moderate-angle impacts as a result of the aforementioned limitations of the program's tire model. The program predicted that glare screens did not significantly destabilize vehicles during impacts at speeds ranging from 30 to 60 mph and angles ranging from 7 to 25 degrees. On the basis of these simulation findings, there is no reason to believe that glare screens adversely affect the performance of concrete safety-shaped barriers under normal crash test conditions. However, the question of the effects of a glare screen for low-angle impacts remains unanswered.

The simulation of concrete safety-shaped barrier impacts involving unusual impact conditions did support findings from the accident data analysis described previously. However, safety-shaped barriers performed relatively well for the majority of impact conditions (moderate-angle, tracking impacts).

Potential Countermeasures

The extent of the rollover problem on concrete safety-shaped barriers is not considered serious enough to warrant retrofitting of existing barriers. Therefore, only potential countermeasures that are applicable to new construction were included in the evaluation. This does not mean that rollover is not a problem for concrete safety-shaped barriers; rather it is believed that retrofitting of existing barriers would not be cost-effective.

Three alternative shapes were selected for evaluation as potential countermeasures to reduce rollover rates: F-shape, constant slope, and vertical wall. The F-shape uses the basic safety-shape configuration with a smaller lower curb face, whereas the constant sloped barrier consists of a single, near-vertical face. Each of these alternate shapes was evaluated through simulation of impact conditions that were identified as potential contributors to rollover for the standard concrete safety-shaped barrier. Results of the evaluation are summa-

TABLE 5 SUMMARY OF HVOSM SIMULATIONS OF IMPACTS WITH MINI-SIZE VEHICLES AT HIGH ANGLES

Impact Speed (mph)	Impact Angle (deg)	Predicted Maximum Roll Angle (deg) Concrete Safety Shaped Barrier	F Shaped Barrier	Constant Sloped Barrier	Vertical Wall
30	35	35	15	14	27
30	45	58	24	53	6
30	60	N/A	> 90	35	8
30	75	N/A	56	15	N/A
45	35	30	23	32	10
45	45	> 90	33	28	17
45	60	> 90	> 90	13	> 90
45	75	N/A	31	15	N/A
60	35	36	> 90	7	27
60	45	> 90	> 90	> 90	54
60	60	> 90	> 90	24	> 90
60	75	N/A	50	13	> 90

TABLE 6 SUMMARY OF HVOSM SIMULATIONS OF IMPACTS WITH MINI-SIZE VEHICLES AT HIGH SLIP ANGLES AND LOW YAW RATES

Impact Speed (mph)	Impact Angle (deg)	Yaw Angle (deg)	Predicted Maximum Roll Angle (deg) Concrete Safety Shaped Barrier	F Shaped Barrier	Constant Sloped Barrier	Vertical Wall
30	15	45	> 90	> 90	> 90	27
30	15	60	> 90	> 90	53	6
30	15	75	25	> 90	49	8
45	15	45	> 90	> 90	> 90	N/A
45	15	60	> 90	> 90	> 90	10
45	15	75	> 90	> 90	> 90	17
60	15	45	> 90	> 90	> 90	> 90
60	15	60	> 90	> 90	56	N/A
60	15	75	> 90	> 90	45	27
30	25	45	> 90	> 90	> 90	54
30	25	60	> 90	> 90	35	> 90
30	25	75	> 90	18	25	> 90
45	25	45	> 90	68	> 90	N/A
45	25	60	> 90	> 90	> 90	10
45	25	75	> 90	> 90	68	17
60	25	45	> 90	45	> 90	> 90
60	25	60	> 90	> 90	12	N/A
60	25	75	> 90	> 90	31	27

TABLE 7 SUMMARY OF HVOSM SIMULATIONS OF IMPACTS WITH MID-SIZE VEHICLES AT HIGH SLIP ANGLES AND LOW YAW RATES

Impact Speed (mph)	Impact Angle (deg)	Yaw Angle (deg)	Predicted Maximum Roll Angle (deg) Concrete Safety Shaped Barrier	F Shaped Barrier	Constant Sloped Barrier	Vertical Wall
30	15	45	10	9	14	27
30	15	60	9	5	53	6
30	15	75	6	6	35	8
45	15	45	16	11	15	N/A
45	15	60	11	6	32	10
45	15	75	6	6	28	17
60	15	45	20	17	13	> 90
60	15	60	> 90	11	15	N/A
60	15	75	7	5	7	27
30	25	45	16	12	> 90	54
30	25	60	10	5	24	> 90
30	25	75	5	6	13	> 90
45	25	45	20	17	15	N/A
45	25	60	> 90	24	32	10
45	25	75	6	5	28	17
60	25	45	24	19	13	> 90
60	25	60	> 90	> 90	15	N/A
60	25	75	10	6	7	27

TABLE 8 SUMMARY OF HVOSM SIMULATIONS OF IMPACTS WITH FULL-SIZE VEHICLES AT HIGH SLIP ANGLES AND LOW YAW RATES

Impact Speed (mph)	Impact Angle (deg)	Yaw Angle (deg)	Predicted Maximum Roll Angle (deg) Concrete Safety Shaped Barrier	F Shaped Barrier	Constant Sloped Barrier	Vertical Wall
30	15	45	12	10	14	27
30	15	60	6	6	53	6
30	15	75	6	7	35	8
45	15	45	19	16	15	N/A
45	15	60	8	5	32	10
45	15	75	7	7	28	17
60	15	45	20	18	13	> 90
60	15	60	22	16	15	N/A
60	15	75	7	7	7	27
30	25	45	18	15	> 90	54
30	25	60	8	5	24	> 90
30	25	75	6	6	13	> 90
45	25	45	20	16	15	N/A
45	25	60	36	19	32	10
45	25	75	7	6	28	17
60	25	45	23	18	13	> 90
60	25	60	63	61	15	N/A
60	25	75	18	8	7	27

rized in Tables 5 through 8. General findings from this simulation effort are as follows.

- The F-shaped barrier offers little performance improvement over the concrete safety-shaped barrier for these impact conditions.
- The constant sloped barrier with an 80-degree slope offers some rollover reductions while slightly increasing lateral vehicle accelerations.
- The vertical wall barrier offers the greatest reduction in rollover potential, but with the greatest increase in lateral accelerations.

Baseline runs were repeated with the vertical wall barrier to generate a basis for comparing its performance with the concrete safety-shaped barrier under the more common impact conditions. As expected, the vertical wall barrier has lower maximum roll angles and climb heights, but also higher lateral accelerations than the standard concrete safety-shaped barrier under these impact conditions. A comparison of the baseline simulations for the concrete safety-shaped and vertical wall barriers is presented in Table 9.

Discussion and Recommendations

Although the vertical wall barrier shows the best potential for reducing the propensity for rollover, it may not be the shape of choice for rigid barriers when all factors are taken into consideration. The propensity for rollover needs to be balanced against factors such as damage to vehicles and potential for injuries to the vehicle occupants, as well as operational factors such as cost and maintenance requirements.

The constant sloped barrier may provide the best compromise solution. It reduces the propensity for rollover compared with the standard safety-shaped barrier and shows less increase in the lateral accelerations, a surrogate for injury potential during nonrollover accidents, than the vertical wall barrier. Construction costs for the constant slope barrier should be only slightly higher than the standard safety-shaped barrier, but the shape can substantially reduce life cycle costs.

In order to maintain safety barrier shape and height during resurfacing operations, the pavement surface has to be planed down before any overlay can be applied. Pavement planing is a costly procedure, and several pavement overlays are normally required during the life of a concrete barrier. On the other hand, a constant sloped barrier can be built to a greater height initially, thereby eliminating the need for removal of the old pavement surface. For example, a 42-in. constant sloped barrier would allow up to 10 in. of overlay before being reduced to the height of a standard 32-in. safety-shaped barrier. These overlay operations would not affect the shape or the minimum height of the constant sloped barrier. A study to develop such a barrier for the Texas SDHPT was recently completed (*6*). Construction bids for constant sloped barriers were not significantly higher than those for safety-shaped barriers. Thus, the reduced costs of pavement overlays associated with the constant sloped barrier should be much greater than the increase in construction costs.

However, to properly compare the overall effectiveness of various barrier shapes, a benefit/cost analysis taking into account all the various factors is needed. The computer simulation runs discussed should provide a basis for determining the relative severity of impact with these barriers for any impact condition. In support of such a benefit/cost analysis, additional research is needed to better identify the distributions

TABLE 9 SUMMARY OF HVOSM SIMULATIONS OF BASELINE IMPACTS

Vehicle Weight (lb)	Impact Speed (mph)	Impact Angle (deg)	Max. 50 ms. Lat. Acc. (g)		Height of Climb (ft.)	
			Concrete Safety Shaped Barrier	Vertical Wall	Concrete Safety Shaped Barrier	Vertical Wall
1800	30	15	2.4	2.9	0.9	0.0
1800	45	15	4.2	5.3	1.3	0.0
1800	60	15	6.5	7.5	1.6	0.1
1800	30	25	4.6	5.3	1.2	0.0
1800	45	25	8.9	9.0	1.7	0.1
1800	60	25	13.3	12.4	2.1	0.5
3800	30	15	1.0	2.2	0.3	0.0
3800	45	15	1.6	N/A	0.7	0.0
3800	60	15	2.3	6.4	1.1	0.0
3800	30	25	2.6	3.9	0.5	0.1
3800	45	25	4.2	6.6	1.1	0.0
3800	60	25	6.0	9.5	1.5	0.1
4500	30	15	1.1	2.2	0.3	0.0
4500	45	15	1.7	4.3	0.7	0.0
4500	60	15	2.4	6.2	1.0	0.0
4500	30	25	2.6	4.0	0.5	0.0
4500	45	25	4.3	6.7	0.9	0.0
4500	60	25	6.1	9.7	1.1	0.1

of barrier impact conditions that can be expected along various highway types.

REFERENCES

1. K. K. Mak and D. L. Sicking. *Rollover Caused by Concrete Safety Shaped Barrier*, Vol. I—*Technical Report*. FHWA, U.S. Department of Transportation, Jan. 1989.
2. H. S. Perera. *Simulation of Vehicular Impacts with Safety-Shaped Barrier*. Ph.D. dissertation. Texas A&M University, College Station, 1987.
3. J. G. Viner. Implications of Small Cars on Roadside Safety. *Proc., 27th Annual Conference of the American Association for Automotive Medicine*, Arlington Heights, Ill., 1983, pp. 357–374.
4. J. Folsom, R. Stoughton, and S. Hawatky. *Effects on a Vehicle Impacting a Concrete Safety Shape Barrier at a Low Speed and a Large Angle*. Report CA/TL-86/02. California Department of Transportation, Sacramento, 1986.
5. J. Folsom, R. Stoughton, and D. Glauz. *A Seat Belt Efficacy Demonstration: A Large Angle Moderate Speed Impact into a Concrete Median Barrier*. Report CA/TL-87/06. California Department of Transportation, Sacramento, 1987.
6. W. L. Beason, H. E. Ross, H. S. Perera, W. Campise, and L. D. Bullard. *Development of a Single-Slope Concrete Median Barrier*. Report 9429CDK-1. Texas Transportation Institute, Texas A&M University System, College Station, 1989.

Development of an IBC MK-7 Barrier Capable of Restraining and Redirecting an 80,000-lb Tractor Van-Trailer

T. J. Hirsch and King K. Mak

This paper summarizes the results of an effort to develop an International Barrier Corporation (IBC) MK-7 barrier that can contain and redirect an 80,000-lb tractor van-trailer. After evaluation of various options, the approach of using portland cement–stabilized sand as the fill material was selected. The developmental effort included laboratory testing to determine the appropriate mix for the stabilized sand fill material, computer simulation to investigate the bending moments and shear strength required for the barrier to contain and redirect various vehicle types and weights, and full-scale crash testing of the smaller IBC MK-9 barrier with automobiles to obtain baseline data for use in developing the MK-7 truck barrier. A mix of 100 lb sand, 10 lb portland cement, and 10 lb water was selected for use as the fill material for the IBC MK-7 truck barrier. The barrier was struck by an 80,000-lb tractor van-trailer at 50.9 mph and 15.0 degrees. The tractor-trailer was contained and smoothly redirected by the barrier. The tractor-trailer rolled considerably toward the barrier during the impact sequence but remained upright with the bottom of the left side of the trailer sliding on top of the barrier. The vehicle was severely damaged, but the barrier sustained only minor damage. The results of the crash test indicate that the IBC MK-7 barrier with stabilized fill material meets the guidelines set forth in *NCHRP Report 230* for a high-performance truck barrier.

The results of an effort (*1,2*) undertaken at Texas Transportation Institute (TTI) to develop an International Barrier Corporation (IBC) MK-7 longitudinal barrier capable of restraining and redirecting an 80,000-lb tractor van-trailer are presented.

The standard IBC MK-7 barrier consists of modules with corrugated side panels attached to vertical bulkheads. Each module is 10.5 ft long, and a barrier installation may consist of any number of modules as required. The side panels and bulkheads are made of 14-gauge galvanized steel sheet metal. The overall cross-sectional dimensions of the barrier are 46 in. high and 44 in. wide. The barrier modules are filled with sand to the top of the barrier and covered with nonstructural 20-gauge galvanized sheet metal lids. The dimensions and details of the IBC MK-7 barrier are shown in Figure 1.

Previous crash tests of the standard IBC MK-7 barrier indicated that the existing barrier did not have the needed capability to restrain and redirect a heavy truck under standard test conditions. (There have been several impacts by large trucks on field installations of IBC MK-7 barriers, all of which resulted in containment of the vehicles without rollover or penetration of the barrier.) Various means of strengthening the barrier for the required loading from impact by a heavy truck were investigated. The approach eventually selected to strengthen the standard IBC MK-7 barrier was to stabilize the fill material by mixing the sand with portland cement and water. In this case, the basic fill material was a "pit run" siliceous sand available from a local quarry. By stabilizing the basic fill material, the strength of the barrier was increased 22 times over the original design with untreated fill material. This was accomplished by making the corrugated side panels work compositely. Another advantage of using stabilized fill material was to significantly reduce damage to the barrier from vehicular impacts.

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

STUDY APPROACH

The study effort consisted of four major activities:

1. Laboratory study,
2. Computer simulation study,
3. Crash testing of MK-9 barriers with and without stabilized fill using 4,500-lb full-size automobiles, and
4. Crash testing of MK-7 barriers with stabilized fill using an 80,000-lb tractor van-trailer.

Brief descriptions of these activities are presented as follows.

Laboratory Study

A limited laboratory study was conducted on the pit run siliceous sand fill material to determine the amount of cement and water to be added to the mixture to achieve the desired strength. A standard compaction test was first conducted on the untreated sand to determine the optimum moisture content for maximum unit weight. The test results indicated that the optimum moisture content for this material was about 10 percent.

Using this moisture content, a number of mixes and test cylinders were prepared using varying amounts of cement to determine how much cement was required to achieve the desired shear strength. Standard compression and split cylinder tests were conducted on the test cylinders for the various mixes to determine their compressive, tensile, and shear strengths as well as their modulus of elasticity.

The mix eventually selected for use with the barrier was 10 lb of cement and 10 lb of water for every 100 lb of sand. This yielded a "stabilized soil" with a dry unit weight of approx-

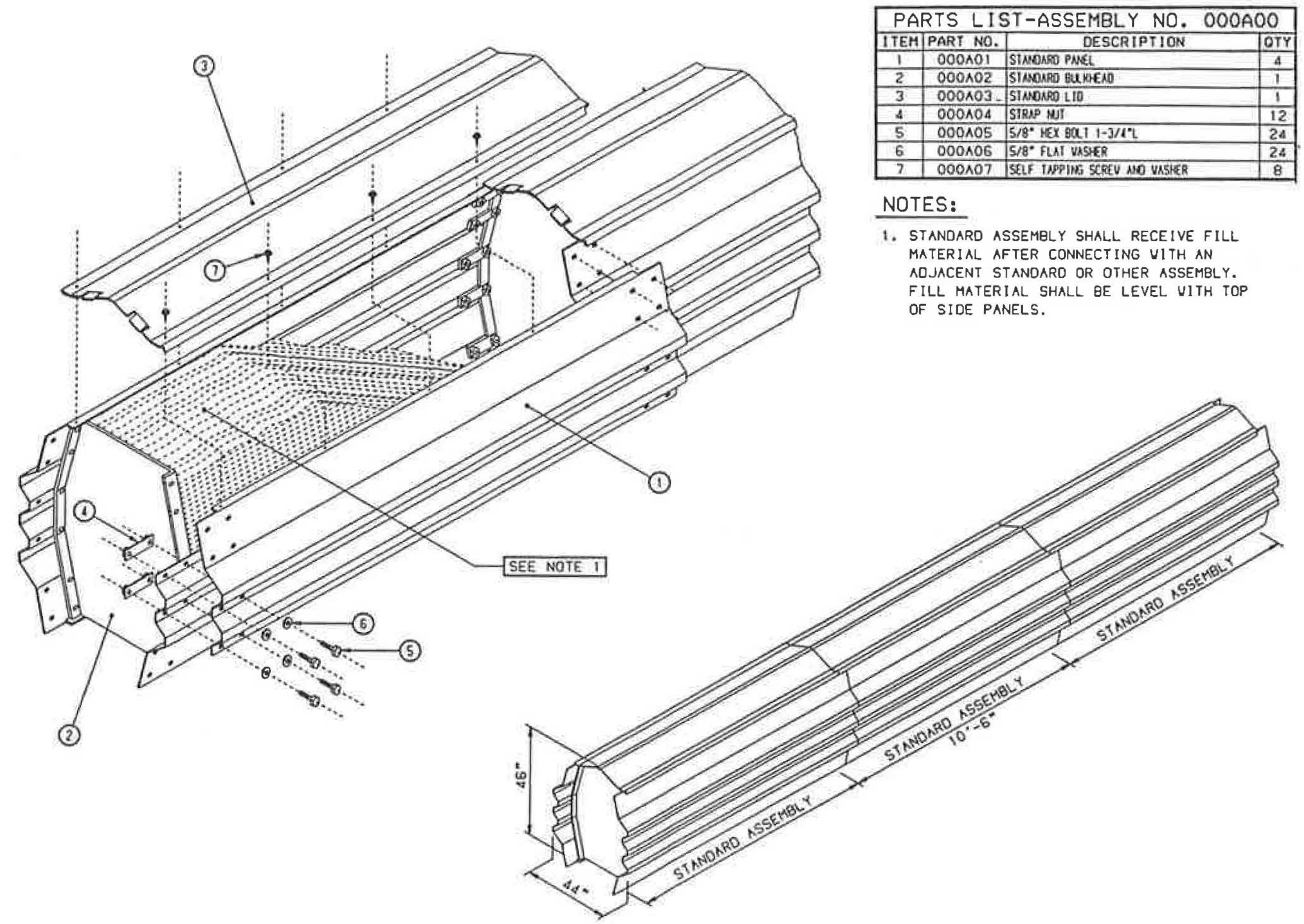


FIGURE 1 Details of MK-7 standard barrier assembly.

imately 110 lb/ft^3, a compressive strength of about 700 psi, a tensile strength of about 85 psi, a shear strength of about 117 psi, and a modulus of elasticity of about 1,000,000 psi.

