TRANSPORTATION RESEARCH

RECORD

No. 1419

Highway and Facility Design

Roadside Safety Features and Landscape and Environmental Design

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
WASHINGTON, D.C. 1993

Transportation Research Record 1419
ISSN 0361-1981
ISBN 0-309-05567-9
Price: $29.00

Subscriber Category
IIA highway and facility design

TRB Publications Staff
Director of Reports and Editorial Services: Nancy A. Ackerman
Associate Editor/Supervisor: Luanne Crayton
Associate Editors: Naomi Kassabian, Alison G. Tobias
Assistant Editors: Susan E. G. Brown, Norman Solomon
Office Manager: Phyllis D. Barber
Senior Production Assistant: Betty L. Hawkins

Printed in the United States of America

Sponsorship of Transportation Research Record 1419

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
Chairman: Charles T. Edson, Greenman Pederson

General Design Section
Chairman: Hayes E. Ross, Jr., Texas A&M University System

Committee on Roadside Safety Features
Chairman: William W. Hunter, University of North Carolina
Robert F. Baker, Maurice E. Bronstad, James E. Bryden, John F. Carney III, Julie Anna Cirillo, Owen S. Denman, Arthur M. Dinitz, John C. Durkos, Vittorio Giavotto, Don Jay Gripne, Malcolm D. MacDonald, Mark A. Marek, Charles F. McDevitt, J. T. Peacock, David H. Pope, Richard D. Powers, Robert Quincy, Hayes E. Ross, Jr., Larry Scofield, Robert K. Seyfried, Rudolph Kenneth Shearin, Jr., Dean L. Sicking, Roger L. Stoughton, E. Debra Swanson, Harry W. Taylor, Thomas Rubell, Tal-Lun Yang

Committee on Landscape and Environmental Design
Chairman: David H. Fasser, New York State Department of Transportation
Secretary: Barbara M. Schaedler, Brentwood, New York
E. Leroy Brady, Nicholas R. Close, Jerrold S. Corush, Lawrence E. Foote, Eugene Johnson, Edward N. Kress, Harlow C. Landphair, Charles R. Lee, Derek Lovejoy, Mark D. Masteller, Teresa Mitchell, Charles P. Monahan, Bob Moore, Carroll L. Morgenson, Barbara Petrarca, Charles R. Reed, James F. Ritzer, Elga Ronis, Jacquelyn A. Ross, Armen S. Sardarov, Richard C. Smardon, Grady Stem, Howard R. Wagner

Daniel W. Dearasaugh, Jr., Transportation Research Board staff

Sponsorship is indicated by a footnote at the end of each paper. The organizational units, officers, and members are as of December 31, 1992.

Transportation Research Record 1419

Contents

Foreword v

Development and Proving Tests of a Four-Rope Safety Fence 1
I. B. Laker and A. W. Naylor

Performance of Cable Guiderail in New York 9
Wei-Shih Yang, Luis Julian Bendaña, Nicholas J. Bruno, and Wayne D. Kenyon

Performance Level 1 Bridge Railings for Timber Decks 21
Ronald K. Faller, Michael A. Ritter, James C. Holloway, Brian G. Pfeifer, and Barry T. Rosson

Vehicle Crash Tests of Concrete Median Barrier Retrofitted with Slipformed Concrete Glare Screen 35
Payam Rowhani, Doran Glauz, and Roger L. Stoughton

Small Sign Support Investigation 43
Dwight Metcalf, Yee-Jung Ho, and Sylvester A. Kalevela

Analysis of Guardrail-End Accidents in Oklahoma 52
J. L. Gattis, John P. Varghese, and Larry E. Toothaker

Comparative Performance of Barrier and End Treatment Types Using the Longitudinal Barrier Special Study File 63
William W. Hunter, J. Richard Stewart, and Forrest M. Council

Attempt To Define Relationship Between Forces To Crash-Test Vehicles and Occupant Injury in Similar Real-World Crashes 78
Forrest M. Council and J. Richard Stewart

Vehicle Crash Tests of Type 115 Barrier Rail Systems for Use on Secondary Highways 86
John Jewell, Roger L. Stoughton, and Doran Glauz

North Central Expressway, Dallas: Case Study of Enhancement-Inclusive Urban Freeway Design 95
John P. Kelly and Rick J. Robles

Visual Prioritization Process 109
Lola Eileen Mason

Use of Computer Animation Technology for the Development of Interpretive Facilities on the San Juan Skyway 119
James L. Sipes and Richard F. Ostergaard

Issues in Planning and Design of Scenic, Recreational, and Parkway Roads 127
Ramey O. Rogness

Foreword

TRB Committees A2A04, Roadside Safety Features, and A2A05, Landscape and Environmental Design, sponsored three sessions during the 1993 Annual Meeting of the Transportation Research Board. The first nine papers in this Record were presented in sessions entitled "Roadside Safety Features, Parts 1 and 2." Laker and Naylor describe the development and approval testing of an improved design of a wire rope safety fence. Agencies whose cable guiderails suffer from loss of tension will be interested in the study by the New York State Department of Transportation to investigate the causes of tension loss and to formulate corrective measures, presented by Yang et al. Faller et al. report on a project to develop and test three crashworthy bridge-railing systems for use with longitudinal timber bridge decks. Rowhani et al. discuss vehicle crash tests that were conducted on a concrete median barrier retrofitted with a slipformed concrete glare screen, and Metcalf et al. report on investigations of a generic small sign support system developed by the Arizona Department of Transportation.

Gattis et al. document attributes associated with accidents in which vehicles have struck a variety of guardrail end sections, mostly exposed or turned down. Hunter et al. recount the findings of an investigation comparing the performance of various types of guardrails, median barriers, and end treatments. Council and Stewart explore the relationship between occupant risk as measured in crash tests and the real-world measure of risk: actual occupant injury. Jewell et al. report on three vehicle crash tests of a tubular bridge-rail system developed in California.

These papers are followed by three papers presented in a session entitled "Highway Aesthetics and the Enhancement Provisions of ISTEA: Looking to the Future and Lessons from the Past." Kelly and Robles explain the "enhancement-inclusive" design approach taken for the total urban freeway reconstruction of the North Central Expressway in Dallas. Mason describes the preliminary development of the visual prioritization process, a planning and implementation guide for prioritizing units and visual elements along a corridor for mitigation and funding. Sipes and Ostergaard use computer animation technology to simulate driving or walking through the interpretive facilities along the San Juan Skyway in southwest Colorado.

In a paper not presented at the TRB Annual Meeting but approved for publication by Committee A2A05, Rogness focuses on recreational and scenic roads and presents the issues and difficulties involved in retaining the intent and function of these facilities when designing their construction or reconstruction.

Development and Proving Tests of a Four-Rope Safety Fence

I. B. LAKER AND A. W. NAYLOR

Early two-rope safety fence needed to be installed on a hardened running surface to avoid undulations in the terrain causing vehicles to contact the fence at varying heights, with the consequent risk of ropes slipping over the car hood or being run over. A new design uses four ropes at two heights with the lower pair of ropes interwoven between the posts. Standard U.K. tests with a 1500-kg car impact at 113 km/hr (70 mph) and 20 degrees showed that this design met U.K. Department of Transport regulations. Further tests with a 750-kg car, at the same speed and angle, demonstrated that rope heights are no longer critical: the fence can now be installed on nonhardened surfaces, thereby reducing the unit costs of installation. Where on-road space for installation of safety fences is restricted, post separations may be reduced from 2.4 m to 1.0 m; this reduces the maximum penetration, under standard impact conditions, from 1.7 to 1.2 m. Analysis of impact severity using the theoretical head impact velocity concept showed the four-rope fence to have impact severity characteristics that match the current U.K. design of semiflexible fence. Other advantages of the fence are that the ropes do not require replacement or retensioning after vehicle impact and that damaged posts are easily removed from ground sockets and replaced with new posts.

Bridon PLC collaborated with the U.K. Transport and Road Research Laboratory in the 1960s on the development of a weak post-and-wire-rope safety fence for the containment of private cars. After a sequence of tests and prototype development, the final design consisted of two wire ropes, mounted above ground at approximately the same height, resting freely in a vertical slot cut into the top of steel posts. The weak-post concept avoided the then common problem of vehicles snagging on posts and spinning out of the fence in a hazardous manner.

The early tests had shown that the performance of the single-height wire rope safety fence was sensitive to rope height above the surrounding surface (*1*). If the ropes were too low they could be ridden over, and if too high they could slide over the hood of a small car. To overcome this, a restricted number of tests were completed with a two-height rope fence, but the tests proved unsuccessful. There was a tendency for the rear lower rope either to be trapped and carried to the ground by the posts or to break free too soon from its attachment to the post, fall to the ground, and then be run over by the vehicle.

The solution adopted at that time, and included in the U.K. Department of Transport (DTp) Technical Memorandum H9/73 (issued in 1973), was to retain the single-height, two-rope system in slotted posts and overcome the rope height problem by ensuring that the wire rope fence was always installed on a hardened running surface. This of course added to the cost, and in consequence, very little single-height wire rope fence was installed on U.K. highways. The longest length is in place on the M62 Motorway across the Pennine Mountains. This road is subject to snow drifting, and one of the benefits of the wire rope fence is that its narrow profile reduces the tendency for drifts to form.

Eventually, because of low demand caused by the high cost of preparing a hard running surface on which to mount the fence, the single-height rope fence was dropped from the U.K. DTp regulations. Nevertheless, the fence has had considerable use overseas, in Europe and the Middle East, where its ability to limit snow and sand drifting has been a most attractive feature.

DEVELOPMENT OF DOUBLE-HEIGHT, FOUR-ROPE SAFETY FENCE

Bridon PLC, in 1986, decided to reexamine the single-height design; its prime aim was to overcome the need to provide a hardened running surface and in so doing reduce the overall cost of the fence installation. To achieve this objective a series of 10 development tests were carried out at the Motor Industry Research Association (MIRA) in the United Kingdom.

First, a second pair of ropes was added (Figures 1 and 2). The distance between the upper and lower ropes and their heights above the ground were selected to permit vehicles to traverse an undulating surface and hit the fence without risk of a rope slipping over the hood. The posts may be held in socketed footings for easy replacement or repair, or soil-mounted posts can be used.

The use of this fence on U.K. roads has been approved by the U.K. DTp in Departmental Standard TD 32/89, which contains detailed drawings of the fence and components. The design has been submitted to and approved by the European Commission and member states of the European Economic Community; it is a patented product available worldwide.

Standard Impact Tests

The standard impact conditions for the testing of safety fences and barriers in the U.K. are quoted in British Standard BS6579 and in DTp Departmental Standard TD32/89. In the United Kingdom, for roads that have a maximum speed limit of 113 km/hr (70 mph), the fence should contain and safely redirect a 1500-kg, 113-km/hr vehicle impact at 20 degrees.

I. B. Laker, Road Safety Consultants, Brockendene, Foxhill, Farley Hill, Berkshire RG7 1UH, United Kingdom. A. W. Naylor, Bridon Ropes Ltd., Carr Hill, Doncaster, South Yorkshire DN4 8DG, United Kingdom.

In addition to the standard test, the U.K. DTp requested of the Bridon design that the maximum dynamic deflection of the fence be less than 2.0 m. Also, the vehicle exit trajectory had to meet the "box" criteria of BS6779, which state in part

> [I]f redirection takes place the vehicle shall be redirected so that no part of it crosses the line drawn parallel with and 2.13 metres, plus the width of the vehicle, from the face of the parapet, within a distance of 10 metres from the break point of vehicle contact with the parapet (fence). The test vehicle should neither turn on its side nor roll over the paved parapet test area.

Bridon Ropes Ltd (a subsidiary of Bridon PLC) was asked to meet all these requirements in the design of its vehicle safety barrier.

One of the main purposes of the tests was to demonstrate that the new four-rope design could perform successfully when mounted on an uneven surface.

The material specification for the running surface over the impact test area led to considerable discussion. On-road sites, where safety barriers are installed, have a wide range of surfaces—from grass, to aggregate, to hardened macadam—all with varying degrees of undulation and hardness. Repeatability of tests was a prime consideration. The solution adopted, in part, followed previous practice using a hardened running surface for the standard 1500-kg saloon car test; this was followed by a second test, with a 750-kg minicar. The smaller, lower profile of the 750-kg car would, to a reasonable degree, represent a standard vehicle that had either penetrated into a soft running surface or was traversing an undulating surface where a rope could slip over the hood when the car was at the lowest part of the undulation. This test would also explore the impact severity and vehicle trajectory response of the fence with a lightweight car.

Design of Four-Rope Safety Fence

Post Spacings and Rope Heights

Bridon Ropes Ltd considered that fence post spacings of 2.4 m would be needed to meet the 2.0-m maximum dynamic deflection criteria, requested by the DTp, when tested under standard impact conditions with the 1500-kg car.

It was foreseen that a stiffer fence would be required where the fence, in an on-road situation, was installed close to roadside features such as lamp columns and gantry signs. To meet this situation, tests were made on a fence with post spacing reduced from 2.4 m to 1.0 m.

All designs were dynamically tested by impact with driverless 1500-kg and 750-kg cars, at a target impact speed of 113 km/hr at 20 degrees.

In the final design, posts of z-shaped cross section were manufactured from 6-mm gauge steel having a yield strength of 335 N/mm^2. A short slot supported the two upper ropes at a height of 585 mm; the lower ropes were supported by small brackets each side of the z-posts at a height of 490 mm Figures 1 and 2). The posts were held in concrete sockets with sufficient clearance for easy removal and replacement of damaged posts.

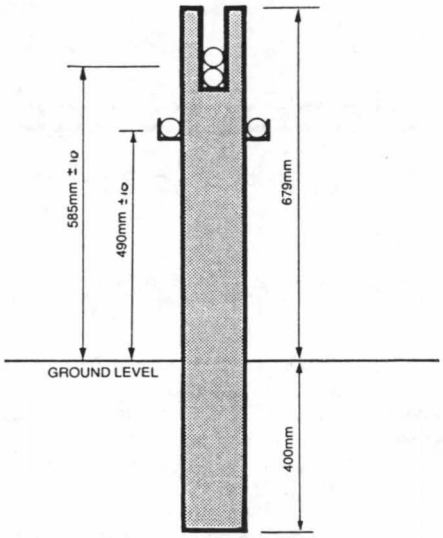


FIGURE 1 Z-section slotted post.

Preliminary Tests

A series of nine tests was carried out during 1987 and 1988; they were followed by two more tests in 1991 on a stiffer fence designed for lower deflection on impact.

In the first of these series, four ropes were placed in pairs in a deep single slot cut into the top of the post. The upper pair was at a height of 635 mm and the lower pair was at 400 mm. On impact by a 1500-kg vehicle, this design failed: the flanges of the slots fractured before the base of the posts started to bend. Without the retaining flanges the ropes were free to break free from the posts ahead of the test vehicle and fall to the ground, where they were subsequently run over. Although the vehicle was contained, this design was abandoned.

In the second 1500-kg car test, the static rope tensions were increased from 13 to 27 kN. Again the vehicle was contained but the lower ropes were run over. This test produced the unexpected result that the higher static rope tension had little influence on the dynamic fence deflection: it increased from 3.1 to 4.9 m.

FIGURE 2 Four-rope safety fence.

The design adopted in all later tests used two ropes located in a shallow slot in the top of the posts, with two lower ropes placed on simple brackets fixed to each side (Figure 1).

In the third test, rope heights were again at 635 and 400 mm, with static tensions 31 and 13 kN in the upper and lower pairs of ropes. In addition, the lower pair was interwoven in a crisscross fashion between every second post. This in effect produced a simple mechanism that trapped the rope between posts and so maintained rope height ahead of the vehicle. The 1500-kg vehicle was contained and the penetration was reduced to 2.4 m, but one rope was run over.

In the fourth test the bending strength of the posts was increased by increasing the material thickness from 5 to 6 mm. The rope heights were 635 and 490 mm with static tensions of 31 kN (upper) and 27 kN (lower). The 1500-kg vehicle was safely contained and redirected in a maximum deflection of 1.8 m.

The fifth test was with a 750-kg car. Rope heights were increased by 50 mm to 685 and 490 mm; rope tensions were the same as the previous test. The maximum deflection was 1.1 m. The vehicle was contained and redirected but the upper ropes slipped over the bonnet. The vehicle path remained close to the fence and the car made second contact; as it came to rest it rolled onto its side, partly because of the impact damage to the front left-side wheel station.

The fifth test was repeated with the upper ropes lowered by 50 mm to 635 mm and the static rope tension reduced to 22 kN. The car was contained by both pairs of ropes, safely redirected, and made second contact with the fence about 60 m from the impact point. The vehicle did not roll over.

In the seventh test the lower pair of ropes were interwoven between every post (Figure 3), rather than every other post as in the previous tests. The purpose was to improve the retention of rope by trapping it against the posts. Together with this modification, the rope heights were reduced by 50 mm, making the new heights 585 and 490 mm. Clearly the lower rope heights made it possible for the cables to be run down. However, if the extra interweaving proved effective and the ropes maintained their heights during impact, the overall lower rope heights would certainly represent an improved configuration and be beneficial for the smaller car. Static tension in all ropes was 22 kN.

This test proved very successful. The configuration of rope heights (585 and 490 mm), post material thickness (6 mm), and static rope tension (22 kN) was retained in the proving tests of the four-rope safety fences with 2.4- and 1.0-m post spacings. The proving tests of the 2.4-m fence and the 1.0-m fence by impact with a 1500-kg car and a 750-kg car are described in detail in the next section.

PROVING TESTS OF FOUR-ROPE SAFETY FENCE

Rope Characteristics

Vehicle impact tests showed that static rope tensions between 13.3 and 26.7 kN (3,000 and 6,000 lb) had little effect on dynamic deflection of the fence. This result is beneficial in service, in that the fence deflection performance is not sensitive to variations in tension brought about by changes in ambient temperature. A static tension value of 22.25 kN (5,000 lb), set at 15°C, is specified for the final design. For a temperature range of −10 to 30°C the rope tension ranges from 36.0 to 14.0 kN.

The ropes are 19 mm in diameter, zinc-coated with a minimum breaking load of 173.6 kN (17.7 T) each. The rope is formed by twisting six wires around a king wire to form a strand; three strands are twisted together to form the rope. The maximum tension recorded in all of the tests was 22.6 kN, and maximum length between anchorages is 626.4 m. Intermediate anchorage overlaps have been successfully designed and tested but are not the subject of this report (*1,2*).

Throughout the series of tests, there was no need to replace or retension any of the ropes; at the end of the program the ropes were in a condition suitable for on-road use. All initial

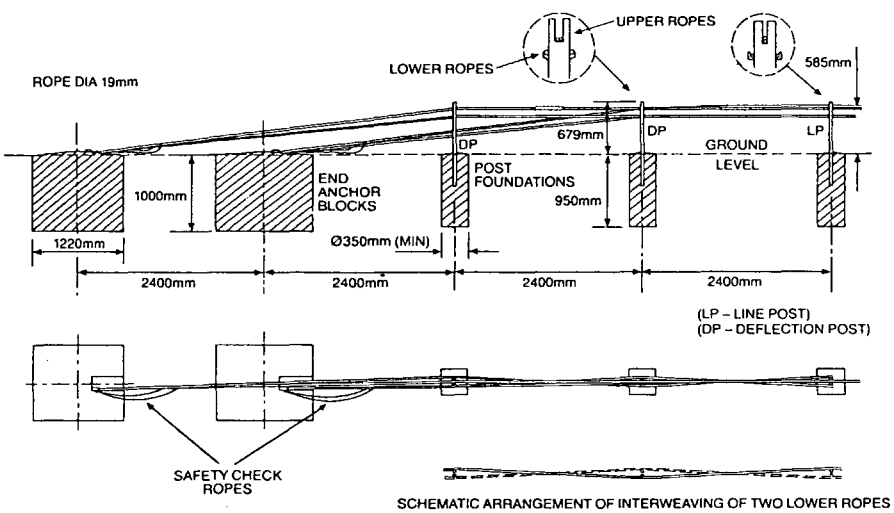


FIGURE 3 General arrangement of four-rope safety fence showing rope interwoven between very post.

impacts occurred at the same point along the fence. Ropes are prestressed (prestretched) before installation to remove all nonelastic stretch; this ensures that tension is maintained in the ropes after they are stretched under vehicle impact conditions.

Four-Rope Safety Fence with 2.4-m Post Spacings

Posts were spaced at 2.4-m intervals to span the impact area of 48 m. The length of rope between anchorages was 626.4 m; turnbuckles were adjusted to set the static tension in all four ropes at 22.24 kN. Rope heights were set at 585 and 490 mm. The lower pair of cables was interwoven at every post.

1500-kg Car Test

The standard weight (1500-kg) car test (Figure 4) at 113 km/hr (70 mph) impacted at an angle of 19 degrees, deflected the fence a maximum distance of 1.7 m, and it was safely redirected onto a departure path of 7 degrees to the line of the fence, with 0 yaw angle.

First contact was over a length of about 19 m. The driverless car then steered back, remained in contact with the fence for a further 19 m, and came to rest about 125 m from the first point of impact (Figure 4).

The main damage to the car was restricted to the area around the impact wheel station; the passenger compartment was undeformed, all four doors could be opened and closed, and all safety glass remained intact. The front left-side corner was pushed in about 300 mm. On the second impact, at about 60 m from impact point, the front left-side wheel station collapsed. The vehicle came to rest about 125 m from the impact point alongside, and just touching, the fence.

In the primary impact, nine posts were damaged over a length of 19.2 m; a similar length was damaged in the secondary impact.

The test complied with the requirements laid down by the U.K. DTp as well as the exit trajectory criteria stipulated in BS6779.

750-kg Car Test

The lightweight (750-kg) car test (Figure 5) is not a formal requirement of the U.K. DTp regulations for safety fences on highways; however, it was carried out, by request of the DTp, to observe whether the lighter car would either snag on the posts and spin out or be redirected after impact at a high angle, due to stored energy in the ropes being returned to it. Additionally, the test represented an impact on the fence of a lower profile car at a lower running height, a condition that could occur with a heavier vehicle on soft ground, both of which may result in the upper rope slipping over the hood.

The fence configuration was identical to the previous 1500-kg car test. The test speed and angle were 116 km/hr (72.1 mph) and 19 degrees. The maximum penetration into the fence was 1.2 m, and the damaged length was about 15 m. The car was safely redirected on an exit path of 7 degrees (center of gravity point) with the car at a yaw angle of about 1.5 degrees to the line of the fence.

On impact, the four wire ropes were forced together and made contact with the front left-hand corner of the car at headlamp height; there was no indication of a rope slipping over the hood. The front left-side wheel suspension damper was damaged, but the wheel remained attached to its upper and lower mountings. The occupant compartment was undeformed, all safety glass remained intact, and both doors could be opened and closed.

The maximum roll angle of 10 degrees occurred at 0.3 sec after impact; at this moment both left-side wheels were clear of the ground. As penetration decreased, the roll angle reduced to 0 degrees and the car left the fence in a stable condition. There was no indication of spinout.

Contact length was over a distance of about 15 m; seven posts were damaged and needed replacement.

The wire rope fence impact test with the 750-kg car successfully met the performance requirements of the DTp, and met the exit path criteria laid down in BS6779: Part 1. In addition, the fence met the running height conditions for soft ground, and the approved fence no longer required mounting on a hardened running surface.

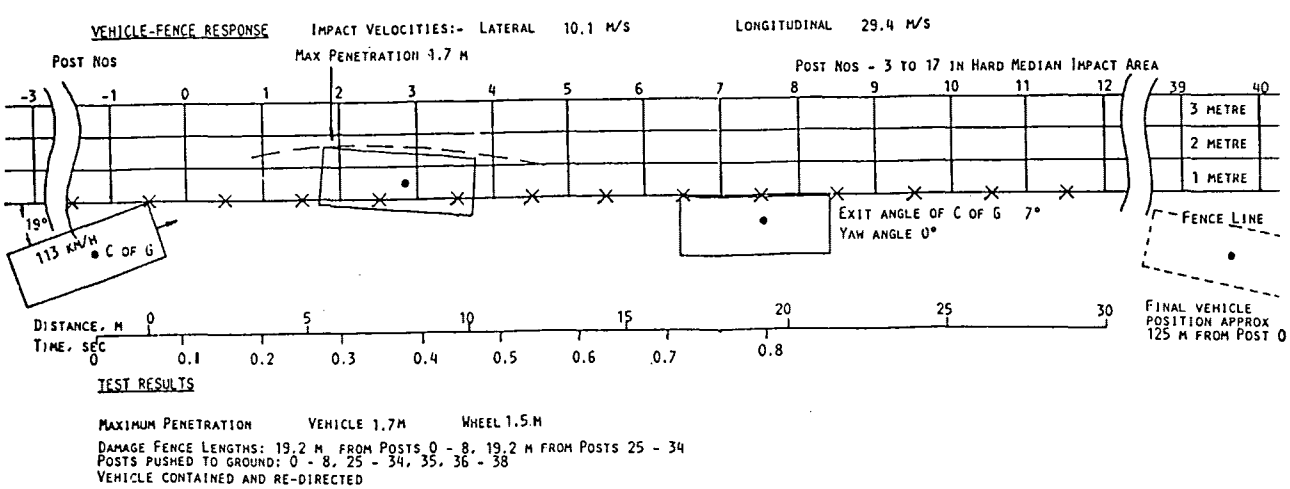


FIGURE 4 Summary of 1500-kg car test (2.4-m post spacings).

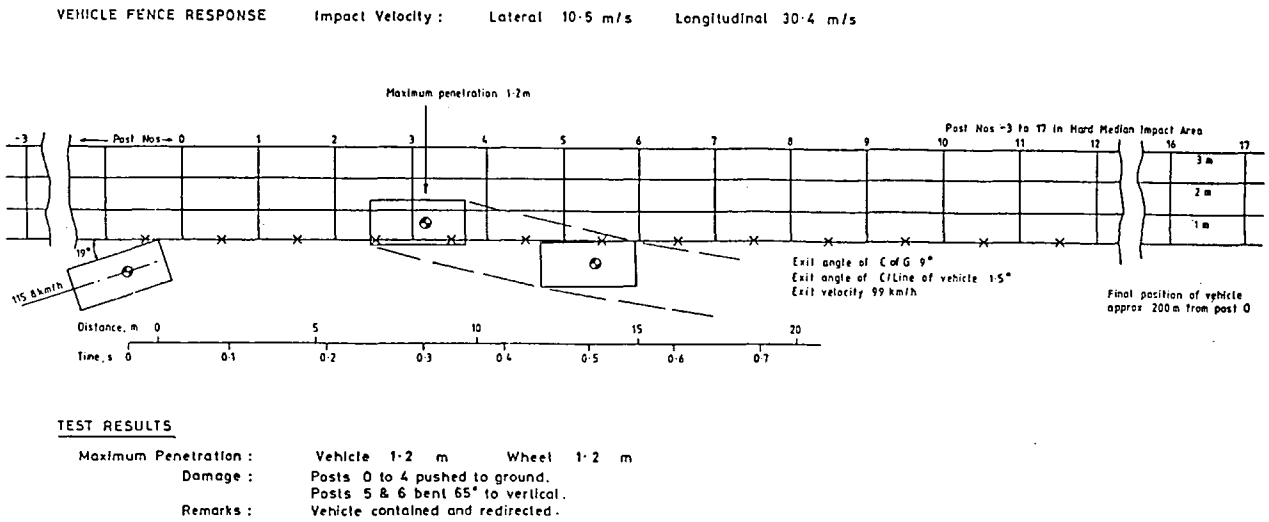


FIGURE 5 Summary of 750-kg car test (2.4-m post spacings).

Four-Rope Safety Fence with 1.0-m Post Spacings

The rope lengths, rope heights, and post cross-sectional dimensions for the fence with the 1.0-m post spacings were identical to those in the 1500-kg and 750-kg car tests, on the 2.4-m fence.

Over a 30-m length of fence, designated the impact area, z-posts were slotted into sockets 1.0 m apart. The remaining lengths of rope near the impact area were supported on posts placed 2.4 m apart. The overall length of the fence was 319 m; the rope static tension was set, by turnbuckles, to 26.5 kN.

1500-kg Car Test

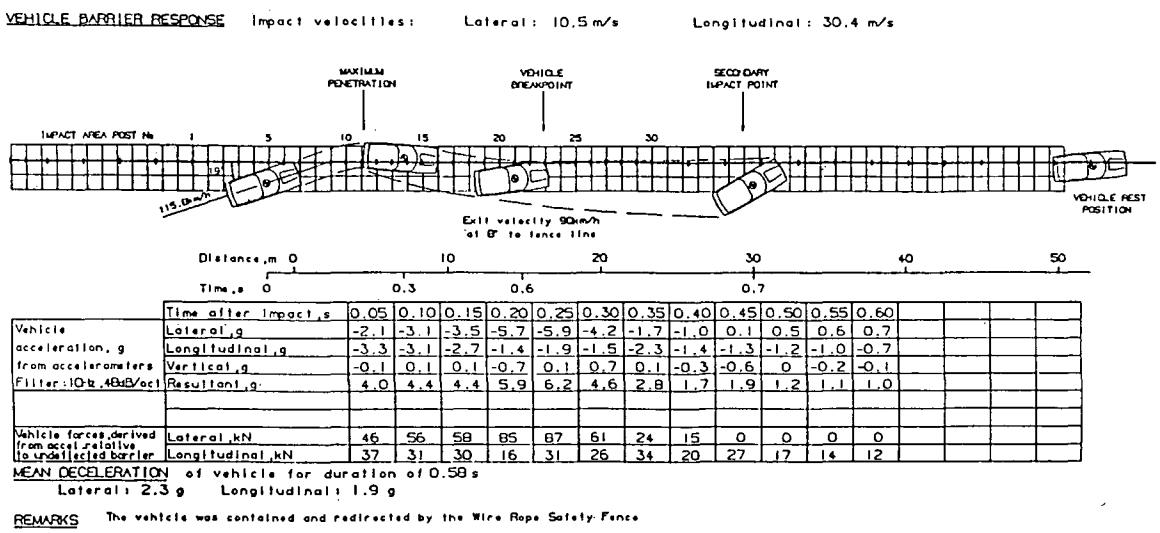
The standard-weight (1500-kg) car impacted at 115.8 km/hr at an angle of 19 degrees (Figure 6). An anthropometric dummy representing a 50th-percentile man was installed in the passenger seat of the car. The exit speed was 90 km/hr at an angle of 8 degrees (Figure 6).

On impact the four ropes were forced together and formed a groove in the vehicle bodywork at headlamp height. At 0.24 sec after impact the car was parallel with the fence and had penetrated 1.1 m; the ropes pressed into the left-hand side body panels, and at this point the maximum deflection was 1.08 m. As the vehicle continued along the fence, the maximum penetration was recorded as 1.12 m where the ropes had cut into the rear left-hand side wheel arch.

The maximum penetrations and deflections are given in the following table:

Time (sec)	*Vehicle Penetration (m)*	*Rope Deflection (m)*
0.10	0.6	0.58
0.24	1.1	1.08
0.30	1.2 (maximum)	1.12
0.30	1.3 (loose hood)	1.12

During the impact 17 posts were damaged. After leaving the fence, at a shallow angle, the car made second contact



	Time after impact, s	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60
Vehicle acceleration, g from accelerometers Filter: 10-Hz, 48dB/oct	Lateral, g	-2.1	-3.1	-3.5	-5.7	-5.9	-4.2	-1.7	-1.0	0.1	0.5	0.6	0.7
	Longitudinal, g	-3.3	-3.1	-2.7	-1.4	-1.9	-1.5	-2.3	-1.4	-1.3	-1.2	-1.0	-0.7
	Vertical, g	-0.1	0.1	0.1	-0.7	0.1	0.7	0.1	-0.3	-0.6	0	-0.2	-0.1
	Resultant, g	4.0	4.4	4.4	5.9	6.2	4.6	2.8	1.7	1.9	1.2	1.1	1.0
Vehicle forces, derived from accel. relative to undeflected barrier	Lateral, kN	46	56	58	85	87	61	24	15	0	0	0	0
	Longitudinal, kN	37	31	30	16	31	26	34	20	27	17	14	12

FIGURE 6 Summary of 1500-kg car test (1.0-m post spacings).

about 13 m from the break point. Another 10 posts (2.4-m spacings) were knocked down, and the vehicle came to rest 52 m from the initial contact point.

The front left-hand corner of the vehicle had been pushed in about 300 mm; superficial rope marks could be seen along the left-hand side of the car at headlamp height. The front left-hand side suspension was badly damaged; the wheel had been pushed forward and had twisted 90 degrees from its true position. There was no visual damage to the occupant area and all four doors could be opened after the vehicle was moved from alongside the fence. Damage is shown in Figure 7. Analysis showed that the test had complied with DTp requirements and also had met the exit path requirement stipulated in BS6779. The results of the analysis are shown in Figure 6.

The effect of reducing the post spacings from 2.4 to 1.0 m reduced the deflection of the wire rope fence from 1.7 to 1.12 m.

FIGURE 7 Final position of 1500-kg car (1.0-m post spacings).

750-kg Car Test

A 750-kg car test (Figure 8), as mentioned earlier, is not a requirement of the DTp regulations; however, there was a possibility that the closer post spacing of 1.0 m could cause wheel snagging and induce a small car to spin out. Also there was the possibility that the lower profile of the smaller car, compared with the 1500-kg car, could permit the ropes to slip over the hood. In addition, the smaller car could represent a larger vehicle running on soft ground, whose height, relative to the fence, was lowered by the wheels penetrating the running surface. In addition, the test with a light vehicle would give an indication of the severity of impact.

Impact speed was 113.4 km/hr at an angle of 19 degrees; the vehicle left the fence at a speed of 90 km/hr at an angle of 1 degree to the line of the fence, with the rear of the vehicle farthest from the fence.

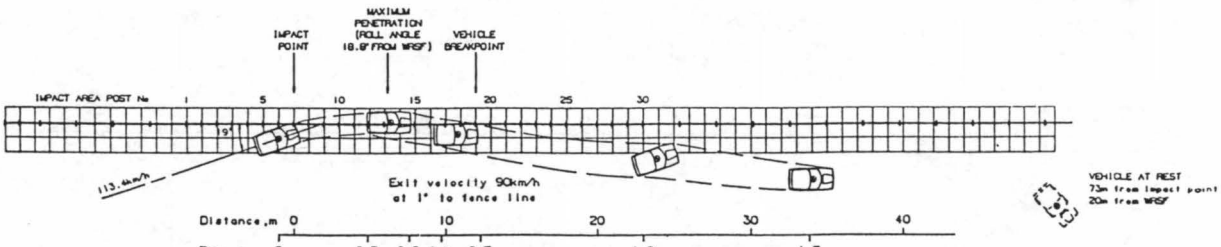
During impact, there was no indication of a rope slipping over the hood. As before, with the 1500-kg car, the four ropes formed a shallow groove in the front left side at headlamp height. At 0.08 sec after impact the windshield shattered. At 0.26 sec the maximum penetration was 0.86 m and the maximum rope deflection was 0.84 m; at this time both left-side wheels were clear of the ground (Figure 9). The car began to move out of the fence while remaining fairly parallel; there was no indication of spinout. The rear of the car yawed slightly away from the line of the fence, and it came to rest 73 m from impact point and about 20 m in front of the fence.

About seven posts were run down and another six posts were bent and needed replacement.

The front left-side quarter was crushed inward about 200 mm. The left-side front suspension unit was detached from the stub axle, and the wheel was trapped in the crushed body panels at 90 degrees to its normal running axis. The passenger door could be opened from the inside with normal manual force but could not be opened from the outside.

The test successfully passed the DTp requirements, and the exit angle complied with BS6779. Also, all ropes remained in contact with the side of the car; none slipped over the hood.

VEHICLE BARRIER RESPONSE Impact velocities: Lateral: 10.3 m/s Longitudinal: 29.8 m/s



	Time after impact, s	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
Vehicle acceleration, g from accelerometers Filter: 10Hz, 48dB/oct	Lateral, g	-3.3	-4.7	-3.8	-3.1	-2.2	-1.1	-0.4	0.4	0.8	0.7
	Longitudinal, g	-5.8	-4.4	-3.1	-2.1	-1.6	-0.6	-0.7	-0.6	-1.0	-0.7
	Vertical, g	0	-1.0	1.2	2.4	2.3	0	-0.8	-0.5	-0.7	-0.8
	Resultant, g	6.7	6.5	5.1	4.5	3.6	1.3	1.1	0.9	1.4	1.2
Vehicle forces, derived from accel. relative to undeflected barrier	Lateral, kN	36	42	32	25	17	8	3	0	0	0
	Longitudinal, kN	33	21	16	12	10	4	5	4	7	6

MEAN DECELERATION of vehicle for duration of 0.47 s
Lateral: 2.0 g Longitudinal: 2.2 g

REMARKS The vehicle was contained and redirected by the Wire Rope Safety Fence

FIGURE 8 Summary of 750-kg car test (1.0-m post spacings).

FIGURE 9 750-kg car during impact (1.0-m post spacings).

The test demonstrated that the four-rope fence met the running height conditions for soft ground.

VEHICLE IMPACT SEVERITY

The severity of impact was estimated using the value of the theoretical head impact velocity (THIV). The THIV value estimates the impact velocity with which a freely moving object, representing an occupant's head, would hit a surface in its path inside the vehicle compartment.

Figure 10 compares THIV values for standard tests with a 1500-kg car and a 750-kg car at 113 km/hr, in collision with a concrete barrier, a steel tensioned corrugated beam (TCB) fence and the wire rope fence with 2.4- and 1.0-m post spacings.

In terms of the THIV values, the 2.4-m wire rope fence is no worse than the TCB fence at about 4 m/sec for the 1500-kg car and about 5.5 m/sec for the 750-kg car. A comparable THIV value for impact into a vertical concrete barrier (VCB) is about 7 m/sec for the standard 1500-kg car.

The THIV value increased from 4 m/sec for the wire rope fence with 2.4-m post spacings to 5.6 m/sec for the 1500-kg car impact into the 1.0-m fence.

The THIV value for the 750-kg car impact into the 1.0-m fence was 6.4 m/sec compared with 8.4 m/sec for its impact into a VCB.

The CEN standard on road restraint systems is likely to recommend that THIV values should not exceed 9 m/sec for Impact Severity Level A and 12 m/sec for Level B. The impact tests demonstrated that both the 2.4- and 1.0-m wire rope fences met the proposed CEN standard for impact severity with THIV values less than the lower recommended value of Level A.

The head injury criterion (HIC) for the passenger dummy was very low at 56, in the 1500-kg car test on the fence with 1.0-m post spacings; the limiting injury value defined in FMVSS in 1,000.

CONCLUSIONS

- The Bridon Ropes wire rope safety fence, with post spacings at 2.4 and 1.0 m, met the impact performance requirements laid down by the U.K. DTp for highway safety fences. The fences also complied with the vehicle trajectory after impact given in BS6779, Part 1.
- The 1500-kg vehicle was safely contained and redirected, after a standard 113-km/hr impact into the 2.4-m four-rope fence. The vehicle departure path was 7 degrees to the line of the fence, and the maximum penetration was 1.7 m.

This test was repeated for the 1.0-m fence; the maximum vehicle penetration was 1.2 m. The reduction in penetration of 0.5 m permits the wire rope fence to be considered for use where site space is restricted. For example, reduced post spac-

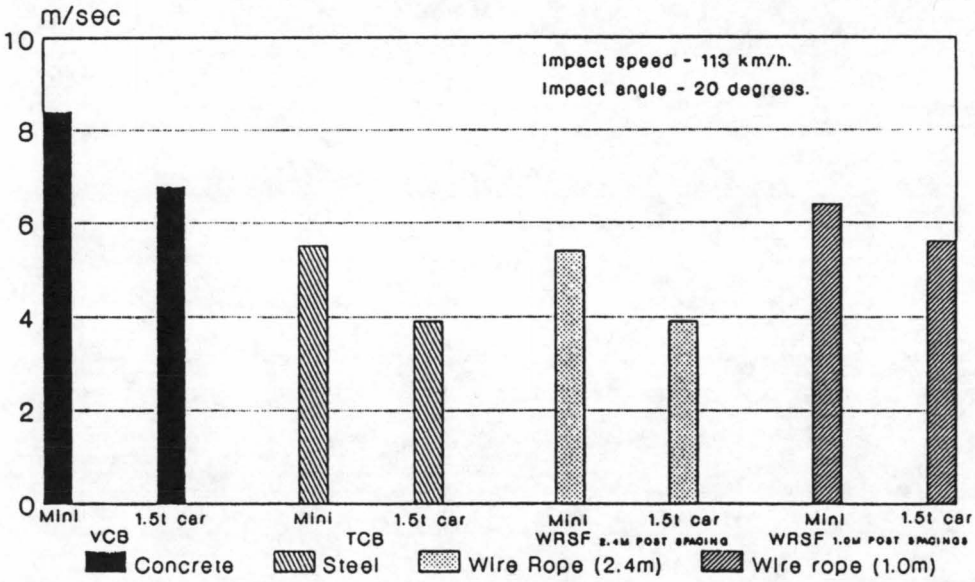


FIGURE 10 Theoretical head impact velocity comparison (mini-750-kg car).

ing would be used to restrict fence deflection where lighting columns were placed close to the safety fence.

- Additional tests with 750-kg car impacts at target speeds of 113 km/hr at 20 degrees also successfully met the U.K. DTp and BS6779, Part 1 requirements. The maximum penetrations were 1.2 m for the 2.4-m fence and 0.86 m for the 1.0-m fence. In addition, these tests demonstrated that the 750-kg car, which has a lower front profile than the 1500-kg car, did not penetrate beneath the fence ropes.
- The maximum tension measured in all vehicle impact tests was 22.6 kN; the breaking load of a single rope is 173.6 kN. The same ropes were used for all the tests; none of the ropes received damage that would require rope replacement at a roadside installation. The fence was quickly repaired after each test by manually extracting the damaged posts from the concrete sockets and replacing them with new ones. In none of the whole series of tests was a post pulled from its socket during vehicle impact. After each test the wire ropes were lifted into place without needing retensioning or mechanical power equipment.
- Vehicle impact severity for the standard 1500-kg car test, using the THIV measure, was 4 m/sec for the 2.4-m post spacing four-rope fence; this is similar to that of the TCB fence for the standard 1500-kg car test. The THIV value increased from 4 to 5.6 m/sec for the 1.0-m fence.

The respective values for the 750-kg car tests on the 2.4- and 1.0-m fences were 5.5 and 6.4 m/sec.

Both the 2.4- and the 1.0-m fences met the THIV level for impact severity given in the draft CEN standard, with THIV values considerably less than the recommended 9 m/sec.

The HIC value of 56 recorded for the standard 1500-kg car test and the 1.0-m post spacing fence was considerably lower than the injury threshold of 1,000 units quoted by FMVSS.

- The U.S. performance requirements (given in *NCHRP Report 230*) (*2*) for safety barriers are under revision. The 750-kg and 1500-kg U.K. car tests are likely to meet the Report 230 requirement for impact by an 1,800-lb car at 15 degrees and 60 mph. The U.K. 1500-kg car test at 20 degrees and 70 mph has an energy level, with reference to the component velocity normal to the fence, that is about 50 percent lower than the U.S. 4,500-lb car test at 25 degrees and 60 mph. However, it is likely that the wire rope safety fence described in this report will be tested according to *NCHRP Report 350* (*3*), the revised version of Report 230.

ACKNOWLEDGMENTS

The authors are indebted to Bridon PLC for permitting the use of the material contained in this report. Andrew Stacey, Technical Director of Bridon Ropes Ltd., was primarily responsible for the project, and his enthusiastic guidance on the design of the double-height wire rope fence is freely acknowledged.

The impact tests were carried out by MIRA under contract to Bridon PLC.

REFERENCES

1. Jehu, V. J., and I. B. Laker. *The Wire-Rope Slotted-Post Crash Barrier*. RRL Report LR 127. Road Research Laboratory, Ministry of Transport, Crowthorne, Berkshire, United Kingdom, 1967.
2. *NCHRP Report 230: Recommended Procedure for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., 1981.
3. *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features*. TRB, National Research Council, Washington, D.C., 1993.

Publication of this paper sponsored by Committee on Roadside Safety Features.

Performance of Cable Guiderail in New York

WEI-SHIH YANG, LUIS JULIAN BENDAÑA, NICHOLAS J. BRUNO, AND WAYNE D. KENYON

Cable guiderail with insufficient tension may deflect excessively on impact, allowing vehicles to contact fixed objects behind the barrier. In 1980 a two-phase study was initiated by the New York State Department of Transportation to investigate causes of tension loss in cable guiderail and formulate corrective measures. The study's first phase documented performance of new cable barriers in the field and the results of laboratory testing. Anchor movement and permanent cable stretch were identified as major causes of tension loss, sufficient to affect barrier performance adversely. Several changes have already been made based on these results: construction specifications and standard sheets were changed to ensure proper soil compaction and better initial and long-term cable tension. In the second phase, field performance of selected improved installations was documented from 1984 to 1987. In addition, prestressed cable was used on some projects in 1985 to investigate its effectiveness in reducing tension loss due to cable stretch. Laboratory stretch tests were conducted using normal and prestretched cable to determine any significant differences appearing in cable strain due to long-term loading. Results from field and laboratory tests indicated that cable guiderail installations continually lose tension and need to be retensioned periodically and that substituting prestretched for normal cable does not reduce the tension-loss problem.

Lightweight cable guiderail now in use in New York State was developed in the late 1960s (*1,2*). The cable is designed to separate from S3 × 5.7 steel posts on vehicle impact, with tension in the cable developing the force necessary to retain and redirect vehicles. The tension in the barrier before impact affects the total cable deflection that must occur to develop this force. The rail elements consist of three ¾-in. galvanized steel cables mounted on the posts by hook bolts. The cables are secured to concrete anchor blocks at the ends of each installation to develop tension. Details of the cable-guiderail system are shown in Figure 1. Spring-compensator devices are included to allow for cable length change due to temperature change. When properly adjusted, these spring compensators should maintain a working range of cable tension between 450 and 1,800 lb throughout the annual temperature cycle without any need for periodic adjustment. The standard sheets require that in cases where the cable run is 1,000 ft or more, springs are required at each end; otherwise they require springs at only one end (*3,4*).

During the 1979 New York State Department of Transportation (NYSDOT) Highway Safety Review, a problem was detected related to the safety of cable guiderails, concerning their inability to redirect traffic because of insufficient tension in the cables. During inspections almost every cable installation observed was found to have insufficient tension. Initially, there was concern that proper installation procedures had not been followed. After further investigation, it became apparent that cable guiderail could become slack even if installed and tensioned correctly.

Besides being unattractive, slack cable guiderail is a potential safety hazard because it limits the ability of the barrier to redirect vehicles within the allowable deflection range. A vehicle that impacts an installation with insufficient tension can be guided into an object while being redirected.

To address these concerns, a research study was initiated in the spring of 1980. Its objectives were fourfold:

1. Determine the extent of slack cable guiderail,
2. Identify the causes,
3. Propose corrective action, and
4. Verify that proposed solutions are effective by conducting long-term follow-up surveys.

The investigation began monitoring several cable guiderail installations. It was determined that these installations quickly lost cable tension, thus reducing their potential effectiveness. In the first phase, several possible causes were investigated, including anchor movement, post settlement, cable creep, post movement, spring compensator failure, accident impacts, inadequate maintenance, incorrect initial installation procedures, and turnbuckle backing off. Major causes were identified as anchor movement, cable creep, and nonuniform tension distribution throughout the barrier caused by frictional drag at the posts. Even after these installations were retensioned, they experienced unacceptable degrees of cable tension loss.

Based on the findings of Phase 1 (*5*), several changes were made. First, construction specifications were changed in 1982 to ensure proper soil compaction during the placement of concrete cable anchors and to reduce anchor movement to acceptable levels. Second, the standard sheet for cable guiderail was revised to ensure better initial and long-term cable tension through improved installation procedures. Third, revised specifications requiring prestressed cable were used on some projects in 1985. With such cable, it was assumed that tension loss due to stretch could be reduced.

To determine the long-term performance of these corrective measures, a second phase was initiated in 1984. The Phase 2 objectives were to monitor the effectiveness of the new specifications and corrective measures and to document the results of field and laboratory tests on prestretched cable.

Engineering Research and Development Bureau, New York State Department of Transportation, State Campus, Albany, N.Y. 12232.

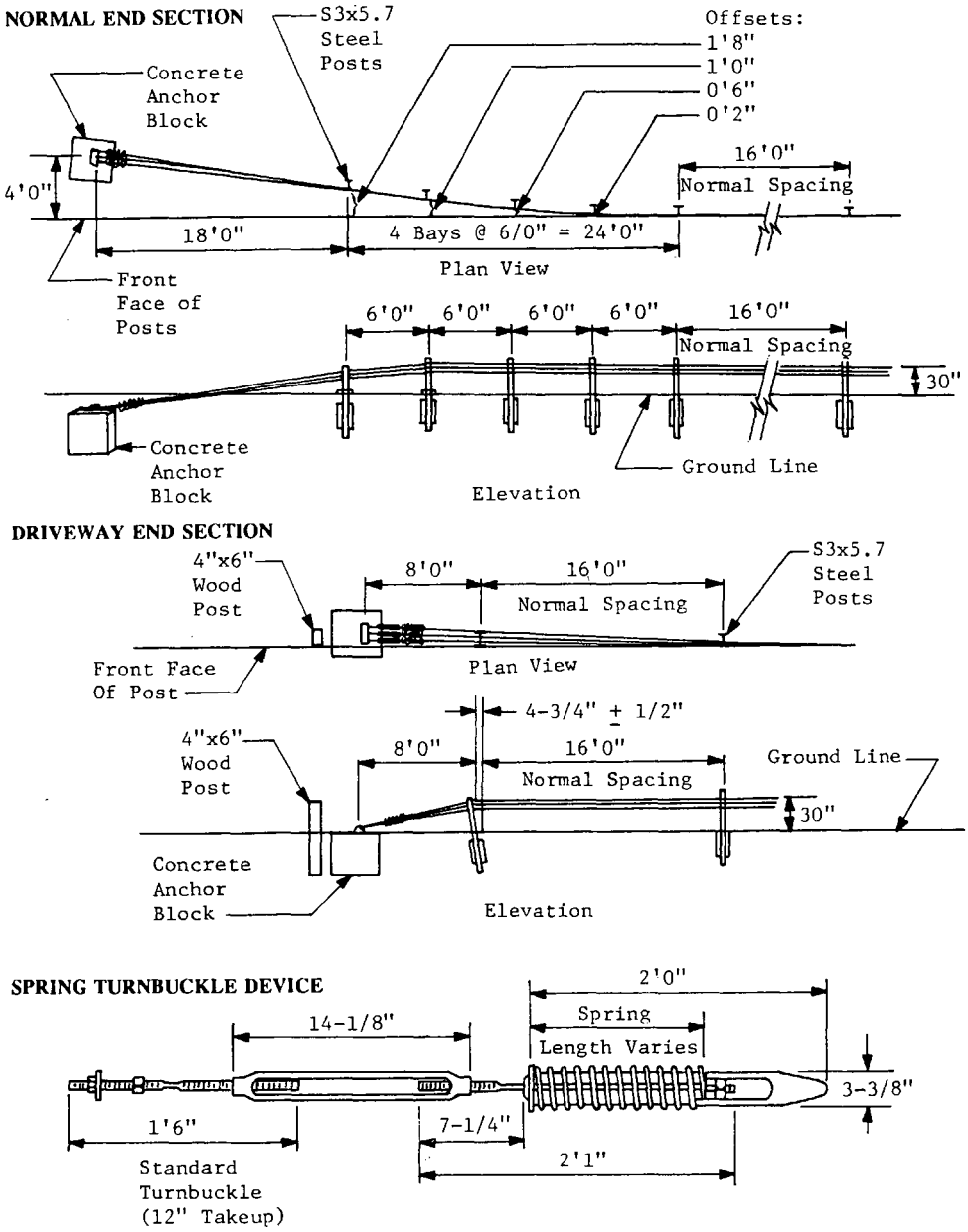


FIGURE 1 Cable-guiderail detail (1 mm = 0.04 in.).

Periodic condition surveys were carried out for 3 years, monitoring both normal and prestretched cables. Laboratory tests were conducted to determine the difference in stretch between the two cable types. The effectiveness of the corrective measures and use of prestressed cable are documented in a final report (*6*).

INVESTIGATION AND RESULTS

Phase 1

In the first phase, possible causes of tension loss were investigated and identified. A total of 53 new installations on 12 construction projects were monitored during 1980 to observe and document installation procedures, and reference systems were installed to monitor changes in the barrier. In addition, individual site parameters were recorded for each sample to determine what effect, if any, they had on tension loss. These parameters included the presence and degree of horizontal and vertical curves, length of run, contractor who installed the barrier, temperature at the time of tensioning, and whether inspections were by state or consultant personnel. Minor alterations in the tensioning procedure were also tried on 15 of these runs in an attempt to remedy the problem of tension loss. After promising corrective measures were formulated, these modifications were included on 21 new installations in 1981 to evaluate their effectiveness. Some of the cables placed

in 1980 were also retensioned in 1981 to see if setting proper tension a second time would aid in maintaining tension.

Cable tension, occurrence of accident damage, and any changes occurring with respect to the reference systems were monitored throughout the year by conducting spring, summer, and fall surveys. Snow and ice made it impossible to obtain winter measurements, but reconnaissance surveys were made throughout winter months to keep track of accident and snowplow damage. If any sample installations were readjusted or experienced damage that would affect tension, they were no longer surveyed. All data obtained from the surveys were organized in tabular form to facilitate statistical analysis. As discussed later, the tensioning procedure, anchor movement, and permanent cable stretch were identified as major causes of tension loss and were found sufficient to harm barrier performance (*5*).

Evaluation of Tensioning Procedure

Four tensioning procedures were compared to verify whether any benefits were obtained by changing normal tensioning practice: (a) a normal-tension group with 154 samples, (b) a 1980 revised-tension group with 78 samples, (c) 1981 revised-tension group with 63 samples, and (d) a retensioned group with 73 samples.

The sequence for installing cable guiderail using these four procedures was as follows:

1. Normal-tension procedure: After posts were driven and anchors placed, the cable was unrolled and cut to the approximate length. The cable was strung through the J-bolts, unloaded spring lengths marked on the compensator rod, and the anchor hardware attached to the cable at one end and secured to the anchor. With the cable now fixed at one end, it was pulled straight by applying tension at the opposite end with a hand winch or pulling with a truck to remove the slack. With this tension held by the truck, or winch, or locking pliers clamped on the cable against posts, the cable was cut to the final length. The anchor hardware was then installed at this end and secured to the anchor, and the slack between the anchor and the point at which the tension was being held was taken up with the turnbuckle. The temporary clamps were then released, leaving the cable secured to both anchors with some initial tension present.
2. 1980 revised-tension procedure: This procedure involved placing 1,600 lb of initial tension on each cable. This value equates to 3.5 in. of spring compression, slightly below the upper limit of 4 in. After 2 to 3 weeks, tension was set to the standard-sheet value if the cables had not already relaxed to that level.
3. 1981 revised-tension procedure: This procedure included two additional modifications of the 1980 revised-tension procedure. First, to overcome the problem of frictional drag, the springs were compressed by applying tension at the opposite end of the barrier. By pulling the cable the entire length of the barrier, the amount of tension at any point had to be at least the value indicated by the springs at the far end. Second, these runs were tensioned according to 10°F temperature intervals corresponding to ¼-in. spring compression increments as presented in Table 1.

TABLE 1 Spring Compression Settings for Cable Tensioning

Previous Settings (20 deg F, ½-in. increments)		Revised Settings (10 deg F, ¼-in. increments)	
Temperature Range, F	Spring Compression, in.	Temperature Range, F	Spring Compression, in.
-20 to -1	4.00	-20 to -11	4.25
0 to 19	3.50	-10 to -1	4.00
20 to 39	3.00	0 to 9	3.75
40 to 59	2.50	10 to 19	3.50
60 to 70	2.00	20 to 29	3.25
80 to 99	1.50	30 to 39	3.00
100 to 120	1.00	40 to 49	2.75
		50 to 59	2.50
		60 to 69	2.25
		70 to 79	2.00
		80 to 89	1.75
		90 to 99	1.50
		100 to 109	1.25
		110 to 119	1.00

Note: $t_C = (t_F - 32)/1.8$
1 N = 0.225 lbf

4. Retensioned procedure: Samples installed with the normal tensioning procedure were retensioned according to the updated temperature-spring compression settings to see whether setting proper tension a second time would help maintain it better.

The cables in the normal-tension group experienced an average 26 percent loss (298 lb) between the time they were tensioned during late summer and fall 1980 and the fall 1980 survey. This loss is based on the difference between actual measured values and theoretical tension values at the measurement temperature. After the first winter, an average 46 percent loss (465 lb) had occurred, and by spring 1983 average loss was 602 lb, or 56 percent. Seventy-seven percent of the total loss measured in spring 1983—after three winters in service—had occurred by spring 1981, and thereafter the gap between theoretical and measured tension widened at a much slower rate. Also, it was found that tension was not distributed uniformly through the cable. Figure 2 shows measured tension values throughout a 1,946-ft barrier just after tensioning by the normal procedure. Tension in the middle portion of the run was about 50 percent less than that indicated by spring compression at the ends. Frictional drag at the cable-post connection caused nonuniform tension throughout the barrier.

The cables in the 1980 revised-tension group experienced an average loss of 19 percent, or 315 lb, during the 2- to 3-week period when tension was left at the high initial level. By spring 1981, after one winter in service, average tension loss was 196 lb, or 19 percent of the theoretical value, referenced to the tension value at the end of the pretensioning period. By spring 1983, actual losses averaged 32 percent of the theoretical value, or 325 lb. Losses occurring by spring 1981 averaged 78 percent of the 1983 loss. Forty-nine percent of the spring 1983 total loss—that was occurring during the 2- to 3-week pretensioning period, plus the loss taking place thereafter—occurred before final adjustments were made at the end of the pretensioning period. The high loss during the high initial tension period was not critical since it occurred after final adjustments were made. After one winter, tension loss in the normal-tension group was 137 percent greater than

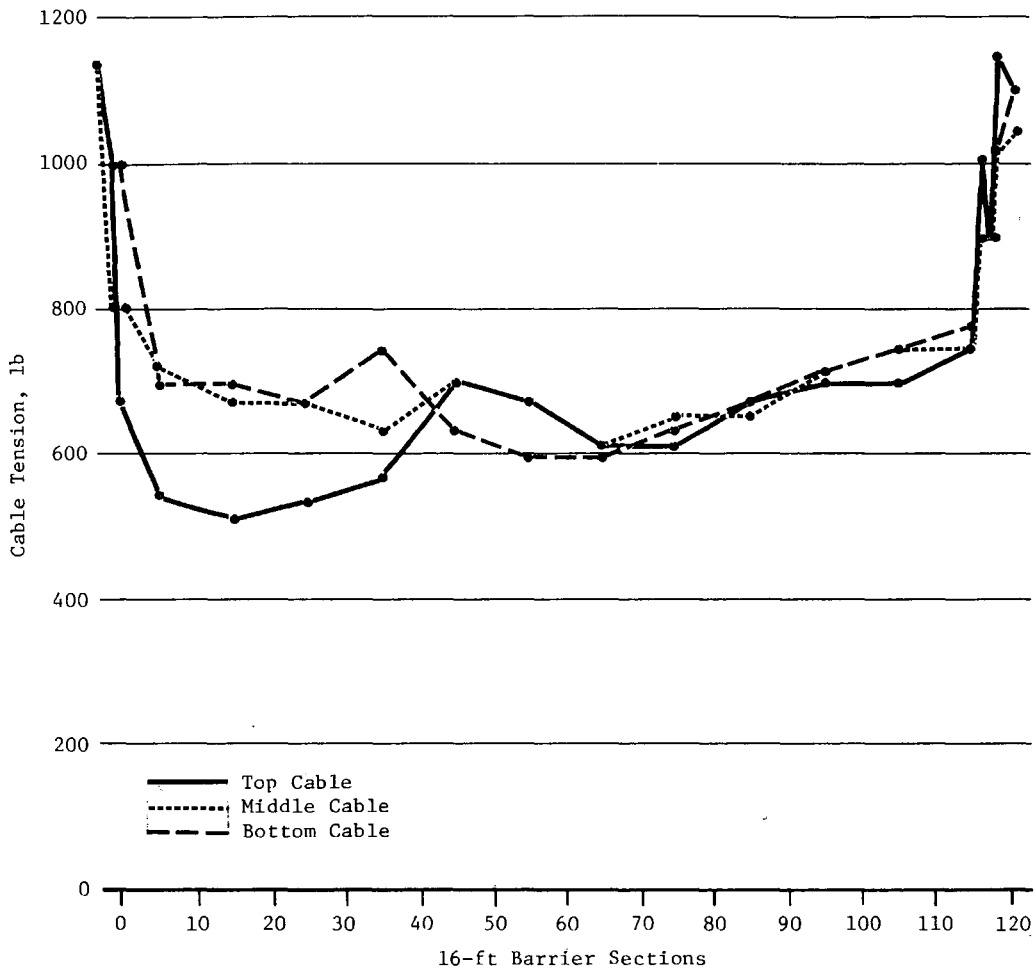


FIGURE 2 Distribution of tension through barrier after tensioning by former normal procedure (1 N = 0.225 lbf, 1 m = 3.28 ft).

in the 1980 revised-tension group, but by spring 1983 tension loss in the normal-tension group was 85 percent greater.

Figure 3 shows that by spring 1983, all samples from both the normal-tension group and the 1980 revised-tension group experienced tension losses greater than 200 lb, considered the maximum acceptable loss. However, only 10 percent of the 1980 revised-tension group had experienced losses greater than 400 lb by 1983, compared with 95 percent of the normal-tension group. The 200-lb maximum tension loss was established so that the minimum tension in the cable would not drop below 450 lb. The rationale for this criterion was developed as follows (*5*): First, initial tensions and field temperatures were recorded for the maximum length of the cable used, 2000 ft. Second, these tensions were adjusted to a maximum design temperature of 95°F. Third, maximum tension loss was calculated so that barriers may not drop below the minimum desired tension of 450 lb at 95°F.

The 1981 revised-tension group, installed in fall 1981, experienced a 483-lb (28 percent) loss during the high-initial-tension period. After final adjustments, additional losses of 430 and 527 lb (37 and 46 percent) occurred by spring 1982 and spring 1983, respectively. By spring 1983, 94 percent of the sample experienced tension loss greater than 400 lb, ranging up to 681 lb. This group lost considerably more tension than the 1980 revised-tension group. The 1981 revised-tension samples were set at higher average tension on final adjustment than the 1980 revised-tension sample for two reasons. First, the 1981 procedure pulled out all slack from the total length of barrier at one end, so average tension throughout those runs was higher than in the 1980 group. Second, the 1981 group was set to 10°F intervals rather than the 20°F intervals used in 1980. Because the 1981 runs were tensioned to a higher level, the greater loss was less critical. Average measured tension values are in the same range as the 1980 revised-tension group, even though the theoretical loss for the 1981 group was greater.

The retensioned group experienced significant tension loss after being readjusted in the fall of 1981. Relative to the final adjustment, this group of 15 runs experienced average tension losses of 253 lb (26 percent) and 308 lb (28 percent) by spring 1982 and spring 1983, respectively. This group contained six runs—two 1980 normal-tension and four 1980 revised-tension—that were retensioned by the contractor in late spring 1981. These six runs thus were retensioned twice. Considering

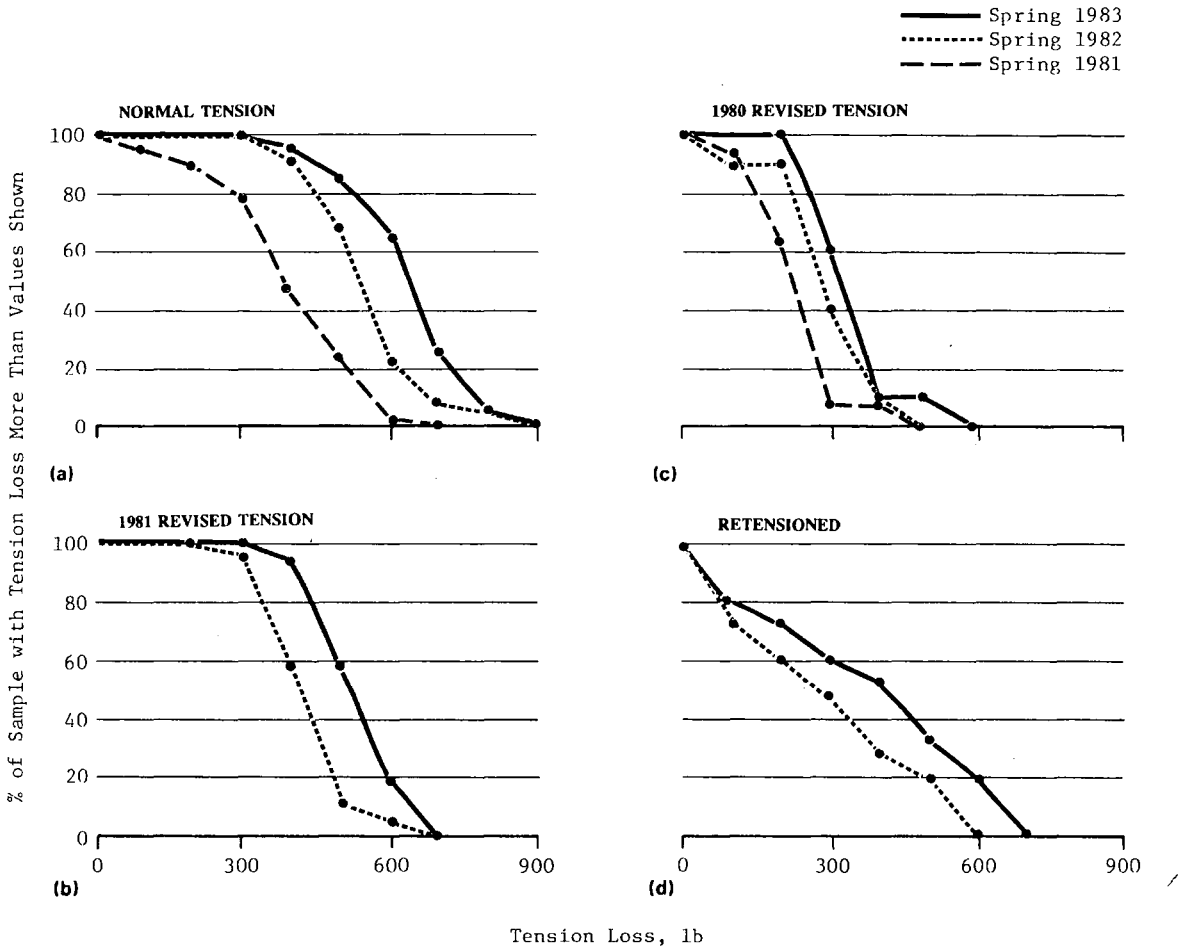


FIGURE 3 Distribution of tension loss after final adjustments 1980 revised tension (1 N = 0.225 lbf).

the six separately reveals a relatively small tension loss by spring 1983 of only 119 lb, or 11 percent.

Tension throughout two barriers tensioned by the 1981-revised procedure is shown in Figure 4. The expected tension distribution did not always occur, and Barrier A shows a slightly lower tension at mid-run than at the ends. However, variation throughout this run is only about 10 percent, compared with as much as 50 percent for barriers tensioned by the normal procedure. Tension in Barrier B is more uniform, with the top and middle cables having tensions higher than the spring-indicated value, and only the bottom cable has some measured tension values slightly lower than the springs indicate. Even though this procedure did not precisely duplicate the expected results, it produced a more uniform distribution of tension throughout the barrier than the normal procedure.

Anchor Movement

The four groups just described were also monitored for anchor movement. Some movement occurred during tensioning of most of the sample runs, and resulted in a visible gap between the anchor and the earth behind it as springs were compressed. By the time the last cable was tensioned, this movement was sometimes large enough to result in decreased spring compression for the other two cables. Research personnel told the contractor about this movement so that corrections could be made. Anchor-movement measurements presented here are the combined movements of both anchors relative to their position immediately after the barrier was tensioned.

Anchor movement for each sample group is summarized in Figure 5. The curves represent average measured anchor movements for the samples in each survey. Most movement occurred immediately after tensioning and between the first fall and spring surveys. After the first spring, a slight decrease in average anchor movement was often noted, probably caused as the anchors settled back. During and after tensioning, the anchors tipped forward, pressing against the fresh fill and compacting the soil beneath the front portions of the anchors. Most guiderail is installed during late summer and fall, and falling temperatures maintain a load on the anchors. Movement probably ceases during winter because soil around the anchor freezes, but high moisture levels and thawing in spring result in low soil support. This poor support is normally coupled with relatively high cable tension from the cool temperatures, compared with installation, and this situation probably produces the significant movement measured in the first spring

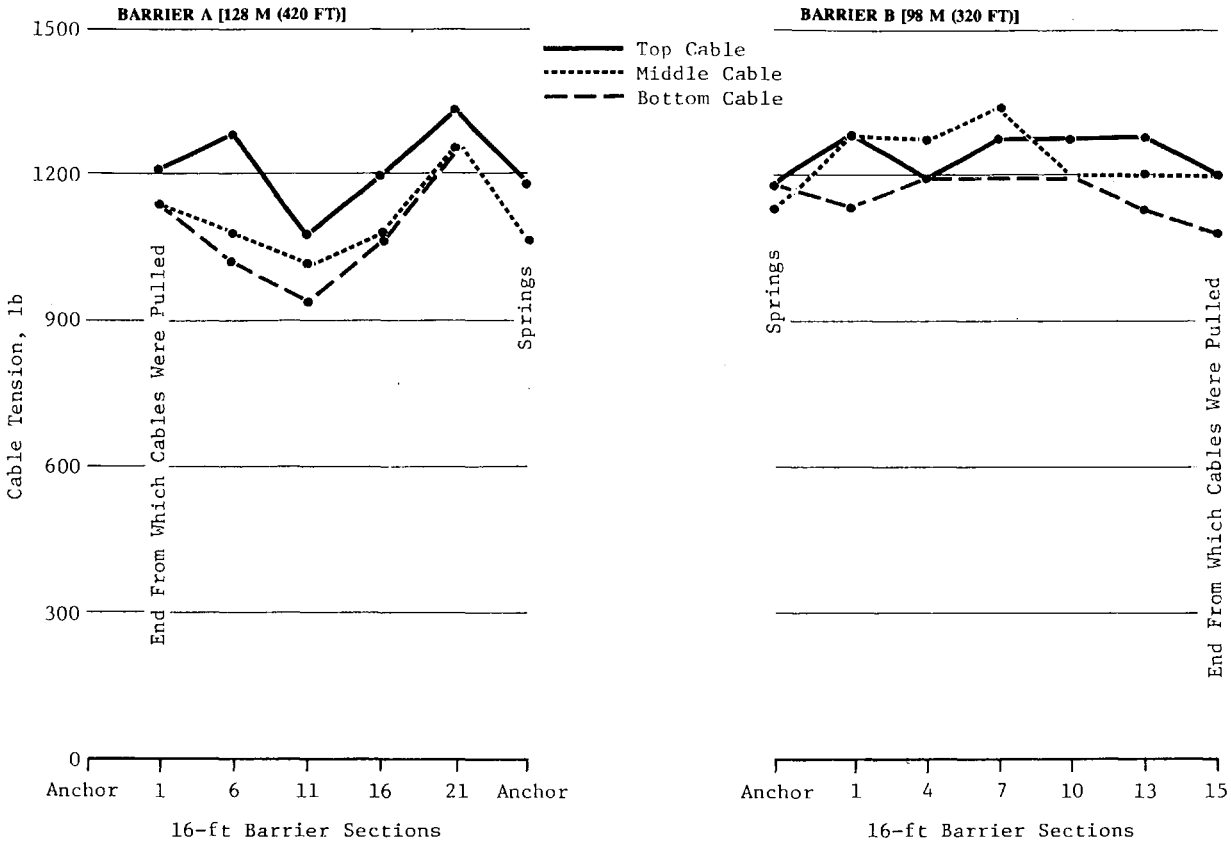


FIGURE 4 Distribution of tension in two barriers after final adjustments for four sample groups (1 N = 0.225 lbf).

survey after installation. As temperatures moderate, cable loads decrease, allowing the anchors to press against the soil behind and beneath them. As this soil compacts, the anchors settle back, thus negating some of the forward movement. After the first spring, anchor movement generally fluctuated slightly, but significant movement rarely occurred unless the barrier was retensioned, at which point movement showed another major increase.

