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Contents

Timber Pole Safety by Design

DON L. IVEY and JAMES R. MORGAN

ABSTRACT

A breakaway design for the modification of timber utility poles that will radically increase the safety of passengers in impacting vehicles has been developed and comprehensively tested. This design is called the Hawkins breakaway system (HBS). The system not only accomplishes the goal of increasing safety but exhibits characteristics of significant advantage to a utility company. A statement of safety philosophy applicable to the evaluation of roadside structures has been prepared. It can be used as the basis for the evaluation of any proposed safety improvement relative to roadside geometry and structures. It was used here to develop compliance tests for breakaway utility poles and to evaluate the results of those tests. Analysis of the literature relative to the cost-effectiveness of breakaway utility poles reveals that there will be a positive societal benefit associated with carefully selected applications.

Timber utility poles carrying power and communication transmission lines on highway rights-of-way are an anachronism. They represent a critical discontinuity in the "forgiving roadside," a concept developed and accepted in the 1960s and that state DOTs have striven to make a reality ever since. Timber utility poles are different from structures such as signs, luminaire supports, and hydraulic structures. They are owned by someone other than the highway or transportation entity responsible for the roadway. These transportation agencies have been hesitant, except under reconstruction conditions, to require a utility company to move or modify its facilities. There has been no consensus as to precisely who should be responsible for the influence on safety of timber utility poles within the highway right-of-way. In the past many utility companies appear to have assumed that highway safety was the responsibility of highway agencies. Although at times that attitude may have been justified, it may no longer be in the best interest of pole owners. Devices now exist that provide cost-effective safety treatments for exposed structures without significant detrimental influence on the primary objective (i.e., the transmission of power and information).

Until 1982 Southwest Research Institute (SwRI) performed most of the work in applying breakaway technology to timber utility poles. Beginning with a 1973 study by Wolfe and Michie (1) various arrangements of holes, grooves, and saw cuts were used to weaken the pole at its base so the pole would fall more easily during a vehicle impact. Another weakened zone was introduced near the top of the pole so that under impact conditions the middle section of the pole would break away leaving the top portion still connected to the utility lines. The best of these designs was called RETROFIX.

It appears that both the utility industry and the FHWA decided that RETROFIX should not be implemented. This was primarily because the pole was significantly weakened in its capacity to withstand environmental loads. To try to overcome the strength problem and other concerns of industry, the FHWA contracted with SwRI to develop a slip base breakaway design. The slip base designed by Bronstad for utility poles and used by Labra et al. (2) appears to be an adaptation of the triangular, three-bolt, multidirectional slip base developed by Edwards (3). It represents the first application of conventional slip base technology to a timber utility pole.

The primary objective of this work was to build on the conventional slip base technology to develop an implementable design. In addition to production of a more effective breakaway shear connection at ground level, this required overcoming the problems of pole detachment, conductor failure and entanglement, and the falling pole. This objective has been realized. A combination of a slip base lower connection and a progressively deforming upper connection has been subjected to five compliance tests. This combination of lower and upper connections has been named the Hawkins breakaway system (HBS) after D.L. Hawkins, who may have been the first to suggest slip bases on roadside structures (4). These tests have been compared on an acceleration, velocity change, and probability of injury basis to calculated values for unmodified poles. They also have been compared with a statistically derived probability of injury estimate for unmodified poles developed by Mak and Mason (5). The compliance tests conducted meet the criteria defined by NCHRP Report 230 (6).

The test selection was made using a new statement of safety philosophy that is described in detail in the full report (7). These comparisons will be detailed in a later section of this paper, but the net result may be stated as follows: In collisions at speeds of from 20 to 60 mph using automobiles of from 1,800 to 4,300 lb gross vehicle weight (GVW), the average probability of severe injury [abbreviated injury scale (AIS) $\geq$ 3] has been reduced by 91 percent. In collisions at speeds of from 40 to 60 mph, the probability of severe injury has been reduced by 97 percent. These reductions are far in excess of what most researchers considered probable. Zegeer and Cynecki (8) use example values of 30 and 60 percent reduction in injury and fatal accidents in their benefit-cost studies for FHWA. Although the 60 percent value may not be unreasonable if AIS injuries of 1 are considered, it appears that injuries would be heavily biased to the minor and moderate injury levels (AIS levels 1 and 2). Thus Zegeer's and Cynecki's use of the 60 percent overall reduction in injury and total accidents may still be too low when accident costs for the break-

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away design are calculated, and the HBS would be cost-effective in a wider spectrum of conditions than was predicted.

The HBS design consists of a slip base similar to those developed by TTI 17 to 20 years ago for use on sign and luminaire supports (4): an upper hinge mechanism and structural support cables (overhead guys) (Figure 1). The slip base connection is unique in that it is a six-bolt connection to reduce weight.

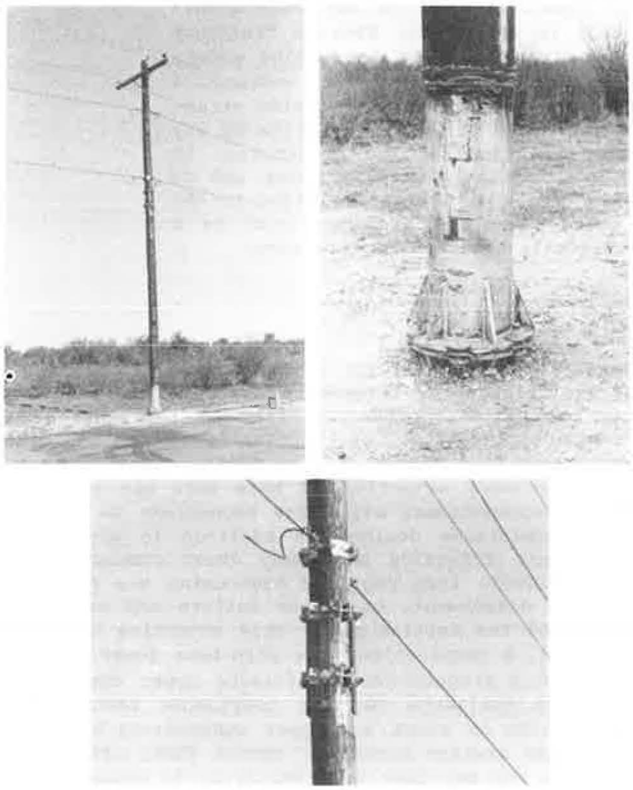


FIGURE 1 Modified utility pole installation.

These mechanisms are activated on impact and are intended to reduce the inertial effects of the pole on the errant vehicle while minimizing the impact on utility service. Typical performance of the HBS is shown in Figure 2. The slip base is designed to withstand the overturning moments imposed by in-service wind loads and, at the same time, slip when subjected to the forces of a collision.

A lower shear plane is created through installation of a slip base at an elevation of 3 in. above grade. The elevation of the slip base is intended to avoid snagging on the underside of an errant vehicle. This shear plane consists of two 5/8-in.-thick plates separated by a 26-gauge keeper plate (intended to maintain a bolt circle diameter of 15 1/2 in.) and by washers 2 1/2 in. in diameter by 1/8 in. The base plates are connected to each other by six 1-in.-diameter high-strength bolts with washers 2 1/2 in. by 1/4 in. These bolts are torqued to 200 ft-lb. Connection of the wooden utility pole to the slip base is through a steel pipe or tubing (Figure 3). These tubes are nominally 12 in. in diameter and 30 in. long and are welded to the base plates. In addition, the base plates are braced by 5/8-in.-thick stiffeners that are welded to both the base plate and the steel tube.

The upper hinge mechanism is sized to adequately transmit service loads while hinging during a collision to allow the bottom segment of the pole to rotate out of the way. This connection consists of two four-part pole bands installed above and below a saw cut through the pole and four straps connecting the two pole bands. The pole bands and straps are further secured to the pole by means of 1-in.-diameter through bolts as shown in Figure 4. At the bottom pole band, the bolts pass through the ends of the straps. At the lower end, the bolt holes are separated from four 1/2-in.-long slots by a 3/16-in. section of steel. Initial bending resistance is provided by the strength of this 3/16-in. margin. When the margin is punched out, resistance is offered by friction between the straps and bolts and by bending of the straps. When significant rotation has occurred, the bolts bear on the end of the slot, thereby providing the required ultimate bending strength. This upper connection reduces the effective inertia of the pole and minimizes the effect of any variation in hardware attached to the upper portion of the pole during a collision. The entire HBS system is designed to achieve the industry standard safety factor of four before ultimate failure. This design has been verified by static tests.

A series of tests was conducted to verify the performance of the HBS. In selecting the test matrix, it was necessary to define and adhere to a specific safety criterion. That criterion is:

A new structural design for a highway auxiliary structure should be strongly considered for implementation if

1. The new design results in significant improvement in safety for the majority of drivers and passengers,
2. The new design does not result in a significant deterioration in safety for any group of vehicle occupants, and
3. There are no other proven designs of equal or better cost-effectiveness that produce a safer condition for a larger spectrum of vehicle occupants.

Although this safety criterion may appear to be self-evident, its acceptance could allow use of structures that vastly improve the safety of the traveling public while not meeting all requirements of NCHRP Report 230 (6) or Transportation Research Circular 191 (9). Although the HBS does meet the requirements of NCHRP Report 230 and Transportation Research Circular 191, it will be demonstrated here how the alternate safety criterion can be applied.

The specific case under consideration is that of utility poles. The questions derived from the alternate safety criterion are:

1. Will breakaway poles result in a significant improvement in safety for the majority of drivers and passengers?
2. Will the design result in a significant deterioration in safety for any group of vehicle occupants (in this case, for drivers of very small cars)?
3. Are there other proven structural designs of equal or better cost-effectiveness that produce a safer condition for a larger spectrum of vehicle occupants?

It will be shown in later sections that breakaway utility poles implemented selectively, as suggested by both Mak and Mason (5) and Zegeer and Cynecki (8), will satisfy the proposed criterion. To prove that compliance, it was necessary to test proposed designs to determine if Element 1 was achieved. The approach to that was to select a series of compliance

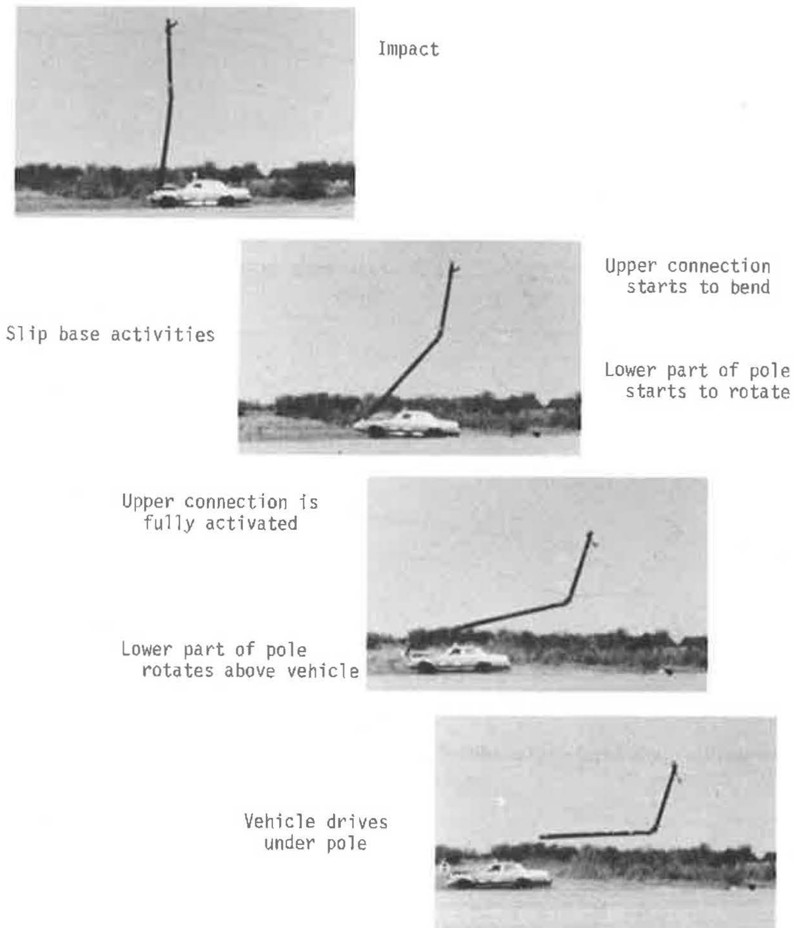


FIGURE 2 Function of Hawkins breakaway system during a vehicle collision.

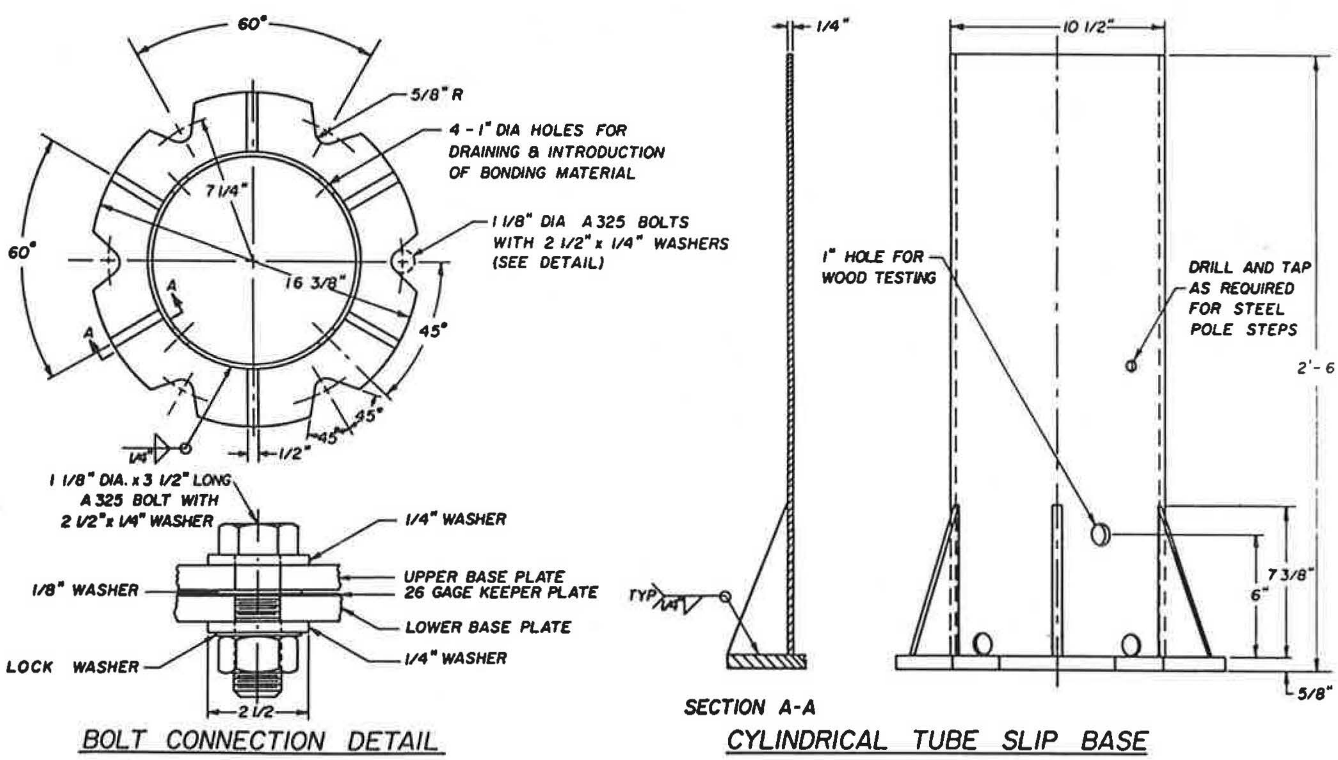


FIGURE 3 Lower connection—slip base.