Computer Simulation Study

To arrive at the barrier design, a TTI computer program called SABS (Simulation of Articulated Barrier Segments) was used to evaluate the structural behavior of the IBC MK-7 and MK-9 barriers as well as the various means of strengthening these barriers. The program indicated the magnitude of the bending moments and shear forces required to redirect various types of vehicles, from an 1,800-lb passenger car to an 80,000-lb tractor-trailer.

The SABS program was first calibrated using crash test data from previous crash tests conducted by or for IBC using 2,100-lb and 4,500-lb cars and a 20,000-lb school bus. The program was then used to investigate various methods of strengthening the barrier. The simulation results, along with a structural analysis of the barrier, indicated that the stabilized fill material needed to have a shear strength of approximately 85 psi in order for the IBC MK-7 barrier to redirect an 80,000-lb tractor-trailer.

Crash Tests of MK-9 Barriers with Automobiles

As part of the developmental effort, two full-scale crash tests were conducted on the smaller IBC MK-9 barrier with 4,500-lb full-size automobiles. The purpose of the IBC MK-9 barrier crash tests was to obtain baseline data for use in developing the IBC MK-7 truck barrier without incurring the high expenses of conducting multiple full-scale crash tests with tractor-trailers.

The standard IBC MK-9 barrier, like the IBC MK-7 barrier, consists of corrugated side panels attached to vertical bulkheads spaced 10.5 ft apart. The side panels and bulkheads are made of 14-gauge galvanized steel sheet metal. The overall dimensions of the barrier are 29.65 in. high (versus 46 in. for the MK-7 barrier) and 33 in. wide (versus 44 in. for the MK-7 barrier). The barrier is filled with sand to about 2 in. below the top of the barrier and covered with a nonstuctural lid made of 20-gauge galvanized steel sheet metal. The approximate weight of the barrier is 500 lb per lineal foot (versus 1,100 lb for the MK-7 barrier).

The test installations for both crash tests were identical except for the fill material. Each installation consisted of 18 bins (10.5 ft for each bin) of MK-9 barrier placed directly on top of a concrete pavement. The total length of the barrier was 189 ft. The first installation was filled with untreated sand and the second installation was filled with portland cement–stabilized sand, except for the bottom 2 to 3 in., which consisted of untreated sand to prevent the stabilized fill material from bonding with the pavement surface. The dimensions and details of the IBC MK-9 barrier are shown in Figure 2. Photographs of the test installation are shown in Figure 3.

The crash test and data analysis procedures were in accordance with guidelines presented in *NCHRP Report 230* (*3*). The test vehicles were instrumented with three rate transducers to measure roll, pitch, and yaw rates and a triaxial accelerometer near the vehicle center of gravity to measure acceleration levels.

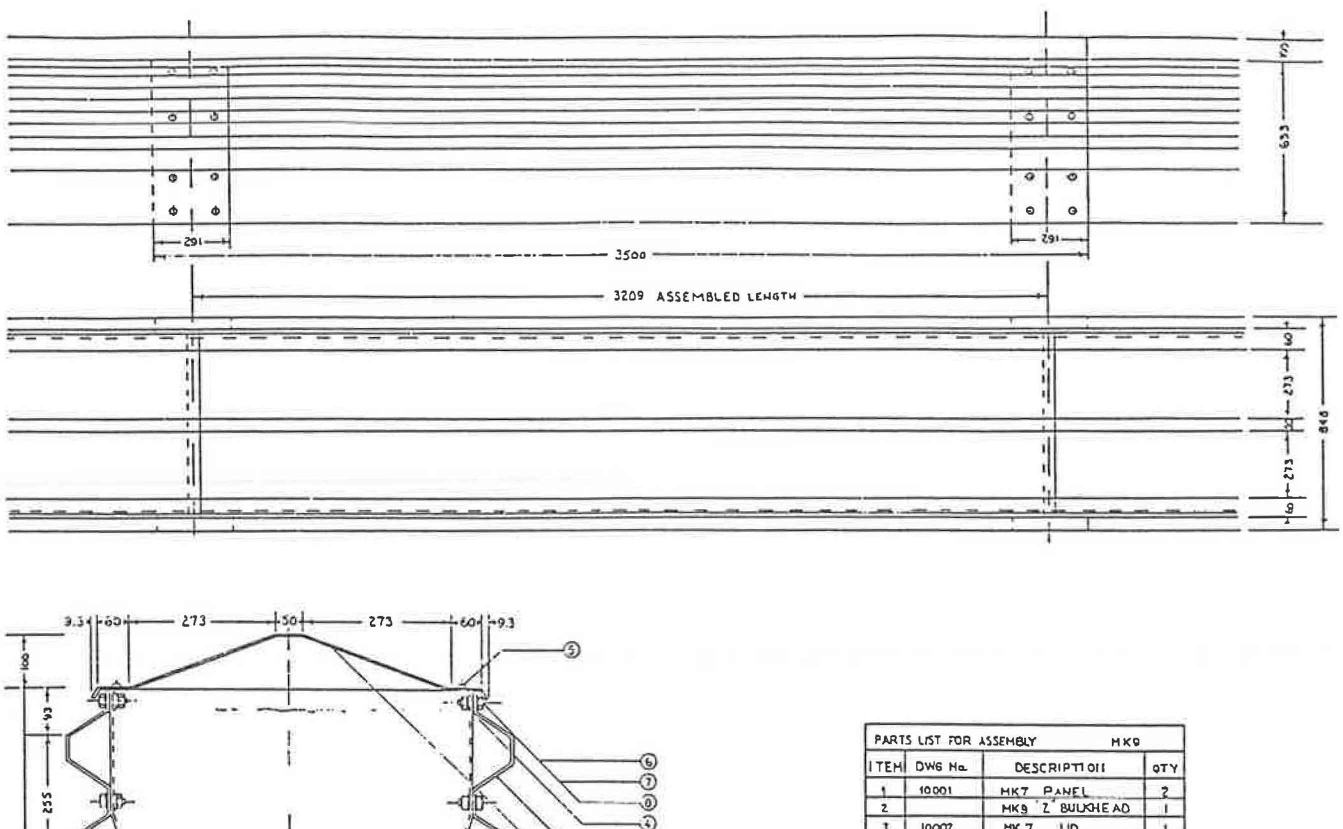
Test 1 (7091-2)

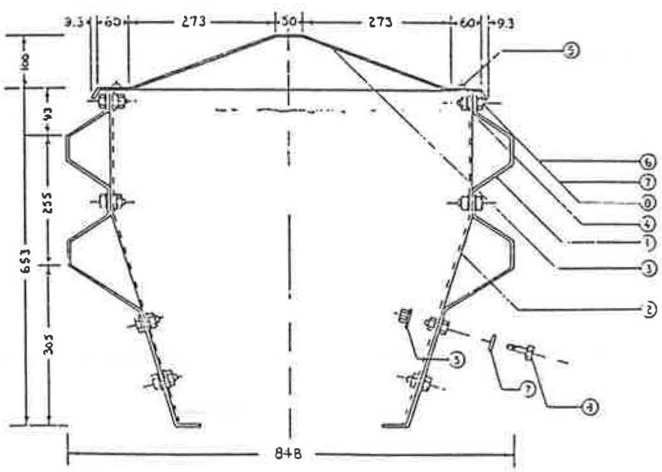
A 1981 Oldsmobile Ninety-Eight struck the standard IBC MK-9 barrier with untreated fill material at 61.96 mph and 25.2 degrees. The point of impact was the midpoint of the eighth bin, approximately 79.0 ft downstream from the beginning of the barrier. On impact, the vehicle began to ride up the face of the barrier. The right front wheel then climbed the barrier and the vehicle became completely airborne. The vehicle came down to the ground behind the barrier 15.0 ft from the point of initial impact. The brakes were then applied. The vehicle bounced and yawed counterclockwise and came to rest approximately 142.5 ft from the point of initial impact.

The barrier sustained severe damage, as shown in Figure 4. The permanent residual deformation was 29.0 in. laterally, located approximately 5 ft from the point of initial impact. The vehicle was in contact with the rail for 22.5 ft.

The vehicle sustained severe damage, as shown in Figure 5. Maximum crush was 13.0 in. at the left front corner of the vehicle. The left front wheel and control arm were severely bent and pushed rearward 9.0 in., causing damage to the floor pan under the driver's feet. The entire left side of the vehicle was dented and scraped. There was also considerable damage to the hood, bumper, grill, and radiator.

Sequential photographs, a summary of the test results, and other information pertinent to this test are given in Figure 6. The maximum 0.050-sec average acceleration experienced by the vehicle was $-6.6\,g$ in the longitudinal direction and $-4.9\,g$ in the lateral direction. Occupant impact velocity in the





PARTS LIST FOR ASSEMBLY MK9			
ITEM	DWG No.	DESCRIPTION	QTY
1	10001	MK7 PANEL	2
2		MK9 'Z' BULKHEAD	1
3	10002	MK 7 LID	1
4		MK9 LID BRACKET	2
5		SELF PLUG RIVET 1/4 x 9/32 LG	4
6	10005	STRAP NUT	8
7		FLAT WASHER 5/8 STD	16
8		HEX HD BOLT 5/8 x 1 1/4 LG	16

FIGURE 2 Details of IBC MK-9 standard barrier assembly.

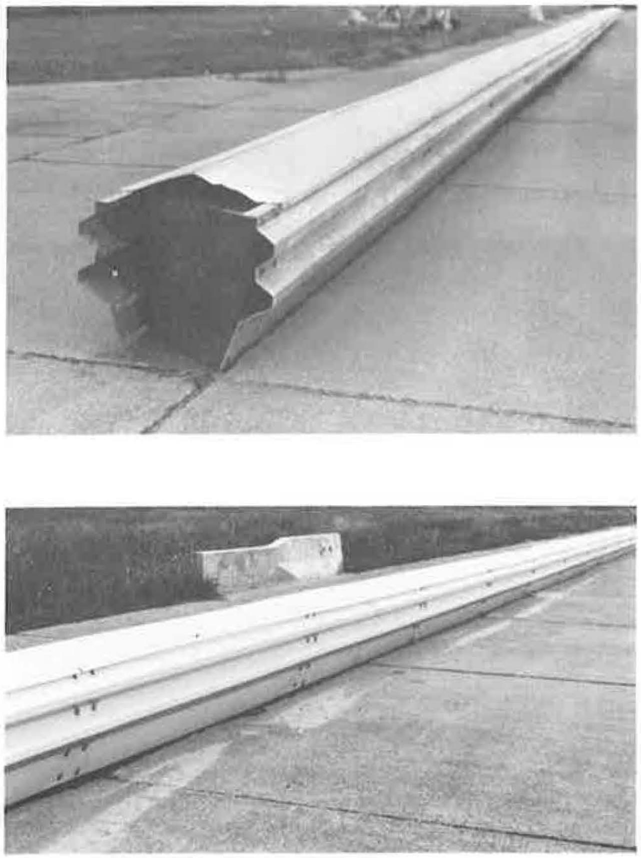


FIGURE 3 MK-9 barrier test installation.

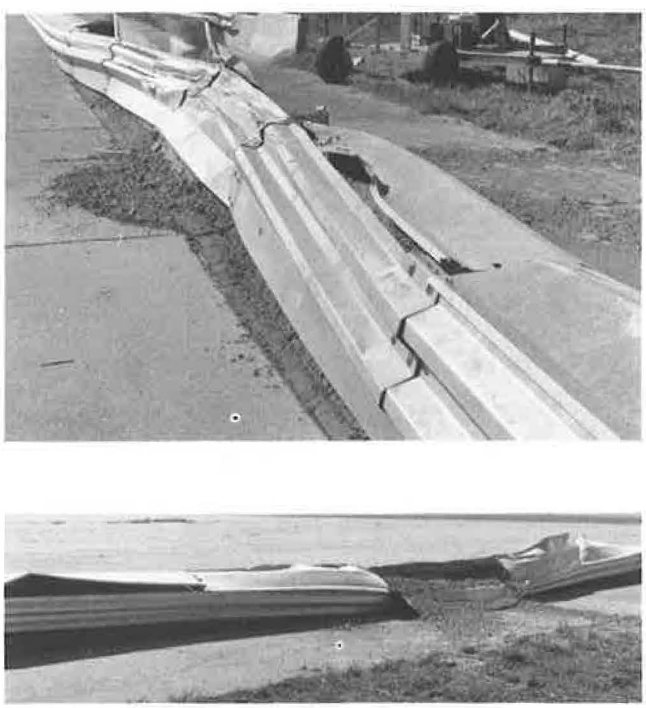


FIGURE 4 MK-9 barrier after Test 7091-2.

FIGURE 5 Vehicle after Test 7091-2.

longitudinal direction was 22.9 ft/sec and 13.0 ft/sec in the lateral direction. The highest 0.10-sec occupant ridedown accelerations were $-7.8\,g$ (longitudinal) and $-5.0\,g$ (lateral). The occupant risk criteria are not applicable to this test but are reported for information purposes.

In summary, the barrier failed to contain and redirect the vehicle. There was severe damage to the barrier due to penetration by the vehicle. The vehicle was also severely damaged, but there was only minimal deformation and intrusion into the occupant compartment. The vehicle, although airborne shortly after impact, remained upright during the initial test period and after leaving the barrier. The researchers had predicted that this test would not be successful, but the test was conducted to provide a baseline for comparison purposes.

Test 2 (7091-10)

A 1979 Cadillac struck the center of the ninth module of the MK-9 barrier with stabilized fill material (or approximately 89 ft from the beginning of the barrier) at 61.7 mph and 24.2 degrees. The vehicle was contained and smoothly redirected with an exit speed of 48.8 mph and an exit angle of 6.3 degrees. The brakes were applied as the vehicle exited the barrier. The vehicle yawed counterclockwise and came to rest approximately 218.0 ft from the point of impact.

The barrier sustained minor damage as shown in Figure 7. The maximum permanent residual deformation to the barrier was 1.0 in. laterally. The barrier moved laterally 4.5 in. on impact. The vehicle was in contact with the barrier for 14.0 ft.

The vehicle sustained severe damage, as shown in Figure 8. Maximum crush was 16.0 in. at the left front corner of the

0.000 s 0.140 s 0.290 s 0.500 s

Test No	7091-2	Impact Speed	62.0 mi/h (99.7 km/h)
Date	03/14/88	Impact Angle	25.2 degrees
Test Installation	IBC MK9 Barrier	Vehicle Accelerations	
Length of Installation	189.0 ft (57.6 m)	(Max. 0.050-sec Avg)	
Vehicle	1981 Oldsmobile 98	Longitudinal	-6.6 g
Vehicle Weight		Lateral	-4.9 g
Test Inertia	4500 lb (2041 kg)	Occupant Impact Velocity	
Vehicle Damage Classification		Longitudinal	22.9 ft/s (7.0 m/s)
TAD	11LFQ6	Lateral	13.0 ft/s (4.0 m/s)
CDC	11FLEK3 & 11LFES3	Occupant Ridedown Accelerations	
Maximum Vehicle Crush	13.0 in (33.0 cm)	Longitudinal	-7.8 g
		Lateral	-5.0 g

FIGURE 6 Summary of results for Test 7091-2.

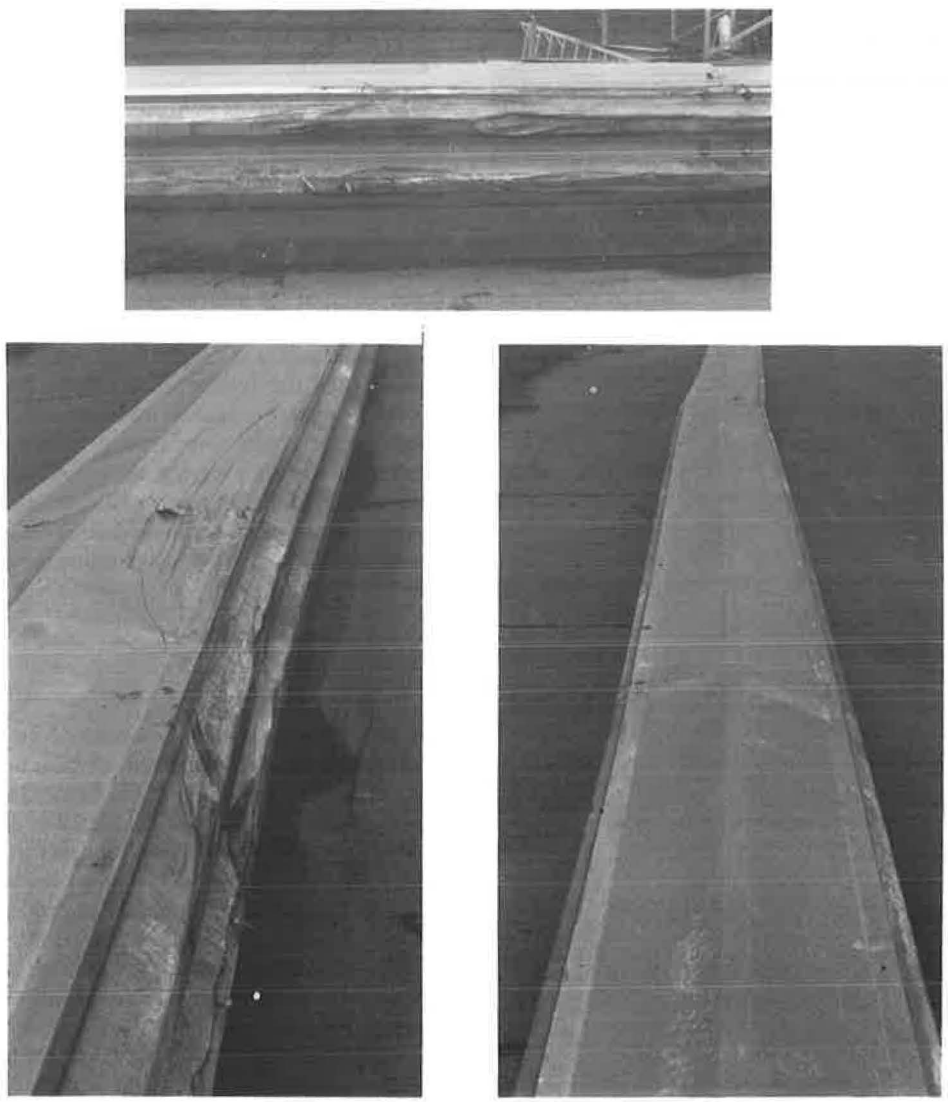


FIGURE 7 MK-9 barrier after Test 7091-10.