For the 1980 revised-tension group, 56 percent of the total spring 1981 anchor movement occurred during the high-initial-tension period, and for the 1981 revised-tension group, 54 percent of the total spring 1982 movement occurred during this period. Total anchor movement for the revised-tension groups—which is the sum of the values above and below the datum line—is greater than the total amount experienced by the normal-tension group, but movement affecting working tension in the barrier is less. Anchor movement for the retensioned samples is also shown in Figure 5. This group of 15 samples contains 6 installations that were retensioned twice—once by the contractor, and then once by researchers. Before retensioning, the anchor movement trend for the whole group was similar to the other sample groups, with substantial movement totaling 2.82 in. by the first spring. Movement then leveled off until the samples were retensioned by researchers in fall 1981, at which point significant movement again occurred, totaling an additional 1.09 in. by spring 1983. The movement that occurred before retensioning in fall 1981 appears below the datum line in Figure 5.

Thirty-seven percent of the normal-tension sample experienced anchor movement exceeding 2 in., ranging up to 6¼ in. By comparison, none of the 1980 revised-tension group, the 1981 revised-tension group, or the retensioned-group experienced anchor movement greater than 2 in. by the first spring after final adjustment. The revised procedures thus resulted in very significant reduction in critical anchor movement compared with the normal-tension group.

Although the revised tensioning procedures reduced the effect of anchor movement on tension loss, additional measurements were desirable to stabilize the anchors. Anchor placement was observed on 12 projects to determine typical procedures used. Generally, the hole was dug with a backhoe, the anchor placed, and soil then backfilled and compacted around the anchor; however, time and effort spent placing and compacting the fill varied. Some crews placed the backfill in five or six lifts, tamping each, while others dumped the backfill around the anchor and simply dropped the backhoe bucket around the top of the fill to compact it. Rocks and large chunks of asphalt pavement were sometimes included in the backfill, making compaction difficult. Careful backfill procedures generally resulted in less movement, but did not guarantee stable anchors. Placing the anchor in the ground a few weeks before tensioning also seemed to have the favorable

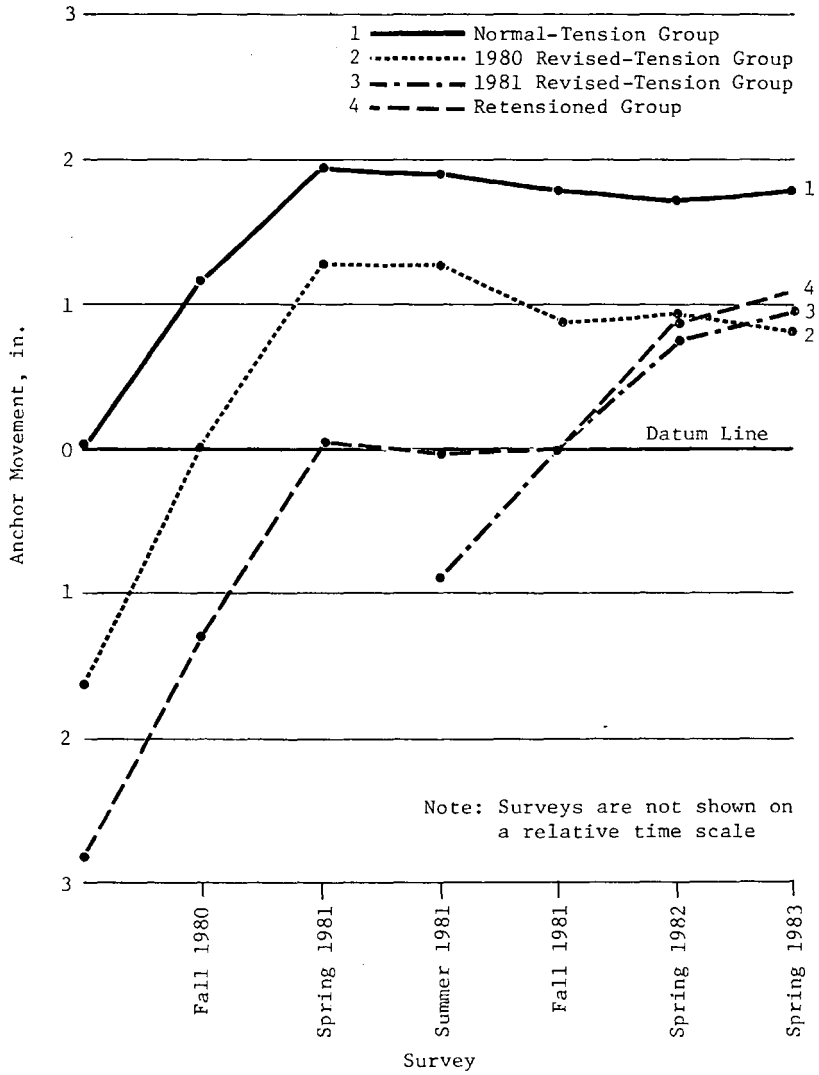


FIGURE 5 Summary of anchor movement for four sample groups (1 mm = 0.04 in.).

effect of reducing movement. Construction specifications permitted considerable latitude in placement procedure, and firm guidelines for backfilling and compaction were not included. This problem was brought to the attention of the NYSDOT Soil Mechanics Bureau, and standard sheets were revised in the hope of ensuring that adequate compaction is achieved around all anchors. Specifically, limits of excavation have been provided, and the revised specification requires suitable fill material to be paced in 150-mm (6-in.) lifts and compacted to 95 percent of standard Proctor maximum density.

Permanent Cable Stretch

Constructional stretch is an inherent property of all wire rope products. This deformation is permanent and remains after load is released. Constructional stretch can be removed by prestretching, which involves subjecting the cable to repetitive loadings of up to 50 to 60 percent of its ultimate strength (7). Manufacturers of a wire rope contacted in this study claimed that guiderail cable may experience a permanent stretch of 0.25 to 0.50 percent of its unloaded length (personal correspondence, S. E. Chehi, Bethlehem Steel Corporation, March 1981). This much stretch would produce large tension loss in cable guiderail. According to industry representatives, guiderail working loads of up to 2,000 lb would never remove all the potential stretch, which would continue as long as the cable was loaded (personal correspondence, S. E. Chehi, Bethlehem Steel Corporation, March 1981). They further indicated that even setting initial tension in the upper range of the working load will not remove all potential stretch, but probably would help reduce tension loss from subsequent cable stretch. Prestretching the cable at loads of 1,200 to 15,000 lb would be the only way to remove all the stretch, but this was not considered feasible by one major manufacturer because of lack of facilities to perform the work. The manufacturer did prestretch one reel containing 1,078 ft of cable. This material was then made available for field instal-

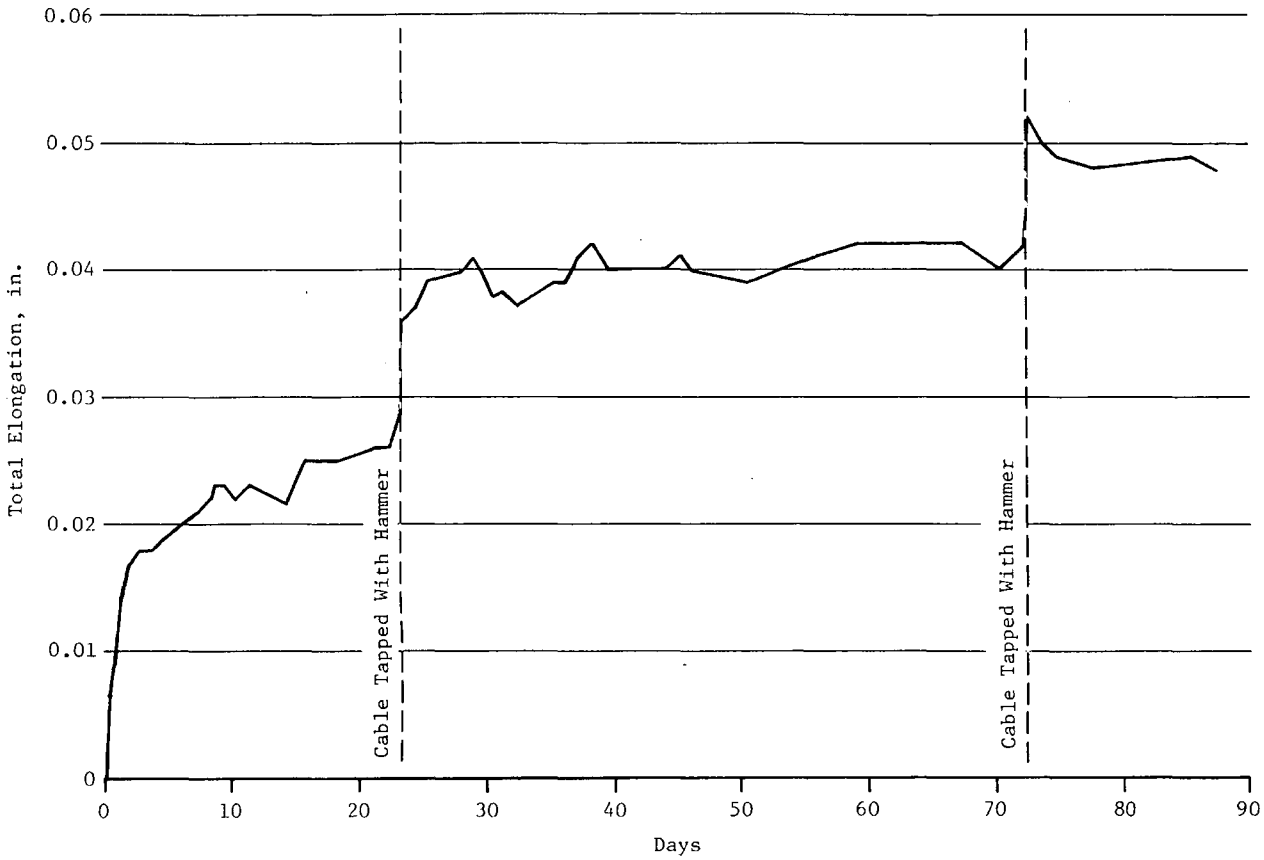


FIGURE 6 Permanent cable stretch over 12-week period (1 mm = 0.04 in.).

lation to monitor its performance. However, no data were recorded during prestretching to determine cable length changes, so testing to determine the behavior of guiderail cable under working loads was initiated as part of this study.

Figure 6 shows the stretch that occurred over a 12-week period under a load of about 1,900 lb. Much of the total stretch occurred during the first 2 days that the cable was placed under load. Additional significant stretch occurred when the cable was tapped with a hammer, creating vibrations that apparently helped seat the wires and lays. By the beginning of the twelfth week, a total of 0.052 in. of permanent stretch had occurred over the instrumented length of 8 ft 10 in., equating to strain of 0.0005 in./in.

This cable stretch testing was initiated late in the project schedule, after learning that the manufacturer did not document the prestretch testing. Thus, time allowed for only one test, which is insufficient for firm conclusions. These preliminary test results, based on only one sample of cable, plus information supplied by manufacturer of wire rope indicate that permanent stretch is a major contributing factor in tension loss. This problem is not easily remedied in the field because normal working loads are not large enough to remove all potential constructional stretch (the stretch during installation). More testing is necessary to determine the total range of constructional stretch to be expected for a large sample of cable.

The reel of prestretched cable donated for this project was installed in the fall of 1982. This 250-ft run was tensioned according to the 1982 revised procedure, except for excluding the period of high initial tension, because all pending constructional stretch was supposed to have been removed. The barrier thus was tensioned according to the revised tensioning table and left at this value. The primary interest in this test was the reaction of other barrier components if the cable did not undergo constructional stretch. By spring 1983, an average tension loss of 681 lb had occurred for the three cables, caused mostly by anchor movement. The combined anchor movement of 2½ in. is equivalent to a theoretical loss of 723 lb for this run. These results support the hypothesis that if cable stretch does not occur, the anchors or some other portion of the barrier yield instead to relieve the load.

In 1984 selected projects were constructed with normal cable guiderail using improved installation procedures. Conditions were surveyed periodically for 3 years from 1984 to 1987 to determine their effectiveness. Methods similar to those used previously and described for Phase 1 again documented installation and long-term performance.

Phase 2

To determine long-term performance of prestretched cable, the NYSDOT Materials Bureau responded to a request by research personnel by issuing a special specification for pre-

TABLE 2 Field Survey Results

Group	Location Tested	Avg Tension Loss, lb (acceptable loss = 200 lb) Fall '87	Spring '87	Fall '86	Spring '86	Fall '85	Spring '85	Fall '84
INSTALLED 1984								
1	Sag	295	521	348	428	220	284	186
	Spring	232	275	185	226	122	164	47
	Avg	264	398	267	327	171	224	117
2	Sag	337	607	453	463	236	240	0
	Spring	399	341	302	313	278	232	94
	Avg	368	474	378	388	257	236	47
3	Sag	378	589	468	458	258	364	95
	Spring	338	327	259	279	230	240	141
	Avg	358	458	364	369	244	302	118
4	Sag	-	661	531	608	180	610	379
	Spring	-	154	158	62	182	151	127
	Avg	-	408	345	335	181	381	253
5	Sag	544	681	582	622	367	560	440
	Spring	512	462	229	207	271	199	217
	Avg	528	572	406	415	319	380	329
7	Sag	313	512	449	338	56	358	0
	Spring	291	279	238	250	231	217	0
	Avg	302	396	344	294	144	288	0
8	Sag	487	739	608	663	600	576	391
	Spring	514	279	246	289	378	255	435
	Avg	501	509	427	476	489	416	408
				Average = 333				
INSTALLED 1985								
10 (Normal)								
	Sag	367	670	451	469			
	Spring	349	488	340	308			
	Avg	358	579	396	389			
				Average = 431				
11 (Prestretched)								
	Sag	397	641	392	408			
	Spring	371	466	290	325			
	Avg	384	554	341	367			
				Average = 412				

Note: 1 N = 0.225 lbf

stretched cable guiderail in 1985. Such cable should experience smaller degrees of permanent stretch, placing greater loads on guiderail components over longer periods. This may or may not have a long-term effect on guiderail components, and ultimately on tension. Using this special specification, additional test sections were installed in 1985, and were also surveyed twice a year in 1986 and 1987 using the same procedures.

During each field condition survey, cable tensions were measured at two positions for sag at a low point on each cable's run between posts and at the spring-compensator. The weight of the cable and cable deflection were used to compute cable tension. Tensions of all three cables (top, middle, and bottom) at these two positions were measured and averaged to represent tension at that particular position. The difference between design tension and measured tension is the tension "loss" given in Table 2. Average tension loss in Table 2 is the average of tension loss measured at sag and at the spring-compensator. Groups 1 through 7 were installed in 1984 and monitored for 3 years. Groups 8 and 9 were installed in 1985 using the revised specification to compare the difference between normal and prestretched cable. These new installations were also monitored twice a year for 2 years. From the field results, overall average tension loss for the first set of installations (Groups 1 through 7) was about 330 lb. For the other installations (Groups 8 and 9) it was greater than 400 lb. Both exceed the acceptable level of 200 lb.

Laboratory stretch tests were conducted using normal and prestretched cable to find any significant differences in cable strain due to long-term loading. Bethlehem Steel Corp. pro-

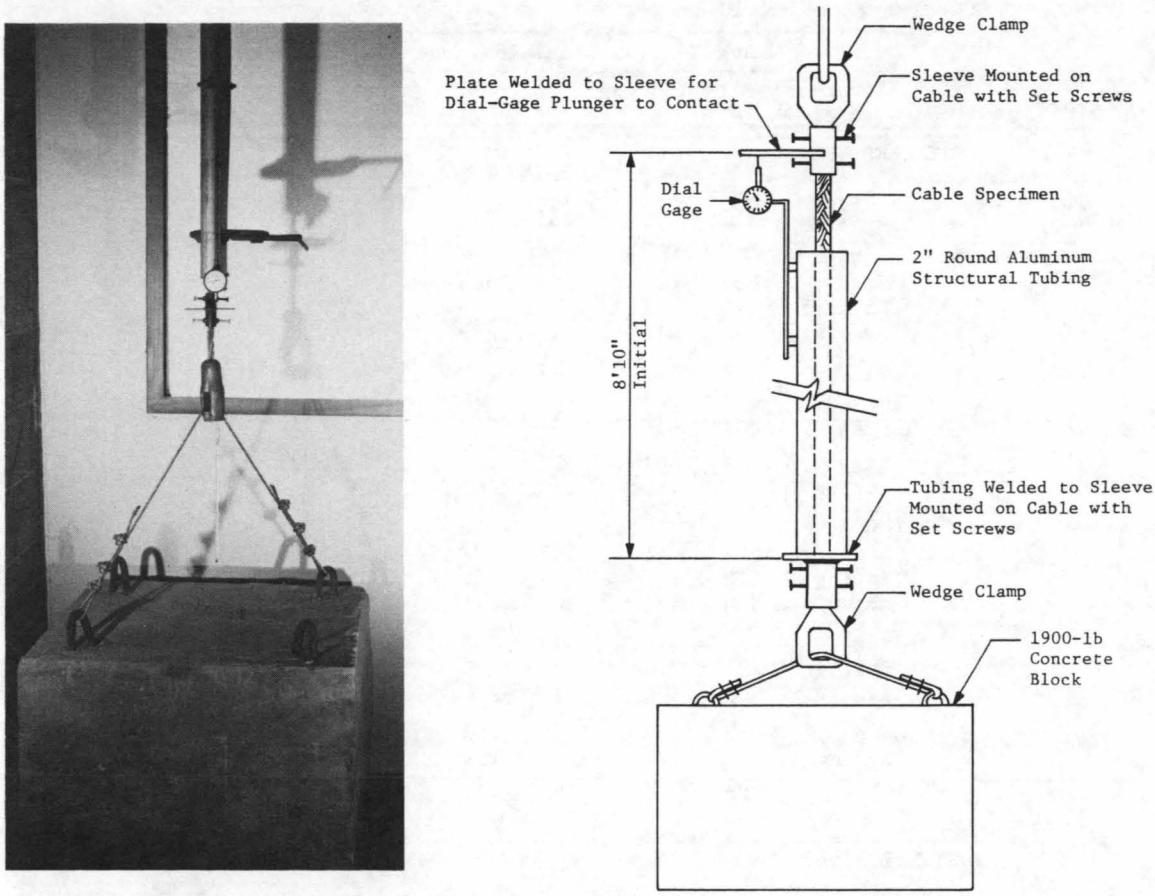


FIGURE 7 Cable stretch tests: as cable specimen elongates under load, structural tubing fixed to cable bottom moves down with it, and gauge mounted on top of tubing registers this movement with respect to plate fixed above cable (1 m = 3.28 ft, 1 mm = 0.04 in., 1 kg = 2.21 lb).

vided a reel of prestretched cable for this purpose. A series of tests was conducted in the laboratory to document the amount and nature of permanent stretch occurring during normal working loads to which guiderail is subjected. This involved suspending a length of new cable and loading it with a concrete block weighing about 1,900 lb—near the upper extreme during the annual temperature cycle, if the spring constant were at the upper acceptable limit of 500 lb/in. The spring limit is specified as 450 ± 50 lb/in. Thus, extreme cable loads would vary from 400 to 2,000 lb with a spring compression range of 1 to 4 in. over the anticipated temperature range. Amount of stretch was measured with a dial gauge reading to thousandths of an inch. Figure 7 shows the apparatus and experimental setup.

To ensure that the tests simulated field conditions, the following observations were made. During field tensioning, spring compression is set at the completion of the tensioning procedure, so that stretch occurring during tensioning does not affect barrier performance. Only stretch occurring after tensioning is of concern in terms of effect on barrier performance. In the laboratory, dial-gauge readings thus began immediately after the 1,900-lb load was applied.

Results of laboratory cable stretch tests are given in Table 3, which gives results of tension loss if the same cable stretch experienced in the laboratory occurred in the field. The data show that cable samples tested under a constant 1,900-lb loading elongated to an unacceptable length. If the same constant tension were placed on guiderail in the field, an unacceptable level of cable stretch would occur. The degree of elongation experienced by these laboratory samples would translate into unacceptable tension loss, if cable used in guiderail installations elongated the same amount, as shown in Table 4. Since the cable used in such installations does not experience constant tension, these laboratory results cannot be used to predict how much tension will be lost; however, they can predict that the cable is capable of stretching to unacceptable lengths if the tension is maintained. This experiment may show that another less plastic material might be used in place of steel cable, although this imaginary product would still have to be plastic enough to allow the barrier to deflect if impacted. Results of the laboratory stretch tests show that prestretched cable elongates less than normal cable, but that elongation is enough that guiderail installations would lose all their tension in a relatively short time.

TABLE 3 Laboratory Cable Stretch Tests

	Strain in Normal Cable, in. (corrected for temperature)								
Days	Test 1	Test 9	Test 10	Test 11	Test 12	Test 13	Test 14	Test 16	Average
25	0.0343	0.005	0.006	0.011	0.0096	0.008	0.017	0.014	0.013*
27	0.0365	0.006	0.006	0.012	0.007	0.009	0.017	0.014	0.013
29	0.0380	0.007	0.006	0.010	0.009	0.009	0.017	0.014	0.014
30	0.0368	0.006	0.006	0.009	0.009	0.009	0.017	0.015	0.014
31	0.0368	--	0.006	0.009	0.009	0.008	0.017	0.015	0.014
36	0.0366	--	--	0.014	0.02	0.010	0.017	0.015	0.019
40	0.0360	--	--	--	--	0.011	0.017	0.016	0.020
48	0.0368	--	--	--	--	0.011	0.017	0.016	0.021
49	0.0388	--	--	--	--	0.011	0.017	0.016	0.022
50	0.0386	--	--	--	--	0.011	--	0.017	0.022
88	0.0468	--	--	--	--	0.008	--	0.020	0.025
141	--	--	--	--	--	0.015	--	0.021	0.018
214	--	--	--	--	--	--	--	0.023	0.023

	Strain in Prestretched Cable, in. (corrected for temperature)							
Days	Test 2	Test 3	Test 4	Test 6	Test 7	Test 8	Test 15	Average
25	0.0165	0.011	0.0063	0.0102	0.0073	0.0053	0.012	0.009
27	0.0165	0.011	0.0068	0.0093	0.0073	--	0.012	0.010
29	0.0155	0.012	0.0068	0.0106	0 0073	--	0.012	0.011
30	0.0158	0.0118	--	--	0.0075	--	0.013	0.012
31	0.0161	0.0103	--	--	0.0075	--	0.013	0.012
36	0.0161	0.0105	--	--	0.0079	--	0.013	0.012
40	0.0164	0.012	--	--	0.0077	--	0.014	0.013
48	0.0168	0.0110	--	--	--	--	0.015	0.014
49	0.0162	0.0127	--	--	--	--	0.016	0.015
50	--	--	--	--	--	--	0.016	0.016
88	--	--	--	--	--	--	0.015	0.015
141	--	--	--	--	--	--	0.019	0.019
214	--	--	--	--	--	--	0.018	0.018

*Value used in Table 4.

Note: 1 mm = 0.04 in.

TABLE 4 Guiderail Tension If Cable Stretch in Laboratory Occurred in Field

		Normal Cable Barrier Length, ft					Prestretched Cable Barrier Length, ft				
Days	Dimension	500	1000	1500	2000	2500	500	1000	1500	2000	2500
25	δ, in.	0.79*	1.57	2.37	3.16	3.95	0.59	1.18	1.77	2.36	2.95
	P, lb	318*	636	954	1272	1590	240	479	719	958	1198
50	δ, in.	1.33	2.66	3.99	5.32	6.65	0.97	1.94	2.91	3.88	4.85
	P, lb	538	1076	1614	2152	2690	391	782	1174	1565	1956
88	δ, in.	1.52	3.04	4.56	6.08	7.60	0.91	1.82	2.73	3.64	4.55
	P, lb	611	1222	1833	2444	3055	367	734	1101	1468	1835
141	δ, in.	1.09	2.18	3.27	4.36	5.45	1.15	2.3	3.45	4.6	5.75
	P, lb	440	880	1320	1760	2200	464	929	1393	1858	2322
214	δ, in.	1.39	2.79	4.18	5.58	6.97	1.09	2.18	3.27	4.36	5.45
	P, lb	561	1121	1682	2243	2803	440	879	1319	1759	2198

*Sample Calculation:

$\Delta = 0.013$ (from Table 3).

$A = 0.22 \text{ in.}^2$

$E = 11 \times 10^6 \text{ lb/in.}^2$

$\varepsilon = \Delta \text{ in./99 in.}$

$\delta = PL/AE$

$\delta = L \times 12 \text{ in./ft} \times \varepsilon \text{ in./in.} = 500 \text{ ft} \times 12 \text{ in./ft} \times 0.013/99 = 0.79 \text{ in.}$

$P = \delta AE/L = (0.79 \text{ in.} \times 0.22 \text{ in.}^2 \times 11 \times 10^6 \text{ lb/in.}^2)/500 \text{ ft} \times 12 \text{ in./ft} = 318 \text{ lb}$

Note: 1 m = 3.28 ft

CONCLUSIONS

From the results of this study, the following conclusions may be drawn:

- Barriers installed using either the normal or revised tensioning procedures experienced greatest tension loss soon after the barrier was first tensioned and over the first winter. Tension losses generally continued after this point, but at a much slower rate. If the barrier is retensioned, a new cycle of tension loss occurs.
- Even with the proposed retensioning procedures, tension loss will continue to occur in cable guiderail, regardless of the installation procedures used. The revised procedure coupled with at least two retensionings will probably be necessary to confine losses to 200 lb.
- Substituting prestretched for normal cable in these guiderail installations does not solve the tension-loss problem. Installations using prestretched cable lose tension at almost the same rate as those using normal cable.
- Cable guiderail installations continually lose tension, and thus must be retensioned periodically. The data, however, were insufficient to estimate how often this must be done.

REFERENCES

1. Burnett, W. C., J. L. Gibson, and R. H. Freer. *New Highway Barriers: The Practical Application of Theoretical Design*. Research Report 67-1. Bureau of Physical Research, New York State Department of Public Works, Albany, May 1967.
2. Whitmore, J. L., R. C. Picciocca, and W. A. Snyder. *Testing of Highway Barriers and Other Safety Accessories*. Research Report 38. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Dec. 1976.
3. *Standard Specification-Construction and Materials*. Office of Engineering, New York State Department of Transportation, Albany, Jan. 1990.
4. *Standard Sheet 606-1R2*. New York State Department of Transportation, Albany, Oct. 1982.
5. Kenyon, W. D. *Cable-Guiderail Tension*. Research Report 124. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, July 1985.
6. Yang, W., N. J. Bruno, and W. D. Kenyon. *Tension Loss in Cable Guiderail*. Special Report 104. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, March 1992.
7. *Bethlehem Wire Rope for Bridges, Towers, Aerial Tramways, and Structures*. Catalog 2277. Bethlehem Steel Corporation, Bethlehem, Pa.

Publication of this paper sponsored by Committee on Roadside Safety Features.

Performance Level 1 Bridge Railings for Timber Decks

RONALD K. FALLER, MICHAEL A. RITTER, JAMES C. HOLLOWAY, BRIAN G. PFEIFER, AND BARRY T. ROSSON

Historically, very little research has been conducted for developing crashworthy railing systems for timber bridge decks. For timber to be a viable material in the new construction of bridges, vehicular railing systems must be developed and crash tested. The USDA Forest Service, Forest Products Laboratory, in conjunction with the Midwest Roadside Safety Facility, undertook the task of developing and testing three bridge railings—two glulam timber railing systems and one steel railing system—for use with longitudinal timber bridge decks. This research effort provided a variety of aesthetically pleasing and economical bridge-railing systems for timber decks. As part of the project, a series of four full-scale crash tests was conducted on the bridge-railing designs. The tests were conducted according to Performance Level 1 (PL1) specified in the AASHTO *Guide Specifications for Bridge Railings* (1989). The safety performance of each of the three bridge railings was acceptable according to the PL1 guide specifications.

Historically, most crashworthy bridge-railing systems located along highways have been developed using materials such as concrete, steel, and aluminum. In addition, most of these railing systems have been constructed on reinforced-concrete bridge decks. The demand for crashworthy railing systems has become more evident with the increasing use of timber bridge decks in secondary highways, county roads, and local road systems. Very little research has been conducted for developing crashworthy railings for timber bridge decks. For timber to be a viable material in the construction of bridges, additional vehicular railing systems must be developed and crash tested for timber bridge decks.

Of the many railing systems that have been crash tested successfully, only one involved a bridge railing for a timber bridge deck. This one study of significance was a 1988 Southwest Research Institute (SwRI) evaluation of a longitudinal glulam timber and sawn lumber curb system attached to a longitudinal spike-laminated timber deck (*1*). The evaluation was conducted according to Performance Level 1 (PL1) criteria specified in the AASHTO *Guide Specifications for Bridge Railings* (*2*).

Although the system met AASHTO PL1 requirements, damage consisting of delamination of the deck timbers and minor pull-out of several spikes was observed. As a result, the system has not been widely implemented in the field, and there continues to be a demand for crashworthy bridge-railing systems that would not cause damage to timber decks.

R. K. Faller, J. C. Holloway, B. G. Pfeifer, B. T. Rosson, Midwest Roadside Safety Facility, Civil Engineering Department, 1901 Y Street, P.O. Box 880601, University of Nebraska–Lincoln, Lincoln, Neb. 68588-0601. M. A. Ritter, U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, One Gifford Pinchot Drive, Madison, Wis. 53705.

OBJECTIVE

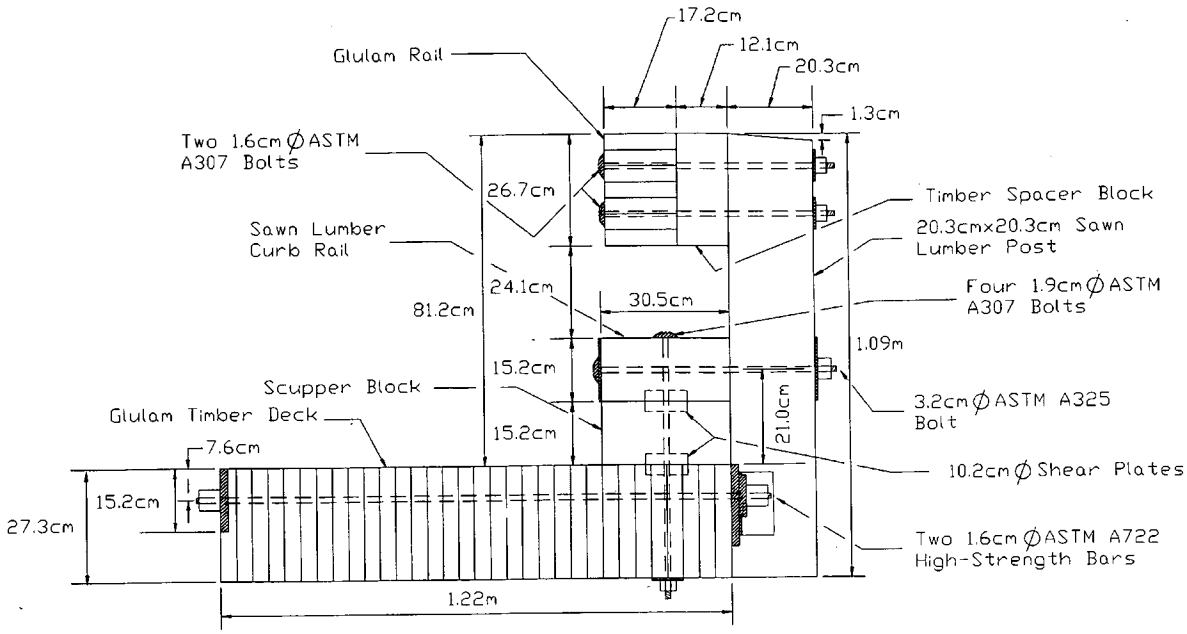
The objective of this research project was to develop bridge-railing systems for timber bridge decks while addressing concerns such as aesthetics and economy. The U.S. Department of Agriculture (USDA) Forest Service, Forest Products Laboratory, in cooperation with the Midwest Roadside Safety Facility (MwRSF), undertook the task of developing three bridge railings—two glulam timber bridge-railing systems and one steel bridge-railing system—that would be compatible with the existing types of longitudinal timber bridge decks. The longitudinal glulam timber bridge deck was selected because it was determined to be the weakest existing timber deck for transverse railing loads. If the bridge railings performed successfully on the glulam bridge deck, then the railing designs would be adaptable to most other longitudinal timber bridge-deck systems with no reduction in the railing performance depending on the specific railing systems. The bridge railings were developed to meet the AASHTO PL1 criteria while causing no damage to the longitudinal glulam timber deck.

The bridge railings included the following:

- A longitudinal glulam timber and sawn lumber curb bridge railing, the "curb system" (Figure 1),
- A single longitudinal glulam timber bridge railing without curb, the "shoe-box system" (Figure 2), and
- A single steel thrie-beam bridge railing, the "steel system" (Figure 3).

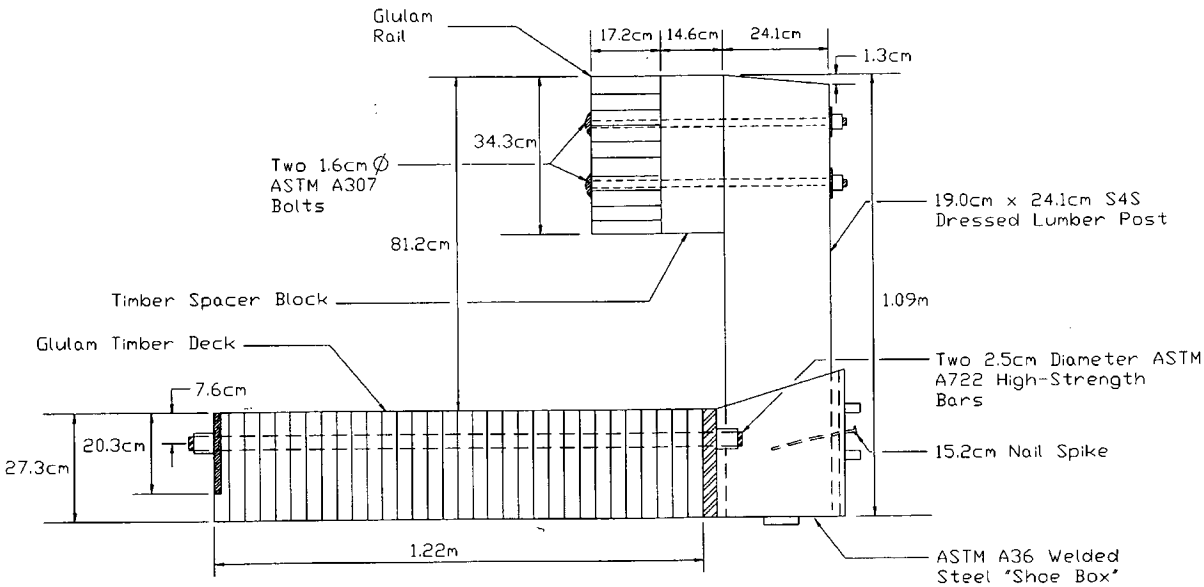
EVALUATION CRITERIA

The three bridge-railing systems were evaluated in accordance with the requirements for PL1 described in AASHTO (*2*). The full-scale crash tests were conducted and reported in accordance with *NCHRP Report 230* (*3*) as required by AASHTO (*2*). To be accepted for use in new construction, the bridge-railing systems were required to satisfy the performance evaluation criteria from two full-scale crash tests. The two required PL1 tests consist of a 2452-kg (5,400-lb) vehicle and an 817-kg (1,800-lb) vehicle impacting at 72.4 and 80.5 km/hr (45 and 50 mph), respectively, with impact angles of 20 degrees.



Notes: (1) Post Spacing 1.90m
(2) Many details have been omitted, refer to reference (10)
(3) 1in=2.54cm

FIGURE 1 Longitudinal glulam and sawn lumber curb bridge railing, or curb system.



Notes: (1) Post Spacing 1.90m
(2) Many details have been omitted, refer to reference (10)
(3) 1in = 2.54cm

FIGURE 2 Single longitudinal glulam bridge railing, or shoe-box system.

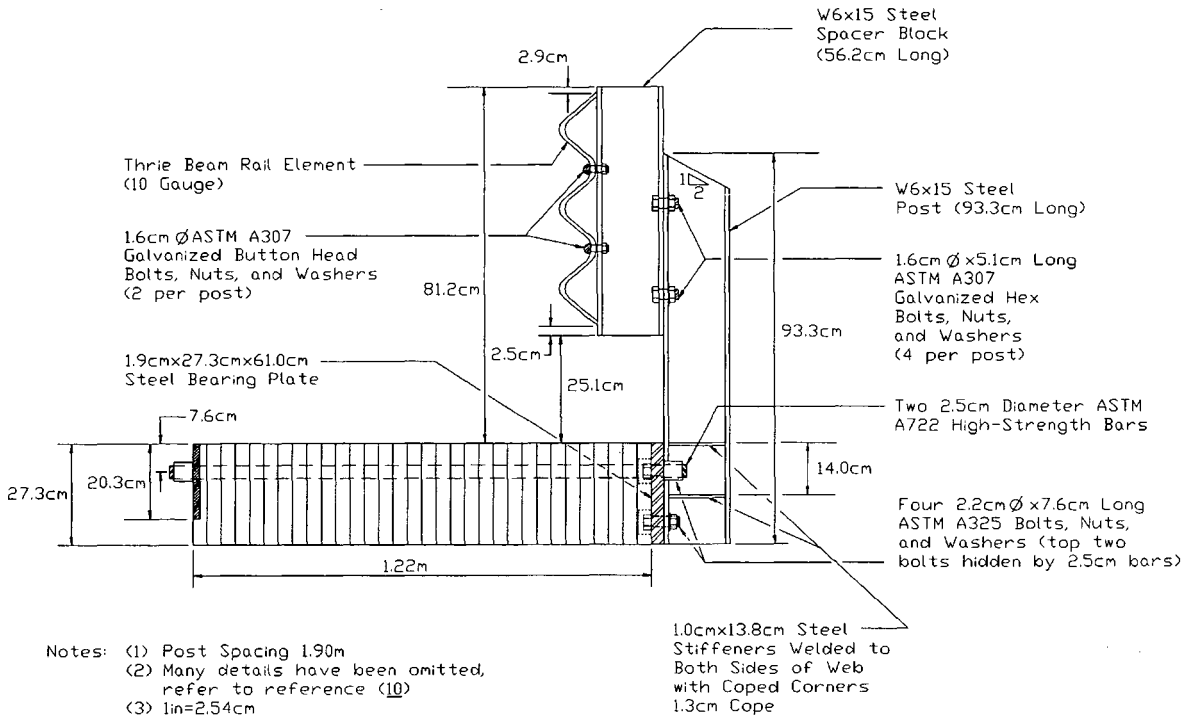


FIGURE 3 Single steel thrie-beam bridge railing, or steel system.

RESEARCH REVIEW AND APPROACH

In designing vehicular railing systems for highway bridges, engineers have traditionally assumed that vehicle impact forces can be approximated by applying equivalent static loads to the post and rail elements. Although rail loads are actually dynamic, the equivalent static load method has been used for years as a simplified approach to standardized railing design. The AASHTO *Standard Specifications for Highway Bridges* (*4*) currently requires that rail posts be designed to resist an outward transverse static load of 44.5 kN (10 kips). A portion of this load is also applied to posts in the inward transverse, longitudinal, and vertical directions and to the rail elements. These requirements are identical for all bridges irrespective of bridge geometry or traffic conditions; for example, railings for a single-lane bridge on a low-volume road must meet the same loading requirements as a bridge on a major highway. Computer simulation modeling and full-scale crash testing, however, have demonstrated that this procedure does not accurately estimate the actual vehicular impact forces transmitted to a bridge railing.

Specific requirements for bridge-rail crash testing have generally followed *NCHRP Report 230* procedures (*3*). In 1989 AASHTO adopted the *Guide Specifications for Bridge Railings* (*2*), which outlined the crash test requirements for three performance levels (PL1, PL2, and PL3); these specifications recommend full-scale vehicle crash testing of all railings used in new construction projects. In recent years, FHWA has recommended that full-scale crash testing be completed in accordance with the AASHTO guide specifications (*2*) for all vehicular railing systems. The testing requirements are based on the performance level of the bridge, which is defined by a number of factors including design speed, average daily traffic (ADT), percentage of trucks, bridge-rail offset, and number of lanes.

CURB SYSTEM

System Development

The timber bridge-railing system tested by SwRI provided the basis for the first design for the longitudinal glulam timber deck, although modifications and improvements were made to improve structural efficiency and reduce the cost of the system. Development of the modified longitudinal glulam timber and sawn lumber curb bridge railing, or curb system, consisted of redesigning the structural components and load transfer mechanisms used with the system previously tested on the spike-laminated timber deck.

The system evaluated at SwRI was constructed and tested with sawn lumber posts measuring 20.3 × 30.5 cm (8 × 12 in.) deep to accommodate AASHTO PL2 requirements. Researchers determined that this post size was in excess of that required for PL1, and that posts 20.3 × 20.3 cm (8 × 8 in.) would provide sufficient strength. The earlier system had also been constructed with a non-standard-size glulam rail of 15.2 × 27.3 cm (6 × 10¾ in.), whereas in the redesigned curb system, the rail section was modified to 17.1 × 26.7 cm (6¾ × 10½ in.) in order to obtain a more universal and economical size (*5*). The curb rail was maintained at a nominal size of 15.2 × 30.5 cm (6 × 12 in.). Hardware was also modified: in the previous system, the curb rail was attached to the deck with four ASTM A325 bolts 1.9 cm (¾ in.) in diameter, whereas for the curb system, it was assumed that four ASTM A307 bolts would provide adequate strength. Totally new to the

curb system design was the concept of using two high-strength bars located transversely in the timber deck at each post location to prevent failure of the longitudinal glulam timber bridge deck.

In order to verify and, if necessary, redesign the structural components and load transfer mechanisms (the posts, glulam and curb rails, structural bolts, and high-strength bars) (Figure 1), it was necessary to determine the lateral dynamic impact loads applied to the bridge rail. Two common methods were used:

1. An approximate method to predict the lateral impact load using a mathematical model taken from *NCHRP Report 86* (*6*) and the 1977 AASHTO Barrier Guide (*7*), and
2. Computer simulation with BARRIER VII (*8*) to analyze the response of the bridge railing during impact.

The first method or mathematical model (*6,7*) is represented by Equations 1 and 2:

$$F_{\text{lat.ave.}} = \frac{M\,g\,V_I^2 \sin^2\theta}{2g[\text{AL}\sin\theta - B(1 - \cos\theta) + D](1{,}000)} \quad (1)$$

$$F_{\text{lat.peak}} = F_{\text{lat.ave.}} \times \text{DF} \quad (2)$$

where

$F_{\text{lat.ave.}}$ = average lateral impact force (kN),
$F_{\text{lat.peak}}$ = peak lateral impact force (kN),
M = vehicle mass (2452 kg),
V_I = impact velocity (20.1 m/sec = 72.4 km/hr),
θ = impact angle (20 degrees),
g = acceleration due to gravity (9.81 m/sec^2),
AL = distance from vehicle's front end to center of mass (2.64 m),
$2B$ = vehicle width (1.98 m),
D = lateral displacement of railing (assumed 0 m), and
DF = dynamic factor ($\pi/2$).

For a 2452-kg (5,400-lb) pickup impacting at 72.4 km/hr (45 mph) and 20 degrees, $F_{\text{lat.ave.}}$ and $F_{\text{lat.peak}}$ were calculated to be 69 and 109 kN (15.5 and 24.4 kips), respectively.

The second method for calculating the lateral dynamic impact force was computer simulation with BARRIER VII (*8*). Impact conditions for AASHTO PL1 tests require a 2452-kg (5,400-lb) pickup at 72.4 km/hr (45 mph) and 20 degrees. Because the BARRIER VII program has been used extensively for computer simulation modeling with 2043-kg (4,500-lb) test vehicles, a 2043-kg sedan was used instead of the 2452-kg (5,400-lb) pickup for an approximate analysis. This reduced weight necessitated an increase in either the impact angle or the impact speed in order to provide similar loading conditions. In this case, an impact with a 2043-kg (4,500-lb) sedan at 72.4 km/hr (45 mph) and 25 degrees was assumed as a conservative estimate for the impact loading. This was in agreement with the results from Equation 1. The simulation runs were conducted at a midspan location between two timber bridge-rail posts. A 1977 Plymouth Fury weighing approximately 2043 kg (4,500 lb) was selected as the simulation test vehicle.

A series of pendulum tests conducted at SwRI on 20.3 × 20.3 cm (8 × 8 in.) Douglas fir posts cantilevered 61.0 cm (24 in.) above the base revealed a peak force approximately equal to 91 kN (20.4 kips) with a failure deflection of approximately 30.0 cm (11.8 in.) (*9*). The 47.0-cm (18.5-in.) effective post height for the curb system was less than the 61.0-cm (24-in.) height used in the SwRI tests. With this reduced height and assuming a linear relationship, an increased peak force of 117 kN (26.4 kips) and a reduced failure deflection of 23.1 cm (9.1 in.) were incorporated into the computer model. The structural properties for sawn lumber posts, glulam rail, and sawn lumber curb rail are shown by Ritter et al. (*10*). Computer simulation predicted that one post would reach a maximum shear force of 125 kN (28.0 kips) and a maximum dynamic post deflection of 17.3 cm (6.8 in.).

Design of High-Strength Bars

The peak lateral impact force was calculated to be approximately 109 kN (24.4 kips) using Equation 1, and 125 kN (28.0 kips) using BARRIER VII. It was assumed that 50 percent of the 125-kN (28.0-kip) impact force was transmitted to both the upper glulam rail and the lower curb rail. The impact load acting on the curb rail was assumed to transfer directly to the bars, while the impact load acting on the upper rail was assumed to transfer to the curb rail through the post-to-curb attachment and subsequently the bars. Based on these assumptions, it was estimated that the bars placed within the deck would be required to resist a force of approximately 125 kN (28.0 kips). High-strength threaded bars complying with the ASTM A722 designation were chosen, with the smallest size available—1.6-cm (⅝-in.) diameter—with an ultimate strength of 194 kN (43.5 kips); two bars were placed through the deck panels 7.6 cm (3 in.) below the top surface of the deck at each post location. This resulted in a somewhat conservative bar design that was maintained so as to ensure a successful test with no damage to the deck. Strain gauges were placed within the ends of selected bolts and bars to determine the transmitted loads.

Timber Deck and Substructure

A full-size simulated timber bridge system was constructed at the MwRSF. To simulate an actual timber bridge installation, the longitudinal glulam timber bridge deck was mounted on six reinforced-concrete bridge supports (*10*). The inner three concrete bridge supports had center-to-center spacings of 5.72 m (18 ft 9 in.), whereas the outer two spacings were 5.56 m (18 ft 3 in.).

The longitudinal glulam timber deck consisted of 10 rectangular panels. The panels measured 1.22 m (3 ft 11⅞ in.) wide, 5.70 m (18 ft 8½ in.) long, and 27.3 cm (10¾ in.) thick. The timber deck was constructed so that two panels formed the width of the deck and five panels formed the length of the deck. The longitudinal glulam timber deck was fabricated with West Coast Douglas fir and treated with pentachlorophenol in heavy oil to a minimum net retention of 9.61 kg/m^3 (0.6 lb/ft^3) as specified in AWPA Standard C14 (*11*). At each longitudinal midspan location of the timber deck panels, stiff-

ener beams were bolted transversely across the bottom side of the timber deck panels per AASHTO bridge design requirements. The stiffener beams measured 13.0 cm (5⅛ in.) wide, 15.2 cm (6 in.) thick, and 2.44 m (8 ft) long.

Glulam Timber and Sawn Lumber Curb Railing Design Details

The curb system consisted of four major components: sawn lumber scupper blocks, sawn lumber curb rail, sawn lumber posts, and longitudinal glulam timber rail. One timber scupper block was bolted to the timber deck at each post location with four ASTM A307 galvanized dome-head bolts 1.9 cm (¾ in.) in diameter and 61.0 cm (24 in.) long. The scupper blocks were 15.2 cm (6 in.) thick, 30.5 cm (12 in.) wide, and 0.91 m (3 ft) long and were attached to the curb rail and timber deck surface with shear plate connectors 10.2 cm (4 in.) in diameter. The sawn lumber curb rail was bolted to the top side of the scupper blocks. The nominal size of the curb rail was 15.2 cm (6 in.) deep and 30.5 cm (12 in.) wide with the top of the curb rail positioned 30.5 cm (12 in.) above the timber deck surface. One ASTM A325 dome-head bolt 3.2 cm (1¼ in.) in diameter and 55.9 cm (22 in.) long was used to attach each of the 15 bridge posts to the curb rail. Two high-strength bars 1.37 m (4 ft 6 in.) long were positioned transversely through the outer timber deck panel and spaced at 55.9 cm (22 in.) at each post. Fifteen sawn lumber Douglas fir posts measuring 20.3 cm (8 in.) wide, 20.3 cm (8 in.) deep, and 1.09 m (3 ft 6¾ in.) long and spaced at 1.90 m (6 ft 3 in.) on centers were used to support the upper glulam railing. The posts were treated to meet AWPA Standard C14 with 192.22-kg/m^3 (12-lb/ft^3) creosote (*11*). The longitudinal glulam rail was fabricated from West Coast Douglas fir and treated in the same manner as the timber deck. The glulam rail measured 17.1 cm (6¾ in.) wide and 26.7 cm (10½ in.) deep. The top mounting height of the glulam rail was 81.3 cm (2 ft 8 in.) above the timber deck surface. The glulam rail was offset from the posts with timber spacer blocks measuring 12.1 cm (4¾ in.) thick, 20.3 cm (8 in.) wide, and 26.7 cm (10½ in.) deep. Two ASTM A307 galvanized dome-head bolts 1.6 cm (⅝ in.) in diameter and 58.4 cm (23 in.) long were used to attach the glulam rail to the timber posts.

Full-Scale Crash Test

The two AASHTO PL1 crash tests conducted by SwRI (*1*) used an 825-kg (1,818-lb) minicompact at 95.3 km/hr (59.2 mph) and 20 degrees, and a 2385-kg (5,254-lb) pickup at 76.5 km/hr (47.5 mph) and 20 degrees. Because the basic geometry of the curb system was unchanged from the system tested by SwRI, it was not necessary to perform a test with a minicompact sedan. Because the structural components and load transfer mechanisms were modified, a test with a ballasted pickup was required to determine the structural adequacy and safety performance of the curb system.

Test C-1 [2,452 kg (5,400 lb), 71.0 km/hr (44.1 mph), 23.4 degrees] was conducted at the midspan location between Posts 7 and 8, which was 0.95 m (3 ft 1½ in.) upstream from a splice in the glulam rail (Figure 4). A summary of the test results and the sequential photographs are shown in Figure 5.

The vehicle became parallel to the timber bridge railing at approximately 0.273 sec with a velocity of 57.6 km/hr (35.8 mph). The vehicle exited the bridge railing at 0.434 sec at 56.4 km/hr (35.0 mph) and 5.3 degrees. The vehicle came to a stop 38.10 m (125 ft) downstream from impact. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic-side face of the rail was 3.10 m (10 ft 2 in.) at 24.38 m (80 ft) downstream from impact.

The moderate test vehicle damage is shown in Figure 4. The superficial damage to the bridge-railing system is shown in Figure 4. Minor scrapes occurred along both the upper glulam and lower curb rails. The length of vehicle contact on the upper glulam rail was approximately 2.74 m (9 ft). The lower curb rail received a gouge 15.2 cm (6 in.) long near impact. During the test, the 15.2-cm (6-in.) square steel plate washer located on the back side of Post 8 was deformed.

FIGURE 4 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test C-1.

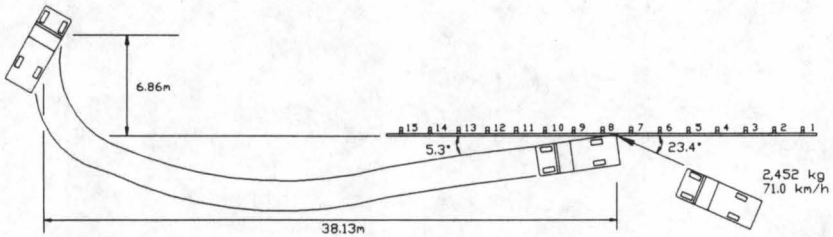
0.000 sec

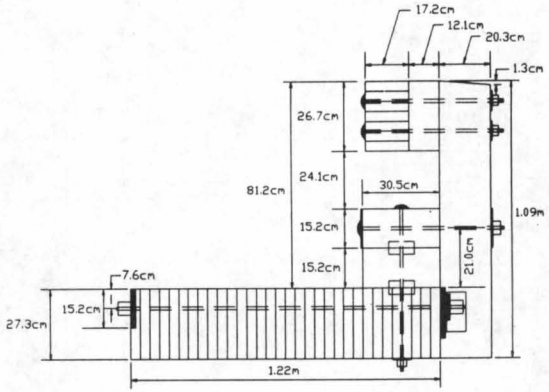
0.081 sec

0.179 sec

0.273 sec

0.540 sec





Test Number	C-1
Date	11/16/90
Bridge Rail Installation	Glulam Timber Bridge Rail With Sawn Lumber Curb Rail
Length	28.59 m
Upper Rail	
Width	17.2 cm
Depth	26.7 cm
Top Mounting Height	81.2 cm
Material	Glulam Rail-Comb. No. 2
Lower Rail (Curb)	
Width	30.5 cm
Depth	15.2 cm
Top Mounting Height	30.4 cm
Material	Sawn Lumber
Posts (No. 1 through 15)	
Size	20.3 cm x 20.3 cm x 108.6 cm
Material	Sawn Lumber
Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1984 Dodge Ram
Test Inertial Weight	2,452 kg
Gross Static Weight	2,452 kg

Vehicle Speed	
Impact	71.0 km/h
Exit	56.3 km/h
Vehicle Angle	
Impact	23.4 deg
Exit	5.3 deg
Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.27 (Fair)
Occupant Impact Velocity	
Longitudinal	3.26 m/s (9.15 m/s) (2)
Lateral	4.45 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	2.3 g's (15 G's) (2)
Lateral	9.6 g's (15 G's) (2)
Vehicle Damage	Moderate
TAD	1-RFQ-4
VDI	01RFEW2
Vehicle Rebound Distance	3.1 m @ 24.4 m
Bridge Rail Damage	Minor Scrapes on Rail and Curb
Maximum Post Deflection	
Permanent Set	0 m
Dynamic	13.2 cm

FIGURE 5 Summary of test results and sequential photographs, Test C-1.

Analysis of the available strain gauge data indicated that the actual loads transmitted to the connecting bolts and bars were lower than anticipated. The strain gauge located in an ASTM A722 bar 1.6 cm (⅝ in.) in diameter at Post 8 transmitted a load of approximately 28 kN (6.2 kips). For two bars, the load transferred to the deck would total approximately 55 kN (12.4 kips). The strain gauge located in the ASTM A325 bolt 3.2 cm (1¼ in.) in diameter at Post 9 carried a load of approximately 26 kN (5.9 kips). The actual loads carried by the structural bolts and bars were much less than the design loads, because the original design assumed the entire lateral load would not be distributed to more than one post, whereas the load was actually distributed over several posts.

SHOE-BOX SYSTEM

System Development

The single longitudinal glulam timber bridge railing or shoe-box system was developed to obtain an alternate glulam bridge railing without a curb. However, a single bridge-rail (side-mounted) design requires larger rail and post members, since the impact force is transferred to the single rail element. Posts with greater flexural stiffness were required to compensate for this increased moment. A sawn lumber post 20.3 cm (8 in.) wide and 25.4 cm (10 in.) deep was chosen for the initial computer simulation analysis; however, it was determined that an S4S dressed lumber post provided better quality control of cross-sectional dimensions. This control was critical to ensure correct placement of posts in the structural steel shoe-box support. The S4S dressed lumber post measured 19.0 cm (7½ in.) wide and 24.1 cm (9½ in.) deep. A single glulam rail 17.1 cm (6¾ in.) wide and 34.3 cm (13½ in.) deep was also selected (*5*). The structural properties for the sawn lumber posts and glulam rail are given by Ritter et al. (*10*). The glulam rail was determined to provide adequate moment capacity while also providing an adequate maximum clearance of 47.0 cm (18½ in.) from the top of the deck surface to the bottom of the glulam rail. For traffic railings, AASHTO (*4*) states that a maximum clear opening below the bottom rail shall not exceed 43.2 cm (17 in.), but with the rail blocked out 31.8 cm (12½ in.), the authors determined that with an 817-kg (1,800-lb) car, vehicle snagging would not occur on the post during impact.

It was necessary to determine the increased lateral dynamic impact loads applied to the bridge rail in the shoe-box system (Figure 2) to design the high-strength bars used to transfer the impact force into the timber deck and to verify the structural adequacy of the rail and post elements. The mathematical model (*6,7*) and computer simulation with BARRIER VII (*8*) were again used for the analysis. Using the mathematical model with a 2452-kg (5,400-lb) pickup impacting at 72.4 km/hr (45 mph) and 20 degrees and an assumed lateral railing displacement equal to 0.15 m (0.5 ft), $F_{lat.ave.}$ and $F_{lat.peak}$ were calculated to be 58 kN (13.1 kips) and 92 kN (20.6 kips), respectively. Computer simulation with BARRIER VII (*8*) was used with the same vehicle and impact conditions as for the curb system. The analysis of the simulation results revealed a maximum post shear force of 82 kN (18.5 kips) and a maximum dynamic deflection of 19.3 cm (7.6 in.).

Design of High-Strength Bars

The peak lateral impact force was calculated to be approximately 92 kN (20.6 kips) (*6,7*) and 82 kN (18.5 kips) (*8*). Considering the compressive bearing force between the sawn lumber post and the steel shoe box, it was estimated that the bars placed within the deck would be required to resist a force of approximately 485 kN (109.1 kips). High-strength threaded bars complying with the ASTM A722 designation were chosen. The next available size of threaded bar larger than 1.6 cm (⅝ in.) was 2.5 cm (1 in.) in diameter with an ultimate strength of 567 kN (127.5 kips). Two bars were placed through the deck panels 7.6 cm (3 in.) below the top surface of the deck. This resulted in a somewhat conservative bar design that was maintained so as to ensure a successful test with no damage to the deck. Strain gauges were placed within the ends of selected bars to determine the loads transmitted, and if necessary, redesign the hardware.

Single Glulam Timber Bridge-Railing Design Details

The concrete substructure used to support the timber deck for the curb system was maintained for the shoe-box system. However, the two rows of timber deck panels (five panels per row) were interchanged for the new bridge-rail configuration.

The shoe-box system consisted of three major components: dressed lumber posts, longitudinal glulam timber rail, and structural steel shoe boxes. The glulam railing was supported with 15 timber posts spaced 1.90 cm (6 ft 3 in.) on centers. The S4S dressed lumber posts measured 19.0 cm (7½ in.) wide, 24.1 cm (9½ in.) deep, and 1.09 m (3 ft 6¾ in.) long. The posts were treated with creosote to 192.22 kg/m^3 (12 lb/ft^3) to meet AWPA Standard C14 (*11*). The timber posts were attached to the longitudinal glulam timber deck with ASTM A36 structural steel shoe boxes. Fifteen welded-steel shoe boxes were fabricated with a steel plate 2.5 cm (1 in.) thick, 27.3 cm (10¾ in.) wide, and 61 cm (24 in.) long for the bearing surface with the remaining three sides of the box fabricated with 1.3-cm (½-in.) steel plate. A galvanized nail spike 0.6 cm (¼ in.) in diameter and 15.2 cm (6 in.) long was driven into the post through a hole located on the back side of the steel shoe box to prevent post pullout. Two high-strength bars 1.37 m (4 ft 6 in) long were positioned transversely through the outer timber deck panel and spaced at 40.6 cm (16 in.) at each post. The longitudinal glulam rail was fabricated from West Coast Douglas fir and treated in the same manner as the timber deck. The glulam rail measured 17.1 cm (6¾ in.) wide and 34.3 cm (13½ in.) deep. The top mounting height of the glulam rail was 81.3 cm (2 ft 8 in.) above the timber deck surface. The glulam rail was offset from the posts with timber spacer blocks measuring 14.6 cm (5¾ in.) thick, 20.3 cm (8 in.) wide, and 34.3 cm (13½ in.) deep. Two ASTM A307 galvanized dome-head bolts 1.6 cm (⅝ in.) in diameter and 61.0 cm (24 in.) long were used to attach the glulam rail to the timber posts.

Full-Scale Crash Tests

Test FSSB-1

Test FSSB-1 [2452 kg (5,400 lb), 72.4 km/hr (45.0 mph), 21.8 degrees] was conducted at the midspan location between Posts

7 and 8, which was 0.95 m (3 ft 1½ in.) upstream from a splice in the glulam rail (Figure 6). A summary of the test results and the sequential photographs are shown in Figure 7.

The vehicle became parallel to the bridge railing at 0.282 sec with a velocity of 62.0 km/hr (38.5 mph). The vehicle exited the bridge railing at 0.456 sec at 57.0 km/hr (35.4 mph) and 7.5 degrees. The vehicle came to a stop 30.5 m (100 ft) downstream from impact. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic-side face of the rail was 0.61 m (2 ft).

The moderate test vehicle damage is shown in Figure 6. The superficial damage to the bridge rail is shown in Figure 6. Both minor scrapes and gouging were evident along the traffic-side face of the bridge rail. Post 8 received some indentations and black marks on the upstream exposed corner; evidence of tire marks on the timber deck near Post 8 are shown in Figure 6. The gouging on the lower portion of the rail occurred from the minor snagging with the right-front wheel well and right-door panel joint. Additional gouging occurred to the top of the timber deck surface. The lower-rear side of Post 8 was compressed 0.3 cm (⅛ in.) due to bearing against the steel shoe box.

FIGURE 6 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test FSSB-1.

The analysis of the strain gauge data indicated that the actual loads transmitted to the high-strength bars were less than anticipated. The maximum load transmitted to an ASTM A722 bar was 164 kN (36.8 kips). The total combined load for the two bars at Post 8 was 301 kN (67.7 kips). A comparison between total load carried by the bars per post location and the maximum lateral dynamic deflection showed significant correlation (i.e., the maximum lateral dynamic post deflection corresponded to the maximum load carried in two bars at a post location) (*10*).

Test FSSB-2

Test FSSB-2 [839 kg (1,849 lb), 80.7 km/hr (50.1 mph), 21.5 degrees] was conducted at the midspan location between Posts 5 and 6, which was 0.95 m (3 ft 1½ in.) upstream from the centerline of Post 6 (Figure 8). A summary of the test results and the sequential photographs are shown in Figure 9.

The vehicle became parallel to the bridge railing at 0.204 sec with a velocity of 63.3 km/hr (39.3 mph). The vehicle exited the bridge railing at 0.442 sec at 60.9 km/hr (37.8 mph) and 6.7 degrees. The vehicle came to a stop 106.68 m (350 ft) downstream from impact. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic side face of the rail was 3.96 m (13 ft).

The minimal test vehicle damage is shown in Figure 8. The superficial damage to the bridge rail is shown in Figure 8. Both minor scrapes and gouging were evident along the traffic-side face of the bridge rail.

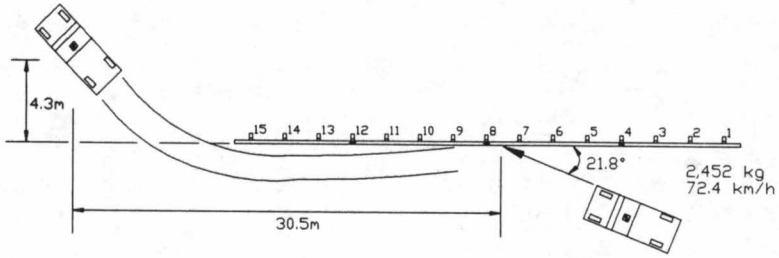
STEEL SYSTEM

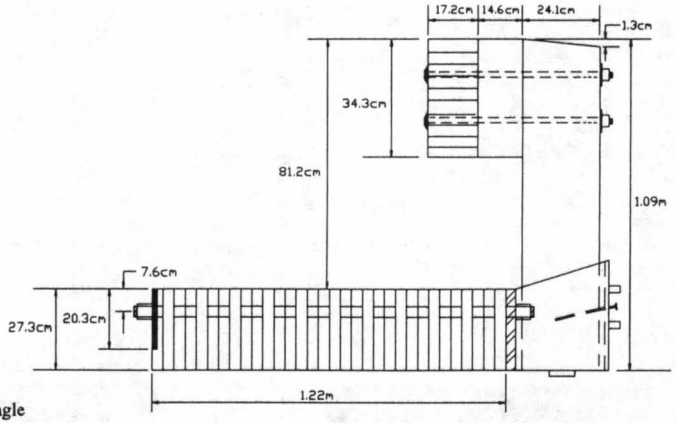
System Development

A single steel thrie-beam bridge railing, or steel system (Figure 3) was developed to obtain a more economical AASHTO PL1 bridge railing for timber decks. The two previously tested glulam bridge railings had higher initial material costs per foot than the steel system. The development of an AASHTO PL1 steel railing for timber decks began with a literature review of existing steel-rail bridge railings. Information gathered in the review suggested that side-mounted steel systems such as the NCHRP SL1 thrie beam (*12*), the Oregon side-mounted thrie-beam bridge rail (PL1) (*13,14*), the California Type 115 bridge rail (PL1) (*15*), and the California thrie-beam bridge rail (PL1) (*15*) could be modified for timber bridge decks. After reviewing data on the steel side-mounted systems, the California thrie-beam bridge rail was selected for modification to the timber bridge deck.

The steel system was attached to the longitudinal glulam timber deck with high-strength bars as previously used in the shoe-box system. Strain gauges were placed within the ends of the bars to determine the loads transmitted.

0.000 sec | 0.082 sec | 0.176 sec | 0.269 sec | 0.456 sec





Test Number	FSSB-1
Date	5/14/91
Bridge Rail Installation	Glulam Timber Bridge Rail With Steel Shoe Box
Length	28.59 m
Glulam Timber Rail	
Width	17.2 cm
Depth	34.3 cm
Top Mounting Height	81.2 cm
Material	Glulam Rail-Comb. No. 2
Posts (No. 1 through 15)	
Size	19.0 cm x 24.1 cm x 108.6 cm
Material	S4S Dressed Lumber
Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1984 Dodge Ram
Test Inertial Weight	2,452 kg
Gross Static Weight	2,452 kg
Vehicle Speed	
Impact	72.4 km/h
Exit	57.0 km/h

Vehicle Angle	
Impact	21.8 deg
Exit	7.5 deg
Vehicle Snagging	Minor Wheel Well and Door Joint gouging
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.20 (Good)
Occupant Impact Velocity	
Longitudinal	3.42 m/s (9.15 m/s) (2)
Lateral	5.19 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	2.0 g's (15 g's) (2)
Lateral	2.5 g's (15 g's) (2)
Vehicle Damage	Moderate
TAD	1-RFQ-3
VDI	01RFEW2
Maximum Vehicle Rebound Distance	0.61 m
Bridge Rail Damage	Minor Scrapes and Gouging on the Rail
Maximum Dynamic Deflection	
Rail	19.6 cm (Visible)
Post	23.6 cm

FIGURE 7 Summary of test results and sequential photographs, Test FSSB-1.

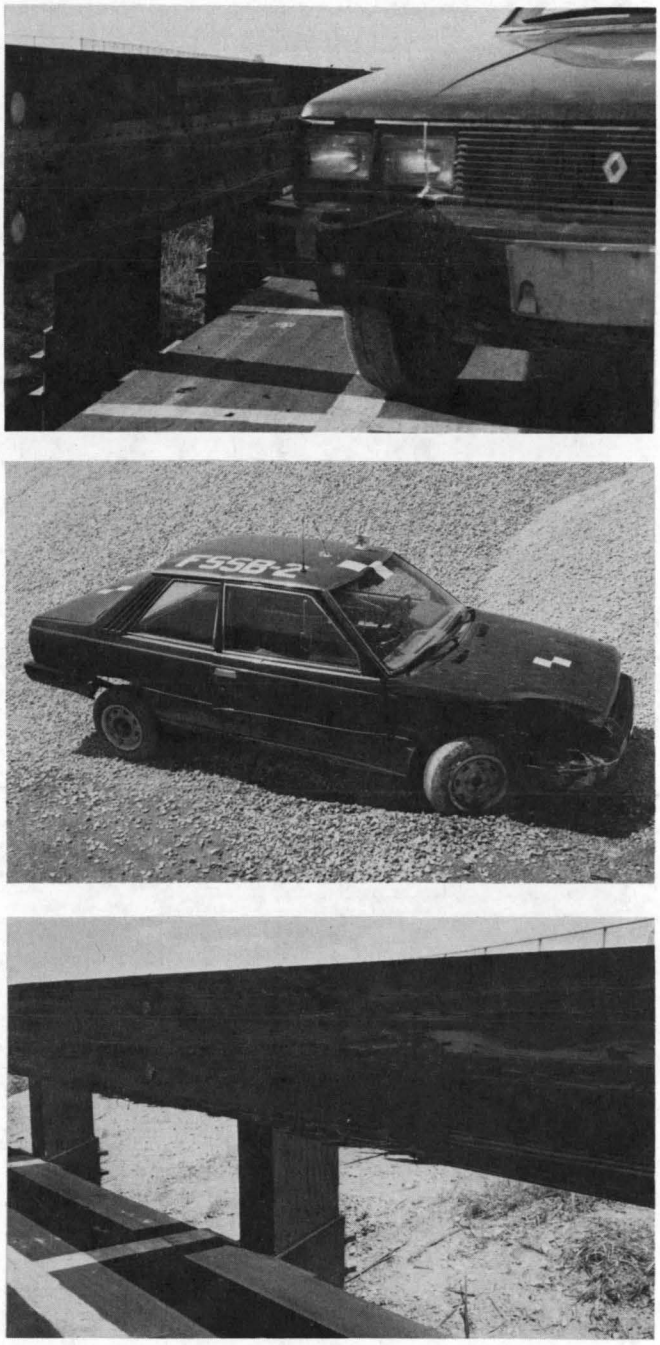


FIGURE 8 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test FSSB-2.