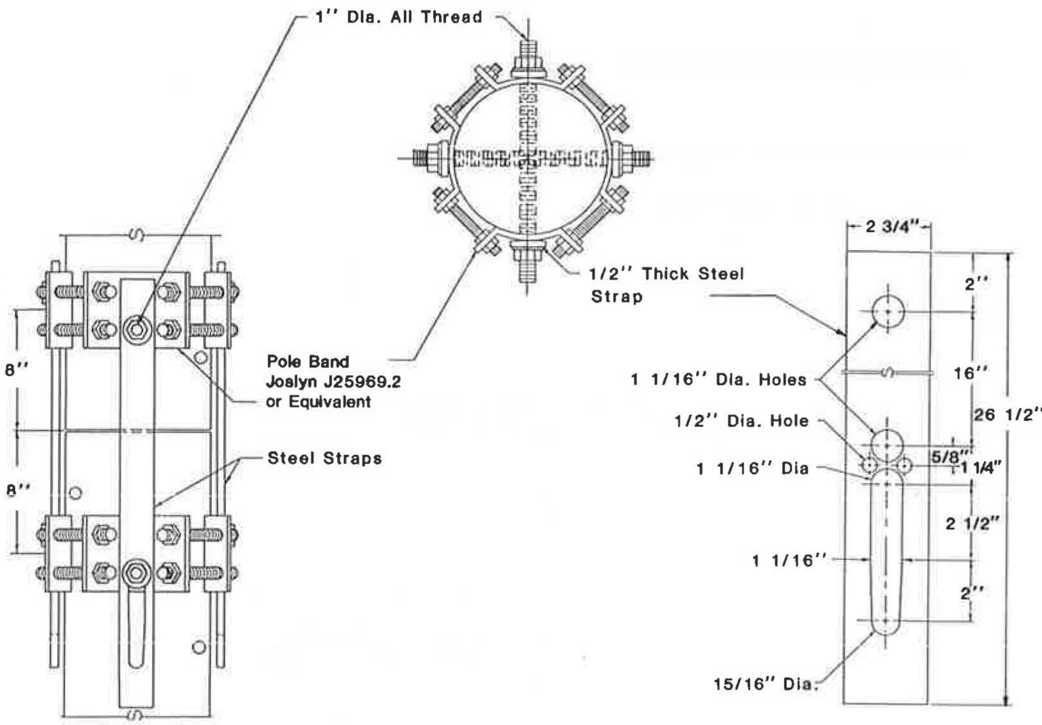


FIGURE 4 Upper connection—pole band with modified slotted straps.

crash tests that would encompass a clear majority of impact conditions.

The tests selected are given in Table 1. The primary purpose of each test is shown in the final column. The actual test conditions achieved are shown in parentheses. For example, in Test 1 the actual vehicle weight was 1,826 lb and the speed determined at impact was 39.9 mph.

HBS PERFORMANCE

The compliance tests outlined in Table 1 were conducted. These tests were performed on 40-ft, Class 4 timber utility poles retrofitted with the HBS. The results are detailed by summary sheets in Figures 5-9. In Table 2 changes in velocity, changes in momentum, and maximum average 0.050-sec accelerations are empirically determined for each test. The probability of injury estimates (percentage AIS $\geq$ 1, percentage AIS $\geq$ 3, and percentage PI) are made in the following ways:

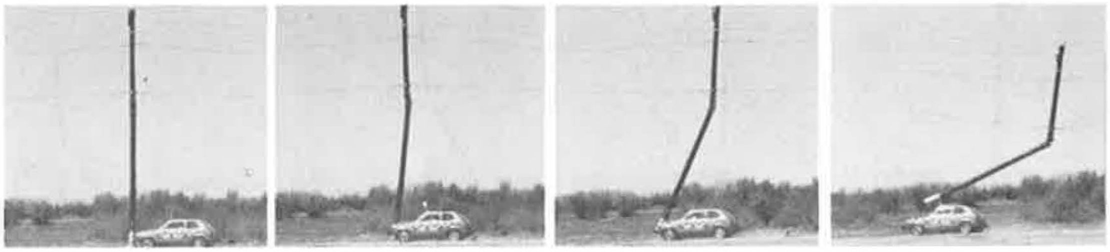
- Method 1, percentage AIS 1 and percentage AIS 3. For the tests conducted, this estimate can be made using Mak's and Mason's equation for velocity change (ΔV) and momentum change (ΔM) (5). For the hypothetical case of the same vehicle conditions on a nonbreakaway pole, a third equation by Mak, depending on vehicle impact speed (V), may be used to make the AIS estimates. Table 3 gives Mak's and Mason's equations.
- Method 2, probability of injury (percent). This estimate can be made using a relationship developed by Buth et al. (10). It depends on the highest average 0.050-sec resultant acceleration level determined from the test. For the hypothetical case of the same vehicle conditions and a nonbreakaway pole, the acceleration level must be calculated to obtain a probability of injury (PI) estimate from the same relationship. Table 3 gives the relationship described.

Although the comparison between any two injury

TABLE 1 Compliance Tests for Breakaway Utility Poles

Test No.	Vehicle Weight (test inertia mass, lb)	Vehicle Speed V (mph)	Vehicle Attitude	Primary Purpose of Test
1 (16)	1,700-1,900 (1,826)	38-42 (39.9)	Frontal, mid-50% (close to center)	Determination of probability of injury reduction for the most critical element of the design spectrum
2 (12)	1,700-1,900 (1,775)	18-22 (19.9)	Frontal, mid-50% (close to center)	Determination of probability of injury reduction for the lowest kinetic energy level at which pole structural activation would be expected
3 (13)	3,200-3,600 (3,365)	38-42 (40.7)	Frontal, mid-50% (close to center)	Determination of probability of injury reduction for the mid-range of automobile kinetic energy
4 (14)	2,300-2,700 (2,500)	58-62 (60.0)	Frontal, outer 50% (quarter point of bumper)	Determination of vehicle dynamic reaction to eccentric collision
5 (15)	4,300-4,800 (4,331)	58-62 (56.8)	Frontal, mid-50% (close to center)	Assessment of pole structural integrity at the highest kinetic energy level encompassed by the design spectrum

Note: numbers in parentheses refer to test numbers described in the text.



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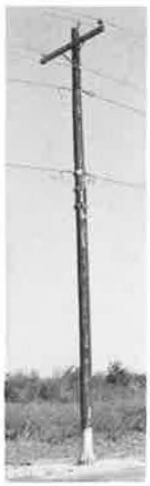
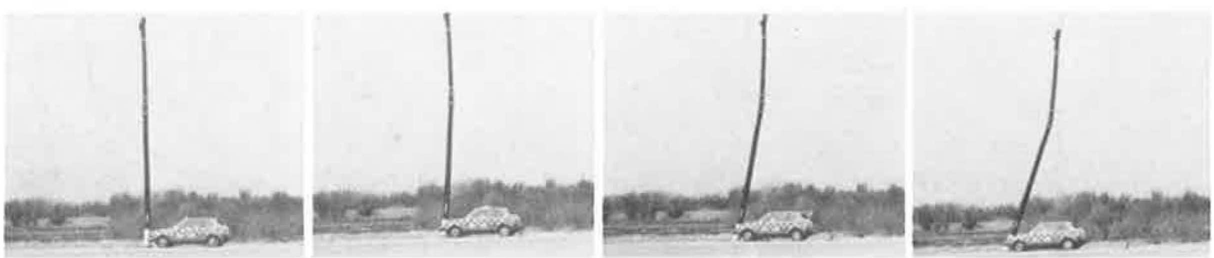


Test No.	4859-16
Date	4/03/85
Test Article	Breakaway Wooden Utility Pole
Lower Connection	Slip Base
Upper Connection	Pole Band No. 3
Vehicle.	1979 Honda Civic
Vehicle Weight	
Test Inertia	1826 lb (829 kg)
Gross Static	2160 lb (981 kg)
Vehicle Damage Classification	
TAD.	12FC2
CDC.	12FCEN2
Maximum Vehicle Crush	
Bumper Height.	10.0 in (25.4 cm)

Impact Speed	39.9 mi/h (64.2 km/h)
Change in Velocity	11.5 mi/h (18.5 km/h)
Change in Momentum	957 lb-s
Vehicle Accelerations (Max. 0.050 s Avg)	
Longitudinal	-8.0 g
Lateral	0.8 g
Occupant Impact Velocity	
Longitudinal.	12.0 fps (3.7 m/s)
Lateral	4.2 fps (1.3 m/s)
Occupant Ridedown Accelerations	
Longitudinal.	-1.0 g
Lateral	0.5 g

FIGURE 5 Summary of results for Test 4859-16 (Compliance Test 1).



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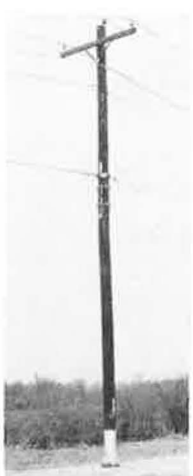
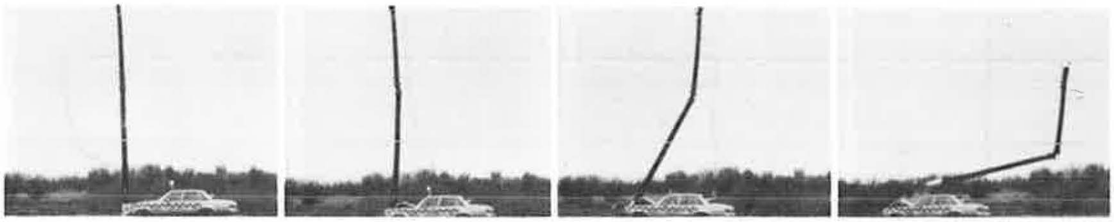


Test No.	4859-12
Date	2/20/85
Test Article	Breakaway Wooden Utility Pole
Lower Connection	Slip Base
Upper Connection	Pole Band No. 2
Vehicle	1979 Honda Civic
Vehicle Weight	
Test Inertia	1775 lb (806 kg)
Gross Static	2115 lb (960 kg)
Vehicle Damage Classification	
TAD.	12FC3
CDC.	12FCEN1
Maximum Vehicle Crush	
Bumper Height.	8.0 in (20.3 cm)

Impact Speed.	19.5 mi/h (31.4 km/h)
Change in Velocity*	11.3 mi/h (18.2 km/h)
Change in Momentum*	915 lb-s
Vehicle Accelerations (Max. 0.050 s Avg)	
Longitudinal.	-6.7 g
Lateral	0.7 g
Occupant Impact Velocity	
Longitudinal.	10.1 fps (3.1 m/s)
Lateral	3.5 fps (1.1 m/s)
Occupant Ridedown Accelerations	
Longitudinal.	-2.1 g
Lateral	1.9 g

*Impulse period computed from 0 to 0.500 sec.

FIGURE 6 Summary of results for Test 4859-12 (Compliance Test 2).



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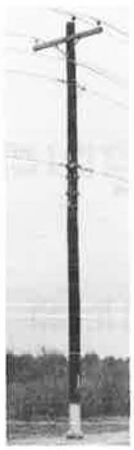
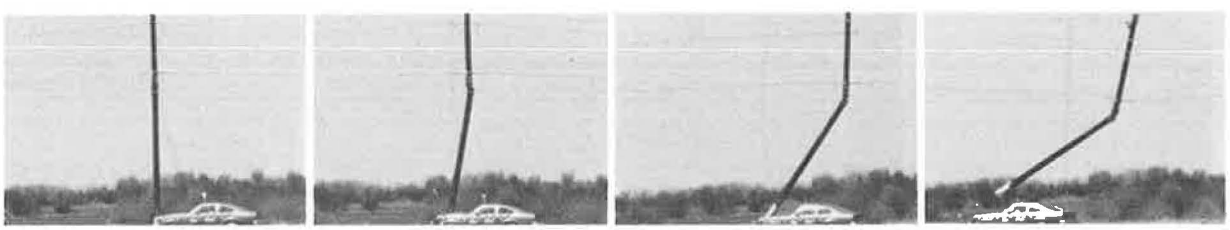


Test No.	4859-13
Date	2/27/85
Test Article	Breakaway Wooden Utility Pole
Lower Connection	Slip Base
Upper Connection	Pole Band No. 2
Vehicle	1980 Chevrolet Malibu
Vehicle Weight	
Test Inertia	3365 lb (1528 kg)
Gross Static	3700 lb (1655 kg)
Vehicle Damage Classification	
TAD	12FC5
CDC	12FCEN2
Maximum Vehicle Crush	
Bumper Height	18.7 in (47.5 cm)

Impact Speed	40.7 mi/h (65.5 km/h)
Change in Velocity	10.8 mi/h (17.4 km/h)
Change in Momentum	1655 lb-s
Vehicle Accelerations (Max. 0.050 s Avg)	
Longitudinal	-6.7 g
Lateral	1.4 g
Occupant Impact Velocity	
Longitudinal	11.9 fps (3.6 m/s)
Lateral	6.3 fps (1.9 m/s)
Occupant Ridedown Accelerations	
Longitudinal	-1.4 g
Lateral	1.1 g

FIGURE 7 Summary of results for Test 4859-13 (Compliance Test 3).



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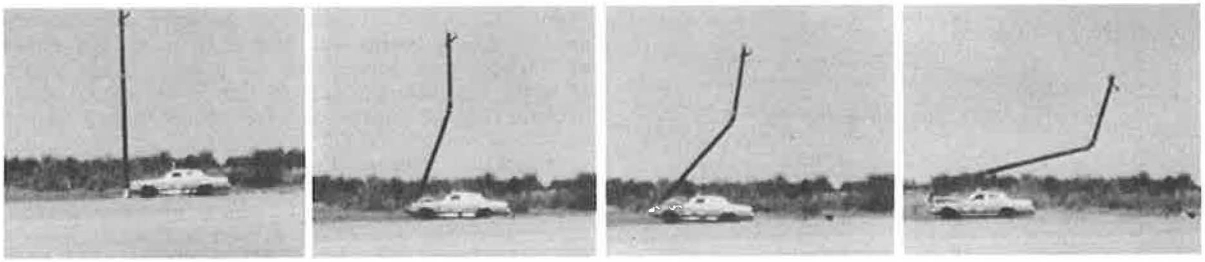
Test No.	4859-14
Date	3/22/85
Test Article	Breakaway Wooden Utility Pole
Lower Connection	Slip Base
Upper Connection	Pole Band No. 3
Vehicle	1975 Chevrolet Vega
Vehicle Weight	
Test Inertia	2500 lb (1135 kg)
Gross Static	2830 lb (1285 kg)
Vehicle Damage Classification	
TAD	12FR3
CDC	12FREN2
Maximum Vehicle Crush	
Bumper Height	15.0 in (38.1 cm)

Impact Speed	60.0 mi/h (96.5 km/h)
Change in Velocity	11.0 mi/h (17.7 km/h)
Change in Momentum	1253 lb-s
Vehicle Accelerations (Max. 0.050 s Avg)	
Longitudinal	-10.2 g
Lateral	- 1.3 g
Occupant Impact Velocity	
Longitudinal	15.6 fps (4.8 m/s)
Lateral	No Contact
Occupant Ridedown Accelerations	
Longitudinal	-1.8 g
Lateral	NA

FIGURE 8 Summary of results for Test 4859-14 (Compliance Test 4).



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Test No. 4859-5
Date 6/29/84
Test Article . . . . Breakaway Wooden Utility Pole
Lower Connection . . . . . . Slip Base
Upper Connection . . . . . . Pole Band No. 2
Vehicle. . . . . 1979 Chrysler Newport
Vehicle Weight
  Test Inertia . . . . 4331 lb (1966 kg)
  Gross Static . . . . 4665 lb (2118 kg)
Vehicle Damage Classification
  TAD. 12FC4
  CDC. 12FCEN3
Maximum Vehicle Crush
  Bumper Height. . . . . . . 28.0 in (71.1 cm)
  Hood Height. . . . . 22.0 in (55.9 cm)

Impact Speed. . . . . 56.8 mi/h (91.4 km/h)
Change in Velocity. . . . . . 7.0 mi/h (11.3 km/h)
Change in Momentum. . . . . . 1487 lb-s
Vehicle Accelerations
  (Max. 0.050 s Avg)
    Longitudinal. . . . . . . -4.9 g
    Lateral . . . . 0.6 g
Occupant Impact Velocity
    Longitudinal. . . . . . . 10.7 fps (3.3 m/s)
    Lateral . . . . None
Occupant Ridedown Accelerations
    Longitudinal. . . . . . . -0.8 g
    Lateral . . . . No Contact

FIGURE 9 Summary of results for Test 4859-5 (Compliance Test 5).

TABLE 2 Injury Rate Levels for Compliance Tests

	Change in Velocity			Change in Momentum			0.050-sec Avg Acceleration		Probability of Injury for Unmodified Pole		
Test No	ΔV (mph)	AIS ≥ 1 (%)	AIS ≥ 3 (%)	ΔM (lb-sec)	AIS ≥ 1 (%)	AIS ≥ 3 (%)	g	PI (%)	AIS ≥ 1 (%)	AIS ≥ 3 (%)	PI (%)
1 (16)	11.5	66.0	1.42	987	52.3	0.38	8.0	21.5	81.3	22.4	100
2 (12)	11.3	65.7	1.39	915	51.5	0.36	6.7	15.1	70.2	2.5	60
3 (13)	10.8	64.9	1.31	1,655	61.5	0.74	6.77	15.1	81.3	22.4	66
4 (14)	11.0	65.3	1.34	1,253	56.8	0.50	10.2	35.0	87.8	76.5	79
5	7.0	57.2	0.83	1,487	59.7	0.63	4.9	8.1	72.6	2.58	26.5

Note: numbers in parentheses refer to test numbers described in the text.