FIGURE 8 Vehicle after Test 7091-10.

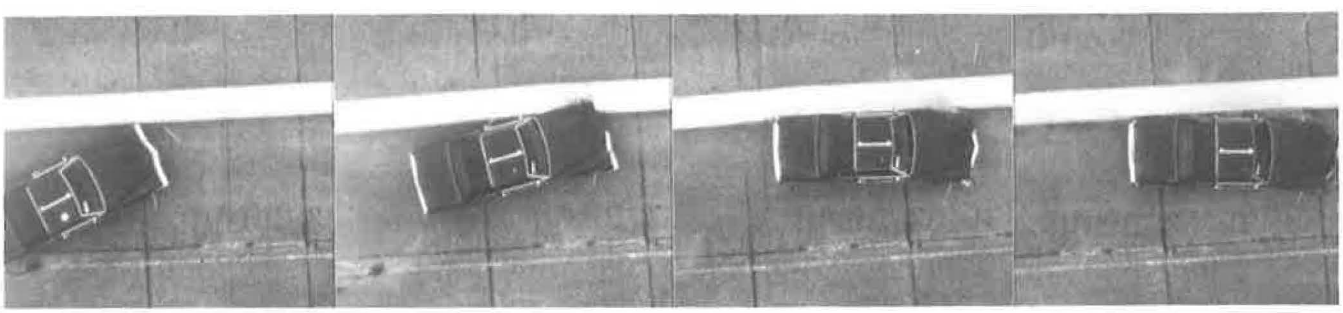
vehicle. The left front wheel was severely bent and pushed rearward 4.3 in. In addition, the subframe was bent. The entire left side of the vehicle, including the front and rear fenders and the door, was dented and scraped. The entire front end of the vehicle sustained considerable damage.

Sequential photographs, a summary of the test results, and other information pertinent to this test are given in Figure 9. The maximum 0.050-sec average acceleration experienced by the vehicle was -6.3 g in the longitudinal direction and -12.5 g in the lateral direction. The occupant impact velocity was 18.6 ft/sec in the longitudinal direction and 27.0 ft/sec in the lateral direction. The highest 0.10-sec occupant ridedown accelerations were -5.9 g (longitudinal) and -10.8 g (lateral). The occupant risk criteria are not applicable to this test but are reported for information purposes.

In summary, the barrier contained and redirected the vehicle. The vehicle sustained severe damage, but the barrier sustained minimal damage. There were no detached elements or debris. There was minimal deformation and intrusion into the occupant compartment. The vehicle remained upright and stable during the initial test period and after leaving the barrier. The exit speed and trajectory of the vehicle indicate minimum potential intrusion into the adjacent traffic lanes.

Test 3 (7132-1)—Crash Test of MK-7 Barrier with 80,000-lb Tractor Van-Trailer

The test installation consisted of 33 modules (10.5 ft for each module) of the standard MK-7 barrier for a total length of



0.000 s 0.125 s 0.249 s 0.374 s

Test No	7091-10
Date	01/17/89
Test Installation	IBC MK9 Barrier
Length of Installation . . .	189 ft (57.6 m)
Vehicle	1979 Cadillac Coupe DeVille
Vehicle Weight	
Test Inertia	4500 lb (2041 kg)
Vehicle Damage Classification	
TAD	11FL4 & 11LD2
CDC	11FLEK4 & 11LDES1
Maximum Vehicle Crush	16.0 in (40.6 cm)

Impact Speed	61.7 mi/h (99.3 km/h)
Impact Angle	24.2 degrees
Exit Speed	48.8 mi/h (78.6 km/h)
Exit Angle	6.3 degrees
Vehicle Accelerations (Max. 0.050-sec Avg)	
Longitudinal	-6.3 g
Lateral.	-12.5 g
Occupant Impact Velocity	
Longitudinal	18.6 ft/s (5.7 m/s)
Lateral	27.0 ft/s (8.2 m/s)
Occupant Ridedown Accelerations	
Longitudinal	-5.9 g
Lateral	-10.8 g

FIGURE 9 Summary of results for Test 7091-10.

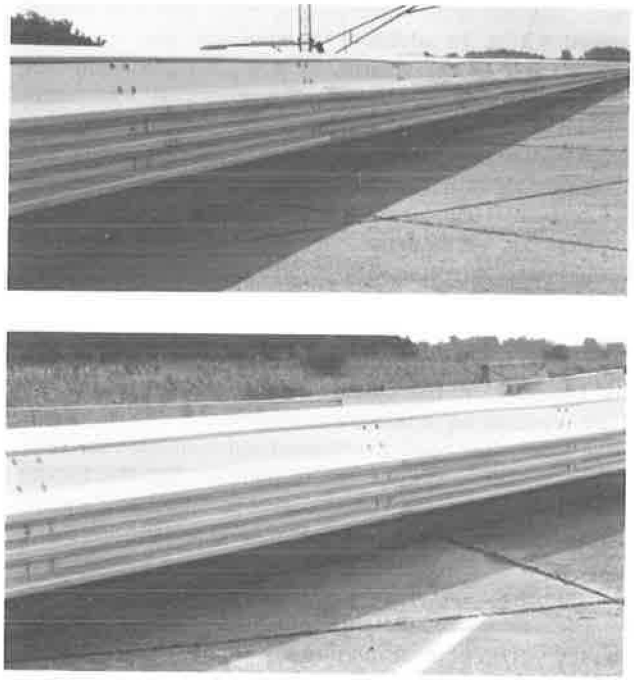
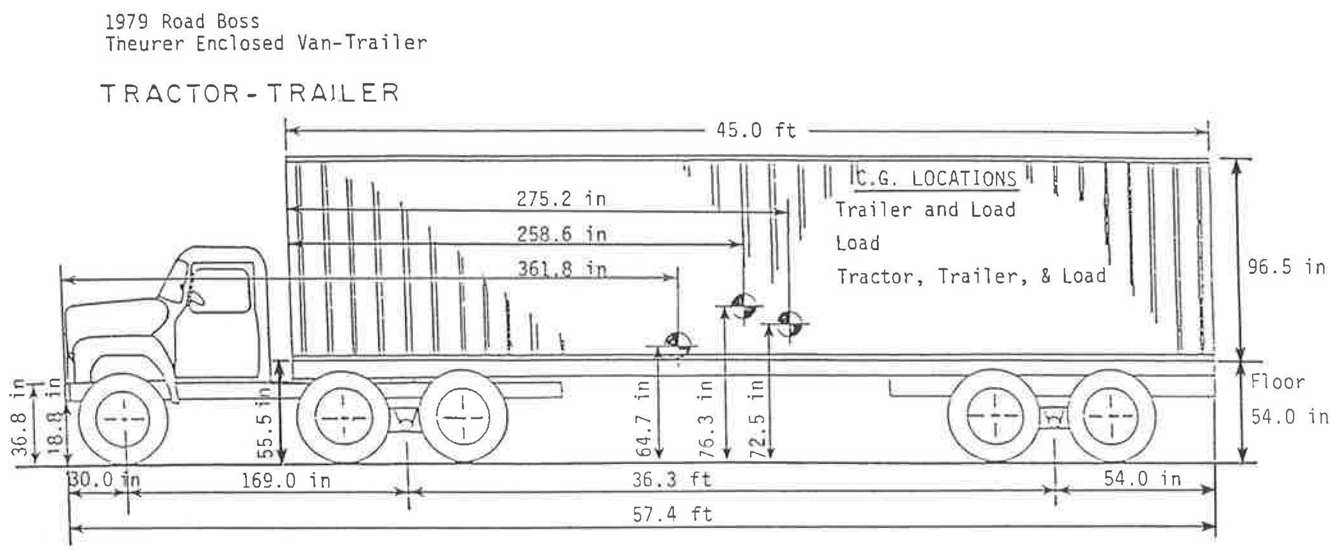


FIGURE 10 IBC MK-7 barrier test installation.

FIGURE 11 Tractor-trailer before Test 7132-1.



EMPTY WEIGHTS:	
Weight on front axle	8,720
Weight on center axles	11,920
Weight on rear axles	8,840
Total Empty Weight	29,480

LOADED WEIGHTS	
Weight on front axle	11,230
Weight on center axles	34,510
Weight on rear axles	34,260
Total Loaded Weight	80,000

FIGURE 12 Vehicle properties for Test 7132-1.

346.5 ft. The barrier modules were filled with portland cement–stabilized sand and the sand was mechanically compacted. The stabilized sand consisted of pit run sand mixed with 10 percent portland cement with a 10 percent moisture content (i.e., 100 lb sand mixed with 10 lb portland cement and 10 lb water). The test installation was placed directly on a concrete pavement surface with no anchorage to the pavement. A layer of untreated sand 2 to 3 in. high was placed at the bottom of the test barrier installation. The purpose of using a layer of untreated sand was to prevent the stabilized sand from bonding with the pavement surface. Photographs of the test installation are shown in Figure 10.

The crash test procedures were generally in accordance with guidelines presented in *NCHRP Report 230*. The test tractor was a 1979 White Road Boss with an empty weight of 16,240 lb. The trailer was a Theurer enclosed van-trailer with an empty weight of 13,060 lb. The combined tractor-trailer empty weight was 29,480 lb. Photographs of the tractor-trailer are shown in Figure 11.

The tractor-trailer was loaded with sandbags and wooden pallets to a test inertia mass of 80,000 lb, in accordance with *NCHRP Report 230* requirements. The height of the center of gravity (c.g.) for the combined tractor-trailer was 64.7 in., which compares with the c.g. height of 65 in. ± 1 in. used in most other 80,000-lb tractor-trailer crash tests. The key dimensions of the tractor-trailer; the actual locations of center of gravity for the load, trailer and load, and the combined tractor-trailer and load; and the empty and loaded axle weights are shown in Figure 12.

The test tractor-trailer was instrumented with three rate transducers to measure roll, pitch, and yaw rates. In addition, the tractor-trailer was instrumented with one set of triaxial accelerometers and three sets of biaxial accelerometers to measure acceleration levels during the impact. The triaxial accelerometers were mounted near the rear of the fifth wheel. The three sets of biaxial accelerometers were located at the front of the tractor, near the front of the trailer, and at the rear of the trailer.

The tractor-trailer struck near the center of the 11th module, approximately 110 ft from the beginning of the barrier, at 50.9 mph and 15.0 degrees. The tractor-trailer was smoothly redirected and remained in contact with the barrier through the end of the barrier. When the truck lost contact with the end of the barrier, the brakes were applied, and the tractor-trailer turned to the left and came to rest almost perpendicular to the barrier. The tractor-trailer traveled 275 ft from the initial point of impact to the point of rest. The tractor-trailer rolled considerably toward the barrier during the impact sequence but remained upright, with the bottom of the left side of the trailer sliding on top of the barrier.

The barrier sustained minor damage as shown in Figure 13. The maximum permanent residual deformation to the barrier was 4.0 in. and the maximum permanent lateral movement was 7.0 in. The stabilized sand fill material remained basically intact after the impact with only localized areas of crushing. Although the vehicle was in contact with the barrier for 239.0 ft, the major damage to the barrier was confined to the first three modules (or roughly 30 ft) downstream from the point of initial impact. Damage to the other modules was limited to scrapes and tears of the side panels as the tractor-trailer slid along the barrier to its final rest position.

The vehicle sustained severe damage, as shown in Figure 14. The front left corner of the bumper was deformed, and the left side of the tractor was damaged. The left front wheel was deformed from impact with the barrier, and the wheel was displaced 18 in. rearward from its normal position into the battery box. The rearward displacement was a result of the fracturing of one right-side and both left-side U-bolts, which mount the front axle to the front leaf springs. The lower left front and upper right front shock absorber mounts were

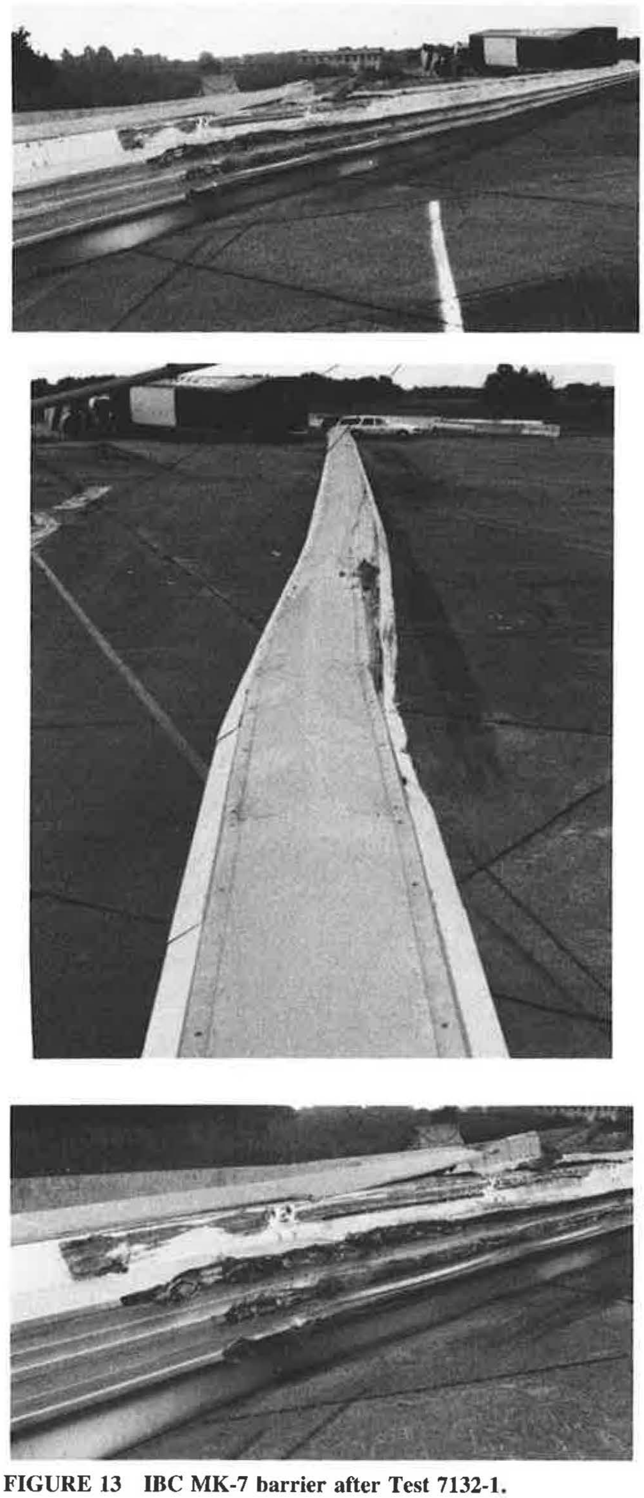


FIGURE 13 IBC MK-7 barrier after Test 7132-1.

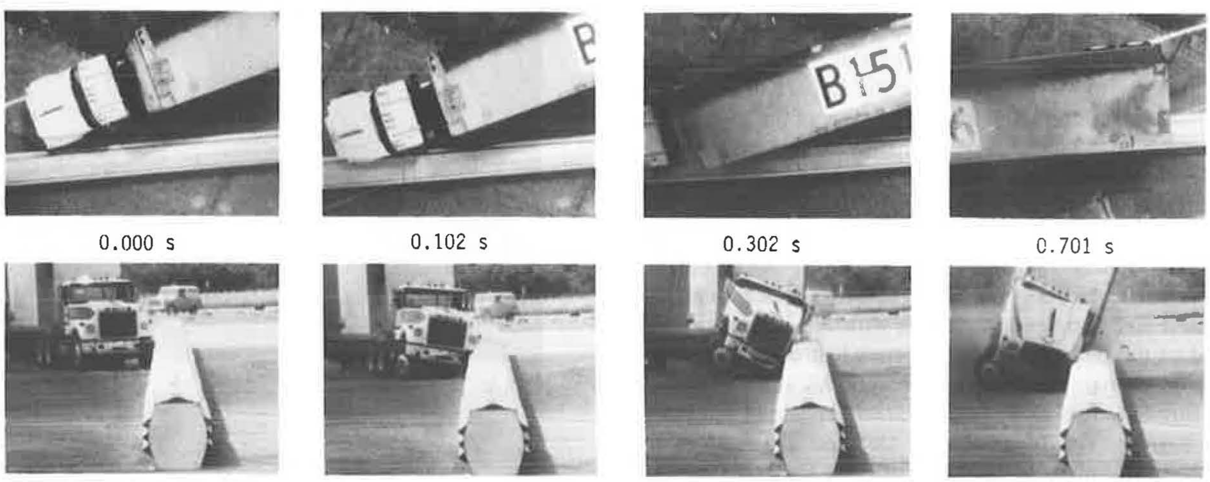
FIGURE 14 Tractor-trailer after Test 7132-1.

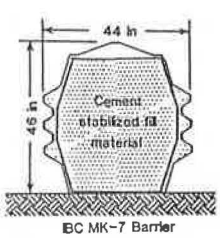
separated from the axle assembly, and the pitman arm separated from the steering assembly as the axle was displaced rearward. The battery box was deformed and displaced rearward into the front surface of the left fuel tank, but the fuel tank remained intact with only minor deformations.

The left frame rail was displaced rearward 6 in. relative to the right rail. The left outer tires of the tractor's rear tandem axles were deflated and the rims were severely deformed as a result of contact with the barrier. The tractor's rear tandem axles were shifted back on the left side approximately 2.0 in.

The trailer sustained direct contact damage along the entire lower left side. The left rear suspension mounting rail was fractured, allowing the floor to drop down approximately 22.8 in., forming a V-shaped left side surface. The left wall of the trailer shifted to the left approximately 40 in. (when viewed from the rear) and separated from the roof structure at the top joint. The right wall was deformed slightly due to induced damage. Both left outer tires on the trailer axles were deflated and the rims were damaged extensively. The trailer landing gear also sustained minor damage.