Single Steel Thrie-Beam Railing Design Details

The concrete substructure used to support the timber deck for the curb and shoe-box systems was maintained for the steel system, as was the configuration of the timber deck panels used during the testing of the shoe-box system.

The steel system consisted of three major components: structural steel posts, steel thrie-beam rail, and structural steel mounting plates. The 10-gauge steel thrie beam was supported by 15 ASTM A36 W6×15 structural steel posts measuring 93.3 cm (3 ft ¾ in.) long. The steel posts were attached to the longitudinal glulam timber deck with ASTM A36 structural steel mounting plates. Fifteen steel mounting plates measuring 1.9 cm (¾ in.) thick, 27.3 cm (10¾ in.) deep, and 61 cm (24 in.) long were attached to the deck with two ASTM A722 high-strength bars 2.5 cm (1 in.) in diameter and 137.2 cm (4 ft 6 in.) long spaced at 40.64 cm (16 in.) and located 7.6 cm (3 in.) below the top surface of the deck. Each steel post was bolted to a steel mounting plate with four ASTM A325 galvanized hex-head bolts 2.2 cm (⅞ in.) in diameter that were welded to the deck side of the steel plate. Four recessed holes were cut into the edge of the timber deck so that the steel mounting plate could be bolted flush against the vertical deck surface. The top mounting height of the thrie-beam rail was 78.4 cm (2 ft 6⅞ in.) above the timber deck surface. A 78.4 cm (2 ft 6⅞ in.) mounting height was selected in order to maintain compatibility with the transition section between the W-beam and the thrie-beam. The steel thrie-beam rail was offset from the posts with ASTM A36 W6×15 structural steel spacer blocks measuring 56.2 cm (1 ft 10⅛ in.) long.

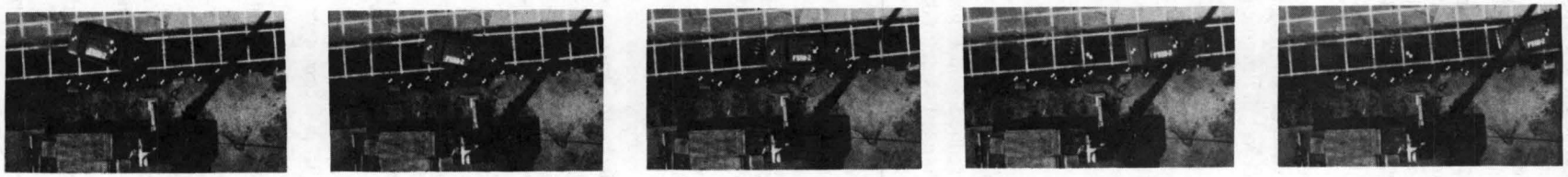
Full-Scale Crash Test

The California thrie-beam bridge rail 81.3 cm (32 in.) high successfully met AASHTO PL1 (*2*) and NCHRP 230 (*3*) requirements for structural adequacy, occupant risk, and vehicle trajectory (*15*). The geometry of the California thrie-beam bridge rail was basically unchanged with only a slight reduction in the effective railing height. It was not necessary to perform a test with an 817-kg (1,800-lb) vehicle impacting at 80.5 km/hr (50 mph) and 20 degrees, since there was no potential for wheel snagging or concern for occupant risk. The only concern with using an existing steel railing system was to verify the structural adequacy of the load transfer mechanism in the post-to-deck connection, which could be determined with a 2452-kg (5,400-lb) vehicle impacting at 72.4 km/hr (45 mph) and 20 degrees.

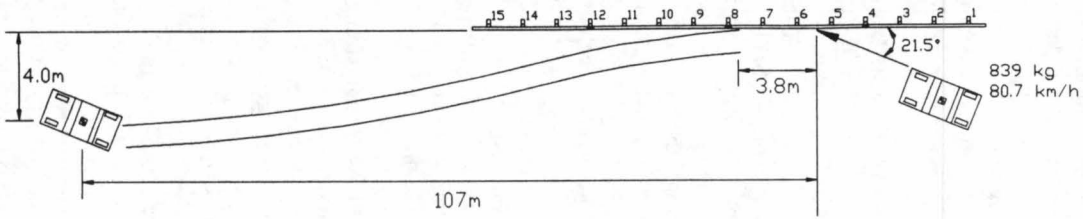
Test FSSR-1 [2542 kg (5,600 lb), 71.2 km/hr (44.2 mph), 19.1 degrees] was conducted at the midspan location between Posts 7 and 8, which was 0.95 m (3 ft 1½ in.) upstream from a central splice in the thrie-beam rail (Figure 10). A summary of the test results and sequential photographs are shown in Figure 11.

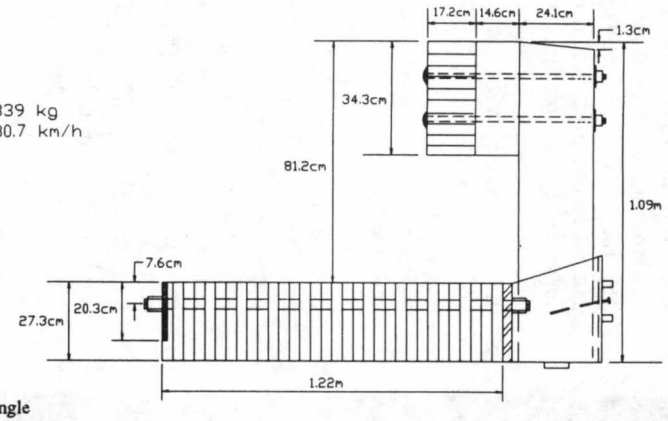
The vehicle became parallel to the bridge rail at 0.240 sec with a velocity of 57.3 km/hr (35.6 mph). The unrestrained onboard dummy impacted the right-side window at 0.291 sec. The vehicle exited the bridge railing at 0.401 sec at approximately 57.3 km/hr (35.6 mph) and 13.3 degrees. During vehicle redirection, the dummy launched out of the right-side window. The vehicle came to a stop 30.78 m (101 ft) downstream from the impact point. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic-side face of the rail was 1.78 m (5 ft 10 in.) at 18.29 m (60 ft) downstream from impact.

The minor test vehicle damage is shown in Figure 10. The moderate bridge-rail and post damage is shown in Figure 10. The physical damage to the thrie beam, consisting of scrapes, tire marks, and deformation, was measured to be 3.94 m (12 ft 11 in.) long.



0.000 sec | 0.084 sec | 0.172 sec | 0.294 sec | 0.442 sec





Test Number	FSSB-2
Date	6/12/91
Bridge Rail Installation	Glulam Timber Bridge Rail With Steel Shoe Box
Length	28.59 m
Glulam Timber Rail	
Width	17.2 cm
Depth	34.3 cm
Top Mounting Height	81.2 cm
Material	Glulam Rail-Comb. No. 2
Post (No. 1 through 15)	
Size	19.0 cm x 24.1 cm x 108.6 cm
Material	S4S Dressed Lumber
Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1984 Renault Alliance
Test Inertial Weight	839 kg
Gross Static Weight	839 kg
Vehicle Speed	
Impact	80.7 km/h
Exit	60.9 km/h

Vehicle Angle	
Impact	21.5 deg
Exit	6.7 deg
Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.40 (Marginal)
Occupant Impact Velocity	
Longitudinal	4.54 m/s (9.15 m/s) (2)
Lateral	6.59 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	1.1 g's (15 g's) (2)
Lateral	6.5 g's (15 g's) (2)
Vehicle Damage	Minimal
TAD	1-RFQ-3
VDI	01RFEW1
Vehicle Rebound Distance	3.97 m
Bridge Rail Damage	Minor Scrapes and Gouging on the Rail
Maximum Dynamic Deflection	
Rail	11.7 cm
Post	11.2 cm

FIGURE 9 Summary of test results and sequential photographs, Test FSSB-2.

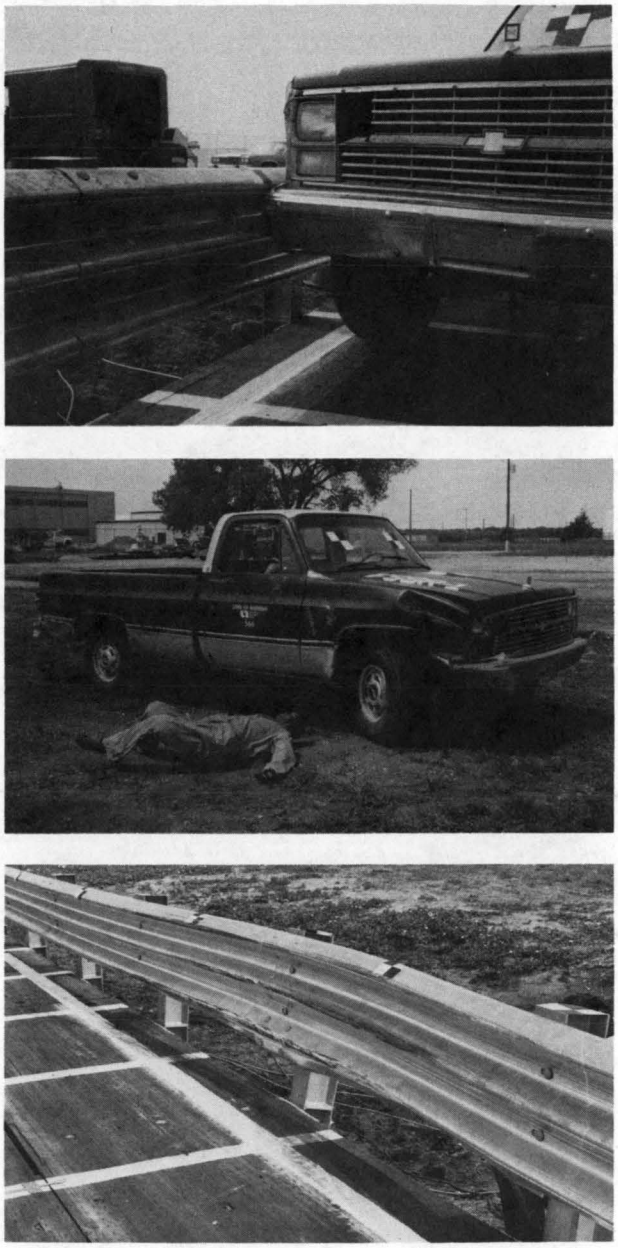


FIGURE 10 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test FSSR-1.

The analysis of the strain gauge data revealed that the maximum load transmitted to an ASTM A722 bar was 186 kN (41.8 kips). For two bars, the load transferred to the deck would total approximately 372 kN (83.6 kips).

RAILING COMPARISONS

Three AASHTO PL1 bridge railings were developed: two glulam timber railing systems and one steel railing system. All three bridge railings received acceptable safety performance evaluations according to the AASHTO guidelines (*2*), but the following comparisons can be made.

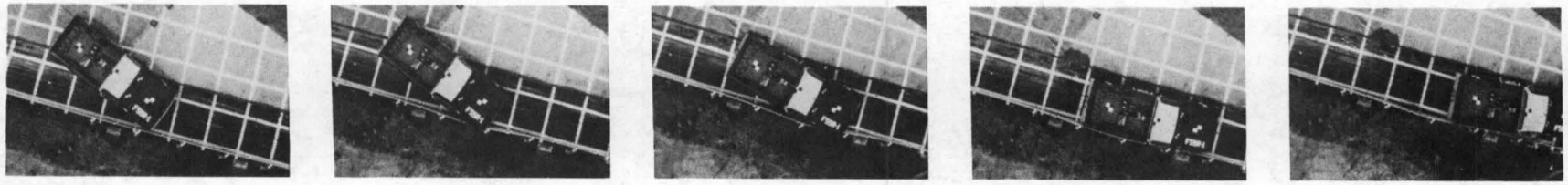
After each crash test, the examination of the top and bottom surfaces of the timber deck laminations revealed that there was no physical damage or separation. Therefore, the maintenance costs of these three railing systems should be less than that of the previously tested system on a spike-laminated timber deck (*1*). In addition, all three bridge railings were easy to install; thus, they should have low construction labor costs. The material costs for the curb system, the shoe-box system, and the steel system were approximately $325/m ($99/ft), $213/m ($65/ft), and $138/m ($42/ft), respectively.

Permanent structural damage as a result of impact occurred only for the steel system. The significant damage (plastic deformation) occurred to 3.94 m (12 ft 11 in.) of thrie-beam rail and three steel posts and mounting plates. This amount of damage would produce higher maintenance costs than for the other two systems. The post and rail members responded in an elastic manner with no measurable permanent set for the curb system and shoe-box system. Damage to the glulam railings for both of these systems was mostly superficial. Therefore, the glulam rails would remain functional from a structural adequacy point of view. From an aesthetical point of view, the glulam rails could be rotated 180 degrees so as to hide the superficial damage from view. Even after the glulam railing surfaces had been scraped, shredded, and exposed to the environment, the preservative treatment came to the surface, essentially providing future protection.

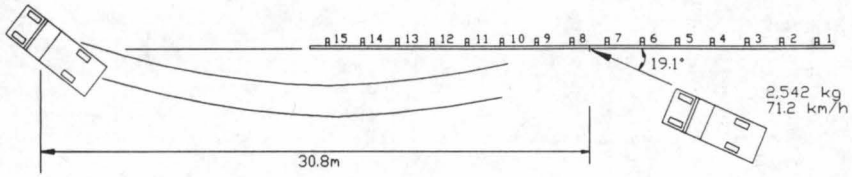
CONCLUSIONS AND RECOMMENDATIONS

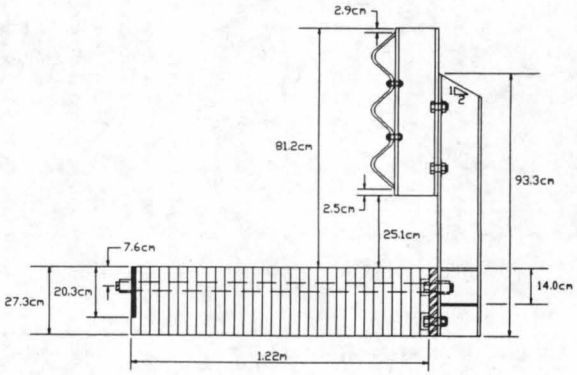
The safety performance evaluations of the curb system, the shoe-box system, and the steel system were acceptable according to the AASHTO PL1 guidelines (*2*). Therefore, three crashworthy bridge-railing systems were developed and are recommended for use on longitudinal timber bridge decks. Although the three bridge-railing systems were tested on a longitudinal glulam timber bridge deck, the three systems could be adapted to other longitudinal timber bridge decks. In addition, the development of the three systems addressed the concerns for aesthetics and economy. The curb system satisfied the concern for aesthetics, whereas the shoe-box system was aesthetically pleasing and more economical than the curb system. The steel system satisfied the basic need for developing an economical railing system for timber decks.

While the strain gauge results for the curb system indicated that one bar would provide sufficient strength, the use of two ASTM A722 high-strength bars 1.6-cm (⅝-in.) in diameter, or similar bars of comparable strength, is recommended for each post location. For economic considerations, additional research is suggested to redevelop the curb system using only one high-strength bar per post with smaller scupper blocks and less structural steel hardware. This redesign would reduce material costs by 14 percent to $279/m ($85/ft). The strain gauge results for the ASTM A325 3.2-cm (1¼-in) diameter bolt at the curb-to-post connection revealed that the load was not sufficient to warrant the use of high-strength material. Therefore, it is recommended that the design be modified to include an ASTM A307 3.18-cm (1¼-in.) diameter bolt. Similarly, it is recommended that two ASTM A722 high-strength bars 2.5-cm (1-in.) diameter or similar bars of comparable



0.000 sec 0.068 sec 0.158 sec 0.251 sec 0.401 sec





Test Number	FSSR-1
Date	6/4/92
Bridge Rail Installation	PL-1 Steel Rail with Steel Posts
Total Length	28.59 m
Steel Thrie Beam Rail	
Size	10 Gauge
Top Mounting Height	78.3 cm
Steel Posts (No. 1 through 15)	W6 x 15
Length	93.3 cm
Steel Spacer Blocks (No. 1 through 15)	W6 x 15
Length	56.1 cm
Vehicle Model	1984 Chevrolet Custom Deluxe 20
Test Inertial Weight	2,542-kg
Gross Static Weight	2,617-kg
Vehicle Speed	
Impact	71.2 km/h
Exit	57.3 km/h
Vehicle Angle	
Impact	19.1 deg
Exit	13.3 deg

Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.43 (Marginal)
Occupant Impact Velocity	
Longitudinal	4.09 m/s (9.15 m/s) (2)
Lateral	5.98 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	-2.7 g's (15 g's) (2)
Lateral	11.4 g's (15 g's) (2)
Vehicle Damage	Minor
TAD	1-RFQ-3
VDI	01RFEW2
Bridge Rail Damage	Moderate
Maximum Vehicle Rebound Distance	1.77 m @ 18.3 m
Maximum Permanent Set Deflection	
Rail	18.8 cm
Post	20.6 cm
Maximum Dynamic Deflection	
Rail	32.0 cm
Post	35.1 cm

FIGURE 11 Summary of test results and sequential photographs, Test FSSR-1.

strength be maintained at each post location for the shoe-box system and the steel system.

ACKNOWLEDGMENTS

The authors would like to thank the following organizations who have contributed to the success of this research project: the American Institute of Timber Construction, Vancouver, Washington, for donating the glulam materials for the deck and rail construction; Laminated Concepts, Inc., Elmira, New York, for donating structural hardware for the bridge rails; Western Wood Structures, Inc., Tualatin, Oregon, for drafting of preliminary designs and shop drawings; Midwest Transportation Center, Ames, Iowa, for graduate student support; and the Office of Sponsored Programs and the Center for Infrastructure Research, University of Nebraska–Lincoln, for matching support.

DEDICATION

This research project is dedicated to Edward R. Post (1934–1991), former Professor of Civil Engineering at the University of Nebraska–Lincoln and founder and Director of MwRSF. The development of crashworthy timber bridge railings was one of his last active research projects.

REFERENCES

1. Hancock, K. L., A. G. Hansen, and J. B. Mayer. *Aesthetic Bridge Rails, Transitions, and Terminals for Park Roads and Parkways.* Report FHWA-RD-90-052. FHWA, U.S. Department of Transportation, May 1990.
2. *Guide Specifications for Bridge Railings.* AASHTO, Washington, D.C., 1989.
3. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., March 1981.
4. *Standard Specifications for Highway Bridges.* AASHTO, Washington, D.C., 1983.
5. Ritter, M. A., *Timber Bridges—Design, Construction, Inspection, and Maintenance.* EM 7700-8. Forest Service, U.S. Department of Agriculture, June 1990.
6. *NCHRP Report 86: Tentative Service Requirements for Bridge Rail Systems.* HRB, National Research Council, Washington, D.C., 1970.
7. *Guide for Selecting, Locating, and Designing Traffic Barriers.* AASHTO, Washington, D.C., 1977.
8. Powell, G. H. *BARRIER VII: A Computer Program for Evaluation of Automobile Barrier Systems.* Report FHWA RD-73-51. FHWA, U.S. Department of Transportation, April 1973.
9. Michie, J. D., C. J. Gatchell, and T. J. Duke. Dynamic Evaluation of Timber Posts for Highway Guardrails. In *Highway Research Record 343*, HRB, National Research Council, Washington, D.C., 1971.
10. Ritter, M. A., R. K. Faller, J. C. Holloway, B. G. Pfeifer, and B. T. Rosson. *Development and Testing of Bridge Railings for Longitudinal Timber Decks by Full-Scale Crash Testing.* Forest Products Laboratory, Forest Service, U.S. Department of Agriculture, 1993.
11. *American Wood-Preservers' Association Book of Standards.* American Wood-Preservers' Association, Woodstock, Md., 1991.
12. Bronstad, M. E., and J. D. Michie. *NCHRP Report 239: Multiple-Service-Level Highway Bridge Railing Selection Procedures.* TRB, National Research Council, Washington, D.C., Nov. 1981.
13. Buth, C. E., D. Kaderka, and W. L. Campise. *Oregon Side-Mounted Thrie-Beam Bridge Rail—1,800-lb car, 52.2 mph, 19.7 deg.* Test Report 7069-17. Texas A&M Research Foundation, Texas Transportation Institute, Texas A&M University System, June 1989.
14. Buth, C. E., D. Kaderka, and W. L. Campise. *Oregon Side-Mounted Thrie-Beam Bridge Rail—5,400-lb pickup truck, 46.1 mph, 20.8 deg.* Test Report 7069-18. Texas A&M Research Foundation, Texas Transportation Institute, Texas A&M University System, June 1989.
15. Stoughton, R. L., D. Glauz, J. Jewell, J. J. Folsom, and W. F. Crozier. *Vehicle Crash Tests of Steel Bridge Barrier Rail Systems for Use on Secondary Highways.* Division of New Technology, Materials and Research, Office of Structural Materials, California Department of Transportation (in preparation).

Publication of this paper sponsored by Committee on Roadside Safety Features.

Vehicle Crash Tests of Concrete Median Barrier Retrofitted with Slipformed Concrete Glare Screen

PAYAM ROWHANI, DORAN GLAUZ, AND ROGER L. STOUGHTON

Two vehicle crash tests were performed on a retrofit concrete glare screen (CGS) slipformed on top of an existing concrete safety shape barrier 0.81 m (32 in.) high. The CGS is intended as a replacement for the current standard expanded metal mesh glare screen. This CGS is 0.51 m (20 in.) high, 0.15 m (6 in.) thick at the base, and tapers slightly to 0.13 m (5 in.) thick at the top. Reinforcing consists of two longitudinal #4 bars tied to vertical #5 hoops (inverted U-shaped stirrups). At the base of the glare screen is a 19-mm (¾-in.) chamfer to match that at the top of the concrete barrier. The two test vehicles included a pickup truck ballasted to 2447 kg (5,390 lb) traveling 89 km/hr (55.3 mph) and impacting at 20 degrees; and a station wagon ballasted to 1979 kg (4,360 lb), traveling 90 km/hr (56.2 mph) and impacting at 25 degrees. Both tests showed that a CGS can successfully withstand the impact of both a pickup truck and a heavy passenger car, and satisfy the requirements for structural adequacy, occupant risk and vehicle trajectory in *NCHRP Report 230* under these impact conditions. Maintenance costs for the CGS should be less than those for the metal mesh glare screen.

Since the early 1970s, headlight glare from opposing traffic has been of concern to traffic engineers. The standard material now used to screen glare in California is an expanded metal mesh mounted on top of concrete median barriers between opposing streams of traffic (*1*). This glare screen is installed only on barriers in medians that are less than 6.1 m (20 ft) in width, to shield driver's eyes from the headlight glare of oncoming vehicles (*2*).

In the mid 1980s the Division of Highway Maintenance concluded that the amount of time spent maintaining expanded metal mesh was excessive and exposed maintenance personnel and the traveling public to potential traffic safety hazards. Glare screens in the narrow medians appeared to be damaged easily by repeated wind gusts from passing trucks, wind, vandalism, roadway debris, and vehicle impacts. A value engineering team from the California Department of Transportation (Caltrans) recommended that the expanded mesh glare screen be replaced with a reinforced concrete glare screen (CGS) because of its greater strength and durability, its excellent glare protection, its low maintenance, and its added barrier protection. A CGS design needed to be crash tested to verify that it would not increase any safety concerns in an automobile crash.

California Department of Transportation, Division of New Technology, Materials, and Research, P.O. Box 19128, Sacramento, Calif. 95819.

SCOPE OF RESEARCH

A retrofit CGS slipformed on top of an existing safety shape concrete median barrier (CMB) was designed and crash-tested to be qualified for use on California state highways. This design, when adopted and implemented, would replace one using expanded metal mesh glare screen mounted on top of the CMB.

TEST BARRIER DESIGN AND CONSTRUCTION

The type 50R barrier that was crash-tested was a standard Caltrans slipformed CMB (Concrete Barrier Type 50) retrofitted with a slipformed CGS on top. The barrier design is shown in Figure 1. The barrier design was a joint effort by the researchers and personnel from the Caltrans Division of Structures. A California barrier contractor with considerable slipforming experience advised the researchers that it would probably be feasible to slipform a CGS on top of a CMB.

The CGS was lightly reinforced. Minimal reinforcement was needed to anchor the CGS to the existing CMB and to hold the CGS together if it was shattered during an impact by a passenger vehicle, to prevent large chunks of concrete from flying into the opposing lanes. Nevertheless, the reinforcement could not be so congested that the concrete could not be vibrated and consolidated properly during the slipform operation.

The CGS was 0.15 m (6 in.) wide where it sat on top of the CMB, the same as the 0.15-m (6 in.) stem width of the CMB. The CGS tapered slightly to a 0.13-m (5-in.) top width. Minimum taper was used to get as thick a CGS as possible for added strength and ease of slipforming, but it was also thought that a slight taper was required to slipform the CGS properly. The minimum concrete strength for the test barrier was specified to be 2.11 MPa (3,000 psi). It was intended that the strength not be too high so the test conditions would be conservative. Test cylinders made during construction showed a 7-day strength of 2.42 MPa (3,440 psi), and a 14-day strength of 2.72 MPa (3,870 psi). It was necessary to control the slump of the concrete during the slipforming operation in order for the concrete to hold its shape.

The height selected for the Type 50R barrier was 1.32 m (52 in.): a 0.81-m (32-in.) CMB plus a 0.51-m (20-in.) CGS. This is 51 mm (2 in.) higher than the minimum height recommended in the NCHRP Synthesis on glare screens (*3*). It is also higher than the vertical center of gravity height for

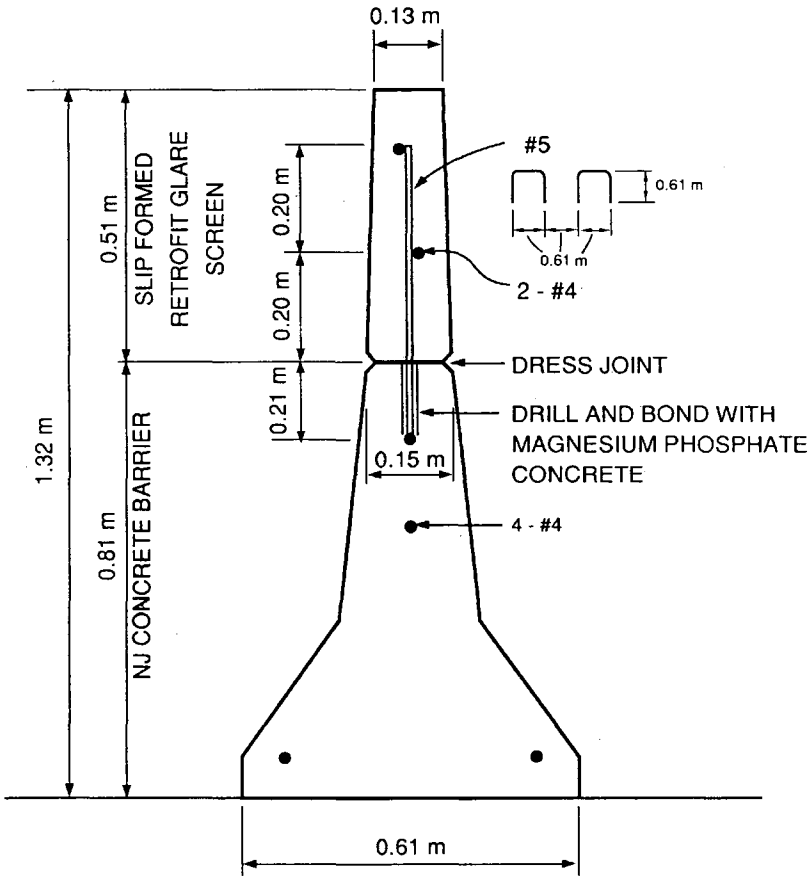


FIGURE 1 Type 50R typical cross section.

most vehicles. It is questionable whether a CGS this thin could be much higher and still be slipformed satisfactorily.

Scuppers, which are rectangular drainage slots in the barrier at ground level, were included in the test barrier design to provide the maximum weakness to the CMB that might be expected in practice.

The length of the test barrier was 45.7 m (150 ft). The construction operation had three major phases. First the CMB was built. Second, the reinforcement for the CGS was constructed on top of the CMB, and finally, the CGS was slipformed. The contractor had to build a new mule to accommodate the shape of the CGS.

TEST CONDITIONS

Test Facilities

The two impact tests were conducted at the Caltrans Dynamic Test Facility in West Sacramento, California. The tests were performed on a large flat asphalt-concrete surface. The test barrier was placed on the pavement.

Test Vehicles

Both vehicles used in these tests were in good condition and free of any major body damage or missing parts. They had front-mounted engines and automatic transmissions. The vehicle models and weights are given in the following table:

Test	*Vehicle*	*Steel Plate Ballast Weight [kg (lb)]*	*Total Test Inertial Weight [kg (lb)]*
481	1985 Chevrolet pickup	386 (850)	2445 (5,390)
482	1982 Mercury station wagon	136 (300)	1977 (4,360)

The vehicles were self-powered in all tests, with the engine being cut off before impact.

Test Instrumentation

Test vehicles were instrumented with two sets of three accelerometers (independently recorded) and rate gyros near the center of gravity of the vehicle. Potentiometers were attached to the top of the CGS in the impact area. They measured the dynamic deflection of the CGS during impact. Several high-speed cameras were used to record the impact.

Other Tests

Two tests were performed to check the integrity of the barrier and its materials. In the first a couple of typical cross sections were cut out to check for rebar arrangement and concrete consolidation. A circular saw was used to cut two cross-sections

0.15 m (6 in.) thick from the barrier. These cross sections showed that the final position of the rebars was the same as the plans. They also showed a homogeneous, well-consolidated concrete mixture with no air pockets.

The second test was a radiography test performed using standard radiographic methods. The main purpose here was to locate the rebar in the barrier so that the typical cross-sections could be cut out at an appropriate location along the barrier. The x-rays were also used to check for air pockets in the concrete, to check whether there was any considerable movement in the rebar during the slipform operation, to check for uniform concrete density along the barrier, and to see whether the concrete was intimately in contact with the rebars. A secondary purpose of these tests was to check for cracks in the concrete.

TEST RESULTS

Test 481

The planned test conditions for Test 481 were 2450 kg (5,400 lb) at 97 km/hr (60 mph) and 20 degrees (Figure 2).

Test Description

The left front tire of the vehicle made first contact with the face of the barrier 32.9 m (108 ft) from the upstream end of the barrier (see Figures 3 and 4). The vehicle impact speed was 89 km/hr (55.3 mph) and the impact angle was 20 degrees. The vehicle rose about 0.8 m (2.75 ft) above the ground as evidenced by the marks on the barrier. The left front corner of the vehicle remained in contact with the barrier for a distance of about 5.2 m (17.2 ft).

The left rear tire touched the lower part of the barrier 34.0 m (111.5 ft) from the upstream end of the barrier. The highest mark of the left rear tire on the barrier was 0.94 m (37 in.). The length of vehicle contact with the barrier was about 5.6 m (18.3 ft). The body contact of the vehicle with the concrete glare screen began 24.5 m (107 ft) from the upstream end of the barrier and ran for a length of about 3.3 m (11 ft). The

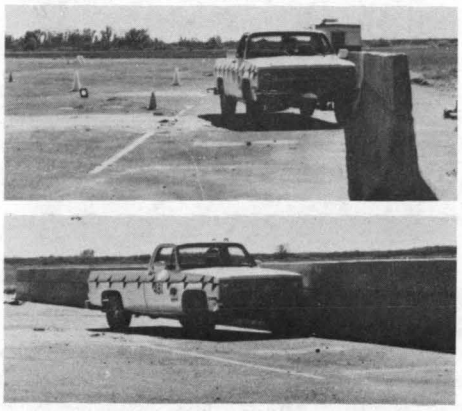
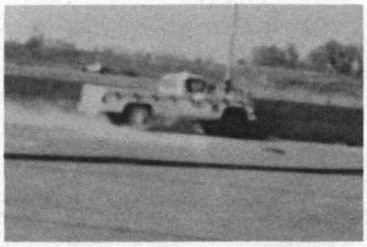
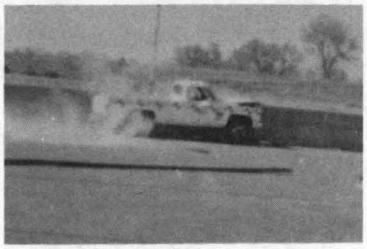


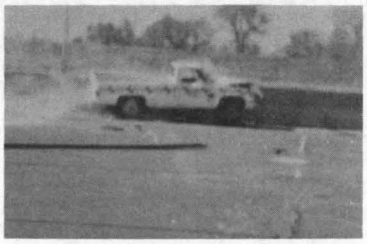
FIGURE 2 Test 481, preimpact photographs.



(a)



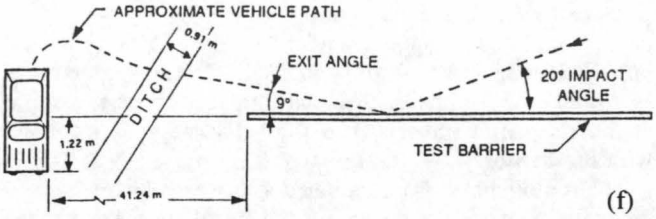
(b)



(c)

(d)

(e)



(f)

FIGURE 3 Test 481, vehicle trajectory and sequential test: (*a*) t = 0.00 sec; (*b*) t = 0.05 sec; (*c*) t = 0.10 sec; (*d*) t = 0.15 sec; (*e*) t = 0.20 sec; (*f*) vehicle trajectory.

FIGURE 4 Test 481, postimpact photographs: (*a*) left front tire of vehicle, (*b*) left front corner of vehicle, (*c–f*) barrier.

maximum height of tire marks on the barrier was 0.94 m (37 in.).

The pickup truck was redirected smoothly and lost contact with the barrier at an exit angle of 6 degrees without exhibiting any tendency to snag or pocket. During barrier impact, the truck experienced a maximum roll towards the barrier of 13.1 degrees and a pitch-up of 9.4 degrees. The vehicle remained upright throughout and after collision. The exit speed was 73.6 km/hr (45.7 mph). Exit velocity and angle were measured at the time after impact when the vehicle first lost contact with the barrier.

The remote brakes were applied after the vehicle lost contact with the barrier and went off the paved area. The vehicle rested perpendicularly at 41.3 m (135.3 ft) downstream from the downstream end of the test barrier. The vehicle was severely damaged. The maximum 50-msec average accelerations were $-11.3\ g$ in the lateral direction and $-4.6\ g$ in the longitudinal direction. The values of occupant impact velocity were 6.3 m/sec (20.6 ft/sec) in the lateral direction and 3.0 m/sec (9.9 ft/sec) in the longitudinal direction. The ridedown accelerations were $-20.7\ g$ laterally and $1.4\ g$ longitudinally.

Barrier Damage

There was no evidence of any structural distress of the CGS or CMB (see Figure 4). A few hair-like cracks were observed but were indistinguishable from cracks due to shrinkage. They may have existed before the crash. Lateral movement of the CGS was measured during the test; dynamic deflection was up to 10 mm (0.39 in.) at the top face, but there was no permanent deflection.

The only damage to the barrier were a few scrapes and tire marks. The length of gouge marks was 3.3 m (11 ft) on the CGS and about 3.6 m (12 ft) on the CMB. The tire marks of the left front wheel scuffed a length of 1.6 m (5.2 ft) on the CMB. The left rear tire marks were 3.5 m (11.4 ft) long on the CMB and 0.76 m (2.5 ft) on the CGS.

Test 482

The planned test conditions for Test 482 were 2043 kg (4,500 lb) at 97 km/hr (60 mph) and 25 degrees (Figure 5).

Test Description

The left front bumper of the vehicle first contacted the barrier face 15 m (49.2 ft) from the upstream end of the barrier (see Figures 6 and 7). The measured impact speed was 90.4 km/hr (56.2 mph), at an impact angle of 25 degrees. The left front tire initially contacted the lower part of the CMB 15.3 m (50.3 ft) from the upstream end. The highest mark on the barrier was 0.8 m (32 in.). The body contact with the CMB extended for a length of 3.8 m (12.5 ft) starting 15 m (49.3 ft) from the upstream end of the barrier. The length of the body contact with the CGS began 15.5 m (50.8 ft) from the upstream end of the barrier.

The test vehicle was redirected smoothly without exhibiting any tendency to snag or pocket with an exit angle of 5 degrees. It remained upright throughout and after collision. The exit velocity was 68.8 km/hr (42.6 mph). Exit velocity and angle

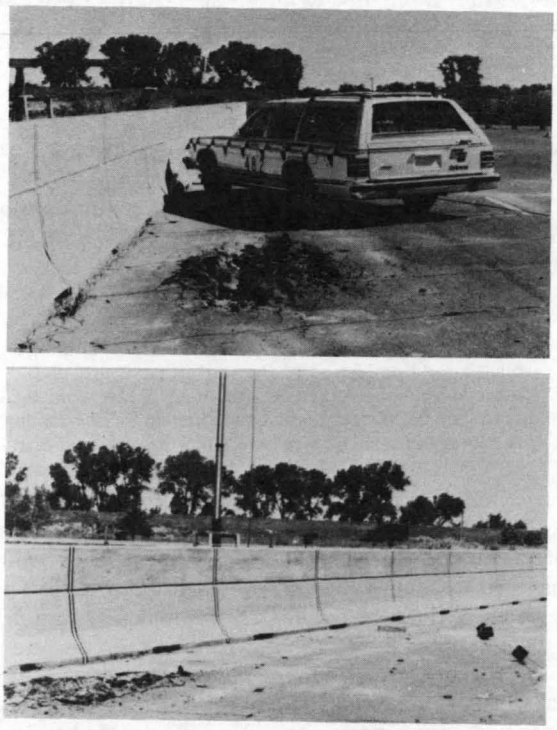


FIGURE 5 Test 482, preimpact photographs: *top*, vehicle; *bottom*, barrier.

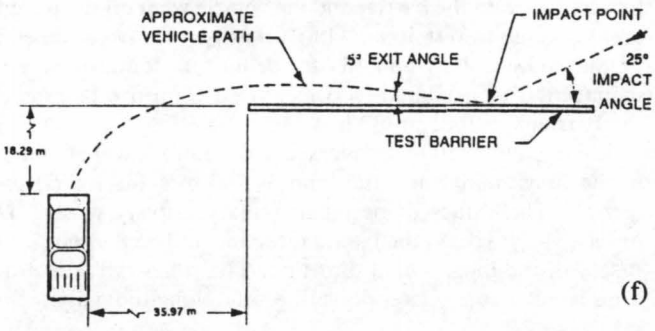
(a)

(b)

(c)

(d)

(e)



(f)

FIGURE 6 Test 482, vehicle trajectory and sequential test: (*a*) t = 0.00 sec; (*b*) t = 0.05 sec; (*c*) 0.10 sec; (*d*) t = 0.15 sec; (*e*) t = 0.20 sec; (*f*) vehicle trajectory.

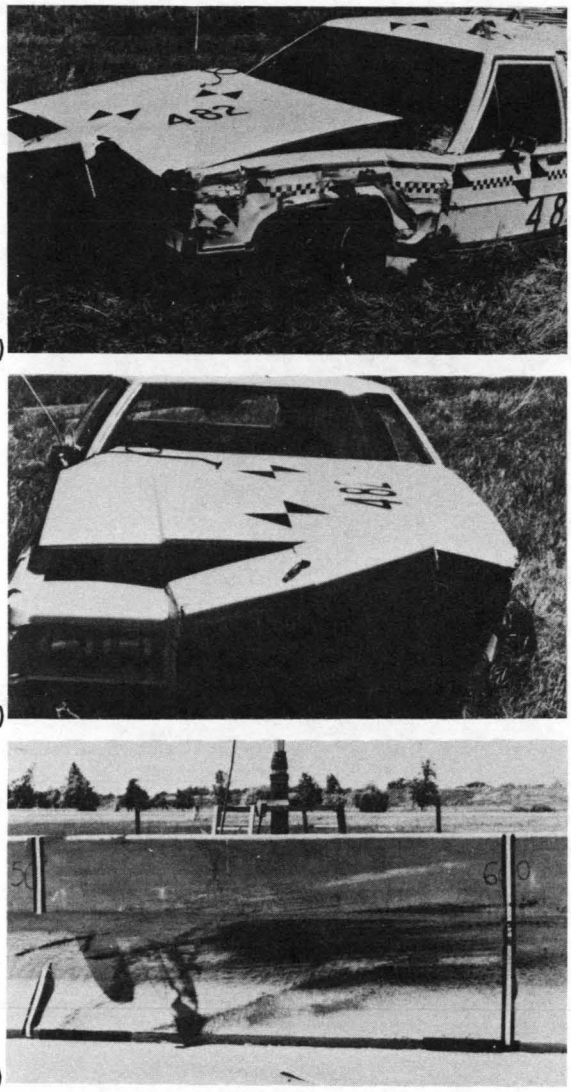


FIGURE 7 Test 482, postimpact photographs: (*a*,*b*) vehicle, (*c*) barrier.

are measured at the time after impact when the vehicle loses contact with the barrier.

During redirection, the test vehicle experienced a maximum roll towards the barrier of 18 degrees and a pitch-down of 5 degrees. The remote brakes were applied after the vehicle lost contact with the barrier and the vehicle went off the paved area. It came to rest 36 m (118 ft) downstream from the end of the CGS and 18.3 m (60 ft) behind it. It was severely damaged. The maximum rise measured from tire marks on the barrier was 0.81 m (32 in.).

The maximum 50-msec average accelerations were $-6.6\ g$ in the longitudinal direction and $-10.0\ g$ in the lateral direction. The values of occupant impact velocity were 6.47 m/sec (21.2 ft/sec) in the lateral direction and 6.68 msec (21.9 ft/sec) in the longitudinal direction. The ridedown accelerations were $-16.3\ g$ laterally and $-5.5\ g$ longitudinally.

Barrier Damage

There was no evidence of any structural distress of the CGS or CMB (see Figure 7). The only damage imparted to the barrier was minor extension of a few preexistent hair line cracks. About eight hair-like cracks 25 to 114 mm (1 to 4½ in.) long developed on the top face of the CGS over a length of 1.1 m (3.5 ft) upstream from the 18.3-m (60-ft) mark. A preexisting crack root on the barrier face at about 18.3 m (60 ft) from the barrier upstream end went on for an additional 38 mm (1.5 in.). Two preexisting contraction cracks on the back of the barrier branched out into five hair-like cracks 76 to 102 mm (3 to 4 in.) long. These preexisting cracks were located at 0.12 m (0.4 ft) and 1.95 m (6.4 ft) downstream from the 18.3-m (60-ft) mark.

The length of gouges from the vehicle was 3.1 m (10 ft) on the CGS and about 3.8 m (12.5 ft) on the CMB. The tire marks covered a length of 3.2 m (10.5 ft) on the CMB. An oil spill on the barrier face covered a length of about 2.7 m (9 ft) starting at 1.4 m (4.7 ft) downstream from the 15.2-m (50-ft) mark.

The maximum dynamic lateral displacement was 5.2 mm (0.21 in.) measured 13 mm (½ in.) from the top, and there was no permanent deflection.

DISCUSSION OF TEST RESULTS

General Safety Evaluation Guidelines *NCHRP Report 230*

Three evaluation factors were used in judging the impact test performance of the test barrier, as recommended by *NCHRP Report 230* (*4*). These factors are (a) structural adequacy, (b) occupant risk, and (c) vehicle trajectory. Tests 481 and 482 were performed to verify the structural adequacy of the CGS. The occupant risk and vehicle trajectory requirements were satisfied in other New Jersey shape barrier tests. Nevertheless, they were analyzed in these tests for comparison with past tests.

Structural Adequacy

The structural adequacy was evaluated by comparison of test results with the following criteria from Table 6 of *NCHRP Report 230* (*4*).

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.
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.

These criteria were met completely in both Test 481 and Test 482. The CMB/CGS demonstrated its ability to retain and redirect the test vehicles under different impact conditions. Vehicle redirection was very smooth in both tests. The vehicles were redirected adequately without penetration, and the overall adequacy of the barrier and the concrete glare screen were demonstrated. In these tests there was no evidence of any structural distress of the barrier; however, there were some minor surface cracks. No pieces of the barrier were broken, and no portions of the barrier showed potential for penetrating the passenger compartment. Both Test 481 and Test 482 were performed on the same barrier with impact

points 15.3 m (50 ft) apart. Lateral movement of the CGS was measured during the test; dynamic deflection was up to 10 mm (0.4 in.), but there was no permanent deflection.

Occupant Risk

The occupant risk was evaluated by comparison of test results with the following criterion from Table 6 of *NCHRP Report 230* (*4*).

> 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.

Table 1 presents maximum roll, airborne distance, and maximum 50-msec average accelerations for both Tests 481 and 482. Included in the table, for comparison, are similar data from previous tests on concrete safety shape barriers tested by Caltrans. Note that the magnitude of roll in Tests 481 and 482 is generally lower than in some other tests of concrete safety shape barriers. In both tests the amount of roll and pitch may be considered low to moderate. Neither of the two test cars showed any indication of being close to rollover. There was no deformation or intrusion into the passenger compartment.

The values of longitudinal occupant impact velocity in Tests 481 and 482 were lower than the NCHRP-recommended maximum value and also lower than in some other Caltrans tests on concrete median barriers. Limiting values of occupant impact velocity are given in Criterion F; however, they apply only to lightweight car tests. Nevertheless, the values were calculated and are reported here for comparison with those in similar tests.

The second part of Criterion F in *NCHRP Report 230* calls for a highest 10-msec average value of longitudinal and lateral vehicle acceleration of 15 *g* after the theoretical occupant/compartment impact occurs. The threshold value is specified as 20 *g*. Even though the lateral ridedown acceleration for Test 481 was 20.7 *g*, we feel that the test essentially met this criterion. Since so many uncontrolled variables contribute to the outcome of these crash tests, the value of 20.7 *g* would be within statistical error bounds. Test 482 had both longitudinal and lateral acceleration values below threshold.

Values of occupant impact velocity were not reported for the previous Caltrans tests on CMB because they were conducted before 1981 (when *NCHRP Report 230* was first published). Occupant impact velocities were introduced for the first time in that report. The maximum 50-msec average value of acceleration is a comparable measurement and was reported for all previous Caltrans CMB tests. These values for the CMB/CGS in Tests 481 and 842 are generally less than the values for the previous tests with similar test conditions. Hence, it can be concluded that the occupant risk for the CMB/CGS barrier is no worse than that for the Caltrans standard CMB.

The maximum vehicle rise in these tests ranged between 10.1 and 20.1 m (33 and 66 in.) compared with 9.8 and 11.3 m (32 and 37 in.) for the CGS tests. It should also be noted that the vehicle roll experienced in the CGS tests are comparable to the lowest roll values from previous tests.

It should also be noted that even though the impact speed of the vehicles was below that recommended in NCHRP 230 and the lateral kinetic energy was not within the bounds given in NCHRP 230, these parameters are used to determine the strength of the barrier, not the CGS. The barrier on which the CGS was mounted for testing was a standard California Type 50 (New Jersey profile) barrier which has been tested extensively and has passed NCHRP 230 criteria. The CGS does not reduce the structural strength of the barrier, it adds to it, and hence the lower speed impacts and lower lateral kinetic energies were sufficient for testing the strength of the CGS.

None of these means of evaluating occupant risk are exact methods of predicting injury levels during impacts. *NCHRP Report 230* states that "whereas the highway engineer is ultimately concerned with safety of the vehicle occupants, the occupant risk criteria should be considered as the guidelines for generally acceptable dynamic performance. These criteria are not valid, however, for use in predicting occupant injury in real or hypothetical accidents." The explanation is given

TABLE 1 Comparison of CGS Tests 481 and 482 to Concrete Safety Shape Barriers Tested by Caltrans

Test#/ Year	Ref. #	Weight (kg) Speed (km/h) Angle (°)	Exit Speed (km/h) Angle (°)	Severity Index (kN.m)	Airborne Distance (m)	Max. Roll (°)	50 ms avg. Accel. Long.	50 ms avg. Accel. Lat.
481 / 1990		***2447 / 89.3 / 20.0***	***73.6 / 9***	***88.0***	***0***	***13.1***	***-4.6 g***	***-11.3 g***
482 / 1990		***1979 / 90.5 / 25.0***	***68.8 / 5***	***111.4***	***0***	***18***	***-6.6 g***	***-10 g***
261 / 1972	5	2252/ 98.2 / 9.5	na / 0	22.8	na	na	0.6 g	3.9 g
262 / 1972	5	2252 / 95.0 / 25.0	91.8 / na	139.6	15.3	na	7.0 g	11.6 g
264 / 1972	5	2206 / 103.0 / 25.0	86.9 / 5	161.1	6.1	na	5.2 g	13.0 g
291 / 1972	5	2206 / 104.7 / 7.0	86.9 / 18	13.8	na	18	1.2 g	3.4 g
292 / 1972	5	2206 / 109.5 / 23.0	na / na	155.4	17.1	61	6.8 g	11.8 g
293 / 1973	5	2206 / 106.3 / 40.0	na / na	396.2	18.3	18	12.8 g	6.5 g
294 / 1974	5	2134 / 62.8 / 25.0	na / 4	57.9	4.9	33	2.7 g	5.5 g
301 / 1974	6	2206 / 109.5 / 27.0	80.5 / 13.8	209.7	9.2	26.5	11.7 g	13.8 g
321 / 1976	7	2134 / 98.2 / 26	72.5 / 7	152.3	1.5	48	4.4g	9.9g

Conversion Factors: 1 kg = 2.20 lb, 1 m = 3.28 ft, 1 km/h = 0.62 mph, 1 kN.m = 0.74 kip.ft,

that "relationship between vehicle dynamics and probability of occupant injury and degree of injury sustained is tenuous, because it involves such important but widely varying factors as occupant physiology, size, seating position, restraint, and vehicle interior geometry and padding." However, low occupant impact velocity and ridedown acceleration values indicate relatively safe roadside safety features.

Vehicle Trajectory

The vehicle trajectory was evaluated by comparison of test results with the following criteria from Table 6 of *NCHRP Report 230* (*4*):

H. After collision, the vehicle trajectory and final stopping position shall intrude a minimum distance, if at all, into adjacent traffic lanes.
I. In tests 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% of test impact angle, both measured at time of vehicle loss of contact with test device.

The same report stresses that "trajectory evaluation for redirectional type of tests is focused on the vehicle at the time it loses contact with the test article, and the subsequent part of the trajectory is not evaluated." The exit angles for both tests did not exceed the recommended upper limit of 60 percent of the impact angle (*4*).

The vehicle speed change was less than the 24.2-km/hr (15-mph) limit for both tests. These low changes in vehicle speed correspond to the relatively low values of longitudinal vehicle acceleration.

Regardless of speed change and exit angles, the barrier demonstrated its ability to retain a vehicle under very severe impact conditions. There was no tendency to pocket or snag the vehicle.

The vehicle post impact trajectories followed the same patterns in both tests. The vehicles were redirected toward the line of the barrier. Following the barrier impact, both vehicles rebounded from the barrier in a disabled condition and traveled 36 to 41.2 m (118 to 135 ft) before coming to a stop. The final positions were across the line of the barrier. If the barrier had extended further downstream, the vehicle would have impacted it a second time in both tests. The differences in vehicle trajectory may be attributed to variations in the timing of brake application and vehicle characteristics, such as weight distribution, suspension system, tires, vehicle stability after impact, and vehicle damage.

For both tests, the post-impact trajectory was as expected for a longitudinal concrete median barrier with or without concrete glare screen. *NCHRP Report 230* (*4*) points out that "the after collision trajectory may be one of the least repeatable performance factors" and that there is no assurance that existing hardware or certain classes of appurtenances will perform within NCHRP 230 limits for exit angle and speed.

In summary, the CMB/CGS met the vehicle trajectory requirements of NCHRP 230 (*4*).

CONCLUSION

- The CMB, as tested previously, was structurally adequate to meet present standards as specified in *NCHRP Report 230* (*4*) including both the strength and stability requirements. The CGS on the test barrier did not diminish the structural adequacy of the CMB.
- The CGS retrofitted on top of the CMB had the structural integrity to fully contain an impact of a 2449-kg (5,400-lb) pickup truck at 88 km/hr (55 mph) and 20 degrees with no evidence of structural distress or debris generation (including debris from vehicle).
- Large passenger vehicles can be redirected smoothly by the barrier with satisfactory occupant risk factors, according to *NCHRP Report 230* (*4*).
- In both tests, the exit speeds and angles of the vehicles met NCHRP 230 (*4*) requirements. The vehicle post-impact trajectory resulted in a smooth redirection of the vehicle back toward the concrete barrier.
- The two impact tests showed that the slipform construction of concrete glare screen to a total height of 0.51 m (20 in.) above a CMB is feasible and that the completed product has considerable structural strength. There is a need for tight control of the slump of the concrete mix with existing construction equipment so that the concrete holds its shape during the slipforming operation. The CMB/CGS should provide long life and low maintenance usually associated with structural concrete.
- Some safety enhancements may result from the additional height and strength of the CGS. For example, depending on the speed, weight, and angle, an errant vehicle might be somewhat less likely to climb over a CGS-equipped barrier.
- There was no penetration of test vehicle parts beyond the face of the CMB/CGS structure.

REFERENCES

1. *Standard Plans*. California Department of Transportation, Sacramento, CA, Jan. 1988.
2. *Traffic Manual*. California Department of Transportation, Sacramento, CA, Jan. 1987.
3. *NCHRP Synthesis of Highway Practice 66: Glare Screen Guidelines*. TRB, National Research Council, Washington, D.C., Dec. 1979.
4. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., March 1981.
5. Nordlin, E. F., et al. *Dynamic Tests of Prestressed Concrete Median Barrier Type 50, Series XXVI*. Report CA-HY-MR-6588-1-73-06. California Division of Highways, Sacramento, March 1973.
6. Parks, D. M., et al. *Vehicular Crash Tests of Unanchored Safety-Shaped Precast Concrete Median Barriers with Pinned End Connections*. California Department of Transportation, Sacramento, Aug. 1976.
7. Nordlin, E. F., et al. *Dynamic Test of a Slipformed Concrete Barrier Type 50 Placed over Lowered Existing Cable Barrier*, Report CA-DOT-TL-6696-1-74-36. California Department of Transportation, Sacramento, Dec. 1974.

Publication of this paper sponsored by Committee on Roadside Safety Features.

Small Sign Support Investigation

DWIGHT METCALF, YEE-JUNG HO, AND SYLVESTER A. KALEVELA

The Arizona Department of Transportation (ADOT), in an effort to comply with FHWA and NCHRP 230 guidelines, and in cooperation with the Texas Transportation Institute (TTI) developed a generic breakaway small sign support system that was implemented in the summer of 1991. The system consists of two 4.5 kg/m (3 lb/ft), 551 MPa (80 ksi) U-channel posts with a 102-mm (4-in.) ground-level splice, (sign post behind the base and separated by two hexagonal spacers) and two Grade 9 bolts 8-mm (5/16-in.) in diameter spaced at 760 mm (3 in.). Shortly after the first installations some single post sign supports reported failed due to winds of approximately 64 km/hr (40 mph). The investigation of the failures involved meetings with ADOT maintenance personnel, U-channel post suppliers, and TTI. Field evaluations of the generic U-channel system were performed. Material property tests were performed on the U-channel posts (purchased as part of the implementation of the generic small sign support). Static and dynamic testing of the U-channel small sign support system were performed as well. Field evaluations indicate that contractor-installed signs have used a variety of spacers. Material property tests indicate that at least some of the U-channels did not meet the 551-MPa (80-ksi) minimum yield point. Static testing indicates that the system should be able to withstand a static load of 160.9 km/hr (100 mph). Dynamic testing, developed specifically for this project by TTI, could not prove that the ADOT generic U-channel single-post sign support can perform satisfactorily under fluttering caused by high winds. Pilot studies, or sign test sites, have been installed in two ADOT maintenance yards and on one ADOT highway. These pilot studies will give field performance data upon which management can make a decision about what ADOT's small sign support of the future will be. In the interim ADOT either will use a minimum of two supports per sign or will specify that square steel tube be used for breakaway small sign supports.

In October 1984 the Arizona Department of Transportation (ADOT) began a research project with the Texas Transportation Institute (TTI) to determine which of the small sign support systems then used by ADOT met FHWA criteria for breakaway sign supports. The project scope was expanded several times and eventually included three phases: Phase 1—crash test program, Phase 2—development of a new small sign support, and Phase 3—benefit/cost analysis.

The results of Phase 1 were that several of the systems ADOT used were in compliance and several were not (*1*). During testing at TTI, FHWA made the recommendation that breakaway small sign supports should conform to the 1985 AASHTO publication *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*. In particular, breakaway small sign supports should have a base that extends no higher than 102 mm (4 in.) above ground level (*2*). The 102-mm stub height is called out in order to prevent the base post from rupturing fuel lines or penetrating the passenger compartment after the sign post has been hit by an errant vehicle. The 102-mm stub height requirement eliminated from consideration the previously used mid-height splice U-channel.

In response to the FHWA requirement ADOT embarked on the second phase of the TTI study: the development of a generic U-channel small sign support system with a ground-level lap splice. In addition to being able to withstand wind loads and meet federal safety standards for crash testing, some of the desired features of the generic small sign support included use of off-the-shelf hardware and easy installation procedures. Several combinations of U-channel unit weights and strengths were considered—4.5-kg/m (3-lb/ft) and 5.9-kg/m (4-lb/ft) U-channel posts, both at 413.4 MPa (60 ksi) and 551.2 MPa (80 ksi)—along with many possible splice configurations. Static and crash testing was performed as part of the generic small sign support development. Static testing was used to verify performance with respect to wind loading and crash testing was used to verify performance with respect to safety. In order to make the installation as easy as possible, a 76.5-mm (3-in.) bolt spacing was used. With a maximum 102-m (4-in.) stub height, the maximum bolt spacing possible (without excavation) is 76 mm.

Upon the conclusion of the second phase of the TTI study, ADOT chose the 4.5-kg/m, 551.2-MPa U-channel system, with a nested lap splice, and the post behind the base. Two Grade 9 bolts 8.0-mm (5/16 in.) in diameter spaced at 76 mm were selected to fasten the splice. A standard drawing and standard specification for the system were then developed. The U-channel small sign support installation drawing is shown in Figure 1.

PROBLEM DESCRIPTION

The U-channel small sign support standard specification was adopted and implemented in 1991. Failures of single support, small sign support installations occurred in large numbers soon after they were installed as part of construction projects. No reports of failures of installations with two or more supports were reported. Reports of single support sign post failures included the following:

- Within 3 months after installation, at a construction project near Phoenix, 32 of the 85 sign supports had failed due to winds that reportedly did not exceed 64.4 km/hr (40 mph);

D. Metcalf, Arizona Transportation Research Center, Arizona Department of Transportation. Current affiliation: Bentson Contracting Company, P.O. Box 37219, Phoenix, Ariz. 85069. Y.-J. Ho, Structures Section, Arizona Department of Transportation, Phoenix, Ariz. 85007. S. A. Kalevela, Arizona Transportation Research Center, Arizona Department of Transportation, 206 South 17th Avenue, Mail Drop 075R, Phoenix, Ariz. 85007.

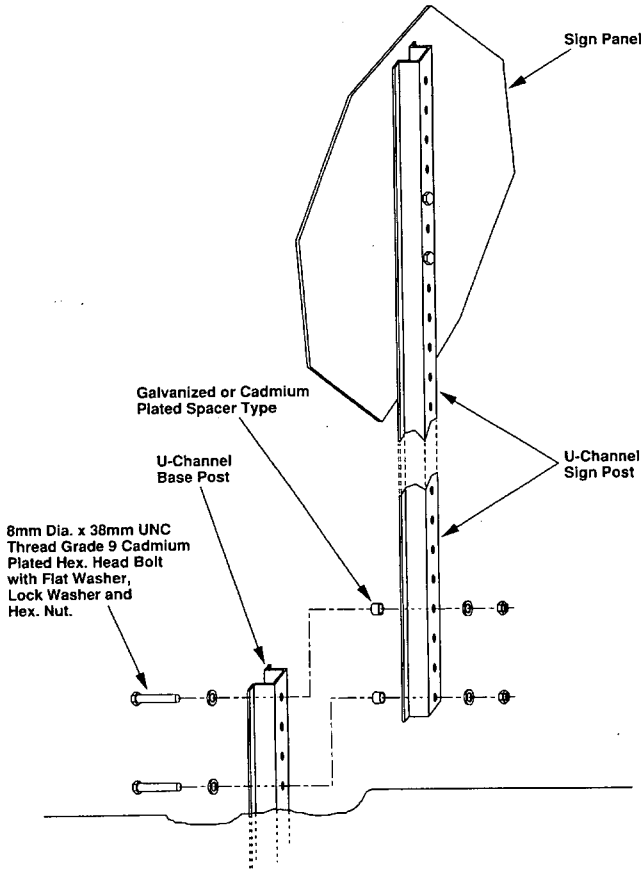


FIGURE 1 U-channel installation pictorial.

- In an airport road project, the U-channel small sign support system was installed without spacers and 40 percent of sign supports failed;
- Seven stop sign supports failed at a construction project south of Phoenix;
- All stop sign supports failed at a construction project in southern Arizona; and
- Some of the stop sign supports failed in a project near Tucson.

For the majority of failures, evidence indicated that the top bolt failed first followed by the U-channel post in the vicinity of the splice. Some posts ruptured with a zipper pattern along the back center line. A number of posts were torn diagonally as though they failed due to torsion stresses.

The couple formed by the two bolts is transferred to the posts through the bearing area of the spacers. Spacers used by some contractors (various diameters of galvanized water pipe and others) have had minimum bearing areas and have deformed easily allowing the connection to become loose. Once the connection is loose the bolts become subject to fatigue failure (*3*).

The U-channel selection chart, which is part of the ADOT U-channel small sign support standard drawing, is shown in Figure 2. The selection chart is used to determine support configurations for a given sign panel. The selection chart is based on a 96.5-km/hr (60-mph) wind speed. AASHTO specifications state that for roadside supports a 10-year mean recurrence interval should be used (*2*). A U.S. map indicating that for a 10-year mean recurrence interval the wind speed is 112.6 km/hr (70 mph) for most of Arizona is also given by

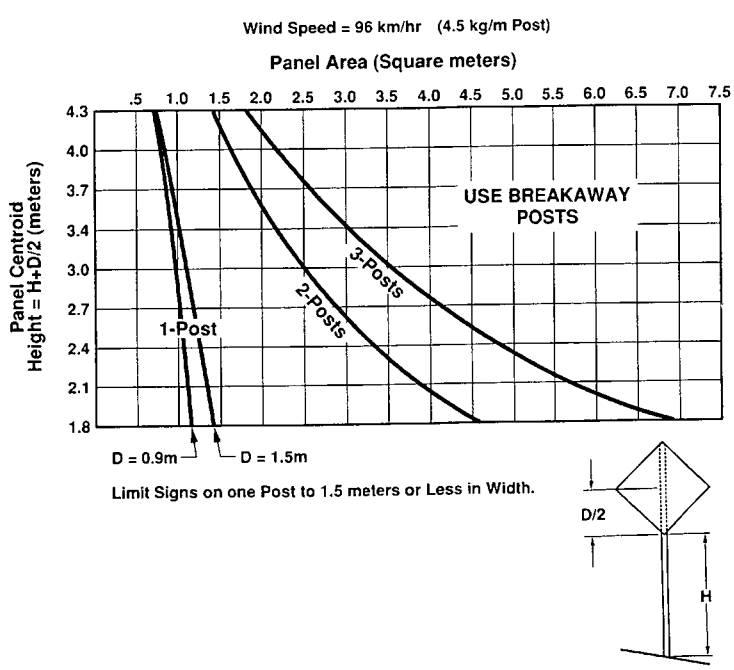


FIGURE 2 U-channel selection chart.

AASHTO. The ADOT selection chart was developed considering only the cross-section of the U-channel posts and not the structural capacity of the splice. The splice has proven to be the critical component of the U-channel small sign support system.

PROJECT APPROACH

The investigation began with a meeting with ADOT maintenance personnel and an assessment of the mode of the small sign support failures. The initial indication was that the U-channel sign post material was too brittle. Therefore, it was decided to conduct physical property tests. Tensile coupon testing was conducted along with chemical analysis of the U-channels that were purchased for use by ADOT maintenance personnel as part of the implementation of the new small sign support standard. These tests indicated that the yield point of some of the U-channel posts was slightly less than the required 551.2 MPa and that the chemistry of the steel was adequate.

The splice was the next target of the investigation. In the original TTI study, when the generic small sign support was developed, static bending, torsion, and combined bending and torsion tests were used to verify the wind load capacity of the system (*4*). Therefore, combined bending and torsion, and static testing was conducted to verify the results of the original study and to compare the effects of various spacers that had been used by contractors. The static testing was conducted in a factorial experiment that included tightness of the connection as a factor. Some of the test specimens with the hexagonal spacers had the bolts secured loosely in order to study the effect.

Structural analysis was performed to determine the wind speed equivalent of the static testing. Indications were that even the loose connection of the U-channel sign supports could withstand a static wind load of approximately 160.9 km/hr (100 mph). At this point, attention shifted to dynamic testing with the intention of simulating the flutter experienced by single sign supports in strong winds.

Vendor Participation

As part of the investigation U-channel post suppliers (Franklin Steel Company and Marion Steel Company) were contacted for their input. U-channel specimens were sent to Marion Steel and Franklin Steel. Franklin Steel representatives came to Arizona and gave a presentation to ADOT in October 1991. They believed that the 551.2-MPa steel was too brittle and that the lap splice was not sufficient to transfer torsion. A Marion Steel Representative came to Arizona in November of 1991 and discussed the problems with ADOT. The Marion Steel representative believed that improper installation was the major problem and offered to conduct training in each of the four ADOT districts.

Tensile Testing

The purpose of the tensile testing was to determine whether the yield point of the Marion Steel 4.5-kg/m (3-lb/ft) U-channel posts met the 551.2-MPa (80-ksi) minimum. Twelve Marion Steel post specimens were selected randomly and shipped to TTI for testing. Four Marion Steel post specimens taken from the TTI scrap yard were also tested for comparison purposes. Table 1 shows the test data.

Coupon tensile tests of the U-channel posts (rated at 551.2 MPa) used by TTI in the development of the generic small sign support system found a yield point of approximately 737.2 MPa (107 ksi) (*4*).

Cross-Section Variability

As stated, all U-channel small sign support failures that have occurred have been single-support installations. Single sign supports are subject to flutter and subsequently torsion. The contact between base post and sign post sides is required if torsion is to be carried by the U-channel posts.

A field evaluation was performed to determine whether contractor-installed sign supports had any contact between the base and sign post. Measurements were taken to determine the variability of in-service U-channel sign supports.

To determine whether the variability inherent in U-channel posts was such that the nesting concept was infeasible, a random sampling of U-channel posts were selected for measurements. Manufacturers of U-channel posts were contacted to determine mill tolerances.

Representatives of Franklin Steel stated that the U-channel posts are formed by rollers. These rollers are constantly wearing and may need to be changed even over the course of 1 day. After the steel is rolled at temperatures of approximately 1,038°C (1,900°F), it is placed on a notched bed for cooling at a temperature of between 206°C and 316°C (500°F and 600°F). Any differential cooling that takes place will induce variations in section dimensions. The U-channel posts are rolled when cool to straighten them. After the section is straightened it is sheared to length.

When asked to send a cross-section drawing that gave the tolerances for each dimension of the cross section, Franklin Steel stated: "it has been difficult to determine the range of variability you asked for, but I have shown on the drawing the internal tolerances we use on overall width, flange to flange. The other dimensions should be considered nominal and will vary slightly from rolling to rolling. The weight per foot of the section is the normal control element from a rolling standpoint and that tolerance for Franklin is plus or minus five percent." The tolerance on the flange-to-flange width mentioned is plus or minus 3 percent. Franklin Steel also stated that cross-section tolerances are not small enough to allow side-to-side contact in all cases. Therefore, it is not reasonable to expect torsion to be carried by side-to-side contact. Information supplied by Marion Steel indicates that their rolling process is similar. The cross-section variability indicated is approximately the same.

Chemical Analysis

A laboratory chemical analysis was conducted for the purpose of investigating chemical composition of the U-channel posts, and to see whether the chemical composition of these posts

TABLE 1 U-Channel Coupon Tensile Test Data (TTI)

Specimen Number	Width (mm)	Thickness (mm)	Load (kN)	Maximum Stress (MPa)
M1	12.672	3.510	40.72	914.99
M2	12.713	3.597	41.30	901.83
M3	12.685	3.708	42.14	895.01
M4	12.680	3.620	41.30	899.28
M5	12.738	3.579	41.25	903.42
M6	12.774	3.660	42.28	902.80
M7	12.700	3.647	42.14	908.72
M8	12.700	3.592	41.21	902.45
M9	12.771	3.617	41.61	899.77
M10	12.692	3.680	41.79	893.63
M11	12.703	3.683	41.30	878.27
M12	12.728	3.581	41.12	900.45
M13	12.664	4.318	45.88	837.69
M14	12.659	4.082	43.30	836.93
M15	12.700	4.293	50.02	916.51
M16	12.713	4.067	48.15	930.70

NOTES:
1) Test Procedure: ASTM 370
2) Specimens M1 through M12 are Marion 4.5 kg/m post specimens and were galvanized.
3) Specimens M13 through M16 were taken from the TTI scrap yard and were painted.
4) M13 and M14 are Marion 4.5 kg/m post specimens.
5) M15 and M16 are Marion 5.95 kg/m post specimens.
6) Thickness includes galvanizing or paint.

met the requirements stated in Article 607-2.04 of the 1990 ADOT Standard Specifications for Road and Bridge Construction (5). Eleven post specimens were analyzed. The following table presents the ADOT chemical composition requirements:

Element	*Composition (%)*
Carbon	0.67–0.82
Manganese	0.70–1.10
Phosphorus (maximum)	0.04
Sulfur (maximum)	0.05
Silicon	0.10–0.25

Table 2 gives the chemical analysis results. Except for the silicon content in Specimen 10, which is slightly above the maximum amount allowed, all the specimens met the chemical composition requirements.