TABLE 3 Probability of Injury Equations

Description	Equation
Mak and Mason *(5)*	% AIS ≥ 1 = −63.5 + 16.87 Ln(ΔM)
Percentage AIS as a function of momentum change, ΔM (lb-sec)	% AIS ≥ 3 = $100/[1 + e^{65-0.00097(\Delta M)}]$
Mak and Mason *(5)*	% AIS ≥ 1 = 22.2 + 16.03 Ln(V)
Percentage AIS as a function of impact speed, V (mph)	% AIS ≥ 3 = $100/[1 + e^{6.08-0.121(V)}]$
Mak and Mason *(5)*	% AIS ≥ 1 = 22.5 + 17.83 LN(V)
Percentage AIS as a function of change in velocity, ΔV (mph)	% AIS ≥ 3 = $100/[1 + e^{5.62-0.12(\Delta V)}]$
Buth and Ivey *(10)*	
Probability of injury (%) as a function of highest resultant 50-msec acceleration, Ar (g 's)	PI = 0.336 Ar P = 0.336 Ar

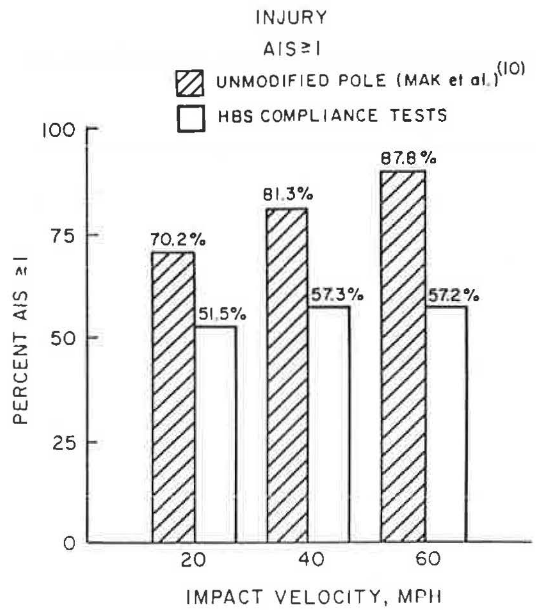


FIGURE 10 Comparison of injury levels from HBS compliance tests with unmodified pole injury levels (% AIS 1).

rate levels for any test can be seen by examining Table 2, it is somewhat easier to compare those levels using Figures 10 and 11. These bar graphs were developed for each test speed using Method 1 and present the average injury level for all tests at that speed. In Figure 10 it is seen that a significant improvement results. The greater improvement, however, is shown by Figure 11. A major decrease in the AIS $\geq$ 3 injury rate is demonstrated. This decrease, for the five compliance tests conducted, averages 91 percent. It is apparent from

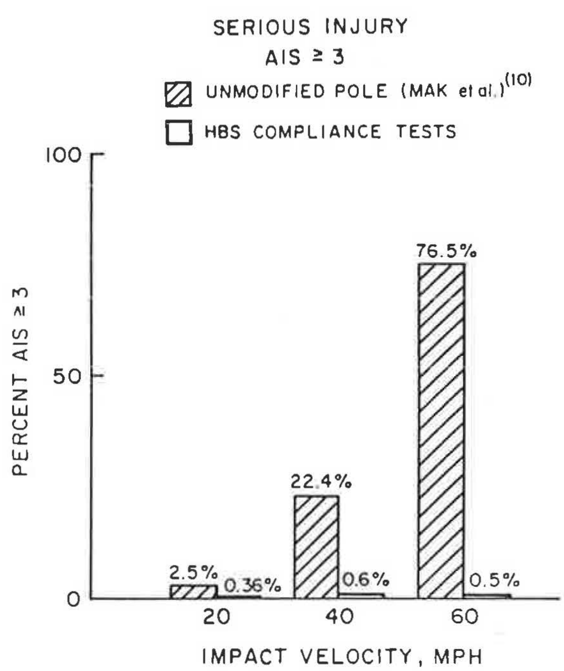


FIGURE 11 Comparison of injury levels from HBS compliance tests with unmodified pole injury levels (% AIS 3).

Figure 11 that the reduction becomes more pronounced as the speed increases. There is a slight advantage at 20 mph that progresses to a major improvement at 60 mph. For the 40- and 60-mph test conditions, the probability of injury greater than AIS = 3 is reduced by 97 percent.

Finally, Figure 12 was constructed using all available test data and a computer simulation. This figure shows the various zones of interaction between vehicles and HBS-modified poles. It also shows the calculated failure boundary for unmodified Class 4 timber utility poles. The activation boundary for the HBS occurs at about 10 mph for small vehicles and will decrease slightly as vehicle weight increases. As speed increases, the next zone is where the lower connection is activated and the pole is pushed in front of the impacting vehicle. The vehicle then stops and the pole leans on or descends on the vehicle. The velocity of the falling pole is so low that significant passenger compartment intrusion will not occur. This was illustrated by Compliance Test 2.

In the next zone the vehicle will go completely under the pole, but the pole will make contact with the roof or truck structure as the vehicle moves through. Passenger compartment intrusion will be minimal in this zone because of the rotation of the lower pole segment to a position where it will glance off or be pulled across the roof structure. The zone is not precisely defined but will vary as vehicle structural stiffness and coefficient of restitution vary. Finally, the zone where the pole clears the vehicle after impact is everywhere to the right of Curve C. This is the zone illustrated by compliance Tests 1 and 3-5.

COMPLIANCE WITH NCHRP REPORT 230

It should be recognized that the recommendations for timber utility poles were considered extremely tentative by the writer of NCHRP Report 230 (6). The development of breakaway devices for these structures was in its infancy and no one was sure it could be done. The recommendations for "Occupant/Compartment Impact Velocity" and "Occupant Ride Down Acceleration" were based more on what the author considered possible than on what would be preferred. In Table 8 of NCHRP Report 230, an acceptance factor of 1.33 was recommended. This resulted in values of ΔV of 30 fps and acceleration of 15 *g*'s.

It appears now that breakaway timber utility poles can be engineered to perform significantly better than the values that were recommended in 1981 would indicate. This can be seen by comparing the results of tests recommended in NCHRP Report 230 for breakaway on yielding supports to the values of velocity change and acceleration given previously in this paper. Table 4 gives this comparison. The required tests are 60 and 61, although in this case test 61 is substituted for 60; 62 is a more demanding test. The other test conducted was not required but is described as a possible supplementary test in Table 4 of NCHRP Report 230 (6). This is Test S64, an 1,800-lb vehicle at 40 mph impacting at the center of the bumper.

As can be seen, the HBS results are well below the maximum values given by NCHRP Report 230 for timber utility poles and fundamentally meet the requirements for signs and luminaire supports. They are well within the requirements for ridedown acceleration and, with one exception, meet the occupant/compartment impact velocity. That exception is Test 61 in which a ΔV of 15.6 fps was observed, compared with a recommended limiting value of 15 fps. Given the variability in crash testing,

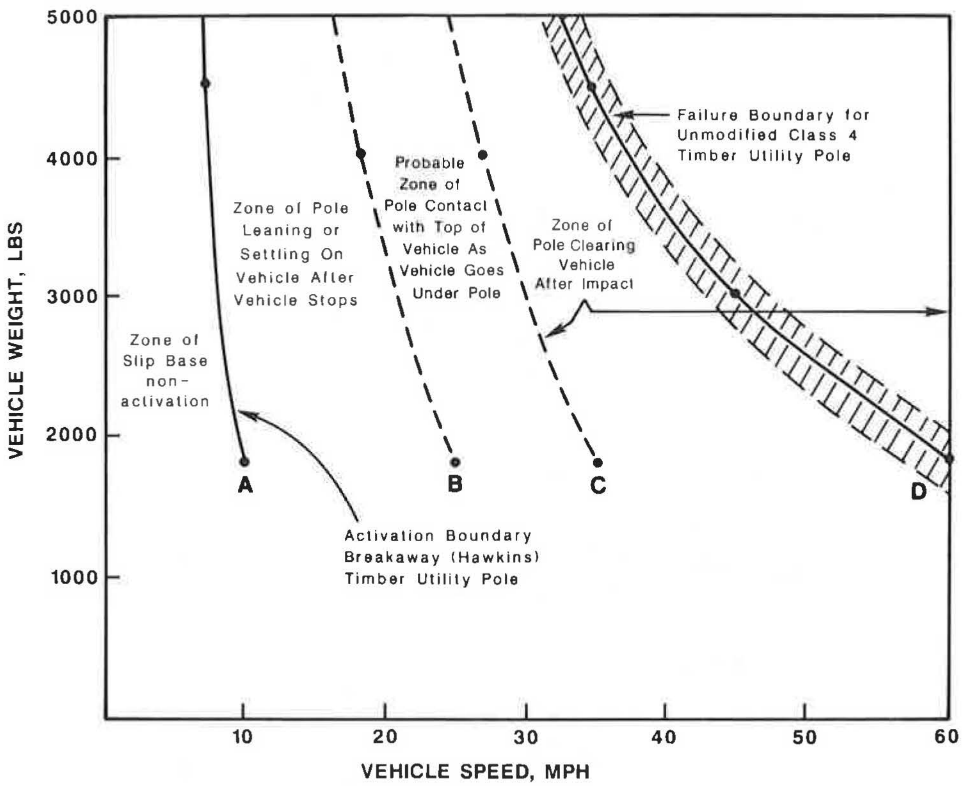


FIGURE 12 Zones of vehicle-pole interaction.

there is no reason to be overly concerned by this result. It appears that an acceptance factor higher than the 1.33 value proposed in 1981 might be considered for timber utility poles.

CONCLUSION

A breakaway design for the modification of timber utility poles that will radically increase the safety of passengers in impacting vehicles has been developed and comprehensively tested. It is called the Hawkins breakaway system (HBS). This system not only accomplishes the goal of increasing safety but exhibits characteristics of significant advantage to a utility company.

An alternate safety criterion to be applied in the evaluation of roadside structures also has been developed. It can be used as the basis for evaluation of any proposed safety improvement relative to roadside geometry and structures. It was used to develop compliance tests for breakaway utility poles, but its applicability is general to the roadside environment.

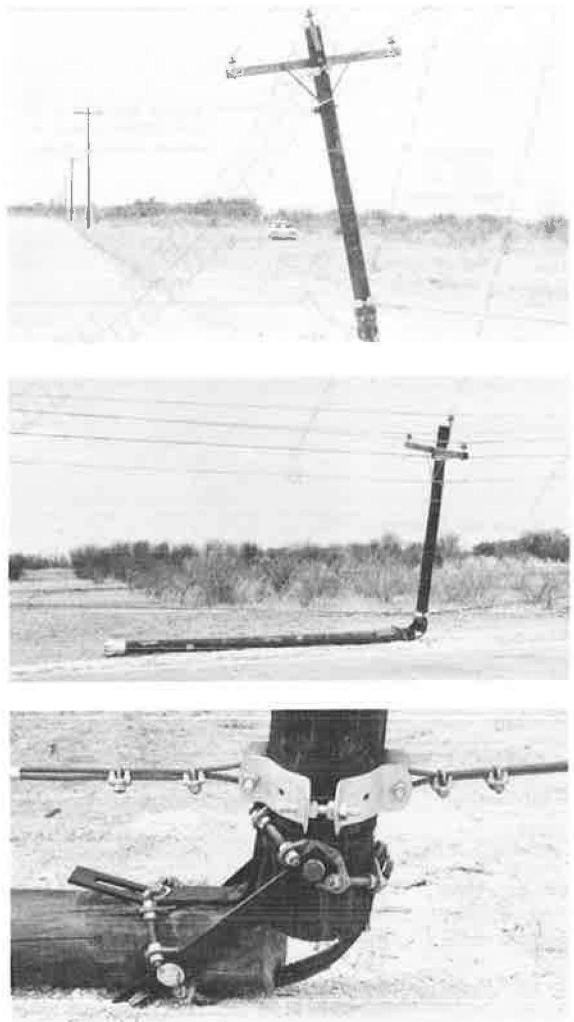
Analysis of the literature relative to the cost-effectiveness of breakaway utility poles reveals that there will be a positive societal benefit associated with carefully selected applications. The work of Zegeer and Cynecki (8) may be used to define appropriate applications, although Sicking and Ross (11) have recently developed a somewhat more comprehensive benefit-cost analysis.

Detailed conclusions are

- The HBS has been adapted and applied to 40-ft, Class 4 timber utility poles (4/0 construction). The primary system developed for this type of construction consists of a slip base, an upper hinge mechanism, and overhead guy support cables. This adaptation of the HBS virtually eliminates the chance of serious injury in a wide range of vehicle collisions.
- Excellent performance has been achieved for vehicles ranging from 1,800 to 4,500 lb at speeds of from 20 to 60 mph. Mak and Mason (5) have found that there is little chance of serious injury at speeds lower than 20 mph, even for an unmodified pole.
- The original cost of the HBS for a single pole modification should be less than $800. It is estimated that a three-person crew with a digger-derrick and insulated aerial device can make all of the necessary repairs within a 4-hr period following an accident. Assuming an area with congested traf-

TABLE 4 NCHRP Report 230 Compliance Tests

NCHRP Test Designation	TTI Test Designation	Weight Suggested (lb)	Weight Achieved (lb)	Speed Suggested (mph)	Speed Achieved (mph)	ΔV Suggested (fps)	ΔV Achieved (fps)	a Suggested (g's)	a Achieved (g's)
61 (substitute for 60)	4859-14	2,250	2,500	60	60.0	30	15.6	15	1.8
62	4859-12	1,800	1,775	20	19.5	30	10.1	15	2.1
564	4859-16	1,800	1,826	40	39.9	30	12.0	15	1.0



End view

Side view

Fully activated
upper connection

FIGURE 13 HBS-modified utility pole after a high-speed collision (Test 4859-3).

fic, energized electric power lines, and night work conditions, the manpower, material (including a new pole but excluding breakaway hardware), and equipment costs are estimated at $875. Because a new pole will not always be required, the average cost may be somewhat lower. In addition, some of the breakaway hardware may need to be replaced (miscellaneous nuts and bolts and a keeper for low-speed impacts, plus two straps in higher speed impacts). The cost for replacement of breakaway hardware should be less than $150.

• On the basis of the results of the compliance tests reported here, it appears that most other types of Class 4 construction could be treated in a similar manner, yielding similar results.

The HBS is ready for implementation. Used selectively, it holds the potential to make a significant reduction in the 1,600 deaths and 100,000 injuries that occur annually as a result of collisions with timber utility poles (12). In addition, significant advantages to utility companies will accrue as selective implementation is undertaken (7). One major benefit is illustrated by Figure 13. After a vehicle collision, a utility maintenance crew will find a shortened pole, with conductors still intact and functioning, instead of a tangle of conductors and broken pole segments.

ACKNOWLEDGMENT

This paper is dedicated to Dr. Robert M. Olson, Professor Emeritus of Texas A&M University, who died suddenly on April 4, 1986. He had a major influence on the lives of both authors and was a continual source of inspiration during a 25-year period as both teacher and associate. Dr. Olson worked with D.L. Hawkins in the late 1960s to develop the nationally accepted breakaway highway signs. The development of the Hawkins breakaway system for utility poles was grounded firmly on their pioneering efforts.

The influence and inspiration of Bob Olson will live on in the students he taught and in the associates he so unselfishly helped. The present authors are two among the many.

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The contents of this paper reflect the views of the Texas Transportation Institute, which is responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official policy of the U.S. Department of Transportation.

Pennsylvania's Guide Rail Standards: A Cost-Effective Change

LOUIS C. SCHULTZ, Jr., et al.

ABSTRACT

In 1985 the Pennsylvania Department of Transportation implemented a systematic technique to inventory the condition and location of guide rail and median barrier along Pennsylvania's highways. This process has been developed to supplement the department's systematic technique to analyze and manage Pennsylvania pavements (STAMPP), the foundation of the department's total roadway management information system. A major recommendation of the task force that developed the guide rail inventory was to review the department's guide rail standards and warrants and, where feasible, to make revisions. A special task force, comprised of district, county, and central office personnel reviewed other states' criteria and research reports and consulted with a number of state and federal officials. The AASHTO cost-effectiveness approach that is detailed in AASHTO's "Guide for Selecting, Locating, and Designing Traffic Barriers" (1) was then used to develop guide rail warrants that are both cost-effective and safe. The task force also recommended several other changes to the department's guide rail standards that will result in a more efficient use of resources without compromising the safety of the roadside environment. The recommended changes and the procedures used are outlined in this paper.

The older, nonstandard guide rail systems on the Legislative Route network are in a general condition of disrepair. The more recently installed sections, which represent just a small portion of the overall system, are in an acceptable condition. Statewide there has been a varying commitment of resources to upgrading and repair. As a result much of the system is not consistent with current standards and not capable of functioning in the desired manner. Because of the large size of the guide rail system, updating and repair within fiscal abilities would require rechanneling of resources from other equally or more important programs, such as resurfacing, surface treatment, or bridge repair.