Sequential photographs, a summary of the test results, and other information pertinent to this test are given in Figure 15. The maximum 0.050-sec average accelerations experienced by the tractor-trailer at the various accelerometer locations along the tractor-trailer are summarized in Table 1. For instance, the maximum 0.050-sec average acceleration experienced by the tractor near the fifth wheel was $-5.4\ g$ in the





Test No. 7132-1
Date 05/31/89
Test Installation . . . . IBC MK-7 Median Barrier w/stabilized fill material
Installation Length . . . 346.5 ft (105.6 m)
Vehicle . . . . 1979 White Road Boss Tractor w/van-trailer
Vehicle Weight
Test Inertia . . . . . . 80,000 lb (36,287 kg)
Gross Static . . . . . . 80,000 lb (36,287 kg)

Note: A layer of untreated sand, two to three inches in depth, was placed at the bottom of the test barrier installation.

Impact Speed . . . . . . 50.9 mi/h (81.9 km/h)
Impact Angle . . . . . . 15.0 deg
Exit Speed . . . . Not Applicable
Exit Trajectory . . . . Not Applicable
Vehicle Accelerations (near fifth wheel)
(Max. 0.050-sec Avg)
Longitudinal . . . . . -5.4 g
Lateral . . . . -10.2 g
Occupant Impact Velocity (passenger compartment)
Longitudinal . . . . . 8.8 ft/s (2.7 m/s)
Lateral . . . . 13.9 ft/s (4.2 m/s)
Occupant Ridedown Accelerations
(passenger compartment)
Longitudinal . . . . . -2.6 g
Lateral . . . . -4.6 g

FIGURE 15 Summary of results for Test 7132-1.

TABLE 1 SUMMARY OF MAXIMUM 0.050-SEC AVERAGE ACCELERATIONS AT VARIOUS LOCATIONS ALONG TRACTOR-TRAILER

Location of Accelerometer	Maximum 0.050-Second Average Acceleration (g) Longitudinal	Lateral	Vertical
Front of Tractor	- 3.2	- 5.5	N/A
Near Fifth Wheel of Tractor	- 5.4	-10.2	3.2
Front of Trailer*	- 1.7	- 4.1	N/A
Rear of Trailer	- 2.5	-10.3	N/A

N/A - Not applicable

* Note: Signal loss for this accelerometer group at 0.460 second after impact.

longitudinal direction, -10.2 *g* in the lateral direction, and 3.2 *g* in the vertical direction.

Occupant impact velocity in the longitudinal direction was 8.8 ft/sec and 13.9 ft/sec in the lateral direction. The highest 0.10-sec occupant ridedown accelerations were -2.6 *g* (longitudinal) and -4.6 *g* (lateral). These occupant impact velocities and ridedown accelerations are from the accelerometers mounted in the passenger compartment at the front of the tractor.

In summary, results of the crash test indicate that the IBC MK-7 barrier with stabilized fill material meets the guidelines set forth in *NCHRP Report 230*. The barrier successfully contained and redirected the 80,000-lb tractor van-trailer. The barrier sustained only minor damage with maximum permanent deformation of 4.0 in. and maximum lateral movement of approximately 7 in. The tractor-trailer sustained severe damage, but there were no detached elements or debris and no deformation or intrusion into the occupant compartment. The vehicle traveled along the barrier after impact to the end of the barrier, indicating minimal potential for intrusion into adjacent traffic lanes. The vehicle remained upright throughout the test.

SUMMARY

The results of an effort to develop an IBC MK-7 barrier that could contain and redirect an 80,000-lb tractor van-trailer were summarized. The developmental effort included laboratory testing, computer simulation, and full-scale crash testing of the smaller IBC MK-9 barrier with automobiles. A prototype IBC MK-7 barrier with stabilized fill material was constructed and crash tested with an 80,000-lb tractor van-trailer at 50.9 mph and 15.0 degrees. The tractor-trailer was contained and smoothly redirected by the barrier. The tractor-trailer rolled considerably toward the barrier during the impact sequence but remained upright, with the bottom of the left side of the trailer sliding on top of the barrier. The vehicle was severely damaged, but the barrier sustained only minor damage. The results of the crash test indicate that the IBC MK-7 barrier with stabilized fill material meets the evaluation guidelines set forth in *NCHRP Report 230* for a high-performance truck barrier.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the support of the International Barrier Corporation, the ATA Foundation, Inc., and the USX Corporation in sponsoring the studies reported in this paper.

REFERENCES

1. *Development of IBC MK-7 Type Longitudinal Barrier to Restrain and Redirect 80,000-lb Tractor-Van Trailer.* Test Reports 7091-1 through 14. Texas Transportation Institute, Texas A&M University System, College Station, 1988–1989.
2. K. K. Mak, T. J. Hirsch, and D. L. Bullard. *Testing and Evaluation of the IBC MK-7 Barrier (with Stabilized Filler Material) with an 80,000-lb Tractor Van-Trailer.* Final report on Project RF 7132. Texas Transportation Institute, Texas A&M University System, College Station, 1989.
3. J. D. Michie. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances.* TRB, National Research Council, Washington, D. C., 1981.

The contents of this paper reflect the views of the authors, who are solely responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the International Barrier Corporation, the ATA Foundation, Inc., or the USX Corporation.

Performance Evaluation of a Movable Concrete Barrier

DORAN L. GLAUZ

A series of crash tests and operational demonstrations of a precast movable concrete barrier (MCB) were performed. Four crash tests of the MCB showed that it can successfully redirect both light and heavy passenger cars at various angles of impact. The crash tests involved two large cars weighing 4,370 lb and 4,300 lb, traveling 59.3 and 59.4 mph, and striking at 24 degrees and 16 degrees, respectively; and two small cars weighing 2,000 lb and 1,895 lb, traveling 57.7 and 58.6 mph, and striking at 15½ degrees and 20½ degrees, respectively. The crash tests satisfied the requirements for structural adequacy and occupant risk in *NCHRP Report 230*. Vehicle trajectory requirements were not satisfied because of large exit angles. The demonstrations consisted of (a) a transfer vehicle straightening a deflected barrier after the last crash test; (b) a transfer vehicle transporting, assembling, and transferring a barrier on a 1,400-ft radius with a 12 percent cross-slope; (c) a transfer vehicle transferring a barrier on a 4 to 5 percent longitudinal grade; and (d) manual movement of the barrier to adjust minor misalignments. The MCB moves laterally under impact. The lateral movement is related to impact severity. Two equations are presented to predict lateral movement as a function of impact severity.

Traffic congestion has increased rapidly in recent years. At many highway and bridge locations there has not been room to add lanes or funds have been insufficient. At those locations where traffic is heavy in one direction in the morning and heavy in the opposite direction in the evening, a need has developed for a median barrier that can be moved easily from one lane boundary to another. With a movable barrier it would be possible to adjust the number of lanes available to peak traffic daily, while maintaining a positive barrier between opposing traffic lanes. The California Department of Transportation (Caltrans) has a pressing need for such a barrier on the Coronado Bridge in San Diego. The relocatable pylons used there now do nothing to retain out-of-control vehicles, and there have been severe head-on collisions. There are other locations where a movable barrier could be used to advantage. These include locations where a permanent system is needed and also construction and maintenance locations where a mobile barrier is needed that would provide greater protection to motorists and workers. Over the years several systems have been proposed to Caltrans. These systems have required an extensive and complicated mechanical installation within the roadbed, introducing a potential maintenance headache and precluding them from temporary use, or have demonstrated inferior performance as a barrier.

Transportation Laboratory, California Department of Transportation, 5900 Folsom Blvd., P.O. Box 19128, Sacramento, Calif. 95819.

DESCRIPTION OF BARRIER AND TRANSFER VEHICLE

A barrier that meets the criterion of simplicity and requires no roadbed modification has been developed. This barrier was conceived, developed, and tested in response to a continuing demand for a movable barrier from the United States and other countries. The Quickchange Movable Concrete Barrier System was invented by Quick-Steel Engineering Pty, Ltd., of Botany, New South Wales, Australia. Barrier Systems, Inc. (BSI), of Sausalito, California, is the North American licensee for the system. Hereafter this system will be referred to as a movable concrete barrier (MCB).

The MCB is a segmented concrete barrier formed similar to a Configuration F-shape modified with a narrowed neck and a T-shaped top (Figure 1). The segments are 3.28 ft (1 m) long, 2 ft (609 mm) wide at the base, and 32 in. (812 mm) high. They are joined together by a pin-and-link hinge.

The MCB is moved from one traffic lane line to another with a transfer vehicle (Figure 2). The vehicle is a mobile steel framework, which may be either self-propelled or towed, with an S-shaped conveyor assembly mounted on it. Closely spaced urethane conveyor wheels ride under the flanges of the T-shape of the stem (Figure 3). The segments are lifted off the pavement by the wheels, guided along the S-shaped conveyor to the new lane position, and lowered back down to the pavement. The barrier segments remain pinned together during the transfer operation. As the vehicle moves forward, the barrier is transferred from left to right (when used as a median barrier), minimizing the exposure of the transfer vehicle to traffic in both directions (Figure 2).

SCOPE OF RESEARCH

A series of crash tests and operational demonstrations of the MCB were performed. Two crash tests indicated a deficiency in the original design. After modification by the manufacturer, four additional tests demonstrated successful redirection of large and small cars. Four operational demonstrations indicated the maneuverability and maintainability of the MCB.

BARRIER DESIGN

Two tests were conducted on two versions of the original Australian design and are described in the full report (*1*). The tests were at impact angles of 15 degrees and 25 degrees with heavy vehicles at 60 mph (27 m/sec). The lateral deflections

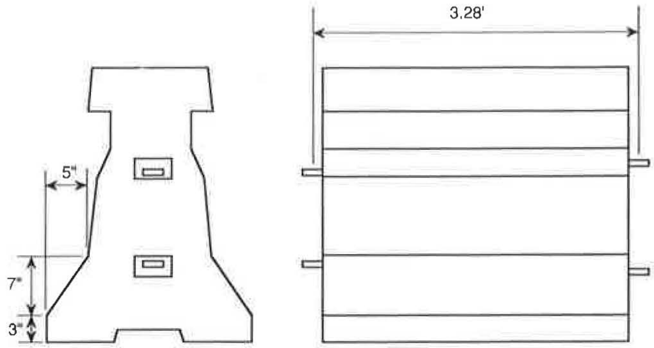


FIGURE 1 End view and elevation of MCB.

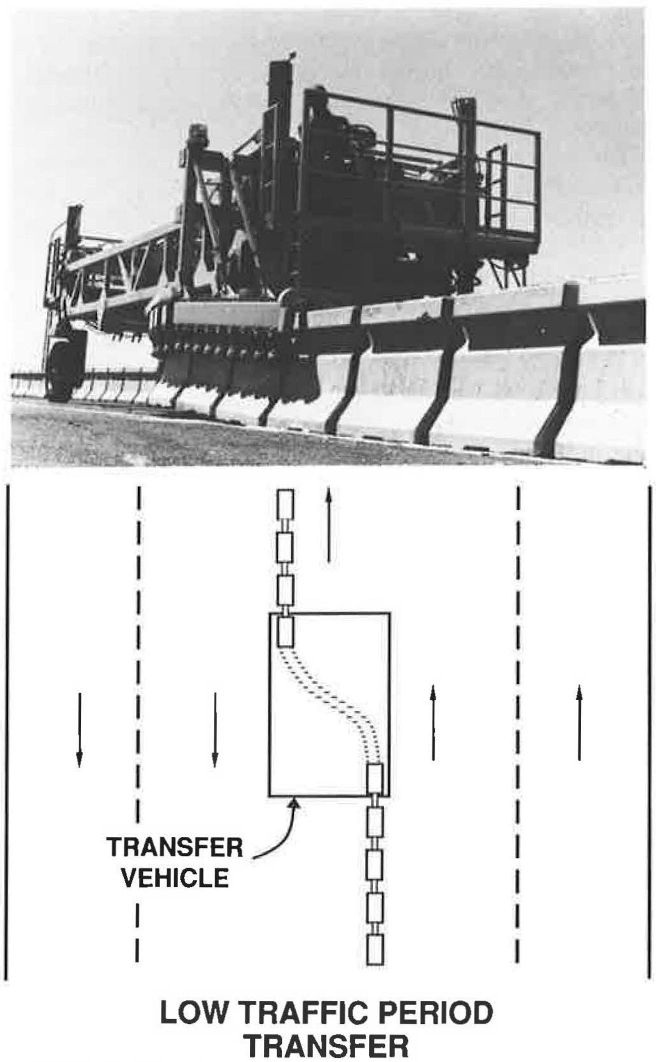


FIGURE 2 Transfer vehicle moves barrier one full lane width.

were 4.56 and 5.77 ft (1.4 and 1.8 m) for the two tests. The strength of the stem proved to be inadequate. Because this barrier was anticipated for use on a permanent installation, the lateral deflection was considered excessive.

The manufacturer, BSI, undertook a testing and development program to design a stronger stem and to determine what factors are important to lateral deflection. The stem was strengthened by thickening the narrow neck section, increasing it from 5⅛ in. (130 mm) to 8⅛ in. (206 mm) and increasing the reinforcement from 6 × 6–W5 × W5 welded wire fabric to two No. 4 reinforcing bars plus 4 × 4–W4 × W4 welded wire fabric. In addition, the wire fabric was bent outward into the top flange (Figure 4).

The method devised to limit the lateral deflection was to reduce the longitudinal clearance in the hinge assembly (Figure 5). The original design had a ±½ in. (12.7 mm) clearance to allow for barrier lengthening and shortening in changing radii and expansion joints on bridges. This clearance was reducted to ±3/16 in. (4.8 mm). By reducing the clearance, more barrier segments (more mass) must be mobilized to effect a unit of lateral movement; thus more energy would be required per unit.

TEST RESULTS

Test 443 (4,370 lb, 59.3 mph, 24 degrees)

The left front bumper of the test vehicle struck the 100-segment barrier at the midpoint of Segment 62 at 59.3 mph (26.5 m/sec) and an angle of 24 degrees. The length of vehicle contact with the barrier was about 39 ft (12 m), from Segments 62 to 74. The car was smoothly redirected and lost contact with the barrier at an exit angle of 14¾ degrees. The car

FIGURE 3 Barrier is lifted by conveyor wheels under the MCB flange.

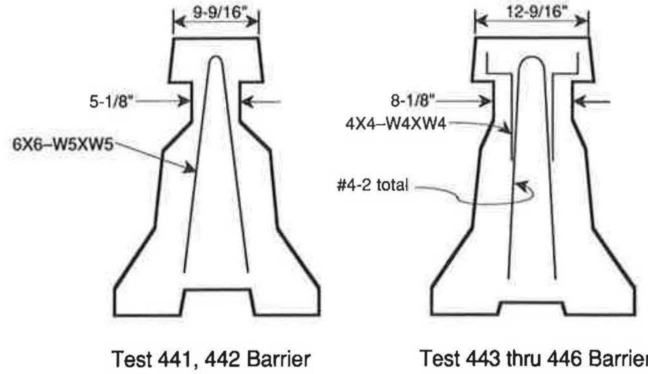


FIGURE 4 Changes from Australian barrier design.

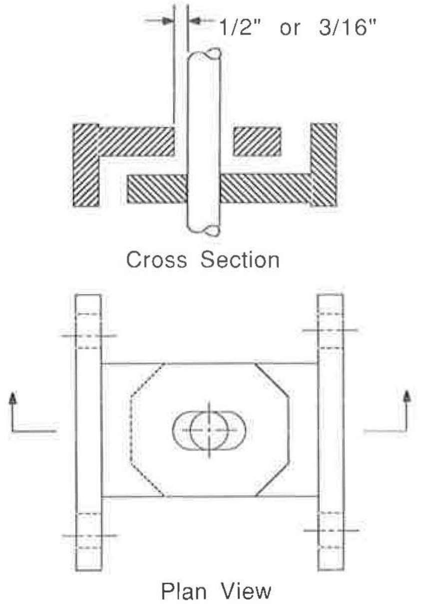


FIGURE 5 Simplified hinge detail.

experienced a maximum roll of −10¼ degrees. The maximum rise of the car was 4 in. (100 mm) 0.73 sec after impact, measured at the right rear corner of the roof. Figure 6 shows sequential photographs and a trajectory diagram.

The trajectory of the car after impact was back toward the line of the barrier. A second impact with the barrier occurred at Segment 93. The car came to rest about 30 ft (9.1 m) beyond the downstream end of the barrier and approximately in line with its face (Figure 7).

The barrier was displaced laterally along a distance of about 66 ft (20 m) (Segments 54 through 75). The maximum lateral displacement was 3.74 ft (1.3 m) at Segment 66 (Figure 8). There was longitudinal movement in the barrier from Segments 22 to 100. The maximum longitudinal displacement in the downstream direction was 0.5 ft (140 mm) at Segment 54. The maximum longitudinal displacement in the upstream direction was 0.15 ft (45 mm).

Test 444 (2,000 lb, 57.7 mph, 15½ degrees)

The left front bumper of the test vehicle struck the 100-segment barrier at the midpoint of Segment 48 at 57.7 mph (25.8 m/sec) and an angle of 15½ degrees. The length of vehicle contact with the barrier was about 16 ft (5 m), from Segments 48 to 52. The car was smoothly redirected and lost contact with the barrier at an exit angle of 10¼ degrees. The car experienced a maximum roll of −14½ degrees and a pitch of +10¼ degrees. The maximum rise of the car was 17 in. (430 mm) 0.36 sec after impact, measured on the right rear tire. Figure 9 shows sequential photographs and a trajectory diagram.

The trajectory of the car after impact was away from the barrier. The car came to rest off the paved area about 15 ft (4.6 m) beyond the downstream end of the barrier and 60 ft (18 m) from its face (Figure 10).

The barrier was displaced laterally along a distance of about 30 ft (9 m) (Segments 47 through 55) (Figure 11). The maximum lateral displacement was 1.78 ft (542 mm) at Segment 51. There was longitudinal movement in the barrier from Segments 36 to 65. The maximum longitudinal displacement in the downstream direction was 0.1 ft (30 mm) at Segment 47. The maximum longitudinal displacement in the upstream direction was 0.1 ft (31 mm) at Segment 55.

Test 445 (4,300 lb, 59.4 mph, 16 degrees)

The left front bumper of the test vehicle struck the 100-segment barrier at the midpoint of Segment 52 at 59.4 mph (26.6 m/sec) and an angle of 16 degrees. The length of vehicle contact with the barrier was about 33 ft (10 m), from Segments 52 to 61. The car was smoothly redirected and lost contact with the barrier at an exit angle of 16½ degrees. The car experienced a maximum roll of +6¼ degrees and a pitch of +5⅜ degrees. The maximum rise of the car was 19 in. (490 mm) 0.54 sec after impact, measured on the right rear bumper. Figure 12 shows sequence photographs and a trajectory diagram.