Static Testing (Bending and Torsion)

The static testing was performed with the sign support in the horizontal position, by placing the base post in a large clamp so that the bottom of the splice was 76 mm (3 in.) from the clamp. The load was applied 2.7 m (9 ft) from the clamp with a 183-mm (7.2-in.) eccentricity. The testing was conducted with the U-channel flanges facing down. Figure 3 shows a schematic of the U-channel clamping and loading configuration for static testing. The load was applied by a two-speed motor (attached to an overhead I-beam) at a rate of 10.3 mm/sec (0.404 in./sec). A load cell was placed between the post and the motor for continuous monitoring of load. Loading was continued until failure.

All bolts used for the testing were Grade 9, 7.9-mm (5/16-in.) in diameter, and spaced at 76 mm (3 in.). Twenty-eight sign support specimens were tested from six spacer groups:

1. Steel bar stock spacer, 13-mm (½-in.) thick × 19-mm (¾ in.) wide × 127-mm (5-in.) long,
2. Hexagonal threaded spacers with bolts tightly fastened,
3. Hexagonal threaded spacers with bolts loosely fastened,
4. Spacers cut from a galvanized water pipe with a 13-mm (½-in.) inside diameter,
5. Spacers cut from a galvanized water pipe with a 19-mm (¾-in.) inside diameter, and
6. Spacers 19 mm in diameter and 13 mm thick (as used in the original TTI testing).

All sign support specimens failed by tensile failure of the top bolt. Table 3 shows the test results. Structural calculations indicate that the minimum load at failure of 1,469 N (330 lb) was equivalent to a "static" wind load of approximately 160.9 km/hr (100 mph) speed on a sign 762 × 762 mm (30 × 30 in.). Therefore, if the wind speed were 112.6 km/hr (70 mph), the factor of safety of the sign support against wind load would be about two. Also, maximum bending stress in the base post was larger than the specified yield stress of 551.2 MPa (80 ksi), which confirms the statement from TTI in their report

TABLE 2 Results of Chemical Analysis of U-Channel Posts

Specimen Number	ELEMENTAL COMPOSITION								
	C	Mn	P	S	Si	Cr	Ni	Mo	Cu
1	0.68	0.84	0.012	0.023	0.16	0.03	0.01	0.01	0.05
2	0.73	0.84	0.023	0.034	0.21	0.03	0.01	0.01	0.01
3	0.78	0.98	0.025	0.014	0.23	0.18	0.02	0.01	0.01
4	0.76	1.02	0.014	0.046	0.25	0.13	0.16	0.03	0.47
5	0.75	0.94	0.012	0.040	0.21	0.09	0.11	0.02	0.34
6	0.75	0.94	0.012	0.040	0.21	0.09	0.11	0.02	0.34
7	0.75	0.91	0.011	0.041	0.21	0.09	0.11	0.02	0.33
8	0.67	0.95	0.014	0.042	0.23	0.10	0.12	0.02	0.37
9	0.71	0.92	0.012	0.040	0.22	0.09	0.11	0.02	0.34
10	0.70	0.86	0.013	0.029	0.26	0.09	0.17	0.02	0.39
11	0.76	0.91	0.022	0.045	0.23	0.12	0.12	0.02	0.42

NOTES: 1) Specimen Numbers 1 through 3 are Franklin Post specimens.
2) Specimen Numbers 4 through 11 are Franklin Post specimens.

on Phase 2 of the original research, that for Marion Steel 4.5-kg/m (3-lb/ft) posts with Grade 9 bolts spaced at 76 mm (3 in.), the splice will develop the nominal yield stress of the posts.

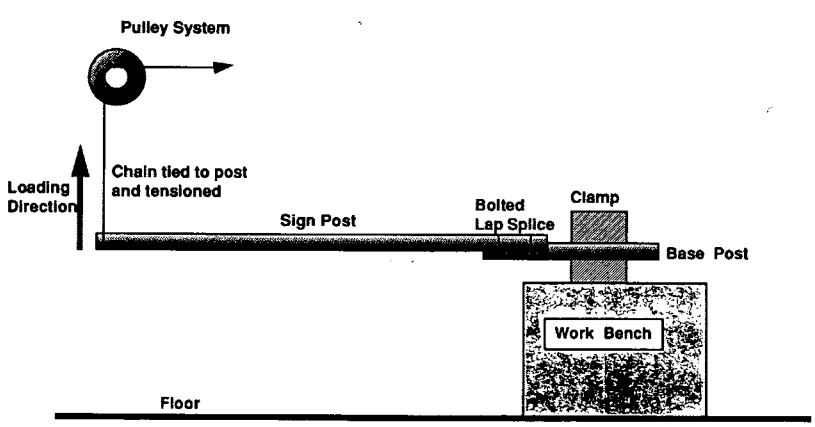
Dynamic Testing

Dynamic testing was conducted to compare the endurance limits of several types of lap splices. Torsional vibration plus bending was induced by a shaker. The shaker consisted of a single-speed motor that drove two counter-rotating masses, one on each side of the motor, 180 degrees out of phase. Each sign support specimen for dynamic testing was installed as shown in Figure 4. The specimen was laid horizontally, with the base post clamped near the splice and the shaker attached to the sign post, 2.7 m (9 ft) away from the clamp that secured the base post. The shaker weighed approximately 445 N (100 lb) and was oscillated at 4 cycles a second.

The level of the dynamic load acting on the post was adjusted by changing the weight of the masses. Three levels of dynamic load were applied. The dynamic testing was intended to simulate wind induced flutter and therefore could not be correlated to wind speed.

Thirty-five sign post specimens were tested from five groups:

1. Marion Steel posts (from ADOT's supply) with two hexagonal threaded spacers and two Grade 9 bolts 7.9 mm (5/16 in.) in diameter, at 76-mm (3-in.) spacing;
2. Marion Steel posts (from ADOT's supply) with one steel bar spacer 13 mm (1/2 in.) thick, 19 mm (3/4 in.) wide, 152 mm (6 in.) long and two Grade 8 bolts 7.9 mm (5/16 in.) in diameter, at 102-mm (4-in.) spacing;



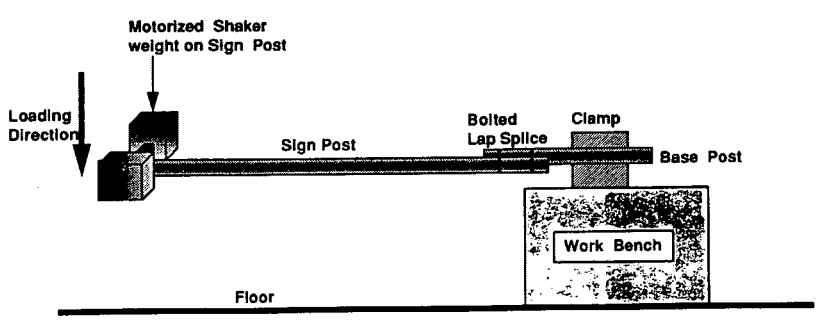
Notes: In this sketch the U-Channel is facing down

FIGURE 3 Schematic of U-channel post assembly and load application for static testing.

TABLE 3 Static Test Results

Post Group	Number of Specimens	Load at Failure (N)	Average Load at Failure (N)	Deflection at Loaded End (mm)	Avg. Deflection at Loaded End (mm)
1	8	1900.15	1851.20	85.34	72.90
		1659.85		60.96	
		2024.75		77.72	
		1726.60		67.56	
		2011.40		73.15	
		1811.15		69.60	
		1766.65		73.66	
		1909.05		75.18	
2	4	1650.95	1610.90	74.68	71.37
		1659.85		72.14	
		1504.10		65.53	
		1624.25		72.64	
3	4	1504.10	1530.80	73.15	73.41
		1637.60		75.69	
		1495.20		72.14	
		1490.75		72.64	
4	4	1593.10	1575.30	73.15	71.88
		1468.50		68.58	
		1615.35		71.12	
		1615.35		74.17	
5	4	1642.05	1579.75	69.60	
		1570.85		59.94	
		1477.40		67.56	
		1624.25		70.10	
6	4	1557.50	1619.80	64.10	66.29
		1628.70		67.56	
		1619.80		67.06	
		1673.20		66.55	

NOTES: 1. 13 mm (½ in.) thick x 19 mm (¾ in.) wide x 127 mm (5 in.) steel bar stock spacer.
2. hexagonal threaded spacers with bolts tightly fastened.
3. hexagonal threaded spacers with bolts loosely fastened.
4. spacers cut from 13 mm (½ inch) inside diameter galvanized water pipe.
5. spacers cut from 19 mm (¾ inch) inside diameter galvanized water pipe.
6. 19 mm diameter x 13 mm thick spacers (as used in the original TTI testing).



Notes: This sketch shows the case for U-Channel facing up

FIGURE 4 Schematic of U-channel post assembly and load application for dynamic testing.

3. Marion Steel posts (from ADOT's supply) with one steel bar spacer 13 mm (½ in.) thick, 19 mm (¾ in.) wide and 152 mm (6 in.) long and two Grade 8 bolts 9.5 mm (⅜ in.) in diameter, at 102-mm (4-in.) spacing;
4. Franklin EZE-ERECT; and
5. Marion Steel post with no splice.

Sign post specimen groups 4 and 5 were included to establish a baseline performance. Group 4, the Franklin EZE-ERECT, is a proprietary lap-splice system that has a 15-year service record. Group 5 was a single test with a continuous Marion Steel 4.5-kg/m (3-lb/ft) post.

The ratio of load cycles (number of revolutions of the sprocket that drives the rotating masses) to observed response cycles (number of oscillations of the sign support) at a given time varies with the level of load. Approximate observed ratios were 1:1 for the low level; 3.87:1 for the medium load level; and 4.32:1 for the high load level. Table 4 presents the results of the dynamic testing.

Pilot Studies

Pilot studies, or field installations, of various U-channel lap splices have been conducted in Arizona in an effort to gain field performance data. To date, two pilot studies have been installed in ADOT maintenance yards and one on an ADOT highway.

The pilot studies consist of a set of signs with four or five lap-splice configurations (including the Franklin EZE-ERECT) at three load levels. To obtain the load levels, the height to the centroid of the sign is held constant at 2.6 m (8.5 ft) and the sign panel size is varied.

Each of the maintenance yard pilot studies has been instrumented with a wind gauge. The maximum peak gust information is captured and stored by the gauges. The wind speed information is gathered periodically and incorporated into the test results data base for the small sign support investigation.

To date, one of the maintenance yard pilot studies, with signs facing west, has had a maximum peak gust speed of 66 km/hr (41 mph) east-southeast. The other maintenance yard pilot study, with signs facing northeast, has had a maximum peak gust speed of 120.7 km/hr (75 mph) south-southeast. All sign supports are still standing at both maintenance yards.

The highway pilot study originally consisted of 10 stop signs. Five had the hexagonal spacer splice and five had the bar spacer splice. Two of the signs had to be removed. None of these signs has failed.

TABLE 4 Dynamic Test Results

Dynamic Load Level	Post Group[a]	Channel Facing Down/up	N[b]	Range of Load Cycles to Failure	Avg. Number of Load Cycles to Failure	Avg. Number of Response Cycles to Failure
High	1	Down	4	4,880 - 10,280	7,280	1,685
High	1	Up	3	2,640 - 7,240	5,170	1,197
High	2	Down	3	5,560 - 10,360	17,050	3,947
High	2	Up	3	3,360 - 9,640	6,510	1,507
High	3	Up	3	11,040 - 13,920	12,240	2,833
High	4	Down	3	10,680 - 16,560	13,960	3,231
High	4	Up	3	20,320 - 71,640	38,560	8,926
High	5	Down	1	Test was halted at 187,480	Test was halted before Failure	Only one test was conducted
Medium	3	Up	3	22,800 - 40,320	31,920	8,248
Medium	4	Down	3	29,760 - 111,360	79,360	20,506
Low	3	Up	3	13,440 - 44,160	25,920	25,920
Low	4	Down	3	42,000 - 191,520	92,240	92,240

NOTES: (a)1. Marion Steel posts (from ADOT's supply) with two hexagonal threaded spacers and two 7.9 mm (5/16 inch) diameter, Grade 9 bolts at 76 mm (3 inch) spacing.
2. Marion Steel posts (from ADOT's supply) with one 13 mm (½ inch) thick x 19 mm (¾ inch) wide x 152 mm (6 inch) steel bar spacer and two 7.9 mm (5/16 inch) diameter, Grade 8 bolts at 102 mm (4 inch) spacing.
3. Marion Steel posts (from ADOT's supply) with one 13 mm thick x 19 mm wide x 152 mm steel bar spacer and two 9.5 mm (⅜ inch) diameter, Grade 8 bolts at 102 mm spacing.
4. Franklin EZE-ERECT.
5. Post with no splice (Marion Steel Post).
(b) N = Number of Specimens

STRUCTURAL ANALYSIS

Effect of Cross Sections

Two cross-section drawings have been received by ADOT from Marion Steel, one dated January 20, 1983, and one dated November 30, 1983. A structural analysis was performed using the January 20, 1983, cross section. A 0.9-m × 0.9-m (3-ft × 3-ft) diamond-shaped sign mounted on a single post, with a 3.5-m (11.5-ft) height to the center of the sign, was used for the analysis since similar signs have been reported failed due to wind. Also signs of this size and height are on the acceptable limit of the current selection chart.

The sign support was analyzed for both 72.4 km/hr (45 mph) and 96.5 km/hr (60 mph) wind speeds (see Table 5). The analysis was performed according to the methodology given by AASHTO (*2*). AASHTO calls for a 40 percent increase in allowable bending stress for wind loading. The 40 percent increase was applied to all bending stress terms in the combined stress ratio (CSR) equation except for the axial compressive stress (F_a), which appears in the denominator of the second term of the equation. The 40 percent increase was not applied to this term in the interest of being conservative.

The analysis considered only the cross-sectional properties and yield point of the U-channel post steel. A yield strength of 551.2 MPa (80 ksi) was used. When subjected to 96.6 km/hr (60 mph) wind, the post is over stressed. However, the post is within the allowable range in the ADOT U-channel selection chart (Figure 2). This is because the U-channel selection chart was developed using the cross section of the U-channel posts supplied to TTI for the development of the generic small sign support.

In January 1992 ADOT received another 4.5-kg/m (3-lb/ft) post cross-section drawing, dated November 30, 1983, from Marion Steel Company. Marion Steel stated that the November 30, 1983, cross section is its final pass design and the January 20, 1983, cross section was sent to ADOT in error. The November 30, 1983, section is slightly weaker than the January 20, 1983, section and has a section modulus that is 23 percent smaller than the posts used in the development of the ADOT U-channel selection chart.

It is very important that in the development of the U-channel selection chart, the weakest possible section from all potential suppliers be used for structural calculations.

Bolt Stresses

An analysis was performed to determine the loads on the bolts at the splice. The splice strength is related to the bolts. The maximum loads on the bolts at the splice, using different wind speeds, are given here:

	Wind Speed (km/hr)	
	72.40	*96.50*
Maximum load on bolt (KN)	18.50	32.90
Factor of safety (based on 33.38-KN proof load)	1.80	1.01

It can be seen that for a 96.5-km/hr (60-mph) wind, the bolts do not have an adequate factor of safety.

Effect of Size and Spacing of Spacers on Stresses in Posts

A structural analysis was performed to check the stresses in the back of the post at the splice using the January 20, 1983, Marion Steel cross section. A wind speed of 72.4 km/hr (45 mph) was used for the wind load. The analysis was performed for spacers 19 mm (¾ in.) in diameter, 16 mm (⅝ in.) in diameter, and 13 mm (½ in.) in diameter, with bolt spacings of 51 mm (2 in.), 76 mm (3 in.), and 102 mm (4 in.). The back of the post was treated as a structural member spanning the sides. Table 6 shows the results of the analysis.

Some of the theoretical stresses shown in Table 6 are not attainable because they are far beyond the failing stress of the steel. The actual stresses would generally be lower because of the effect of local yielding and stress redistribution. How-

TABLE 5 Results of Structural Analysis on U-Channel Posts

	Wind speed (km/hr)	
	72.40	96.50
Axial compressive stress f_a (kPa)	558.10	558.10
Allowable axial compressive stress F_a (kPa)	8336.90	8336.90
Bending stress f_b (MPa)	228.67	405.61
Allowable bending stress F_b (MPa)	463.00	463.00
Shear stress from shear and torsion f_v (MPa)	127.53	226.48
Allowable shear stress F_v (MPa)	254.65	254.65
Combined stress ratio (CSR) $= \frac{f_a}{F_a} + \frac{f_b}{(1-f_a/F_a)F_b} + \frac{f_v^2}{F_v^2}$ (should be < or = 1)	0.863	1.824

TABLE 6 Maximum Flexural Stresses in Back of Post at Splice

	Maximum flexural stress (MPa) in back of post		
Bolt Spacing (mm)	19 mm diameter spacer	16 mm diameter spacer	13 mm diameter spacer
5.1	1149.3	1485.5	1816.2
7.6	685.6	877.1	1065.9
10.2	556.0	705.5	853.7

ever, it can be seen that the stress decreases as the size of spacer and bolt spacing increase. The optimum diameter of the spacer is 19 mm (¾ in.).

CONCLUSIONS

ADOT developed a generic U-channel small sign support system with a breakaway ground-level lap splice. Shortly after implementation of the new system, failures of single-support sign installations occurred. The failures have been associated with the splice.

The investigation into the failures began by ensuring that the U-channel steel met specifications. The chemistry of the steel met specifications but the yield point of some of the specimens tested was slightly less than the required 551.2 MPa (80 ksi). On reviewing the original TTI work, it was determined that the U-channel posts that they used were rated 551.2 MPa (80 ksi) but that the as-tested strength was 737.2 MPa (107 ksi). However, the U-channel posts purchased by ADOT (from the same vendor who supplied the original U-channel posts) that were rated at 551.2 MPa (80 ksi) were actually much closer to 551.2 MPa.

The combined bending and torsion static testing was conducted next. The purposes of this testing were to verify the original work done by TTI and to determine whether various spacers used by contractors had caused the failures. The results of the static testing indicated that all the spacers were sufficient from a static loading standpoint.

A dynamic test conducted at the TTI laboratory was used to induce flexural and cyclic torsional stresses on the sign support system. The test indicated that the proprietary Franklin EZE-ERECT performed better than any of the other systems tested. This was presumably due to the four bolts and special strap used in that system. Although the dynamic test was meant to simulate sign flutter caused by wind, the simulated flutter could not be correlated with real system flutter because there were no wind data for the analysis.

All failures during the testing occurred at one of the two splice bolts. The failure of only single sign support installations in the field led to a study of the effects of torsion. For torsion to be carried efficiently, it is necessary for the sides of the U-channels to be in constant contact. If they are not, the bolts will have to carry all the torsion. The results of variability studies of the U-channel post cross sections indicated that side-to-side contact is not a reasonable expectation. The result of this work, for ADOT, is that sign supports previously scheduled (designed) for a single post will now be installed with two posts. ADOT will explore other options for small sign support in the future.

REFERENCES

1. Ross, H. E. Jr., D. L. Sicking, W. Campise, and R. A. Zimmer. *Small Sign Support Analysis: Phase 1 Crash Test Program*. Report FHWA-AZ88-202-IV. Arizona Department of Transportation, Phoenix, Aug. 1988.
2. *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*. AASHTO, Washington, D.C., 1985.
3. Shigley, J. E., and C. R. Mischke. *Mechanical Engineering Design*. McGraw-Hill, New York, 1989.
4. Morgan, J. R., H. E. Ross, Jr., R. E. Schuler, and W. L. Campise. *Small Sign Support Analysis: Phase II Static, Pendulum, and Full-Scale Crash Test Program Volume I (Report)*. Report FHWA-AZ88-202-II. Arizona Department of Transportation, Phoenix, Aug. 1984.
5. *Standard Specifications for Road and Bridge Construction*. Highways Division, Arizona Department of Transportation, Phoenix, Aug. 1990.

Publication of this paper sponsored by Committee on Roadside Safety Features.

Analysis of Guardrail-End Accidents in Oklahoma

J. L. GATTIS, JOHN P. VARGHESE, AND LARRY E. TOOTHAKER

Researchers at the University of Oklahoma documented attributes associated with accidents in which vehicles struck guardrail ends. The data base included accidents at a variety of guardrail-end types, but most ends were either exposed or turned down. The severity of exposed and of turned-down guardrail-end accidents in relation to lateral location of the guardrail, and to vehicle rolling and vaulting, was investigated. Individual accident reports were read carefully to obtain information for the analyses. The results showed that on divided roads, vehicles struck median guardrail ends about as often as right-side ends. On undivided roadways, right-side ends were struck about 60 percent of the time. Approximately a sixth of the accidents were fatal or incapacitating-injury accidents. In most of them, the vehicle did not vault or roll. The research did indicate that turned-down guardrail ends were associated with more vehicle rolling and vaulting than the exposed ends. Roughly a third of all guardrail end accidents involved an inattentive driver striking a guardrail end. Most guardrail-end accidents on the state system occurred on a small portion of the system, namely the higher-volume roadways. The researchers suggested that accident reporting methods be enhanced, and that rumble strips be tested as a means to reduce guardrail end strike accidents. If newer, more expensive end treatments were installed, concentrating efforts on a small portion of the system could address a majority of the end accident sites.

Exposed ends and turned-down ends are the two predominant end types currently used on Oklahoma highway guardrails. Researchers studied reports of guardrail-end accidents which occurred on highways maintained by the Oklahoma Department of Transportation (ODOT) from 1988 through 1991 to evaluate the performance of the guardrail-end treatments when struck by vehicles.

BACKGROUND

Accumulated experience and technological changes have led to changed perspectives about roadside safety. Years ago, observers noted that errant vehicles were sometimes impaled on the commonly used exposed guardrail ends. Researchers developed the turned-down-twist guardrail end as a remedy; however, these designs caused some vehicles striking the end to go out of control. In response, designers modified the ends in hopes that vehicles would "ride down the rail" and not roll or vault.

J. L. Gattis, Oklahoma Transportation and Infrastructure Center, University of Oklahoma, 202 West Boyd, Room 334, Norman, Okla. 73109. J. P. Varghese, School of Civil Engineering and Environmental Science, University of Oklahoma, 202 West Boyd, Room 334, Norman, Okla. 73109. L. E. Toothaker, Department of Psychology, University of Oklahoma, 455 West Lindsey, Room 705, Norman, Okla. 73109.

Now, changes in the composite vehicle fleet have resulted in a higher proportion of smaller, lightweight vehicles being on the road. Because some vehicles, especially lightweight vehicles, have flipped when they ride up on a turned-down guardrail end, and because newer end types with better reported performance are on the market, there is a move to replace the turned-down ends with newer guardrail-end designs. A recent FHWA memorandum contained the following statements:

- Turned-down terminals should not be used on new installations of guardrails for freeway, expressway, or other high-speed, high-volume facilities.
- Safety improvement projects, hazard elimination projects, or 3R/4R projects on high-speed, high-volume facilities should require replacement of turned-down end terminals with approved terminals.
- Use of turned-down terminals on low-speed or any low-volume facility may be allowed based on reasonable risk management considerations.

FHWA has asked states to act on this policy.

Thousands of turned-down ends were installed and are still in use. State transportation departments are understandably reluctant to incur the cost of replacing existing turned-down ends unless they can be assured that the existing ends are in fact causing problems. Some states have shown interest in investigating the performance of their present guardrail end designs. Two related publications have recently been prepared by the Texas Transportation Institute (*1,2*).

RESEARCH OBJECTIVES

ODOT wanted to evaluate recent guardrail-end accidents on state highways. An initial study objective was to determine whether, and to what degree, the turned-down guardrail ends used on state highways were associated with

- Vehicle overturning,
- Vehicle vaulting, or
- Accidental death and injury.

The original plans called for a study of accidents occurring between June 1, 1987, and May 31, 1990. It was assumed that not all police would describe a certain type of accident with the same terms, and guardrail-end accidents possibly could be found in more than one "Object Struck First" category. Therefore, the initial study pool was to have included acci-

dents in the "Object Struck First" categories of barrier, bridge rail, and guard post, as well as guardrail. However, ODOT decided that the study would consider only the "guardrail" category, but added an extra year of data to cover accidents between 1988 and 1991 on ODOT Interstate (excluding turnpikes), U.S., and State highways. The study utilized a state accident data base, accident reports, video highway logs, traffic volume maps, as well as other reference materials.

Computerized Accident Data Base

When police investigate an accident, they fill out an "Official Police Traffic Collision Report." Information from these reports is encoded into a state accident data base. ODOT furnished a computer file containing details of guardrail accidents.

Accident Reports

Reports were retrieved for accidents that had been encoded as "guardrail." The research team gleaned information from these reports.

Videotapes

ODOT had more videotapes of Oklahoma highways during that general period, from 1988 through 1991, when the accidents under review occurred. It was assumed that conditions at the accident sites had not changed between the time the video was taken and the time the accidents occurred. Although the videotapes permitted the viewing of many roadway features without visiting the site, tape viewing did not always allow the researchers to find needed information. Viewing impediments included roadside vegetation at the accident location, or an unclear video. At a few accident locations, the guardrail was not in the field of vision in the videotape; guardrails located in the median on sharp horizontal curves were especially susceptible to this problem. A few telephone calls and field visits were needed to gather some details.

Average Daily Traffic Volumes

Accident sites were spotted on ODOT average daily traffic (ADT) maps. The ADT nearest to the accident site was taken as the volume for that accident location.

BUILDING AND ANALYZING THE DATA BASE

The overall study goal was to define certain characteristics of guardrail-end accidents. A guardrail accident data base was created and analysis was performed so the researchers could gain insight into certain issues.

Location and Direction

The initial data base included all accidents which had been encoded with "guardrail" being the first object struck. The researchers had to separate the end hits from others, and they exercised judgment as to whether each accident involved an actual guardrail-end strike. If the accident report led the researchers to presume that the vehicle struck the guardrail end, then the accident was coded as a "presumed end hit." If the report indicated that the impact was possibly but not likely near the end, then the accident was coded as a "questionable end hit."

The accidents were coded initially with respect to the direction in which the vehicle was travelling. If a vehicle crossed over into the oncoming side and hit the oncoming trailing end, it was considered head-on from the perspective of the vehicle. Later, sorting routines were used to identify vehicles that crossed over the median or centerline and hit a trailing end head-on.

When the accidents were grouped later, the terms "approach end" and "trailing end" were used with respect to the normal or intended direction of travel in a lane or lanes. The "approach end" is the guardrail end initially encountered at the beginning on the right side of an undivided road; on a divided road, it is the end on the right or left of the lanes intended for one direction of travel. A "trailing end" is the one last encountered at the end of a guardrail installation. When a driver crossed the centerline or the median, the vehicle was said to have struck the trailing end. Accidents were categorized as follows:

- End hits—all guardrail-end accidents, and
- Approach end/same side and trailing end/cross over/undivided accidents—only those in which a vehicle hit the approach end, or crossed over the centerline of an undivided roadway and struck the trailing end on the driver's left side, excluding ends struck from behind.

ODOT reported that, on undivided roads, trailing ends and approach ends are the same type. On divided roads, the trailing end may not be the standard turned-down terminal. To reflect the possible levels of uncertainty about what was actually struck, the data sets eventually created were as follows:

- End hits—presumed (P);
- End hits—presumed plus questionable (P + Q);
- Approach end/same side and trailing end/cross over/undivided—presumed;
- Approach end/same side and trailing end/cross over/undivided—presumed plus questionable;

If the vehicle hit more than one guardrail end, then each end hit was treated as a separate accident by entering it twice. To keep the project scope under control, some analyses were not performed on all data sets.

Some accidents were not classified due to the absence of a collision diagram, or a duplicate accident report. If the vehicle struck the connecting point of the guardrail and bridge parapet wall, then the accident was placed under the "connection with fixed object" category. If the vehicle had struck a concrete bridge barrier or concrete guardrail, then the accident was placed under the "not a guardrail accident" category. A few vehicles approached the guardrail from behind and struck the end; these were placed in the "end hit from behind" category.

The researchers also coded the guardrail lateral location with respect to the direction in which the vehicle was traveling. Codes were given for accidents on ramps, frontage roads, and cross streets.

Type of End Struck

The researchers assigned codes to reflect the end type which had been hit. Few accident reports furnished this information, so usually the researchers obtained this from videotapes. On the videos, it was not possible to differentiate breakaway cable terminals (BCTs) with rounded ends from "normal" rounded exposed ends; both were classified as rounded ends. Some BCTs may have been categorized as "exposed" ends. Although the size of any misclassification error is unknown, it is expected to be small.

The researchers used judgment to determine whether guardrail ends were flared. If the end appeared in the video to be significantly set back, they called it flared. In the subsequent analysis, flared ends were not analyzed separately because of their small number.

Vehicle Rolling and Vaulting

The researchers found many of the accident reports contained wording which did not indicate clearly whether the vehicle vaulted. After reviewing the police accident report, the researchers concluded that the vehicle had vaulted in conjunction with striking the guardrail end if the vehicle went airborne, went over the guardrail, or slid on top of the guardrail. If the language in the report was such that the researchers were not sure whether the vehicle vaulted, then the accident was classified as "not sure to have vaulted." If the vehicle did not do any of the preceding actions, then a "vehicle did not vault" classification was made.

If after hitting the guardrail end, the vehicle immediately turned on its side or top, then it was said to have rolled. Sometimes researchers categorized an accident as "rolling was not sure to have occurred," such as when an embankment was close to the end and the researchers could not determine from the report whether the guardrail end or the embankment caused the rolling. If no roll occurred, then the accident was classified as "did not roll."

Injury Accident Severity

The existing data base categorized accidents as fatal, injury, or property damage only (PDO). To further define the severity of injury accidents, the researchers added codes for the three injury severities. Injury A is incapacitating, Injury B is non-incapacitating, and Injury C is a complaint of injury.

Driver Alertness

The researcher added a code to the data base if the wording led them to conclude that driver inattention or drowsiness contributed to the accident. The code was not added when seizures or driving under the influence was mentioned, except in a few cases in which the accident report wording led the researchers to conclude that the driver was not severely impaired.

Data Analysis

After building the data base, the researchers performed a number of analyses. Some classification categories were combined to obtain a sufficient number of occurrences per cell.

The researchers found total numbers of guardrail-end accidents exhibiting various types of attributes, such as the number involving a median-end hit. A regression equation was derived to relate the percentage of accidents with the percentage of vehicle miles of travel.

The researchers investigated the effects of combinations of certain factors. Contingency tables were formulated from the "presumed" data sets to investigate the relationship of end type, severity, and rolling and vaulting. The following statistical tests were performed:

- The chi-square test of independence, to determine whether, for a number of data groups combined, the frequency of occurrence of an event (e.g., roll/vault) for a data group differed from that of another data group (e.g., exposed versus turned-down ends);
- The Games-Howell (GH) multiple comparison statistic on cell means, to determine whether the means of two groups of data were statistically different; and
- The binomial proportions test, to determine whether the proportions of two specific data groups were statistically different.

The chi-square and binomial tests are well known. The lesser known GH procedure is especially suited for data groups with unequal sample sizes. It uses the test statistic

$$t_{jk} = \left(\overline{Y}_j - \overline{Y}_k\right)/\sqrt{(s_j^2/n_j) + (s_k^2/n_k)} \quad (1)$$

where

$\overline{Y}$ = sample mean,
s^2 = unbiased sample variance, and
n = sample size for each pair of means,
$j = k\ (3)$.

Because α is chosen to be controlled for each comparison, the null hypothesis H_0 is rejected if

$$p\left(t_{\mathrm{df}_{jk}} \geq |t_{jk}|\right) \leq \alpha \quad (2)$$

Otherwise, one does not reject H_0. The degrees-of-freedom df_{jk} for the observed t_{jk} is

$$\mathrm{df}_{jk} = (s_j^2/n_j + s_k^2/n_k)^2/[(s_j^2/n_j)^2/(n_j - 1)] + [(s_k^2/n_k)^2/(n_k - 1)] \quad (3)$$

TABLE 1 Longitudinal Location of Guardrail Accidents

Classification	Number	Same Side	Cross-over	Percent
ALL GUARDRAIL ACCIDENTS				
Not able to determine, duplicate	23			1.33
Not guardrail end accident	1064			61.36
Questionable guardrail end accident	118			6.80
Presumed guardrail end accidents	435			25.09
Guardrail connection with fixed object	67			3.86
Not a guardrail accident (e.g.,concrete barriers)	27			1.56
Total	1734			100.0
ONLY GUARDRAIL END ACCIDENTS				
Questionable trailing end	27	17	10	4.88
Trailing guardrail end--undivided road			9	
Questionable approach end	91	81	10	16.46
QUESTIONABLE	118			
Trailing guardrail end	89	19	70	16.09
Trailing guardrail end--undivided road			62	
Approach guardrail end	336		5	0.90
Head end of vehicle--approach end		234		42.32
Side of vehicle--approach end		92		16.64
Rear of car--approach end		5		0.90
Approach or trailing guardrail end hit from behind	10	4	6	1.81
PRESUMED	435			
TOTAL PRESUMED-PLUS-QUESTIONABLE	553			100.0

RESULTS

Table 1 gives the summary of guardrail accidents. ODOT furnished a total of 1,731 guardrail accident reports. In three accidents, a vehicle struck two guardrail ends, so there were 1,734 entries in the file. Of these, the researchers did not classify 1.3 percent because of missing data or because the accident report was a duplicate.

Most of the guardrail-end accidents on Oklahoma highways were at either exposed ends or turned-down ends. There were a few accidents involving breakaway cable terminals, parabolic end sections, and double-faced turned-down ends. Oklahoma turned-down ends typically have an initial wooden post, and all posts have block-outs. The turned-down end design specifies 7.62 m (25 ft) between the embedded end and the first post, and 3.81 m (12.5 ft) between the first two posts. Figures 1 and 2 show examples of the two end types at which almost all of the guardrail-end accidents occurred.

Table 2 presents a summary of the guardrail-end accident lateral locations. The data showed that more guardrail-end accidents on the State highway system occurred on divided roads. On divided roads, the chances of a right-side end accident were almost the same as a median end-strike. On un-

FIGURE 1 Example of typical exposed guardrail end.

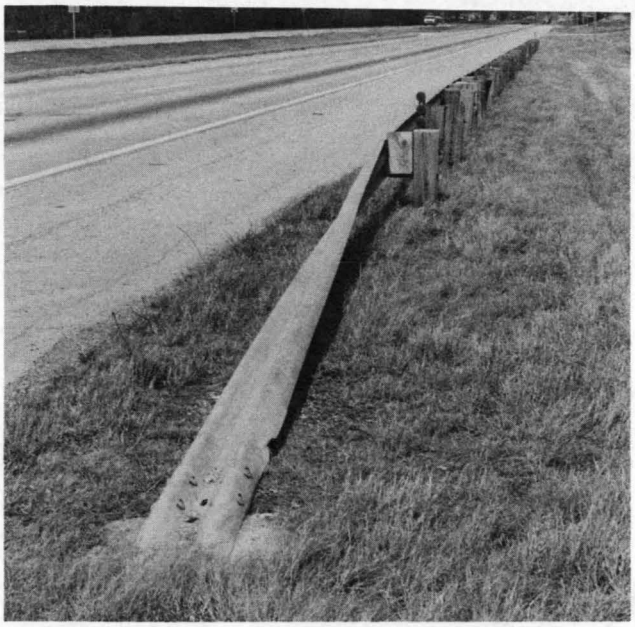


FIGURE 2 Example of typical turned-down guardrail end.

divided roads, 60 percent of accidents involved the vehicle striking the guardrail end on the right side.

Type of Guardrail End Struck

Table 3 presents the number of end strikes by end type. The researchers were unable to define a few of the end types, perhaps because the guardrail had been removed or because of an inadequate location description. The presumed and the presumed-plus-questionable data sets exhibited similar proportions of end-type categories.

Because the percent of accidents occurring on "other than exposed and turned-down ends" was very small, the researchers did not perform analyses on the "other" group. The reseachers merged the flared-end groups into the corresponding nonflared turned-down or exposed end categories.

Vehicle Rolling and Vaulting

The researchers examined vehicle rolling and vaulting trends in relation to the exposed or the turned-down guardrail-end types. The roll and vault characteristics were combined into three groups: "no roll/vault," "roll/vault," and "not sure." Table 4 gives the number and percentage of accidents in these categories. Roughly one-fourth to one-third of guardrail-end accidents appear to result in vehicle rolling and vaulting. In most of the guardrail-end accidents, the vehicle did not vault or roll.

Guardrail-End Accident Severity

The researchers found the number of fatal, Injury A, Injury B, Injury C, and PDO accidents separately for exposed and for turned-down end terminals. Table 5 gives the results and also relates end accident magnitude with that of two other categories.

TABLE 2 Lateral Location of Guardrail-End Accidents

	Left Side	Median Left	Median Middle/ Right	Right Side	Other[a]	Total
PRESUMED END HITS						
Roadway with median						
Frequency	2	16	105	119	5	247
Percentage	0.81	6.48	42.51	48.18	2.02	100.0
Roadway without median						
Frequency	65	na	na	111	9	185
Percentage	35.13	na	na	60.00	4.87	100.0
Not Sure Frequency						3
PRESUMED-PLUS-QUESTIONABLE END HITS						
Roadway with median						
Frequency	3	18	138	161	7	327
Percentage	0.92	5.50	42.20	49.24	2.14	100.0
Roadway without median						
Frequency	82	na	na	131	10	223
Percentage	36.77	na	na	58.74	4.49	100.0
Not Sure Frequency						3
PRESUMED -- APPROACH END/SAME SIDE and TRAILING END/CROSSOVER/UNDIVIDED						
Roadway with median						
Frequency	0	1	99	110	5	215
Percentage	0.0	0.46	46.05	51.16	2.33	100.0
Roadway without median						
Frequency	62	na	na	105	9	176
Percentage	35.23	na	na	59.66	5.11	100.0
Not Sure Frequency						2

[a]"Other" location includes frontage roads, ramps, and cross roads/drives

TABLE 3 Type of Guardrail End Struck

	END HITS				APPROACH END/SAME SIDE and TRAILING END/CROSS OVER/UNDIVIDED			
	Presumed		Presumed -plus- Questionable		Presumed		Presumed -plus- Questionable	
Type of end	#	%	#	%	#	%	#	%
UNCOMBINED CATEGORIES								
Not able to determine	13	2.99	17	3.07	11	2.80	14	2.90
Exposed end	140	32.18	168	30.38	126	32.06	143	29.61
Turned-down end	241	55.40	316	57.14	218	55.47	280	57.97
Exposed end with significant flare	17	3.91	22	3.98	17	4.33	21	4.35
Turned-down with significant flare	8	1.84	10	1.81	6	1.53	7	1.45
Parabolic end	5	1.15	7	1.27	5	1.27	7	1.45
Rounded end	8	1.84	9	1.63	7	1.78	8	1.65
Other end type	3	0.69	4	0.72	3	0.76	3	0.62
Total	435	100.0%	553	100.0%	393	100.0%	483	100.0%
COMBINED CATEGORIES								
Exposed end	157	38.67	190	36.82	143	38.96	164	36.36
Turned-down end	249	61.33	326	63.18	224	61.04	287	63.64
Total	406	100.0%	516	100.0%	367	100.0%	451	100.0%

TABLE 4 Roll/Vault in Connection with Guardrail-End Accidents

	END HITS				APPROACH END/SAME SIDE and TRAILING END/CROSS OVER/ UNDIVIDED			
	No Roll/ Vault	Roll/ Vault	Not Sure	Total	No Roll/ Vault	Roll/ Vault	Not Sure	Total
ALL END TYPES								
PRESUMED								
Number	273	105	57	435	240	96	57	393
Percentage	62.76	24.14	13.10	100.0	61.07	24.43	14.50	100.0
PRESUMED-PLUS-QUESTIONABLE								
Number	354	125	74	553	298	112	73	483
Percentage	64.02	22.60	13.38	100.0	61.70	23.19	15.11	100.0
EXPOSED ENDS								
PRESUMED								
Number	116	24	17	157	104	22	17	143
Percentage	73.88	15.29	10.83	100.0	72.73	15.38	11.89	100.0
PRESUMED-PLUS-QUESTIONABLE								
Number	140	32	18	190	118	28	18	164
Percentage	73.69	16.84	9.47	100.0	71.95	17.07	10.98	100.0
TURNED-DOWN ENDS								
PRESUMED								
Number	133	78	38	249	115	71	38	224
Percentage	53.41	31.33	15.26	100.0	51.34	31.70	16.96	100.0
PRESUMED-PLUS-QUESTIONABLE								
Number	182	90	54	326	153	81	53	287
Percentage	55.83	27.61	16.56	100.0	53.31	28.22	18.47	100.0

TABLE 5 Severity of Accidents on State DOT System, 1988–1991

Severity	Total # of ALL Accidents 1988-1992	Presumed Approach End/Same Side and Trailing/ Crossover/ Undivided % of ALL	Presumed Guardrail End Hits % of ALL	Presumed-plus-Questionable End Hits % of ALL	Fixed Object-Culvert Acc. % of ALL	Fixed Object-Utility Pole Acc. % of ALL
Fatal	1,315	1.05%	1.14%	1.45%	2.05%	1.22%
Injury A	8,270	0.65%	0.69%	0.80%	1.81%	1.05%
Injury B	9,048	0.74%	0.82%	1.07%	2.01%	1.24%
Injury C	17,773	0.29%	0.32%	0.39%	0.74%	0.77%
PDO	63,757	0.32%	0.37%	0.47%	0.51%	0.84%
Total	100,163	0.39%	0.43%	0.55%	0.82%	0.89%

	GUARDRAIL END ACCIDENTS							
	END HITS				APPROACH END/SAME SIDE and TRAILING END/CROSS OVER/UNDIVIDED			
	Presumed		Presumed -plus- Questionable		Presumed		Presumed -plus- Questionable	
Severity	#	%	#	%	#	%	#	%
ALL END TYPES COMBINED								
Fatal+A	72	16.55	85	15.37	68	17.30	78	16.15
Inj B+C	130	29.89	167	30.20	119	30.28	147	30.43
PDO	233	53.56	301	54.43	206	52.42	258	53.42
Total	435	100.0%	553	100.0%	393	100.0%	483	100.0%
EXPOSED ENDS								
Fatal+A	24	15.29	30	15.79	22	15.38	27	16.46
Inj B+C	52	33.12	61	32.11	48	33.57	52	31.71
PDO	81	51.59	99	52.10	73	51.05	85	51.83
Total	157	100.0%	190	100.0%	143	100.0%	164	100.0%
TURNED-DOWN ENDS								
Fatal+A	46	18.47	52	15.95	44	19.64	48	16.73
Inj B+C	70	28.12	97	29.75	64	28.57	88	30.66
PDO	133	53.41	177	54.30	116	51.79	151	52.61
Total	249	100.0%	326	100.0%	224	100.0%	287	100.0%

Driver Inattention and Guardrail-End Accidents

Table 6 indicates that roughly a fifth of all guardrail-end accidents involved an inattentive driver striking the right-side guardrail end. On divided roads, the chance of an unalert driver hitting a guardrail end on the right side was slightly more than that for hitting an end on the near side or center of the median. On undivided roads, the chance for hitting a right-side guardrail end was almost double that for hitting a left side end. The actual portion of inattentive drivers involved in guardrail-end accidents may be greater; this categorization was made only if the police mentioned a form of inattention in the report.

Accident Frequency and Travel

The researchers performed analyses to determine whether guardrail-end strike frequency was a function of the amount of travel on the roadway. ODOT provided a file containing the number of miles of State highways for each 1,000 vehicles per day (ADT) volume increment. Volume data for 1989 were used as representative of the period 1988 through 1991.

For each ADT volume increment, the midpoint of the volume range was multiplied by the kilometers of road in that range to arrive at the vehicle kilometers of travel (VkT). For instance, for the volume range 1,000 to 1,999, VkT were calculated as follows:

$$1{,}500 \text{ vehicles per day} * 5016.9 \text{ km } (3{,}117.37 \text{ mi.}) = 7\ 525\ 381 \text{ VkT} \quad (4)$$

The percentage of VkT for each of the ADT groups was also determined.

It can be concluded from Table 7 that about 47 percent of the guardrail-end accidents occurred on 17 840 km (11,085 mi) of roads having volumes less than 10,000 ADT. These lower-volume roads constituted more than 90 percent of the length of the state highway system. About 53 percent of guardrail-end accidents were concentrated on the 10 percent of the system length having the higher ADTs. Figure 3 shows that a close relationship exists between the proportion of accidents and the proportion of VkT.

Multiple Factors: End Type Versus Roll/Vault Versus Severity

In addition to investigating the individual attributes of the guardrail-end accident problem, the researchers studied the

TABLE 6 Driver Inattention and Lateral Location of Guardrail-End Hit

	END HITS			
	Presumed		Presumed-plus-Questionable	
	#	%	#	%
ROADWAY WITH MEDIAN				
Left side	1	0.40	1	0.31
Median - left side	2	0.81	2	0.61
Median - right, center	35	14.17	42	12.84
Right side	43	17.41	53	16.21
Other	1	0.41	2	0.61
No mention	165	66.80	227	69.42
Total	247	100.0%	327	100.0%
ROADWAY WITHOUT MEDIAN				
Left side	19	10.27	23	10.31
Right side	42	22.70	52	23.32
Other	1	0.54	1	0.45
No mention	123	66.49	147	65.92
Total	185	100.0%	223	100.0%
NOT ABLE TO DETERMINE IF MEDIAN EXISTS	3		3	

TABLE 7 Number of Guardrail-End Accidents in Relation to Volume, Distance, and Distance of Travel

	PRESUMED END HITS							
Volume Range	Assumed Volume Midpoint	Presumed Accidents #	%	Length of Road on System # km	# mi.	%	Percent of VkT
<1000	500	18	4.14	5786.3	3595.41	29.40	3.54
1000-1999	1500	43	9.89	5016.9	3117.37	25.49	9.20
2000-2999	2500	45	10.34	2804.1	1742.41	14.25	8.57
3000-3999	3500	24	5.52	1522.7	946.17	7.74	6.52
4000-4999	4500	16	3.68	826.6	513.61	4.20	4.55
5000-6999	6000	31	7.13	1054.5	655.25	5.36	7.74
7000-9999	8500	27	6.21	828.5	514.78	4.21	8.61
10000-14999	12500	67	15.40	926.8	575.87	4.71	14.17
15000-24999	20000	44	10.11	495.6	307.94	2.52	12.12
25000-34999	30000	22	5.06	145.4	90.37	0.74	5.34
35000-44999	40000	7	1.61	52.4	32.57	0.27	2.56
45000-54999	50000	21	4.83	82.6	51.33	0.42	5.05
55000-64999	60000	15	3.45	34.7	21.54	0.18	2.54
65000-74999	70000	22	5.06	64.0	39.75	0.33	5.48
75000-84999	80000	7	1.61	13.8	8.56	0.07	1.35
85000-94999	90000	24	5.52	17.9	11.11	0.09	1.97
95000-100000	97500	2	0.46	5.9	3.69	0.03	0.71
Total		435	100.0%	19678.7	12227.73	100.0%	100.0%

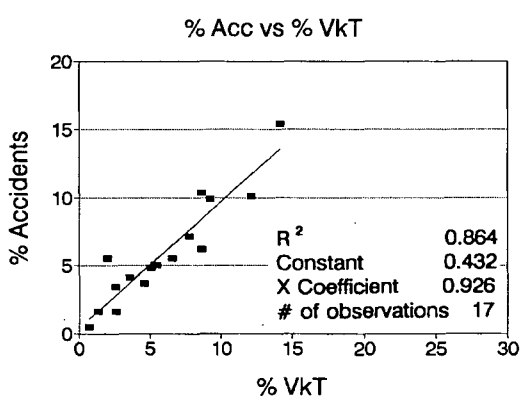


FIGURE 3 Guardrail-end accidents as function of VkT.

interrelationship of end type, occurrence of rolling and/or vaulting, and resulting accident severity. The Table 8 contingency tables give the number (#) of accidents having various combinations of attributes. Because previous examinations showed the proportions in the "presumed" data set to be similar to those in the "presumed-plus-questionable" set, the researchers developed these contingency tables for only the "presumed" sets. The chi-square test of independence with an α of 0.05 was performed on the presumed end-hit values. The test statistic χ^2 was 42.34 and χ^2 critical was 21.03. It was concluded that the classifications were not independent; that is, some kind of relationship between end type, roll/vault, and severity existed.

To investigate the relationship further, the GH statistic for comparing two means was computed. The following weights were assigned to the severity classes.

TABLE 8 End Type versus Roll/Vault versus Severity

	PRESUMED END HITS								PRESUMED APPROACH END/SAME SIDE and TRAILING END/CROSS OVER/UNDIVIDED							
	No Roll/ Vault		Roll/ Vault		Not Sure		Total		No Roll/ Vault		Roll/ Vault		Not Sure		Total	
Severity	Obs	*Exp*	Obs	*Exp*	Obs	*Exp*	Obs	*Exp*	Obs	*Exp*	Obs	*Exp*	Obs	*Exp*	Obs	*Exp*
EXPOSED																
Fat + Inj A	12	*16.6*	9	*6.8*	3	*3.7*	24	*27.1*	11	*14.9*	8	*6.3*	1	*3.7*	20	*24.9*
Inj B+ Inj C	41	*28.9*	7	*11.9*	4	*6.4*	52	*47.2*	37	*26.5*	7	*11.3*	6	*6.7*	50	*44.4*
PDO	63	*50.8*	8	*20.8*	10	*11.2*	81	*82.8*	56	*43.9*	7	*18.7*	10	*11.0*	73	*73.6*
Total	116	*96.3*	24	*39.4*	17	*21.3*	157	*157*	104	*85.3*	22	*36.2*	17	*21.4*	143	*143*
TURNED-DOWN																
Fat + Inj A	21	*26.3*	18	*10.8*	7	*5.8*	46	*42.9*	20	*23.3*	17	*9.9*	7	*5.9*	44	*39.1*
Inj B+ Inj C	31	*45.9*	31	*18.8*	8	*10.1*	70	*74.8*	27	*41.5*	29	*17.6*	8	*10.4*	64	*69.6*
PDO	81	*80.5*	29	*33.0*	23	*17.8*	133	*131.3*	68	*68.8*	25	*29.2*	23	*17.3*	116	*115.4*
Total	133	*152.7*	78	*62.6*	38	*33.7*	249	*249*	115	*133.7*	71	*56.8*	38	*33.7*	224	*224*

Type of Severity	*Weight*
Fatal + Injury A	5
Injury B + Injury C	3
PDO	1

Thus, each accident that had a severity classification of fatal or injury-A received a score of 5 for severity, and so on. Using severity as the dependent variable, the mean value and standard deviation for each combination of the factors (end type and roll/vault) was then calculated.

The four comparisons in Table 9 were statistically significant; the p-values were less than the 0.05 α-value. It can be concluded that the severity associated with roll/vault accidents for both exposed and turned-down ends was higher than the severity associated with no roll/vault accidents.

Three statistical tests for comparing proportions were performed for presumed end hits to determine whether any significant differences existed in the proportions of roll/vault and the associated severity for the exposed and turned-down ends.

TABLE 9 Games-Howell Analysis

		MEAN SEVERITY OF END TYPE AND ROLL/VAULT			
		PRESUMED END HITS		PRESUMED APPROACH END/SAME SIDE and TRAILING END/ CROSSOVER/UNDIVIDED	
Group		Mean	Standard deviation	Mean	Standard deviation
EXPOSED (EX)					
No Roll/Vault	EX-N	2.12	1.35	2.13	1.36
Roll/Vault	EX-RV	3.08	1.72	3.09	1.69
Not sure	EX-NS	2.18	1.59	1.94	1.25
TURNED-DOWN (TD)					
No Roll/Vault	TD-N	2.10	1.51	2.17	1.54
Roll/Vault	TD-RV	2.72	1.54	2.77	1.53
Not sure	TD-NS	2.16	1.59	2.16	1.59

	PAIRWISE COMPARISON OF SEVERITY MEANS					
	PRESUMED END HITS			PRESUMED APPROACH END/SAME SIDE and TRAILING END/ CROSSOVER/UNDIVIDED		
Groups	t-statistic	df	p	t-statistic	df	p
TD-N vs. TD-RV	-2.8504	158.8743	0.0049	-2.6271	149.2597	0.0095
EX-N vs. EX-RV	-2.5853	29.1839	0.0150	-2.4923	27.0481	0.0191
TD-N vs. EX-RV	-2.6345	29.7348	0.0132	-2.3884	28.1324	0.0239
EX-N vs. TD-RV	-2.7837	150.7217	0.0061	-2.8390	138.2259	0.0052

The raw values were obtained from Table 8. The following assumptions apply for the three following tests.

H_0: $p_{exp} = p_{td}$

H_{alt}: $p_{exp} \neq p_{td}$

$\alpha = 0.05$

critical $z_{\alpha/2} = 1.96$

- Test 1: compare the proportion of exposed end, Fatal + Injury A accidents having roll/vault with the proportion of turned-down end, Fatal + Injury A accidents having roll/vault.

$p_{exp} = 9/24 = 0.375$

$p_{td} = 18/46 = 0.391$

test $z = 0.131$

H_0 was not rejected as the test z did not exceed the critical $z_{\alpha/2}$. Comparing exposed with turned-down ends, there was no significant difference in the proportions of Fatal + Injury A accidents which had rolling or vaulting.

- Test 2: compare the proportion of exposed end, roll/vault accidents having a severity of Fatal + Injury A with the proportion of turned-down end roll/vault accidents having a severity of Fatal + Injury A.

$p_{exp} = 9/24 = 0.375$

$p_{td} = 18/78 = 0.231$

test $z = 1.399$

H_0 was not rejected as the observed z did not exceed the critical $z_{\alpha/2}$. Given that a roll/vault accident had occurred, the proportion of the accidents that were Fatal + Injury A was higher for exposed ends than for turned-down ends, but the difference was not statistically significant. A larger sample size could have produced a finding of statistical significance.

- Test 3: compare the proportion of exposed end, Fatal + Injury A accidents out of total exposed end accidents to the proportion of turned-down end, Fatal + Injury A accidents out of total turned-down end accidents.

$p_{exp} = 24/157 = 0.153$

$p_{td} = 46/249 = 0.185$

test $z = 0.81$

H_0 was not rejected as the observed z did not exceed the critical $z_{\alpha/2}$. The proportion of Fatal + Injury A accidents associated with turned-down ends was not significantly different from the proportion of Fatal + Injury A accidents associated with exposed ends. Even though there was a significantly higher likelihood of rolling or vaulting associated with turned-down ends, tests showed no significant differences between severe accident proportions at exposed ends and at turned-down ends.

FINDINGS SUMMARIZED

The majority of the guardrail accidents in the data set occurred along the guardrail midsection; a quarter of the total were presumed guardrail-end accidents. The front end of the guardrail was struck much more often than the trailing end. In most of the end accidents, the front or side of the vehicle struck the guardrail.

On divided roads, the chances of a vehicle hitting the guardrail end on the right side of the road or the end in the median were similar. The chance of a vehicle's crossing the median and hitting the guardrail end on the far left side (that is, right side of the oncoming main road) was small. On undivided roads, about 60 percent of accidents involved the vehicle striking the guardrail end on the right side.

Roughly one-third of all guardrail-end accidents involved an inattentive driver striking a guardrail end. For roads with and without medians, the right-side guardrail end was most often struck when inattention was mentioned in the accident report.

About 47 percent of the guardrail-end accidents occurred on 90 percent of the State highway system length, while 53 percent were concentrated on the 10 percent of the system having higher ADTs. There was a close relationship between the percentage of accidents and the percentage of VkT.

The majority of the guardrail-end accidents were PDO accidents. For all end types combined, about a sixth of the accidents were fatal or incapacitating injury (Injury A) accidents. The proportion of all accidents having fatalities or incapacitating injuries was about the same for both exposed ends and turned-down ends.

In most of the guardrail-end accidents, the vehicle did not vault or roll. There was a relationship between type of end, roll/vault, and accident severity. The severity associated with roll vault accidents for both exposed and turned-down ends was significantly higher than the severity associated with no-roll/vault accidents. The proportion of vehicles rolling or vaulting after hitting turned-down ends was higher than the proportion of vehicles rolling or vaulting after hitting exposed ends. When a roll/vault did occur, the results were more severe with exposed ends than with turned-down ends, although the difference was not statistically significant.

In addition to percentages and proportions, actual numbers have to be considered. The data from 1988 through 1991 indicated that there were about four to five fatal guardrail-end accidents per year. The turned-down ends accounted for just under 60 percent of the fatalities (slightly more than 60 percent of all end accidents were at turned-down ends). In conjunction with all end impacts, there were about 15 or 16 Injury-A accidents per year, and about 20 to 25 A-injuries per year. About 60 percent of the injuries occurred at turned-down ends. The accident reports indicated that at roughly three-fourths of those accidents where there was or may have been rolling or vaulting, the vehicle occupants suffered either B, C, or no injuries.

RECOMMENDATIONS

While reviewing the accident reports and analyzing the data, the researchers made a number of observations. These observations led to the following suggestions.

Accident Reporting

The quality of an accident study is constrained by the quality of the data base, in this case the quality of individual police accident reports. While the majority of the reports were adequate, some were not. Because police may not have experience with using accident reports to find solutions to traffic safety problems, it may be difficult for police in the field to appreciate the needs of other users of accident reports. Training sessions in which police are given actual examples of unclear accident reports, then asked to identify accident details, may help them improve the quality of their reports.

Accident report quality would be improved if police had global positioning devices to report accident locations while physically at the accident site. With the proper codes, police could report to within a few feet the first "point of error" and the final resting place of vehicles in accidents. This would reduce the amount of time later spent in offices, trying to figure out where the accident took place. It would enhance the ability of office staff to identify locations with elevated accident frequencies or rates.

Response Strategy

The research suggests that any program of installing newer guardrail-end treatments should first target the more-traveled roads. Installing newer end types along a relatively small portion of the state system could address the majority of guardrail-end accident occurrences. The results also suggest targeting those lateral locations that were most likely to be struck.

Roughly a third of all guardrail-end accidents involved an inattentive driver striking a guardrail. Recent research suggests that rumble or chatter strips constructed on the shoulder at the lane edge may reduce the number of accidents, by alerting inattentive drivers about to run off the road. At sites with lesser probabilities of end strikes, states may wish to test the benefits of rumble strips as an inexpensive countermeasure.

Currently, a major issue in guardrail-end treatment is replacing the existing ends with, and specifying on new projects, the newer, more expensive end treatments. The results of this research should assist those who evaluate these policy options.

ACKNOWLEDGMENT

The authors thank the Oklahoma Department of Transportation for supporting this study.

REFERENCES

1. Ivey, D. L., M. E. Bronstad, and L. I. Griffin III. Guardrail End Treatments in the 1990s. In *Transportation Research Record 1367*, TRB, National Research Council, Washington, D.C., 1992, pp. 63–74.
2. Griffin, L. I. III. *An Analysis of Accidents on Turned Down Guardrail Ends in the State of Texas*. Texas Transportation Institute, Texas A&M University, College Station, May 1991.
3. L. E. Toothaker. *Multiple Comparisons for Researchers*. Sage Publications, Newbury Park, Calif., 1991, pp. 98–99.

Publication of this paper sponsored by Committee on Roadside Safety Features.

Comparative Performance of Barrier and End Treatment Types Using the Longitudinal Barrier Special Study File

WILLIAM W. HUNTER, J. RICHARD STEWART, AND FORREST M. COUNCIL

By using the Longitudinal Barrier Special Study file developed in the 1980s for FHWA as part of the National Highway Traffic Safety Administration's National Accident Sampling System data gathering, the performance of various types of guardrails, median barriers, and end treatments as well as the risk to the driver when striking a barrier length of need (LON) versus an end section are compared. Data elements describing the barriers and end treatments were much more detailed than is typically the case in police investigations. Multi-vehicle crashes were excluded, because of the difficulty in determining when driver injury occurs. Most of the analysis focuses on cases in which a vehicle strikes only a single barrier, though sometimes more than once. It was found that weak post barriers were less associated with driver injury than other barrier types. In regard to subsequent impact, rollover produced the highest rate of driver injury. Higher risks of serious driver injury were associated with blunt and turndown end treatments than with LON. End hits were more likely to result in serious driver injury than LON by being more likely to produce rollover and by producing more serious injuries when no rollovers occur.

For roadway and roadside design engineers and others interested in the performance of hardware, a persistent topic of interest has been the performance of various types of guardrails, median barriers, and end treatments, and the comparison of risk to the driver when striking a barrier length of need (LON) versus an end section. This topic was explored using the Longitudinal Barrier Special Study (LBSS) file developed in the 1980s for FHWA as part of the National Highway Traffic Safety Administration's (NHTSA) National Accident Sampling System (NASS) activity. Data were generated by NASS investigators working through various zone centers in the United States. For an eligible case, detail far beyond routine police crash investigation was obtained, with barrier descriptors including items such as post spacing, presence of block-out, end treatment type, angle of impact, yawing angle, and barrier performance (*1*).

From 1982 to 1986, about 1,200 crashes involving roadside barriers were input to the file for analysis. Almost half of the crash data was obtained in 1982 and 1983 (almost 300 cases each year). Within the file there are subsets of data pertaining to barrier, accident, vehicle, driver, occupant, and pedestrian variables. These variables can be linked to generate a vehicle-based contacts file (that is, to describe a vehicle's path through the crash, which could involve multiple barrier contacts and contacts with other vehicles or objects). A problem with building the file and conducting subsequent analysis was that the coding for the "object contacted" variable in the barrier subset was not uniform across the 5 years of data collection. Considerable recoding was required to develop the consistency necessary to ensure appropriate sample sizes for analysis.

University of North Carolina, Highway Safety Research Center, CB 3430, Chapel Hill, N.C. 27599-3430.

REVIEW OF LITERATURE

The design of longitudinal traffic barriers has been influenced greatly by two basic assumptions: (a) occupants are subjected to the highest risk of injury during the vehicle's initial collision with a barrier, and (b) the probability of severe occupant injury is directly related to the intensity of vehicle collision accelerations (*2*). To assess these assumptions, Ray et al. (*2*) conducted sled tests and analyzed data from several accident files (including 1982 and 1983 LBSS data) and full-scale crash tests. They found that occupants rarely sustain serious injuries in collisions with longitudinal barriers if the vehicle remains upright and is redirected smoothly by the barriers. They concluded that the vehicle's post-impact trajectory is an important component of barrier performance and should be considered more carefully in future barrier development and testing.

A follow-up study by Ray et al. analyzed the risk of occupant injury in second collisions (*3*). Crash data from North Carolina and New York suggested that occupants were three times more likely to suffer severe injury in a second collision after the vehicle had been redirected successfully from a longitudinal barrier. Criteria for eligible cases included the following: (a) the longitudinal barrier was the first object struck, (b) the vehicle was a passenger vehicle, (c) the impact was in the midsection (LON) of the barrier, and (d) the impact angle was oblique. For the North Carolina data, nontracking vehicles were eliminated.

Bryden and Fortuniewicz analyzed data from completed field investigations at 3,302 traffic barrier accident sites in New York State to determine the effects of various parameters on barrier performance (*4*). Their research showed that collisions with traffic barriers resulted in low occupant injury rates than roadside accidents in general. Traffic barriers performed best for midsize passenger automobiles in terms of injury severity as well as vehicle containment and secondary collisions. They did not perform as well for vans and light trucks because of secondary rollovers. The severe injury rates of heavy trucks were comparable to those of automobiles, but

heavy trucks often penetrated barriers and were involved in secondary collisions.

As part of a study on severity measures for roadside objects and features, Mak et al. analyzed two data sources containing information on real-world impact conditions for 472 pole accidents and 124 bridge accidents (*5*). They found that a gamma function provided the best fit for both univariate impact speed and impact angle distributions. Impact speed and angle probability distributions were developed for various functional classes of roadways. The authors cautioned that their results are limited only to reported accidents and that accidents involving poles and safety features at bridge sites are not necessarily representative of all run-off-road crashes.

Troxel et al. used the 1980–1985 Fatal Accident Reporting System and the 1982–1985 NASS data bases to extract the characteristics of side-impact accidents with fixed roadside objects (*6*). Such accidents involved tall, narrow objects such as trees and utility poles. Narrow-object collisions appeared to be twice as likely to result in fatalities as were broad-object collisions. Guardrail accidents accounted for 9 percent of all side-impact, fixed-object crashes and 4 percent of fatal side-impact, fixed-object crashes. All guardrail fatalities were caused by collisions with end sections and transitions. The authors suggested that the most effective countermeasures for guardrail collisions would involve improving the performance of terminals and transitions.

Pigman and Agent analyzed 110 accidents involving breakaway-cable-terminal (BCT) end treatments and 36 accidents involving median breakaway-cable-terminal (MBCT) end treatments as used in Kentucky (*7*). The BCT end treatment performed properly in 73 percent of accidents, with the wooden posts breaking away or the guardrail redirecting the vehicle. Proper performance ranged from 60 percent for end sections with no offset to 69 percent for a simple curve offset and 79 percent for a parabolic flare offset. The MBCT performed properly 63 percent of the time. The authors recommended that the MBCT end treatment design be modified or eliminated because of its stiffness and the problems associated with impacts at shallow angles.

Rollovers are an undesirable result of barrier crashes. To evaluate the performance of the concrete safety-shape barrier with the New Jersey profile, Perera and Ross used a modified version of the Highway-Vehicle-Object-Simulation Model (HVOSM) (*8*). They determined that overturns could be expected for small cars in nontracking or high-angle impacts with the concrete safety-shape barriers. The overturn problem could be mitigated by installing a barrier with a constant-slope face or a vertical wall. A retrofit design for the barrier consisting of a longitudinal member placed on the side of the barrier near the top also showed promise in reducing the problem.

Through statistical analyses of three accident data files and computer simulation using a modified version of HVOSM, Mak and Sicking also examined rollover accidents caused by concrete safety-shape barriers (*9*). Three impact conditions were identified as potential contributory factors to rollovers: (1) high impact angle and moderate to high impact speed; (2) high slip angle, low to moderate yaw rate, and moderate to high impact speed; and (3) high impact speed and low impact angle for vehicles in a tracking mode. The authors considered three alternative barrier shapes as countermeasures for reducing rollover rates. Of these, the F-shape offered little performance improvement and the vertical wall offered the greatest reduction in rollover potential, but with the greatest increase in lateral accelerations. The constant-sloped barrier was suggested as perhaps the best compromise solution.

CREATION OF THE ANALYSIS FILE

The LBSS data collected by field investigators were partitioned into accident, vehicle, occupant, driver, contacts, and impacts subfiles. The main file used for the analyses related to barrier and end types and risk in LON versus end crashes was a vehicle-oriented file built from these subfiles, with one record per vehicle. The analysis file was developed from an original accident-oriented file that contained information on all impacts by all vehicles involved in a crash and the object contacted for each impact. The file was first restricted to only single-vehicle accidents, with the reasoning being that the deletion of multiple-vehicle crashes would help ensure that the harm to the driver was related predominantly to the barrier impact rather than the impact with another vehicle, either before or after striking the barrier. Some 168 variables on the merged file were selected as candidates for analysis from the accident, vehicle, occupant, driver, contacts, and impacts subsets.

The analysis file also contained a "flag" variable that allowed it to be categorized into three types of analysis records, all of which involved single-vehicle crashes: (a) an "all hits" subfile, in which a vehicle may strike a barrier and then another barrier or object, (b) a "barrier hits only" subfile, in which a vehicle strikes no objects other than one or more longitudinal barriers, and (c) a "clean hits" subfile, in which a vehicle strikes only a single barrier, though sometimes more than once. Thus, using the clean hit file, the instances in which a vehicle struck a barrier, and nothing else, or the vehicle struck the same barrier several times could be examined.

The clean hits and barrier hits only subfiles are contained within the all hits file. The analyses used the clean hits and all hits data. Using the clean hits data is the most appropriate way to examine harm caused by a specific barrier type, in that any driver injury would be the result of striking the barrier. If a vehicle strikes another vehicle or object and then a barrier, determination of when the injury occurred is speculative at best. Thus, for most of the analyses, the clean hits file was used to verify the results from the larger all hits file. The clean hits file contained 665 vehicle records, and the all hits file contained 1,062 vehicle records.

CLEAN HITS FILE

Much of the analyses involved the clean hits subfile. Results from the all hits subfile were quite similar. From the clean hits data, 665 single vehicle impacts were available, 450 pertaining to LON and 215 to end-of-barrier crashes (with end hit defined as an impact occurring within the first 25 ft of the barrier). The percentages in the following data pertain to the total of 665 impacts.

- Guardrail types—23 percent G4 (1S) (blocked out W-beam with steel posts), 20 percent W-beam strong post.

- Median barrier types—58 percent concrete median barrier.
- Blocked-out presence—55 percent blocked out.
- End treatment type—41% blunt, 30 percent turndown, 10 percent breakaway cable.
- Location of end treatment in direction of vehicle travel—78 percent upstream (the first end a vehicle would encounter in normal direction of travel).
- Distance from end of barrier to initial point of impact—21 percent within first 3 m (10 ft).
- Length of longitudinal barrier section—87 percent longer than 30 m (100 ft).
- Location of barrier in direction of vehicle travel—61 percent off left side of road, 36 percent off right side. (When the item was coded, guardrail crashes split as 44 percent off of left side and 56 percent off of right side.)
- Curb presence—present 16 percent of time.
- Curb height—61 percent 25–127 m (1–5 in.), 39 percent 152–254 m (6–10 in.).
- Perpendicular distance from curb to barrier—62 percent 1 m (3 ft) or less.
- Total change in elevation—13 percent no change, 57 percent below edge of roadway, 30 percent above edge of roadway. (For LON, virtually the same percentages. For ends, 12 percent no change, 67 percent below edge of roadway, 21 percent above edge of roadway.)
- Longitudinal barrier height—32 percent 660–737 m (26–29 in.) in height, 30 percent 762–838 m (30–33 in.) in height. [For LON, 29 percent 660–737 m (26–29 in.) in height, 36 percent 762–838 m (30–33 in.) in height. For ends, 38 percent 660–737 m (26–29 in.) in height, 17 percent 762–838 m (30–33 in.) in height.]
- Total horizontal distance to barrier—3 percent barrier at edge of road, 22 percent 0.3–1.2 m (1–4 ft) from edge, 26 percent 1.5–2.4 m (5–8 ft) from edge, 29 percent 2.7–3.7 m (9–12 ft) from edge, and 20 percent 4 m (13 ft) and greater from edge.
- Length of direct contact with barrier—63 percent 0.3–6.1 m (1–20 ft) of contact, 27 percent 6.4–15.3 m (21–50 ft) of contact.
- Impact angle—1 percent at zero degrees, 21 percent at 1–8 degrees. (For LON, 1 percent at zero degrees and 15 percent at 1–8 degrees. For ends, 3 percent at zero degrees and 24 percent at 1–8 degrees).
- Yawing angle at impact—4 percent at zero degrees, 14 percent at 1–8 degrees, 32 percent greater than 27 degrees. (For LON, 3% at zero degrees, 11 percent at 1–8 degrees, 31 percent greater than 27 degrees. For ends, 5 percent at zero degrees, 22 percent at 1–8 degrees, 33 percent greater than 27 degrees).
- Impact speed—36 percent at 34–56 km/hr (21–35 mph), 30 percent at 58–72 km/hr (36–45 mph), 12 percent at 74–89 km/hr (46–55 mph), and 9 percent greater than 89 km/hr (55 mph). (For LON and ends separately, virtually the same percentages.)
- Separation angle—63 percent at 0–8 degrees. (For LON ends separately, virtually the same percentage.)
- Barrier performance—65 percent redirected, 11 percent snagged, 12 percent overrode. (For LON, 77 percent redirected, 9 percent snagged, 7 percent overrode; for ends, 38 percent redirected, 16 percent snagged, 22 percent overrode.)
- Postimpact trajectory—50 percent remained on roadside, 25 percent returned to roadway, 16 percent went on top of/over/through. (For LON, 52 percent remained on roadside, 28 percent returned to roadway, 11 percent went on top of/over/through. For ends, 47 percent remained on roadside, 18 percent returned to roadway, 25 percent went on top of/over/through.)
- Subsequent impact—10 percent rollover (9 percent for LON and 13 percent for ends).
- Driver age—0.3 percent less than 16 years; 22 percent 16–20 years; 20 percent 21–24 years; 31 percent 25–35 years; 20 percent 36–55 years; 5 percent 56–75 years; 1 percent 76 and over.
- Driver injury, MAIS scale—40 percent no injury, 45 percent minor injury, 9 percent MAIS 3 and above. (For LON, 42 percent no injury, 48 percent minor injury, 3 percent MAIS 3 and above. For ends, 40 percent no injury, 43 percent minor injury, 11.5 percent MAIS 3 and above.)
- Driver injury, KABCO scale—51 percent no injury, 12 percent possible or C injury, 24 percent non-incapacitating or B injury, 11 percent incapacitating or A injury, 0.8 percent killed. (For LON, 52% no injury, 12 percent C injury, 24 percent B injury, 10 percent A injury, and 0.7 percent killed. For ends, 48 percent no injury, 10 percent C injury, 23 percent B injury, 18 percent A injury, 1.4 percent killed.)
- Vehicle type—83 percent passenger cars, 15 percent light trucks and vans, and 3 percent heavy trucks.