OBJECTIVES

A task force was assembled to analyze the problem and recommend appropriate actions for reaching an overall solution. To accomplish this end, the task force was to

1. Evaluate and reestablish guide rail warrants using a cost-effectiveness analysis,
2. Identify areas where existing guide rail can be removed,
3. Review design standards and recommend areas of cost reduction, and
4. Recommend an implementation program.

METHODOLOGY

An extensive literature search was conducted to determine existing warrants for guide rail placement, cost-effectiveness approaches to guide rail selection, and reduced criteria for guide rail selection on primarily low-volume roadways. As a result of the literature search, it was determined that the existing warrants for guide rail, presented in the 1977 AASHTO "Guide for Selecting, Locating, and Designing Traffic Barriers" (1) and adopted by the Pennsylvania Department of Transportation (PennDOT) as warrants, were for the most part used on a nationwide basis. However, the need to consider the cost-effectiveness of guide rail installations is emphasized in a number of publications.

Calcote (2) notes that "it has become of critical importance that a cost-effectiveness formulation be included as an aid in the decision-making policy." The 1977 AASHTO barrier guide includes an entire chapter, Chapter VII, that addresses guide rail use based on encroachment frequencies, which depend on average daily traffic (ADT); severity of impact against a warranting feature; embankment slopes and heights; and available clear zone. This analysis evaluates three alternatives for every situation:

- Remove or reduce the hazard so that shielding it is not necessary (e.g., flatten slopes);
- Install a barrier; or
- Do nothing (leave the hazard unshielded).

The task force also reviewed a revised cost-effectiveness procedure that was presented by the National Highway Institute and included in a 1980 supplement to the AASHTO guide (3). The Georgia Department of Transportation developed criteria for cost-effective guide rail selection that contains reduced clear zone widths based on lower ADTs, embankment slope, and operating speed, as well as significantly reduced warrants for fill heights and embankment slopes based on ADT. The Georgia criteria, which have been accepted by their FHWA division office, were used as a model procedure for determination of revised warrants based on traffic volumes and roadway geometry (see Appendix).

In any analysis of or decision on use of guide

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rail at a specific site, several criteria are evaluated in their order of importance:

- Embankment slope and height,
- Presence of fixed objects that may present a hazard, and
- Clear zone.

The present PennDOT Design Manual, Part 2, criteria allow calculation of reduced clear zone width based on traffic volume. The task force thought that the widths obtained from these calculations could be more easily presented as a table of numbers for site-specific operating speed and ADT and developed Table 1. Previous research had established that 85 percent of all vehicles that leave the roadway recover within 30 ft of the edge of pavement. This 30-ft-wide clear recovery area has long been a part of Pennsylvania's design criteria. After careful review and discussion, it was the consensus of the task force that this 30-ft figure should be retained as the desirable maximum clear recovery area, with site-specific reductions as indicated in Table 1.

TABLE 1 Clear Zone Widths (ft) by ADT and Operating Speed

Slope	ADT: More Than 6,000	2,000-6,000	800-2,000	250-800	Fewer Than 250
Operating Speed = 40 mph					
3:1 fill	20	18	17	15	13
4:1 fill	18	17	15	14	12
6:1 fill	17	16	14	13	11
Flat	15	14	13	11	10
6:1 cut	15	14	13	11	10
4:1 cut	15	14	13	11	10
3:1 cut	15	14	13	11	10
Operating Speed = 50 mph					
3:1 fill	30	30	30	30	30
4:1 fill	27	24	22	20	18
6:1 fill	21	19	17	16	14
Flat	20	18	16	15	13
6:1 cut	20	18	16	15	13
4:1 cut	18	16	15	14	12
3:1 cut	16	15	13	12	11
Operating Speed = 60 mph					
3:1 fill	30	30	30	30	30
4:1 fill	30	30	30	30	28
6:1 fill	30	30	27	25	22
Flat	30	27	25	23	20
6:1 cut	28	25	23	21	19
4:1 cut	25	23	21	19	17
3:1 cut	20	18	17	15	14

An in-depth analysis of Georgia's embankment slope and fill-height criteria was performed to evaluate the applicability of Georgia's numbers to typical conditions in Pennsylvania. A table for reduced criteria was developed (Table 2). The analysis was highly dependent on the severity index assigned to sets of slope and height conditions and was calculated using a formula developed through testing results reported in NCHRP Reports 115 (4) and 174 (5): $\log SI = 0.556 + 0.160 \log h + 0.324 \log s$, where SI is the severity index, h is the embankment height, and s is the side slope of the embankment.

A reanalysis was conducted using computer programs containing Pennsylvania-specific criteria and cost data. Finally, a review of the same cost-effectiveness methods was made for the warrants for fixed objects and nontraversable hazards. Again, a revised DM-2 table was developed (Table 3).

TABLE 2 Guide Rail Warrants (ft) for Fill Heights and Slopes

Slope	ADT: More Than 5,000	751-5,000	400-750	Fewer Than 400	Current Criteria (all ADT)
1 1/2:1	4	6	9	17	2
2:1	8	10	16	31	5
2 1/2:1	12	16	25	49	8

FINDINGS AND RECOMMENDATIONS

1. Guide rail warrants should be revised to reflect the fill heights given in Table 2.
2. Guide rail should not normally be used to protect utility poles and trees.
3. Design Manual, Part 2, Chapter 12 should be revised as discussed in this paper.
4. The height of weak-post systems should be reduced to 30 in. for all new construction.
5. The height of all strong-post systems should be reduced to 27 in. and rub rails should be eliminated for all new construction.

TABLE 3 Warrants for Fixed Objects

Fixed Objects Within Clear Zone	Guide Rail Required: Yes	No
Sign support (ground mounted)		
Post of breakaway design		X
Sign bridge supports	X	
Concrete base extending 6 in. or more above ground	X	
Lighting poles and supports of breakaway design		X
Bridge piers and abutments at underpasses	X	
Retaining walls and culvert headwalls 6 in. or more above ground	X	
Trees		X
Utility poles		X
Lighting poles with high-mast lighting	X	

6. Bridge approach treatments as shown in the standard drawings should be modified. If guide rail is needed strictly to protect a parapet end, it should consist of 25 ft of Type 2-SC with no rub rail and 25 ft of Type 2-S with no rub rail and a standard end treatment. If the rail height cannot be kept at 27 in., the rub rail should be used.
7. Drawing RC-54, sheet 1 of 3, Note 4 should be revised. The minimum distance from a solid obstruction to the beginning of the guide rail should be changed from 125 to 50 ft.
8. Use of the training tape on guide rail design should be discontinued until it is modified. This modification should be done so that issuance of the new tape can coincide with that of the standard revisions noted previously.
9. The department's administration should formulate a request to the legislature that the tort laws be revised to place the burden of safe operation of motor vehicles on the driver. The right to initiate litigation or a tort claim should be denied to those operating outside the provisions of the law.
10. When an engineer is reviewing plan details and considering the need for guide rail, he should evaluate the previous accident history, the roadway

geometry, and the like in deciding, on the basis of his engineering judgment, whether to install guide rail.

11. Although the proposed guide rail warrants provide for a cost-effective approach to guide rail installation or removal, the district engineer retains the option of providing guide rail treatment at locations with high impact potential, where personal safety would be compromised, or in socially sensitive locations.

IMPACTS OF CHANGES

It is estimated that the recommended modifications to the guide rail criteria will save the department approximately $4 million each year. The savings accrue from the following sources:

Reduced length of protection at bridge parapets	$2,184,000
Elimination of rub rail on 2-S guide rail	1,700,000
Reduced need for guide rail due to fill height revisions	1,235,000
Reduced length of 2-S guide rail at fixed objects	101,250
Subtotal	$5,220,250
Annual cost to remove unwarranted guide rail	1,800,000
Net annual savings	$3,420,250

The task force estimates that 40 percent of the guide rail in Maintenance Functional Classification (MFC) D and E routes can be removed at a cost of $1.00 per foot. This work would be accomplished during a 4-year period and would result in the $1,800,000 annual cost figure. Other calculations and data supporting these findings are as follows:

1. Impact of changing the present minimum 100 ft of 2-S guide rail to 25 ft of 2-S for bridge parapet connections only. Annually an average of 719 bridges undergo updating of three of the four lengths of guide rail warranted for the bridge ends: 719 x 225-ft reduction x $13.50 per linear foot (LF) = $2,184,00.

2. Impact of changing strong post from 33-in. height with rub rail to 27-in. height without rub rail. Annually 400,000 LF of 2-S and 2-SC are placed; rub rail cost = $4.25: 400,000 x $4.25 = $1,700,000.

3. Impact of deletion of guide rail due to fill height revisions. 400,000 x 25% = 100,000 LF (50% 2-S, 50% 2-W); average run = 500 LF; 200 runs x 2 = 400 end treatments. Annual savings:

2-S	50,000 LF x $13.50	$ 675,000
	200 breakaway cable terminal end treatments x $950	190,000
2-W	50,000 LF x $6.00	300,000
	200 2-W end treatments x $350	70,000
Total		$1,235,000

4. Impact of changing minimum 100 ft of 2-S to 25 ft of 2-S for fixed-object warrants other than bridges at 100 locations annually: 100 x 75 ft x $13.50 = $101,250.

5. Impact of changing post spacing on low-volume, narrow roads with speed of 40 mph or less from 6 ft 3 in. to 12 ft 6 in. (not adopted): post ($43.42) + offset bracket ($7.52/12.5 ft) = $4.07 per foot (rounded to $4.00 per foot); D and E W-beam rail annually is maintained or upgraded and 50 percent of that could be installed at 12 ft 6 in.: 3,000,000 LF x 5% x 50% x $4.00 = $300,000.

IMPLEMENTATION

The task force recommends that steps be taken to immediately implement the proposed revisions to the guide rail criteria and standards. It is recognized that there are certain procedural clearances that must be given before final adoption as policy. However, the potential savings of $3.5 million per year is considered significant enough to warrant issuing interim criteria via a strike-off letter. This will enable the immediate application of these criteria on all projects currently being designed or scheduled for design this fall and winter.

It has been estimated that, if these criteria are followed, approximately 40 percent of all guide rail on MFC D and E roads can be eliminated, thus eliminating a maintenance problem and improving roadside aesthetics. The task force recommends that all unwarranted guide rail be removed as soon as feasible, with all work to be completed within 4 years. This work can be accomplished either by department forces, within their annual work plans, or by contract, as is now done for guide rail repair and upgrading.

Finally, the task force recommends that the estimated $4 million annual savings that will be realized be applied to upgrade substandard guide rail that will still be required under the new criteria. This is imperative if the real benefits, more miles of improved guide rail along Pennsylvania's highways, are to be realized. An annual program to systematically upgrade substandard guide rail in accordance with these criteria can produce savings in terms of improved highway safety, reduced tort liability, and decreased maintenance needs. Moreover, it is recommended that each district, as part of its annual Energy Conservation, Congestion Reduction, and Safety Improvement Program, include a project to upgrade guide rail protection at bridge parapets. This is an area of increasing tort claims, and the department may be able to significantly reduce its tort liability exposure by doing so. More important, obvious safety benefits will be realized by protecting the motoring public from blunt bridge ends.

The task force identified one additional consideration, use of strong-post guide rail at 12 ft 6 in. spacing on low-speed, low-volume roads. It was the consensus of the task force that motorists would be unable to strike this guide rail at sharp angles, thus minimizing the probability of pocketing. Moreover, savings of $300,000 per year are projected. However, because of the lack of adequate research, this was not included in the package of modifications presented for department-wide review. It is recommended that this suggestion be considered for future research to determine what the effects will be when 12 ft 6 in. strong-post guide rail is struck at flat angles.

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Raymond R. Radomsky, Manager, Susquehanna and Wyoming County Maintenance Districts, Montrose; and William Tritle, Assistant District Maintenance Manager, Engineering District 6-0, St. Davids. Robert K. Burk, Assistant Program Coordinator, Federal Highway Administration, Pennsylvania Division, Harrisburg, also served on the task force.

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APPENDIX--CALCULATIONS SUPPORTING REVISIONS TO EMBANKMENT WARRANTS

Concept

The concept used to develop new warrants for guide rail installation was the cost-effectiveness selection procedure detailed in AASHTO's "Guide for Selecting, Locating, and Designing Traffic Barriers," (1) published in 1977 and supplemented in 1980 (2). The 1980 supplement includes a set of design charts, which were developed by the state of Georgia and approved by the FHWA, based on this procedure. These charts served as a starting point for the task force in the development of new guide rail warrant criteria.

The AASHTO cost-effectiveness procedure is a technique that objectively compares alternative solutions at problem locations. The alternatives that are routinely considered by the designer are

1. Can the fixed object be eliminated?
2. Can the fixed object be relocated?
3. Can the fixed objects' impact severity be reduced (e.g., made breakaway)?
4. Should the object be shielded?

Obviously, one of the first three alternatives is clearly a preferred action because overall roadway safety will be enhanced. However, it is not always possible to eliminate, relocate, or convert to breakaway all fixed objects along the highway. Thus the designer must evaluate the cost-effectiveness of installing guide rail to shield the object or leaving the object unshielded. Recognizing that guide rail is itself a hazard, the cost-effectiveness procedure has been used to evaluate the costs associated with installing guide rail versus the costs of leaving a slope unprotected. The break-even point has been determined on the basis of the degree of slope and roadway volume. At fills above this break-even height, guide rail is found to be more cost-effective. Below this height, it is more cost-effective to leave the slope unprotected.

The cost-effectiveness procedure uses two principal formulas:

$$Cat = Ci\ (CRF) + Cd\ Cf + Cm + Covd\ Cf - Cs\ (SF)$$

and

$$Cad + Ci\ (CRF) + Cd\ Cf + Cm - Cs\ (SF)$$

where

Cat = total annual cost associated with the obstacle,
Cad = total annual direct cost associated with the obstacle,
Ci = initial cost of the installation of the obstacle,
Cd = average cost of damage sustained by the obstacle per accident,
Cm = average maintenance cost per year for the obstacle,
Covd = average occupant injury and vehicle damage cost per accident,
Cs = estimated salvage value of the obstacle,
CRF = capital recovery factor,
SF = sinking fund factor,
Cf = collision frequency (accidents per year) based on

$$Cf = [Ef/10{,}560]\ [(L + 62.9)\ P1 + 5.14\ P2]$$

where

Ef = encroachment frequency (encroachments per mile per year) based on AASHTO's Table 5.1.16 (1) for given ADTs,
L = horizontal length of the roadside obstacle (ft),
P1 and P2 = probability of an encroachment equaling or exceeding a given lateral displacement (A), and
A = lateral placement of the roadside obstacle from the edge of pavement.

To develop warrant criteria for guide rail on slopes, Cat was calculated for guide rail installations for the following conditions:

	A				
ADT	Rail (ft)	Slope (ft)	Ef	P1 (%)	P2 (%)
20,000	10	12	7.5	93	90
5,000	8	10	2.0	95	93
2,000	6	8	3.4	97	95
750	4	6	1.4	98	97
400	2	4	0.8	99	98

For an ADT of 20,000 vehicles per day, this means that guide rail was assumed to be placed 10 ft from the edge of pavement, and the slope is assumed to begin at a point 12 ft from the edge of pavement. As noted in a subsequent section of this Appendix, the Ef was further modified by multiplying by 1.5 to account for Pennsylvania's accident history.

Guide rail lengths of 150, 300, 500, 750, and 1,000 ft were analyzed to determine the Cat for each. The Cat for a slope of the same length was then set equal to the Cat for guide rail to, in essence, work backward to determine the maximum height of slope that would be acceptable from a cost-effectiveness standpoint. With all other factors known, this equation was solved for Covd, the average accident cost associated with the slope. This dollar value was equated with a severity index from the Covd/SI chart (Table A-1). The height of slope was then determined using the formula (3):

$$\log SI = 0.566 + 0.160 \log h + 0.324 \log s$$

where

SI = severity index determined previously,
h = height of fill, and
s = slope of fill.

Three slopes were analyzed: 1 1/2:1, 2:1, and 2 1/2:1. Values of h were determined for both a weak-post and a strong-post installation for each ADT, length, and slope condition. In each case, the most conservative height was then selected.

TABLE A-1 Modified Covd Table, Based on Pennsylvania Accident Cost Data

Severity Index	PDO Accidents (%)	Injury Accidents (%)	Fatal Accidents (%)	Total Accident Cost, Covd ($)
0	100	0	0	1,680
1	85	15	0	3,390
2	70	30	0	5,100
3	55	45	0	6,810
4	40	59	1	11,380
5	30	65	5	23,961
6	20	68	12	45,122
7	10	60	30	97,746
8	0	40	60	184,692
9	0	21	79	239,036
10	0	5	95	284,799

Note: based on fatal accident = $299,100, injury accident = $13,080, and property-damage-only (PDO) accident = $1,680.