The trajectory of the car after impact was away from the barrier. The car came to rest off the paved area at the toe of an earth berm about 79 ft (24 m) beyond the downstream end of the barrier and 41 ft (12.5 mm) from its face (Figure 13).

The barrier was displaced laterally along a distance of about 59 ft (18 m) (Segments 47 through 65). The maximum lateral displacement was 2.85 ft (870 mm) at Segment 59 (Figure 14). There was longitudinal movement in the barrier from Segments 26 to 81. The maximum longitudinal displacement in the downstream direction was 0.4 ft (110 mm) at Segment 58. The maximum longitudinal displacement in the upstream direction was 0.1 ft (34 mm) at Segment 70.

Test 446 (1,895 lb, 58.6 mph, 20½ degrees)

The left front bumper of the test vehicle struck the 100-segment barrier at Segment 55 at 58.6 mph (26.2 m/sec) and an angle of 20½ degrees. The length of vehicle contact with the barrier was about 20 ft (6 m), from Segments 55 to 60. The car was smoothly redirected and lost contact with the barrier at an exit angle of 19½ degrees. The car experienced a maximum roll of −15 degrees and a pitch of +12½ degrees. The maximum rise of the car was 30 in. (760 mm) 0.44 sec after impact, measured on the right rear bumper. Figure 15 shows sequence photographs and a trajectory diagram.

The trajectory of the car after impact was away from the barrier. The car came to rest about even with the downstream end of the barrier 37 ft (11 m) away from its face (Figure 16).

The barrier was displaced laterally along a distance of about 42 ft (13 m) (Segments 52 through 64). The maximum lateral displacement was 2.24 ft (684 mm) at Segment 59 (Figure 17). There was longitudinal movement from Segments 37 to 84. The maximum longitudinal displacement in the downstream direction was 0.15 ft (48 mm) at Segment 55. The maximum longitudinal displacement in the upstream direction was 0.2 ft (54 mm) at Segment 64.

DISCUSSION OF TEST RESULTS

In Tests 443 through 446 the MCB demonstrated its ability to retain and redirect a vehicle under a variety of impact

Impact + 0.014 s

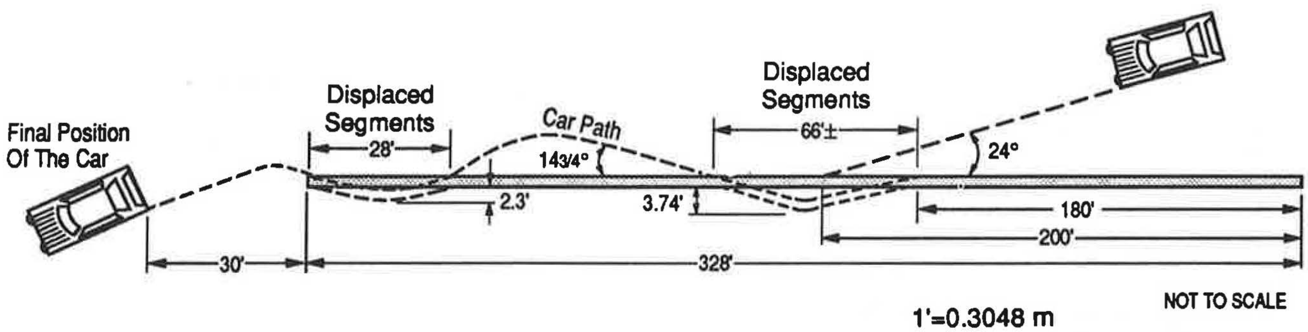
I + 0.074 s

I + 0.204 s

I + 0.674 s

I + 1.32 s

I + 1.68 s



Test Barrier:
- Type: Movable Concrete Barrier (Simple Hinge Connections with Reduced Clearance)
- Length: 328 ft (100 m) - 100 segments

Test Date: November 18, 1987

TestVehicle:
- Model: 1982 Olds Station Wagon
- Inertial Mass: 4370 lb (1982 kg)
- Impact Velocity: 59.3 mph (26.5 m/s)
- Impact; Exit Angle: 24 deg; 14 3/4 deg

Test Dummy:
- Type: Part 572, 50th Percentile Male
- Weight / Restraint: 165 lb (75 kg)/ none
- Position: Driver's seat

Test Data:
- Occupant Impact Velocity (long): 27.0 fps. (8.2 m/s)
- Max 50 ms Avg Accel: long –8.3 g, lat -7.7 g, vert -2.0 g
- HIC / TAD / VDI: 121 / LFQ6 / 11LDEW2
- Max Roll;Pitch;Yaw : –10 1/4 deg; NA; NA
- Barrier Displacement: 3.74 ft (1.14 m) at segment 66
- Max Dynamic Deflection (film): 4.10 ft (1.25m)
- Barrier Damage: Minor scratches on 11 segments at the area of contact with test car

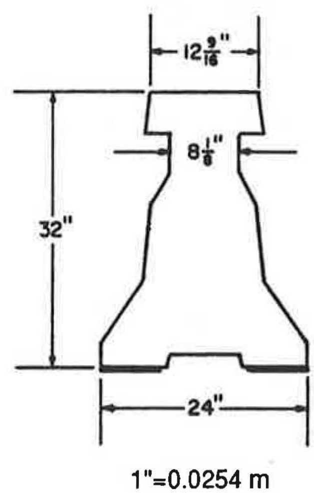


FIGURE 6 Summary of data for Test 443.

FIGURE 7 Car and barrier after impact, Test 443.

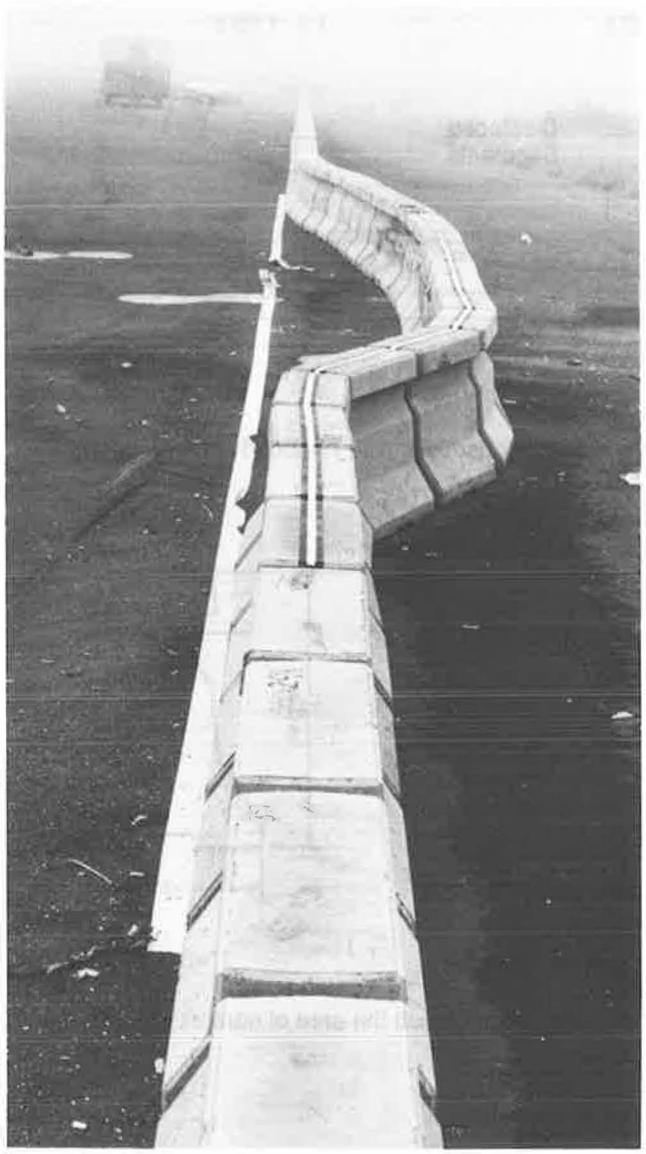


FIGURE 8 Deflected barrier after impact, Test 443.

conditions. Vehicle rederection was smooth in all these tests. There was no tendency for the barrier to pocket or trap the vehicles. There was no evidence of any structural distress of the barrier segments. All four tests were performed on the same set of barrier segments without replacing any segments, welded hinge plates, or steel hinge pins. Segments were shifted after each test so fresh segments would be located in the main impact zone.

There was significant lateral displacement of the test barrier during each test (Table 1). The barrier displacement was closely related to impact severity (IS). The data from these tests were statistically analyzed to obtain an equation for lateral displacement as a function of IS.

Two equations (Table 2) were found to fit the experimental data. These equations are represented in graphical form with the experimental data in Figure 18. The correlation is significant at the 5 percent level (*2*, pp. 462–463). Data from other tests were also used in deriving these equations (*1*; E. F. Nordlin, unpublished data).

For very small values, up to 3.8 ft-kips (5.2 kJ), no deflection is predicted by Equation 1. Although the second equation approaches a zero displacement as IS approaches zero, it can be considered to evaluate to zero for IS less than 1 ft-kip (1.4 kJ).

For small impacts, up to 15 ft-kips (20 kJ), it is believed that Equation 1 understates the displacement that might be expected. Within this impact severity range, Equation 2 probably gives a better value of lateral displacement. The reason why the lateral displacement is probably larger than that predicted by Equation 1 lies in the action within the hinge during impact. In high-IS impacts, like those used to derive Equation 1, many of the barrier segments move. For each segment that moves, the entire longitudinal clearance in the hinge is taken up, effecting a lengthening of the barrier to allow lateral movement. During low-energy impacts many fewer segments are brought into the movement zone, down to the limiting case where only two segments move at all. In an impact when only two or three segments move, all the longitudinal clearance in the hinge may not be used, thus allowing movement with very low energy input.

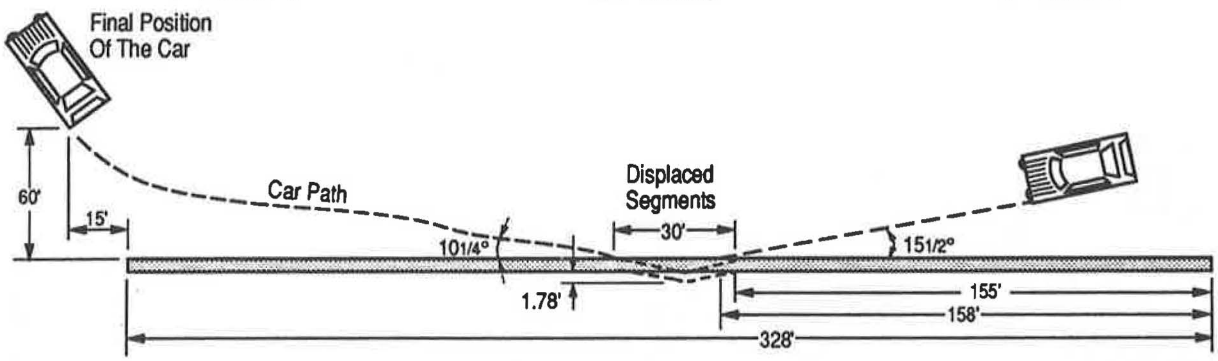
In the range of 15 to 130 ft-kips (20 to 175 kJ), the two equations give the same answer within the accuracy that can be expected from such an estimator. Caution must be exercised when using these equations to extrapolate beyond 100 ft-kips (135 kJ), because that is beyond the value of any data used to derive the equations. At some unknown value of impact severity some structural elements of the barrier may fail, thus invalidating any attempt at predicting deflection.

Table 3 shows roll, pitch, and yaw values, maximum 50 msec average accelerations, occupant impact velocities, and ridedown accelerations. For comparison, Tests 443 through 446 are included with data from previous tests on continuous concrete safety shaped barriers done by Caltrans.

Note that the magnitude of roll in Tests 443 through 446 is generally lower than in other tests of concrete safety shaped barriers. The amount of roll and pitch is low to moderate in all MCB tests. None of the test cars showed any indication of being close to rollover. Scuff and rub marks on the face of the barrier indicated that the projecting cap of the MCB restricted the climb of the car, thereby minimizing the roll angle.

Impact + 0.054 s I + 0.209 s I + 0.401 s

I + 0.591 s I + 1.288 s I + 2.322 s



Test Barrier:
- Type: Movable Concrete Barrier (Simple Hinge Connections with Reduced Clearance)
- Length: 328 ft (100 m) - 100 segments

Test Date: December 18, 1987

TestVehicle:
- Model: 1981 Honda Civic
- Inertial Mass: 2000 lb (907 kg)
- Impact Velocity: 57.7 mph (25.8 m/s)
- Impact; Exit Angle: 15 1/2 deg; 10 1/4 deg

Test Dummy:
- Type: Part 572, 50th Percentile Male
- Weight / Restraint: 165 lb (75 kg)/ none
- Position: Driver's seat

Test Data:
- Occupant Impact Velocity (long): 15.1 fps. (4.6 m/s)
- Max 50 ms Avg Accel: long –4.6 g, lat –6.7 g, vert 1.7 g
- HIC / TAD / VDI: 30 / LFQ4 / 12LDEE2
- Max Roll;Pitch;Yaw : –14 1/2 deg; 10 1/4 deg; NA
- Barrier Displacement: 1.78 ft (0.54 m) at segment 51
- Max Dynamic Deflection (film): 1.92 ft (0.58 m)
- Barrier Damage: Minor scratches at the area of contact with test car

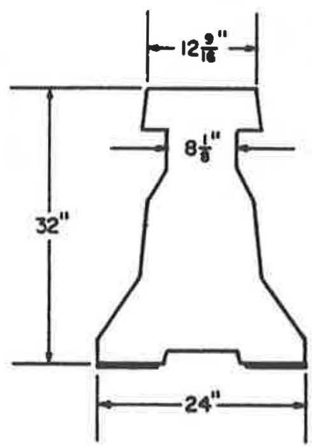


FIGURE 9 Summary of data for Test 444.

FIGURE 10 Car and barrier after impact, Test 444.

FIGURE 11 Deflected barrier after impact, Test 444.

The longitudinal occupant impact velocity in Test 444 (see Table 3) was below the *NCHRP Report 230* (*3*) recommended maximum value and also smaller than in other Caltrans tests on permanent concrete median barriers. Although this was the only test required to meet Section F of the occupant risk requirements of *NCHRP Report 230*, the criterion was also met in Tests 443, 445, and 446.

In all four tests the exit angle exceeded 60 percent of the impact angle, the recommended limit in *NCHRP Report 230*, though only slightly in Tests 443 and 444 (Table 4). In Test 443 the velocity change also exceeded the recommended limit, 15 mph. In that test, the vehicle steered back toward the MCB and struck a second time. In Tests 444, 445, and 446 the vehicle speed change was 11 to 12 mph (4.9 to 5.3 m/sec), and the vehicles then crossed the traveled way and came to rest 40 to 60 ft (11 to 18 m) from the barrier face. The vehicles were disabled in all four tests and stopped 150 to 200 ft (45 to 62 m) from the impact point.

THE TRANSFER VEHICLE

The transfer vehicle is 49 ft (15 m) long and 8.2 ft (2.5 m) wide and weighs 30 tons (27 000 kg) (Figure 19). It is self-powered; a 200-hp (150-kW) diesel engine powers a hydraulic drive and steering. Each wheel can be independently raised and lowered. A barrier can be transferred onto or off a curb up to 12 in. high. The lateral move of the barrier can be varied from 6 ft to 16 ft. Up to 15 segments of the barrier can be carried and transported as a unit. The transfer vehicle operates in either direction and is operationally symmetrical. Each end of the vehicle is independently steered with its own steering wheel. Movement can be controlled from either end.

DEMONSTRATIONS OF TRANSFER VEHICLE

A prototype transfer vehicle was used for four demonstrations. The demonstrations consisted of (a) straightening a deflected barrier after the last crash test, (b) transporting and assembling a 10-segment length of barrier, (c) transferring a barrier on a 1,400-ft radius with a 12 percent cross slope, and (d) transferring a barrier on a 4 to 5 percent longitudinal grade.

The first demonstration showed the ability of the transfer vehicle to realign a deflected barrier. The barrier was deflected by Test 446 a maximum of 2.24 ft (683 mm). The barrier was back to a straight alignment in its original position after two passes (Figure 19). It appeared that with more experienced operators the barrier could have been made straight with only one pass. Realignment was accomplished without placing

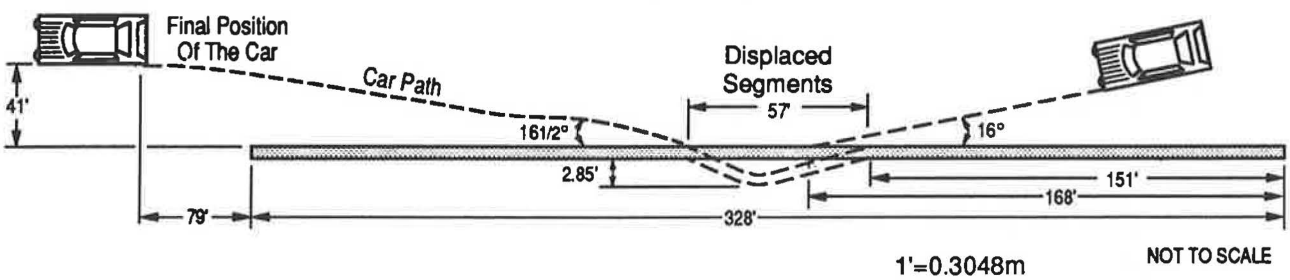
Impact

I + 0.118 s

I + 0.238 s

I + 0.458 s

I + 0.71 s



Test Barrier:
Type: Movable Concrete Barrier (Simple Hinge Connections with Reduced Clearance)
Length: 328 ft (100 m) - 100 segments
Test Date: January 21, 1988
TestVehicle:
Model: 1982 Olds Station Wagon
Inertial Mass: 4300 lb (1950 kg)
Impact Velocity: 59.4 mph (26.6 m/s)
Impact; Exit Angle: 16 deg; 161/2 deg
Test Dummy:
Type: Part 572, 50th Percentile Male
Weight / Restraint: 165 lb (75 kg)/ none
Position: Driver's seat
Test Data:
Occupant Impact Velocity (long): 14.3 fps (4.4 m/s)
Max 50 ms Avg Accel: long –3.3 g, lat –5.9 g, vert –1.7 g
HIC / TAD / VDI: 45 / LFQ4 / 12LDEE2
Max Roll;Pitch;Yaw : 61/4 deg; 53/8 deg; NA
Barrier Displacement: 2.85 ft (0.87 m) at segment 59
Max Dynamic Deflection (film): 3.04 ft (0.93 m)
Barrier Damage: Minor scratches and spalling at the area of contact with test car

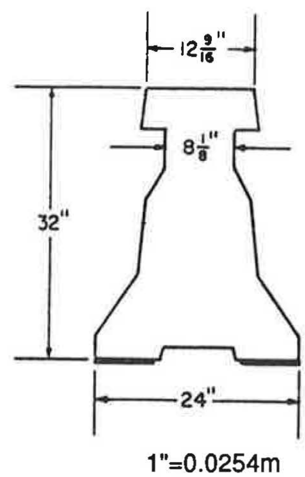


FIGURE 12 Summary of data for Test 445.