REPRESENTATIVES OF THE LBSS FILE

Because of the nature of the questions that can be analyzed with the LBSS file, it is important to have some understanding of how "representative" the file is of barrier impacts in the United States. The LBSS file cannot be used for determining frequency or rate of barrier impacts, so questions concerning representativeness are related to the severity of the crash.

To examine the issue of representativeness, information was extracted from accident files from North Carolina, Michigan, Utah, Maine, and Illinois. The latter four states are part of the Highway Safety Information System, the FHWA data base used in many of their internal analyses. A number of tables were examined. In summary, fatal driver injuries occurred almost twice as often in the LBSS file as in all other files, and serious injury occurred slightly more often than in the North Carolina towaway file and significantly more often than in the Michigan towaway file or in the other states. Part of the difference in severity may be related to the selection of the LBSS sample of crashes for investigations. The emphasis on fatal crashes in some NASS procedures may have biased this file to a certain extent. In short, while much more like a towaway file than a total crash file, the LBSS file may indeed contain a slightly more severe set of guardrail impacts than is the case in the comparison groups of states. The analyses that follow are based exclusively on LBSS data and are comparative, so that representativeness is much less of a concern.

ANALYSIS AND RESULTS

The following section presents the methods and results of the analyses conducted. Again, the major questions being ex-

plored were (a) the comparative injury-related (injury severity to drivers) and vehicle trajectory-related (redirected, snagged, vaulted) performance of different types of barriers and different types of barrier end treatments, and (b) the comparison of performance for LON versus ends. Data pertaining to the exposure of vehicles to barriers and ends were unavailable for analysis, as were data pertaining to low severity impacts (driveways) where no crash data are reported to police or other investigating units. The analyses thus compare various barrier and end types.

LON Analysis

Comparisons of Barrier Types Within LON

Based on what was available in the file, nine types of guardrails (GR) and median barriers (MB) were grouped for analyses:

Barrier Type	*LBSS Description*
GR-1 (weak post)	G1, G2, G3
GR-2 (strong post)	G4 (1W), G4 (2W), G4 (1S), G4 (2S), G9
GR-3 (rigid)	Concrete safety shape
GR-4 (other)	Other guardrail type
GR-9 (W-beam strong post)	W-beam (strong post)
MB-5 (weak post)	MB1, MB2, MB3
MB-6 (strong post)	MB4W, MB4S, MB9
MB-7 (rigid)	Concrete median barrier (MB5)
MB-8 (other)	MB7, other median barrier type

(Note that GR-9, W-beam strong post, was examined as a separate category and not merged with the other types in GR-2. Much of the data referred to an older type of guardrail system likely to be located on lower-volume and lower-speed roadways, which is no longer installed.)

Distributions of severity of injuries to drivers involved in crashes into the LON of these barriers are shown in Tables 1 and 2 in terms of the KABCO and MAIS injury severity scales, respectively. These tables are based on all barrier hits, not just clean hits. The right-most column of Table 1 also gives the percentage of A or K injuries for each barrier type, and at the bottom of this column are results of a significance test comparing these A or K percentages. In Table 2 the last three columns show percentages having MAIS $\geq$ 1, MAIS $\geq$ 2, and MAIS $\geq$ 3, respectively (representing any injury, moderate to severe injury, and fairly severe injury), with significance test results given at the bottom of the columns. Note that no X^2 is shown for MAIS $\geq$ 3 because the data were too sparse for the test to be valid. The severe (A or K) KABCO injury differences across barrier types were marginally significant (p = .055), but the MAIS $\geq$ 3 injury differences were based on too few data to make valid comparisons. Because there were relatively few injuries at MAIS level 2, differences in the percentage with MAIS $\geq$ 2 were also only marginally significant (p = .10). On the other hand, the MAIS $\geq$ 1 differences were highly significant (p = .000).

Although the KABCO results look a bit worse, Tables 1 and 2 show that relatively few serious (or fatal) injuries resulted from hits into the LON for any of the barrier types. Thus, if differences in injury severity distributions between barrier types exist, they appear to be occurring primarily at the lower end of the severity scale. This is confirmed by the statistical tests associated with the MAIS data in the last three columns of Table 2.

Results were similar when the data were limited to clean hits into the barriers. Again, the significance tests showed significant differences across the barrier types with respect to driver injury versus no injury, but nonsignificant differences for MAIS $\geq$ 2. Both the all hits and the clean hits data suggest higher injury rates associated with barrier types GR-3 (rigid guardrail), MB-6 (strong post median barrier), and MB-7 (concrete median barrier).

All subsequent comparisons of barrier type were based on driver injury (MAIS $\geq$ 1) versus no injury (MAIS = 0) using MAIS. This is because the MAIS data were obtained from medical records and are considered more reliable than the police codes. A limitation is that the MAIS data are not as complete. Rows 1 to 5 (guardrails) and rows 6 to 9 (median barriers) of Table 2 were also analyzed separately as subtables relative to MAIS $\geq$ 1. For all hits the respective X^2 statistics and p-values were X_4^2 = 6.762 (p = .149) for guardrails and X_3^2 = 14.874 (p = .002) for median barriers; for clean hits these quantities were X_4^2 = 9.975 (p = .041), and X_3^2 = 10.298 (p = .016). Thus, when considered separately the differences in injury rates across the different types of median barriers were statistically significant. The injury rate differences across guardrail types were only marginally significant. Because injury rates for guardrails and median barriers were relatively similar, it seemed most efficient to analyze guardrails and barriers together instead of splitting a moderate-sized data set into two rather small subsets.

To further investigate differences in barrier type while taking into account the effects of certain covariates, logistic models of the form

$$\log \frac{p}{1-p} = \beta_o + \sum_{j=1}^{J} \beta_j X_j \qquad (1)$$

were fit to the data using SAS PROC LOGISTIC (*10*). The quantity $R = (p/1 - p)$ in Equation 1 can be thought of as the risk of injury. Taking exponentials of both sides of Equation 1 shows that injury risk (R) can be expressed as the product of factors

$$R = (e^{\beta_o})(e^{\beta_1 X_1})(e^{\beta_2 X_2}) \cdots (e^{\beta_j X_j})$$

A model of this type formulated to make comparisons among the five guardrail and four median barrier types contained eight dummy or indicator variables to flag the various guardrail and barrier types:

X_1 = 1 if GR-2, 0 otherwise.

X_2 = 1 if GR-3, 0 otherwise.

.
.
.

X_8 = 1 if MB-8, 0 otherwise.

The model also contained three covariates: impact speed, vehicle curb weight, and impact angle. Thus, the estimated

TABLE 1 Driver Injury Severity (KABCO) by Barrier Type: All Hits into LON

	KABCO Injury Severity						
Type	0	C	B	A	K	Total	Percent A or K
GR-1	35	8	10	0	0	53	
	(66.0)	(15.1)	(18.9)	(0.0)	(0.0)		0.00
GR-2	72	13	36	20	3	144	
	(50.0)	(9.0)	(25.0)	(13.9)	(2.1)		16.0
GR-3	4	1	8	1	0	14	
	(28.6)	(7.1)	(57.1)	(7.1)	(0.0)		7.14
GR-4	49	10	15	6	0	80	
	(61.3)	(12.5)	(18.8)	(7.5)	(0.0)		7.5
GR-9	25	6	10	6	1	48	
	(52.1)	(12.5)	(20.8)	(12.5)	(2.1)		14.6
MB-5	23	3	5	3	0	34	
	(67.7)	(8.8)	(14.7)	(8.8)	(0.0)		8.8
MB-6	16	6	11	6	1	40	
	(40.0)	(15.0)	(27.5)	(15.0)	(2.5)		17.5
MB-7	55	17	47	20	3	142	
	(38.7)	(11.9)	(33.1)	(14.8)	(2.1)		16.2
MB-8	12	4	7	3	0	26	
	(46.2)	(15.4)	(26.9)	(11.5)	(0.0)		11.5
						580	$X^2_8 = 15.2$
							$p = .055$

Key:

Barrier Type

GR-1 (weak post)
GR-2 (strong post)
GR-3 (rigid)
GR-4 (other)
GR-9 (W-beam strong post)
MB-5 (weak post)
MB-6 (strong post)
MB-7 (rigid)
MB-8 (other)

model coefficients provide estimates of the relative injury risk associated with the different barrier types while taking into account that the different types of barriers may have been struck by vehicles of different sizes with different speeds and impact angles. Ranges of the variation in these factors are given in the introductory section.

Other variables tested as covariates but found not to make a statistically significant improvement in the model were yaw angle, effective barrier height, separation angle, horizontal distance to barrier, length of barrier, and driver age. For a vehicle striking guardrail type GR-1, all of the variables $X_1 = X_2 = \cdots = X_8 = 0$, so the estimated injury risk has the form

$$R_1 = (e^{\beta_0}) \text{ (covariate effects)}$$

For a vehicle striking guardrail type GR-2, the dummy variable $X_1 = 1$ and all other dummy variables are zero, so the injury risk has the form

$$R_2 = (e^{\beta_0})(e^{\beta_1}) \text{ (covariate effects)}$$

For fixed values of the covariates then

$$R_2 = (e^{\beta_1})\, R_1$$

Thus, the estimated model coefficients $\beta_1, \ldots, \beta_8$ gives multiplication factors for injury risks of GR-2, . . . , MB-8 relative to GR-1. Table 3 gives the estimated values of these coefficients when the model was fit to the all hits data. The table also gives standard errors and p-values for the estimated pa-

TABLE 2 Driver Injury Severity (MAIS) by Barrier Type: All Hits into LON

	MAIS Injury Severity								Percent Injured		
Type	0	1	2	3	4	5	6	Total	MAIS ≥ 1	MAIS ≥ 2	MAIS ≥ 3
GR-1	27	4	2	1	0	0	0	54			
	(50.0)	(44.4)	(3.7)	(1.9)	(0.0)	(0.0)	(0.0)		50.0	5.6	1.9
GR-2	50	62	13	4	0	1	2	132			
	(37.8)	(47.0)	(9.9)	(3.0)	(0.0)	(0.8)	(1.5)		62.1	15.2	5.3
GR-3	3	8	3	0	0	0	0	14			
	(21.4)	(57.1)	(21.4)	(0.0)	(0.0)	(0.0)	(0.0)		78.6	21.4	0.0
GR-4	40	33	5	3	0	0	0	81			
	(49.4)	(40.7)	(6.2)	(3.7)	(0.0)	(0.0)	(0.0)		50.6	9.9	3.7
GR-9	22	18	3	2	0	0	1	46			
	(47.8)	(39.1)	(6.5)	(4.4)	(0.0)	(0.0)	(2.2)		52.2	13.0	6.5
MB-5	21	12	0	1	0	0	0	34			
	(61.8)	(35.3)	(0.0)	(2.9)	(0.0)	(0.0)	(0.0)		38.2	2.9	2.9
MB-6	10	21	5	2	0	0	1	39			
	(25.6)	(53.9)	(12.8)	(5.1)	(0.0)	(0.0)	(2.6)		74.4	20.5	7.7
MB-7	36	80	16	4	1	0	2	139			
	(25.9)	(57.6)	(11.5)	(2.9)	(0.7)	(0.0)	(1.4)		74.1	16.6	5.4
MB-8	9	16	1	0	0	0	0	26			
	(34.6)	(61.5)	(3.9)	(0.0)	(0.0)	(0.0)	(0.0)		65.4	3.9	0.0
								559	X^2_8=30.5 p = .000	X^2_8=13.3 p = .102	

Key:

Barrier Type
GR-1 (weak post)
GR-2 (strong post)
GR-3 (rigid)
GR-4 (other)
GR-9 (W-beam strong post)
MB-5 (weak post)
MB-6 (strong post)
MB-7 (rigid)
MB-8 (other)

rameters. Because barrier types 2 to 9 are compared in this model with barrier type 1, the statistical significance of the estimates $B_1, \ldots, B_8$ is a function of the variability in injury severity associated with hits into barrier type 1, and with each of the other types in turn. Thus, the variability (or inversely the stability) of the behavior of barrier type 1 is a factor relative to the statistical significance of each of the other barrier effect estimates. The estimate of β_1 is statistically significant and has the value $\beta_1 = .84$. Thus, for equal speed, curb weight, and impact angle, the injury risk for a crash into guardrail type GR-2 is estimated to be $(e^{.84}) = 2.32$ times the injury risk of a crash into a guardrail of type GR-1.

Guardrail type 1, weak post systems, served as a reference group or baseline for the model of Table 3. Thus, each barrier type is compared with weak post systems, a design that provides a more forgiving impact by allowing large deflection. These systems should be limited to locations where such large deflections are permissible. A disadvantage of these systems is that barrier damage in a crash is generally extensive. That all of the estimates are positive indicates that the estimated injury risk was higher for each of the other barrier types than for weak post systems. However, the standard error and *p*-values show some of these differences to be not statistically significant (GR-4, "other" guardrail; GR-9, W-beam strong post guardrail; and MB-5, weak post median barrier).

To further examine differences in barrier types, some grouping of similar barriers was done. This resulted in six groups of barriers:

Group 1—GR-1, MB-5 (Weak post guardrail and median barrier).
Group 2—GR-2 (Strong post guardrail).
Group 3—MB-6 (Strong post median barrier).
Group 4—GR-3 (Rigid guardrail).
Group 5—MB-7 (Rigid median barrier).
Group 6—GR-4, MB-8, GR-9 (Other guardrail and median barrier).

Table 4 gives results from a logistic model similar to that of Table 3, but based on the six groups of barriers. From the *p*-values shown in the top portion of the table, it can be seen that groups 2, 3, and 5 all differ significantly from group 1;

TABLE 3 Logistic Model Results Comparing Nine Barrier Types: All Hits into LON

Variable	Parameter Estimate	Standard Error	P-value
GR-2	.84	.40	.03
GR-3	1.53	.88	.08
GR-4	.57	.44	.19
GR-9	.35	.50	.48
MB-5	.19	.54	.72
MB-6	1.25	.53	.018
MB-7	1.76	.46	.0002
MB-8	.92	.61	.127
Speed	.05	.01	.0001
Curb Weight	-.05	.01	.0013
Impact Angle	.02	.007	.001

Key:

Barrier Type

GR-1 (weak post)
GR-2 (strong post)
GR-3 (rigid)
GR-4 (other)
GR-9 (W-beam strong post)
MB-5 (weak post)
MB-6 (strong post)
MB-7 (rigid)
MB-8 (other)

group 6 does not differ significantly from group 1; the significance of group 4 is marginal. The positive signs of the coefficients indicate that groups 2 to 6 are estimated to be associated with injury risks greater than that of group 1. By estimating other models with groups 2 to 6 omitted one at a time, the table of pairwise comparisons shown in the middle portion of Table 4 was generated. This table shows, for example, that group 2 differs significantly from groups 1 and 5 but not from groups 3, 4, and 6. The bottom portion of the table lists the relative risks of injury for each significant difference. Fitting models to the clean hits data file produced similar results.

Barrier Performance in Terms of Vehicle-Barrier Interaction and Postimpact Trajectory

Although the analysis so far has involved performance in terms of harm to the driver, additional analyses were conducted to examine differences in vehicle-barrier interaction and vehicle trajectory following impact. All the analyses in this section are limited to vehicle hits into barrier LON only. In many cases separate analyses are carried out for all barrier hits and clean hits only. Table 5 gives a tabulation of barrier performance by barrier type. This table shows that 75 percent of the case vehicles were redirected by the barrier, 8.5 percent snagged, 9 percent overrode the barrier, and 3 percent or less each vaulted, penetrated, or had some other involvement with the barrier. Guardrail type GR-1, weak post systems, had the highest percentage of snags (23 percent), but some snagging is expected for this more forgiving design. Type GR-4 also had a high snag percentage (18 percent) and the highest percentage of overrides (21 percent). Type GR-9, W-beam strong post, had a high percentage of overrides (16 percent) and the highest percentage vaulting the barrier (12 percent).

A tabulation of barrier performance by driver injury (injured versus not injured based on MAIS) is shown in Table 6. Injury rates vary relatively little across the performance categories as confirmed by the nonsignificant X^2-statistic. Logistic models were run to further investigate relationships between barrier performance and driver injury while taking into account impact speed, curb weight, and impact angle. In these analyses, the injury risk to drivers of vehicles that were snagged, overrode the barrier, or fell into any of the remaining three performance categories were compared with the injury risk to drivers of vehicles that were redirected by the barrier. Results from one such analysis—a model fit to data from all barrier hits—are shown in Table 7. None of the barrier performance indicator variables was statistically significant (compared with redirected), nor were there statistically significant differences between the performance estimates, as can be seen from an examination of their standard errors. In other words, for any driver injury versus no injury, categories such as snagging and overriding were not statistically different from redirection. Because of small sample sizes, the categories of vaulted, penetrated, and other were combined for the modeling. The results were similar for the clean hit data.

These injury results are not what would normally be expected. In particular, vaulting and penetrating a barrier are considered barrier failures and more likely to produce injury than redirection. Table 6 showed vaulting to produce injury 86 percent of the time, but penetration produced injury only 42 percent of the time. Thus, the combination of these two outcomes with the category of "other" may partially account for the modelling results, which indicate no difference in these failure modes and redirection.

Further exploration through three-way cross-tabulations showed a few possible coding errors by investigators, in that eight of the vehicles indicated to have either overrode, vaulted, or penetrated a barrier also were coded to have returned to the roadway or crossed to the other side. Injury to drivers of vehicles that overrode, vaulted, or penetrated was present more than 80 percent of the time when the vehicle either struck another object or rolled over.

Examination of a number of variables for each case of vaulting and penetrating a barrier also helped in understanding some of the injury results. For the vaulting cases, almost all involved W-beam barriers, with about half being the older W-beam strong post design. Many of the barrier heights were less than 711 mm (28 in.) high, and several had curbs. Almost half of the impact angles exceeded 28 degrees. Only one case had an impact speed in the 74–89 km/hr (46–55 mph) range, but four others were not coded, perhaps implying high speeds. Six of the cases involved rollover. Most of the crashes occurred on Interstate routes, with only two on county routes.

For the penetrating cases, about half involved cable guardrail, which is a forgiving system that allows large deflections. Most of the barrier heights were less than 711 mm (28 in.), and only a few curbs were present. About half of the impact

TABLE 4 Logistic Model Comparing Six Groups of Barrier Types: All Hits into LON

Variable	Estimate	Standard Error	P-Value
Group 2 (strong post GR)	.77	.34	.022
Group 3 (strong post MB)	1.18	.49	.016
Group 4 (rigid GR)	1.46	.85	.087
Group 5 (rigid (MB)	1.69	.42	.001
Group 6 (other GR and MB)	.49	.33	.138
Speed	.05	.01	.001
Curb Weight	-.05	.01	.001
Impact Angle	.02	.007	.001

Pairwise comparisons: p-values

Group	Group 2	3	4	5	6
1	.02	.02	.09	.001	ns
2		ns	ns	.02	ns
3			ns	ns	ns
4				ns	ns
5					.002

Relative risks for significant differences:

$R_2/R_1 = 2.17$ $R_5/R_2 = 2.49$
$R_3/R_1 = 3.27$
$R_4/R_1 = 4.31$ $R_5/R_6 = 3.30$
$R_5/R_1 = 5.40$

angles exceeded 28 degrees. Two of the impact speeds exceeded 97 km/hr (60 mph) and eight more were not coded, perhaps implying high speeds. Most of the cases involved no subsequent impacts, and there was only one rollover. About half of the crashes occurred on local or county routes, and only three involved large trucks.

Subsequent Impact Following First Barrier Impact

Frequencies of subsequent impacts with certain other objects are shown in Table 8 by barrier type. The data used for these analyses were the all hits data because in clean hits subsequent impacts were restricted to the same longitudinal barrier. Note that no subsequent impacts with other vehicles are shown in Table 8; these cases have been excluded from all the data files for this analysis. Approximately 9 percent of the vehicles rolled over after striking the barriers. The rollover rate for barrier type MB-7, the concrete median barrier, is nearly double this overall rate. (The rollover rate for GR-3 was also quite large, 20 percent, but was based on only 15 cases.)

Injury distributions associated with subsequent impacts are shown in Table 9, which shows rollover to have the highest rate of driver injury. However, in a logistic model that compared the injury risk of rollover with that for all other subsequent impacts (including none), the estimated rollover effect was not statistically significant ($p = .119$).

End Treatment Analyses

Tables 10 and 11 give the maximum data in the file on type of end treatment cross-classified by driver injury severity. In these tables, end treatment type and LON are determined by the barrier subset variable called "end treatment type" for the first impact. The data are not restricted to clean hits. Following some initial analyses comparing injured and uninjured drivers, it appeared that differences between types of barrier ends and ends versus LON were more pronounced in the more serious injuries than in the "any injury" versus "no injury" comparison. This can be seen in the last columns of the tables, which show percentages with A or K injuries and

TABLE 5 Vehicle-Barrier Interaction by Barrier Type: All Hits into LON

Barrier Type	Performance						
	Redirected	Snagged	Overrode	Vaulted	Penetrated	Other	Total
GR-1	34 (60.7)	13 (23.2)	4 (7.1)	0 (0.0)	2 (3.6)	3 (5.4)	56
GR-2	112 (75.7)	10 (6.8)	10 (6.8)	6 (4.1)	7 (4.7)	3 (2.0)	148
GR-3	13 (86.7)	0 (0.0)	2 (13.3)	0 (0.0)	0 (0.0)	0 (0.0)	15
GR-4	43 (50.6)	15 (17.7)	18 (21.2)	0 (0.0)	4 (4.7)	5 (5.9)	85
GR-9	29 (59.2)	5 (10.2)	8 (16.3)	6 (12.2)	1 (2.0)	0 (0.0)	49
MB-5	28 (82.4)	4 (11.8)	1 (2.9)	0 (0.0)	0 (0.0)	1 (2.9)	34
MB-6	36 (87.8)	2 (4.9)	1 (2.4)	1 (2.4)	0 (0.0)	1 (2.4)	41
MB-7	135 (91.2)	0 (0.0)	8 (5.4)	0 (0.0)	0 (0.0)	5 (3.4)	148
MB-8	21 (77.8)	2 (7.4)	2 (7.4)	1 (3.7)	1 (3.7)	0 (0.0)	27
Total	451 (74.9)	51 (8.5)	54 (9.0)	13 (2.2)	15 (2.5)	18 (3.0)	603

Key:

Barrier Type

GR-1 (weak post)
GR-2 (strong post)
GR-3 (rigid)
GR-4 (other)
GR-9 (W-beam strong post)
MB-5 (weak post)
MB-6 (strong post)
MB-7 (rigid)
MB-8 (other)

TABLE 6 Barrier Performance by Driver Injury (MAIS): All Hits into LON

Performance	Injured (MAIS ≥ 1)	Not Injured	Total
Redirected	258 (61.4)	162 (38.6)	420
Snagged	30 (61.2)	19 (38.8)	49
Overrode	31 (59.6)	21 (40.4)	52
Vaulted	12 (85.7)	2 (14.3)	14
Penetrated	5 (41.7)	7 (58.3)	12
Other	11 (61.1)	7 (38.9)	18
Total	347 (61.4)	218 (38.9)	565

X^2_4 df = 5.536 p = .354

TABLE 7 Logistic Model Results for Injury Risk as Function of Barrier Performance: All Hits into LON

Variable	Estimate	S.E.	P-Value
Snagged	.34	.37	.36
Overrode	.02	.40	.95
Vaulted, penetrated, other	-.55	.46	.24
Speed	.05	.009	.0001
Curb Weight	-.05	.01	.0005
Impact Angle	.02	.007	.0017

TABLE 8 Subsequent Impact Following First Barrier Impact by Barrier Type: All Hits into LON

	Subsequent Impact					
Barrier Type	None	Other Roadside Object	Same Barrier	Other Barrier	Rollover	Total
GR-1	35 (67.3)	8 (15.4)	6 (11.5)	3 (5.8)	0 (0.00)	52
GR-2	67 (48.2)	26 (18.7)	20 (14.4)	14 (10.1)	12 (8.6)	139
GR-3	6 (40.00)	1 (6.67)	5 (33.33)	0 (0.00)	3 (20.00)	15
GR-4	47 (55.3)	20 (23.5)	8 (9.4)	6 (7.1)	4 (4.7)	85
GR-9	25 (55.6)	9 (20.0)	4 (8.9)	3 (6.7)	4 (8.9)	45
MB-5	22 (64.7)	2 (5.9)	7 (20.6)	2 (5.9)	1 (2.9)	34
MB-6	25 (62.5)	4 (10.0)	6 (15.0)	2 (5.0)	3 (7.5)	40
MB-7	72 (49.7)	2 (1.4)	45 (31.0)	3 (2.1)	23 (15.9)	145
MB-8	16 (61.5)	2 (7.7)	6 (23.1)	1 (3.9)	1 (3.9)	26
Total	315 (54.2)	74 (12.2)	107 (18.4)	34 (5.9)	51 (8.8)	581

Key:

Barrier Type

GR-1 (weak post)
GR-2 (strong post)
GR-3 (rigid)
GR-4 (other)
GR-9 (W-beam strong post)
MB-5 (weak post)
MB-6 (strong post)
MB-7 (rigid)
MB-8 (other)

TABLE 9 Subsequent Impact by Driver Injury: All Hits

Subsequent Impact	Injured (MAIS ≥ 1)	Not Injured	Total
None	164 (55.4)	132 (44.6)	296
Other roadside object	47 (68.1)	22 (31.9)	69
Same barrier	58 (58.6)	41 (41.4)	99
Another barrier	20 (64.5)	11 (35.5)	31
Rollover	43 (87.8)	6 (12.2)	49
Total	328	215	544

$X^2_4 = 20.51$ $p = .001$

TABLE 10 End Treatment by Driver Injury (KABCO): All Barrier Hits

End Treatment Type	Injury Severity						
	No Injury	C	B	A	K	Total	Percent A or K
Length of Need	294 (50.4)	68 (11.7)	149 (25.6)	64 (11.0)	8 (1.37)	583	12.4
Blunt	60 (44.8)	18 (13.4)	31 (23.1)	22 (16.4)	3 (2.2)	134	18.7
Non-Breakaway Cable	10 (41.7)	2 (8.3)	6 (25.0)	6 (25.0)	0 (0.0)	24	25.0
Turndown	51 (47.2)	10 (9.3)	26 (24.1)	16 (14.8)	5 (4.6)	108	19.4
Breakaway Cable	14 (41.2)	2 (5.9)	10 (29.4)	8 (23.5)	0 (0.0)	34	23.5
Anchoring to Backslope	6 (46.15)	3 (23.08)	3 (23.08)	1 (7.69)	0 (0.00)	13	7.7
Attached to Parapet	5 (22.73)	3 (13.64)	9 (40.91)	5 (22.73)	0 (0.00)	22	22.7
Other	3 (37.50)	0 (0.00)	1 (12.50)	4 (50.00)	0 (0.00)	8	50.0
Total	443 (47.8)	106 (11.5)	235 (25.4)	126 (13.6)	16 (2.2)	926	15.3

TABLE 11 End Treatment Type by Driver Injury (MAIS): All Barrier Hits

End Treatment Type	Driver Injury Severity (MAIS) 0	1	2	3	4	5	6	Total	Percent MAIS≥3
Length of Need	221 (38.8)	274 (48.1)	48 (8.4)	18 (3.2)	1 (0.18)	1 (0.18)	6 (1.1)	569	4.7
Blunt	52 (40.9)	51 (40.2)	11 (8.7)	5 (3.9)	3 (2.4)	4 (3.2)	1 (0.8)	127	10.3
Non-Breakaway Cable	9 (40.9)	8 (36.4)	2 (9.1)	2 (9.1)	1 (4.6)	0 (0.0)	0 (0.0	22	13.6
Turndown	37 (36.6)	43 (42.6)	11 (10.9)	5 (5.0)	0 (0.0)	0 (0.0)	5 (5.0)	101	10.0
Breakaway Cable	9 (27.3)	15 (45.5)	5 (15.2)	3 (9.1)	1 (3.0)	0 (0.0)	0 (0.0)	33	12.1
Anchoring to Backslope	6 (46.15)	5 (38.46)	1 (7.69)	1 (7.69)	0 (0.0)	0 (0.0)	0 (0.0)	13	7.7
Attached to Parapet	5 (26.32)	10 (52.63)	4 (21.05)	0 (0.00)	0 (0.0)	0 (0.0)	0 (0.0)	19	0.0
Other	2 (25.00)	3 (37.50)	1 (12.50)	1 (12.50)	1 (12.50)	0 (0.0)	0 (0.0)	8	25.0
Total	341 (38.2)	409 (45.9)	83 (9.3)	35 (3.9)	7 (0.8)	5 (0.6)	12 (1.4)	892	6.7

percentages with MAIS ≥ 3, respectively. It can also be seen from the tables that blunt and turndown end treatments predominated. In most of the analyses that follow all the remaining end treatment types were combined into a single "other" category. This collapsing appears reasonably justified in terms of the serious injury percentages given in Tables 10 and 11.

A problem that arose in the analysis of end treatment types was that, although impact speed has been shown to be a significant factor relative to driver injury, estimated impact speed was available for only about 27 percent of the end hit cases. Neither curb weight nor impact angle was found to have a significant effect.

Table 12 shows results from a logistic model for comparing three types of end hits with hits into LON, using impact speed as a covariate. This model shows estimated risk of serious injury (MAIS ≥ 3) to be significantly greater for blunt ends and turndown ends than for LON. Estimated injury risks for the combined other end types are not significantly greater than that for LON; however, the standard errors shown in Table 12 suggest that the injury risks do not differ significantly across the three end types either.

Comparisons of the three end types are further explored in Tables 13 and 14, which show injury severity classified by the three end types for two more-restrictive types of end hits. In Table 13 only upstream, end-on hits (from the all hits file) are considered. Upstream hits within 25 ft of the end are tabulated in Table 14. In neither case are there significant differences between end types.

Further Analysis of Rollovers

Vehicle rollovers associated with barrier impacts were studied further by examining associations between rollovers and driver injury severity and between rollovers and barrier end hits versus LON. In particular, these analyses address the question of whether the higher injury risk associated with end hits is primarily a result of more rollovers associated with end hits. The analyses that follow were based on clean hits data. Table

TABLE 12 Logistic Model Results for Comparing Three End Types Versus LON Relative to Risk of Injury at MAIS ≥ 3: All Hits

Variable	Estimate	Standard Error	p-value
Blunt ends	1.78	.71	.012
Turndown ends	1.42	.70	.041
Other end types	.96	.71	.174
Speed	.03	.02	.027

TABLE 13 Comparison of End Types for Upstream Hits, End On

End Type	MAIS ≥3	<3	Total
Blunt	7 (11.9)	52 (88.1)	59
Turndown	4 (13.8)	25 (86.2)	29
All other	5 (16.7)	25 (83.3)	30
Total	16	102	118

$X^2_2 = .393$ $p = .822$

TABLE 14 Comparison of End Types for Upstream Hits, End On to 25 ft

End Type	MAIS ≥3	<3	Total
Blunt	9 (11.1)	72 (88.9)	81
Turndown	7 (10.6)	59 (89.4)	66
All other	7 (13.5)	45 (86.5)	52
Total	23	176	199

$X^2_2 = .259$ $p = .879$

TABLE 15 Rollover Versus Driver Injury: Clean Hits Only

	Percent Injured		
Rollover Status	MAIS ≥ 1	MAIS ≥ 3	A+K
No Rollover	59.4%	5.8%	13.5%
Rollover	84.7%	15.2%	30.2%
X^2_1d.f.	20.9	11.2	18.6
p-value	.000	.001	.000

TABLE 16 Three-Way Table of Rollover by End Versus LON by MAIS Injury Severity

		Injury Severity		
Rollover	End/LON	MAIS < 3	(MAIS ≥ 3)	Total
Yes	LON	42 (85.7)	7 (14.3)	49
	End	30 (83.3)	6 (16.7)	36
	Total	72	13	85
		$\chi^2_1 = .051$	p = .763	
No	LON	498 (96.5)	18 (3.5)	516
	End	254 (90.4)	27 (9.6)	281
	Total	752	45	797
		$\chi^2_1 = 12.791$	p = .000	

*In this table any hit in which an end type was coded was considered an end hit; length-of-need was "no" end hit.

15 shows results from contingency tables of rollover versus the three characterizations of driver injury used in previous analyses. All three characterizations show significantly higher injury or serious injury rates associated with rollovers.

Comparisons of rollover rates for hits into LON with rollover rates for end hits yielded the following rates:

- 8.46 for LON,
- 13.62 for all end hits,
- 17.16 for upstream, end-on hits, and
- 17.43 for upstream hits within 25 ft of barrier end.

All three end rates differed significantly from the LON rate with $p < .02$ in each comparison. Tables 16 and 17 show three-way breakdowns of rollover by end and LON by injury severity (MAIS and KABCO, respectively), in which end hits refer to any end hit. These tables show no significant differences in injury rates between ends and LON when rollovers

TABLE 17 Three-Way Table of Rollover by End Versus LON by KABCO Injury Severity

		Injury Severity		
Rollover	End/LON	O,C,B	A,K	Total
	LON	34 (68.0)	16 (32.0)	50
Yes	End	33 (71.7)	13 (28.3)	46
	Total	67	29	96
		χ^2_1 = .159	p = .690	
	LON	474 (89.3)	57 (10.7)	531
No	End	235 (81.3)	54 (18.7)	289
	Total	709	111	820
		χ^2_1 = 10.107	p = .001	

occurred, but when no rollovers occurred significantly higher injury rates were associated with end hits. Similar tables were analyzed for comparing LON hits with upstream end-on hits and upstream hits within 25 ft of barrier end. The results were the same; no significant differences when rollovers occurred, and higher injury rates associated with end hits when rollovers did not occur. Thus, it seems that end hits are more likely to result in serious injury than LON hits by being more likely to produce rollover and by producing more serious injuries when no rollovers occur.

SUMMARY

The following is a brief summary of these analyses:

- Weak post barriers were less associated with driver injury (MAIS ≥ 1) than other barrier types.
- In regard to subsequent impact, rollover produced the highest rate of driver injury (MAIS ≥ 1).
- Higher risks of serious driver injury (A + K, MAIS ≥ 3) were associated with blunt and turndown end treatments than with LON.
- Rollover was associated with both higher driver injury (MAIS ≥ 1) and serious injury (A + K) rates.
- End hits are more likely to result in serious driver injury than LON by being more likely to produce rollover and by producing more serious injuries when no rollovers occur.

These descriptive statistical analyses and modeling results have produced a wealth of information. Some of the findings are provocative and in need of further clinical exploration. It is recommended that this be done in subsequent research, primarily because design engineers could learn a great deal from seeing the outcomes of various kinds of real-world barrier impacts. Although the LBSS file is certainly not free of error, it remains a comprehensive information source.

REFERENCES

1. Mak, K. K., and A. Magaro. *National Accident Sampling System (NASS) Longitudinal Barrier Special Study Coding/Editing and Field Procedures Manual.* Southwest Research Institute (prepared for National Highway Traffic Safety Administration), San Antonio, Tex., 1984.
2. Ray, M. H., J. D. Michie, and M. Hargrave. Events That Produce Occupant Injury in Longitudinal Barrier Accidents. In *Transportation Research Record 1065*, TRB, National Research Council, Washington, D.C., 1986, pp. 70–75.
3. Ray, M. H., J. D. Michie, W. W. Hunter, and J. C. Stutts. An Analysis of the Risk of Occupant Injury in Second Collisions. In *Transportation Research Record 1133*, TRB, National Research Council, Washington, D.C., 1987, pp. 17–22.
4. Bryden, J. E., and J. S. Fortuniewicz. Traffic Barrier Performance Related to Vehicle Size and Type. In *Transportation Research Record 1065*, TRB, National Research Council, Washington, D.C., 1986, pp. 69–78.
5. Mak, K. K., D. L. Sicking, and H. E. Ross, Jr. Real World Impact Conditions for Run-Off-the-Road Accidents. In *Transportation Research Record 1065*, TRB, National Research Council, Washington, D.C., 1986, pp. 45–55.
6. Troxel, L. A., M. H. Ray, and J. F. Carney III. Side Impact Collisions with Roadside Obstacles. In *Transportation Research Record 1302*, TRB, National Research Council, Washington, D.C., 1991, pp. 32–42.
7. Pigman, J. G., and K. R. Agent. Performance Evaluation of Breakaway-Cable-Terminal End Treatments. In *Transportation Research Record 1198*, TRB, National Research Council, Washington, D.C., 1988, pp. 1–10.
8. Perera, H. S., and H. E. Ross, Jr. Prediction of Rollovers Caused by Concrete Safety-Shape Barriers. In *Transportation Research Record 1233*, TRB, National Research Council, Washington, D.C., 1989, pp. 124–131.
9. Mak, K. K., and D. L. Sicking. Rollover Caused by Concrete Safety-Shaped Barrier. In *Transportation Research Record 1258*, TRB, National Research Council, Washington, D.C., 1990, pp. 71–81.
10. *SAS User's Guide, Version 6.* SAS Institute, Inc., Cary, N.C., 1990.

Publication of this paper sponsored by Committee on Roadside Safety Features.

Attempt To Define Relationship Between Forces To Crash-Test Vehicles and Occupant Injury in Similar Real-World Crashes

FORREST M. COUNCIL AND J. RICHARD STEWART

Roadside safety devices are designed to protect vehicle occupants from injuries. Because new designs cannot be tested with human occupants, the safety of new designs has traditionally been measured in crash tests, with criteria for success being structural adequacy of the device, vehicle trajectory after collision, and occupant risk. The relationship between occupant risk as measured in crash tests and the ultimate measure of occupant risk—driver injury—is explored. Vehicles from the 1973 through 1986 North Carolina crash files were matched with similar crash-test vehicles on the basis of feature struck; make, model, and year; and Traffic Accident Data Project impact location and severity. Contingency table analysis and logistic regression modeling were used to explore the potential relationship between crash-test measures and injury. Results indicated the lack of a strong relationship between driver injury and peak 50-msec longitudinal and lateral forces to the vehicle or momentum change. With respect to the newer proposed "flail space" measures of occupant risk, the limited amount of data available made conclusions virtually impossible to draw. Because of the continuing need for a strong link between crash-test measures and injury, recommendations for modification of the methodology, the data files, the test matrix, and the measures themselves are provided.

Roadside safety features protect occupants of errant vehicle from serious injury (e.g., guardrail shielding steep sideslopes or bridge piers and bridge rail shielding waterways) or enhance the driving or living environment (e.g., signs, luminaire supports, utility poles). This hardware is designed to "either gently stop the vehicle, or readily break away, without subjecting occupants to major injury producing forces" (*1*).

In the best of all hardware-development environments, alternative designs for each safety device would be tested in the real world, with changes in injury patterns to vehicle occupants being the major criteria of success. However, because of cost reasons related to available sample sizes, limitations in police-reported accident data bases related to details on the specific design of a roadside feature, and moral and legal reasons requiring that designs be tested before use, such a research and development scenario is not possible.

Instead, research conducted by FHWA and the states related to roadside safety device performance has involved crash-testing of vehicles striking a given device. In such testing, which is designed to represent practical worst-case scenarios, evaluation criteria used include measures of (a) structural adequacy of the device (e.g., whether the vehicle penetrated the guardrail), (b) vehicle trajectory after collision (e.g., whether the vehicle rebounds into lanes of traffic), and (c) occupant risk.

Perhaps the most important criterion, and the one with which the most problems arise in measurement, is occupant risk. Ultimate occupant risk is measured by occupant injury distributions. However, because human subjects cannot be put in crash-test vehicles, the criterion traditionally used is a measure of *g*-forces (accelerations) acting on the vehicle or a crash dummy, some measure of change in speed of the vehicle upon striking an obstacle (delta-*v*), or some measure of momentum change. Detailed crash-test procedures, measurements, and acceptance guidelines in terms of "preferred" and "acceptable" vehicle accelerations (e.g., the peak 50-msec average acceleration measured near the vehicle center of mass, or the maximum allowable momentum change) have been established for many years (*2–4*).

Unfortunately, although the test conditions, the measurement conditions, and the acceptance guidelines are specified in great detail, the same publications that document the guidelines note that "These criteria are not valid, however, for use in predicting occupant injury in real or hypothetical accidents" (*4*). Although logical attempts to link these vehicle accelerations with occupant injury have been made [e.g., the TTI "severity index" (*5*)], they are not suitable for "direct assessment of human injury" (*6*). Clearly, at that time, the linkage between crash-test and occupant injuries was tenuous at best.

Supporting criticisms of these measures in an earlier work (*7*), Michie (*8*) hypothesized that, although these measures might indicate in some overall manner the degree of occupant risk, they are inconsistent, inadequate, and overly conservative. Michie further noted that momentum change would be expected to be the best indicator, average vehicle accelerations in crash-cushion impacts second best, and maximum 50-msec *g*-forces the least adequate.

Michie then attempted to define a better linkage between the crash test and occupant injury—the "flail-space" model. Here, the occupant is considered to be a free body that travels across the compartment space after the vehicle has struck an object and begun to decelerate. The occupant, continuing to travel at the original impact speed, then contacts a part of the vehicle interior (e.g., windshield, dashboard), which has

University of North Carolina, Highway Safety Research Center, Chapel Hill, N.C. 27599-3430.

decelerated to a lower speed. The difference between the occupant speed and the vehicle interior speed (referred to here as delta-v, or change in velocity for the occupant) is the first critical measure in the flail space model and is referred to as longitudinal or lateral "occupant risk." Following the impact, the vehicle and occupant continue to experience deceleration as a unit, and the second critical measure of acceleration to the occupant is considered to be the highest average vehicle acceleration over any subsequent 10-msec interval, referred to as longitudinal or lateral "occupant ridedown."

A pass-fail judgment of the barrier is then based on comparison of these two measures with critical levels of accelerations. These critical levels were based by Michie on what are considered to be acceptable impact velocities and g-forces from earlier studies related to dummy head impacts into windshields and car interiors (*9*), on limited accident analysis related to reconstruction of forces in side impacts (*10*), and on a variety of past attempts to determine the peak survival acceleration forces in other laboratory experiments (*11,12*).

Although Michie's hypotheses concerning deficiencies in the traditional measures and his flail-space measure of occupant risk are based primarily on vehicle dynamics, virtually no accident-based verification of his criticisms of the three original models or of the efficacy of his newer model has been conducted. Thus, while the relationship between forces to crash-test vehicles and ultimate occupant injury in real-world crashes is both logical and based in the physics of crashes, the specific nature of the relationship remains somewhat undefined. Such definition is important for several reasons.

- The measure used must reflect ultimate occupant injury, because decisions concerning which designs to approve for use are based on the measure used. Criteria not reflecting injury would lead to erroneous acceptance/rejection decisions.
- Because initial hardware design and subsequent design changes are based on differences in the measured crash test criteria (i.e., a design resulting in a lower measured criterion is considered more desirable than one resulting in a higher measured criteron), the lack of a relationship with occupant injury would mean that meaningless (or erroneous) design changes would be made—costs would be incurred without additional benefit.
- If differences in crash-test measures can be shown to reflect differences in occupant injury, they could be used with accident data to define better severity indexes for roadside features. Such severity indexes are used in economic analyses that help guide the design of the roadside—the use of safety features to shield roadside hazards. If the crash test measures are sensitive to changes in ultimate injury, the indexes based on accident analyses could at least be fine tuned or modified for specific feature types based on the crash test results. For example, although accident data might provide a general severity index for guardrail on the shoulder, the crash test results might allow modification of the general index to define specific indexes for different types of guardrail (e.g., W-beam versus thrie beam versus cable). Such refinements may never be possible using accident data alone because of sample size problems.

The analyses reported in this paper attempted to quantify this relationship better by linking mass accident data to the crash-test information. We attempted to examine Michie's hypothesized criticisms and, to the extent possible, his flail-space solution. Because of data and methodology problems, we were not very successful. However, because of the continuing importance of defining such a relationship, the details of the methodology used have been included for the benefit of other researchers considering such an approach. In addition, suggestions of future analytical efforts and methods that might prove to be more successful were provided.

METHODOLOGY

To study the relationship of crash-test measures to injury, a sample is needed of vehicles in crashes that have both the crash-test measure and injury specified. Such a sample does not exist because we cannot place human drivers in crash-test vehicles and cannot measure true acceleration forces in real-world crashes. Because the methodological goal was to use injury data from the real world, a sample had to be constructed in which real-world crashes that include injury data were supplemented (appended) with measured acceleration forces from similar situations in crash tests. To ensure a close match, as many vehicle, crash type, and severity variables as possible were matched and then controlled for other possible injury-related factors in the modeling.

On the basis of this goal, the unit of analysis in this study is a vehicle that was involved in a crash in North Carolina between 1973 and 1986. The sample of vehicles in the analysis file had to closely match those vehicles in crash tests. The match is based on (a) vehicle factors (i.e., make, model, and year), (b) crash specifics (i.e., type of object struck and location of impact on vehicle), and (c) crash severity (as measured by the Traffic Accident Data (TAD) project vehicle deformation scale). The crash-test data are then appended to the matching-vehicle record.

Data

The crash-test data used in this effort were extracted from recently computerized summary sheets found in FHWA's Roadside Safety Library. Information on crash tests conducted since the early 1970s included (a) the type of test and type of feature tested; (b) make, model, year, and weight of the test vehicle; (c) test specifics including impact angle, impact speed, and location of impact on the feature and on the test vehicle; and (d) results of the test. This final subset includes information on vehicle damage measured by the TAD and Vehicle Deformation Index (VDI) scales, the behavior of the vehicle, and information related to 50 msec average longitudinal and lateral accelerations, delta-v and momentum change, and occupant "flail space" measures as found in a work by Michie (*8*).

The complete file received contained some information on almost 1,000 cases, but the number of usable cases was much smaller because of incomplete summary sheets. Using the requirement that the crash-test case had to have usable in-

formation on the feature struck, car make and model, TAD level, and at least one of the pertinent crash forces (i.e., either lateral or longitudinal acceleration or momentum change), 223 usable crash tests were available for analysis—36 for guardrails, 20 for median barriers, 57 for bridge rails, and 8 for sign supports. The remainder were for features that could not be linked with the North Carolina crash file either because a limited number of matches were found (e.g., guardrail-end treatments and crash cushions) or because the objects crash tested were not similar to the objects in the real-world crashes (e.g., utility poles and mailboxes, which are considered "breakaway" in the crash tests). A number of cases that required editing were found. Although case-by-case quality control checks were not possible, repeated use of the variables in the matching and analysis efforts resulted in corrections of problems when found and lead one to believe that the data used in the final analysis files are relatively accurate.

The nature of crash testing is such that the tests do not include a random sample of the entire range of vehicles or crash situations that are found in real-world crashes. Indeed, as prescribed under NCHRP 230 and earlier guidelines, the tests are designed to represent practical worst-case situations—the upper end of the crash distributions. Thus, the testing protocols result in numerous crash tests involving the same vehicle striking the same longitudinal barrier at very similar speeds and angles, and resulting in the same TAD rating. In these cases, accelerations, changes in momentum, and measurements of delta v were averaged for all crash tests in which the vehicle, the barrier struck, and the TAD were the same. (There may have been differences in the impact angle and impact speed, but because neither of these variables is obtainable from the real-world crash data, they were not used to differentiate one crash test from another.) The final number of independent crash-test records used in matching, after similar cases were averaged for the hardware classes ultimately analyzed, is given in Table 1. A total of 75 independent data points resulted.

The real-world crash information used in this study was extracted from annual files of North Carolina crash data, files that are considered relatively accurate because of statewide uniformity of report forms and enhancements through repeated use in research efforts. The annual statewide files contain records of 150,000–200,000 accidents and 300,000–370,000 vehicles per year. Because the crash tests studied included vehicles with model years in the early 1970s, annual accident data tapes from 1973 to 1986 were used (with the exception of 1979 where the data tape was unusable), which provided a sample of approximately 4,000,000 vehicles against which to match the crash-test vehicles. Files from later years were not used because of inaccuracies in reported occupant restraint use after passage of a mandatory belt law in 1987.

File Linkage

The key variables on which the two sets of data were to be matched included the specific make and model of the vehicle involved, the type of feature struck in the crash test, the TAD-related location and severity of vehicle deformation, and the estimated original speed before accident as a surrogate for impact speed. A primary goal was to maximize the sample size, a goal that also defined the nature of the matching parameters.

With respect to make, model, and year, both exact matches between the two files and matches with "clones" of the crash-test vehicle were allowed. Clones are vehicles of very similar design (e.g., a 1974 Oldsmobile 98 could be matched with a 1971–1976 Oldsmobile 98 and a 1971–1976 Buick Electra). Clones were defined in discussions with representatives from automobile manufacturers and researchers at the National Highway Traffic Safety Administration who have done similar work using clones, a review of published material from the Insurance Institute for Highway Safety, and a detailed review of wheelbase and gross vehicle weight information found in the *Branham Automobile Reference Book* (*13*).

With respect to type of feature struck, vehicles in the North Carolina crash file were retained as case vehicle if

1. They were involved in single-vehicle accidents in which the "first harmful" and "most harmful event" was either ran-off-road or collision with a fixed object. This eliminated cases in which driver injury might have resulted from crashes with other vehicles instead of hardware.
2. The "object struck" code matched the crash-test object by indicating a "guardrail face," "median barrier face," "bridge rail," or "sign support."

TABLE 1 Sample Sizes of Usable Crash Test Cases, Cases After Averaging Similar Tests, and Matched Vehicle Records in Final Analysis File

Feature	Original Crash Tests	No. Crash Tests After Averaging	Matched Vehicle Records
Guardrail	36	23	53
Median Barrier	20	16	5
Bridge Rail	57	28	122
Sign Support	11	8	52
Total	223	75	232

For the third linkage variable, impact severity, the measure used was the TAD vehicle deformation scale (*14*), which is provided by both the investigating police officer and the crash-test engineer. Preliminary research has shown that the deformation rating can be made reliably by police officers and is superior to the officers' estimate of speed in terms of control for crash severity (*15,16*). When this information was missing or obviously erroneous in the crash-test file, a second deformation scale, the VDI was used to edit erroneous TADs and to generate missing ones. Because early file-matching runs indicated that matches with "exact" TADs led to very small samples, a decision was made to accept a match if the location of the impact was in the same area and if the TAD severity rating was within plus or minus one level of the crash-test severity value. In the final analysis file, approximately 50 percent of the vehicles had exact TAD matches.

The 75 crash-test data points referred to earlier were then matched with the 1973–1986 North Carolina crash file. The expansion of both the vehicle make and model year to clones and the TADs to plus or minus one severity level resulted in a file of 232 vehicle records suitable for analysis. These restrictions resulted in the final sample sizes used in the modeling as shown in Table 1.

Analysis Methodology

The basic methodology used to analyze the matched data involved the development and analysis of a series of contingency tables (cross-tabulations) to define potential relationships between injury and crash-test measures, and the development of a series of predictive models based on logistic regression. The outcome variable in these models was some measure of driver injury, and the independent variables included the various crash-test measures and vehicle and crash descriptors. An attempt was made to develop such a model for each feature type tested—guardrails, median barriers, bridge rails, and sign supports.

[A preliminary modeling effort examined the question of whether TAD—one of the primary matching variables—was related to crash forces in the crash tests. By using general linear regression models, various measures of crash-test forces (longitudinal acceleration, lateral acceleration, and resultant acceleration) were predicted as a function of TAD severity (coded 1 to 7), car size (grouped into small, median, and large car groups), impact speed, and impact angle. Unlike later modeling in which the unit of analysis is a vehicle in a real-world crash, the unit of analysis here is a vehicle in a crash test. The results indicated that when TAD, impact speed, and impact angle were included in the same model as predictors of crash forces in guardrail impacts, TAD was not a strong predictor. When impact speed and angle were excluded from the model, TAD was a significant predictor, meaning that it was acting as a proxy for impact angle and speed. TAD was a strong predictor for median barriers (along with car size) and for bridge rails (along with impact speed). The results of this modeling did not dissuade us from using TAD as a matching variable but did point out the need to attempt to include some measure of vehicle speed in the subsequent injury models.]

RESULTS FOR TRADITIONAL MEASURES OF VEHICLE FORCES

The contingency table analysis pointed out a very serious limitation of this investigation—the extent of data available for analysis, in terms of both total sample size of matched vehicles where complete crash-test data existed (as shown in Table 1) and in the range of variation in many of the relevant variables. For example, for the already limited sample of 53 matched guardrail-related vehicles, examination of the resulting distribution of R-force values indicated that, although the values of R-force (resultant force) ranged from 3.47 to 19.02 *g*, 33 of the 53 (62 percent) had the same value of 5.98. This was a result of the fact that some of these matches were groups of similar vehicles in the North Carolina crash file that matched the same crash-test vehicle (or groups of similar crash-test vehicles for which data were averaged). Further analysis indicates that the 52 matches (vehicles in the real world) were the result of only 17 crash tests (of the total of 36) and only 12 distinct data points (after averaging similar tests). The resulting problem, of course, is that the sample available for analysis is small, in terms of number of cases, and has only limited variability, in terms of crash forces assigned to vehicles. Similar problems were noted with the median barrier sample. Bridge rails were less problematic in that a larger, more variable sample was available for analysis. For all longitudinal barriers combined, while nearly 179 total observations (vehicles in crashes) were available, more than half were concentrated at just a few specific R-force values.

Given these limitations, the first phase of the contingency table analysis involved examination of the distributions of driver injury corresponding to specific R-force values when all matched vehicles that had struck guardrails, median barriers, and bridge rails were combined, and when 10 or more accident observations were found for the same R-force level. (Note that this and all later combinations of the data continued to retain the integrity of the original match, in that each record represented a vehicle that had already been matched to a specific crash-test object, and was assigned the crash-test forces for that object. Thus, a vehicle striking a guardrail in the real world was not assigned force values from median barrier crash tests. Only entire matched records were combined for analysis.) As in all subsequent analyses, driver injury is defined by the KABCO scale, with K indicating "killed," and A, B, and C denoting progressively less severe injury.

The analyses consistently indicated a wide range of driver injuries corresponding to specific values of R-force. There was no apparent trend of more severe injuries corresponding to higher R-force values. Neither of these findings indicated a strong relationship between injury and R-force.

Momentum change values (ΔMV) were assigned to 52 crashes into sign supports; none were available for luminaire crashes. The distribution of these values ranged from 1,834 to 10,511 N/sec (412 to 2,362 lbf/sec). Although the spread of values for the matched vehicles was somewhat better than for the *g*-force measures, again, 25 of the 52 cases were matched to only two different crash-test momentum change values. All of the 11 crash-test cases with usable data available for analysis were matched, but they generated only eight different data points.

A cross-tabulation of driver injury by the highest frequency momentum changes was generated and examined. Although no strong relationship between injury and ΔMV was indicated, there was perhaps a hint of injury severity increasing with increasing values of ΔMV, at least at the higher injury categories.

In the comparisons thus far, other variables such as car size have not been controlled for. To further explore relationships between *g*-forces and driver injury, the final analyses involved fitting a series of logistic regression models to the data. The logistic regression model fits a linear function of the independent variables to the quantities

$$\text{lot}\ \frac{\text{proportion injured at specified level}}{\text{proportion with lesser or no injury}}$$

by the method of maximum likelihood. Logistic models are particularly useful when both continuous and categorical dependent variables are being studied, and where some of the groups may have a zero value.

For each of these models, driver injury was characterized as a dichotomous (grouped) variable ranging from serious (K or A) injury versus lesser injury, to any injury (K, A, B, or C) versus no injury. In models involving crashes into any of the longitudinal barriers, either R-force or a variable labeled "test" were included as independent variables, where "test" was defined as follows:

Test = 1 if lateral acceleration, longitudinal acceleration, and R-force are all in the preferred range (i.e., lateral ≤ 3 *g*, longitudinal ≤ 5 *g*, R-force ≤ 6 *g*).

= 2 if not all in preferred range but all in acceptable range (i.e., lateral ≤ 5 *g*, longitudinal ≤ 10 *g*, R-force ≤ 12 *g*).

= 3 if at least one *g*-force beyond acceptable range.

The definitions of "preferred" and "acceptable" range were taken from Transportation Research Circular 191. Other independent variables included car size [large = 1, where vehicle weight is greater than 1816 kg (4,000 lb); medium = 2, where vehicle weight is between 908 and 1816 kg (2,000 and 4,000 lb); small = 3, where vehicle weight is less than 908 kg (2,000 lb)], seat belt use (yes = 0, no = 1), and the police estimate of original speed before accident. (The police estimate of "impact speed" is uncoded in many cases and is of questionable accuracy.) For vehicles crashing into sign supports, ΔMV was the force-related independent variable. Preliminary analyses indicated that "driver age," usually an important variable in injury prediction, was not significantly related to injury in this data set. Thus, it was not included in the models.)

Two sets of analyses of longitudinal barriers were carried out, the first set with no restrictions on estimated vehicle speed. Because an early reviewer noted that TAD alone may not be an entirely adequate matching variable (given that the same TAD may result from two different changes in velocity if the initial speeds differ), an attempt was made to provide some control for the speed of the crash-involved vehicle. Unfortunately, there is no adequate measure of "impact speed" provided in a police investigation. Instead, the only "surrogate" available—the police estimate of "original speed prior to accident—had to be used." Since all crash-test vehicles in the final matching file had crash-test impact speeds of approximately 89–105 km/hr (55–65 mph), the analysis file was restricted to include only those vehicles with estimated original speeds greater than 64.4 km/hr (40 mph). (Further restriction would have reduced the sample to an unusable size.) Approximately 81 percent of these vehicles had estimated original speeds between 72 and 113 km/hr (45 and 70 mph).

The results of these speed-restricted analyses are given in Table 2. Here, the column labeled "Injury Level" defines one of the two contrast injury groups used in the model. Thus, in the first model, the proportion of drivers with K or A injury was compared with the proportion with less severe injuries (i.e., B, C, or no injury). As indicated in the column referring to feature type, in most models all three types (guardrails, median barriers, and bridge rails) were combined.

The R-force values are significant predictors of injury in four of the nine final models developed. More specifically, R-force values are significant predictors for contrasts involving any injury versus no injury for all barriers combined (Models 3 and 4) but are not significant predictors for contrasts involving more serious injury, where one might have expected to see the greatest effect. The categorical measure of force ("test") is a significant predictor for contrasts of fatal plus serious plus moderate injuries (K, A, B) versus other injuries (Models 5 and 7). Vehicle speed, even within this restricted subsample, appears to be a fairly consistent predictor of injury. It may also be noted that the estimated car size effect, although statistically significant in Models 3 and 4, is contrary to intuition in that higher injury rates are associated with large cars relative to smaller cars. This is most likely because the large cars are tested at higher impact angles than are the small cars under existing testing procedures.

In logistic regression, there is no measure of "fit" of the model comparable to R^2 in general linear regression. Instead, the model is used to classify each case into one of the two injury categories (e.g., injury versus no injury), and the results of the predicted classification are then compared with the true injury/no injury class for each case. Here, for example, such verification of Model 4 (chosen since both R-force and speed are significant predictors of total injury at the $p < .08$ level) indicates that the predictive capability is low. The model correctly classifies 77 percent of the no-injury cases, but it correctly classifies only 45 percent of the injury cases.

Additional models involving vehicles crashing into sign supports were developed, but detailed results are not presented here. Briefly, the variable related to momentum change was a statistically significant predictor of moderate or more severe injury, but the strength of the model was quite low.

In summary, the analysis of the relationship between the traditional crash-test measures—longitudinal and lateral 50-msec *g*-forces and momentum change—and driver injury indicated some relationship in only four of nine final models examined. The predictive strengths of the models were quite low. This lack of relationship could have been partially a function of available sample sizes and of the use of the "expanded TAD" matches.

TABLE 2 Logistic Regression Results for Models Involving Longitudinal Barriers

Feature[a] Type	Injury Level	Parameter Estimate (Standard Error) R-Force	Test	Belts	Car Size	Speed
G,MB,BR	K or A	.06 (.07)	--	-.44 (.75)	.07 (.31)	.06* (.02)
G,MB,BR	K,A,B	.08 (.05)	--	.40 (.59)	.04 (.22)	.04* (.02)
G,MB,BR	K,A,B,C	.12* (.06)	--	.56 (.55)	-.50* (.20)	.03 (.02)
G,MB,BR	K,A,B,C	.12* (.06)	--	--	-.43* (.18)	.04 (.02)
G,MB,BR	K,A,B	--	.61* (.29)	--	-.30 (.26)	.04* (.02)
G,MB,BR	K,A,B	--	.52 (.29)	--	--	.04* (.02)
B,MB,BR	K,A,B	--	.62* (.29)	--	--	--
MB,BR	K,A,B,C	.12 (.07)	--	--	-.41 (.35)	.02 (.03)
MB,BR	K,A,B,C	.09 (.05)	--	--	--	--

[a] G = Guardrail, MB = Median Barrier, BR = Bridge Rail
* Significant at p<.05

RESULTS FOR OCCUPANT RISK AND RIDEDOWN MEASURES

The lack of a strong relationship between the traditional crash-test measures and injury was hypothesized earlier by Michie (*8*) and others. As a result, Michie developed alternative "flail space" measures to be used in crash tests—occupant "risk" and "ridedown."

Unfortunately, even though these measures have been used since 1981, the computerized data set available is very limited. Indeed, there were only 39 usable guardrail tests, 34 median barrier tests, 3 bridge-rail tests, and no matchable tests for sign supports. After similar tests were combined, there were even fewer unique measures, ranging from 2 for bridge rails to 24 for median barriers.

Even though limited, the longitudinal barrier tests were linked with crashes in the North Carolina crash file, using only those cases in which the most harmful event was related to strike a fixed object, and then restricting those by estimated initial speed. With the speed restriction in place, the linkage resulted in 34 matched vehicles striking guardrails, 24 striking median barriers, and 4 striking bridge rails.

It is obvious that these samples of available vehicles were quite small. Again, as with the earlier *g*-force data, these samples were also restricted in terms of the variability of the recorded risk and ridedown measures. For example, using the data restricted by both harm and speed, of the 34 cases in which a measure of longitudinal risk was available for crashes with guardrails, 21 (62 percent) had the same value (22.2). Of the 35 guardrail cases in which there was a measure of lateral risk, 21 (60 percent) had the same value (14.4). Although the measures of longitudinal and lateral ridedown were less clustered, there were only 21 guardrail cases in which any data existed in the matched analysis file when restricting only on harmful event, and only 15 when further restricting on speed.

Attempts were made to analyze these data through the development of contingency tables of each flail-space measure versus injury and the development of a limited number of models, but the results were relatively meaningless. This was not unexpected. Because of the small sample sizes and the lack of variability in the data, a relatively strong relationship between flail space and injury would have had to exist to be apparent. In addition, one other factor of note operated against finding such a relationship in these limited data—the use of driver injury as the outcome variable.

Driver injury was the outcome variable of choice throughout all the analyses, primarily because it is a constant measure

in that a driver is always present and because roadside hardware must be designed to protect the occupant most likely to be in the vehicle—the driver. However, flail space is based on an occupant compartment distance through which the occupant can travel before striking part of the interior during the deceleration. Not only does the distance (and thus the impact speed differential) for the driver and the right front passenger differ—0.305 m (1 ft) forward for the driver (to the steering wheel) versus 0.610 m (2 ft) for the passenger (to the dashboard)—but the nature of which occupant measure is critical in a crash test depends on the impact point on the vehicle. For left front or side impacts (e.g., into median barriers), the driver measure would be the most important. For right front or side impacts (e.g., into shoulder guardrails), the passenger measure would be critical. Thus, flail-space measures on the crash tests are based on what is considered the most critical occupant. (This is somewhat problematic in a philosophical sense in that there appears to be a need to define some measure of risk for the driver in right-side impacts, particularly given that a driver is always present, but a passenger only sometimes present.) In the data analyzed here, further division into driver and passenger injury or right- versus left-side impacts for matching purposes would even more severely restrict the available insufficient sample size.

In short, because of the small sample size and the need to further restrict to specific occupants, no conclusions can be drawn concerning the relationship between flail-space measures and injury. Clearly, many more data are needed before the ability of these measures to correctly predict subsequent driver injury can be assessed appropriately.

DISCUSSION OF RESULTS

There are strong reasons for attempting to better examine and define the relationship between crash-test measures and occupant injury. These reasons are related both to the need to be certain that the measures being used are resulting in the correct acceptance or rejection decisions and design modifications for roadside features and the need to expand the use of these relatively expensive crash-test data to other uses, such as the development and refinement of severity indexes. This attempt to use existing computerized crash-test data and North Carolina accident files to examine occupant risk measures and driver injury was not successful in defining such a relationship.

To some extent, these analyses appear to support the crash-test community's earlier reservations about use of the traditional measures. Although the lack of an indicated relationship could have resulted partially from the small sample sizes and the use of the expanded TAD match (which would lead to more variability in the injury/measure match), it is also likely to have resulted from the hypothesized underlying weak relationship. Unfortunately, these efforts to determine whether the proposed improved measures—the flail-space measures—are indeed superior in terms of predicting true occupant risk were not successful. Part of the problem stemmed from the methodology and part from the nature of the files used.

With respect to the files, the major problems include the following:

- In FHWA's computerized Roadside Safety Library, there appear to be significant amounts of missing data, particularly data related to the flail-space measures.
- By design, the nature of the test matrix severely limits the variability in the impacts tested, in terms of both vehicles and speeds, and this greatly restricts matches with vehicles in accidents and the range of measures (both traditional and flail-space) that can be analyzed.
- In the North Carolina accident file and those from other states, there is no good measure of impact velocity or impact angle, and the KABCO injury scale does not adequately subdivide serious injuries.

With respect to the flail-space measure itself

- There is a need for a method to combine the risk and ridedown measures to predict occupant risk. In this work, an attempt was made to analyze each of the four flail-space measures (lateral and longitudinal risk and lateral and longitudinal ridedown) separately. In contrast, for peak vehicle accelerations, a combination of lateral and longitudinal *g*s were combined to form a resultant force vector for study. If the flail-space measures more accurately capture the results of the complex movements of an occupant within an impacting vehicle, perhaps some combination would be even better predictors than the individual measures.
- From the point of view of testing for (and thus protecting) the vehicle occupant most often exposed to impact—the driver—and a parallel need to increase the available sample size of cases that can be studied in these correlational efforts, there is a need to define a flail-space measure for the driver in off-side impacts.

With respect to the methodology explored—the linking of crash-test and real world data—the major problem was in the limitation on the preciseness of the real-world and crash-test match. The lack of impact speed estimates in the North Carolina files and the lack of variability in vehicles and crash speeds in the crash-test file led to the inability to match on speed or angle and the need to use an expanded rather than an exact TAD severity match. Clearly, both these problems erode the analytical ability to detect a relatively weak relationship and should be corrected in future research. Additional crash-test cases, which include angles, speeds, and vehicles not now in the computerized file, are needed.

In addition, the use of vehicle crush as defined by TAD may not be an entirely sufficient match variable for analysis of occupant ridedown. Under the flail-space assumption that the occupant and interior decelerate as a unit after occupant impact with the interior, deceleration peaks to the vehicle frame (e.g., the frame snagging on a guardrail post) could be transmitted to the interior and could result in injury but not in changes in vehicle crush. In these cases, TAD would not capture the necessary information, and the resulting true correlation would be lessened.

In terms of what should be considered by both the crash-test community and other researchers attempting similar efforts, the following is offered:

1. Maximize the use of the existing Roadside Safety Library through a case-by-case review of post-1980 test documenta-

tion such that available flail-space and other key measures are placed in the computerized file. Concentrate this effort to capture impact situations not now captured (i.e., angle, speed, and vehicle size gaps in the analysis file).

2. Attempt to identify additional crash-test conditions from non-FHWA or new FHWA-NCHRP tests not now in the library, concentrating on combinations that increase the variability of the test conditions.

3. Consider future (research, rather than compliance) funding for tests that increase the variability in the crash-test data by varying the vehicles and the speed and impact angles tested. For research, the vehicles tested within the weight classes should constitute as large a share of vehicles in the existing population as possible. This could be determined by examination of vehicle registration files supplemented by knowledge of possible clones.

4. Examine the possible use of other accident files, such as the National Accident Sampling System or the Longitudinal Barrier Special Study, which include information on estimated impact speed and angle and enhanced injury data, or the possibility of new large-scale crash reconstruction efforts targeted to vehicles used in crash testing. (Sample size will be a problem in the existing files if the clone requirement is retained.)

5. Consider alternative methodology involving "pseudo" measures of injury such as Head Injury Criteria and that of the Texas Transportation Institute through comparison of flail-space or traditional measures with data from anthropomorphic dummies. Problems with such dummy-related measures exist (e.g., correlation with injury and variability due to out-of-position dummies) and must be considered carefully in such research.

In summary, this has been a limited effort at attempting to define relationships between crash-test measures and occupant severity. Although it was not successful, there remains a clear need to determine whether the measures currently used in the design and testing of roadside features indeed maximize protection for occupants of the vehicles while minimizing cost.

ACKNOWLEDGMENT

This study was conducted under the auspices of the University of North Carolina Injury Prevention Research Center. It was supported by the Centers for Disease Control and Prevention, U.S. Department of Health and Human Services, and FHWA.