All calculations were performed using a computer program on a microcomputer. The results are discussed in a subsequent section of this Appendix.

A similar approach was used to evaluate the cost-effectiveness of installing guide rail to protect trees and utility poles. Single poles 1 ft in diameter, single trees 1 ft in diameter, and lines of trees 150, 300, 500, 750, and 1,000 ft in length were all analyzed to determine the most cost-effective treatment. Cat values for guide rail were compared with Cat values for trees and poles, with the lowest Cat identifying the most cost-effective approach.

Modified Covd Table

The AASHTO guide includes a table that translates severity index into Covd by multiplying NHTSA's cost figures for fatal, injury, and property-damage-only (PDO) accidents by the percentage of each that is assumed to occur at each severity index. This table was modified by using accident cost data that are more current and appropriate to Pennsylvania (Table A-2).

Ef Modification

The Ef factor is used in the formula to compute the frequency with which a roadside hazard is struck and is based on the ADT of the roadway.

The Ef values shown in Figure 5.1.16 of the AASHTO guide are representative of encroachment data based on observations conducted on relatively flat medians along tangent sections of multilane facilities.

It was thought that these values were not representative of encroachments along the secondary system

TABLE A-2 AASHTO Table of Accident Costs Based on NHTSA Accident Cost Data

Severity Index	PDO Accidents (%)	Injury Accidents (%)	Fatal Accidents (%)	Total Accident Cost ($)
0	100	0	0	500
1	85	15	0	1,550
2	70	30	0	2,250
3	55	45	0	3,650
4	40	59	1	7,425
5	30	65	5	20,025
6	20	68	12	41,200
7	10	60	30	94,500
8	0	40	60	183,000
9	0	21	79	238,575
10	0	5	95	285,375

Note: it is assumed that fatal accident cost = $300,000, injury accident cost = $7,500, and PDO accident cost = $500.

in Pennsylvania and that they should be modified to represent conditions on Pennsylvania's highways.

Glennon and Wilton (4) found that accident rate is directly proportional to encroachment frequency. Therefore the accident rate on Pennsylvania's secondary roadways compared with the accident rate on multilane facilities was computed and then the AASHTO Ef values were increased in the same proportion.

Figure A-1 graphically depicts the results of the analysis and shows that a factor of 1.5 should be used at all ADT levels.

Modified Severity Index

The AASHTO guide contains a table (5.1.12) of severity indices for a multitude of fixed objects and hazardous features. The task force attempted to verify the indices for selected features by comparing the percentage of fatal, injury, and PDO accidents attributed to each with the actual Pennsylvania percentages taken from the 1982, 1983, and 1984 Statistical Summary of Accidents on All State Highways. The results of this analysis are given in Table A-3.

It can be seen that there is little correlation between the percentages assumed by AASHTO and the actual percentages that occurred in Pennsylvania from 1982 to 1984.

To better equate severity index to actual conditions and to permit its use in the equation, it was decided to use the formula developed by Glennon to compute SI for each roadside feature under consideration.

The use of Glennon's formula,

$$SI = (24F + 6I + P)/N$$

where

F = number of fatal accidents for the condition,

TABLE A-3 Severity of Accidents

	Pennsylvania (actual percentage)			AASHTO (assumed percentage)		
Feature	Fatal	Injury	PDO	Fatal	Injury	PDO
Guide rail	1.5	50.5	48.0	1	53	46
Bridge end	5.0	64.2	30.8	84	16	0
Trees	2.3	65.5	32.2	60	40	0
Utility poles	1.2	64.3	34.5	30	60	10
Piers and abutments	2.0	61.9	36.1	84	16	0
Sign supports	1.3	51.0	47.3	4	63	33
Culverts	1.8	60.5	37.3	60	40	0
Ditches	1.5	58.2	40.3	NA	NA	NA

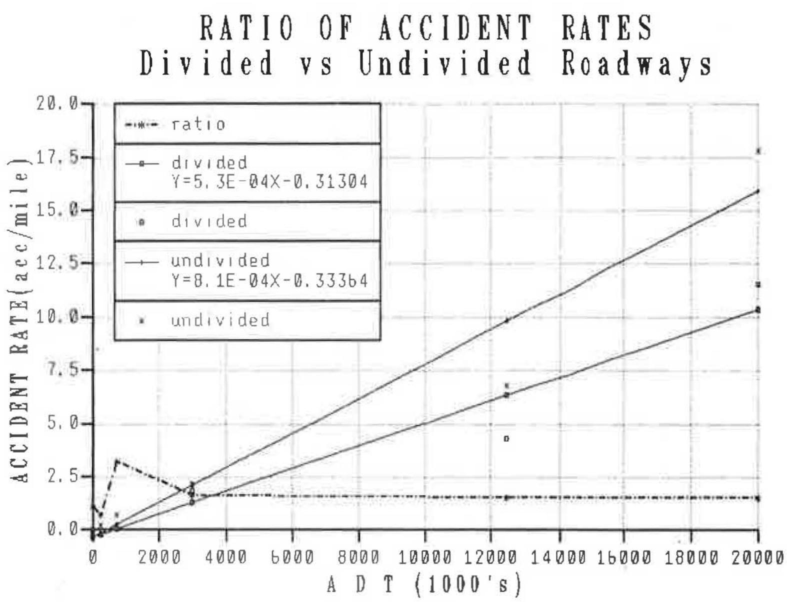

FIGURE A-1 Ef modification.

I = number of injury accidents for the condition,
P = number of PDO accidents for the condition, and
N = total number of accidents for the condition,

resulted in the SI values and their accompanying Covd values given in Table A-4 (see also Table A-1).

Examples of SI Calculations

Guide Rail

SI = (276 x 24 + 9,168 x 6 + 8,724 x 1)/18,168

SI = 3.87 (rounded to 3.9)

GR Covd = $10,923

Trees

SI = (366 x 24 + 10,348 x 6 + 5,076 x 1)/15,790

SI = 4.81 (rounded to 4.8)

Tree Covd = $21,445

Utility Poles

SI = (318 x 24 + 17,559 x 6 + 9,435 x 1)/27,312

SI = 4.48 (rounded to 4.5)

Pole Covd = $17,671

Cost Data

For all calculations, the task force used cost data that were specific to Pennsylvania. These assure that the figures derived are appropriate to the conditions experienced in the commonwealth. The following list gives the figures and the sources from which they were derived:

1. Accidents: based on figures used in Pennsylvania's Highway Safety Improvement Program
   - Fatal accidents: $299,100
   - Injury accidents: $13,080
   - PDO accidents: $1,680
2. Cost of installation: based on contract bid prices for the installation of guide rail
   - Weak-post guide rail: $10.00 per foot
   - Strong-post guide rail: $16.50 per foot

These costs include

1. Average damage cost: $400 per incident, based on assumed damage of 25 ft if strong post and 37.5 ft if weak post
2. Average maintenance cost: $1.50 per foot, based on actual costs recorded in the department's highway maintenance management system
3. Estimated salvage value: $3.00 per foot, based on contract estimates for W-beam in place ($5.00 per foot) versus W-beam installed by contract but supplied by the department ($2.00 per foot)

TABLE A-4 Modified Covd

	No. of Pennsylvania Accidents (1982-1984)					
Feature	Fatal (F)	Injury (I)	PDO (P)	Total (N)	SI = (24F + 6I + P)/N	Covd (see Table A-1) ($)
Guide rail	276	9,168	8,724	18,168	3.9	10,923
Bridge end	56	733	351	1,140	5.3	30,309
Trees	366	10,348	5,076	15,790	4.8	21,445
Utility poles	318	17,559	9,435	27,312	4.5	17,671
Piers and abutments	16	489	284	789	4.6	18,929
Sign supports	48	2,134	1,980	4,142	3.8	10,466
Culverts	39	1,345	769	2,153	4.5	17,671
Ditches	54	3,030	2,080	5,164	4.2	13,896

4. Economic factors: based on a 20-year life for the guide rail and a 6 percent interest rate (the rate assumed in developing Pennsylvania's 1984-1996 Twelve-Year Transportation Improvement Program)

- Capital recovery factor: 0.087
- Sinking fund factor: 0.027

Program Output

The microcomputer program output provides the following data:

- Cat (weak)--total cost of installing and maintaining a weak-post guide rail system.
- Cat (strong)--total cost of installing and maintaining a strong-post guide rail system.

Each Cat value was then divided by the collision frequency (Cf) factor for the appropriate ADT classification to compute the average occupant injury and vehicle damage cost (Covd) that is expected to occur for that type of installation.

To determine the SI of a slope that will incur the same average accident cost, the following steps were then followed:

1. The SI value for each Covd was determined from Table A-1.
2. The formula log SI = 0.56 + 0.160 log h + 0.324 log s was solved for each combination of ADT, length, and slope (1 1/2:1, 2:1 and 2 1/2:1) to find h.
3. The resulting heights of slopes are given in tabular form in Table A-5.

TABLE A-5 Permissible Unprotected Slope Heights (ft)

Slope	Length (ft)	ADT 20,000	5,000	2,000	750	400
1 1/2:1	150	4	7	6	9	17
	300	4	9	6	11	19
	500	5	9	6	11	21
	750	5	9	6	11	22
	1,000	5	9	6	13	22
2:1	150	8	14	10	16	31
	300	8	16	10	21	35
	500	9	16	10	27	37
	750	9	16	10	21	39
	1,000	9	16	10	24	39
2 1/2:1	150	12	22	16	25	49
	300	12	25	16	33	55
	500	14	25	16	33	58
	750	14	25	16	33	62
	1,000	14	25	16	38	62

Analysis of Data

An analysis of the values given in Table A-5 reveals the following general principles:

1. As ADT decreases, reduced accident frequency permits greater slope height.
2. As rate of slope decreases, reduced severity permits greater slope height.
3. Within the same rate of slope and ADT range, the greater the length of slope, the greater the slope height.

These findings were expected and, to a certain degree, validate the method of analysis. One exception occurs in the 5,000-ADT column. The values shown for this volume in the table appear to be unreasonable compared with the rest of the data. A further review of the assumptions and input values uncovered a discrepancy in the Ef value used in this volume range. Figure 5.1.16 in the AASHTO guide shows a break in the equation between 0 and 6,000 ADT with (in the present case) both the 2,000- and 5,000-ADT ranges severely affected.

A detailed analysis of Pennsylvania's accident data on roadways in all ADT ranges showed a more linear relationship with no apparent spike in the lower volume ranges. It was therefore decided to smooth out the peak in the AASHTO data by compressing the volume ranges and using an average value to describe the range between 750 and 5,000 ADT.

Because each rate of slope within an ADT range was represented by a range of permissible slope heights, it was decided to introduce a factor of safety by selecting the lowest height regardless of length.

Table A-6 gives the results of the compression of volume ranges and the minimum height selections. It represents the recommended warrants for the use of guide rail on embankments.

TABLE A-6 Embankment Warrant Criteria

Slope	ADT >5000 (slope height f1)	751-5,000 (slope height f+)	400-750 (slope height f+)	<400 (slope height f+)
1 1/2:1	4	6	9	17
2:1	8	10	16	31
2 1/2:1	12	16	25	49

The analysis performed for trees and poles indicates that it is most cost-effective not to place guide rail in front of individual trees and poles. The calculations further indicate that it is more cost-effective to install guide rail than to leave lines of trees unprotected for all ADTs and lengths (150 ft or longer) analyzed. However, this is contrary to current department policy. The task force considers the existing policy (to not routinely protect lines of trees) to be appropriate because each condition must be evaluated on its own merits; sound engineering judgment must be exercised in deciding whether guide rail should be used or trees should remain exposed.

References

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This paper was prepared for internal use by the Pennsylvania Department of Transportation. The contents of this paper reflect the views of the Guide Rail Task Force, which is solely responsible for the facts and data presented herein. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation, nor does this paper constitute a standard, specification, or regulation.

Events That Produce Occupant Injury in Longitudinal Barrier Accidents

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ABSTRACT

Since the early days of highway safety research the design of longitudinal traffic barriers has been greatly influenced by two basic assumptions about the mechanism of occupant injuries. First, it has been assumed that the severity of occupant injury is directly related to the intensity of vehicle collision accelerations in the first barrier collision. It has been thought that the risk of occupant injury would be decreased by developing roadside features that would prevent high values of vehicle acceleration. The second assumption has been that occupants of vehicles involved in multiple-impact accidents would be subjected to the highest risk of injury in the first collision. Because vehicle speed and kinetic energy are generally greatest in the initial collision, it has been reasoned that the most severe occupant trauma occurs during the first collision event. Recent research at Southwest Research Institute has indicated that ensuring a smooth redirection is a more effective means of improving occupant safety than trying to limit vehicle lateral accelerations. It was found that occupants are rarely injured severely in a collision with a longitudinal barrier that smoothly redirects the vehicle. In the light of these recent findings, many of the typical assumptions made in designing and evaluating highway safety hardware may not be as appropriate as was once thought. Data from sled tests, accident data analysis, and full-scale crash tests indicate that the likelihood of an occupant sustaining serious injury in a collision with a longitudinal barrier is quite low if the vehicle remains upright and is smoothly redirected.

Since the early days of highway safety research, the design of longitudinal barriers such as guardrails, bridge rails, and median barriers has been greatly influenced by two basic assumptions about the causes of occupant injuries when vehicles collide with such devices. It has been assumed that occupants are subjected to the highest risk of injury during the vehicle's initial collision with a longitudinal barrier; subsequent collisions with the same or other roadside features have been presumed to be less hazardous because of lower vehicle speeds. Second, the probability of severe occupant injury has been assumed to be directly and primarily related to the intensity of vehicle collision accelerations. It has been thought that by designing roadside hardware to limit high values of vehicle accelerations the frequency and severity of occupant injuries would be diminished.

A recent study performed at Southwest Research Institute (SwRI) and sponsored by the FHWA produced findings that indicate that these traditional assumptions may not be completely accurate. The results of this study indicated that (a) even when subjected to what have generally been considered severe impact conditions, occupants are not severely injured and (b) vehicle trajectory and stability after the initial collision are major factors in the causation of occupant injuries.

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FLAIL SPACE MODEL

Traditionally, the dynamic performance evaluation of longitudinal barrier systems was accomplished by assessing vehicle kinematic and dynamic quantities derived from carefully controlled crash tests. In addition to requiring that the vehicle be smoothly redirected and remain upright, the peak 50-msec average lateral and longitudinal accelerations were acquired and evaluated on the assumption that the severity of occupant injury in a longitudinal barrier collision was primarily a function of the vehicle's collision dynamics. Chi (1) provides an informative historical evaluation of the many pre-NCHRP Report 230 injury evaluation criteria.

NCHRP Report 230 (2) advocated the use of the flail space concept and occupant risk criteria that linked vehicle kinematics to the occupant's risk of sustaining physical injuries. The occupant risk factor is the hypothetical impact velocity of the occupant with the vehicle interior: the greater the occupant impact velocity the more severe the resulting injuries. The occupant is assumed to behave as a free missile that continues to travel along the precollision trajectory and at the precollision velocity while the vehicle responds to the collision forces. In essence, the vehicle compartment moves toward the occupant, striking the occupant at a determinable velocity. This concept allows all of the previous occupant severity indices to be unified in a single value: the occupant risk factor.

At the time NCHRP Report 230 was written, there was little evidence to establish threshold values for the occupant-to-passenger compartment impact velocity required to prevent severe injuries. Some data were available for frontal occupant impacts into the windshield from crash cushion studies. No data were available, however, for occupant lateral impacts into the door during redirectional collisions. In addition, there were no comprehensive data available to establish appropriate flail space dimensions for calculating the occupant risk factor.

To better define the flail space envelope, a survey was made of typical 1978 to 1984 vehicle interior dimensions to determine the distribution of flail space distances. The following equation, which can be used to calculate the occupant's impact velocity with the vehicle interior when the vehicle is not yawing, illustrates the importance of the flail dimension (s).

$$\upsilon = 2As^{1/2} \qquad (1)$$

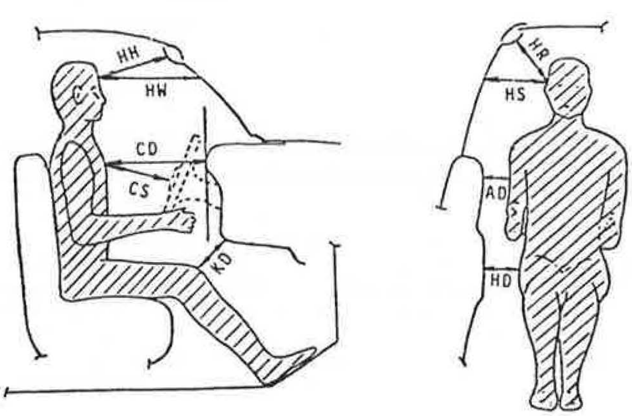
where

υ = occupant-compartment impact velocity (fps),
A = average vehicle accelerations (ft/sec²), and
s = flail distance (ft).