FIGURE 13 Car after impact, Test 445.

FIGURE 14 Deflected barrier after impact, Test 445.

workers on the ground to manually adjust the barrier. Two additional passes were made over the barrier to demonstrate simple transfer operation. All the functions of the transfer vehicle—lifting, lateral transport, and deposit of the modules—were smooth and continuous, and the vehicle moved at about 6 mph (2.7 m/sec).

The second demonstration showed how lengths of barrier can be transported and reattached to a standing barrier. (Such an operation might be performed to move the lane closure zone of a progressing construction site.) A length of barrier, 10 segments, was loaded onto the conveyor of the transfer vehicle, carried to the location of the third demonstration, and reassembled (Figure 20). The transport distance was about 0.5 mi (800 m), and the travel speed on the paved road was about 10 mph (4.5 m/sec). To reassemble the MCB, the barrier on the ground was aligned with the barrier within the vehicle and a hinge pin was inserted. Alignment was accomplished by loading the portion on the ground partway into the conveyor (Figure 21) until it came in contact with the carried barrier. There was some difficulty inserting the pin because the joint to be connected was sometimes pushed too far into the vehicle, to a place that hampered insertion. Even with that problem, though, assembly of the barrier was much faster than if it had been set one segment at a time.

The third demonstration consisted of transferring a barrier plus and minus 6 ft (1.8 m) from its original position on a curve of radius 1,400-ft (426.7 m) with a 12 percent cross slope (Figure 22). Two reference lines were laid out for use by the vehicle operators to place the barrier on each transfer run. A total of 70 segments were used to compose a barrier 230 ft (70 m) long. Two four-movement cycles were performed. In each cycle, the barrier was first moved outward to a 1,406-ft (428.5-m) radius, then inward two times to a radius of 1,394 ft (424.9 m), then outward to its original position.

The last demonstration, transferring a barrier on a 5 percent longitudinal grade, was done in Lodi, California, at the BSI test site. The barrier consisted of 76 segments for a total length of 250 ft (76 m). The whole barrier was transferred laterally back and forth 6 ft (1.8 m) each time from the middle, initial position. The speed of the transfer vehicle was about 5 mph (2.2 m/sec) both uphill and downhill. The barrier segments were freestanding in the first eight transfers and tethered in the second set of eight transfers.

Measurements of the joint displacements were taken across a set of four joints located about 50 ft (15 m) from each barrier end. The measurements were taken after each lateral transfer. The net change in length was near zero after each complete transfer cycle. Stretching of the barrier apparently occurred during travel of the transfer vehicle uphill, and contraction occurred during downhill transfers. However, the number of transfers was too small for a definite pattern to be discerned.

The lateral transfers resulted in a gradual longitudinal movement of the barrier system downhill. Measurements of longitudinal movement were made at the downhill end of the barrier. The total longitudinal movement was 4¾ in. (120 mm) after eight lateral transfers. Because the length of the barrier did not change, as shown by the measurements above, the whole barrier must have moved longitudinally downhill.

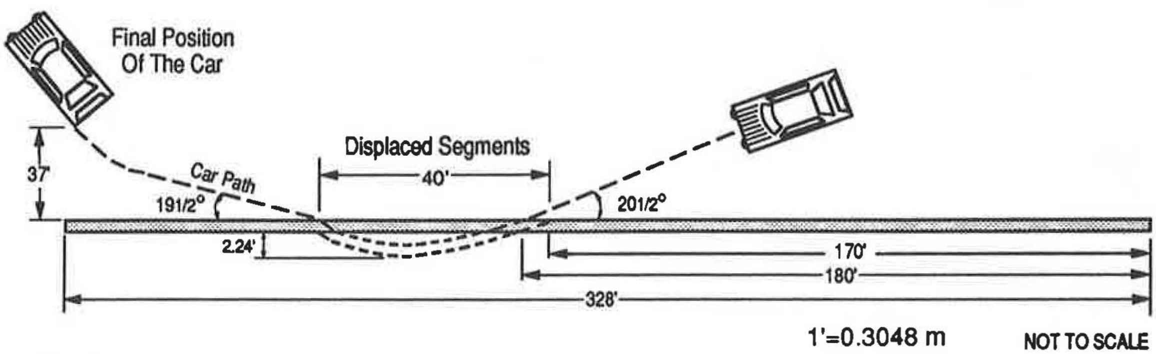
To counteract this tendency, the upstream end of the barrier was tethered with a cable tensioned to 1,000 lb (450 N) at the beginning of each downhill run (Figure 23). The same measurements as for the freestanding barrier were performed. The measurements indicated an apparent stretching of the barrier after each transfer cycle. The stretch was about 0.1 in. (2.5 mm) per joint. A total longitudinal movement of 3⅜ in. (84 mm) occurred after eight lateral transfers. Because the upstream end of the barrier was tethered, the downhill creep may be explained by the stretch in the barrier noted.

Although creep appeared to be restricted by pulling at the upstream end, it was not eliminated. A definite pattern or determination cannot be drawn from these data because the number of repetitions was limited.

Longitudinal creep has been reported in a similar barrier system installed in Paris, France (*4*). The total longitudinal movement of the French barrier 1.5 mi (2.4 km) long on a downhill grade of 1.5 to 2.0 percent was 3.3 to 6.6 ft (1 to 2 m) during the initial months of operation. The French solution to retard longitudinal creep was manual jacking of the uphill end of the barrier system before starting each daily barrier

Impact + 0.015 s | I + 0.098 s | I + 0.168 s

I + 0.303 s | I + 0.515 s | I + 1.208 s



Test Barrier:
- Type: Movable Concrete Barrier (Simple Hinge Connections with Reduced Clearance)
- Length: 328 ft (100 m) - 100 segments

Test Date: March 9, 1988

TestVehicle:
- Model: 1984 Nissan
- Inertial Mass: 1890 lb (857 kg)
- Impact Velocity: 58.6 mph (26.2 m/s)
- Impact; Exit Angle: 20 1/2 deg; 19 1/2 deg

Test Dummy:
- Type: Part 572, 50th Percentile Male
- Weight / Restraint: 165 lb (75 kg)/ none
- Position: Driver's seat

Test Data:
- Occupant Impact Velocity (long): 16.9 fps (5.2 m/s)
- Max 50 ms Avg Accel: long -7.6 g, lat –11.3 g, vert 2.8g
- HIC / TAD / VDI: 86 / LFQ4 / 11LDEE2
- Max Roll;Pitch;Yaw : –15 deg; 12 1/2 deg; NA
- Barrier Displacement: 2.24 ft (0.68 m) at segment 59
- Max Dynamic Deflection (film): 2.41 ft (0.73 m)
- Barrier Damage: Minor scratches on 2 segments at the area of contact with test car

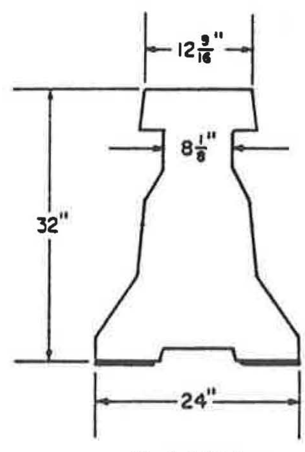


FIGURE 15 Summary of data for Test 446.

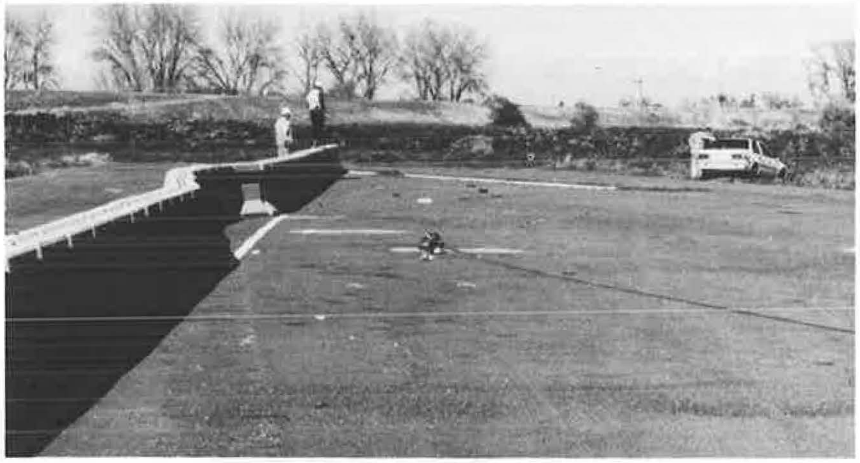


FIGURE 16 Car and barrier after impact, Test 446.

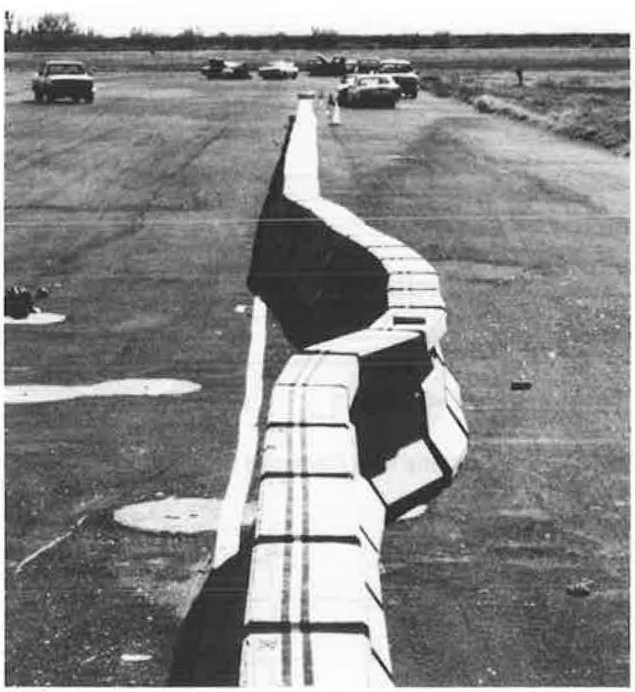


FIGURE 17 Deflected barrier after impact, Test 446.

transfer in the downhill direction, similar to what was done in this demonstration.

MANUAL MOVEMENT

Another method of moving the MCB is by hand. This would be useful in making minor alignment adjustments either while assembling the barrier or after an impact. Movement by hand was done by a single person using a pry bar 6 ft (2 m) long during installation of the test barrier. BSI also demonstrated that a vehicle access 9 ft (2.8 m) can be made by one person in 3 min (*5*).

CONCLUSIONS

Based on the results of impact tests on this movable concrete barrier, the following conclusions can be drawn: Small cars can be smoothly redirected by a MCB with satisfactory occupant risk factors. The MCB is strong enough to fully contain a 4,500-lb (2040-kg) vehicle, striking at 60 mph (26 m/sec) and 25 degrees with no structural failure and little debris generation. The vehicle exit angle tends to be slightly more than 60 percent of the impact angle. The flanged top that is

TABLE 1 LATERAL DISPLACEMENT OF BARRIER

TEST #	Vehicle Inertial Mass lbs (kg)	Impact Speed, mph (m/s)	Impact Angle, degrees	Impact Severity ft-kips (kJ)	Max. Permanent Lat. Displacement D, ft. (m)
443	4370 (1982)	59.3 (26.5)	24	85.0 (115)	3.74(1.14)
444	2000 (907)	57.7 (25.8)	15 1/2	15.9 (21.5)	1.78(0.54)
445	4300 (1950)	59.4 (26.6)	16	38.4 (50.8)	2.85(0.87)
446	1895 (857)	58.6 (26.2)	20 1/2	26.7 (36.1)	2.24(0.68)

TABLE 2 EQUATIONS TO PREDICT LATERAL DISPLACEMENT

Eq. #	Equation	Coefficients A	B	C	Applicable IS Range ft-kips (kJ)	r
1	$D = A + B \ln(IS)$	-1.62	1.21	-	15–130	.993
		(-0.592)	(0.365)		(20–175)	
2	$D = A + B^{(1/IS)}\, IS^{C}$	0.961	0.0125	0.319	1–130	.985
		(0.266)	(0.00263)	(0.319)	(1–175)	

For D in meters, use the coefficients in parentheses.

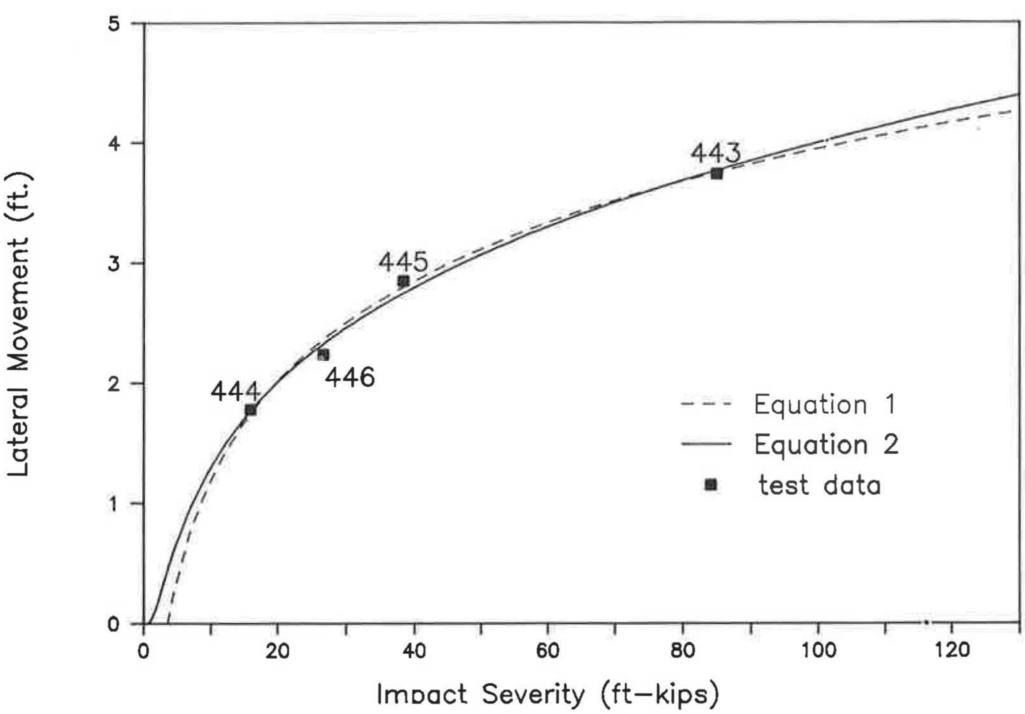


FIGURE 18 Plot of predictive equations and test data.

TABLE 3 TEST RESULTS

Test #	443	444	445	446	451(6)	431(7)	262(8)	264(8)	301(9)	321(10)
Concrete Barrier Type	MCB	MCB	MCB	MCB	New Jersey	New Jersey	Type 50	Type 50	Type 50	Type 50
Car Mass, lbs	4370	2000	4300	1895	3575	1860	4960	4860	4860	4700
(kg)	(1982)	(907)	(1950)	(857)	(1622)	(844)	(2250)	(2200)	(2200)	(2130)
Impact Angle,deg	24	15 1/2	16	20 1/2	45	52	25	25	27	26
Speed, mph	59.3	57.7	59.4	58.6	40.3	27.4	59.0	64.0	68.0	61.0
(m/s)	(26.5)	(25.8)	(26.6)	(26.2)	(18.0)	(12.2)	(26.4)	(28.6)	(30.4)	(27.3)
Roll, degrees	-10 1/2	-14 1/2	6 1/2	-15	7 1/2	71	>90	NA	27	48
Pitch, degrees	NA	10 1/4	5 3/8	12 1/2	NA	-2	NA	NA	NA	NA
Yaw, degrees	NA	NA	NA	NA	NA	-12	NA	NA	NA	NA
Maximum rise, in.	4.4	16.7	19.3	29.6	NA	NA	34	36	38	66
Max. 50 ms Average Acceleration, g										
Longitudinal[1]	8.3	-4.6	-3.3	-7.6	-11.2	-12.4	7.0	5.2	11.7	NA
Lateral[2]	-7.7	-6.7	-5.9	-11.3	-8.7	-5.5	11.6	13.0	13.8	NA
Occupant Impact Velocity Vlimit fps (m/s)										
Longitudinal[3]	27.0	15.1	14.3	16.9	28.6	32.9	NA	NA	NA	NA
	(8.2)	(4.6)	(4.4)	(5.2)	(8.7)	(10.0)				
Lateral (digital recorder)		18.0	NA	14.0	NA	NA	NA	NA	NA	NA
NA										
	(5.5)		(4.3)							
Ride down Accelerations, g										
Longitudinal	-5.6	-6	-3.9	-5	NA	-15	NA	NA	NA	NA
Lateral	7.6	-10	10.6	-13	NA	-10	NA	NA	NA	NA

1. TRC 191 recommended value: -5g (acceptable value: -10g)
2. TRC 191 recommended value: -3g (acceptable value: -5g)
3. NCHRP Report 230 1. recommended value: 30fps (9.1 m/s)

TABLE 4 IMPACT AND EXIT CONDITIONS

Test number	Impact Angle, deg.	60% of Impact Angle, deg.	Exit Angle, deg.	Impact Speed, V_I mph (m/s)	Exit Speed, V_E mph (m/s)	Speed Change, $V_I - V_E$ mph (m/s)
443	24	14 1/2	14 3/4	59.3 (26.5)	27.0 (12.1)	32.3 (14.4)
444	15 1/2	9 1/4	10 1/4	57.7 (25.8)	45.8 (20.5)	11.9 (5.3)
445	16	9 1/2	16 1/2	59.4 (26.6)	48.0 (21.5)	11.4 (5.1)
446	20 1/2	12 1/4	19 1/2	58.6 (26.2)	47.6 (21.3)	11.0 (4.9)

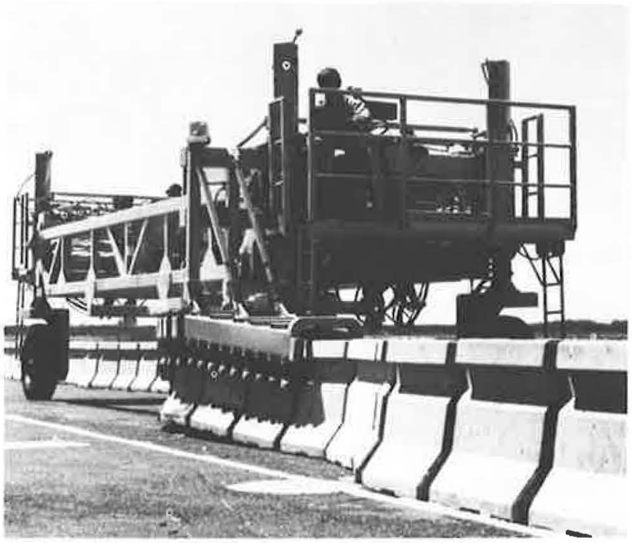


FIGURE 19 Transfer vehicle straightening deflected barrier.