REFERENCES

1. Michie, J., *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., March 1981.
2. Shoemaker, N. *Summary Report of Highway Barrier Analysis and Test Programs.* Report VJ-1472-V-3. Cornell Aeronautical Laboratory, Cornell University, Buffalo, N.Y., 1961.
3. Bronstad, M., and J. Michie. *NCHRP Report 153: Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., 1974.
4. *Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances.* Transportation Research Circular 191. TRB, National Research Council, Washington, D.C., 1978.
5. Ross, H., E. Post, E. Nixon, D. Hustace, and E. Kristaponis. E. V. Warrents for Guardrails on Embankments. In *Highway Research Record 460*, TRB, National Research Council, Washington, D.C., 1973, pp. 85–96.
6. Young, R., E. Post, and R. Holcomb. *Simulation of Vehicle Impact with the Texas Concrete Median Barrier, Vol. 1. Test Comparison and Parameter Study.* Texas Transportation Institute, College Station, Tex., 1972.
7. Cook, J., and A. Bodocsi. *Criteria for Yielding Sign Supports.* University of Cincinnati, Cincinnati, Ohio, 1970.
8. Michie, J. Collision Risk Assessment Based on Occupant Flail Space Model. In *Transportation Research Record 796*, TRB, National Research Council, Washington, D.C., 1981.
9. Begeman, P., A. King, P. Weigt, and W. Patrick. *Safety Performance of Asymmetric Windshields.* SAE Paper 791009. Society of Automotive Engineers, New York, 1978.
10. Kornhauser, M. and A. Gold. Application of Impact Sensitivity Method to Animate Structures. *Impact Acceleration Stress.* NRC Publication 977. National Research Council, Washington, D.C., 1961, pp. 333–344.
11. Chi, M. *Assessment of Injury Criteria in Roadside Barrier Tests.* Report FHWA-RD-75-74. Federal Highway Administration, Washington, D.C., 1976.
12. Snyder, R. *State-of-the-Art—Human Impact Tolerance.* SAE Paper 700398. Society of Automotive Engineers, New York, May 1970.
13. *Branham Automobile Reference Book.* Branham Publishing Co., Santa Monica, Calif., various years.
14. *Vehicle Damage Scale for Traffic Accident Investigators (Third Edition).* National Safety Council, Chicago, Ill., 1984.
15. Rouse, W., and F. Gendre. *Field Experience and Evaluation of the TAD Vehicle Damage Rating Scale.* University of North Carolina, Highway Safety Research Center, Chapel Hill, 1969.
16. Vilardo, F. *Vehicle Damage Scale for Traffic Accident Investigators: An Investigation of Its Use and Potential for Predicting Driver Injury.* University of North Carolina, Highway Safety Research Center, Chapel Hill, August 1972.

Publication of this paper sponsored by Committee on Roadside Safety Features.

Vehicle Crash Tests of Type 115 Barrier Rail Systems for Use on Secondary Highways

JOHN JEWELL, ROGER L. STOUGHTON, AND DORAN GLAUZ

A total of three vehicle crash tests were performed on a California Type 115 bridge rail. The Type 115 bridge-rail test barrier consisted of two steel tube rails (10 × 10 × 0.64 cm) supported from the edge of deck by steel W8×31-posts at 244-cm spacing. The height of the top rail was 76 cm. There were two impact tests on a Type 115 bridge rail and one on a Type 115 bridge-rail transition. Although it was planned to test the Type 115 bridge rail to Performance Level 2 (PL2) and it was tested with a 2450- and 810-kg vehicle at 95 km/hr and 20 degrees, the snagging of wheels in Tests 471 and 472 led to the recommendation that the Type 115 be used as a PL1 bridge rail for lower-speed narrow bridges where impact angles are expected to be less. The Type 115 bridge-rail transition, when tested with a 2450-kg vehicle, produced some moderate pocketing, but could easily be stiffened to lessen the pocketing problem. Otherwise, the transition met all test criteria.

California has used the Type 115 bridge rail (Figure 1) for a number of years on lower service level highways. Better visibility, more usable deck width on existing bridges, and local agency aesthetics were needed. The Type 115 railing consists of two 10 × 10-cm tube rails mounted on W8×31 posts. It was designed under old AASHTO bridge-rail specifications that only required an analysis using a 44.5-kPa static load applied to the rails, but no crash testing.

All old or new bridge-rail designs used to replace old railings or for new construction now must be crash tested to qualify for federal aid. The crash tests must use current test and evaluation guidelines according to *NCHRP Report 230* (*1*) and AASHTO *Guide Specifications for Bridge Railings* (*2*). Hence, to continue using the Type 115 railing, the California Department of Transportation (Caltrans) needed to conduct crash tests that would confirm compliance with current guidelines.

The recently published AASHTO guide specifications for the first time provide for crash testing of performance level one (PL1) rails. PL1 rails are intended for local roads. Recent FHWA policy allows PL1 rails to be used on California highway rehabilitation work that is federally funded. The California Type 18 (Figure 2) metal railing (tubular) has met current crash-test requirements but is much more costly and takes up more deck width than the older steel rail design, the Type 115 (Figure 1). Therefore, it was decided to quality the Type 115 rail for use by crash testing. The PL2 level of testing was selected to determine the limits of performance of the Type 115.

California Department of Transportation, Division of New Technology, Materials and Research, 5900 Folsom Boulevard, Sacramento, Calif. 59819.

SCOPE

Three crash tests were performed, two on Type 115 bridge rail and one on a transition to that rail. The bridge-rail test conditions followed the AASHTO *Guide Specifications for Bridge Railings* (*2*) for a PL2 bridge rail (except no 8200-kg truck test was planned). The transition barrier was tested at the PL1 level, a decision made after the first two bridge-rail tests had been analyzed. The tests were conducted and evaluated using the procedures and criteria in *NCHRP Report 230* (*1*) and also evaluated using the AASHTO guide specifications (*2*). Intended impact conditions are given in Table 1.

BRIDGE-RAIL AND TRANSITION DESIGN

The Type 115 metal-tube bridge railing has two 10 × 10 × 0.64-cm structural steel tube rail elements attached to W8×31 steel posts. Spaced 244 cm on center, the tube rails are attached directly to the posts with no block outs. Rail-to-deck clearance is 41 cm and the overall height is 76 cm. Two 10-cm high rail faces are exposed to traffic, and there are no energy-absorbing elements. The posts were attached to the edge of the bridge deck with high strength anchor bolts. The two top bolts were 3.2 cm in diameter and the two bottom bolts 2.5 cm in diameter.

Steel rails and posts were used to minimize the rail area obstructing the vision of motorists. Structural steel concentrates strength in a small area of material. The two tube rails were needed to provide a broad impact surface for vehicles of varying height. The top edge of the top tube rail is 76 cm above the deck; thus, it prevents vehicles with high centers of gravity from getting over the rail better than do 68.6-cm high barriers (the lowest height now generally allowed).

The W8×31 posts and anchor bolts embedded in the edge of the deck met the old AASHTO bridge-rail static load design requirement. The anchor bolts used for testing were previously used in the testing of the Type 18 bridge rail and were judged to be undamaged and acceptable for additional tests with the Type 115.

The concrete-simulated deck was a block of reinforced concrete 25.6 m long, 1.07 m wide, and 0.91 m deep with a cantilevered section the length of the deck that was 1.07 m wide and 0.30 m thick. At the time of the first test the compressive strength (f'_c) of the concrete in the deck was 30.9

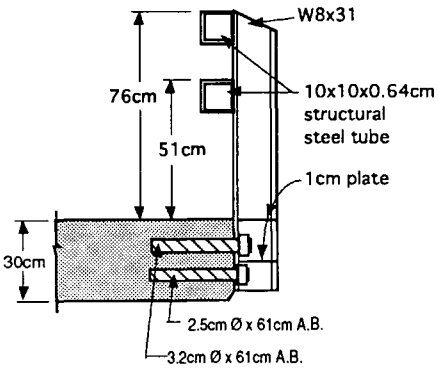


FIGURE 1 Type 115 bridge rail.

MPa. The cantilevered deck had steel reinforcement typical for a bridge deck, and all rebar conformed to ASTM A615, grade 60. The deck surface was flush with the surrounding asphalt-concrete pavement.

The transition from a thrie-beam guardrail to a Type 115 (Figure 3) bridge rail was accomplished through a 30 × 51 × 0.64-cm steel plate. A structural steel tube 30 cm long 10 × 10 × 0.64 cm was welded onto the plate. The flat side of the plate was bolted to the thrie beam. The side of the plate with the steel tube was fitted between the two tubes on the Type 115 and bolted to the post (Figure 4). The thrie beam was connected to a standard metal beam guardrail using a standard W-beam to thrie-beam transition piece. The guardrail was terminated with a breakaway cable terminal. The bridge rail was designed by the Caltrans Division of Structures.

CRASH TESTING

Test Vehicles

The test vehicles complied with *NCHRP Report 230* (*1*). For all tests, the vehicles were in good condition and free of major body damage and missing structural parts. All equipment on the vehicles was standard. The engines were front mounted. Ballast was used on Tests 472 and 477 only. Vehicle types used in the tests and their masses are shown in Table 2. The vehicles were self-powered in all tests.

Test Dummy

An anthropometric dummy with three accelerometers mounted in its head cavity was placed in the driver's seat of the test vehicle to obtain motion and acceleration data. The dummy was placed in the driver's seat and not restrained except in Test 472.

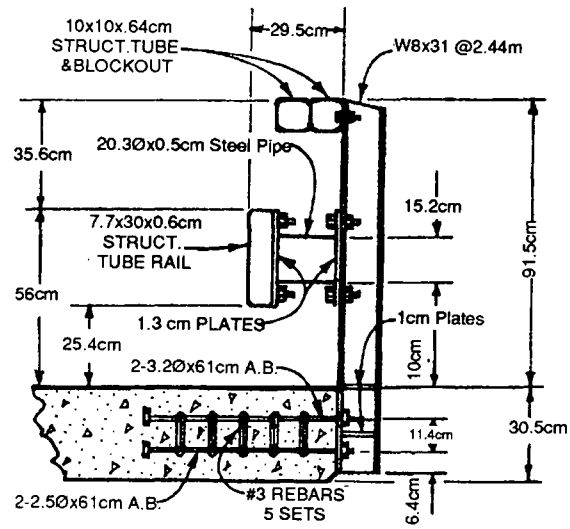


FIGURE 2 California Type 18.

Test Instrumentation

Six accelerometers were attached to the floor of the vehicle near the center of gravity to measure motion in the longitudinal, lateral, and vertical directions. Rate gyro transducers were also placed at this location to measure the pitch, roll, and yaw of the vehicle. The accelerometer data were used in calculating the occupant impact velocity.

CRASH TESTS RESULTS

Test 471

Impact Description

The vehicle weighed 810 kg and measured impact speed was 94.9 km/hr. The actual impact angle was 19.0 degrees (Figures 5 and 6*a*). The vehicle right side contacted the barrier face 38 cm downstream from Post 6. Contact continued for a distance of about 300 cm.

The right front tire contacted the lower rail face for about 61 cm starting at 37 cm downstream from Post 6 (Figure 6*b*). The right rear wheel touched the lower rail face for about 160 cm and left contact 42 cm downstream from Post 7 (Figure 6*c*). The right front alloy wheel was damaged on Post 7.

The right corner of the car bumper was slightly snagged and sliced off on Post 7 between the lower and upper rails. With the exception of the snagging wheel, the car was redirected smoothly. The vehicle remained upright throughout and after the collision. The exit angle and speed of the car

TABLE 1 Target Impact Conditions

Test No.	Barrier type	Mass (kgs)	Speed (kph)	Angle (deg)
471	115	810	95	20
472	115	2450	95	20
477	115 transition	2450	72	20

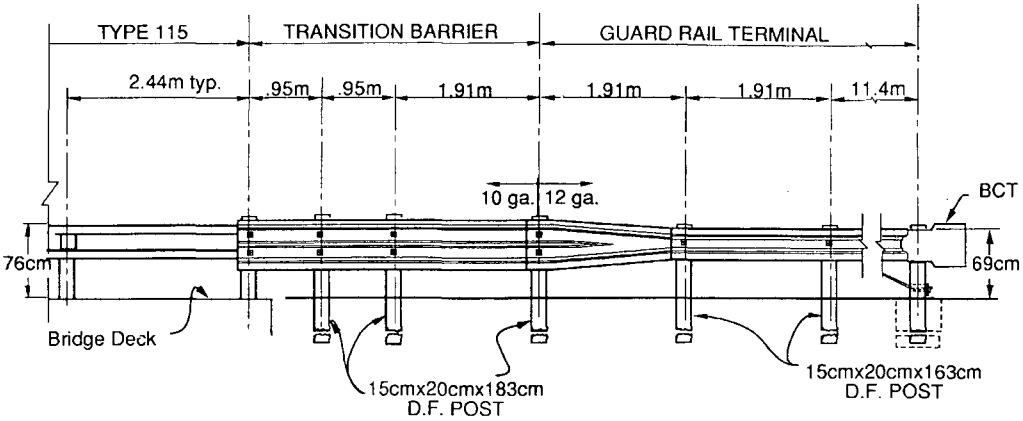


FIGURE 3 Type 115 transition.

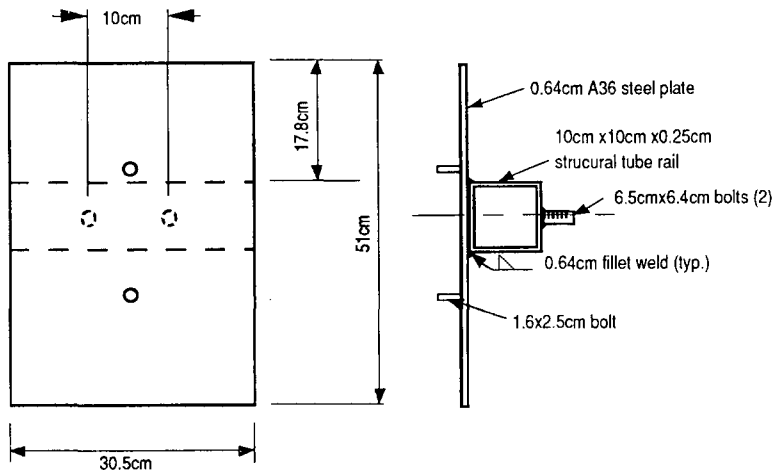


FIGURE 4 Thrie-beam to Type 115 transition piece.

were 5.0 degrees and 87.5 km/hr, respectively. The car stopped about 26 m downstream from the barrier after the brakes were applied.

Vehicle Damage

The test car was moderately damaged, with considerable crumpling of the right side body sheet metal but little frame damage (Figure 6*d*). The contact pattern extended from the right front corner for about 25 cm across the right side of the vehicle. The front frame members under the engine were bent on the right side. The windshield was cracked on the lower right corner. The right front fender was severely crushed by the penetration of the upper rail for a depth of about 18 cm. The right front door was jammed. Both front wheels were broken, and the tires were flattened and wheel movement was completely restricted.

TABLE 2 Test Vehicle Weights

Test No.	Vehicle	Ballast-(kg)	Mass-(kg)
471	1985 Isuzu 1-Mark	0	810
472	1985 Dodge Truck	562	2480
477	1985 Chevy Truck	230	2450

Barrier Damage

Barrier damage consisted of a slight bend of the rail and backward displacement of post tops in the impact area. The maximum dynamic lateral deflection was 4.45 cm at Post 7. The residual lateral displacement was 2.3 cm at the same post. The lateral displacement of post tops ranged between 1.9 and 1.4 cm. The maximum depth of car contact on the top of the upper rail was 3.7 cm at 57 cm upstream from Post 7.

The right front fender and each of the right tires contacted the barrier. The length of the right front fender contact with the top of the upper rail was 245 cm and ended 74 cm further downstream from the point where the rail lost contact with the car's right front fender. The tire marks of the right front wheel on the lower rail face were 51 cm high for a length of 61 cm. The right rear tire marks on the lower rail face were also 51 cm high for a length of 160 cm.

Dummy Response

During collision, the unrestrained dummy was thrown to the right. Its shoulder hit the right front door, bending it outward. The dummy's final position was on its back across the pas-

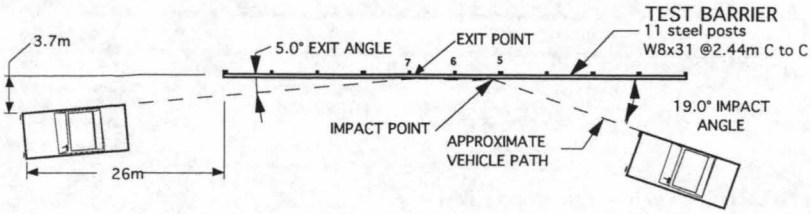


FIGURE 5 Vehicle trajectory, Test 471.

senger floor area with its legs wedged under the steering wheel. The dummy had a gash in its head, torn covering on the left foot, and torn coveralls.

Test 472

Impact Description

The vehicle weighed 2480 kg and speed at impact was 103 km/hr (Figures 7 and 8). The measured angle of impact was 21.0 degrees (Figure 8*a*). The right front corner of the truck bumper struck the lower rail face 17 cm downstream from the centerline of Post 5. The truck body's first contact with the upper rail was 37 cm upstream of the centerline of Post 5 (20 cm upstream of the bumper contact point).

The right front tire contacted the lower rail face at the centerline of Post 5 and remained in contact with the rail for about 494 cm. During impact, the right front tire contacted the top of the lower rail first at 10 cm downstream of Post 5 for a length of 43 cm and second at Post 6 for 20 cm. The truck's right front wheel snagged on Post 6 and was torn

FIGURE 6 Crash sequence, Test 471.

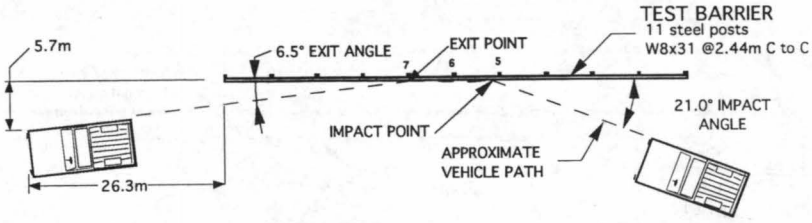
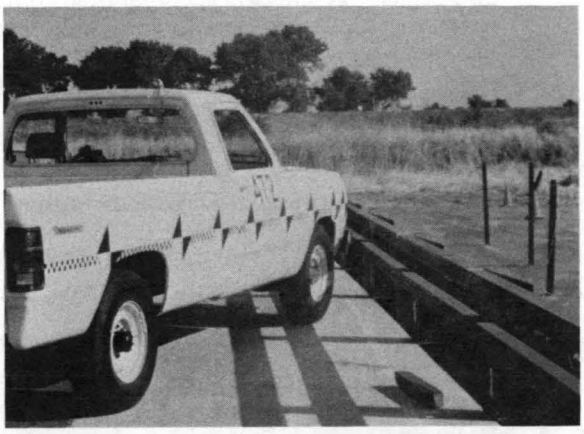
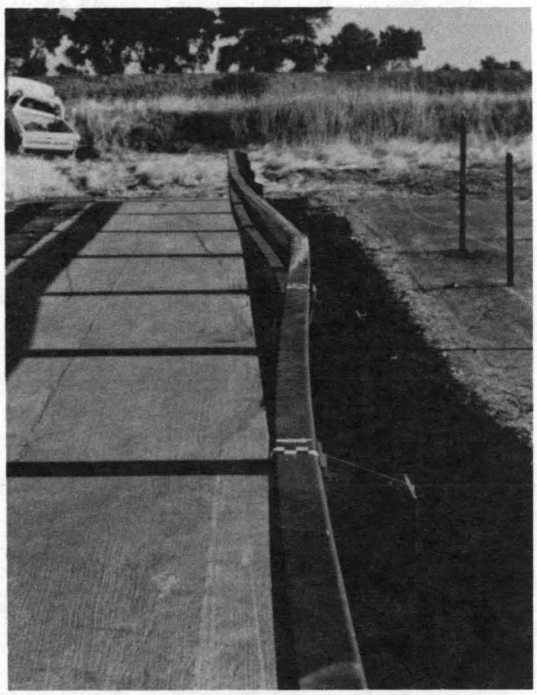


FIGURE 7 Vehicle Trajectory, Test 472.

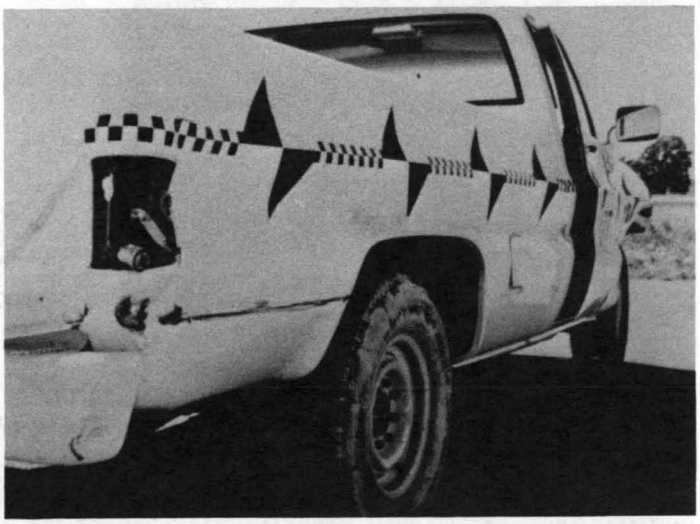


(a)



(d)

(b)



(c)

(e)

FIGURE 8 Crash sequence, Test 472.

forcefully off the truck. The wheel rolled and tumbled alongside the redirected truck and came to rest behind it.

The right rear tire contact with the lower rail started 76 cm downstream of Post 5 and ended 6.3 cm downstream of Post 7. The contact with the upper rail started 120 cm upstream and ended 18 cm downstream of Post 7. The only post contact by the right rear tire was with Post 6.

The truck continued to turn to the left after it left the pavement (Figure 8*b*). The truck was redirected relatively smoothly and came to rest about 26.4 m downstream from Post 11. The exit angle and speed were 6.6 degrees and 77.4 km/hr, respectively.

Vehicle Damage

The vehicle sustained extensive damage to various areas (Figure 8*c*). The right front bumper was crushed and bent under the crushed right front fender. the right front wheel, including the suspension assembly, was torn from under the truck during impact. The right door was crushed and jammed. The floor on the right side was crushed and pushed up into the passenger compartment. The windshield was broken, partially popped out of its frame, and pushed to the right. The radiator was pushed back to the fan. The battery broke loose from the mounts. The right side of the truck bed was crushed for the entire length at rail height. The bed was twisted and pushed rearward, toward the right side of the truck. The right wheel well was crushed by the shifting of the ballast (a 540-kg steel plate broke free from mounting brackets during impact).

Barrier Damage

Post and rail damage were limited to the impact area. The upper rail separated at the splice 0.8 cm at the face and 2.0 cm at the back. The lower rail also separated at the splice 0.64 cm at the face and 0.5 cm at the back.

The lateral deflection of the upper rail, measured at the posts, ranged from 0.64 to 28.2 cm, in a smooth long curve between Posts 4 and 8 (Figure 8*d*). The maximum deflection was measured at Post 6. The lateral displacements at the top of the steel Posts 4, 5, 6, 7, and 8 were 12.1, 27.6, 40.3, 20.3, and 10.8 cm, respectively. The same posts experienced some bending ranging from 1.3 to 5.1 cm. The washers on the top studs of Posts 5 and 6 were pulled through the holes (Figure 8*e*). After impact, all posts remained attached to the bridge deck and the two rails remained attached to the posts.

Tire marks from the right front wheel on the upper rail face were about 76 cm high for a length of 110 cm between Posts 5 and 6. On the lower rail, marks from the same tire appeared for 260 cm. The same tire contacted the top of the lower rail for 43 cm downstream from Post 5 and for 20 cm downstream from Post 6.

The right rear tire marks on the upper rail face were 28 cm long 110 cm downstream from Post 5 and 140 cm long 120 cm downstream from Post 6. The right rear tire marks on the lower rail face were about 420 cm long, and started 76 cm downstream from Post 5.

Dummy's Response

During collision, the restrained dummy remained almost in its place. The dummy did not experience any damage as a result of the collision. The dummy's final position was in the driver seat.

Test 477

Impact Description

The height to the lower edge of the truck's bumper was 44.5 cm and it was 66.8 cm to the top of the bumper. The vehicle weighed 2450 kg and was freewheeling and unrestrained just before impact. The measured impact speed was 74.8 km/hr and the angle was 19.2 degrees (Figures 9 and 10).

The vehicle bumper contacted the middle ridge of the transition face 15 cm upstream from wood Post 3. Contact continued for a distance of about 381 cm. The right front tire contacted the thrie-beam corrugation at Post 3 and ended at Post 1. The right rear tire touched the lower ridge about 40 cm upstream from Post 2 for about 84 cm. The sheet metal body contact of the vehicle with the upper ridge of the rail began 44.5 cm upstream from Post 3 and ended 15 cm downstream from metal Post 1. The body contact of the vehicle with the middle and lower ridges was noted as discontinuous scratches that ended about 15 cm past steel Post 1.

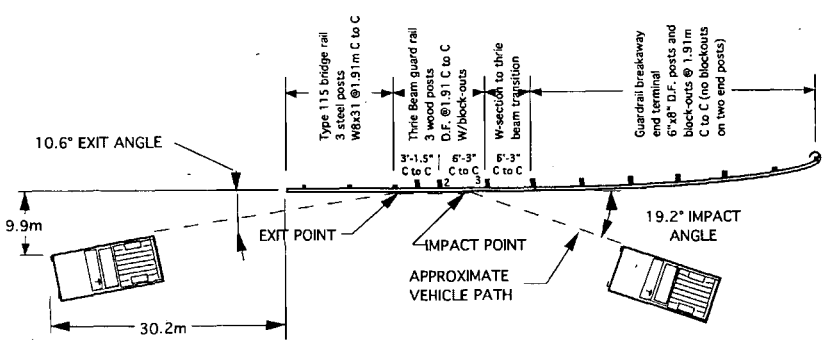
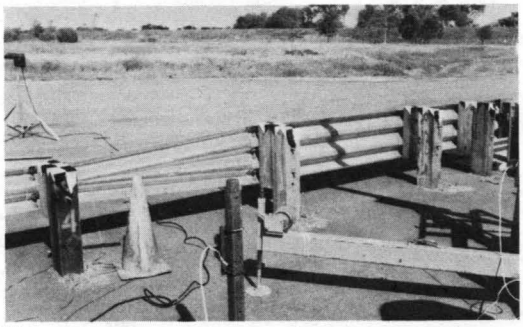
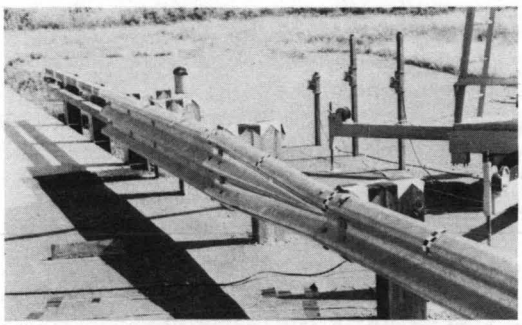


FIGURE 9 Vehicle trajectory, Test 477.

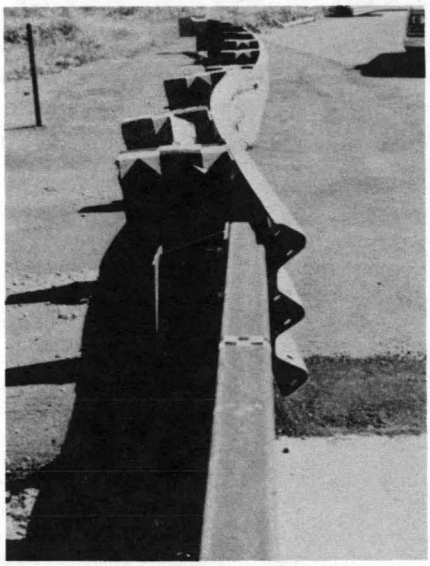
(a)



(b)



(c)



(d)

FIGURE 10 Crash sequence, Test 477.

The vehicle was redirected when pocketing occurred upstream from Post 1. It remained upright throughout and after the collision. The exit speed and angle were 59.2 km/hr and 10.6 degrees, respectively. The remote brakes were applied after impact. The vehicle went off the pavement and stopped on a berm. The final location of the vehicle was 40 m from the impact point and 11.7 m in front of the barrier.

Vehicle Damage

The vehicle was damaged, with severe crushing of the right front side and moderate impairment on both sides. The bumper was jammed, split at the right corner, and pushed to the left side (Figure 10*d*). The entire right side of the vehicle was scraped. The tires were intact but movement of the right front wheel was restricted because the front bumper was pushed against it. There was no intrusion into the passenger compartment.

Barrier Damage

Post and rail damage was limited to the impact area. The top of the wood posts 1, 2, 3, 4, and 5 had permanent lateral displacements that were 15, 25, 23, 8, and 2.5 cm, respectively, and dynamic lateral deflections of 20, 36, 33, 10, and 8 cm, respectively. Wood Posts 4 and 5 were deflected at ground level. The first steel post of the bridge rail (metal Post 1) had permanent and dynamic deflections of 1.3 and 3.8 cm, respectively.

The length of vehicle scratch on the upper ridge of the rail was 445 cm. The tire marks of the right rear wheel covered a length of 84 cm on the lower ridge starting 41 cm upstream from Wood Post 2.

Dummy Response

During collision, the unrestrained dummy was thrown forcefully to the right side of the vehicle. The dummy's final position was on its side with its upper body across the passenger side and its legs under the steering wheel.

DISCUSSION OF CRASH-TEST RESULTS

Structural Adequacy

Tests 472 and 477, using the 2450-kg pickup trucks, tested the structural adequacy of the bridge rail and transition. The barriers were not penetrated or vaulted and there were no detached barrier elements; thus the design is adequate for the tested conditions. The bending of the bridge-rail post with the partial pull-through of the nuts and washers at the flange indicated that posts were being stressed significantly when struck.

The small car had alloy (instead of steel) wheels, which may have lessened the deceleration during the wheel snagging. The pickup truck had its front wheel torn off completely. The value of μ calculated for Test 472 using the formula in the AASHTO guidelines was unacceptably high. Type 115 is

recommended as a PL1 bridge rail for lower-speed narrow bridges where impact angles are expected to be less than the 20 degrees tested.

Occupant Risk

Each test required the test vehicle to remain upright with acceptable (low) levels of yaw, pitch, and roll. All the tests produced vehicle reactions that were acceptable within this criterion.

The AASHTO guidelines limit the occupant impact velocity in the lateral direction to 7.6 m/sec, in the longitudinal direction to 9.1 m/sec, and the ridedown acceleration in both directions to 15 *g*. Table 3 shows the test data as compared with NCHRP 230 criteria (*1*).

Vehicle Trajectory

NCHRP Report 230 (*1*) stresses that "trajectory evaluation for redirection type of tests is focused on the vehicle at the time it loses contact with the test article, and the subsequent part of the trajectory is not evaluated."

The exit angle for all tests was less than the recommended upper limit of 60 percent of the impact angle; hence, all tests passed the criterion. The speed reduction of 25.9 km/hr is greater than the recommended 24 km/hr maximum for Test 472, but the 1.8 km/hr difference is not significant enough to fail Test 472. The vehicle exit speeds and angles are shown in Table 4.

The Type 115 bridge rail and the thrie-beam transition to the bridge rail passed all the NCHRP 230 (*1*) and AASHTO guidelines (*2*) criteria (Table 5) except for a minor exceedance of the AASHTO guide specification criteria 3.f. in Test 472. This failure led us to classify the Type 115 as a PL 1 barrier.

RECOMMENDATIONS

The researchers propose to design a bridge rail similar to the Type 115 that will have three steel-tube rails instead of two to eliminate wheel snagging on the post. It is intended to crash test this design with passenger vehicles using PL2 test procedures and criteria.

It is also recommended that the possible modification of a terminal connector of the type shown in Figure 11 be used to establish a viable solution for vehicle impacts occurring at the transition connection, but coming from the opposing direction. This terminal connector is currently approved in California for use in transitions from thrie beam to the concrete safety shape. The researchers and bridge engineers involved in the design and testing of the transition will be reviewing this terminal connector design and other options to decide whether additional testing is warranted.

CONCLUSIONS

On the basis of the results of the two impact tests on the Type 115 bridge railing and the one impact test on the Type 115 bridge railing transition, the following conclusions can be drawn:

- The Type 115 bridge rail design presented here can successfully contain a 2450-kg ballasted pickup truck striking at a 20-degree angle at 97-km/hr. However, the snagging of a wheel on Test 472 disqualifies the Type 115 bridge rail for PL2. But, because of passing the AASHTO guide specification and NCHRP 230 criteria, and otherwise smooth redirection, the Type 115 bridge rail is acceptable for PL1 impact conditions.
- The Type 115 bridge rail can smoothly redirect a small car and a pickup truck without any signs of undesirable be-

TABLE 3 Occupant Risk Test Results Compared with NCHRP 230 Criteria

	Test 471	Test 472	Test 477
Vehicle Mass, kg	810	2480	2450
Vehicle Speed, kph	94.9	103.2	74.8
Vehicle Angle, degrees	19	21	19.2
Occupant Impact Velocity - Long. (m/s) (Threshold - 9.1 Limit - 12)	4.08	5.49	3.81
Occupant Impact Velocity - Lat. (m/s) (Threshold - 6.1, Limit - 9.1)	5.61	5.91	5.52
Ridedown Acceleration - Long. (g's) Threshold - 15, Limit - 20)	NA	-8.0	0.2
Ridedown Acceleration - Lat. (g's) Threshold - 15, Limit - 20)	-8.9	10.6	7.6
Maximum 50ms Avg. Accel. - Long (g's)	-5.7	-5.2	2.9
Maximum 50ms Avg. Accel. - Lat. (g's)	-8.3	7.7	5.9

TABLE 4 Vehicle Trajectories and Speeds

	Angles			Speeds		
Test Number	Impact Angle (deg)	60% of Impact Angle (deg)	Exit Angle (deg)	Impact Speed, V_i (kph)	Exit Speed, V_E (kph)	Speed Change V_i - V_E (kph)
471	19.0	11.4	5.0	94.9	87.7	7.2
472	21.0	12.6	6.5	103	77.4	25.9
477	19.2	11.5	10.6	74.8	59.2	15.6

TABLE 5 Summary of Crash-Test Criteria

NCHRP 230	Test 471	Test 472	Test 477
Structural Adequacy			
A. Containment and smooth redirection	P *	P *	P
D. No debris hazard to traffic or passenger compartment	P	P	P
Occupant Risk			
E. Vehicle upright, moderate pitch, roll and yaw; no passenger compartment intrusion	P	P	P
F. Occupant Impact Velocity & Ridedown Acceleration criteria	P	P **	P **
Vehicle Trajectory			
H. Minimal intrusion in traffic lanes	P	P	P
I. Minimal speed change and exit angle	P	F ***	P
AASHTO Guidelines			
1. Tests per NCHRP 230 criteria; report max. loads that can be transmitted from bridge railing to bridge deck	P/NA †	P/NA †	
2. Within vehicle speed and angle tolerances	P	P	
3. a. Vehicle contained	P	P	
b. Vehicle/barrier debris was no undue hazard to passengers or traffic	P	P	
c. No intrusion of passenger compartment	P	P	
d. Vehicle remains upright	P	P	
e. Smooth vehicle redirection, yaw of rear of vehicle < 5°	P	P	
f. Smooth vehicle to barrier interaction, $\mu < 0.35$	P μ=0.067	F μ=0.517	
g. Occupant Impact Velocity and Ridedown Acceleration criteria	P	P	
h. Vehicle trajectory and exit angle	P	P	

* Smooth redirection despite wheel snagging.
** Not required to pass Criteria in these tests.
*** Exceeded change of velocity limit by only 1.8 kph.
† These loads have not been measured or calculated.
P = passed
F = failed

havior (except the wheel snagging) and without exceeding occupant risk evaluation guidelines.

- The Type 115 bridge-rail transition produced some pocketing, but can easily be stiffened to lessen the pocketing problem. It passed all criteria for transitions in NCHRP 230. After consideration of the transition design, however, it was concluded that there may be a problem with impacts in the reverse direction. Therefore, until further investigation can yield a reasonable level of safety in both directions, the transition design discussed in this paper should not be used where impacts can occur in the reverse direction.

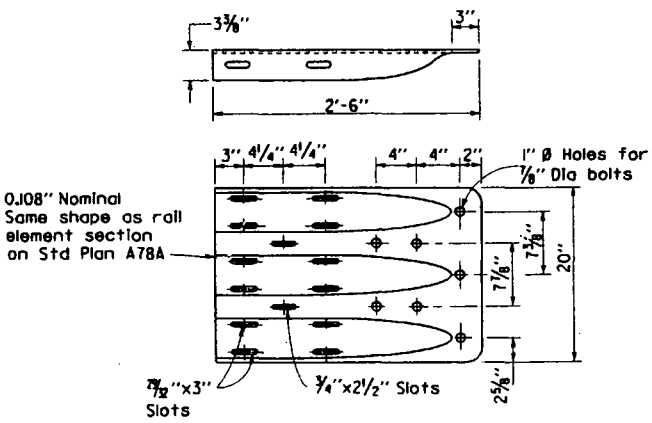


FIGURE 11 California standard plan A78B terminal connector.

ACKNOWLEDGMENT

The work reported in this paper is the result of a research project federally funded through the highway planning and research program. Caltrans conducted the crash tests and collected and analyzed the data. Appreciation is due to Anne Hernandez, who contributed to this report.

REFERENCES

1. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., March 1981.
2. *Guide Specifications for Bridge Railings.* American Association of State Highway and Transportation Officials, 1989.

Publication of this paper sponsored by Committee on Roadside Safety Features.

North Central Expressway, Dallas: Case Study of Enhancement-Inclusive Urban Freeway Design

JOHN P. KELLY AND RICK J. ROBLES

The North Central Expressway (US-75) project in Dallas, Texas, is a total urban freeway reconstruction that provides an excellent case study for the new range of opportunities available to transportation engineers under the new 1991 Intermodal Surface Transportation Efficiency Act. This 10-mi-long, $640 million job was planned from the outset as an innovative project that will expand the envelope of urban freeway design and be a precursor of 21st century ideas. This project encompasses many of the enhancement elements listed in the act (e.g., pedestrian and bicycle facilities, landscaping and other scenic beautification, preservation of rail corridors for bicycle trails, archaeological planning and research). The expressway offers a case study of a new environmentally sensitive "enhancement-inclusive" design approach, methods used to solicit public input into the final design, the end product of this new approach, and advice for others embarking on similar undertakings based on the experiences on this project.

Dallas is a thriving metropolis that boomed to a population (in the city proper) of almost 1,000,000 people during the 1980s. But Dallas is a relatively young city typical of the southwestern United States. It was founded on the banks of the Trinity River in the mid-1800s and enjoyed growth brought by the development of the national rail system in the late 19th century.

Along with the progress in transportation came issues that had to be addressed, such as comfort, safety, and travel speeds. Today, as highways are rebuilt and upgraded throughout the United States, the traveling public, residential communities, and abutting property owners have new demands. There are increased expectations that new urban highways will be an aesthetic enhancement to and sensitively situated within communities, in addition to carrying large volumes of traffic efficiently.

Knowledge and technology are now available to design highways with the same attention that has in the past been reserved for buildings and special theme areas. But doing this well requires a synthesis of all the parts into a comprehensive approach.

A case study that offers an opportunity to address these aesthetic challenges in an existing urban context is the reconstruction of the North Central Expressway (US-75) in Dallas, Texas. This project offers more than surface decoration—it offers the ultimate chance to reevaluate the design of various highway elements.

Texas Department of Transportation, 8150 North Central Expressway, Suite 801, Dallas, Tex. 75206.

Because of the unusually high profile of the project—the existing expressway is infamous across Texas—and this is the first total reconstruction of an existing urban freeway in Texas, the Texas Department of Transportation (TxDOT) designers were given wide latitude to explore new concepts for reconstruction. A tabula rasa, or slate, approach was used, somewhat like the approach General Motors took with its Saturn Division.

In 1911 the Dallas City Planner, George E. Kessler, suggested that the right-of-way of the Houston and Texas Central Railroad be purchased to provide for the construction of a "Central Avenue." It would run north from the heart of downtown Dallas radially out some 5 mi.

On March 3, 1947, almost 40 years after Kessler's first recommendation, construction began on what had by then become "Central Expressway." This was the dawn of controlled-access freeways in Texas.

The existing facility is a first-generation freeway with almost continuous parallel service roads. Traffic volumes have grown to nearly twice the design volume and exceed theoretical capacity by 50 percent or more.

The lower 3 mi of the existing expressway consist of six main through lanes and six parallel lanes of service roads. The upper 7 mi consist of four main lanes and four or six service lanes.

In the 1970s several variations of a plan to construct elevated express lanes to relieve increasing congestion were developed by the Texas Highway Department (TxDOT's predecessor). Most plans met with some skepticism or public criticism. By 1981, a tenuous agreement had been struck to construct new elevated lanes above the existing facility, which would remain in place. However, strong community opposition (commercial and residential) doomed this controversial plan.

In 1984, a community task force (North Central Task Force, or NCTF) developed a publicly supported consensus plan that incorporated a totally reconstructed, depressed-main-lane facility with continuous service roads and in the southern 3 mi, a twin-bored light-rail transit tunnel. In the north 7 mi, light rail would run parallel and adjacent to the expressway in an abandoned railroad corridor. The rail system is a project of the Dallas Area Rapid Transit Authority (DART). After intense discussion and public involvement, the Texas Highway Commission in 1986 approved this consensus plan.

The new facility is an eight-lanes-plus-auxiliary-lanes design with continuous parallel frontage roads, with an expected capacity of 230,000 vehicles/day. In the south 6 mi of the 10-

mi project, the main lanes will be depressed in a "concrete canyon," which will cross under 15 cross-streets. In the north 4 mi, a more typical at-grade or above-grade profile will prevail. This area will have the freeway crossing over four cross-streets (Figure 1).

COMMUNITY IMPACT AND RESPONSIVE MITIGATION

The North Central Expressway, carrying more than 200,000 vehicles/day, will have many of the same environmental impacts any other facility of this magnitude would have—noise, air quality, water quality, visual intrusion, and so forth. In addition, it can be perceived as a neighborhood barrier, segregating communities and disrupting the urban fabric. The perceived barrier effect was one of the strongest rallying points for those who had opposed the elevated-lanes concept. One of the initial goals was therefore to identify communities of interest in the corridor and try to find ways to reknit these areas so that the barrier effect of the facility is minimized. Noise and visual impacts were concerns in two single-family residential areas. The community wanted to be shielded from the freeway.

For TxDOT to truly adhere to its commitment to take a totally new approach to designing highways, an architectural firm was chosen to provide a broad range of new perspectives in design concepts, as well as plan-specific structural and landscaping design. The first step was to study the entire corridor, identify and understand the various socioeconomic communities of interest, and develop an overall project concept. The second step was for a team of architects, landscape architects, and urban planners, accompanied by two TxDOT engineers, to visit the most aesthetically advanced projects in the nation, on the basis of FHWA and other recommendations, to gain firsthand knowledge of the state of the art in highway design. In addition to gaining valuable information, the visits confirmed that the most innovative highways emerged when they were considered as unique opportunities to be good neighbors in their communities, assets to adjoining private property owners, and sources of pride for the communities and the agencies that build them.

THREE TYPES OF EXPRESSWAY ENVIRONMENTS

The planning and architectural analysis determined that many districts of the city abut and sometimes cross the North Central Expressway. The variety reflects the city's northward growth. Neighborhoods of 1930s bungalows in the south give way to 1960s suburban homes in the north, just as rows of low commercial buildings in the south precede freestanding office towers further north.

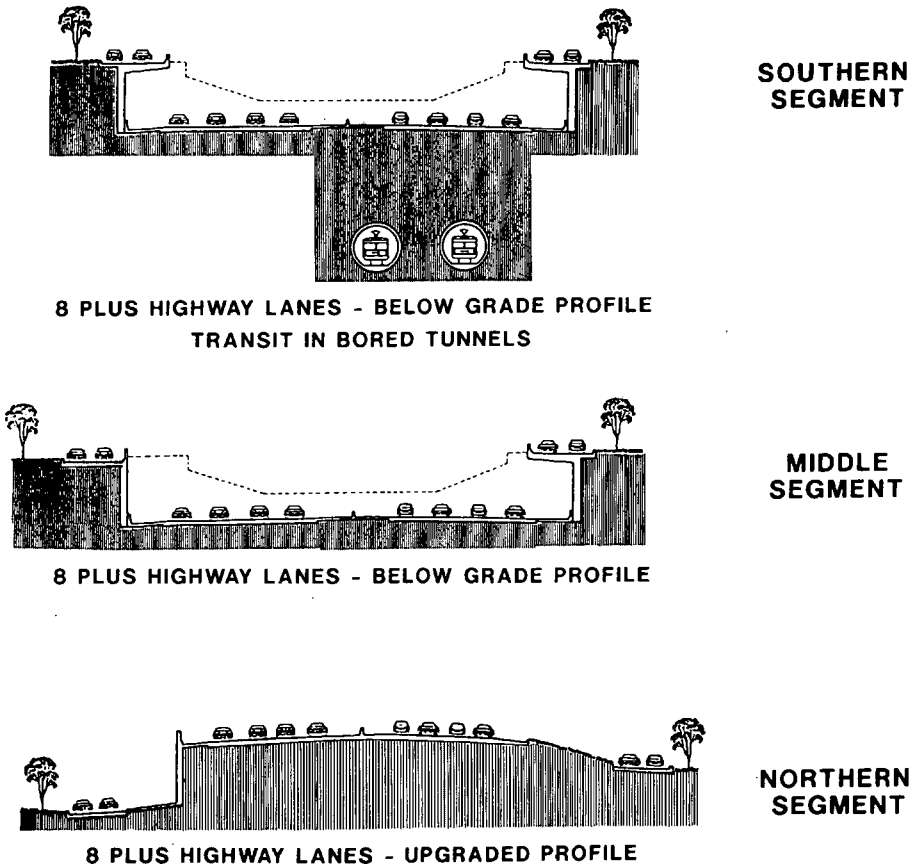


FIGURE 1 Typical sections.

The presence of the railroad had established a boundary, so that near downtown the construction of the expressway created no significant new barrier. As the alignment progressed farther north into then-undeveloped areas, the highway delineated developing neighborhood boundaries. Near residential areas that abut but do not cross it, the expressway's presence created a beneficial boundary to hold commercial development at bay. In other areas, such as older, single-story retail neighborhoods, the barrier may be detrimental. In those areas, the new expressway design for crossings focused on reknitting those communities.

The closest-in neighborhoods have small businesses and single-family homes of modest dimensions. Farther out, the development of mostly residential areas adjacent to the corridor took place in the 1930s and 1940s. Southern Methodist University (SMU) is located adjacent to and west of the expressway in this area. From about 4 mi and farther out, commercial development, mostly office and retail, paralleled at least one side of the alignment. From Northwest Highway (Loop 12), which the expressway crosses about 5 mi from downtown, north to Interstate 635 (Lyndon B. Johnson Freeway), primarily commercial and office development later occurred on both sides of the highway.

In general, the more affluent areas of Dallas are north of downtown, and the Central Expressway developed over the 1960s, 1970s, and 1980s into a primary commuter route for workers who moved farther and farther north to the burgeoning northern suburbs.

The three general types of environment identified are urbanscape, linear greenway, (geographically within the limits of the urbanscape), and openscape (Figure 2).

Urbanscape

South of Northwest Highway, the expressway is primarily urban in scale and character. An urbanscape is created by office and commercial buildings lining the service roads. Small individual buildings, typically one to three stories high, located directly adjacent to the expressway create a dense urban environment with little open space. This low urban fabric is punctuated by mid-rise office buildings.

A variation of the urbanscape will occur when the DART light-rail line is constructed immediately adjacent to the expressway between Mockingbird Lane and Lovers Lane. The depressed roadway section will be bordered by adjacent partly elevated track, creating a unique urban environment.

The urbanscape will undergo many immediate changes with the new depressed expressway configuration. Frontage road improvements will increase access. In exchange, business and property owners will have reduced visibility from the freeway mainlanes. In addition, right-of-way expansion and reconstruction will remove the mature trees lining the existing urbanscape, changing the environment and increasing the sterile impact of the expressway.

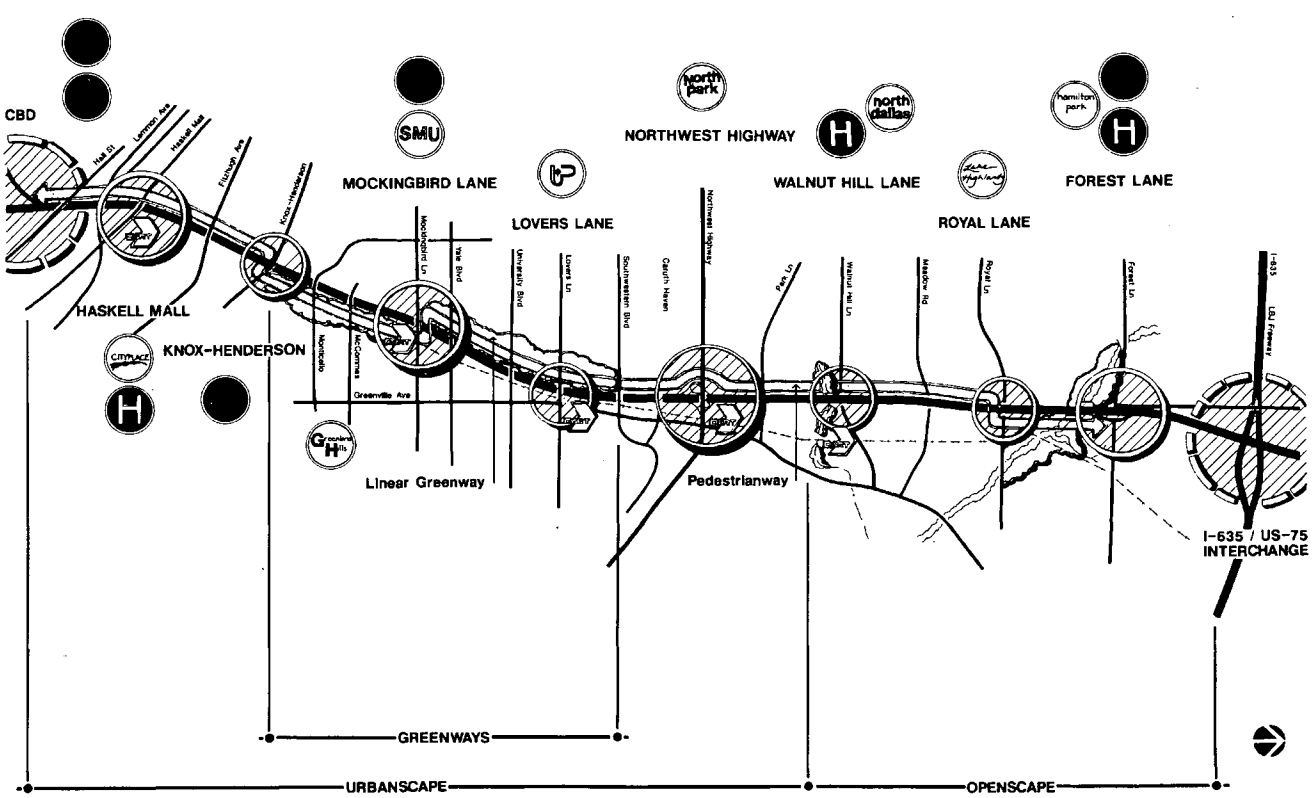


FIGURE 2 Design agenda, final scheme.

Redevelopment of office and commercial buildings can be anticipated following completion of the new expressway. Construction of the DART light-rail line may facilitate high-density office and commercial development in proximity to the light-rail stations at the intersections of Haskell Mall, Mockingbird Lane, and Lovers Lane.

Linear Greenway

The linear greenway is defined by mature vegetation at the edge of older residential areas south of Northwest Highway. Residential neighborhoods line the east side of the expressway from Knox-Henderson to Mockingbird Lane, and the west side from Yale Boulevard to Southwestern Boulevard. The low single-family residences and large trees of the greenway areas currently provide a sharp visual contrast to the other environments.

Although reconstruction of the North Central Expressway will affect the linear greenway areas by reducing the tree cover or eliminating the trees altogether, it will not bring the change in scale of development that can be expected in the other two environments.

The existing stability of both residential areas warrants preservation of their residential character with the assistance of landscape and screening improvements.

Openscape

North of Northwest Highway, the character of the expressway is much different. Although this area is also lined with office and commercial buildings, development is more scattered along the roadway and individual structures are frequently larger than in the southern section. This type of environment can be characterized as an "openscape."

Most changes to the openscape will follow completion of the improved expressway, when reconstruction will serve as a catalyst for new commercial development. The potential exists for the openscape environment to become a corridor lined by mid- and high-rise office buildings, as vacant property is developed and apartments and small commercial buildings are redeveloped.

The new expressway alignment will be approximately the same as the current alignment, both vertically and horizontally. Separated at intersections, the frontage roads and main lanes will often be at roughly the same grade, thereby increasing the impact of the expressway.

PUBLIC INVOLVEMENT

Once the analysis of the highway's urban context was complete, the next step was to create alternative concepts to be presented for public input and acceptance. To do this, a North Central Amenities Task Force (NCATF) was set up under the umbrella of the NCTF. A design symposium was held on the SMU campus early in the process to present the initial concepts to the public in hopes of inspiring community input and participation. Reaction and comments received at the symposium were evaluated by the architectural planners.

Subsequently, the 10-mi corridor was divided into six geographic areas. Smaller meetings, sponsored by TxDOT, were held with interested persons in each area. An exchange of ideas and concepts was led by the architectural consultants.

To better integrate the facility into its environs, one concept was to reflect, through architectural elements, the character of the various neighborhoods in the related stretch of highway. However, a totally neighborhood-reflective approach may result in a somewhat discontinuous effect for the through traveler. In the case of the North Central Expressway, the presence of continuous service roads allows an opportunity to have the best of both worlds in the sense that through-project continuity can be maintained for the main-lane design, and neighborhood-reflective design can be incorporated at the service road and cross-street intersections, which are the actual interface points between the freeway and the community.

ALTERNATIVE CONCEPTS

As a part of the design process, the planning team explored alternative approaches to the design of the North Central Expressway. Three design themes were investigated in a search for the right fit between the expressway and the surrounding environment. The process also allowed testing of individual component designs. The concepts were prepared for presentation to TxDOT, local cities, and the general public with the intent of incorporating their responses into a final design package.

Drawing on three different relationships between the expressway and the surrounding city, the alternatives explored the relationships through design of the key elements.

- Alternative A considered the expressway as part of a regional transportation network that passes through the city. Designed as a major state-of-the-art freeway, Alternative A is inspired by a streamlined freeway focus (Figure 3).
- Alternative B considered the expressway as a boulevard through the city, reflecting the character of each district it passes through. In this scheme, the character of individual neighborhoods is expressed along the expressway (Figure 4).
- Alternative C considered the expressway as a municipal monument, a statement unique to Dallas. This alternative presented the expressway as a major public facility, designed with the same attention that might be given to an important municipal structure, such as a city hall or a public library or an international airport (Figure 5).

Each alternative focused attention on different aspects of the corridor. The emphasis on particular components changed with each alternative. The end product of the three concepts produced varied results.

DESIGN OBJECTIVES

Designing a freeway necessitates special interpretation of traditional engineering, architectural, landscape, and urban design principles. The unusually large scale and high speeds present a challenge requiring a unique set of design objectives.

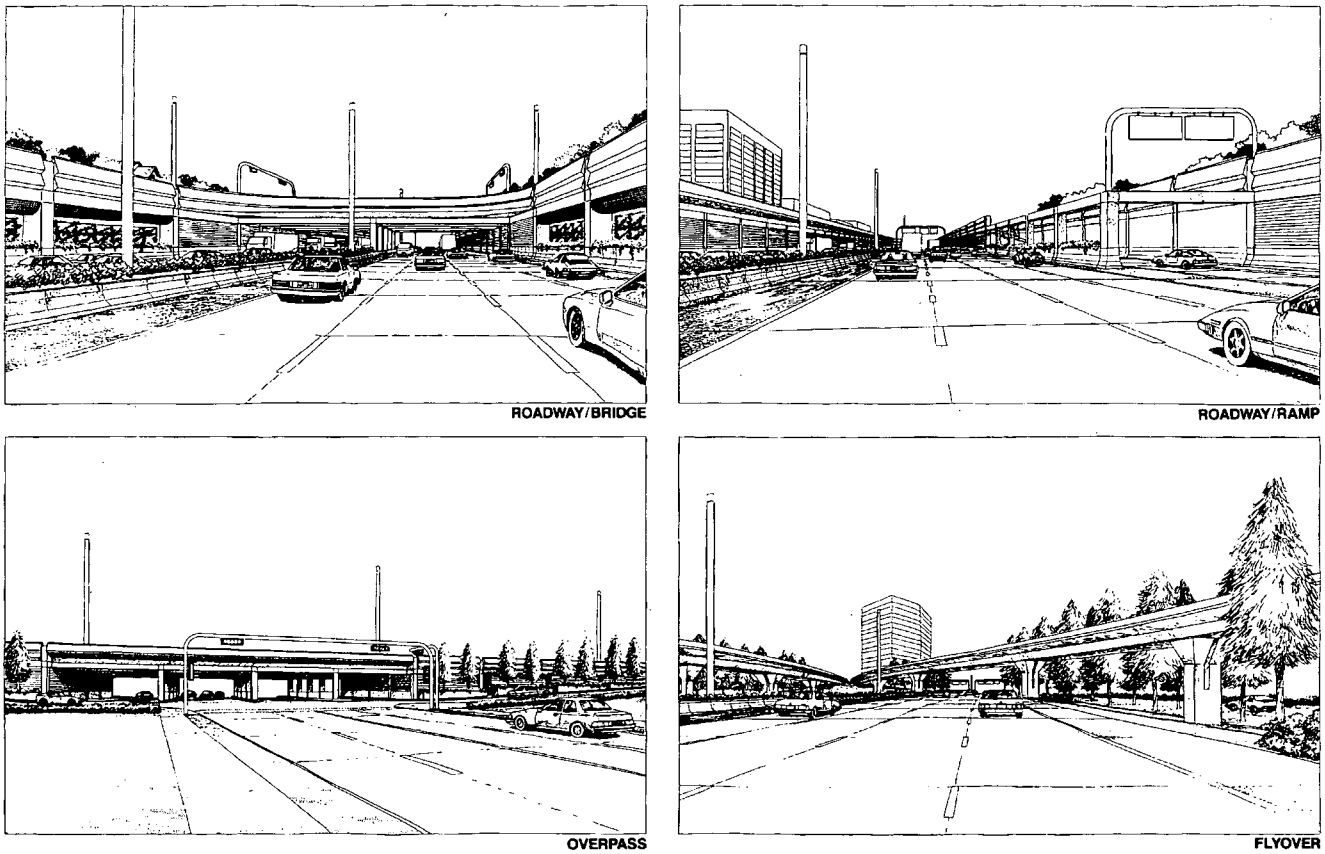


FIGURE 3 Perspectives, Scheme A.

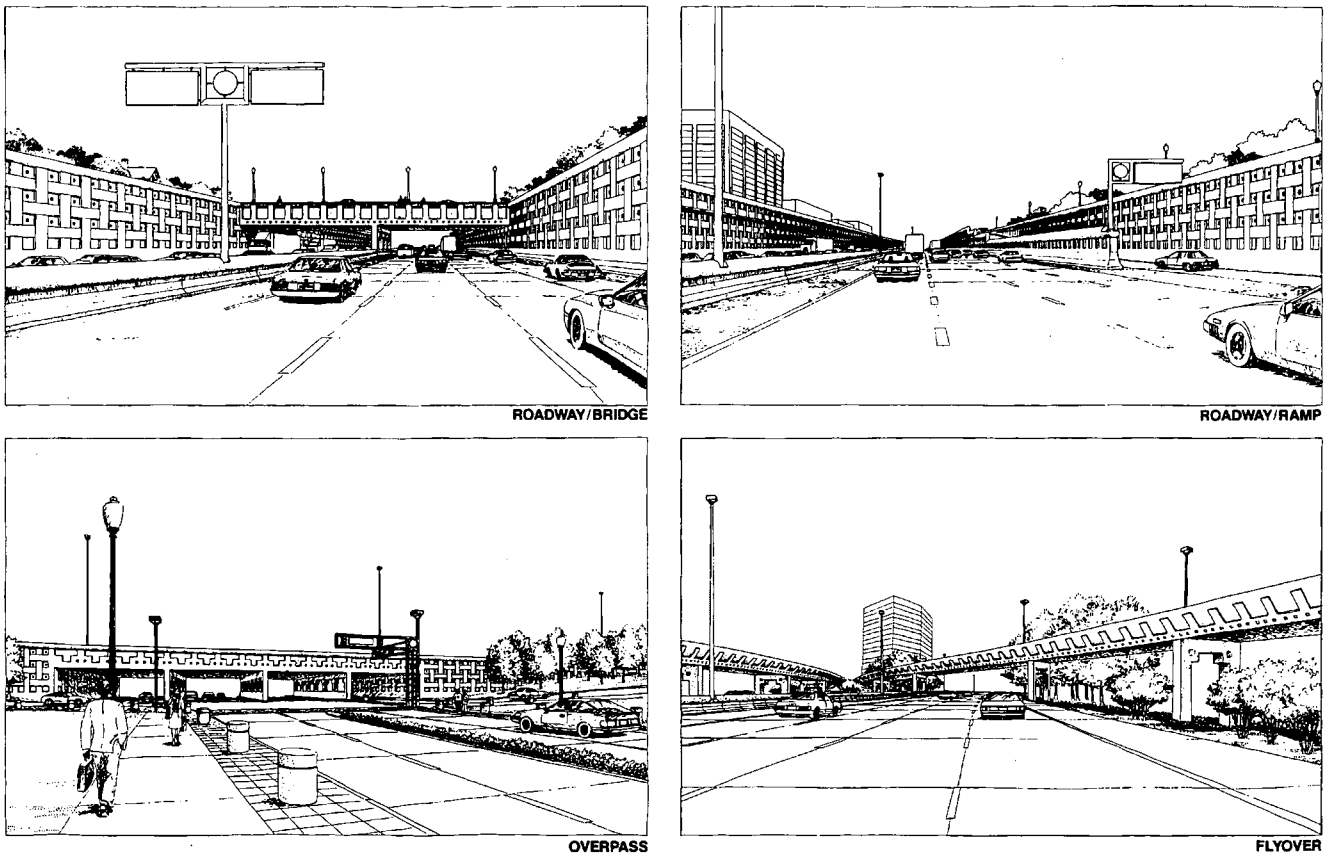


FIGURE 4 Perspectives, Scheme B.

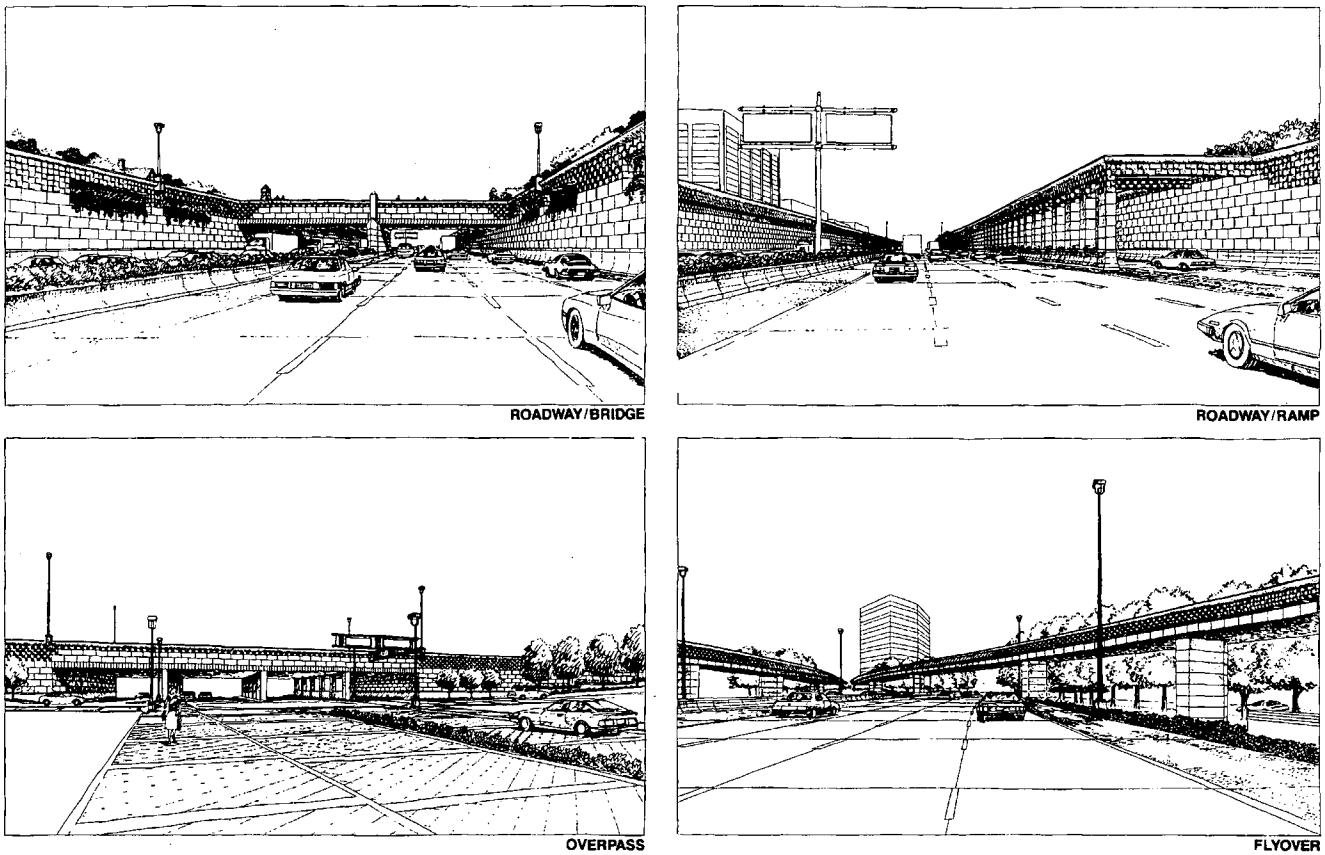


FIGURE 5 Perspectives, Scheme C.

The following objectives served as guidelines for the designers on the North Central Expressway project:

- Visual organization,
- Timeless design,
- Appropriate scale and speed for two environments, and
- Views to and from the road.

Visual Organization

Much of the challenge of this project lay in the task of organizing the engineering of the expressway and subtracting the clutter. TxDOT standards strictly govern the design of highways, but they do not preclude or assure a visually appealing design. Organization and coordination of all the elements in the expressway corridor are very important objectives.

Timeless Design

With a life expectancy of many decades, the North Central Expressway must endure time and trends. Designing the expressway as a public facility to serve many generations requires a timeless approach, one that will weather changes in style and public opinion.

Appropriate Scale and Speed for Two Environments

The expressway actually involves two separate design problems. The main lanes and the frontage roads are different environments, the scale and speed of each requiring special attention.

Driving at 55 mph on the main lanes, a driver will not have time to study the details of the expressway. Textures and patterns must be at a scale that will not become lost or confusing. In contrast, when traveling on the frontage road, the driver will be able to absorb a more intricate scale of design.

Views to and from the Road

Views from the expressway will be dominated by the wall structures in the southern 6-mi "canyon" section. Several concerns with this configuration were readily apparent. One was driver disorientation because of being visually cut off from the roadside landmarks. The harsh concrete environment, particularly in a hot Texas summer, was another.

In the northern section, the new highway will be viewed from high-rise office buildings, one-story shopping centers, and automobiles. The expressway must fit comfortably into the fabric of the city as a good neighbor when viewed from these perspectives.

After the concepts were presented and public response was evaluated, design goals were established and a final cohesive plan developed. The goals were that the facility was to be sensitive to its urban environment, respond effectively to user demands, be aesthetically pleasing to the motorists and adjacent residents and businesses, and be as technically "leading edge" as possible, providing for 21st century smart highways (IVHS) technology.

DESIGN ELEMENTS

Although the design of the North Central Expressway involves the execution of a total design concept, the expressway environment is composed of several distinct elements. These elements are repeated over the expressway's length, providing a rhythm and identity.

- Structures/built elements: bridges, retaining walls, and barriers/railings;
- Signs: structures and panels;
- Lights
- Traffic signals
- Streetscape: stamped tinted concrete, concrete unit pavers, bollards, planters, and special features;
- Landscape: urbanscape, openscape, and linear greenway.

Structures and Built Elements

Bridges

Low-profile bridge structures will be achieved through the use of precast concrete boxes. Although their unit costs are more than some other systems, the boxes represent an actual cost savings systemwide because of the decrease in required depressed-section excavation and retaining-wall height. An aesthetic benefit to the precast boxes is the clean appearance they will give to the edge and underside of bridges. Bridge profiles will be further streamlined by the shadows created from cantilevered overhangs. The angled cap of the retaining walls curving onto the bridge face will reinforce the impression of a sleek structure. Street names located on bridge facades will be a unique orienting device along the expressway. Columns with flared capitals will form a graceful transition to the bridges (Figure 6).