For relatively long collision events, such as redirectional collisions, the occupant impact velocity increases as the square root of the appropriate flail space distance given the same average acceleration. This implies that occupants in "spacious" compartments where the flail space is maximized are more at risk. Table 1 gives a summary of the results of a passenger compartment survey that was performed using data from the New Car Assessment Program (NCAP) on 1978 to 1984 passenger sedans. To create a "worst case" scenario the passenger was assumed to be small (i.e., 5th percentile female) and seated in the right front passenger position with the seat in the rear-most position. The NCHRP Report 230 (2) value of 2 ft was found to be an appropriately conservative yet realistic value for the longitudinal flail distance compared with 22 in. shown in Table 1 for the dimension HW. For the lateral flail distance, values in Table 1 range from 7 to 13 in. for the dimension HS, and a 12-in. lateral flail distance, as suggested in NCHRP Report 230, was deemed appropriate. The data in Table 1, then, indicate that the NCHRP Report 230 suggestions of 1 ft in the lateral direction and 2 ft in the longitudinal direction are, indeed, representative of flail distances in the vehicle population.

TABLE 1 Typical Passenger Compartment Clearance Dimensions

Dimension[a]	Range[b]	Median[b] Distance	75th Percentile[b] Distance
HW	15-24	20	22
CD	19-24	21	22.5
CS	10-17	13	15
HS	7-13	9	10
AD	1-7	4.5	5.5
HD	5.5-9.5	6	8
HH	11-20	14	15
HR	4-10	6	7.5
KD	3-10.5	7	8



[a]Dimensions are for a 5th percentile female seated in the driver position with the seat in its rearmost position.
[b]The dimensions are, to a small degree, functions of vehicle weight. The values reported are for 1978 to 1984 passenger automobiles with core weights greater than 3,680 lb.

ANTHROPOMETRIC DUMMY SLED TESTS

To establish a link between the flail space model and occupant protection standards used by NHTSA, a series of sled tests was conducted in which unrestrained anthropometric dummies were observed during simulated small car frontal and side impacts. Three frontal tests were performed in which the passenger compartment underwent velocity changes of 25, 35, and 45 fps at acceleration rates of 4.7, 9.8, and 16.6 g's, respectively. Four side impact tests were performed in which the passenger compartment experienced velocity changes of 20, 30, 35, and 45 fps at constant accelerations of 2.6, 9.4, 14.1, and 18.4 g's, respectively.

A 1979 Honda Civic passenger compartment body buck with standard bucket seats and glass windows was used in these seven tests. A Part 572 5th percentile female dummy instrumented according to FMVSS 208 was positioned in a normal attitude with the seat in the rearmost position for the frontal tests. A 165-lb, 50th percentile male side impact dummy (SID) was used in the side impact tests. Figures 1 and 2 show sequential photographs from the test series and illustrate typical tajectories of the occupant in frontal and side impacts. A summary of the sled test findings is given in Table 2.

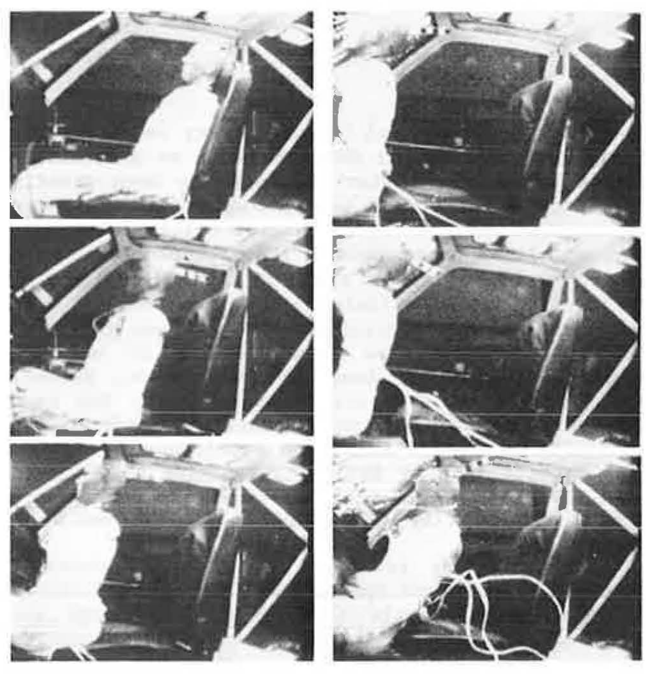


FIGURE 1 Typical frontal impact.

The findings given in Table 2 generally confirm the hypothesis that the simulated occupant behaves

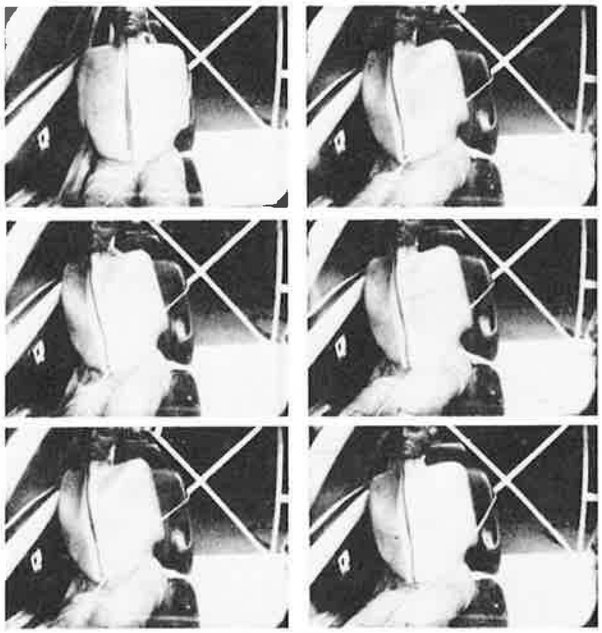


FIGURE 2 Typical side impact.

like a free missile. The occupant risk factor computed from the sled acceleration pulse using the free missile assumption compares favorably with test signals produced from the dummy accelerometers for both frontal and side impacts. The calculated occupant impact speed was reasonably close to the observed values given in Table 2 although the calculated values become more accurate as the accelerations increase and thus there is a tendency to overestimate the occupant risk factor at low accelerations.

For frontal impacts, the dummy responses tend to support the 30-fps occupant risk value suggested in NCHRP Report 230. A head injury criteria (HIC) score of 1345 occurred when the anthropometric dummy head struck the windshield at about 46 fps. By interpolating the data in Table 2 it was estimated that a head impact velocity of 40 fps would result in a HIC of 1000, the critical value established in FMVSS 208. In NCHRP Report 230 a safety factor of 1.33 was applied to the 40-fps limit to arrive at the 30-fps design limit. Chest accelerations also exceeded the FMVSS 208 60-g criterion for the 46-fps dummy impact condition. In redirectional tests, however, the longitudinal occupant risk is generally not a critical parameter because longitudinal accelerations are rarely sufficient to propel the occupant to the instrument panel. For this reason, the remainder of this paper will be primarily concerned with the lateral occupant risk factor.

For the side impact sled tests, the anthropometric dummy responses were surprisingly low. In NCHRP Report 153 (3) lateral vehicle accelerations of 5 g's were considered high. For Test 2540 in Table 2, the sled was accelerated laterally at 18.6 g's, and the resulting HIC was a mild 316, well below the FMVSS 208 threshold of 1000. The maximum occupant risk factor was calculated to be about 25 fps, which exceeds the design limit of 20 fps suggested in NCHRP Report 230. It should be noted that the actual lateral flail distance of 6.5 in. rather than the 12-in. value suggested in NCHRP Report 230 was used

TABLE 2 Sled Test Results

Left Side Impacts				
Test No.	2534	2533	2535	2540
Sled response[a,b]				
Change in velocity (fps)	20	30	35	40
Acceleration (g's)	-3.6	-8.0	-15.0	-18.4
Occupant risk data				
Time to head impact (sec)[c]	0.092	0.049	0.048	0.042
Average sled acceleration (g's)	-2.6	-9.4	-14.1	-18.4
Measured occupant impact velocity (fps)	7.7	14.8	21.8	24.9
Calculated occupant impact velocity (fps)	9.5	18.1	22.2	25.3
Head injury criteria data				
HIC	37	121	193	316
HIC duration (sec)	0.012	0.006	0.010	0.006
Head severity index	52	163	221	569
Thoracic trauma index data				
Sping g's–T12y	12.5	36.4	32.1	65.2
Upper rib g's–LUR_y	10.7	30.4	47.7	46.7
Assumed age (yr)	41	41	41	41
Weight (lb)	165	165	165	165
TTI	69	91	97	113
Probability of AIS ≥ 3 (%)	0	3	6	16

Frontal Impacts			
Test No.	2538	2537	2539
Sled response[a,b]			
Change in velocity (fps)	25	35	45
Acceleration (g's)	-5.6	-10.9	-16.8
Occupant risk data			
Time to head impact[c]	0.140	0.105	0.085
Average sled acceleration (g's)	-4.7	-9.8	-16.6
Measured occupant impact velocity (fps)	21.1	33.2	45.6
Calculated occupant impact velocity (fps)	23.5	34.4	45.8
Head injury criteria data			
HIC	87	468	1345
HIC duration (sec)	0.061	0.030	0.014
Head severity index	30	55	94
Peak chest acceleration (g's)	29.7	55.0	94.4

[a] Buck was a 1979 Honda Civic passenger compartment.
[b] Side impact dummy was used in side impacts and Part 572 5th percentile female in frontal collisions.
[c] Flail distances were measured as 22.5 in. longitudinal and 6.5 in. lateral and used in velocity calculations.

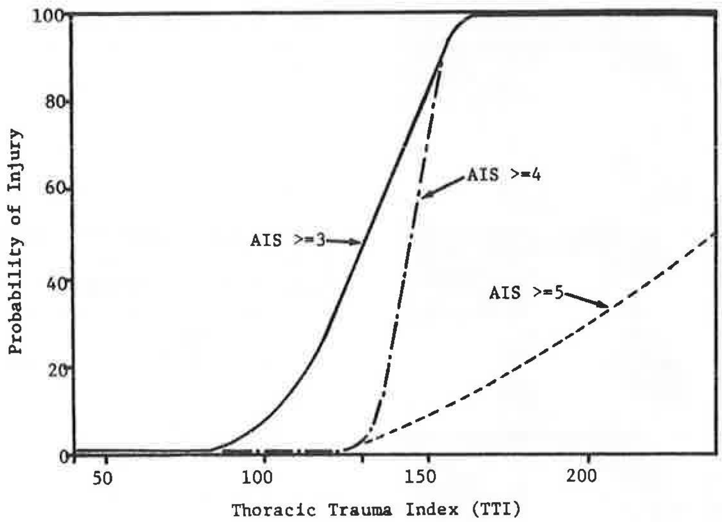


FIGURE 3 Probability of injury versus TTI.

in determining the occupant impact velocities given in Table 2.

Another injury measure better suited to side impacts is the thoracic trauma index (TTI). Eppinger et al. (4) developed a family of curves that relate the TTI to the probability of sustaining a given level of injury. This relationship is shown in Figure 3 and the TTI is defined by the following equation:

$$TTI = 1.4 \text{ Age} + 0.5 \, (LUR_y + T12_y) \, [\text{Weight (165 lb)}] \quad (2)$$

where

Age = occupant age (years),
LUR_y = left upper rib y acceleration (g's), and
$T12_y$ = Spinal y acceleration (g's).

Even at a vehicle lateral acceleration of 18.6 g's and an occupant impact velocity of 24.9 fps, the probability of a hypothetical 41-year-old, 165-lb occupant (TTI = 113) sustaining an AIS of 3 or greater is only 0.16, and the probability of sustaining an AIS of 4 or greater is nil as shown in Figure 3. The probability of severe injury (AIS $\geq$ 4) is quite remote for this occupant even under impact conditions that are generally considered to be severe.

The sled tests illustrated two important points. First, simulated occupants do behave like free missiles during collisions and their impact velocities can be calculated if the compartment geometry and the vehicle accelerations are known. Second, the lateral occupant risk design limit of 20 fps suggested in NCHRP Report 230 as well as vehicle acceleration values contained in NCHRP Report 153 may be unnecessarily conservative.

ACCIDENT ANALYSIS OF OCCUPANT INJURIES

To further investigate this apparent noncriticality of the lateral occupant risk factor during redirectional crash tests, a number of longitudinal barrier accident cases were examined in detail. Because 5 percent (5) of all fatal accidents, as the data in Table 3 indicate, can be attributed to an impact with a longitudinal barrier, the conclusions of the last section might reasonably be questioned.

TABLE 3 Distribution of Most Harmful Events Where the First Object Struck Is a Longitudinal Barrier *(5)*

	Longitudinal Barrier Is First Harmful Event	
Most Harmful Event	No.	Percentage
Overturn, noncollision	453	31
Other noncollision	116	8
Non-fixed objects	115	8
Longitudinal barrier[a]	442	31
Pier/abutment/parapet end	73	5
Other fixed objects	242	17
Total longitudinal barrier[b]	1,441	100
Total fatal accidents	27,516	

[a]Most harmful event may not necessarily be the first harmful event. It may include subsequent impact with same bridge rail or a bridge rail across the highway.
[b]Longitudinal barrier accidents represent 5.2 percent of the 27,516 fatal accidents.

To examine the importance of the lateral occupant impact velocity in real highway accidents, it was necessary to isolate those accident cases in which the lateral occupant impact velocity was the principal injury-producing mechanism. All cases in which some other aspect of vehicle dynamics or barrier performance could have caused the occupant injuries were screened from the data base leaving only those cases in which the

- Barrier was the first item struck by a passenger sedan;
- Vehicle was tracking before the first impact (i.e., heading angle and velocity vector were within 10 degrees);
- Vehicle was smoothly redirected after the first impact; there were no signs of vaulting, penetration, or severe post-wheel snagging in the first impact;
- First impact was not with a bridge pier, barrier terminal, or end treatment; and
- Vehicle did not roll over as a result of the first impact.

Using these criteria, 26 accident cases were selected from the narrow bridge study data base of 124 bridge-related accidents (6). Of the 124 narrow

TABLE 4 Characteristics of 81 Bridge Rail Accidents[a]

	No Second Impact	Result of Second Impact: Redirected or Spun Out to Rest	Redirected into Another Roadside Feature	Rollover	Vault/ Override	Total
Vehicle tracking at first impact						
Redirected or skidded to stop	7	9	8		2	26
Snagged	4	2	1			7
Penetrated			1			1
Vault/override	2	2	3	1		8
Rollover	2					2
Total	14	13	13	1	2	44
Vehicle not tracking at impact						
Redirected or skidded to stop	7	13				20
Snagged	3	5	1	1		10
Rollover	3					3
Vault/override	1	2	1			4
Total	15	20	2	1		37
Total	29	33	15	2	2	81

[a]Data from Calcote and Mak *(6)*.

bridge cases, 43 were eliminated because they involved a first collision with an end treatment or guardrail-bridge rail transition. Table 4 gives characteristics of the remaining 81 narrow bridge accidents that occurred along the midspan of the barrier system. The vehicle was not tracking in 46 percent of the cases, and, of the cases in which the vehicle was tracking, only about half met the performance criteria listed previously. Occupants suffered serious to critical injury in only 3 of the 26 eligible cases.

To supplement this small sample size, the Longitudinal Barrier Special Studies (LBSS) data base from the National Accident Sampling System (NASS) for the years 1982 and 1983 was surveyed and 139 cases out of a total of 555 were deemed eligible. The total number of eligible cases was therefore 165.

One of the most basic and widely used measures of occupant injury is the Abbreviated Injury Scale (AIS) (7):

AIS	Injury	
1	Minor	Non-life threatening
2	Moderate	
3	Serious	
4	Severe	Life threatening
5	Critical	
6	Unsurvivable	

Each individual injury is assigned an AIS score by the accident investigator. For example, minor cuts and scratches on the face may be scored as an AIS of 1 and a broken rib may be reported as an AIS of 3. A frequently used measure of the severity of all occupant trauma is the maximum AIS (MAIS). The MAIS is the highest AIS experienced by the occupant. Thus the MAIS of the occupant with facial cuts (AIS = 1) and broken ribs (AIS = 3) would be 3.

Injuries of AIS 4 or above are defined as life threatening. The intent of NCHRP Report 230 was to select an occupant risk design limit such that occupants would not sustain an injury of AIS 4 or greater.