FIGURE 20 Transfer vehicle carrying barrier.

FIGURE 21 Aligning barrier for connecting carried barrier and placed barrier.

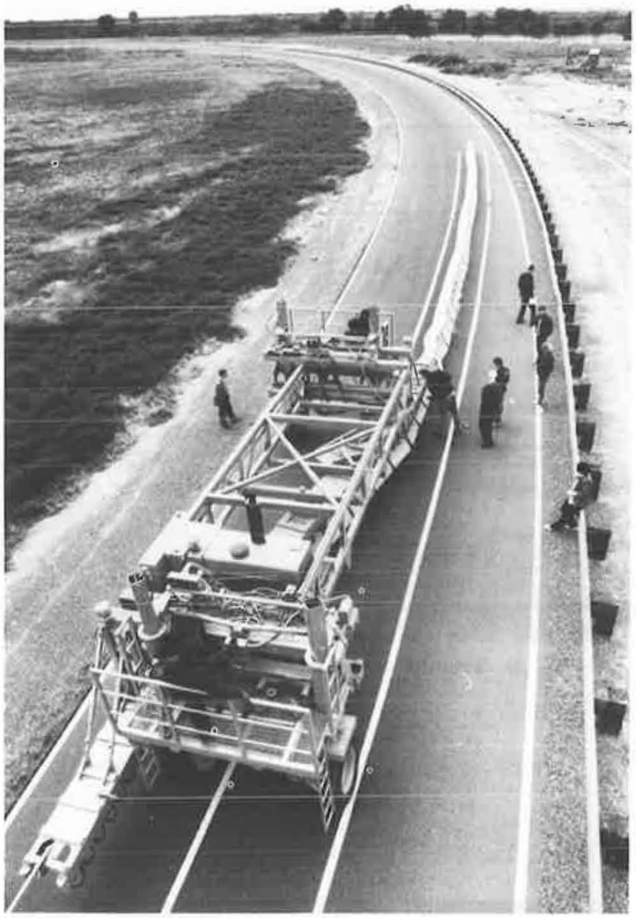


FIGURE 22 Transfer vehicle on curve with 1,400-ft radius.

used to lift the barrier appears to limit the distance a vehicle climbs the face of the barrier, thus limiting the roll angle of the vehicle. The MCB deflects laterally under impact. The lateral deflection of the MCB has a strong statistical relation to impact severity.

Based on the results of the demonstrations of moving the MCB both with a transfer vehicle and by hand, the following conclusions can be drawn: The transfer vehicle can easily and smoothly move the barrier one full lane width at speeds up to 6 mph (2.7 m/sec). Transporting, assembling, and transferring an MCB on a curve of radius 1,400 ft (427 m) with a 12 percent cross slope and transferring a barrier on a 5 percent longitudinal grade can be successfully performed by the transfer vehicle. And a barrier deflected as much as 2.24 ft (0.7 m) can be straightened by the transfer vehicle or can be pushed back into place with a pry bar by one person.

FIGURE 23 Tensioning the tether cable.

ACKNOWLEDGMENTS

This federally funded research project was initiated by Caltrans and was a joint effort of Caltrans and BSI. BSI supplied a test barrier in place and conducted demonstrations of the transfer vehicle. Caltrans conducted the crash tests, collected and analyzed data, and wrote the final research report (*1*).

REFERENCES

1. *Vehicle Crash Tests of a Movable Concrete Barrier*. California Department of Transportation, Sacramento, 1989.
2. *Basic Statistical Methods for Engineers and Scientists*. Harper and Row, 1976.
3. J. D. Michie. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., 1981.
4. *Report on the Movable Median Barrier Highway A-15 Bridge over the Seine River, Sennevilliers, France*. Traffic Institute, Northwestern University, Evanston, Ill. 1987.
5. *Movable Median Barrier for the Golden State Bridge*. Barrier Systems, Inc., Sausalito, Calif., 1988.
6. *A Seatbelt Efficacy Demonstration: A Large Angle Moderate Speed Impact into a Concrete Median Barrier*. California Department of Transportation, Sacramento, 1987.
7. *Effects on a Vehicle Impacting a Concrete Safety Shape Barrier at a Low Speed and a Large Angle*. California Department of Transportation, Sacramento, 1986.
8. E. F. Nordlin, et al. *Dynamic Tests of a Prestressed Concrete Median Barrier Type 50 Series XXVI*. California Division of Highways, Sacramento, 1973.
9. E. F. Nordlin et al. *Dynamic Tests of a Slipformed Concrete Barrier Type 50 Placed over a Lowered Existing Cable Barrier*. California Department of Transportation, Sacramento, 1974.
10. *Vehicular Crash Tests of a Continuous Concrete Median Barrier Without a Footing*. California Department of Transportation, Sacramento, 1977.

Temporary Asphalt Medians for Two-Lane, Two-Way Operation

BENJAMIN H. COTTRELL, JR.

The objectives of this research were to evaluate the performance of temporary asphalt medians for use in two-lane, two-way operations as an alternative to portable concrete barriers and, if appropriate, to develop guidelines for the use of temporary asphalt medians. Use of the temporary asphalt median was evaluated at one site. The median was installed and removed at least twice as fast as concrete barriers, thereby reducing the time traffic was exposed to such activities by at least 50 percent. There was no difference in the cost per linear foot between the median and the concrete barrier because of the relatively high contract cost for the median compared with median costs in other states. However, use of the median will save a minimum of \$40,000 (\$80,000 on this project) by eliminating the use of impact attenuators at the concrete barrier end sections. It is expected that the costs will decrease as more medians are used; the cost per linear foot of the median was 40 percent lower on the second project in Virginia using the median. There is no evidence to suggest that the temporary asphalt median directly contributed to any accidents. However, it does appear that the presence of an intersection within two-lane, two-way operation may have been a factor in some accidents. The median performed well. Guidelines were developed for the use of the median.

Two-lane, two-way operation (TLTWO) describes the traffic flow pattern that results when one side of a four-lane divided highway is closed for reconstruction or repair and its traffic is diverted to the other side. Traffic flowing in opposing directions is limited to two lanes. A TLTWO is used when there is no feasible alternative. In Virginia, portable concrete median barriers are typically used to separate opposing streams of traffic in TLTWO.

Temporary barriers have four specific functions: to protect traffic from entering work areas, to provide positive protection for workers, to separate two-way traffic, and to protect construction (*1*). When struck, the portable concrete barrier provides protection by redirecting vehicles. The need for such positive protection to enhance safety is an important factor to consider in determining the type of treatment for TLTWO. The use of the barriers should be based on an engineering analysis that includes such factors as traffic volumes, traffic speeds, offset, and duration (*1*). There is no consensus on specific warrants for temporary barriers (*1*).

Because the portable concrete median barrier is expensive and it may not be needed or desired under certain traffic conditions, there is a need for a safe, cost-effective alternative for separating opposing traffic streams in TLTWO. Moreover, although experience with TLTWO in Virginia is limited, its use is expected to increase given the current and expected levels of bridge rehabilitation activities.

Virginia Transportation Research Council, Box 3817 University Station, Charlottesville, Va. 22903.

A typical temporary asphalt median (also called an island) is 12 in. to 18 in. wide and 4 in. high, is painted with reflectorized yellow paint, and has orange tubes with reflectorized white collars mounted about 50 ft apart as shown in Figure 1 (*2*). The median is highly visible and provides more positive delineation than the concrete barrier, especially at night. Because the median is narrower than the barrier, it occupies less of the travel lane. Several state departments of transportation, including those of North Carolina, Florida, Ohio, and Pennsylvania, have successfully used the medians, typically on roads with average daily traffic (ADT) volumes under 30,000. The medians are generally not recommended where physical (protective) separation of the opposing lanes is required or where the traffic volume is high, for example, where the ADT is above 50,000.

The estimated costs of installing, maintaining, and removing temporary asphalt medians was expected to be about one-third to one-sixth of those for portable concrete median barriers in Virginia. The time required to install and remove an asphalt median was found to be substantially less than that required for installing and removing a concrete barrier (*3*). This difference in time is an important safety consideration if the installation and removal must be done during exposure to traffic.

Because of limited operational experience with the medians, there is no consensus on the traffic and geometric conditions that warrant the use of temporary asphalt medians (*1*).

OBJECTIVES AND SCOPE

The objectives were to evaluate the performance of temporary asphalt medians for use in TLTWOs as an alternative to the portable concrete median barrier and, if appropriate, to develop guidelines for the use of temporary asphalt medians for the Virginia Department of Transportation (VDOT). Both medians and concrete barriers were studied. Emphasis was on a comparison of the installation and removal costs and the performance of the asphalt median.

METHODS

Five activities were conducted to accomplish the study objectives.

Development of Specifications

Specifications for the temporary asphalt medians were developed primarily on the basis of a telephone survey of state

FIGURE 1 Temporary asphalt median.

departments of transportation (DOTs) that have used the median. Following the survey, the respondents sent additional information, such as specifications and guidelines on the temporary asphalt medians. A computerized literature search and a literature review supplemented the survey. A synthesis of this information resulted in a proposed specification that was reviewed and revised by VDOT staff.

Selection of Sites

Much effort in soliciting sites for this study was directed toward the district traffic engineers and the Location and Design and Traffic Engineering Divisions. Criteria for site selection were developed. The solicitations were made periodically throughout the study period.

Field Evaluation

Data were collected at the study site on three phases of the field evaluation: installation of the median and TLTWO, maintenance of TLTWO, and removal of the median and TLTWO. Traffic volume, speed, and vehicle classification data were collected. Research Council staff collected data during the installation and removal phases, and they also collected the traffic data. The VDOT on-site project inspector provided data on the maintenance of the TLTWO work zone.

Comparative Analysis

An attempt was made to comparatively analyze the temporary asphalt median and the concrete barrier. Comparisons were made of the rates and costs of installation and removal.

Development of Guidelines

Guidelines for the use of the temporary asphalt median for TLTWO were developed based on the study activities.

RESULTS AND DISCUSSION

Survey Results and Specifications Development

Survey Results

Information on the use of temporary asphalt medians in five states was obtained through telephone surveys of five state DOTs. The median cross section was either the shape of a trapezoid or a rectangle with rounded corners. The base width ranged from 12 in. through 18 in. with a height of 4 in. The median was painted with yellow reflectorized paint. A curb machine was typically used to install the median (a small pavement-widening machine may also be used). Orange tubular markers 18 to 36 in. high, spaced 50 to 55 ft apart, with white reflectorized sleeves or collars provided additional delineation for the median. Raised pavement markers were used in one state as an alternative delineator. Drainage openings were provided in the median at a spacing of 25 to 500 ft, depending on drainage requirements.

Use of a temporary asphalt median is not a factor in determining the speed limit for TLTWO. The speed limit is determined on the basis of factors such as work zone conditions, road geometrics, and traffic volumes. In addition, lower speed limits through work zones are often ignored by motorists. The *Roadside Design Guide* conservatively suggests that the temporary asphalt median be used with speeds of 45 mph or less until there is more operational experience with the median (*1*). From the survey findings, the speed limit was eliminated as a factor in the use of the temporary asphalt median.

There have been no accident problems as a result of using the median on four- and six-lane divided roads under ADT volumes ranging from 10,000 to 60,000. On one project, a six-lane, divided roadway was converted to a four-lane, two-way operation on one side of the road using the shoulder as a lane. Overall, the median was successfully used by all five state DOTs.

Specifications

The specifications used for the study are shown in Figure 2. They were developed from a synthesis of the survey of state DOTs with minor revisions by VDOT personnel.

Site Selection

The following three criteria were used for selecting the site for the temporary asphalt median: ADT between 6,000 and 30,000 vehicles per day, TLTWO maintained a minimum of 2 months, and a four-lane highway. In addition, sites that satisfied these criteria and that used the New Jersey concrete barrier to separate traffic were of interest as comparison sites.

One site was selected for use of the temporary asphalt median. Additional study sites were not found because of infrequent use of TLTWO, which limited the number of potential sites, and reservations by VDOT to using the median.

The temporary asphalt median for TLTWO was selected for installation on the eastbound approach of a U.S. primary

VIRGINIA DEPARTMENT OF HIGHWAYS AND TRANSPORTATION
SPECIAL PROVISION FOR
TEMPORARY ASPHALT MEDIAN

October 29, 1986

I. Description:

This work shall consist of the construction, maintenance and removal of a temporary asphalt median for maintenance of traffic.

II. Materials:

(a) Asphalt median shall be Type I-2 bituminous concrete conforming to Section 212 of the Specifications.

(b) Raised pavement markers shall conform to Section 243 of the Specifications, except both sides of the pavement marker shall be yellow.

(c) Tubular pavement markers shall be from the Department's approved products list.

III. Construction Methods:

The bituminous materials shall be placed and compacted, on a clean pavement surface without using a tack coat, at the locations and to the dimensions shown in the provisions or as directed by the Engineer.

Drainage openings shall be 12 inches in length and spaced at 300 foot intervals or as directed by the Engineer.

The Department will paint the temporary asphalt median, before the Contractor installs the tubular and raised pavement markers; installation shall be in accordance with the provisions and manufacturer's recommendations.

The Contractor shall maintain the temporary asphalt median until its removal is required and replace any missing or damaged tubular or raised pavement markers within 24 hours of notification by the Engineer.

IV. Method of Measurement:

Temporary asphalt median will be measured in units of linear feet.

Tubular pavement markers will be measured in units of each.

V. Basis of Payment:

Temporary asphalt median will be paid for in units of linear feet, complete-in-place, which price bid shall include furnishing, placing and maintaining raised pavement markers, removal of temporary median and markers and all materials, labor, tools, equipment and incidentals necessary to complete the work.

Tubular pavement markers will be paid for in units of each, complete-in-place, which price bid shall include furnishing, placing and removal of tubular pavement markers and for all materials, labor, tool, equipment and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	Pay Unit
Temporary Asphalt Median	Linear Foot
Tubular Pavement Markers	Each

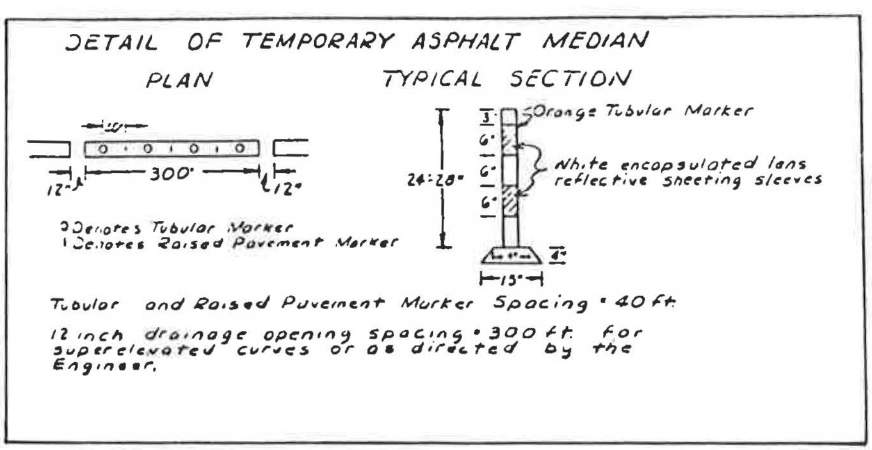


FIGURE 2 VDOT special provision.

route during reconstruction of the westbound bridge across a river. The study site is a four-lane, divided, rural highway section. A unique feature of this site is the presence of an intersection with a two-lane local road within the TLTWO section. The vertical alignment was relatively level. There was a slight horizontal curve near the western end of the TLTWO. The temporary asphalt median was used from the beginning of the crossover transition through the tangent section to the beginning of the exiting crossover transition.

Field Evaluation

Median Installation

A temporary asphalt median was installed on August 10–11, 1987. The length of the median was 2,280 ft. About one-half of the median was installed on the first day before the asphalt curb machine malfunctioned. It was noted that the asphalt mixture (VDOT Type I-2) was crumbling. Consequently, on the second day, a finer, modified Type I-2 asphalt mixture was used. Four workers were involved in the median installation: (a) the curb machine guider, (b) the dump truck driver, (c) the monitor of the asphalt entering the machine, and (d) the inspector of the median and cleaner of loose asphalt (Figure 3). The asphalt median was installed at a rate of 420 ft/hr.

After a section of the median was installed, the median was painted yellow with a spray gun, and reflectorized glass beads were manually spread on the top of the median. Three problems were encountered in the paint process: (a) the manual painting with the spray gun was slow, (b) the paint was absorbed by the hot asphalt, and as a result (c) the glass beads did not adhere to the paint. Therefore, there was little or no reflectorization from the paint. Next, raised pavement markers and tubular markers were installed on the median. The two markers were alternated every 40 ft.

Traffic Data

The ADT was about 7,000 vehicles, 87.7 percent of which were passenger cars and long, two-axle, four-tire vehicles; 5.4 percent were two-axle vehicles with six tires or three- or four-axle vehicles; and 6.7 percent were vehicles with five or more axles. The 85th percentile speeds for westbound and eastbound approaches were 48 mph and 58 mph, respectively. The speed of the traffic on the westbound approach may have been lower because it was the approach that crossed over the existing median and the data collection point was about 200 ft from the first crossover. Data collected on lateral placement and headway were omitted because of an equipment malfunction.

Monitoring TLTWO

The VDOT project inspector monitored and recorded the activities related to work zone traffic control (Figure 4). The contractor maintained the TLTWO. A summary of the incidents is presented in Table 1.

FIGURE 3 Median installation process.