Retaining Walls

Refining the beam-on-column framework developed in Alternative A, the retaining walls establish an organizational motif for the expressway. Their regular pattern is repeated over the length of the expressway, providing a system for the

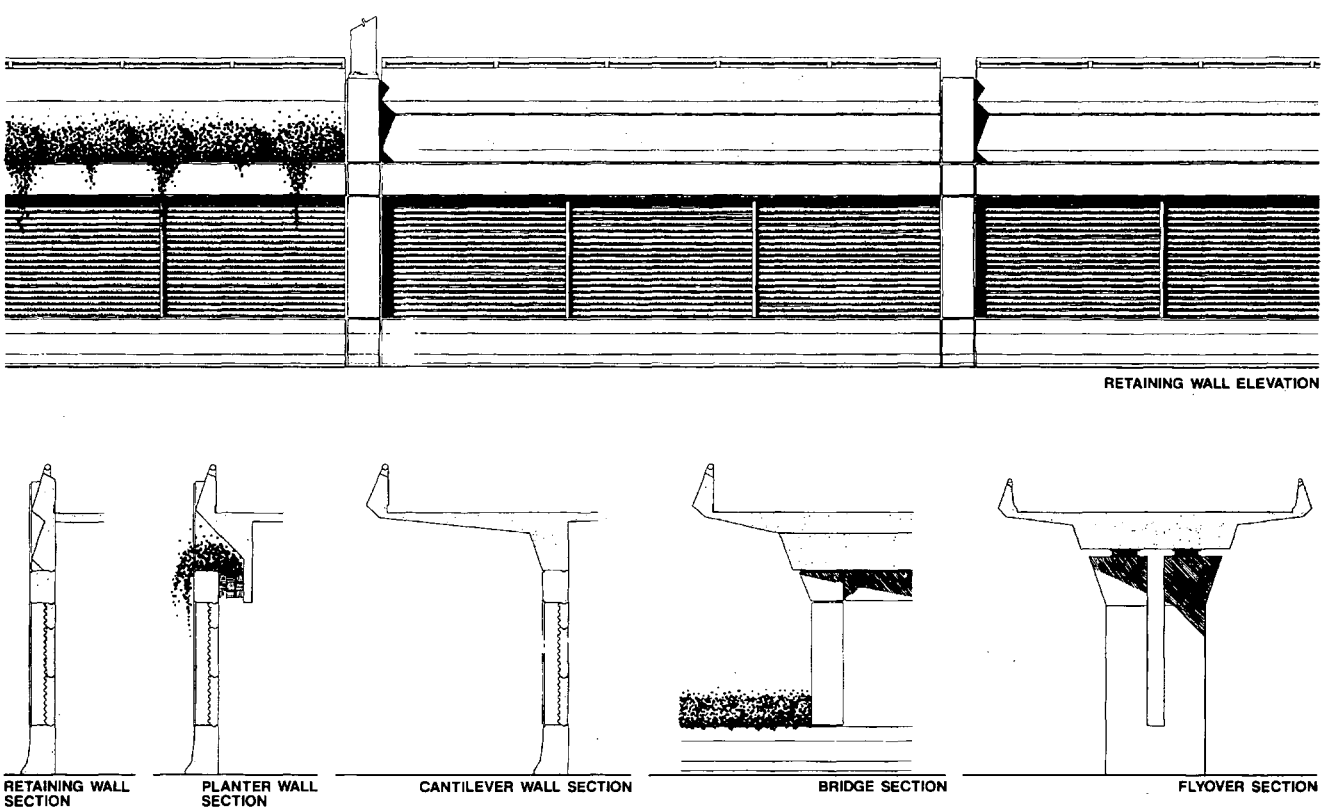


FIGURE 6 Built elements, final scheme.

introduction of all the elements. The retaining walls are designed for maximum efficiency of construction. Changes in the main-lane roadway grade are absorbed by a cast-in-place concrete base that incorporates the Jersey barrier. The precast panels installed on top of the base will be constructed in modular heights, allowing the necessary flexibility to accommodate the varying wall heights.

The deeply ribbed texture of the panels contrasts with the smooth structural framework, a contrast that is reinforced by the use of two colors of paint. Indented planters in the retaining walls provide for cascading plantings. Visible to drivers on both the main lanes and the frontage roads, the greenery in the windows will relieve the monotony of the depressed freeway walls. In addition, where space allows, tree planting will be introduced in shelves that are stepped to the top of retaining walls. The grid pattern of the walls will be extended into the expressway with a freestanding grid between the main lanes and ramps (Figure 6).

Signs

The design of signs for the expressway involves a system functioning on many levels and providing different types of information (Figure 7).

Guide Signs

A principal criterion for locating guide signs in this system is the consolidation of information on single supports wherever possible, thereby eliminating unnecessary supports, which create visual confusion. In addition, signs will not be attached to bridge structures.

As part of the overall expressway design, guide sign supports are a major architectural element. Formed of rectilinear tubing, the supports are an extension of the beam-on-column pattern of the expressway. A field of metal mesh provides a surface to which to attach the guide sign and exit number sign.

Regulatory and Warning Signs

Design criteria result in a large number of individual signs on single supports. Where locations are not limited by specific standards, signs will be consolidated on sign bridges or traffic control supports. Individual supports for these signs are smaller-scale versions of the guide signs and use compatible materials and colors.

Lighting

The North Central Expressway lighting system will use a vertical lamp design, setting an important new standard for highway lighting. The foot-candle level achieved with the vertical lamp is an improvement over conventional highway lighting (Figure 8).

Disc-shaped fixtures with a domed top will be attached to square posts. Mounted closely to the posts, these fixtures are less obtrusive than the typical "cobra head" fixtures used on

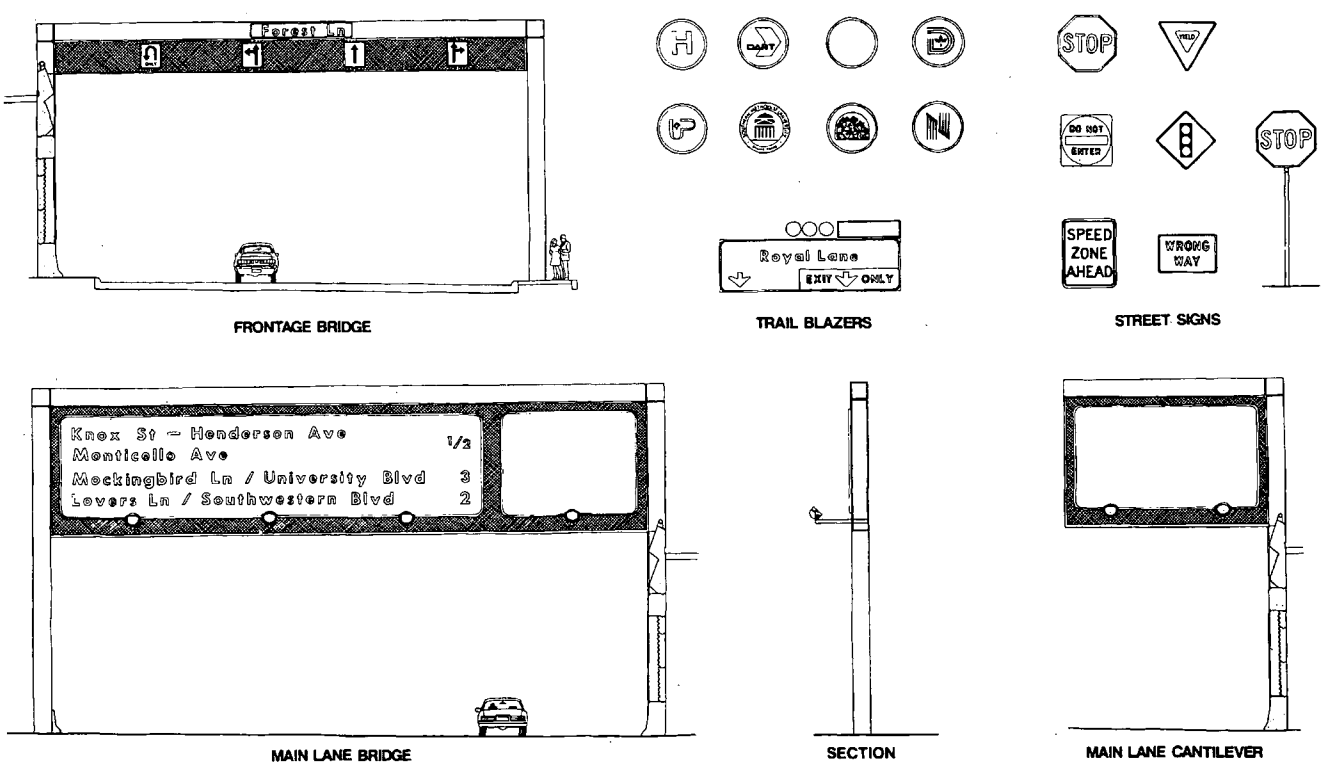


FIGURE 7 Signage, final scheme.

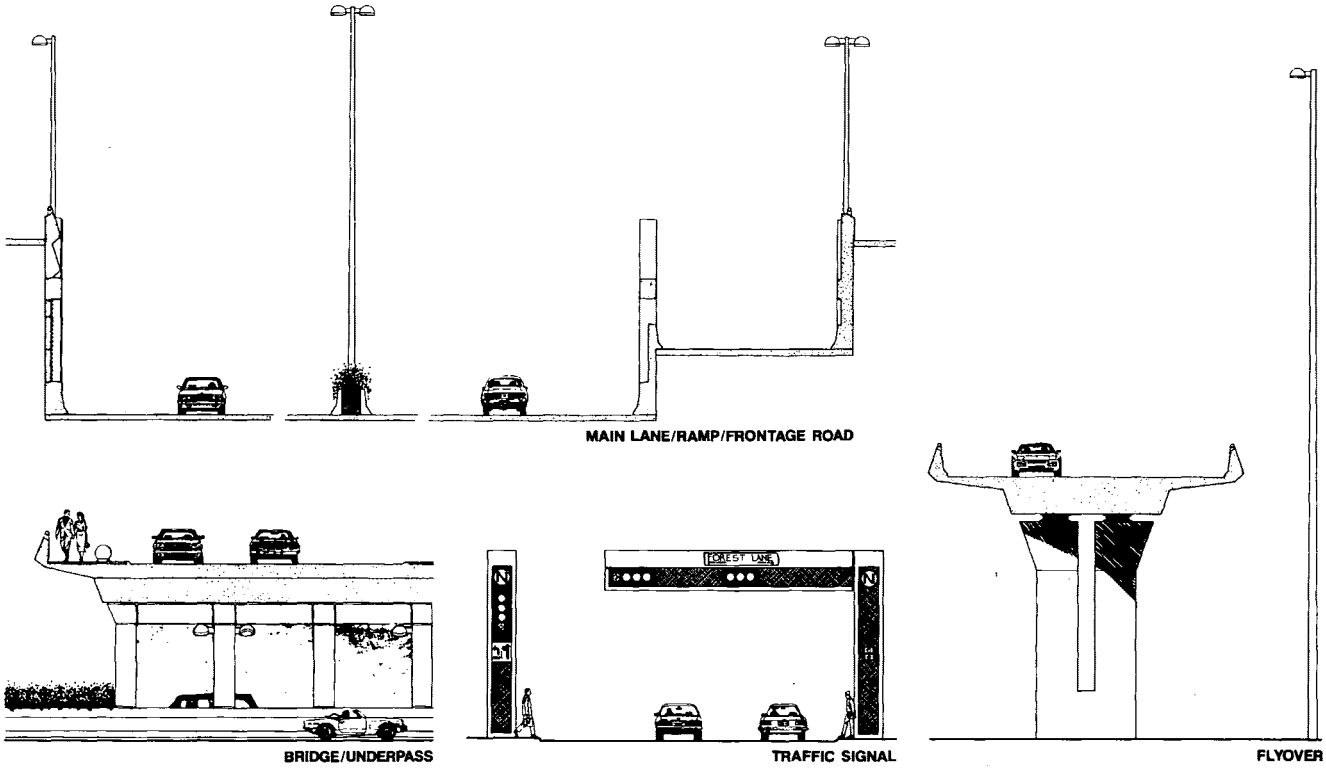


FIGURE 8 Final lighting scheme.

many highways. The dome shape of the standard fixtures will be repeated in fixtures under bridges, on high masts, and on sign supports. Compatible with the rectilinear sign supports, the square posts are part of the "family" of North Central Expressway components.

Set on 50- to 65-ft poles, main-lane fixtures will be located within the center median. Frontage road lighting along the depressed roadway section will be on the pilasters of the retaining wall. The vertical lamp allows wider light distribution with sharp cut-off, eliminating the need for additional edge lighting at on- and off-ramps. Light intended for the roadway will remain on the roadway, reducing intrusive light into the neighborhoods.

Signal Lights

Specially designed traffic signals will be used at all intersections. Repeating the mesh-on-tubular-frame construction of the guide sign supports, traffic signal supports will integrate frontage road signs, street signs, and traffic signals (Figure 8).

Streetscape Development

Streetscape development of the frontage roads and cross streets will continue the highly ordered framework of the North Central Expressway design. The streetscape will also respond to the character of the surrounding city. Elements unique to the expressway are repeated at intersections over its length. These human-scaled elements have been introduced to define spaces and create a sense of place. Streetscape improvements include paving, planters, signs, lighting, and bollards (Figure 9). Some improvements extend throughout the corridor; others are allocated to specific locations.

Pedestrians crossing the expressway corridor will be accommodated by sidewalks on all cross streets. Bridges without U-turns will have 8-ft sidewalks, and bridges with U-turns will have 16-ft sidewalks. Bollards lining the sidewalks will define the edge between pedestrians and vehicles and provide a pedestrian-friendly environment.

Landscape Development

Landscape in and on Structures

Many cross-street bridges will incorporate special landscape developments. These improvements are allocated by bridge location and driving lane configuration.

Median and Edge Plantings

In the median between lanes of expressway traffic, a double row of the safety barriers will be separated to allow planting in between. With provisions for drainage and irrigation, the utilitarian safety barriers will create a pocket for plant material that will soften the environment, shield drivers from oncoming headlight glare, and cut down on rubbernecking at incidents.

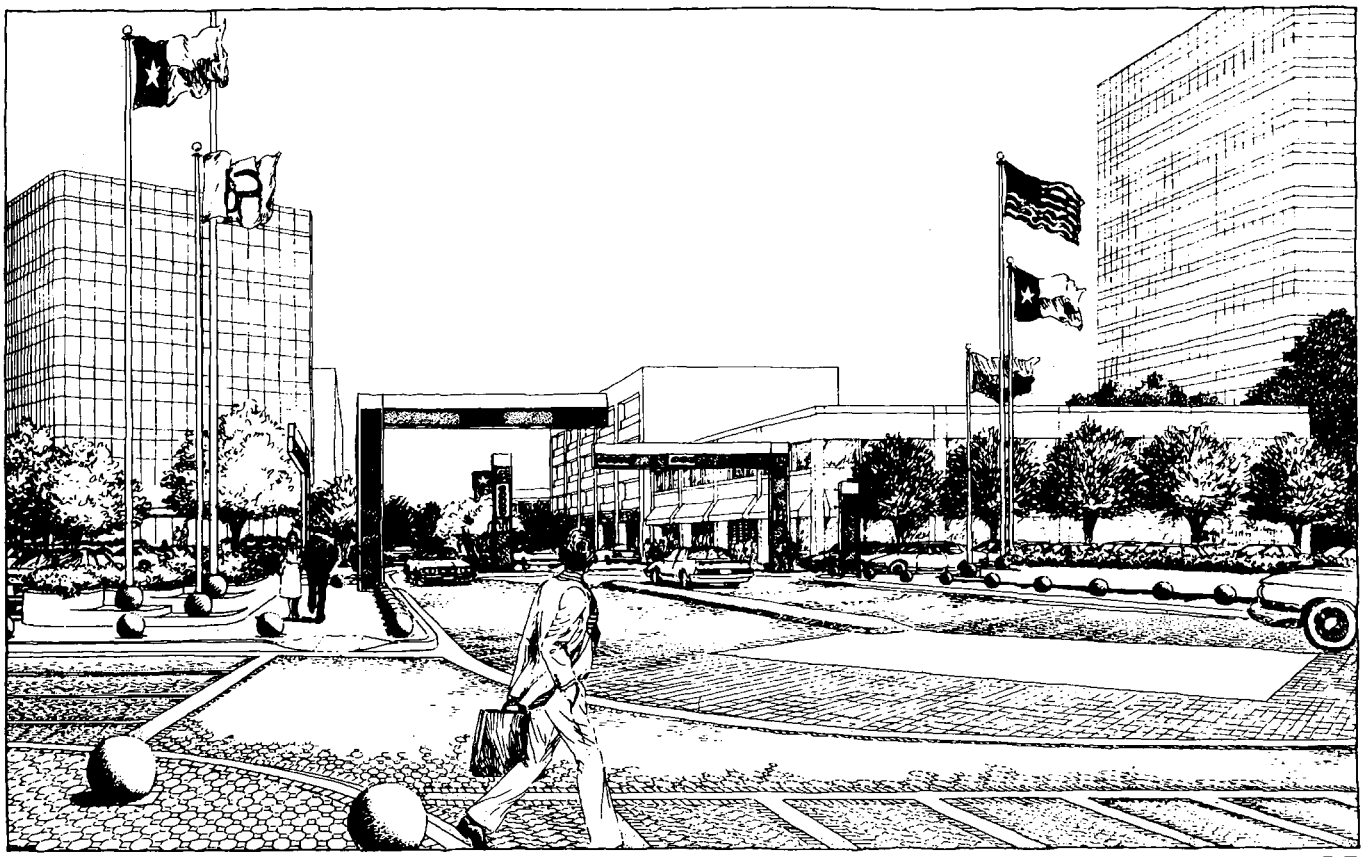


FIGURE 9 Cross Street bridge.

Frontage Road Landscape

Landscape opportunities along the frontage roads are narrow. Planting patterns and materials for the expressway are bold and sometimes highly organized. Where trees and shrubs are adjacent to vehicular traffic (but outside safety clear zones) and particularly vulnerable to being hit, planting patterns are more informal. One tree removed from a cluster of trees will be much less noticeable than a tree removed from a formal planting.

Irrigation and Maintenance

Installation of a maintainable irrigation system is essential to the successful development of the corridor.

MULTIMODALITY

Concern for other transportation users of the expressway received strong attention. Capacity and user-friendliness for nonmotorized traffic—pedestrians, bicyclists, wheelchair users, and so forth—were considered. All the street crossings, including the 15 on bridge structures (where the main lanes are depressed below), are very wide. This is good for vehicles, but can be intimidating for pedestrians, bicyclists, and others. To bring a sense of order and safety to the crossings, tinted concrete pavers, bollards, and irrigated, raised planters were designed. These features create an ambiance of pedestrian-friendliness, which should make pedestrians and cyclists less apprehensive about crossing the expressway.

There are eight designated on-street bike routes crossing the expressway. For all the designated crossings, exterior lanes of 14 or 15 ft are provided.

"Intermodality" is the transportation buzzword of the 1990s. The potential for students to bike to the suburban DART rail station, stow the bike on board, travel the rail line, disembark at the DART Mockingbird Station, and then bike a mile across the expressway to the SMU campus seems to be a perfect example of functional intermodalism that is successfully incorporated in this design.

In addition to the on-street bike routes, two hike and bike trail crossings of the expressway are being accommodated. In the reconstruction of the existing trail at White Rock Creek, the plans established a maximum number of calendar days of safety-related trail closure allowed to the contractor in the 3-year contract.

The impacts and requirements of the Americans with Disabilities Act are still being analyzed, but extensive use of sidewalks with handicapped-accessible ramps has already been included in the design. The pedestrian-friendly intersection design is expected to be advantageous for handicapped users.

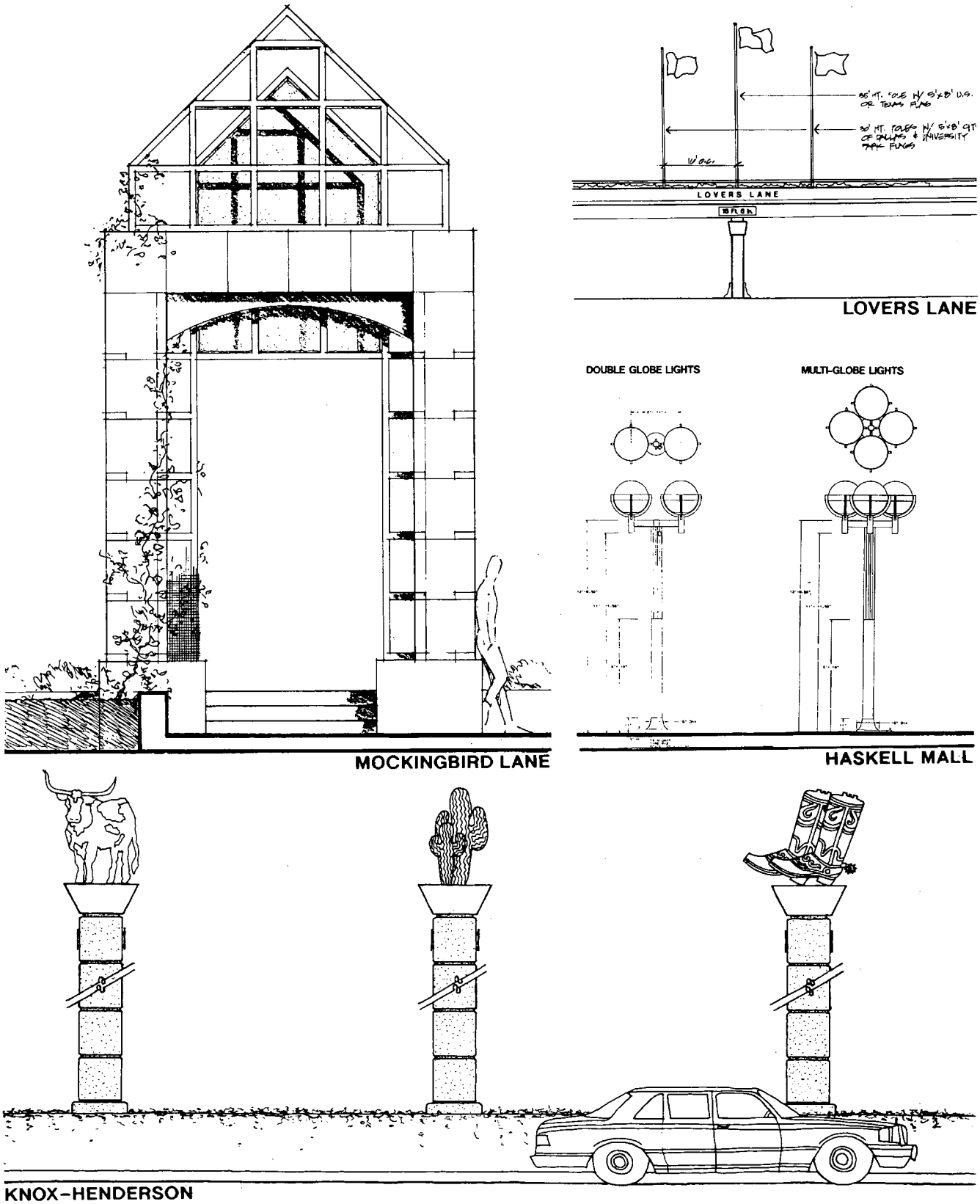


FIGURE 10 Special features.

SPECIAL FEATURES

In addition to the usual pedestrian-friendly design, five intersections also have special architectural features to highlight the major crossings, provide architectural interest for motorists and pedestrians, provide visual (and somewhat subliminal) landmarks to prevent disorientation for drivers in the 6-mi concrete canyon, and allow an expression of the community within the expressway.

Each special feature is different. At Haskell Mall, the enhancement will be decorative lighting reflecting special lighting in a planned development either side of the expressway (Figure 10). At the Knox-Henderson structure, the special feature is lighted twin colonnades of five columns on each side situated in a deck planter. Each column will be topped by a metal sculpture reflecting the electric mix of neighborhoods near this crossing (Figure 10).

The next special-feature crossing is Mockingbird Lane, another community "node." At opposing diagonal corners of the deck in the planter areas, twin pavilions will be constructed. The pavilions are purely architectural. They will be lighted at night and provide a gateway effect for motorists crossing the expressway in either direction on Mockingbird Lane (Figure 10).

At Lovers Lane, three flagpoles with up-lights will be installed in each bordering deck planter (Figure 10).

The Northwest Highway Interchange creates significant open landscape areas around the interchange ramps. A major developer is expected to participate (at his expense) in additional enhancements to the interchange landscaping. Two 130-ft-tall vertical sculptural elements are planned for diagonally opposite sides of the interchange (Figure 11). A sculptor will design these elements to provide a focal point for travelers along the expressway and along Loop 12. A major regional shopping center is located at this intersection and it is a significant areawide traffic generator.

The archaeological relocation of approximately 1,500 Civil War–era graves from one of the largest African-American cemeteries in the United States is also being done on this project. The archaeological research will open a window into an important unrecorded aspect of Dallas's past.

ADVICE BASED ON EXPERIENCE

Any time new concepts are incorporated into complex construction activities, resistance can be expected. And any deviation from long-accepted standard practice opens up opportunities for construction glitches. In general, the more dramatic the new concepts, the greater the resistance and the opportunity for problems. Some measures can be taken to mitigate these difficulties.

First, conceptual design and architectural input must have sufficient lead time to develop the range of optional ideas that may be placed "on the table." Then, additional time is needed for soliciting public input regarding the magnitude of

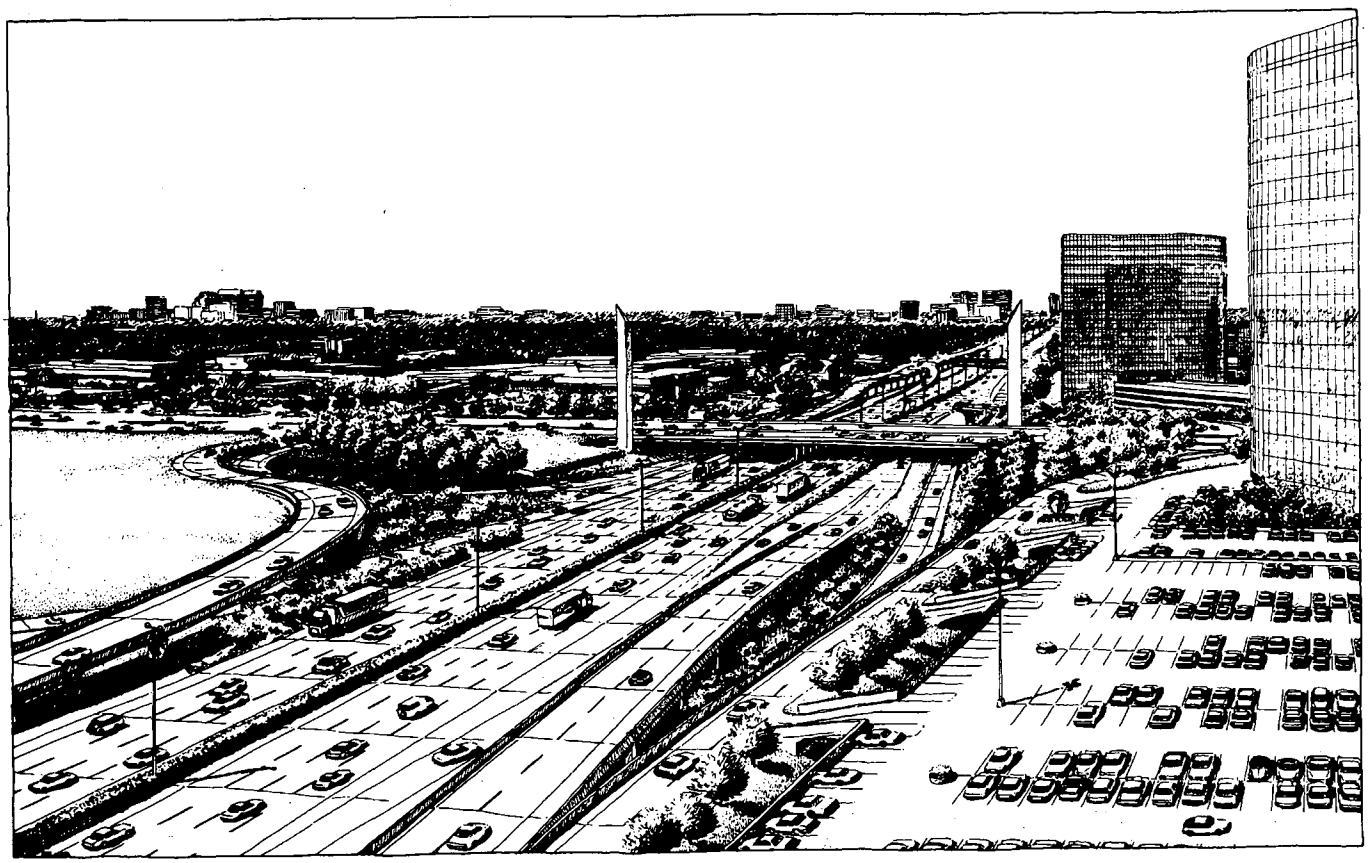


FIGURE 11 Loop 23 interchange.

enhancements and the specific designs favored. Effectively soliciting the input of the community can often be difficult. A professional-level public affairs program is needed for most large urban areas to develop the type of sophisticated audience targeting wanted. It is of particular importance that all interest groups are included (e.g., bicyclists, environmentalists). In general, some competing goals will arise between commercial and residential interests. Good political skills are needed within the project management and public affairs staffs. Only after the project concept and architectural and landscaping ideas have jelled should the engineering designs begin. Simultaneous architectural and engineering work starts will inevitably result in numerous revisions for both disciplines, which is inefficient and costly, and also can create an adversarial relationship between the architects, planners, and the design engineers.

It is also very important to bring in construction and maintenance personnel for their input regarding new concepts and modifications of existing design/construction practice. Input from construction personnel can be valuable in reviewing constructability issues and can often provide good suggestions on how modifications can improve constructability. Maintenance forces usually have ultimate responsibility for upkeep of the finished facility, so their input can also be valuable in creating concepts that work for the long term.

Most major highways pass through several local jurisdictions. It is critical to consult with appropriate local staffs to apprise them of various concepts under consideration and solicit their concurrence and support. In many areas, local jurisdictions will have a role in maintenance and upkeep of the facility, and it is important that the plan has their blessing.

In addition to the official local jurisdictions, adjacent commercial and residential property owners are often interested in and willing to participate in installation and maintenance of special features such as landscaped areas. Ideally, the level of such participation will have been ferreted out in the public involvement process. It is also important that the ultimate plan include a fall-back maintenance level in case early private-sector enthusiasm dwindles or financial circumstances change.

CONCLUSION

The area of enhancements gives a new and exciting opportunity for transportation projects if approached with enthusiasm and a positive attitude. Unquestionably, the status quo may be disturbed by these new ideas. But the traveling public, whether traveling on foot, by wheelchair, by bicycle, by car, by bus, or by truck will benefit from these efforts. It should

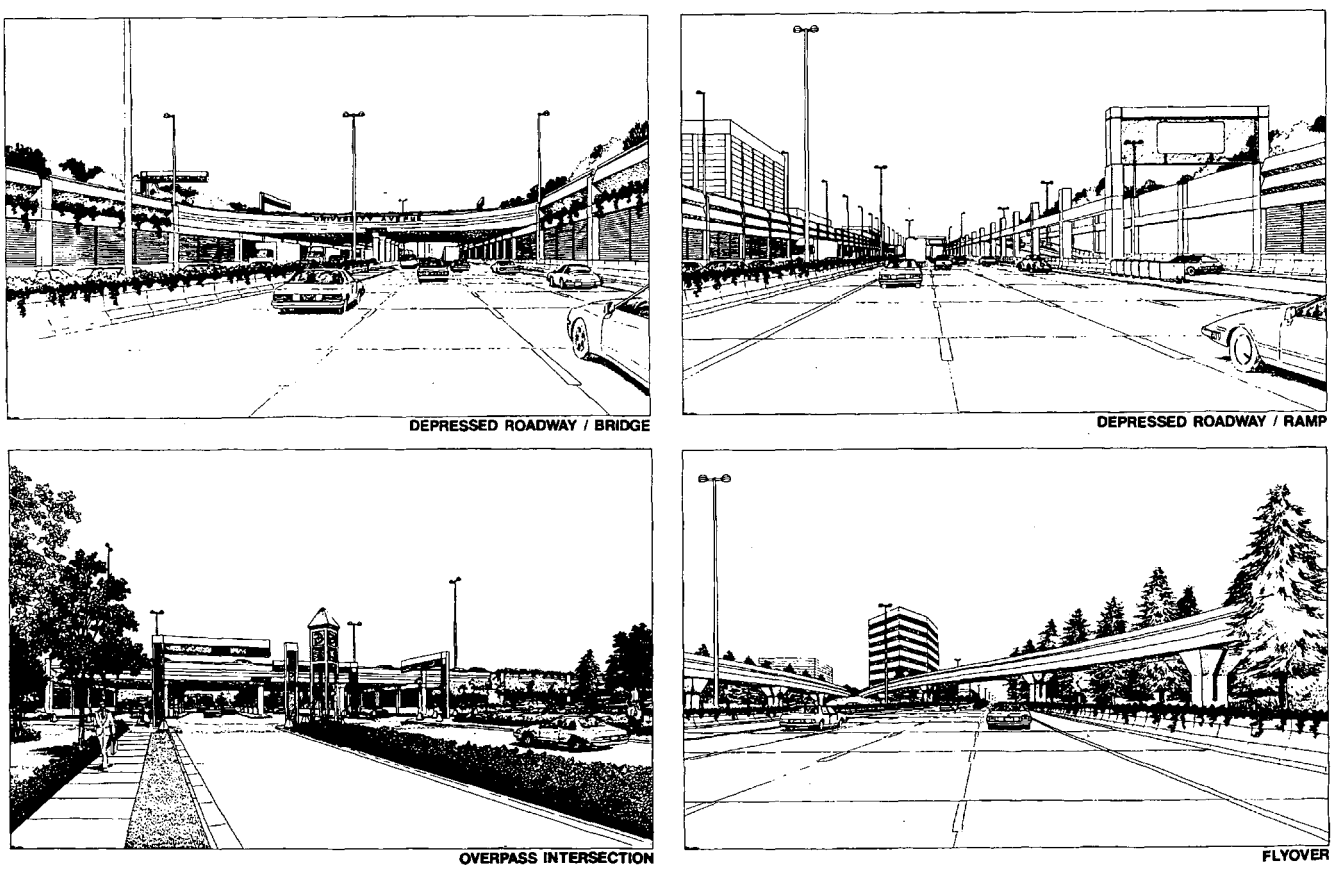


FIGURE 12 Perspectives, final scheme.

be recognized that major transportation facilities have a significant impact on the urban fabric. All residents and users are affected by the quality of these facilities and by the sensitivity with which they have been placed in their environment. Years ago, Main Street was the focal point of a community, usually built wider and with fancier street lights than the more mundane thoroughfares. These streets helped give communities a sense of place. Today, and in the future, the same attention to the importance of new or rehabilitated transportation facilities as a source of civic pride can be achieved by proper application of enhancements (Figure 12).

ACKNOWLEDGMENTS

The authors gratefully acknowledge the architectural and planning work of Hellmuth, Obata and Kassabaum, Inc., Architects, Engineers, Planners & Landscape Architects; and Carr, Lynch, Hack and Sandell, Urban Design Consultants, under contract to TxDOT for providing graphics and research material for this paper.

Publication of this paper sponsored by Committee on Landscape and Environmental Design.

Visual Prioritization Process

LOLA EILEEN MASON

As part of the design of every roadway or other corridor construction/reconstruction project, environmental concerns about the visual resources of the corridor need to be considered. Visual management guides for mitigation of the visual resources in corridor design vary with each governmental entity. Existing guidelines cover only planning, with little or no guidance for implementation. The result is often a final design consisting of an even distribution of mitigation measures over the entire corridor. The guides give little consideration to the design engineer being able to understand the process and the decisions made for the final design. There are variations in the significance of visual elements along a corridor. Available funding can be more wisely used and environmental concerns better met by varying the amount of mitigation in various sections along the corridor according to the visibility of proposed impacts. Numerical values based on formulas would not only help determine the variations between the sections but would also bridge the communication and understanding gap between the landscape architect and the engineer. This became the basis for the preliminary development of the Visual Prioritization Process (VPP). VPP is a planning and implementation guide for prioritizing units and visual elements along a corridor for mitigation and funding.

In 1988, Joanne Gallaher, a landscape architect with Wheat-Gallaher and Associates, was hired by the Coronado National Forest in Tucson, Arizona, to conduct a visual analysis and design the mitigation for the reconstruction of the nearby Mt. Lemmon Highway project (see Figure 1). Gallaher had experience with many visual resource management guides that were strong in planning the necessary level of mitigation for the area, but offered no guidance in the implementation of the mitigation. During the first phase of the project, she developed the Visual Prioritization Process (VPP) as a planning and implementation guide based on the Forest Service Visual Management System (VMS) (*1*) and other agency visual management guides for design. VPP recognizes that, within a visually managed area, variations of visual resources occur. Because of this, different levels of mitigation can achieve the same visual management objective. This approach is more cost-effective than a blanket or uniform mitigation treatment. VPP makes it easier to express the need for the mitigation to the design engineer and others in a manner they understand through the use of numerical scores to represent the variations.

After completion of the first phase of the Mt. Lemmon Highway, VPP was the recipient of a National Endowment of the Arts Federal Design Achievement Award and many requests were received for more information on the process. In 1989, Gallaher applied for Coordinated Federal Lands Highway Technology Implementation Program (CTIP) funding from the U.S. Department of Transportation, FHWA. VPP needed further refinement and distribution. The CTIP committee recognized that road construction and reconstruction projects are under ever-increasing scrutiny and criticism of aspects of environmental impacts and costs (see Figure 2). Providing measures in road projects to mitigate environmental concerns, including visual quality objectives, often threatens the economic viability of needed projects—especially in visually sensitive terrain. VPP was considered a means of achieving the necessary mitigation in a cost-effective manner. Based on this, VPP was chosen as a CTIP study. The study was to be conducted by an interagency task force of the CTIP agencies. CTIP agencies are U.S. governmental agencies that manage federal lands and highways, such as the National Park Service, Forest Service, FHWA, Bureau of Land Management, and Bureau of Indian Affairs. The Forest Service San Dimas Technology and Development Center (SDTDC) was chosen to manage this project.

U.S. Department of Agriculture Forest Service, San Dimas Technology and Development Center, 444 East Bonita Avenue, San Dimas, Calif. 91773-3198.

PROCESS REFINEMENT

For the CTIP study, Gallaher originally proposed to refine, develop, and distribute associated publications and visual aids, including videos, to facilitate use of VPP under a broader range of conditions by highway design agencies. As a start, further work was needed to refine the process to accommodate conditions not handled by the original formulas. Refinements relating to distances, angles, and rankings within the models were required. Gallaher stated that through the use of other publications—including FHWA's *Visual Impact Assessment for Highway Projects* (*2*)—comparisons and possible alterations could be made.

The following were the specific objectives to be met through the CTIP study:

1. Review VPP development, background, and use.
2. Review, redefine, analyze, and revise factors as needed to equalize the values. Add adjacent land use, such as campgrounds, trails, and residential development, as well as topographic analysis and "seen areas" to the matrix.
3. Review, analyze, and revise rankings, formulas, and calculations to standardize and validate the process. Investigate and develop the capability of weighting the various factors.
4. Ensure the applicability of VPP to other linear projects with visual management impacts, such as roads, trails, and utilities.
5. Develop a package for technology transfer of VPP, such as a publication, computer program, or slide/tape presentation to be distributed to CTIP agencies and others that may request it.

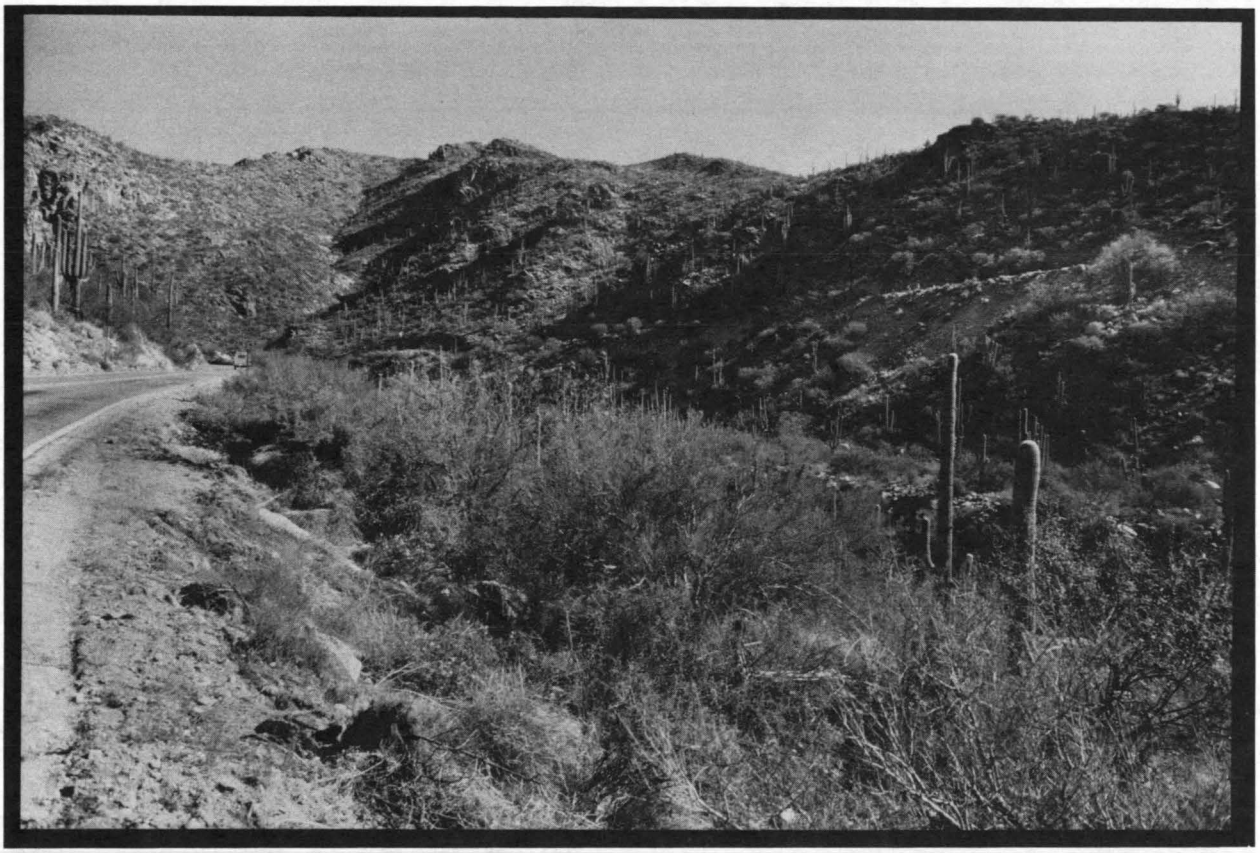


FIGURE 1 Mt. Lemmon Highway, Tucson, Arizona.

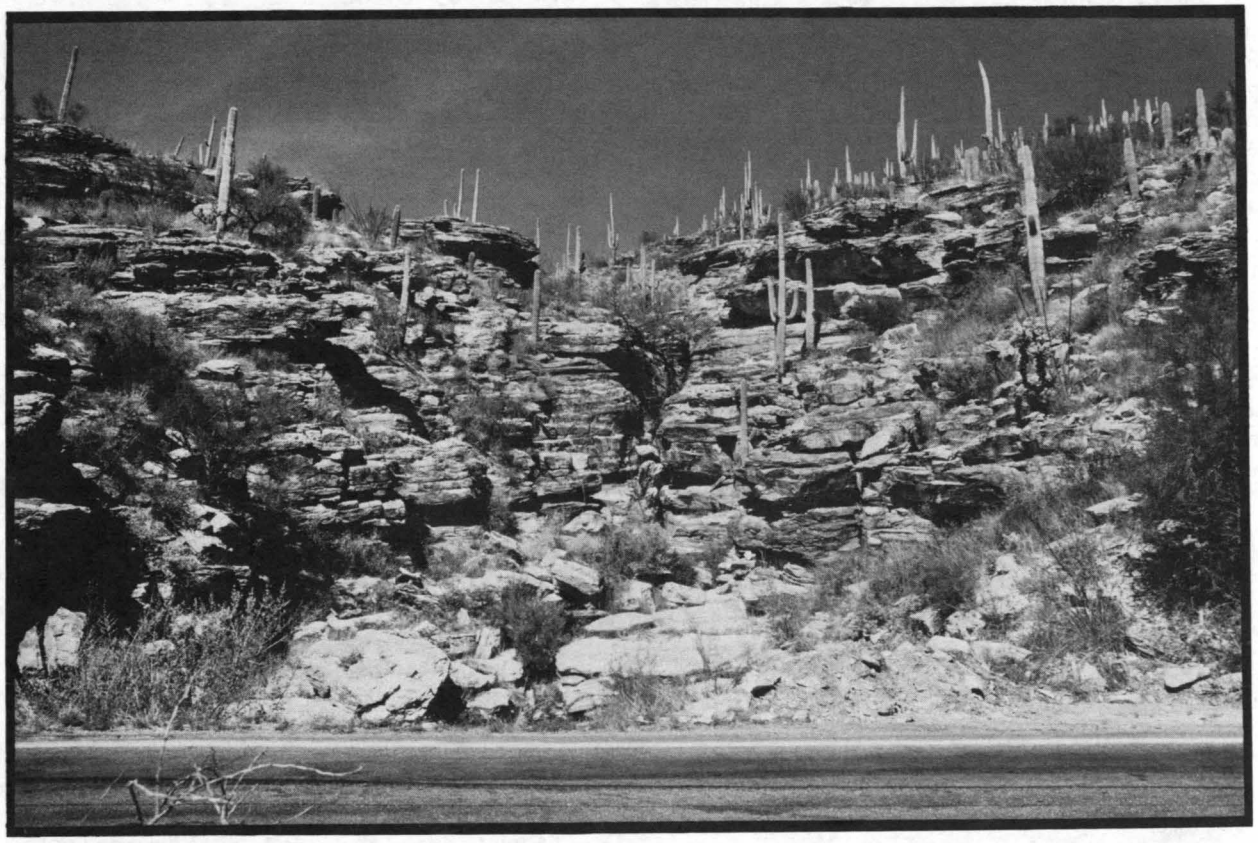


FIGURE 2 Mt. Lemmon environment, possible impacts.

FIGURE 3 Najavo bridge project.

VPP has been used on six different projects (see Figure 3). Landscape architects and civil engineers agree that VPP gives direction to a reasonable and effective mitigation approach. VPP gives guidance on distributing mitigation fairly by showing areas that need more mitigation and those that need only the minimum level. Other guides only help to determine the visual objective, resulting in a homogeneous mitigation. This often results in excessive mitigation in some areas and under mitigation in other areas. At the design level, VPP results in varying mitigation levels that are balanced between the visual objective and the funding available. At the planning level, VPP is used to guide the project's alternative selection and preliminary design, which also leads to less required mitigation. VPP successfully links land management planning to project design and implementation and construction. Because VPP uses numerical scores, it is much easier to communicate where the mitigation is needed and why.

KEY POINTS OF CONCENTRATION

The initial step of the study was to form an interagency task force. Those who were requested and agreed to become members of the original task force were

- Joanne Gallaher, Landscape Architect, Coronado National Forest;
- Mark Taylor, Civil Engineer, FHWA;
- Tom McGovern, Civil Engineer, McGovern, MacVittie, Lodge & Dean;
- Jill Easley, Landscape Architect, Colorado Highway Department;
- Gary Johnson, Landscape Architect, National Park Service;
- Steve Galliano, Landscape Architect, Forest Service-Southeastern Region;
- Bill Makel, Forester, SDTDC; and
- Lola Mason, Civil Engineer, SDTDC.

The original one-page description of VPP has been developed into a user's guide (*3*) (unpublished data) with four accompanying case studies in mountain, urban, and rural United States settings. The task force concentrated on enhancing VPP in four key areas.

1. The formulas used in VPP. By incorporating standard formulas, VPP would become easier to understand and develop into a software package. In addition, design engineers might be able to use this numerical approach.
2. The use of references to other visual management manuals. It was important for agencies to feel comfortable using VPP and references for how VPP can be used with other processes.
3. The use of VPP nationwide. It was important to concentrate on incorporating the southern and eastern areas of the United States. Most visual resource manuals are based

on conditions found in the West, which can be very different from the East in terms of visual management.

4. The need to incorporate all the steps of corridor planning. Originally, the description of VPP concentrated on the variables and values used to prioritize the visual elements and units during design. The guide had to cover all the steps from area-wide planning to corridor construction.

The draft of the user's guide will be sent to landscape architects and engineers in various agencies throughout the United States for peer group review. The comments will be incorporated into the final publication.

VPP USER'S GUIDE

The initial work of the CTIP project was to enhance the process and write a user's guide. The guide needed to be written so that it could either be incorporated into other agencies' visual resource management guides or be used as is. To meet this need, the manual would cover both planning and implementation.

The planning process was developed in accordance with the Forest Service VMS (*1*). Like many other agency guides, VMS is a large-scale visual inventory and management process. It is used to inventory and analyze existing visual resources and then determine management objectives (see Figure 4). The frustrating aspect for landscape architects and engineers is that these agency guides result in an overall objective but no guidance on implementation. The design becomes a single mitigation measure for the entire corridor without taking into account that the visual resources are not homogeneous. This typically results in a cost-prohibitive design.

VPP incorporates the planning process to determine a visual management objective based on existing funding resources. It also can be used to inventory and analyze proposed visual resources based on the engineer's proposed design. What sets VPP apart from other visual management systems is that it can be applied to the project-level implementation stage. It is a means of numerically showing and comparing the proposed impacts to the importance of visual resources within units along the corridor.

Units are sections making up the corridor that consist of similar significant visual resources. Priority levels can be assigned to the proposed visual elements within units, based on the numerical values. Mitigation measures can then be varied, based on the priority levels, yet still guarantee that a minimum level of mitigation is always met (see Figure 5). In this manner, the visual objective can be achieved with varied levels of distribution, resulting in a cost-effective design that meets the visual goals of the project. As stated in the user's guide,

> The project level implementation phase of VPP allows the designers (i.e., civil engineers, planners, and landscape architects)

FIGURE 4 Various existing visual resources.

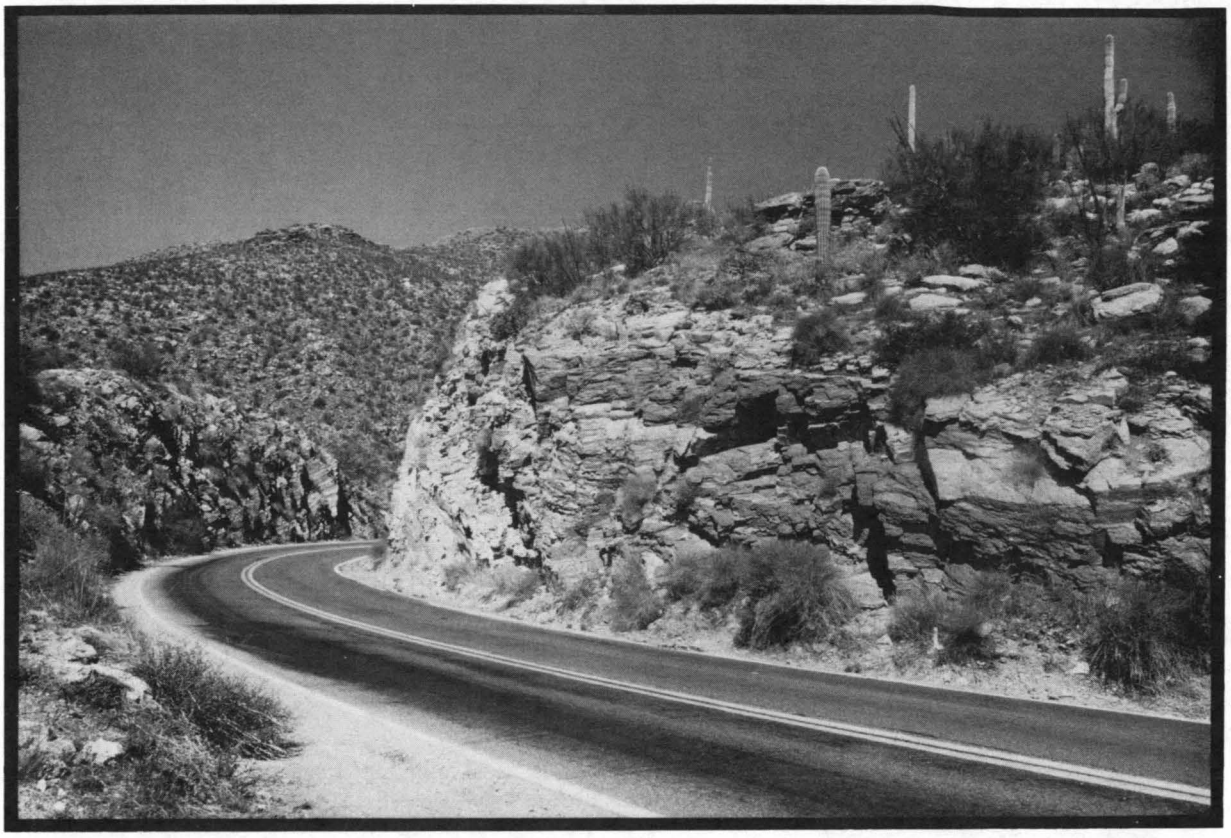


FIGURE 5 Mitigation measures for specific area.

to assess their design in regards to visual resource impacts, modify the design, and incorporate mitigation measures to lessen the impact. By prioritizing areas along the corridor, the designer can allocate the budget proportionately for mitigation measures based on highest importance per dollar. VPP creates a design process which balances the work of the civil engineer with that or the landscape architect and the objectives of aesthetics with those of safety and cost.

The manual is written in three sections based on the process phases. The phases are area-wide planning, corridor- or project-level planning/design studies, and project-level implementation. The manual also includes four case studies. The first case study is the Mt. Lemmon Highway, located in the mountainous, arid region of the southwestern United States. It is an example of how VPP can be used only to prioritize mitigation for cuts and fills, the most significant visual impacts on the project. The second case study is the Navajo Bridge, located in the high plateau, arid southwestern region of the United States. It is an example of how VPP can be used for all visual impacts proposed on the project. The third case study is River Road located in Tucson, Arizona. It is an example of how VPP can be used for all visual impacts on a project in an urban area. The fourth case study is the Natchez Trace, located in the southeastern region of the United States. It is an example of how VPP can be used to compare two alternatives, a bridge and an at-grade road.

For all the case studies, VPP gave much more insight than traditional guidelines on where highly sensitive visual units were located. This helped during discussions with the engineer on various proposed designs and specific changes that would benefit the project visually. VPP was extremely helpful in deciding between alternatives. It measures the proposed impact on existing visual resources, as well as the addition of proposed new visual resources. It was also extremely helpful in determining the impacts on possible funding reductions. The numerical values of each section quickly and easily displayed why the mitigation funding was necessary to meet visual management objectives. Mitigation varied in line with natural variations of each area. As a result, the trade-off from a reduction in mitigation funding could be evaluated.

PHASE 1—AREA WIDE PLANNING

The initial phase of VPP is the area-wide planning and determination of the visual management objective or the visual goals for the project area (see Figure 6). Many times, this phase is completed with no planned corridor project but with the possibility of future projects in mind. The four steps in this phase are directly from the Forest Service VMS. Other agency systems have very similar steps, which are in the user's guide. The steps encompass determining the natural, cultural, and historical resources of the area; defining the uniqueness of the visual elements; defining the concerns of the user; and defining the management objective for the visual resources.

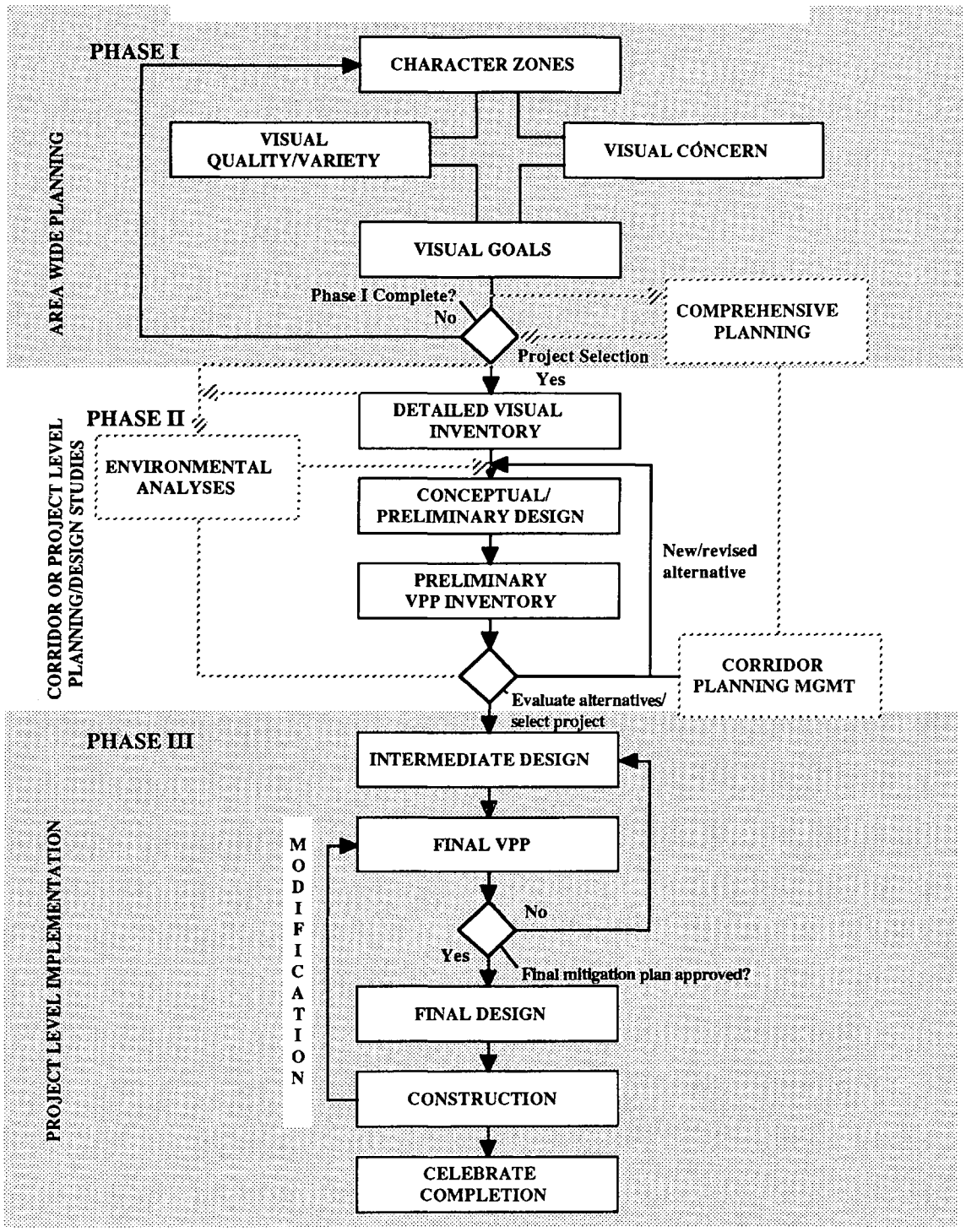


FIGURE 6 VPP flowchart.

As part of the first phase, the comprehensive planning for the area would be considered; this would include management zoning, land management planning, and any capital improvement programs. Other resource inventories previously performed for the area would be considered. This phase is a part of most visual management guides for design and must be completed to successfully accomplish the remainder of the process.

PHASE II—CORRIDOR OR PROJECT LEVEL PLANNING/DESIGN STUDIES

The results of the first phase—the definition of the existing visual resources and visual goal—are used in the second phase, called the corridor- or project-level planning/design studies (see Figure 6). During this phase, designs of route alternatives are considered and compared. Typically, for corridor designs,

the visual analysis is not adequately considered until after the engineering design alternative is chosen. The visual analysis performed during this phase could be valuable input to the engineering design and potential alternatives and mitigation costs derived later. The evaluation of the alternatives could then be based on the necessary mitigation and funding for the visual management as well as for the corridor design. During this phase, it is critical that a strong communication link be maintained between the landscape architect and engineer. Each party must understand the other's analysis and design of each alternative.

During this phase, input from the environmental assessment (EA) or environmental impact statement (EIS) would be included. An EA or EIS is a report covering an investigation into the effects of a construction project on the environment in that area. It is a requirement for all U.S. governmental agencies to document environmental concerns to determine that all environmental rules and regulations will be met. There are three steps in this phase: determining the site-specific resource inventory; designing the preliminary designs or route alternative; and determining a preliminary inventory, which is the basis for the numerical scores.

The preliminary inventory is used to determine the prioritization and estimated costs for the units along the corridor, based on the new visual elements and loss of existing visual resources resulting from each preliminary design or route alternative being considered (see Figure 7). There are 10 tasks that basically cover the completion of several forms and the validation and use of the forms' numerical scores. The tasks begin with listing the significant resources and defining the variables and values for the numerical scores (see Table 1). The variables are determining viewing distances (see Table 2 and Figure 8), calculating visual element sizes, determining horizontal and vertical viewing angles, calculating length of viewing time, and determining visual element backgrounds.

At this point, forms are used to list each visual element and its variable values, which are totaled to determine Visual Priority Levels (VPLs) and Unit Totals (UTs). These values are then field verified (see Figure 9). From the unit totals, the Total Visual Change (TVC) and the Net Visual Change (NVC) are calculated. Mitigation measures are designed and distributed throughout the project according to combinations of some or all of the following factors:

- Units where TVC and NVC are highest;
- Units where significant positive and neutral visual elements that will be lost are highest;
- Units where detrimental new visual elements are highest;
- Units where highest visibility will occur (highest VPLs per negative element), where opportunities for enhancing positive elements and views remain, and where increasing visual quality and variety are greatest;
- Units where visual concern is highest; and
- Each element or unit's importance/cost with the ranking (total unit value) being the importance. The higher the importance/cost, the wiser the use of funds.

The level of mitigation measures distributed shall not be less than the minimum level required to meet visual goals. The tasks are completed with a preliminary cost estimate and evaluation of the overall mitigation plan.

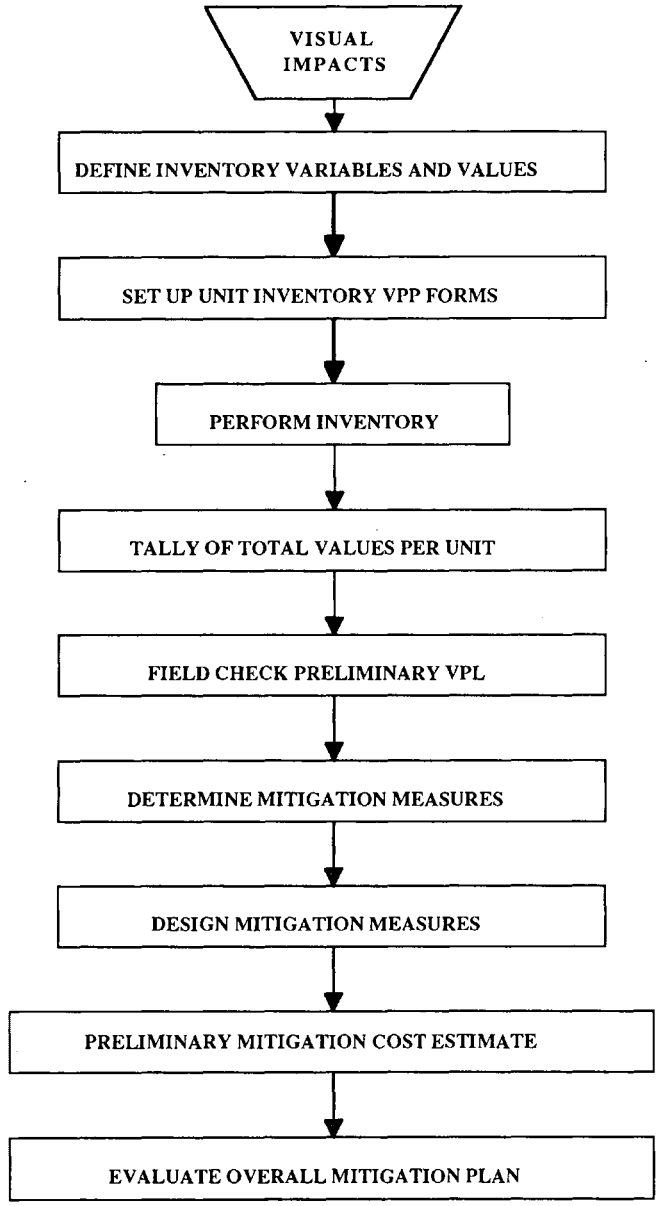


FIGURE 7 VPP preliminary inventory flowchart.

This completes the second phase. Before going on to the third phase, the alternatives need to be evaluated and a design or route alternative selected. The EA or EIS should be complete and included in the evaluation. During the evaluation, the decision may be made to design a new alternative or revise an existing one. As part of the evaluation, the planning and management of the corridor will need to be considered. This includes planning the road system or utilities and future routes on the corridor.

PHASE III—PROJECT LEVEL IMPLEMENTATION

After an alternative is selected, the third phase, project-level implementation, begins (see Figure 6). There are five steps, beginning with the detailed work on the chosen alternative. The design engineer and landscape architect need to work

TABLE 1 Inventory Variables and Numerical Scores

INVENTORY VARIABLES	NUMERICAL SCORE
1. Distance from the viewer:	N/A
Foreground: up to 660' (1/8 mile)	
Middleground: 1/8 mile to 3 miles	
Background: 3 miles and greater	
2. Magnitude:	
0 - 600 sf	1
600 - 4,000 sf	2
4,000 sf+	3
3. Angle of the view:	
46 degrees - 90 degrees	1
16 degrees - 45 degrees	2
0 degrees - 15 degrees	3
4. Duration of the view:	
0 - 7 seconds (less than or equal to 300')	1
7 - 12 seconds (300' - 500')	2
12+ seconds (500'+)	3
5. Silhouette condition:	
No silhouette	0
Background is vegetation	1
Background is sky	2
6. Aspect:	
Angles flat to away from viewer	1
Angles 45 degrees to flat	2
Angles vertical to 45 degrees	3

closely through this step so that each has a chance to determine design and funding changes that will result from the other's design changes. The phase continues with the final determination of the prioritization of the units, which is the basis of the final mitigation measures and is similar to the third step of the second phase.

The final mitigation plan goes through approval. The approval is mainly based on evaluation of the mitigation costs and how well the goals are met. At this point, the fine tuning between the engineering design and visual management concerns should occur. The final design, which incorporates the concerns of engineering and visual management, is completed and is now ready for construction. Construction impacts are evaluated with the understanding that new field conditions may arise that may not have been addressed previously. New decisions can be made to ensure that the goals are being met consistently. Modifications may need to be made, which could be based on the previous inventory.

SUMMARY

VPP is a means of meeting a corridor area's visual management goals, while targeting project mitigation and funding by

TABLE 2 Determination of Distance Zones

DESIGN SPEED (mph)	FOCUSING DISTANCE (ft)	ANGLE OF VISION (degrees)	PERIPHERAL ANGLE (degrees)
30	800		
40	1,100	37	60
50	1,400	29	55
60	1,800	20	45

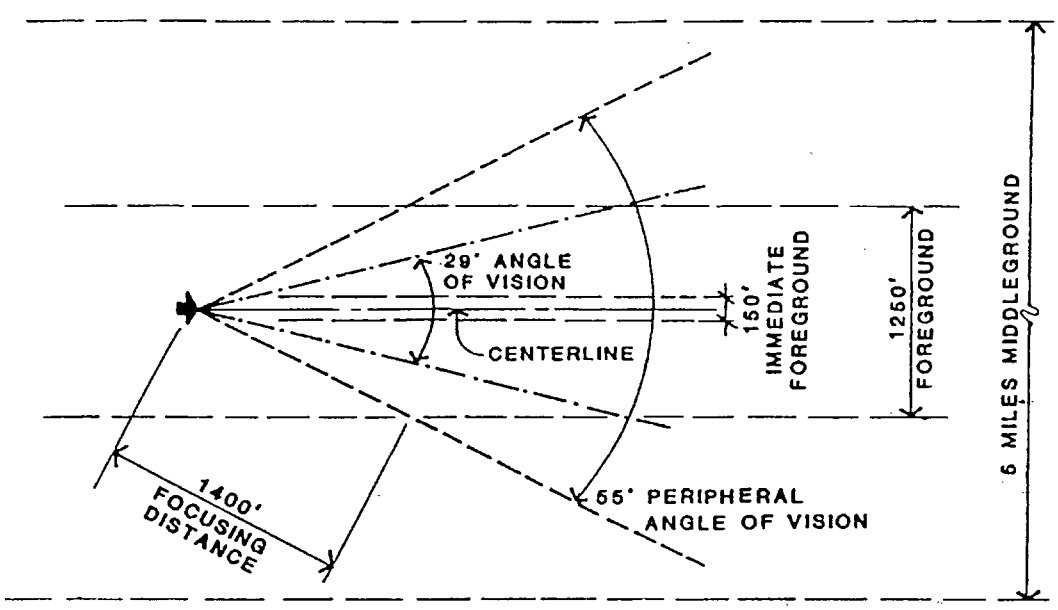


FIGURE 8 Determination of distance zones.

UNIT NO. A, B

STATION

UNIT VPP INVENTORY—NEW VISUAL ELEMENTS

CUTS	MAGNITUDE				ANGLE HORIZONTAL				ANGLE VERTICAL				DURATION/ VISIBILITY				SILHOUETTE				ASPECT				SUB TOTAL	TOTAL/ ELEMENT
	I	F	M	B	I	F	M	B	I	F	M	B	I	F	M	B	I	F	M	B	I	F	M	B		
A1 1+00 - 2+00	1	2			1	1			2	2			3	2			2	2			3	3			24	
A2 2+25 - 3+50		2	3	1		3	1	2		3	2	3		1	3	3		2	1	3		3	3	3	42	
B1 4+00 - 4+50	1	1			2	1			1	3			1	1			1	2			1	1			16	82

VISUAL PRIORITY LEVELS—NEW ELEMENT RATINGS

CUT RANKINGS			FILL RANKINGS			BRIDGE RANKINGS			WALL RANKINGS			STRUCTURE RANKINGS			VIEW RANKINGS		
SCORE	SPECIF AREA	VPL	SCORE	SPECIF AREA	VPL	SCORE	SPECIF AREA	VPL	SCORE	SPECIF AREA	VPL	SCORE	SPECIF AREA	VPL	SCORE	SPECIF AREA	VPL
42	A2	1															
24	A1	2															
16	B1	3															

UNIT TOTALS

UNIT	SPECIF AREA	CUT	VPL	FILL	VPL	WALLS	VPL	BRIDGE	VPL	NAV COMM AREA	VPL	STUCTURES	VPL	CUMULATIVE UNIT TOTAL NEW	LOSS
A	1	24	2												
	2	42	1												
UNIT TOTAL		68												68	
B	1	16	3												
UNIT TOTAL		16												16	

FIGURE 9 Three forms for determining priorities.