Table 5 gives the distribution of the MAIS in each of the three data sources. Nearly 90 percent of the eligible cases in Table 5 (134 minor cases and 14 serious cases) exhibit injuries that are below the design injury limit of AIS 4. Only 2 percent of the eligible cases exhibit severe injury. It appears that the majority of vehicle occupants escapes severe injury when the vehicle is smoothly redirected and remains upright after a longitudinal barrier collision. Unfortunately the severity of occupant injury is unknown in almost 9 percent of the eligible cases (eight AIS-7 and six AIS-9 cases). There are two ways in which an NASS investigator can code an unknown injury. An AIS of 9 is used when the occupant cannot be located or departed the accident scene before any officials arrived. Generally an AIS of 9 indicates no injury or only minor injury because the occupant was capable of leaving the scene.

An AIS of 7 indicates that there was an injury but its severity is unknown. Unlike the AIS of 9, an AIS of 7 is often used by NASS investigators when

TABLE 5 Distribution of Injury in Three Data Bases

Source	Total Cases in Data Base	Eligible Cases[a] — Known Injury Severity: Minor, $0 \leq MAIS < 2$ No.	Percentage	Serious, $2 \leq MAIS < 4$ No.	Percentage	Severe, $4 \leq MAIS < 7$ No.	Percentage	Unknown Severity: MAIS = 7 No.	Percentage	MAIS = 9 No.	Percentage
1982 NASS LBSS	292	61	20.9	6	2.1	1	0.3	4	1.4	1	0.3
1983 NASS LBSS	263	50	19.0	7	2.7	0	0.0	4	1.5	5	1.9
Narrow bridge	124	23	18.5	1	0.8	2	1.6	0	0.0	0	0.0
Three data bases, combined	679	134	19.7	14	2.1	3		8	1.2	6	2.3

[a]Eligible cases are those in which (a) longitudinal barrier was struck by a passenger automobile; (b) vehicle was tracking before impact (i.e., heading angle and velocity vector are within 10 degrees); (c) vehicle was smoothly redirected after first impact; no vaulting, rollover, severe snagging or penetration; and (d) first impact was not with an end treatment or transition.

TABLE 6 Relationship Between an AIS of 7 and Police-Reported Accident Severity

Police-Reported Injury Severity	Probability of[a] AIS ≥ 2	AIS ≥ 4	No. of Eligible[b] AIS-7 Cases	Probable No. Above AIS 2	Probable No. Above AIS 4
O–none	0.0050	0.0001	0	0.0	0.0
C–possible	0.0927	0.0016	1	0.0927	0.0016
B–nonincapacitating	0.1592	0.0057	3	0.4776	0.0171
A–incapacitating	0.4181	0.0438	3	1.2543	0.1314
K–fatality	0.6104	0.4416	0	0.0	0.0
U–unknown	0.2210	0.0166	1	0.2210	0.0166
Total			8	2.0456	0.1667
Figure used				2	0

[a]From 1984 NASS CSS data.
[b]These are the eight cases with AIS 7 from Table 5.

severe injury occurs but supporting documentation such as autopsy or hospital records cannot be obtained.

The NASS Continuous Sampling System (CSS) data for 1984 were used to calculate the probability of an AIS of 7 being coded when severe injury occurred. As the data in Table 6 indicate, the probability that any of the eight cases coded as AIS 7 included injuries greater than or equal to an AIS of 4 is quite low. All eight AIS-7 cases and all six AIS-9 cases can therefore be grouped with those below the AIS of 4 guideline. Thus the eight AIS-7 cases and the six AIS-9 cases can be grouped with the 134 minor injury cases and the 14 serious injury cases to show that 98 percent of the eligible cases indicate an acceptable level of occupant injury. Severe injuries were noted in only 2 percent of the eligible cases.

The 17 serious and severe injury cases in Table 5 were reconstructed in detail to determine exactly what feature of the accident caused these injuries. Each of the 17 cases studied with serious to unsurvivable injuries would have passed the two provisions of the NCHRP Report 230 criteria that require the vehicle (a) to be smoothly redirected and (b) to remain upright. With only three exceptions, all of the cases in Table 7 involved a subsequent collision with the same or another roadside feature. The reconstruction process therefore involved determining the speed and angle for two or three collisions. The vehicle deformation energy was calculated using the damage analysis portion of CRASH3 (8), barrier deformation energy was estimated using BARRIER VII (9), and energy dissipated by tire-ground friction along the trajectory was estimated by hand analysis methods. By proceeding from the last event to the first and summing all of the energies of vehicle and barrier deformation with the energies lost through tire-ground friction and braking, a reliable estimate of the impact speed can be produced.

Occupant injuries were assigned to particular impact events with, in most cases, a high degree of certainty. When there was uncertainty the injury was assigned to all phases equally. Figure 4 shows a typical diagram of vehicle trajectory, occupant injuries, and vehicle interior. Using these pieces of information, it is possible to match injuries with the events that caused them. For example, the dislocation of the occupant's left shoulder shown in Figure 4 can be assigned to the first collision. This is confirmed by the damage to the driver's side door shown in the interior sketch and the vehicle's position shown in the trajectory sketch. The lacerations on the right side of the head can be assigned, on the basis of the occupant contact points in the interior sketch, to the second collision. Because it is difficult to determine which phase of the accident caused the concussion it was attributed equally to both impacts. The occupant risk factor can be calculated from the impact conditions using a method

TABLE 7 Summary of Cases with Serious to Unsurvivable Injuries

Data Base	Case No.	Role[a]	Vehicle Weight (lb)	No. of Impacts	First Impact Speed (mph)	Angle (degrees)	MAIS	Occupant Risk (fps)	Second Impact Speed (mph)	Angle (degrees)	MAIS	Object Struck
NASS	83-53-010T	PUI	3,365	2	90	26	2	37	66	38	2	Bridge rail
NASS	82-81-078V	DUI	3,397	3	70	2	1	6	67	16	2	Guardrail
NASS	82-75-507V	DUI	1,813	2	46	15	0	15	?	90	4	Bridge pillar
NASS	83-32-532V	DUN	2,546	2	56	3	0	7	37	90	2	Utility pole
NASS	83-53-010T	DUN	3,365	2	90	26	1	37	66	38	2	Bridge rail
NASS	83-39-131V	DUN	3,161	2	69	5	0	12	37	45	3	Median barrier
NASS	82-52-083T	DRI	3,444	1	31	34	3	46				None
NASS	82-35-125V	DRI	3,541	2	?	2	0	?	?	9	2	Guardrail
NASS	82-78-511T	DRI	4,535	1	57	35	2	48				None
NASS	83-02-071T	DRI	2,338	2	49	17	0	16	28	90	3	Tree
NASS	82-55-293V	DRN	3,041	2	46	7	0	8	38	72	2	Ditch
NASS	82-06-513Z	DRN	3,981	1	34	10	3	8				None
NASS	83-30-516T	DRN	3,062	2	71	3	0	10	49	17	3	Median barrier
NASS	83-77-517T	DRN	2,811	2	23	2	1	2	9	45	2	Median barrier
NASS	83-02-523W	DRN	4,208	2	64	10	0	12	59	29	2	Median barrier
NBS	80-03-04-068	DUN	3,977	2	61	8	1	5	52	19	3	Bridge rail
NBS	80-03-22-071	DUN	3,980	4	48	10	0	12	33	90	5	Bridge pillar
NBS	79-12-03-049	DRN	4,318	3	52	8	0	4	20	90	4	Wingwall

[a]P = passenger, U = unrestrained, I = impact side, D = driver, R = restrained, and N = nonimpact side.

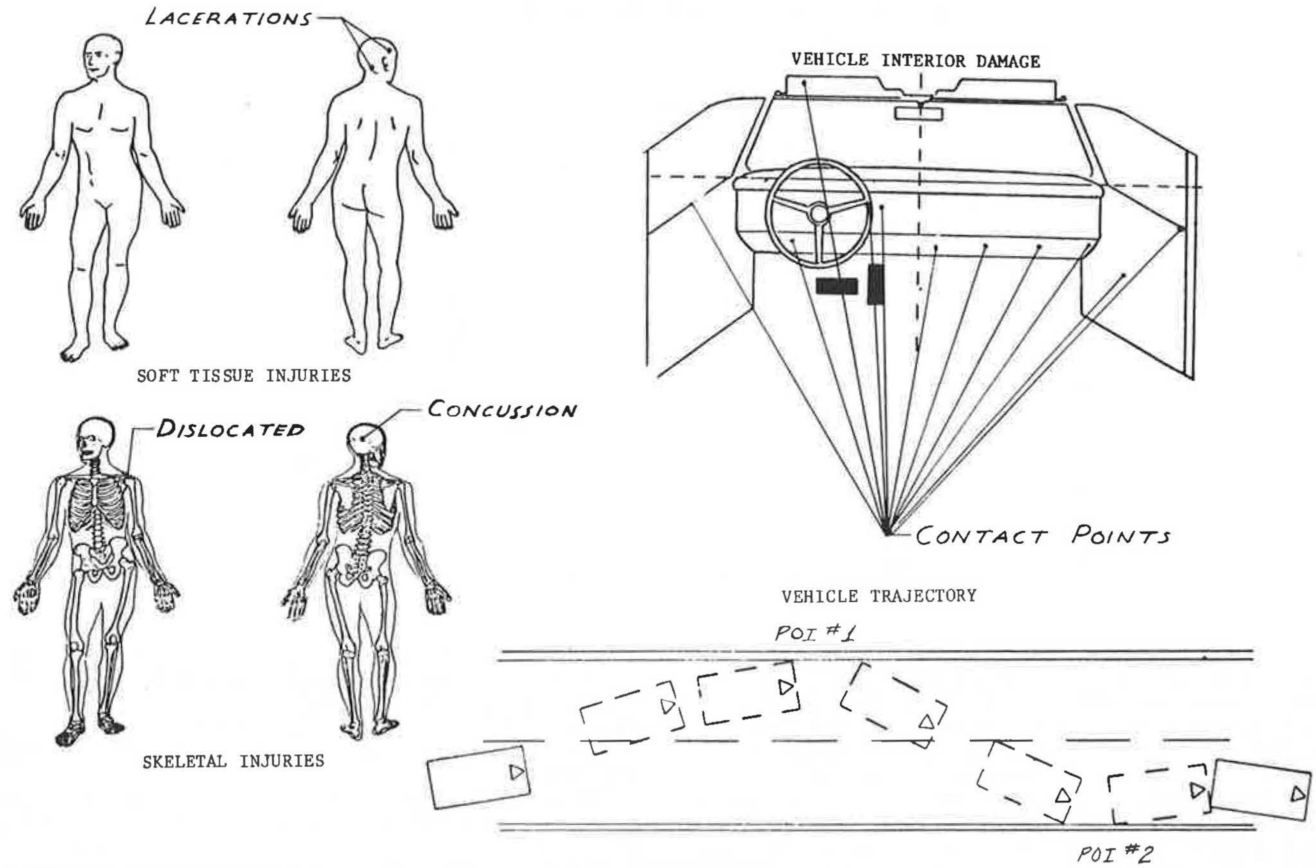


FIGURE 4 Typical accident reconstruction summary sheet, Case NASS 82-02-078V.

developed in another phase of this project and then compared with the actual level of injury experienced in each phase of the accident.

A summary of the 17 cases studied in detail is given in Table 7. When the MAIS for each of the multiple impacts is examined, it becomes apparent that none of the occupants suffered severe injury in the first impact. Recalling that these 17 cases are those 2 percent of eligible cases in which severe or serious injury occurred, it appears that the first impact in all 165 eligible cases in Table 5 resulted in injuries less than the design limit of AIS 4. Indeed, 96 percent of all eligible cases resulted in only minor injuries: 134 minor injury cases from Table 5, 13 of the 17 cases summarized in Table 7, 6 of the 8 AIS-7 accident cases, and all AIS-9 cases.

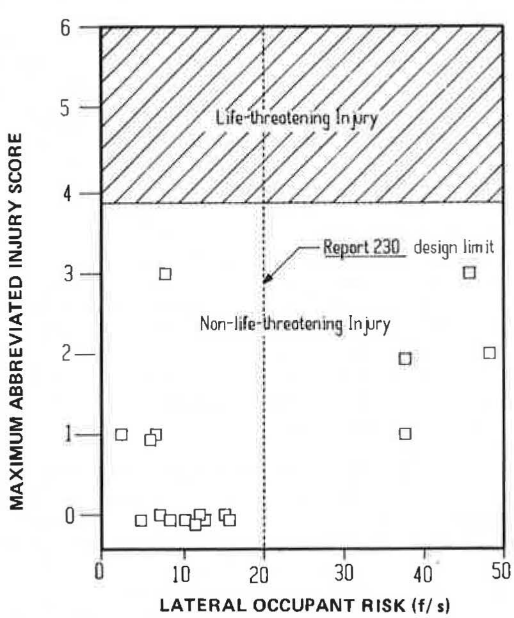


FIGURE 5 Occupant injury versus lateral occupant risk factor.

The original intent of this research was to discover some relationship between the occupant risk factor and the actual level of injury sustained in real highway accidents. The data proved to be surprising. Figure 5 shows a plot of the occupant risk factor versus the MAIS for the first impact of each of the serious and severe injury cases in Table 7. None of the 17 accident cases resulted in a life-threatening injury after the first impact. Figure 5 illustrates the apparent relationship between the occupant risk factor and the MAIS. Injuries greater than or equal to an AIS of 4 do not appear likely until the occupant risk factor is in excess of 40 fps, twice the design limit suggested in NCHRP Report 230.

TYPICAL VALUES IN FULL-SCALE CRASH TESTS

The sled test data indicated that serious injuries were not likely to occur under what have generally been considered to be severe impact conditions. How useful, then, is the occupant risk factor for evaluating longitudinal barriers?

Since NCHRP Report 230 was published in 1981, nearly 300 full-scale crash tests have been performed at SwRI. Rarely has a test device been disqualified

TABLE 8 Occupant Risk Values for 15 Bridge Rail Crash Tests (10)

Test No.	Impact Conditions[a] Speed (mph)	Angle (degrees)	Occupant Risk Frontal (fps)	Side[b] (fps)	50-msec Avg[b] Vehicle Acceleration Front (g's)	Side (g's)	Comments
NBR-1	60.7	19.3	7.2	21.8	-5.8	12.6	Smooth redirection
NBR-2	61.4	24.9	3.0	21.6	-6.3	8.4	Smooth redirection
NBBR-1	61.4	20.0	5.3	20.7	-4.9	13.5	Smooth redirection
NBBR-2	58.4	24.3	–[c]	9.9	-5.9	8.2	Smooth redirection
NCBR-1	59.7	18.8	14.0	22.7	-8.1	12.9	Smooth redirection
NCBR-2	60.0	25.0	16.6	31.2	-6.9	17.9	Smooth redirection
OBR-1	58.6	18.8	0.8	19.9	-3.3	10.2	Smooth redirection
OBR-2	60.8	24.3	20.9	26.0	-5.2	13.7	Snagged hood
OBR-3	60.0	25.0	0.9	28.2	-5.2	15.9	Smooth redirection
KBR-1	61.9	20.3	11.5	20.4	-7.5	11.2	Smooth redirection
KBR-2	60.5	24.0	30.0	23.3	-8.3	13.4	Severe snagging
OHBR-1	60.6	19.6	7.3	20.6	-5.6	11.4	Smooth redirection
OHBR-2	60.0	25.0	7.0	25.1	-6.1	12.1	Smooth redirection
LABR-1	60.4	18.8	–[c]	23.6	-4.4	12.8	Smooth redirection
LABR-2	59.7	19.1	14.0	22.8	-5.3	10.8	Smooth redirection

[a]20-degree tests utilized Honda Civics and 25-degree tests utilized Plymouth Furys.
[b]From transducer data.
[c]Hypothetical occupant did not displace the required 24 in.

because of the occupant risk criteria alone. Table 8 gives a brief summary of the impact conditions, occupant risk measurements, and 50-msec average accelerations from a research project (10) that involved a number of crash tests of operational bridge rails. Bridge rails are generally rigid barrier systems and therefore provide minimal energy dissipation during collisions; the highest values of the occupant risk factor should be observed during bridge rail tests. The data in Table 8 indicate that even in rigid barrier collisions the occupant risk factors are generally in the same range that was shown to be noncritical for the sled tests in Table 2. The probability of an occupant sustaining injuries of AIS 4 or greater is remote for these 15 typical rigid barrier installations.

Clearly there are two problems with using the occupant risk criteria for evaluating longitudinal barrier crash tests. First, as the sled test and accident data imply, serious injury does not appear likely at the current NCHRP Report 230 design limit of 20 fps or even at a more liberal value of 30 fps. The accident data imply that severe occupant injury is not likely until occupant lateral impact velocities of at least 40 fps occur. Second, the occupant risk is nearly always below 30 fps even in rigid barrier tests. Hence, although the flail space concept is both accurate and simple to use, it does not provide a measure that is meaningful in assessing longitudinal barrier crash tests.