There were nine incidents in which a total of 23 tubular markers were hit by vehicles, primarily farm machinery. Cold weather and wind appeared to cause six tubes to break, and hot weather resulted in the bending of five tubes. A total of 34 tubular markers were replaced. Initially, 29 tubular markers were installed and 4 more were added to mark the ends of the median for snowplows. The replacement rate for the tubes was about 100 percent. Four of the six incidents of Type II barricades being hit in the transition occurred in the first five weeks of TLTWO. Accidents at the intersection occurred throughout the duration of the project. Accidents are discussed in the next section. From the tire marks, it appeared that in one incident an eastbound vehicle drove onto the right shoulder and then crossed over the median. It is not known whether this incident was intentional. This was the only incidence of a vehicle crossing the median, and it occurred about 2 months after installation. No damage was reported.

Accidents

Between August 11, 1987, and August 8, 1988, 11 accidents occurred during TLTWO: 9 angle accidents at the intersection; 1 overturn, alcohol-influenced accident; and 1 run-off-the-road (ROR) accident. There were five injury accidents and six property damage accidents. None of the accidents involved a fatality.

The following trends were noted: (a) 7 of the 11 accidents occurred during the daytime on the weekend, (b) 9 of the 11 accidents occurred during the daytime with clear weather, (c) 5 of the 9 angle accidents involved drivers aged 60 or over failing to yield (3 of the 9 angle accidents involved a driver aged 79 or over failing to yield the right-of-way), and (d) 7 of the 9 angle accidents involved a westbound vehicle on the U.S. primary route and a northbound vehicle on the local road.

In the 1-year period before the installation of TLTWO—August 11, 1986, through August 9, 1987—there were no accidents. Two years before, there were three accidents, one angle at the intersection and two ROR accidents. In the third

Route 360 King & Queen and King William Counties
Approaches and Bridge over Mattaponi River

WORK ZONE TRAFFIC CONTROL LOG

Month and Year: ______________________

Name and Title: ______________________

Date	Damage Description	Location (see reverse)	Action Taken and Date

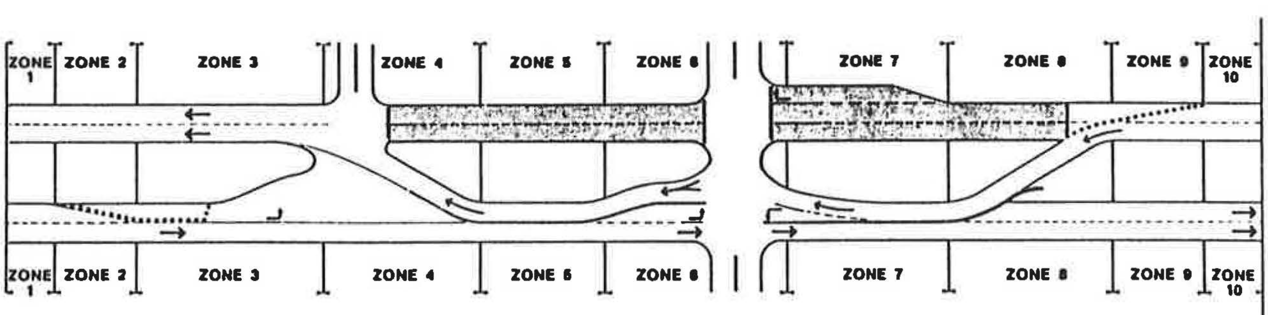


FIGURE 4 Monitoring form.

TABLE 1 INCIDENTS BY FREQUENCY OF OCCURRENCE

Type of Incident	Number of Incidents
Tubular markers broken/damaged	11
Type II barricades hit	6
Accidents at the intersection	5 (9 based on accident reports)
Type III barricades or barrels vandalized	2
Warning signs hit	2
Type II barricade stolen	1
Vehicle crossed median	1
TOTAL	28

year before, there was one accident, an angle accident at a store entrance.

Intersection control beacons (flashing overhead caution signals) were installed at the intersection several years before construction as a countermeasure to reduce intersection accidents. The beacons were removed during construction because the support poles were in conflict with temporary pavement construction. Maintaining intersection control beacons during construction may have resulted in a lower accident frequency.

In the traffic control planning phase of this project, there was some concern about the presence of this intersection within TLTWO. A special effort was made to provide temporary and permanent warning signs on the approaches of the local road and delineation of travel through the intersection.

It is suspected that violations in driver expectancy at intersections of four-lane divided roads may have contributed to the seven angle accidents involving a westbound vehicle on the U.S. primary route and a northbound vehicle on the local road. Usually, the driver of a northbound vehicle stops at the intersection, looks left (for eastbound vehicles) to see if it is safe to proceed, proceeds to the median opening, and stops and looks right (for westbound vehicles) to see if it is clear. But with TLTWO on the eastbound approach, it is necessary for the driver of a northbound vehicle to look both left and right to see if it is clear before entering the intersection.

Some older drivers have some difficulty at intersections with information processing and decision making (*4*); these difficulties are compounded by the presence of TLTWO.

It is further suspected that the conditions of the intersection would be worse with the use of the concrete barrier instead of the median because the higher barrier would further restrict sight distance at the intersection.

Initially, a before-after accident study with a comparison group was planned. This study was eliminated because the lack of accidents 1 year before reconstruction would have resulted in division by zero in the analysis (*5*).

Because the accident experience during reconstruction was high, it appears that the accidents were connected with the presence of an intersection within the TLTWO. There is no evidence to suggest that the temporary asphalt median directly contributed to any of the accidents.

Removal of the Median and TLTWO

TLTWO ended on August 8, 1988, with the opening of the new westbound bridge. The median was removed on August 8–9, 1989, in three phases (Figure 5): a front-end loader was used to push the asphalt median into a pile toward the left shoulder, a tractor with a sweeper attachment swept the loose asphalt from the travel lane toward the shoulder, and a second front-end loader loaded the asphalt from the median onto a dump truck for transport to a storage area. During the first two phases, the second front-end loader was removing the median crossover pavement.

FIGURE 5 Median removal: *top*, front-end loader removing median; *bottom*, sweeping loose gravel.

The median was removed under traffic conditions. The front-end loader operator moved the median out of its position and into the closed left lane during gaps in the traffic stream. The median was then pushed into piles about 500 ft apart. The average rate of removal by the loader was 606 ft/hr (with a standard deviation of 144 for four samples).

Field Evaluation Finding

The temporary asphalt median performed well at the study site.

Comparative Analyses

Two comparative analyses were made between the temporary asphalt median and the New Jersey concrete barrier for TLTWO: (a) installation and removal rate and (b) cost.

Installation and Removal Rate

Using an asphalt curbing machine, the temporary asphalt median was installed at a rate of 420 ft/hr. The concrete barrier was installed at a rate of 200 ft/hr based on a study on a four-lane interstate highway on September 1, 1987. The temporary asphalt median can be installed about twice as fast as the concrete barrier. Therefore, the time that traffic is exposed to installation activities is about 50 percent lower for the median than for the barrier. The rates are not for a complete installation because painting the median, installing markers for the median, installing warning lights on panels, and painting the temporary pavement marking adjacent to the barrier are not included. Because these activities are typically done concurrently with the median or barrier installation, it is expected that the additional time would be relatively small.

The removal rate for the median was 606 ft/hr. It is estimated that the removal rate for the barrier is equal to the installation rate of 200 ft/hr because the procedure is reversed. If the removal rate for the median is reduced to 450 ft/hr to allow for complete removal, this will mean that the median can be removed 2½ times faster than the barrier, thus reducing exposure to traffic by 60 percent.

Cost

The contract price for the complete installation and removal of the temporary asphalt median was $10.00 per linear foot. Because the average cost of the concrete barrier is $10.00 to $11.00 per linear foot, there was no difference in cost in Virginia. The tubular markers were priced at $50.00 each. Average costs for temporary asphalt median projects in other states are shown below.

State	*Asphalt Median Cost ($/linear foot)*	*Unit Cost ($/tubular marker)*
Pennsylvania	2.10	18.00
Ohio	2.35	18.70
West Virginia	7.18	34.00

In every case, especially for Pennsylvania and Ohio, the costs are substantially lower than for Virginia. It is expected that the prices will decrease as VDOT uses more temporary asphalt medians. In the second project in Virginia using a median, the contract price was $6.00 per linear foot. This parkway project on a six-lane, divided highway in a suburban area was initiated near the completion of this study.

In the initial project plans, four G.R.E.A.T. impact attenuators were proposed for the two ends of the concrete barrier and at both sides of the break in the barrier at the intersection. At a unit cost of $20,000 for the impact attenuators, $80,000 was saved by eliminating them when the temporary asphalt median was chosen over the concrete barrier. In addition, sight distance at the intersection was improved with the use of the temporary asphalt median instead of the concrete barrier.

Development of Guidelines

The second project objective was to develop guidelines for the temporary asphalt median, if appropriate. The Location and Design Division was directed to develop guidelines for the use of temporary asphalt medians to provide information and instruction and to promote use of the median. The guidelines were developed with input from the Traffic Engineering Division and the principal investigator of this research. Consequently, instead of developing separate guidelines for the temporary asphalt medians in this research, this researcher reviewed the Location and Design Division's guidelines and special provision and made comments and suggestions.

Guidelines

The VDOT's guidelines for the use of temporary asphalt medians are shown in Figure 6. The guidelines consist of two parts: general notes and a detailed drawing of the temporary asphalt median.

Considering the median for TLTWO with traffic volumes between 4,000 and 15,000 vehicles per day is restrictive compared with the approach of other state DOTs. However, this restriction reflects VDOT's cautious approach to using the median. Although it is mostly used on four-lane divided roads, the median in TLTWO is suitable for use on four-lane undivided roads. The guidelines do not address four-lane, two-way operations (FLTWO). However, FLTWO was used on the parkway project; therefore, FLTWO should be mentioned in the guidelines. The decision to use the asphalt median is made on a project-by-project basis, typically using traffic analysis methods. The volume guidelines are not very useful compared with the traffic analysis. Therefore, volume guidelines may be omitted.

From the experience at the study site, the following two suggestions are noted:

1. Tubular markers should be placed at the ends of the median to delineate them for snow removal activities.
2. Special attention should be given to traffic control at an intersection that is within a TLTWO, especially the side street approaches of the intersection. Special attention may include extensive warning signing, supplemental pavement markings, and intersection control beacons.

May 16, 1988

ERRATA SHEET

INSTRUCTIONAL AND INFORMATION MEMORANDUM
LD-87(D) 93.8
CONSTRUCTION ZONE SAFETY
Sheet 7 of 10 (7 sheets added)

This revision is to add, under the subheading GENERAL, the following guidelines for the use of Temporary Asphalt Medians and for the use of Police Patrols in construction zones:

Temporary Asphalt Medians

- Temporary asphalt medians are to be considered on two-lane, two-way temporary detours for traffic volumes between 4000 and 15,000 VPD.
- Each location is to be reviewed and have the joint approval of applicable District, Traffic Engineering and Location & Design personnel.
- Each location should use geometrics that provide an operating speed equal to that of the existing roadway, where possible, to minimize operational problems. (See Standard GS-10)
- The SEQUENCE OF CONSTRUCTION/TRAFFIC CONTROL PLAN is to include the required temporary asphalt median layout details along with the included "DETAIL OF TEMPORARY ASPHALT MEDIAN" that is available in the CADD SECTION for inclusion in the plans.
- Payment will be made under:

Pay Item	Pay Unit	Item Code
Temporary Asphalt Median	Lin. Ft.	24285
Flexible Post Delineator	Each	24286

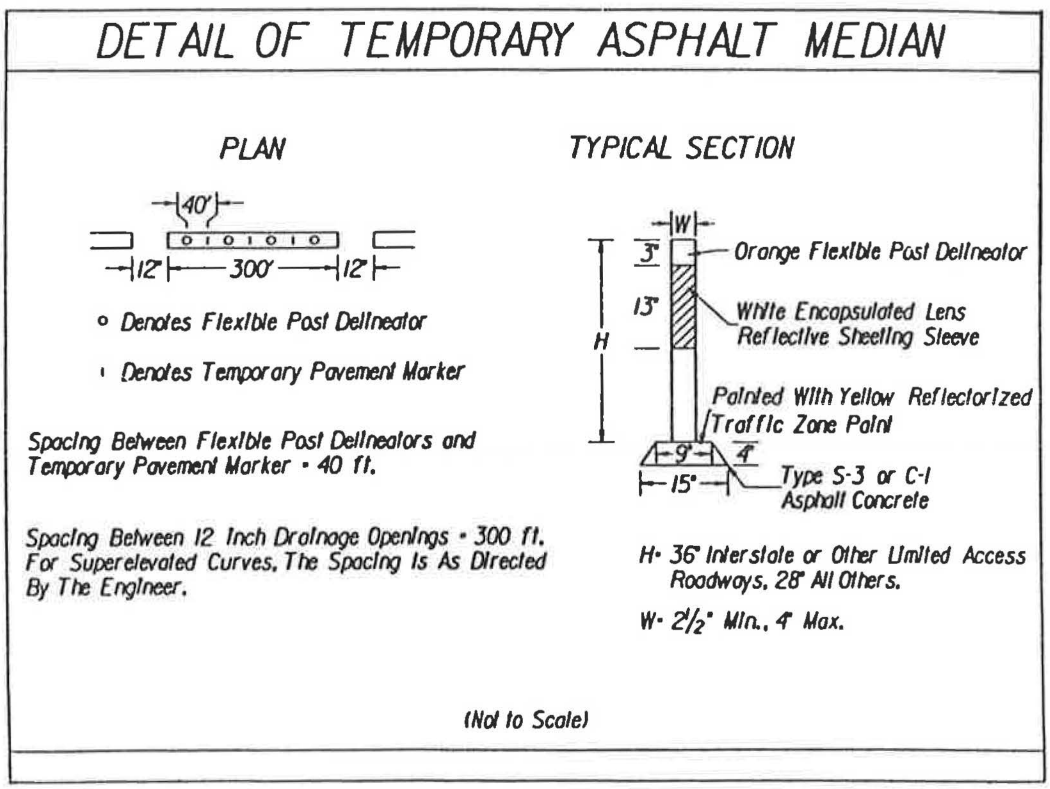


FIGURE 6 VDOT guidelines for use of a temporary asphalt median.

Special Provision

From the experiences at the study site and this review, additional notes and changes on field practices for the installation, maintenance, and removal of the median are suggested below.

- For better quality, faster application, and better reflectivity, the median should be painted with a paint truck instead of manually with a spray gun. (At maximum height the paint truck carriage can apply a 10-in. swath of paint and glass beads, so two passes are necessary.) The hot asphalt median should be allowed to cool before painting for better paint adhesion and less paint absorption. Other options to consider are the use of (a) temporary pavement marking tape on the side of the median and paint on the top and (b) raised pavement markers on the side of the median to supplement the paint.
- Some districts prefer that the contractor instead of VDOT be responsible for painting the median. In some districts, much of the painting related to construction, as well as other construction-related traffic activities, is done by contract. This not only allows VDOT traffic forces to focus on maintenance activities but also relieves them of tying up a paint crew that is dependent on the contractor's schedule. It is suggested that the district determine whether VDOT or the contractor will paint the median.
- The two 6-in. reflective sleeves on the tubular marker should be replaced by the option of a 13-in. reflective sleeve (as shown in the guidelines) or a 6-in. (at the top) and 4-in. sleeve spaced 2 in. apart. The 13-in. sleeve was recommended based on a study to optimize the tubular marker design (*6*), whereas the latter option is in accordance with a recent change in the *Manual on Uniform Traffic Control Devices* (*7*).
- The contractor should be encouraged to use efficient methods for the installation and removal of the median.

These VDOT guidelines and the special provision should be expected to change as VDOT gains experience with the temporary asphalt median. To aid in the evolution of the guidelines and the special provision, it is necessary to document each use of the temporary asphalt median by VDOT. The report should include

1. The project title and location;
2. The time period of median use and the location of the TLTWO;
3. The contract price for the median and tubes;
4. Median installation and removal methods;
5. A description of any deviations from the guidelines, including the reason and the result;
6. A general description of incidents and accidents during TLTWO;
7. A description of any problems encountered and their solutions; and
8. The name, address, and telephone number of the project inspector or person submitting the report.

The report should be submitted to the Traffic Engineering Division.

CONCLUSIONS

The following conclusions can be drawn from this study:

- The temporary asphalt median was installed two times faster than the concrete barrier, thereby reducing the time traffic was exposed to installation activities by about 50 percent.
- The median was removed roughly 2½ times faster than the concrete barrier, thereby reducing the time traffic was exposed to removal activities by about 60 percent.
- There was no difference in the cost per linear foot between the median and the concrete barrier because of the relatively high contract price for the median ($10.00 per linear foot) compared with median costs in other states (28 to 79 percent less). However, an $80,000 savings was achieved with the median by eliminating the need for impact attenuators at concrete barrier end sections. It is expected that the cost will decrease as VDOT uses more medians. (The median cost per linear foot was 40 percent lower on a recent project.)
- There is no evidence to suggest that the temporary asphalt median directly contributed to any accidents. However, it does appear that several accidents can be attributed to the presence of an intersection within TLTWO.
- The temporary asphalt median performed well at the study site.
- VDOT has a special provision and guidelines for the use of temporary asphalt medians for TLTWO. Suggestions were made to improve these items.

ACKNOWLEDGMENTS

The research reported here was financed from Highway Planning and Research funds administered by the Federal Highway Administration.

REFERENCES

1. *Roadside Design Guide*. AASHTO, Washington, D.C., 1989.
2. *Temporary Asphalt Islands for Maintenance of Two-Way Traffic Through Construction Zones*. FHWA, U.S. Department of Transportation, 1983.
3. *Design Considerations for Two-Lane, Two-Way Work Zone Operations*. Report FHWA/RD-83/112. FHWA, U.S. Department of Transportation, 1983.
4. *Special Report 218: Transportation in an Aging Society: Improving Mobility and Safety for Older Persons*. Vol. 1. TRB, National Research Council, Washington, D.C., 1988.
5. *Evaluation of Highway Safety Projects: Procedural Guide*. FHWA, U.S. Department of Transportation, 1979.
6. R. F. Pain, H. W. McGee, and B. G. Knapp. *NCHRP Report 236: Evaluation of Traffic Controls for Highway Work Zones*. TRB, National Research Council, Washington, D.C., 1981.
7. *Manual on Uniform Traffic Control Devices*. FHWA, U.S. Department of Transportation, 1978.

The opinions, findings, and conclusions are those of the author and not necessarily those of the sponsoring agencies.