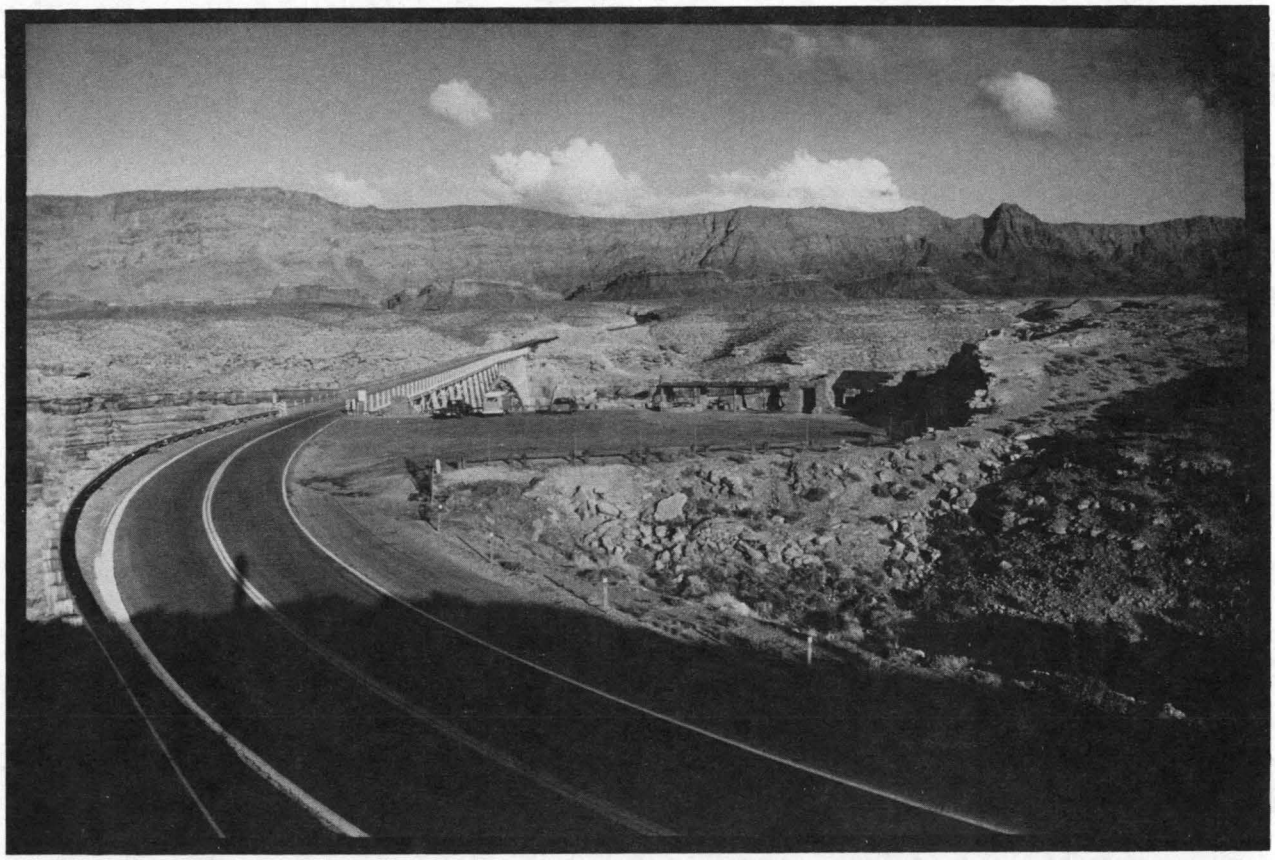


FIGURE 10 Navajo bridge corridor area.

recognizing variations in resources within the corridor area (see Figure 10). VPP is also a means of increasing communication and understanding of the design engineer concerning the visual management concerns along a project corridor. VPP is broken down into three phases, which cover the entire process of a corridor project. Phase I is the area wide planning, which documents the area's visual resources, visual management concerns and objectives, and the public's concerns. Phase II is the corridor- or project-level planning/design studies, which detail corridor alternatives, significant new elements, and lost resources to determine preliminary design mitigation and cost estimates. From this information, alternatives are evaluated and the best, considering engineering and visual management concerns, is chosen. Phase III is project-level implementation, which is the completion of the engineering and landscape architect design for construction, including any possible modifications during construction. VPP can be completed in its entirety or only through the initial phase, with the others completed later. VPP can be an excellent tool in completing any corridor project from both the engineering and visual management aspects. Various government entities may want to consider including VPP as an addition to their visual management guides for design.

REFERENCES

1. U.S. Department of Agriculture, Forest Service, *National Forest Landscape Management, Volume 2*. Agricultural Handbook Number 462, Ch. 1, April 1974.
2. Jones and Jones. *Visual Impact Assessment for Highway Project*. American Society of Landscape Architects. Contract DOT-FH-11-9694, FHWA, U.S. Department of Transportation, undated.
3. McGovern, T., J. Gallaher, L. Mason, M. Taylor, and B. Makel. *The Visual Prioritization Process*. CTIP Study 90-F-11, FHWA, U.S. Department of Transportation, 1993 (unpublished data).

Publication of this paper sponsored by Committee on Landscape and Environmental Designs.

Use of Computer Animation Technology for the Development of Interpretive Facilities on the San Juan Skyway

JAMES L. SIPES AND RICHARD F. OSTERGAARD

It is important that the skills and experiences of designers be complemented with a collection of tools that allow them to deal with complex issues and problems. Landscape architects at Washington State University and the U.S. Department of Agriculture Forest Service are working to determine the potential use of computer animation as a design, visualization, and analytical tool. The San Juan Skyway, in southwest Colorado, was selected as a test project because of its national visibility and the high expectations concerning the planning and design of facilities along the route. Forty-nine sites along the Skyway have been designated as possible feature locations for interpretive facilities, the purpose of each facility being to enhance the experience of visitors by providing interpretation of the surrounding scenery, history, and forest management practices. The Weminuche Wilderness Overlook was selected from the 49 sites to be modeled and animated. Photo-realistic 3-D animation sequences were developed to simulate driving or walking through and around the proposed interpretive facilities in an attempt to understand the spatial experiences and dynamic interrelationships of the landscape.

The process of analyzing, creating, and communicating design solutions is often hindered by an inability to consistently generate adequate levels of reality recognizable to nondesigners. Exploring the use of computer animation to generate more effective representations offers the opportunity for designers to enhance design and communication skills.

Design professions have always relied on visual simulations to explore and communicate thoughts and ideas. Early techniques were dependent primarily on pen and ink, pencil, charcoal, watercolors, oil paints, and later photography (*1*). These techniques, rooted in tradition, were used to generate plans, sections, elevations, and perspectives and have remained virtually unchanged for generations. Research has indicated, however, that traditional graphics reflect only one component of the perceptual experience of landscape and, as such, are limited when it comes to modeling the manner in which people relate to the three-dimensional (3-D) aspects of a landscape (*2*). Traditional graphics ignore the environment's surrounding, engulfing nature, and fail to reflect the animated, dynamic character that people ascribe to their surroundings (*3*). They also ignore whatever psychological meanings may have become attached to physical elements as well as the nonvisual senses that occur in connection with ephemeral events (*4*). These spatial experiences are especially important along scenic roads and highways.

There has been a growing trend to adopt more sophisticated technological innovations to create simulations (*1*). Orland has given a thorough review of the technology through the mid-1980s (*1*). Sheppard reviewed more than 300 simulations of proposed landscape changes in the San Francisco Bay Area and studied 30 of them in detail (*5*). These landscape portrayals included renderings, models, photomontages, and computer graphics. Respondents placed greatest confidence in models and photographs and least confidence in line renderings and computer graphics. Lindhult and Dines suggested a strategy for using the capabilities of computers to generate multiple perspective sketches from diverse positions in combination with traditional hand-drawn perspectives (*6*).

Computer technologies are evolving so rapidly that compelling reasons for using or not using some technique today may not hold true 6 months or a year from now (*7*). The somewhat abstract nature of the state of computer graphics makes it difficult to determine its communications effectiveness. For designers, the most intriguing technological advancements may be in computer animation. Designers who work with exterior sites place emphasis on creating dynamic spatial experiences because sites are usually experienced by a moving observer (*8*). Not only are users of a site likely to move past, into, or through a space (on foot, in a car, on a bicycle, and so forth), the site itself is ever-changing. The sun, moon, and clouds are always on the move, playing games with light and shadow, vegetation grows and changes colors with the passing of seasons. A site can become alive as the eye of an observer scans the horizon for new vistas or focuses on nearby surface textures. A single view is not as important as the cumulative effect of a sequence of views, and the transitions from the human experience to large elements around us in the environment are critical (*9*). Once attention is shifted from object to experience, all kinds of environments that provoke aesthetic experience can be included in the discussion, and variations in the responses of individuals can then be recognized (*10*).

PROJECT DESCRIPTION

Landscape architects at Washington State University (WSU) and the U.S. Department of Agriculture Forest Service are

J. L. Sipes, Department of Horticulture and Landscape Architecture, Washington State University, Pullman, Wash. 99164-6414. R. F. Ostergaard, San Juan National Forest, 701 Camino Del Rio, Durango, Colo. 81301.

working to determine the potential use of computer animation as a design, visualization, and analytical tool. The assumption that aesthetically pleasing environments provide valued experiences that can improve people's quality of life underlies many government landscape policies (*11*), including those of the Forest Service. The San Juan Skyway, in southwest Colorado, was selected as a test site because of its national visibility and the high expectations concerning the planning and design of facilities along the route. The area is rich in cultural resources ranging from the Archaic and Anasazi habitations to the colorful mining era in the San Juan Mountains in the 1800s, including the development of the narrow-gauge railways through the area (*12*).

The Skyway itself is a 232-mi route through the San Juan Mountains of southwestern Colorado. It connects the historic towns of Durango, Silverton, Ouray, Telluride, and Cortez and includes the historic section of the Million Dollar Highway. Traversing some of the most spectacular, rugged, and primitive country in North America, the Skyway is often described as the most scenic drive in America (*12*). The Skyway was designated by Forest Service Chief Dale Robertson (*13*) as a National Forest Scenic Byway on November 11, 1988, and by the State of Colorado Scenic Byway Commission as a State Scenic and Historic Byway on October 3, 1989. Both were the first such designations in the state. The nationally popular drive is the second longest Forest Service scenic byway and represents one of the largest and most complex recreation development undertakings in southern Colorado. As a result, it requires creative and innovative planning, design, and management strategies.

In keeping with the primary goals of the national scenic byway program, the physical development of highway scenic overlooks for natural or historical interpretation is critical. Forty-nine sites (*12*) along the Skyway have been designated as possible future locations for interpretive facilities. Managing changes in the dynamic systems of these diverse landscapes poses the challenge of determining what kinds of change are acceptable in the landscape composition.

The purpose of the proposed facilities is to enhance the experience of visitors by providing interpretation of the surrounding scenery, history, and forest management practices. Three basic conceptual design modules were developed: (a) a camera point plan, (b) a standard plan, and (c) an expanded plan (*12*) (Figure 1). These modules are designed to allow continuity from site to site, to fit aesthetically on the landscape, and to be functional for the intended use. The camera point plan is intended for smaller sites and provides an opportunity for a brief stop with a simple interpretation or informative sign. The standard plan provides space for 15 to 25 visitors and parking for up to five cars and two recreational vehicles (RV). The expanded plan is essentially the same layout but is larger to accommodate 30 to 40 visitors and parking for eight cars and three RVs. Common to the standard and expanded plans are benches, interpretive signs, and a staging area between the parking area and viewing area that provides for a large landscape rock with a plaque attached crediting the partnerships responsible for the facility. Other improvements such as curbs, retaining walls, bicycle racks, and steps or ramps will be incorporated as dictated by the site.

The Weminuche Wilderness Overlook was selected by Forest Service landscape architects to be modeled and animated. Four classified wilderness areas are accessible from the Skyway, of which the Weminuche Wilderness is one. The Weminuche Wilderness is the oldest classified wilderness in the state and also the largest at 459,804 acres (*12*). It includes some of the steepest and most complex terrain in the world, including the Needles, the most precipitous terrain in the nation. Mining opened the area to settlement, resulting in many stories about lost gold and silver mines. Many legends were the result of illegal mining during the 17th century by the Spanish, who did not keep records of their operation lest they be taxed by the king of Spain (*12*).

TRADITIONAL VERSUS COMPUTER ANIMATION

Traditionally, animation techniques were not an alternative for developing animation sequences of proposed interpretive facilities. Traditional hand-generated animation is very time-consuming and labor-intensive because a tremendous number of frames must be generated to create smooth, realistic movements. A minimum of 16 images per second are needed or movements will appear jerky. In traditional animation, 24 frames are typically displayed for every second of animation with computer animation requiring a slightly higher standard of 30 frames per second (*14*). Consequently, to prepare a realistic animation sequence for 1 minute requires 1,440 hand-drawn images or 1,800 computer-rendered images (*15*). Even classic cartoons from the past were tedious for the animators who toiled to create them, requiring months to produce the thousands of drawings necessary to give the illusion of fluid motion.

Computer animation was used on the Weminuche Wilderness Overlook project for several reasons. Computer animation systems eliminated much of the repetitive work associated with traditional animation and reduced the amount of time that would have been required for such a project. Computer animation also allowed designers and administrators to move around and through the proposed interpretive facility and to examine the site from a variety of views. Once 3-D computer models were constructed, the designers were able to visualize different design decisions and create simulations with real-world properties such as surface texture, transparency of objects, and the appearance of rain or fog (*16*).

COMPUTER COMPONENTS FOR THE PROJECT

Three major components collectively make up an animation system: the main computer, animation software, and specialized peripherals (*9*).

Computer

Historically, only mainframes or minicomputers were used for computer animation because animation is one of the most demanding computer graphics applications in terms of memory and processing power. In the past few years, however, microcomputer hardware and software have advanced to the point where animation is becoming affordable. Microcomputer platforms still cannot render as fast as mainframe sys-

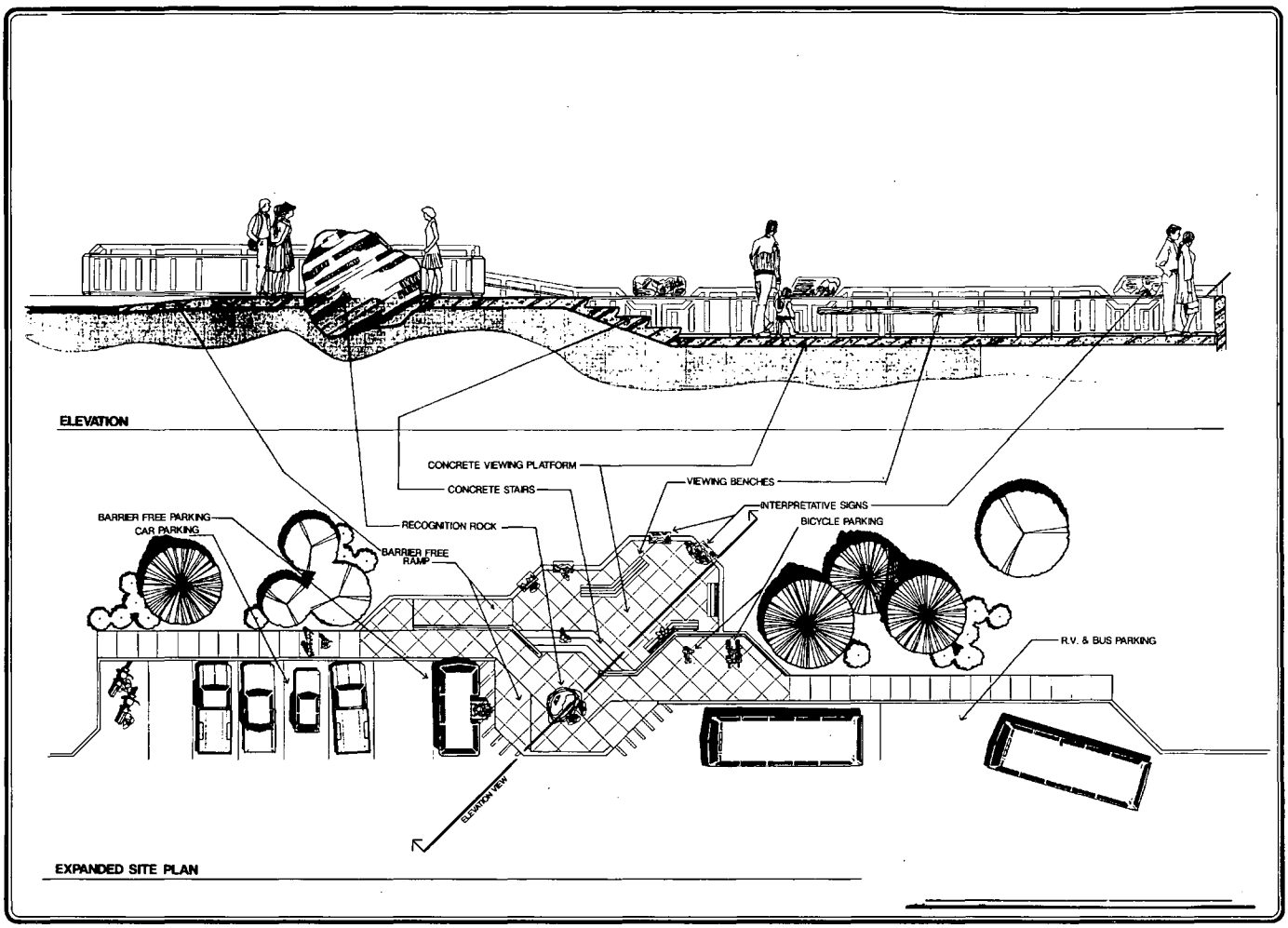


FIGURE 1 Section detail (*top*) and plan (*bottom*) of standard plan for proposed interpretive facilities.

tems, but now that hardware has increased in power and software has incorporated sophisticated animation features, it has become virtually impossible to distinguish a microcomputer animation from a mainframe animation (*17*).

Microcomputers were selected to create the animation sequences for this project for several reasons. Even at a major university such as WSU, only a handful of mainframe computers are capable of generating animation sequences, and getting adequate time on one of these computers for a project such as this one is virtually impossible. Microcomputers, on the other hand, are plentiful, easily accessible, and affordable. This is true not only at WSU, but in the Forest Service offices as well. One of the objectives was to take computer animation out of the exclusive research laboratories and demonstrate its potential as a design tool for even low-budget projects.

Animation Software

In general, animation software can be divided into three major categories: electronic slide show, two-dimensional (2-D) animation, and 3-D animation (*9*). Unlike true animation, an electronic slide show creates the illusion of animation by providing smooth transitions from one image to the next (*8*) and is used primarily for presentations. The 2-D animation programs simulate traditional animation by combining sequences of images created with "paint" software. These animation systems are typically less expensive and easier to use than 3-D systems but are not as powerful. This project did not use 2-D animation because such systems create a flat "painting" that does not adequately capture the essence of moving through a site. Rather, 3-D animation software was used for the Weminuche Wilderness Overlook, because it provides a wide range of choices for designing precise, accurate models and creating photo-realistic simulations. Autodesk's 3D Studio was selected because of its combination of power, affordability, and ease of use.

Specialized Peripherals

Although the main computer provides the primary computational and managerial functions, other hardware adapts the system to animation and actually eclipses the personal computer in terms of sophistication and price. This equipment typically includes a high-resolution color monitor and peripherals such as a color video camera, video camera recorder (VCR), VCR controller, sync generator, encoder, and mass-

storage system (*7*). Specialized equipment is often needed because microcomputers typically do not have adequate memory or storage capacity for high-resolution, true-color animation sequences (*17*).

Most computer animation sequences are compiled and recorded directly to tape because of the tremendous storage space required to store the individual images that make up the sequence. For most animation software, the only way to render an animation sequence is directly to tape, one frame at a time. However, the slow rendering speed of microcomputers makes it difficult to render an animation directly to tape. Because of the expense involved, most video decks are shared among departments and individuals and it is not practical to tie up the video deck for days, weeks, or months at a time. For the Weminuche Wilderness project, preliminary animation sequences were rendered initially with low resolution and a limited color palette so they could be displayed on a standard computer screen. Final animation sequences were rendered one frame at a time directly to a hard drive, then compressed, stored on a removal tape cartridge, and recorded on ¾-in. or ½-in. video tape.

DEVELOPING 3-D COMPUTER MODELS

Designs for the three proposed interpretive facility modules were developed by landscape architects with the San Juan National Forest before the decision was made to use computer animation. Before creating animation sequences, the basic design modules had to be converted from traditional hand-drawn plans to three-dimensional computer shapes using a process called "modeling" (*15*). Modeling is the step in which a designer describes to the computer the shape and dimensions of every object or character in the animation sequence. Most animation programs use a tool called "object editor" (*17*) for modeling the objects to be used in animation. Most object editors use the same functions and terminology as do computer-aided drafting (CAD) and solid-modeling systems.

There are two basic ways to create 3-D computer models (*15*). The first is to "extrude" or "loft" 2-D shapes to create 3-D objects. The second consists of creating simple geometric primitives, such as rectangles, cylinders, and cones, then combining them to create more complex objects. The first approach proved to be the most efficient because there were already detailed plan views of the interpretive facilities. Because the designs had not originally been generated on a computer system, the hand-drawn plans of the standard and enhanced modules had to be converted into a digital format that would serve as the foundation for developing computer animation sequences. These plans were digitized using a standard CAD program, AutoCAD Release 11, to create a digital 2-D line drawing. Although digitizing these plans was very quick and easy, the accuracy of the subsequent CAD images was not acceptable for use in 3-D modeling. The final images had to be very precise because 3-D modeling and animation is defined mathematically.

The only way to ensure that the final drawings would be accurate and precise enough for detailed computer modeling was to draw them from scratch. Although this took approximately twice as long as did digitizing, the results were free of even minor flaws or mistakes. The plans were exported out of AutoCAD as a standard data exchange file (DXF) file and imported into the 2-D Shaper module of 3D Studio where they were converted into 2-D polygons. The shapes were extruded using the 3D Studio's 3-D Lofter program (*18*) to add the "Z" dimension of height to the original images. Major design elements such as roads, curbs, steps, and ramps were modeled this way. Other site features, including rails, benches, light fixtures, signs, rocks, and automobiles, were created by combining simple geometric primitives to create more complex shapes.

One of the most time-consuming tasks in 3-D computer animation can be building the models that form the foundation of the sequences. Modeling the proposed interpretive overlook for the Weminuche Wilderness was not very difficult, primarily because the original design was fairly simple and very geometric. Creating a model of the proposed site along the Weminuche Wilderness was a much greater task. Trying to model intricate shapes typically is a laborious and repetitive task (*9*) and can create special problems. Traditional modeling methods do not adapt well to objects with the complicated shapes, such as trees, water, and topography, necessary for landscape architects. The first step in creating a computer-based landform model of the site was to digitize contours from a topographic map. Each line was defined with a Z coordinate to represent its respective elevation, an algorithm generated spot elevations at intersecting points on a grid, and the points were combined to create a rectilinear wire mesh formed by polygons. The most accurate representation of the actual site was obtained by a 5- × 5-ft mesh, but the resulting CAD file was too large (4.7 Mb) to work with efficiently. Larger meshes, 20 × 20 ft and 30 × 30 ft, were unacceptable because the interpolation process eliminated too many points in the landscape. A 10 x 10 ft mesh provided an acceptable level of detail to model the site faithfully and was not beyond the capabilities of current hardware and software.

Generating 3-D vegetation for the final computer model was also a challenge. In many ways, the difficulties in creating a 3-D model of a tree epitomize—more than any other elements—the problems inherent in modeling a site (*7*). Trees take on a bewildering variety of forms under the influence of their environment, and those forms change as the plant grows, ages, and adapts to its environment. Each species has its own habit of growth and its own way in which branches and leaves are arranged. The problems of modeling and rendering a single tree in a foreground are quite different from those of a forest seen in a background. When seen from a distance, trees are large elements defined by their shape. On closer inspection they break down into a connected system of branches, twigs, leaves, buds, and flowers (*7*). Because of the complexities involved, it is little wonder many people avoid trying to develop detailed 3-D trees.

The first attempts focused on creating 3-D trees with a high number of polygons to achieve a high level of realism. Although these trees surpassed expectations of visual quality, each consisted of more than 100,000 polygons and was approximately 1.2 Mb in size. They looked great, but required so much computing power to render that it was difficult to put more than a few in an animation sequence. The next tree was a more simplistic version, created with a series of overlapping "shingles" draped over a simple geometric cone. To create the final 3-D trees, two panels were combined to create

a "plus," then texture-mapped with scanned photographs of the original trees, and combined with transparency maps to allow a viewer to "see" through the branches of a tree. Tree panels were created as self-illuminating to minimize problems with shadows and to create the illusion of being irradiated by the sun. The benefit of these "panel" trees is that they provide the realism of a photograph while maintaining the benefits of a 3-D object.

After models of the proposed interpretive facilities were modeled, a "scene editor" (*17*) was used to render the individual 3-D objects created with the object editor. The final rendering step is the equivalent of adding ink and paint in the traditional animation process. Colors are added to each object, and the surfaces were enhanced with textures, shading, reflections, and motion blur. Surfaces of the objects were enhanced with bump maps, surface properties, and texture maps (*19*) (Figure 2). Bump-mapping creates the appearance of roughness on a surface. Surface properties refer to the degree of transparency of an object and the way its surface reflects light and objects around it. In texture mapping, an image of a texture, material, or photograph is overlaid on a 3-D computer model of an object.

To provide an acceptable level of realism for this project, background images were created by texture-mapping a sequential series of images onto large panels, which functioned much like billboards. The same basic technique was used to create the trees described earlier. The graphic images were actual photographs taken of the Weminuche Wilderness Overlook site. Eight different photographs were taken from the same spot, with the photographer turning slightly to create a panoramic view while ensuring that each image overlapped, to eliminate any gaps. Truevision TIPS (*7*), an image process program, was used to combine the images so they would appear as one.

QUESTION OF REALISM AND VALIDITY

An important question when considering computer animation for the Weminuche Wilderness Overlook was what level of realism and accuracy was needed. During the planning stage of this project, one of the first assumptions was that the final images needed to be both "valid" and "realistic" (*1*) because the final computer models had to illustrate that the interpretive facilities would visually "fit" the existing site (*12*). It was assumed that crude models would not have given the degree of detail necessary for members of the public to develop a level of understanding and confidence that would enable them to make informed decisions. But as work progressed, a number of questions needed to be answered. Are wire-frame images sufficient to convey a design concept? Do hidden-line algorithms need to be implemented to remove "visual noise?" Do textures and colors play an important role in the visualization process? Is the search for "realism" appropriate and necessary? Is it acceptable to create images that simply "look" real or should the complexities of a site be duplicated?

In the past, these questions were mostly rhetorical because hardware and software limitations made it virtually impossible to achieve a high level of detail or realism (*15*), especially for landscape architects with limited budgets and little or no computer training. Even with advances in computer technology, providing a high level of realism is difficult. It is neither practical nor appropriate to attempt to include every site element in infinite detail (*7*). Not only are resources unavailable to generate such a computer model, but the rendering and seen-area algorithms available today do not naturally accommodate the complexities of the entire visual field (*7*).

Foley and Van Dam (*20*) see the quest for realism as the production of "images which are so realistic that the observer believes the image to be that of a real object rather than that of a synthetic object existing only in the computer's memory." Appleyard (*21*) has suggested that "simulations should be realistic and accurate; that is, they should convey how a project will be experienced." Kaplan and Kaplan (*4*), on the other hand, advocated the use of "a highly simplified physical model of the physical environment." The limited evidence (*22*) suggests that the most realistic simulations, those that have the greatest similitude with the landscapes they represent, provide the most valid and reliable responses (*23*). Early computer-generated simulations (*24*) were found to be inaccurate in comparison with photographs of the actual completed projects. Research by Sheppard (*5*) showed that design professionals and planners placed the greatest confidence in models and photographs and the least in line renderings and computer graphics because of their limited realism.

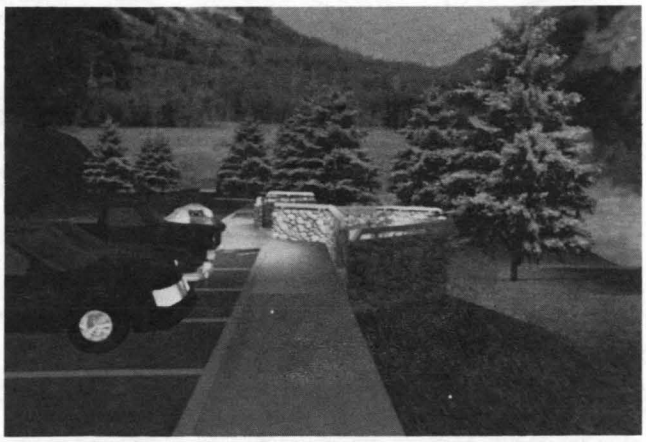


FIGURE 2 Photorealistic 3-D computer model of standard interpretive facility at Weminuche Wilderness Overlook.

DEVELOPING ANIMATION SEQUENCES

After 3-D models of the site elements were completed and surface materials defined, light sources, atmospheric conditions, and camera positions were added to the scene. Lighting is one of the most important aspects of the final rendering. In 3D Studio, three types of lighting can be added—ambient, omnidirectional, or spot lighting (*25*). Ambient light (*25*) provides a minimum level of illumination throughout a defined scene but has no direction or definable color. Omnidirectional lighting (*25*) is a consistent light source much like the sun, except that it does not cast shadows. A spotlight is used to highlight a specific part of a scene and is the only light source which casts shadows (*25*). Shadows can be important in the final rendering, especially when the objective is to add a touch

of realism (*26*). Each rendering of the final animation sequences for the Weminuche Wilderness Overlook uses a combination of the three different light sources, but spot lights have the greatest impact. The lighting was arranged in attempt to create the illusion of the sun peeking through a dense ceiling of clouds.

Different shading techniques were used at various stages of the design process. The three most common shading techniques are Flat, Gouraud, and Phong (*25*) shading. For initial massing studies where a high degree of detail is not necessary, models were rendered using flat shading. Flat shading provides the fastest rendering but offers the least amount of surface detail. To create a more realistic-looking image, computer models were rendered using Gouraud or Phong shading options. Gouraud shading provides smooth rendering, but Phong shading provides a more realistic representation and must be used to render shadows and highlights. Although Phong shading takes a considerable amount of processing time, the final Weminuche Wilderness Overlook animation sequences were all rendered with Phong shading.

The final step in the process was to script motion from the beginning to the end of the animation sequence (Figure 3). This motion scripting allows a designer to define the movements of the objects, characters, lights, and cameras and is often the most difficult task in advanced animation. Our objective was to give the viewer a feeling of what it would be like to drive down a road to the proposed interpretive center overlooking the Weminuche Wilderness.

SUMMARY

Landscape architects at both WSU and the Forest Service were interested in using this project to help determine the potential use of computer animation as a design, visualization, and analytical tool. Animation sequences of the proposed Weminuche Wilderness Overlook were developed to understand the spatial experiences and dynamic interrelationships of the landscape. But beyond this overall goal, the two participating groups had different perspectives on what would make this project a success or failure.

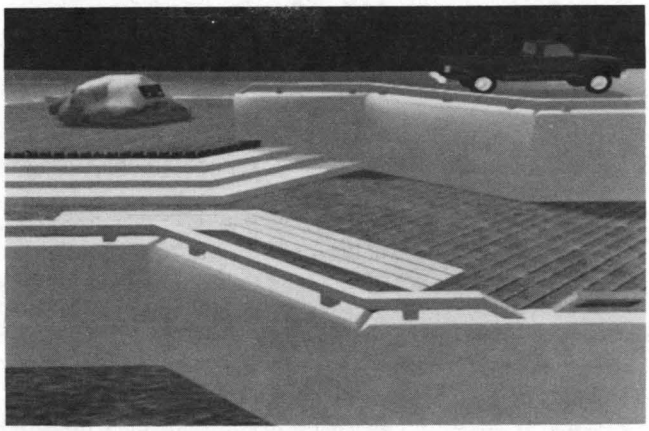


FIGURE 3 Sequential images of animation sequence that shows use of different materials for proposed interpretive facility.

Forest Service Perspective

The original goal of the Forest Service was for the computer animation to aid the designers in making decisions on proposed design concepts for interpretive facilities and how these concepts would fit into a selected site. It was hoped that there would also be some adaptation of these animation sequences in approaching possible sponsors of sites along the Skyway. The technology would open other opportunities for adaptation of other projects throughout the Forest Service system.

The actual benefits of this project are varied, and some benefits may not be fully discernible at this time. From the standpoint of the Forest Service landscape architects, the previously mentioned goals and expectations generally were met and, in some areas, exceeded. The two major benefits to them were as a design and analytical tool and as a marketing tool. Being able to visualize a proposed facility on the landscape in a 3-D depiction has helped the Forest Service designers in refining the scale, function, form, and choice of materials that would successfully harmonize with the setting. It also helps meet the agency's planning goals and guidelines as suggested in the Recreation Opportunity Spectrum and Landscape Management programs.

As a marketing tool, computer animation has aided the Forest Service in developing partnerships with parties who may be interested in participating in the development of interpretive sites along the scenic byway. The use of these animation sequences, or others done for specific sites, will give a convincing simulation of what the finished facility will look like.

The use of this computer animation technology for the San Juan Skyway project has also opened up other applications possible for other projects throughout the Forest Service system. There have been a number of inquiries from other field units considering the applications that computer animation has for some of their projects.

WSU Perspective

One of the major reasons WSU landscape architects participated in this project was to develop a series of animation sequences to be used for further studies addressing issues such as the effectiveness of microcomputer animation, the benefits of animation over traditional graphic media, and perceptions concerning animation sequences and still images. Although a detailed analysis of the Weminuche Wilderness Overlook animation project has not been completed, preliminary observations have yielded some interesting results.

- For the designers involved with this project, the use of computer animation did not result in a greater level of creativity. Although basic design decisions had been made before any attempts to develop computer animation sequences, the physical separation between WSU and the San Juan National Forest, the lack of computer knowledge by Forest Service employees, and the tremendous time required to render the final images would have presented major problems in trying to use computer animation as a design tool.
- Creating 3-D computer models of the various design proposals for the interpretive centers were a fairly simple,

straightforward task. Even the most complex design alternatives could be modeled in days, with modifications being made in hours. Although time and effort were devoted to modeling the proposed interpretive facilities, most of this time focused on trying new techniques or procedures.

- The general feeling of movement and motion created by the animation sequences did a lot to make up for the overall quality of an individual image (*27*). Reviewers of the animation sequences appeared to have difficulty distinguishing between 8-bit (256 colors), 16-bit (32,000 colors), and 24-bit (16,700,000 colors) images when they were part of an animation sequence. In contrast, they quickly noticed a difference in the overall quality of images when comparing individual frames from a sequence. This was in contrast to our original belief that high-resolution, high-color images were necessary to convey the aesthetic concerns of the site and validates past research (*28*) that concluded that, although greater color and pixel resolution give more accurate simulations and more reliable impact measures for development proposals, video editing based on 8-bit graphics systems is viable.
- Rendering time was the greatest limitation in using computer animation to explore different design alternatives or modifications. With current microcomputer technology, it is not cost effective to create 24-bit (16,700,000 colors), photo-realistic animation sequences in an acceptable amount of time. After modifications were made, it would take several days to render an animation sequence before the results could be analyzed. This effectively eliminated any interactiveness or spontaneity. An acceptable substitute appears to be 8-bit (256 colors) animation sequences, which provide a good representation (*28*) of a proposed alternative, yet enable most sequences to be rendered overnight.
- In general, computer animation sequences were viewed very favorably by viewers. There is concern, however, that this had more to do with the media itself and not how effectively the proposed interpretive overlooks were illustrated.
- There needs to be a greater understanding of the numerous visual cues, identified by psychologists (*27*), by which our minds interpret 3-D space. These visual cues include motion parallax, size, linear perspective, angle of regard, interposition, aerial perspective, light and shade, texture, disparity, convergence, and accommodation. Motion parallax, disparity, and convergence (*27*) are not easily addressed by current computer technology, but an understanding of the other visual cues is critical if computer animation is truly going to become a useful analytical, design, or visualization tool.
- Shading appeared to play an important role in making an image or sequence more understandable or more realistic, but the addition of shadows had very little positive impact. This information appeared to contrast with past research (*27*), which indicates that the full use of a number of visual cues requires both shade and shadows.
- The patterns in the mind appear to be as important as the objects of sight. Most students believed they saw objects in animation sequences that were not really there. When asked, several viewers said shadows were helpful in making an animation sequence more believable even though there were no shadows in the sequences they observed. They "saw" shadows because they expected to see shadows, not because they were actually present in the scene. These results agree with Land's retinex theory (*26*), which states that we do not determine the color of an object in isolation; instead, we are always comparing the object with its surrounding.
- The perception of realism (*27*) seemed important to viewers, yet the question of how valid or accurate the final scenes were did not seem to be a concern. None of the viewers, even those familiar with the area, questioned whether these scenes faithfully depicted the Weminuche Wilderness area.

FIGURE OF ANIMATION IN LANDSCAPE ARCHITECTURE

Computer animation has had limited implementation outside the entertainment industry. Design professionals have been especially slow in embracing computer animation for a number of reasons. There are still a number of technical limitations that make animation very laborious and time-consuming. Most firms or organizations do not have anyone with the necessary skills or training to be proficient at creating computer animation sequences. In addition, a new computer tool, such as computer animation, typically generates an abundance of enthusiasm, making it difficult to determine its long-term value as a design tool. It is easy to get caught up in using a new toy like computer animation without having a clear objective of why and how it should be used.

Zube et al. (*1*) reviewed available simulation techniques and believed that the potential of computer graphics and animation in landscape architecture was very high and that, given some cost reduction, "a major transformation in graphic communications is likely to occur in all of the environmental design fields." The communications effectiveness of computer animation, however, is not very well understood and there are still a number of problems and questions that need to be addressed. As computer animation is used more frequently for public communications, the need to address these problems becomes more urgent.

Perhaps the key to using computer animation efficiently lies in the ability to be flexible, to match technique with the situation at hand in a way that will best reveal the essence of what design professionals are trying to communicate or study (*7*). During the early stages of a design or a large-scale project, massing studies consisting of wire-frame models, simple geometric shapes, and minimal detail may be appropriate. For a final design solution and projects involving sensitive aesthetic considerations, photo-realistic images, accurate shadow casting, and complex three-dimensional animation sequences with a high degree of detail may be required.

REFERENCES

1. Zube, E. H., D. E. Simcox, and C. S. Law. Perceptual Landscape Simulations: History and Prospect. *Landscape Journal*, Vol. 6, No. 1, 1987, pp. 62–80.
2. Kaplan, S. Perception and Landscape Conceptions and Misconceptions. In *Our National Landscape: Proceedings of a Conference on Applied Technology for Analysis and Management of the Visual Resources* (G. Elsner and R. C. Smardon, technical coordinators). USDA Forest Service General Technical Report PSW-35. Pacific Southwest Forest and Range Experiment Station, Berkeley, Calif., 1979.
3. Conron, J. *The American Landscape*. Oxford University Press, New York, 1974.

4. Kaplan, S., and R. Kaplan. *Cognition and Environment*. Praeger, New York, 1982.
5. Sheppard, S. R. J. How Credible are Visual Simulations? *Landscape Architecture*, Vol. 73, No. 1, 1983, pp. 83–84.
6. Lindhult, M. S., and N. T. Dines. Perspective Sketching With Microcomputers. *Landscape Architecture*, Vol. 75, No. 4, 1985, pp. 56–57.
7. Sipes, J. L., and B. Sumption. *High Color*, Sept./Oct. 1992, pp. 42–45.
8. Harbison, D. Landscape Architects Tap CAD & GIS. *Computer Graphics World*, Vol. 11, No. 12, Dec. 1988, pp. 32–38.
9. Sipes, J. L., W. G. Hendrix, and R. F. Ostergaard. Computer Animation in Landscape Architecture. *Landscape Architecture*, Aug. 1992, pp. 68–71.
10. Koh, J. An Ecological Aesthetic. *Landscape Journal*, Vol. 7, No. 2, Fall 1988, pp. 177–191.
11. Stewart, T. R., P. Middleton, M. Downton, and E. Ely. Judgments of Photographs vs Field Observations in Studies of Perception and Judgment of the Visual Environment. *Journal of Environmental Psychology*, Vol. 4, 1984, pp. 283–302.
12. Ostergaard, R. F. *A Development and Interpretive Guide for The San Juan Skyway: A National Forest Scenic Byway*. Student Project for Professional Development for Outdoor Recreation Management Program, Clemson University, April 1990.
13. *Protection Techniques for Scenic Byways: Four Case Studies*. Final Case Study for the National Scenic Byways Study. FHWA, U.S. Department of Transportation, Sept. 1990.
14. Rivlin, R. *The Algorithmic Image: Graphic Visions of the Computer Age*. Microsoft Press, Redmond, Wash., 1986.
15. Robertson, B. Animation Goes to Work. *Computer Graphics World*, Vol. 11, No. 7, July 1988, pp. 38–48.
16. Biggs D. Simulating and Animation Project Designs. *CADENCE*, June 1990. pp. 59–60.
17. MacNicol, G. PC Animation Hugs the Inside Lane. *Computer Graphics World*, July 1989, pp. 58–63.
18. Yares, E. 3D Studio Makes Professional Animation Tools Affordable. *PC WEEK*, Vol. 7, No. 48, Dec. 1990, p. 8.
19. Foley, J. D., and A. Van Dam. *Fundamentals of Interactive Computer Graphics*. Addison-Wesley Publishing Company, Reading, Mass., 1984.
20. Foley, J. D., and A. Van Dam. *Fundamentals of Interactive Computer Graphics*. Addison-Wesley Publishing Company, Reading, Mass., 1984.
21. Appleyard, D. Understanding Professional Media. In *Human Behavior and Environment, Volume 2* (I. Altmand and J. F. Wohlwill, eds). Pleanum Press, New York, 1977.
22. Bishop, I. D., and P. N. A. Leahy. Assessing the Visual Impact of Development Proposals: The Validity of Computer Simulations. *Landscape Journal*, Vol. 8, No. 2, Fall 1989, pp. 92–100.
23. Daniel, T. C., and J. Vining. Methodological Issues in the Assessment of Landscape Quality. In *Human Behavior and Environment* (I. Altmand and J. F. Wohlwill, eds). Pleanum Press, New York, 1983.
24. Zube, E. H., J. L. Sell, and J. G. Taylor. Landscape Perception: Research, Application and Theory. *Landscape Planning*, Vol. 9, 1982, pp. 1–34.
25. Yares, E. 3D Studio Makes Professional Animation Tools Affordable. *PC WEEK*, Vol. 7, No. 48, Dec. 1990, p. 8.
26. Montgomery, G. Color Perception: Seeing With the Brain. *Discover*, Dec. 1988, Vol. 9, No. 12, pp. 52–59.
27. Armstrong, W. P., and R. P. Burton. Perception Cues for n Dimensions. *Computer Graphics World*, March 1985, pp. 11–28.
28. Bishop, I. D., and P. N. A. Leahy. Assessing the Visual Impact of Development Proposals: The Validity of Computer Simulations. *Landscape Journal*, Vol. 8, No. 2, Fall 1989, pp. 92–100.

Publication of this paper sponsored by Committee on Landscape and Environmental Design.

Issues in Planning and Design of Scenic, Recreational, and Parkway Roads

RAMEY O. ROGNESS

Planning and design for recreational and scenic roads present a somewhat different set of criteria and restrictions than for other roads. Since the first parkways were built in the 1920s and 1930s and provided the impetus and criteria for all scenic and recreation roads, several important factors have changed: the age of these roads, the age of drivers, vehicle characteristics, and drivers' desires. Some road managers and agencies desire to retain the intent and function of scenic, recreational, and parkway roads, in terms of either new construction or reconstruction. It is important to recognize the role and nature of the recreational, scenic, or parkway road relative its users' needs. Comparisons are made between the early road standards, existing standards, and proposed lower road standards. The need to retain the role of the recreational, scenic, and parkway roads while addressing vehicle and driver factors is examined. One means to accomplish this is through more specific detailing in the design and construction stages. Differences within agencies and units show difficulties from a lack of consistency in philosophy and in design that results in inconsistencies in roadway geometrics, signing and marking, and especially lane width.

Planning and design for recreational, scenic, and parkway roads present a somewhat different set of criteria and restrictions than for other roads. These restrictions arise from three factors: (a) many of these roads are up to and over 50 years old; (b) these roads are themselves considered scenic or recreational only, or pass through scenic areas; and (c) any roadway is itself an obtrusion into the vista and scenic view. Generally these situations cause most recreational or scenic road managers to want to keep or design the road to a minimal standard. More importantly, the roads typically have an assumed lower design speed and reduced lane width.

Most of the original recreational or scenic roads were built to older standards and the lower design speeds of the 1930s and 1940s. Recreational road managers are hesitant to widen and flatten existing roads because of the cost and the visual impact. This is similarly true for new recreational and scenic roads since the desire is for a recreational and scenic road, not a thoroughfare.

This is illustrated by the FHWA as

> A scenic road or byway has roadsides or corridors of aesthetic, cultural, or historical value. An essential part of this road is its scenic corridor. The corridor may contain outstanding scenic vistas, unusual geological formations, dramatic urban scenes, scientific features, or other elements—all providing enjoyment for the highway traveler. (*1*)

A similar situation exists when a state designates an existing roadway as a scenic roadway. Does the agency need to make any operational, signing or maintenance improvements to the roadway before it is officially designated and recreational travel encouraged? Obviously, the roadway is selected because it is scenic. This typically means that the roadway is low volume and probably historical in the sense of older design standards. The road may even have been built before the adoption of highway design standards.

Department of Civil Engineering, Ohio State University, 470 Hitchcock Hall, 2070 Neil Avenue, Columbus, Ohio 43214.

HISTORICAL DEVELOPMENT

Newton describes the early development of recreational parkways (*2*). The first parkway was the Bronx River Parkway, started in 1913 and substantially completed in 1923. This parkway and three others in Westchester County, New York—the Hutchinson River, Saw Mill River, and Cross County—influenced future parkways by providing an environment that was extremely pleasant for driving and yet functionally efficient for commuting traffic. Manning describes the intrinsic beauty of these early parkways (*3*).

At the federal level, the Mount Vernon Memorial Highway in Virginia was completed in 1932. This parkway made provisions for scenic overviews and historical features and was an early model for federal parkways. The adjoining George Washington Memorial Parkway, started in 1935 and completed in 1965, included many of the parkway design and construction details developed on the Mount Vernon Memorial Highway (*4*). In the Blue Ridge Mountains, Skyline Drive, designed in 1930 and finished in 1934, was built in the early scenic parkway tradition. These parkways have provided design elements and philosophies that continue to be used. Today, many public use roads designed by the National Park Service (NPS), U.S. Department of the Interior (*5*) within the national parks are constructed in the parkway tradition.

Some believe that the crown jewel is the Blue Ridge Parkway, started in 1937 and completed in 1987. The concept for this scenic road connecting the Shenandoah Valley to the Great Smoky Mountains developed mostly during the Depression years. Scenic, historical, and cultural heritage features are combined in its coverage, development, and preservation. One prime goal, then and today, is to provide a user with a living museum of natural and manmade form. This obligation has led to certain mindsets in terms of land and vegetation management practices (*6*) that have influenced road design and maintenance practices.

The other major national parkway that has influenced both perceptions and design attitudes is the Natchez Trace, which runs through Mississippi, Alabama, and Tennessee. It has also presented design, construction, and maintenance challenges.

State-initiated parkways have included Oregon's start in 1913 and the multistate Great River Road, which was conceptualized in 1938 and has major portions still to be built. This concern for incorporating the scenic value of areas into highway building was a major factor in the development of scenic roads and parkways, and its influence has carried over into many of the scenic road design philosophies.

By the late 1930s, technical information had been gathered concerning the special needs for a parkway (e.g., ecology, vistas, and right of way). Differences in roadside vegetation by regions and climatic zone were identified and considered in the parkway development process. At the same time geometric features and roadside improvement were being considered.

Much attention was also given in the 1930s to landscaping improvements to the roadside. One example is the work by Boddy and Taylor (*7*) that gives roadside landscaping specifications.

By the 1940s, detailed specifications of roadside ecological construction were developed for specific applications. Curtiss in 1942 (*8*) wrote about roadside concerns in the national forests. Bell (*9*) described roadway standards for western scenic areas.

Dupre (*10*) reviewed the accomplishments and progress of roadside development in Ohio. One unique aspect was the development in the Lake Region states, especially in Ohio, of a specific grassed shoulder design to address the problem with vehicle tracking near the pavement edge on state roads (*11,12*).

After the 1940s the interest in research concerning the scenic roads and parkways and the roadside environment waned. The resurgence of interest in scenic roads returned in the 1960s with the emphasis on highway beautification. In 1966, the national program for scenic roads was initiated.

Pragnell (*13*) in 1970 wrote a report concerning scenic roads in forested lands. In 1984 the NPS published a report identifying and describing road standards for park roads (*14*). Interest in scenic roads has been growing. In 1988 a conference entitled "Scenic Byways—A National Conference To Map the Future of America's Roads and Highways" was held (*1*). Such activities emphasize the renewed interest in scenic roads.

HISTORICAL PERCEPTIONS VERSUS PRESENT SITUATIONS

The earliest parkways were built in Westchester County, New York, 75 years ago—really almost before the advent of recreational travel, much less automobile travel. Their purpose was "not to provide the fastest or most direct route between . . . origin and destination. These parkways were designed for *moderate* driving speeds to permit fullest enjoyment of the scenery" (*1*). It is doubtful that these parkways still carry out the same function for motorists, with today's heavy commuter traffic. Unfortunately, this purpose still guides most recreational road planners, who are unwilling to acknowledge that motorists may not have the same objectives or interests. Interestingly enough, the description of the George Washington Parkway by the NPS shows these differences:

> Considered a commuter route by many local residents, the George Washington Memorial Parkway offers the traveler much more than convenience. It is a route to scenic, historic and recreational settings offering respite from the urban pressures of metropolitan Washington. It also protects the Potomac River shoreline and watershed. The Parkway links a group of parks that provide a variety of experiences to over 9 million people each year. (*1*)

Not only are the Westchester County parkways and the George Washington Parkway heavily used by commuter traffic, but other scenic and recreational roads and parkways in the United States also exhibit these characteristics. Part of the problem is that the intent of the scenic road and its purpose do not match its use by the motorists. The definition of a park road is that it is a "means to enable visitors to reach their goals and provides a goal in and of itself," whereas a parkway is "an elongated park featuring a road designed for pleasure driving and embracing scenic, recreational, or historical features of national significance. Park roads and parkways are designed with care and sensitivity with respect to the resource and visitor experience. Roads in the national parks are treated as scenic roads" (*1*).

Looking at major NPS parkways—Baltimore-Washington Parkway, Colonial Parkway, Rock Creek and Potomac Parkway, and Suitland Parkway around the Washington, D.C., area—it is apparent that these parkways carry heavy volumes of commuter traffic and do not ideally match the definition of a parkway, especially in terms of speeds and driver objective.

Other parkways or park roads in nonurbanized areas have similar characteristics. An example would be the entrance drive near Gatlinburg, Tennessee, to the Great Smoky Mountains National Park and its park road to Cherokee (both US-441), which not only carries commuter and commercial traffic but also is the only direct route (not an alternative route, as is sometimes assumed for a park road). Another example is Florida's Everglades National Park, where the park road not only serves the park visitor but acts as the major commuting route for local fisherman headed to Flamingo for the weekend with their oceangoing boats on trailers, to go fishing in the Florida Keys. The John D. Rockefeller Parkway between Grand Tetons and Yellowstone National Parks in Wyoming represents a rural parkway environment.

Since the initial recreational roads (parkways) were built, three major changes have occurred. Different design standards have evolved for the design of new roads. The design standards for parkways built in the 1930s were developed to accommodate traffic moving at approximately 40 mph.

The second, more important, concern is that vehicles have changed. This issue is more complicated than automobile characteristics' having changed. Recreational vehicles (RVs) now are becoming common, and they are becoming larger. It is notable that an RV may be allowed on a recreational road while commercial traffic is restricted. In many cases, the RV and the commercial vehicle have similar characteristics.

The third change is in the driver. With the inherent aging of the population, the recreational driver has become much older and has a reduced driving capability. This problem is greater if an elderly driver is operating an RV. Motorists driving such vehicles typically are not familiar with their performance since they use them infrequently, and thus they control the vehicles less effectively.

This situation is compounded for an RV driver on a scenic or recreational road. The motorist not only has lower capability or driving skill with the RV, but also must become familiar with and adapt to lower road design features. The driver is used to wider pavement surfaces (12-ft lanes) and has initial (or sometimes continued) difficulty adapting to a lower design standard road, all while driving a vehicle that is typically wider, longer, and less stable than the driver is used to.

Many park visitors expect higher service and less inconvenience in their activities. Some motorists are in more of a hurry to complete the drive and are not tolerant of slow-moving vehicles. The same is true of the driving speed versus the speed limit (or design speed) of the roadway. This problem is compounded by the type of motorist. Near urbanized areas, many of these roads become a bypass, alternative, or even principal route for commuter traffic. Obviously, the speed and number of vehicles are much higher than for a purely recreational road.

The recreational road user today has different characteristics. One aspect having a more pronounced effect on the roadway designer is the trend of the recreational user wanting to see and do everything from the vehicle. Another ramification is the drive-through visitor. This type of visitor is obviously much different than the classical (and still present) visitor who wants a more leisurely and thorough visit, including side trails, walks, and programs. Overall, these trends translate into more driving loops and less use of trails and walkways. These conditions are compounded by changes in size and type of vehicles and driver performance in the user population. A further complication is that the number of visitors (users) has increased. Also, weekend or color season visitors provide even higher volumes, to a level that overburdens the roadway.

Further emphasizing the problem is that adjacent (access) roads probably were built at the same time as the scenic road with similar design standards but over time have been improved—especially with respect to pavement width. The roads may be similar in design but they meet a higher design standard and are wider in surface.

The realization that must develop is that retaining historical design features on recreational and scenic roads means that drivers will have more difficulty operating on them. This arises from drivers' lesser familiarity with their RVs or towed trailers and their performance. The average age of motorists is increasing with the attendant decreasing driving capability. Motorists are used to driving on wider (12-ft lane) roads and tend to position themselves by the same reference point when driving on narrower roads. Thus they are closer to the pavement edge. Recreational roads typically do not have shoulders, whereas at least a minimal shoulder is provided on the access road. Motorists get accustomed to interstate standards and 55-mph roadways and are reluctant to drive slower on other roadways. These problems need to be recognized and appropriately considered in roadway planning, design, and operation.

Trade-offs between design aethestics and design capability may need to be modified. Many older roads have been widened on the same alignment, so on access roads, drivers have typically made only minor adjustments in speed and *no* adjustment in lane positioning. Drivers' expectations need to be considered, and adequate transition distances are needed.

The visual intrusion of a widened roadway surface needs to be weighed against the driver, vehicle, roadway, and environmental characteristics. Retention of a 1930s design for a recreational road may not be effective. Modifications, especially in terms of initial transitions and commuter roads, need to be more extensive. For those roadways that are considered recreational or scenic but exhibit the higher-volume characteristics of a thoroughfare, the need is to recognize the actual situation and incorporate appropriate modifications.

Examples of the difficulty of keeping an RV within its lane are illustrative. Figure 1 shows an RV "cheating the lane" on a park road at Elkmont, in the Great Smokies. Figure 2 shows the same vehicle off the pavement edge on the tangent section and the associated edge deterioration problem.

Other concerns arise. One is the high frequency of informal pulloffs on scenic and park roads, even when the practice is officially restricted. Edge dropoff, edge failure, and shoulder deterioration can result. Figure 3 shows the effect of roadside "tunnel/vista effect" and the presence of a "vee" at the tangent-to-spiral of a curve on driver behavior and vehicle positioning. Figure 4 shows what can happen if a vehicle outside tire does leave the road for a length of time.

FIGURE 1 Vehicle cheating centerline.

FIGURE 2 Edge deterioration problem.

FIGURE 3 Example of "vee" at tangent-to-spiral of curve.

FIGURE 4 Example of extreme deviation outside pavement.

Obviously, there must be government concerns with the safety and operation of RVs on these scenic, recreational, and parkway roads. An example is the Scenic Byways Study Act of 1989 (*15*), which called for forecasts of significant changes in traffic volumes and safety consequences.

A limited study conducted by the Ohio Department of Transportation as part of its Highway Safety Improvement Program of 1990 (*16*) looked at general accident characteristics on designated state scenic roads as compared with all other two-lane roads. The 4 years of data showed that scenic roads had a higher accident rate than the other two-lane roads. The study results showed that for scenic roads accidents were overrepresented (compared with other two-lane roads) in several categories: driver had been drinking, dry road conditions, at curved sections, and for other vehicle types (RVs). For the general analysis, these factors were significantly different. The number of scenic road accidents was 3,621 as compared with 36,752 on other two-lane roads for the 4-year period. Scenic highways have 8.7 percent more accidents than the average two-lane highways (without divided sections) in Ohio. Figure 5 illustrates the historical accident rate trends. The provisional limits were developed using a chi-square test (*16*).

These difficulties should be considered not only in the rehabilitation of existing roads but also in the planning and design of new recreational and scenic roads and parkways. These concerns should extend to the new National Scenic Byways program of the U.S. Department of Agriculture (USDA) Forest Service.

CONSTRUCTION OF RECREATIONAL ROADS VERSUS MAINTENANCE NEEDS

A basic premise of recreational road managers is that recreational and scenic roads are used by recreational travelers. However, many scenic and park roads and parkways carry commuter traffic. In some locations, the parkways carry heavy volumes of traffic and act as commuter roads or urban bypasses. An example is the Blue Ridge Parkway in Roanoke, Virginia.

At the same time, driver characteristics are changing with the growing predominance of the older (retired) driver. Historically the recreational traveler was a younger (family) vacationer. Thus, the mix of visitors to recreational areas has changed. This factor has not been considered in the design and planning process in terms of older drivers' declining capabilities.

Also changing are vehicle characteristics. The greater number of larger RVs and longer trailers is compounded by the increasing presence of fifth-wheel trailers. Also more common is a large RV towing a motor vehicle. For many of these combinations, overall vehicle size approaches that of commercial trucks, which typically are restricted from operating

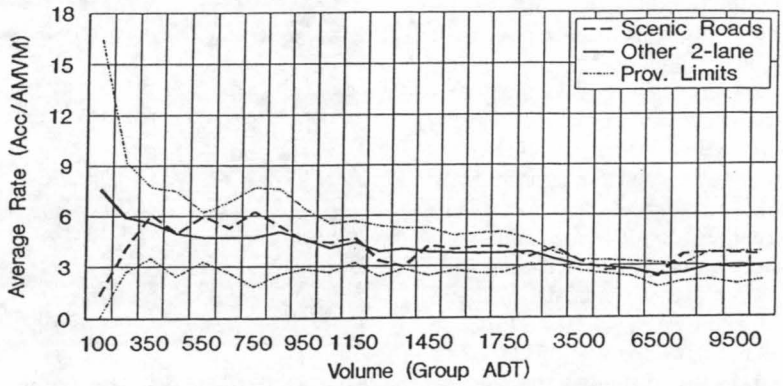


FIGURE 5 Accident rates on Ohio scenic and other two-lane roads.

on a recreational road. RVs are typically as wide and as long and as heavy as some single-unit trucks. When a 50,000-lb 45-ft motor home tows a boat trailer or car, its length and characteristics approach those of an over-the-road truck.

These vehicle characteristics are important when combined with the roadway and driver characteristics present. Because it is a recreational road, the lower design standards also include reduced lane widths. A driver typically positions a vehicle in a lane on the basis of experience. Being more experienced with the typical 12-ft lane, a driver will try to position the vehicle at the same distance from the roadway centerline (as for a 12-ft lane). Obviously, this places the outside wheels of the vehicle nearer the edge of the pavement. It might be expected that the driver would reposition the vehicle within the narrower lane; however, the combination of opposing traffic, tighter geometrics, and larger vehicle tends to make the driver unwilling to do so. Drivers are also not used to being as close to the roadway centerline as is necessary in narrower lanes. However, they do recognize the narrowness of the lane and the tighter geometrics. Drivers will compensate for the geometrics and narrowness typically by "cheating" the centerline if there is no opposing traffic. If there is opposing traffic, then a motorist will move away from the centerline and reposition the vehicle closer to the edge of the lane. This will also happen at horizontal curves where motorists can not observe whether opposing traffic is present. This repositioning can result in the outer wheels' riding on or outside the paved surface.

All of these circumstances—narrower roadways, tighter geometrics, older drivers, and larger and longer vehicles—result in vehicle wheels being positioned close to the pavement edge or, in some locations, off the pavement edge on the shoulder. This leads to two problems: pavement edge failure, and rutting of the adjacent shoulder edge. Even if pavement edge failure or rutting has not occurred, the wheel loads are causing deterioration of the pavement or shoulder structure, with fatigue and failure of the pavement edge to follow. Interestingly enough, if rutting of the shoulder occurs, the pavement edge fatigue is less extensive, but the situation is a greater potential hazard to motorists.

EDGE PROBLEM

Vehicle positioning by a driver has a major influence on the occurrence of pavement edge deterioration. While the combination of driver capability, vehicle performance, and change in alignment consistency interact, the other factors of driver expectations, actual (or potential) presence of opposing traffic, and roadway width versus vehicle path determine the presence or lack of pavement edge problems.

An important but not obvious factor is that the roadway pavement must match the actual and perceived path of the vehicle wheels. This aspect is important because the margin for location error both by the driver (in terms of lateral wheel placement relative to the pavement edge) and in the constructed location of the pavement is much smaller because of the narrower lane width. Thus in constructing a recreational road or improving its surface, better quality control of the pavement placement, in terms of lateral position, is necessary because small differences relative to driver positioning can lead to pavement edge fatigue and failure problems or shoulder rutting. Thus there is a greater need for accuracy in lateral positioning of the pavement to match the vehicle placement for the narrower lanes than for normal-width lanes. An alternative is to use a wider lane to compensate for the geometrics and positioning problems. If a wider pavement surface is unacceptable, there is a need for higher quality control and tighter controls of the paving operations and pavement positioning relative to vehicle wheel location than is current practice. This means more detailed work for design and more specific and rigorous operational controls for the construction and reconstruction of a roadway paved surface.

The reluctant use or nonacceptance of a widened pavement surface is not the only problem that generates concern. Just as there was with centerline striping of park and scenic roads in the 1960s, there is strong opposition to the use of painted edge stripes on park and scenic roads because it distracts from the scenic vista. This same reluctance carries over into the use of raised pavement markers, delineators, pavement widening, and paved shoulders. This dilemma arises from the fundamental problem of trying to preserve the natural surroundings while allowing access for its use and enjoyment.

For many parkways, the retention of an antiquated concept of the parkway is not in tune with the actual use of the roadway. Problems arise from differences in treatments, such as curve design. Other examples are the lack of advisory warning signs and the intentional lack of informational signs on some recreational roads. Other examples are the use of curve warning and advisory signs, and the nonuse of post-mounted delineators for curves. Practices vary widely.

It becomes obvious that uniformity does not exist for the motorist. Significant changes between design elements (e.g., degree of curvature being greatly different between two consecutive curves) can be troublesome. These differences become more of a concern with larger vehicles, higher speeds, and—especially—situations with large traffic volumes or adverse weather.

Although a road may be designated a parkway, it may operate as a thoroughfare. To better design, operate, and maintain scenic roads and parkways, the operational practices of those with low-volume characteristics must be distinguished from those with high-volume characteristics.

Unfortunately, most recreational road agencies and their managers still cling, rightly or wrongly, to the old concept and definition of a scenic or park road, or parkway. Even if a parkway designed in the 1930s has a moderate speed design for today, the operating behavior of motorists is appropriate for a higher-speed roadway. Driver expectations are difficult to change with a turn from an access road to a parkway or scenic road. With good intentions, most agencies have allowed some transitional design to be provided. A good example is the Blue Ridge Parkway's general treatment of prohibiting direct access from an Interstate highway. In the one instance where there is direct access, a transition roadway several miles long is provided.

For other parkways, however, these transitions usually are inadequate or nonexistent. A telling illustration of a poor transition is the access to the Foothills Parkway from I-40 to the Great Smoky Mountains National Park. This is a new parkway, only a few miles of which are completed that is intentionally being built in the 1970s tradition. There is a very

short and severe transition from the interstate ramp to the parkway.

Differences between and within recreational road agencies arise in their approaches to addressing the trade-offs of the function of a scenic or park road versus motorists's objectives and use of a roadway. Enjoying a leisurely drive along a scenic road is a preferred form of recreation for many Americans. Since the original conceptualization of parkways, more families and individuals possess the resources, time, and desire to enjoy the recreational driving experience. The recreational road has retained this idyllic image for its managers even though many of its actual characteristics have changed with heavy traffic and commuting volumes and speeds. It must be remembered that while the recreational road is important, so is the safety of the road user, especially with current liability issues.

DESIGN STANDARDS

The design standards of the 1930s are quite different than current design standards. They are illustrated both in the AASHTO Green Book (*17*) and the FHWA FAR 75 Manual (*18*). Early design standards for parkways are still in place because reconstruction of these roads appears to be on a nearly 50-year cycle. For some, such as the Blue Ridge Parkway, Natchez Trace, and Great River Road, 50 years covers the span of building the road. Thus design standards have changed since the earliest parkways were started (and also as each section was being built).

In 1984 the NPS recommended design standards for the construction of new park roads and for upgrading older park roads (*14*). Design widths indicated are primarily determined by the traffic volume. In the new standards, however, increased design speeds do not necessarily provide for increased design width.

Glennon (*19*) presented lane and shoulder width guidelines for low-volume rural roads. Here the criterion is that as design speeds increase, recommended design widths also increase. Downs and Wallace (*20*) make shoulder-width recommendations for rural roads with fewer than 400 vehicles daily. they suggest a 2-ft minimum graded shoulder on rural collectors and 4 to 8 ft of usable (paved or stabilized) shoulder on rural arterials.

Table 1 provides a comparison of lane and shoulder widths for low-volume roads as given in the NPS 1984 design standards for park roads and in Glennon's guidelines. The following table presents a comparison between the Blue Ridge Parkway design standards (as built) and the 1984 NPS standards:

	Blue Ridge	*NPS 1984*
Travel lane width (ft)	10	11
Shoulder width (ft)	3	4
Shoulder surface type	Unpaved (grass)	Paved

As the first comparison shows, park and scenic roads meeting the 1984 standards compare favorably with Glennon's recommended design widths at lower design speeds. At higher design speeds, however, the comparison is less favorable. As the table indicates, the comparison with the design standards to which the Blue Ridge Parkway was built is also unfavorable.

The concern here is not only with the older park roads and parkways that have not been upgraded. Some recently upgraded park roads, particularly those with higher design speeds, are considered to be narrower than the guidelines specify and therefore potentially troublesome.

Another real problem occurs when a new parkway is built and designers attempt to retain a 1970s design, such as in the Foothills Parkway at Great Smoky Mountains National Park. On other updated park roads, such as Trail Ridge in the Rocky

TABLE 1 Comparison of Recommended Lane and Shoulder Widths for Low-Volume Roads

	NPS (84)	Glennon
Design Speed (Mountain) (mph)	20-30	20-30
Lane Width (ft)	9*	10-11
Shoulder Width (ft)	1-2+	0
Total Width (single direction)	10-11	10-11
Design Speed (Rolling)	25-40	25-40
Lane Width (ft)	9*	11-12
Shoulder Width (ft)	1-2+	0
Total Width (single direction)	10-11	11-12
Design Speed (Flat)	30-50	30-50
Lane Width (ft)	9*	11-13
Shoulder Width (ft)	1-2+	0-2
Total Width (single direction)	10-11	11-15

* The National Park Service recommends an additional foot of lane width on roads where RV's exceed 5% of design volume or where tour buses are permitted. Lane widening may be used on inside edges of sharp curves.

Mountain National Park (Colorado), increased lane and shoulder widths were used. A good example of a park road that blends with its surroundings is in Wilson Creek Battlefield National Park in Missouri. It is clear that there is not yet total consensus on parkways and scenic road design parameters, especially with respect to design speeds and geometrics, and that the newest design standards are meeting with resistance in terms of their use for recreational roads.

Obviously, some agencies, managers, and designers believe that it is not suitable for a scenic or park road to be designed as a rural arterial. However changes in design standards will not entirely solve the recreational road problem. Part of the problem is the informal pulloff or stopping along a scenic or park road by motorists and their disregard for the roadside ecology. This is occurring along with a demand for more drive-through roads so that motorists do not have to get out of their vehicles to see the attractions. These developments present another set of challenges.

DETAIL NEEDS

A continuing theme is that a recreational road—be it a scenic, parkway, or access road—requires more attention to detail. If narrower lane widths are present or planned to be less than 12 ft, then even greater quality control, lane positioning, and detailing are needed in the design and construction phase. This is particularly true for longer scenic roads and parkways.

CONCLUSION

The challenge, as always, is to find acceptable design and construction procedures and approaches that continue to address the concerns and issues of these roads in terms of user, setting, and agency objectives while incorporating needed changes to address current situations. More detailed design and construction steps are a means to alleviate part of the problem. Maybe the title of W. H. Simonson's 1933 paper states it best: "the roadside picture: a hindrance to traffic? or an inspiring asset to travel?" (*6*). After 60 years, has anything changed?

REFERENCES

1. *Scenic Byways.* Report FHWA-DF-88-004. FHWA, U.S. Department of Transportation, July 1988.
2. Newton, N. Parkways and Their Offspring. In *Design on the Land: Development of Landscape Architecture.* Belknap Press, 1971.
3. Manning, W. H. Travelways of Beauty. *Landscape Architecture,* Vol. 20, 1930.
4. Zapatka, C. The American Parkway: Origins and Evolution of the Park-Road. *Lotus International,* Vol. 56, 1988.
5. Monis, F. *Bibliography on Roadside Control.* U.S. Bureau of Public Roads, 1949.
6. Simonson, W. H. The Roadside Picture: A Hinderance to Traffic or an Inspiring Asset to Travel? *Landscape Architecture,* Vol. 23, No. 2, 1933.
7. Boddy, J., and A. D. Taylor. Landscape Construction Notes XLIV—Supplemental Specifications for Roadside Improvement. Landscape Architecture, Vol. 26, No. 3, 1936.
8. Curtiss, H. L. The Roadside Zone in Our National Forests: An Administrative Policy in Answer to a Challenge. *Landscape Architecture,* Vol. 32, No. 1, 1942.
9. Bell, J. H. New Road Construction in Western National Parks: Notes on the Building of Roads in Scenic Areas. *Landscape Architecture,* Vol. 31, No. 2, 1941.
10. Dupree, D. D., Jr. Roadside Improvement: Landscape Design or Pansy Planting? *Landscape Architecture,* Vol. 30, No. 2, 1940.
11. Dupree, D. D., Jr. What Ohio Is Doing in the Roadside Development Program. Short Course on Highway Development. Architecture. *Ohio Transportation Engineering Conference Proc.,* Department of Architecture and Landscape Architecture, Ohio State University, 1941.
12. Committee of the Associated Landscape Architects of Cleveland Ohio. A Permanent Policy of Roadside Improvement: Improved Planning and Planting of Highways. *Landscape Architecture,* Vol. 26, No. 2, 1936.
13. Pragnell, R. C. *Scenic Road: A Basis for Its Planning, Design and Management.* Engineering Technical Report 7700-2. Division of Engineering, USDA Forest Service, 1970.
14. *Park Road Standards.* National Park Service, U.S. Department of the Interior, 1984.
15. *Scenics Byways Study Act.* 23 C.F.R., 1989.
16. *Highway Safety Improvement Programs, Progress and Evaluation Report 1990.* Ohio Department of Transportation, Columbus, 1990.
17. *Guide for Geometric Design of Urban Streets and Highways.* AASHTO. Washington, D.C., 1985.
18. *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects.* FHWA, U.S. Department of Transportation, 1985.
19. Glennon, J. C. *NCHRP Report 214: Design and Traffic Control Guidelines for Low-Volume Rural Roads.* TRB, National Research Council, Washington, D.C., 1979.
20. Downs, H. G., Jr. and E. W. Wallace. *NCHRP Report 254: Shoulder Geometrics and Use Guidelines.* TRB, National Research Council, Washington, D.C., 1982.

Publication of paper sponsored by Committee on Landscape and Environmental Design.