DISCUSSION

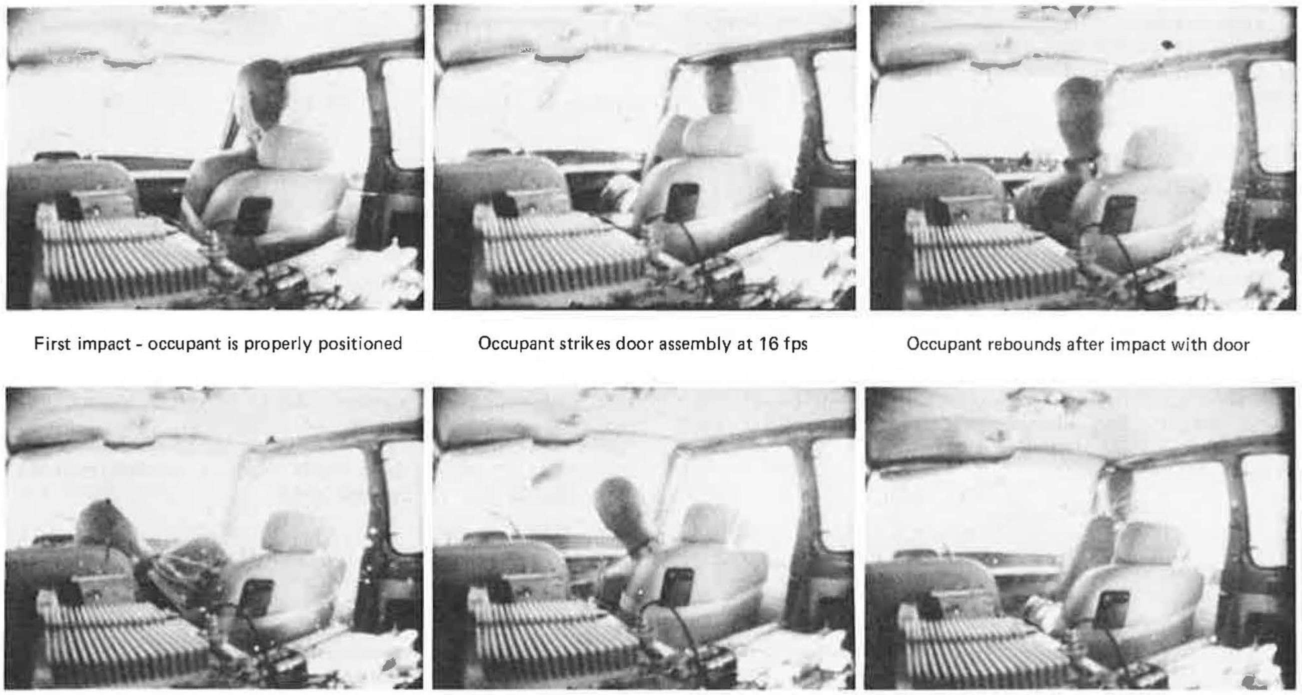
How then are occupants being injured and killed in the nearly 1,500 fatal longitudinal barrier accidents that occur each year (5)? Some clues may have been suggested earlier in this paper.

In more than 80 percent of the cases summarized in Table 7, the vehicle struck another roadside object after being successfully redirected from the first collision. For all of the vehicle occupants that experienced secondary impacts, the MAIS was greater in the second impact than the first, sometimes by a large margin. For example, after the first barrier impact in NASS Case 83-02-071T the occupant had sustained no injuries. After the vehicle was redirected, however, it collided with a tree; the MAIS for the second collision was 3. Often, in the cases summarized in Table 6, the occupant sustained no injuries during the first redirection only to become involved in another, much more serious, subsequent collision. Clearly, redirection into other roadside features poses a serious hazard to vehicle occupants.

There are several possible reasons for this increase in injury rate for occupants of vehicles that are redirected from a longitudinal barrier and subsequently strike other roadside features. Although the impact speed is nearly always less in second collisions, the angle frequently increases. In Table 7, the second impact angle was larger than the first in all of the multiple-impact cases. Frontal impacts may be more injurious than side impacts because of the greater amount of flail space in which the occupant may accelerate as discussed earlier in this paper. Therefore, as the impact angle becomes larger, the impact will become more frontal. Because occupants have larger flail distances available in frontal collisions they may be at greater risk of sustaining injury.

Another important feature of the secondary collision is the occupant's position in the passenger compartment. At the time of the initial collision the occupant is usually positioned correctly in the seat. During the first redirectional collision the occupant will strike the door surface and rebound beyond his preimpact position. Thus, if a second collision occurs, a larger flail space is available in which to accelerate to a higher velocity. Figure 6 shows a set of sequential photographs of an anthropometric dummy taken during a longitudinal crash test in which the vehicle unintentionally struck two barriers. The dummy struck the door in the first collision, rebounded beyond its original seating position, and then struck the door again at a higher velocity during the second collision. The dummy's flail distance was more than two times greater in the second collision.

Although considerable attention and effort have been devoted to defining and measuring vehicle accelerations during longitudinal barrier crash tests, little effort has been directed to affecting the after-collision trajectory of the vehicle. This lack of attention to the postimpact trajectory can be attributed to both the unrecognized importance of this phase of the test by the technical community and the unpredictability and frequently erratic be-



First impact - occupant is properly positioned

Occupant strikes door assembly at 16 fps

Occupant rebounds after impact with door

Second impact - occupant out of position at second impact

Occupant accelerates toward door

Occupant strikes door assembly at 35 fps

FIGURE 6 Effect of occupant position on occupant risk factor.

havior of the vehicle caused by wheel and frame damage as well as imprecise braking controls. Even with improved braking controls, the authors are not confident that the after-collision vehicle trajectory would be a good crash test assessment criterion.

On the other hand, by changing the emphasis of barrier design from reducing vehicle accelerations during a collision to effecting more predictable vehicle trajectories, longitudinal barrier developers may be able to improve the vehicle's postimpact trajectory and greatly increase the safety of the vehicle occupants.

Although a number of possible reasons have been suggested for the causes of occupant injuries after the initial collision with a longitudinal barrier, the data are not sufficient to suggest the magnitude of the redirection problem. Cumulatively, however, postimpact vehicle trajectory and stability appear to be crucial to providing protection to vehicle occupants.

The ultimate objective of longitudinal barrier designers is to protect occupants by shielding vehicles from more hazardous roadside objects and to shield pedestrians from traffic. It is often a difficult task to determine what specific aspects of a design will work best toward these goals. For many years longitudinal barrier designers have attempted to find a balance between the often conflicting goals of barrier flexibility for vehicle occupant protection and barrier strength for vehicle containment.

The discussions in previous sections have suggested that these goals need not conflict. A longitudinal barrier system that performs correctly, smoothly redirecting the vehicle without serious snagging, vaulting, penetration, or rollover, will not subject the occupant to lateral collision forces of a magnitude great enough to cause severe injury. Thus, if designers ensure that longitudinal barriers perform "correctly," vehicle occupants will generally be well protected in redirectional collisions.

Although the foregoing discussion indicates that the occupant risk factor may not be the critical evaluation factor in longitudinal barrier tests, the authors recommend that these measurements continue to be taken especially because they are easily calculated from vehicle dynamics. Moreover, the vehicle kinematics and occupant risk determinations are critical for other roadside hardware evaluation tests such as those of crash cushions and breakaway supports.

CONCLUSIONS

There are two principal conclusions to this study. First, when a tracking vehicle strikes a longitudinal barrier and is smoothly redirected and remains upright, the risk of severe occupant injury in that collision is quite small. Although the flail space model and the occupant risk criteria are useful and simple tools for estimating the behavior of occupants in a collision environment, they do not appear to be a discerning assessment factor for redirectional tests. In the absence of snagging, barrier penetration, or rollover, it is not likely that high values of occupant-interior impact velocity will be observed. Because NCHRP Report 230 already requires smooth redirection and an upright vehicle, the occupant risk factor is a redundant evaluation criteria.

Second, the postimpact trajectory of the vehicle, though difficult to predict or control, is an important feature of barrier performance and should be more carefully considered in future longitudinal barrier development and testing. Although it is doubtful that postimpact trajectory can be explicitly used as a test evaluation criteria, it is a feature of motor vehicle collisions that should receive more attention from the highway safety community. The authors are confident that this aspect of vehicle and appurtenance interaction can be used to develop even more creative and innovative methods of providing an even higher level of safety on our nation's highways.

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Discussion

John G. Viner*

The authors' first conclusion states that, in tracking vehicle impacts with longitudinal barriers in which the outcome is a smooth redirection with no overturn, "the risk of severe occupant injury in that collision in quite small," and high values of occupant-interior impact velocity are "not likely." Thus the authors state that the occupant risk factor is a redundant evaluation criterion in redirection crash tests conducted according to NCHRP Report 230 procedures. The relevant data in this paper (four side-on dummy tests and 165 accidents) do not support this conclusion. When viewed from the broader perspective of other data from NASS, a somewhat inverse hypothesis may be supportable.

DUMMY DATA

The calibration procedures recommended by the NHTSA were not followed in the dummy tests. The data in Table 2 indicate that measured values of spine and upper rib acceleration (used in calculating TTI, which in turn is used to estimate the probability of AIS $\geq$ 3) do not consistently increase with increasing test ΔV. The values of TTI for the 40-fps test are in the area of the AIS $\geq$ 3 versus TTI curve (Figure 3), where small changes in TTI produce relatively large changes in this estimate. The 20- and 30-fps tests are close to this region of the curve. These side-on tests were made with a flail space distance of 6.5 in.; yet, as noted by the authors, the measured flail space values from the 1978-1984 NCAP tests ranged from 7 to 13 in. and the NCHRP Report 230 procedure uses 12 in.

The apparent inconsistency in the dummy data suggests that the failure to follow the recommended calibration procedures has affected the validity of the data. If a 12-in. flail space had been used, as recommended by NCHRP Report 230 (and the authors), the dummy accelerations would have been larger. Because the estimate of probability of injury (AIS $\geq$ 3) is quite sensitive to increases in dummy accelerations, a repeat of these tests using a 12-in. flail space is likely to result in significantly larger estimates of injury probability.

ACCIDENT DATA

The authors' conclusion that the risk of severe injury (AIS $\geq$ 4) is "quite small" in tracking vehicle impacts with longitudinal barriers, if the vehicle is smoothly redirected without overturning, is

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debatable. The 2 percent (3 of 165) of the eligible longitudinal barrier cases with AIS $\geq$ 4 is comparable to the national estimate from NASS 1984 data that 1.5 percent of all run-off-road accidents (trees, rollovers, ditches, guardrails, etc.) result in AIS $\geq$ 4 injuries. The NASS estimate is calculated from 4,911 investigated accidents in which the first harmful event occurred outside the shoulder of the road.

A similar comparison can be made with the AIS $\geq$ 2 cases in this paper. The 17 AIS $\geq$ 2 cases represent 11 percent of the eligible accidents. From the 1984 NASS data on accidents in which the first harmful event occurred outside the shoulder, 12 percent had AIS $\geq$ 2. Thus, at both the AIS $\geq$ 2 and the AIS $\geq$ 4 levels, the eligible longitudinal barrier cases (tracking vehicles that strike in the length of need and that are redirected by the barrier and remain upright) were found to be comparable with roadside accidents in general.

End impacts, overturns, penetrations, and vaulting accidents with longitudinal barriers, which were excluded from the eligible longitudinal barrier accident cases in this study, are more severe than impacts that result in smooth redirection. This suggests that tracking vehicle impacts with longitudinal barriers are more likely to result in AIS $\geq$ 2 and AIS $\geq$ 4 injuries than are run-off-road accidents in general.

Although the intent of longitudinal barriers is to protect the traveling public from the more serious roadside hazards, the finding in this study that under favorable conditions (no end impacts, rollovers, etc.) the severity of injury to occupants of tracking vehicles in longitudinal barrier impacts was the same as that of roadside accidents in general deserves further attention. The authors' observation that the 300 tests examined by NCHRP Report 230 criteria have rarely resulted in occupant risk criteria alone disqualifying a device thus suggests further study to see if lowering or revising the occupant risk criteria should be considered.

LATERAL OCCUPANT RISK DESIGN LIMIT

The measure of effectiveness used by the authors for the accident analysis, likelihood of AIS $\geq$ 4, was selected because "The intent of NCHRP Report 230 was to select an occupant risk design limit such that occupants would not sustain an injury of AIS 4 or greater." This is not the case. As stated on page 30 of NCHRP Report 230, "Accident statistics from France (22) indicate that injuries of AIS 3 or greater were sustained in 50 percent of side impact cases for a ΔV of at least 30 fps (9.4 m/s). Where the compartment space is not intruded, an upper lateral occupant impact velocity of 30 fps (9.1 m/s) appears to be a reasonable limit. . . ." NCHRP Report 230 recommends that a factor of safety of 1.5 be used with this limit value giving a 20-fps design limit to lateral ΔV in the appropriate crash tests.

This interpretation of AIS 4 as a design limit rather than a 50 percent chance of AIS $\geq$ 3 makes a difference because the AIS scale is not a linear scale of injury outcome. For example, from the 1984 NASS estimates of accidents with first harmful events outside the shoulder, accidents with a maximum AIS of 3 result in fatalities in 5.4 percent of the cases whereas, with an AIS of 4, fatalities result in 15.6 percent of these cases. For comparison, 1.2 percent of all such accidents are estimated to result in fatalities.

In Figure 5, the authors compare the calculated ΔV from the 17 reconstructed accidents (in which AIS $\geq$ 2) with the actual injuries sustained in the first impact in these cases. The authors state that the original purpose of this research was to discover some relationship between lateral ΔV and likelihood of injury. Looking at the data from the point of view of the quote from NCHRP Report 230 (see Figure 5), only four cases had a calculated ΔV of at least 30 fps. One case was AIS 3 and two were AIS 2. This is consistent with the 50 percent chance of AIS $\geq$ 3 in this selected limit.

CONCLUSIONS

1. The dummy data used to support the authors' first conclusion are questionable because (a) recommended calibration procedures were not followed, (b) the data showed apparent contradictions, (c) the flail space used in these tests was less than either that found from vehicle measurements or NCHRP Report 230 recommendations, and (d) the calculated likelihood of AIS $\geq$ 3 injuries is quite sensitive to the dummy data values.
2. The accident data do not support the authors' assertion that the risk of severe occupant injuries in the selected longitudinal barrier cases is small. Rather, from NASS 1984 data, the outcome of these longitudinal barrier collisions (under the favorable conditions of excluding end impacts, rollovers, vaulting, snagging, and underride) was found to be comparable to that of roadside accidents in general.
3. The authors found that the current occupant risk criteria are rarely a discerning assessment factor in redirection tests. Yet, a set of longitudinal barrier accidents with characteristics associated with successful crash test outcomes (no vaulting, no overturn, redirected vehicle, etc.) was found to have severities identical to roadside collisions in general.
4. In summary, the data in this paper, when supplemented by a comparison with roadside accidents in general, do not support the authors' conclusion that the occupant risk factor in redirectional crash tests is redundant. Rather, the data indicate that either the allowable lateral limit of ΔV should be lowered from the current value of 20 fps or the severity of the impact conditions (test speed-angle combinations) should be increased.

The link between measurements made on the crash test pad in redirectional-type crash tests and probability of injury has been recognized as a research need by specialists in this area for a number of years. The authors' use of the relatively new side impact dummies and reconstructions of accidents that have been investigated in depth is indeed valuable in increasing our current tenuous understanding in this area. Further study to see if the lateral ΔV limit of 20 fps should be lowered or test severity increased should be considered. The NASS and National Crash Severity Study data bases can be used to help interpret the results of such studies.

Authors' Closure

In the preceding discussion by a member of Committee A2A04, the discussant correctly states in his final summary that "the link between measurements made on the crash test pad in redirection-type crash tests and [the] probability of injury has been recognized as a research need by specialists in this area for a number of years." There have indeed been few investigations into the relationship between measurements made during full-scale vehicle crash tests and the risk of injury to vehicle occupants in real-world accidents. The data discussed in the paper represent a first step in an area that demands much more attention from the research community. The discussant has raised a number of topics and has helped to focus critical and creative thinking on these important issues. The authors are indebted to the discussant's diligence and insight and for this opportunity to further clarify our findings.

There are a number of specific questions in the discussion, but nearly all of them hinge ultimately on one of two issues: (a) the value and validity of data taken in the anthropometric dummy sled tests and (b) the acceptable level of injury specified in NCHRP Report 230 (1).

SLED TEST DATA

Figure 7 shows a plot of the dummy response data for the side impact sled tests given in Table 2 of the paper. The discussant states that because the spinal and lower rib accelerations vary slightly the data are flawed. Figure 7 shows that all of the data are within normal experimental tolerances. Furthermore, data for frontal impacts, also given in Table 2 of the paper, confirm that an occupant head impact velocity of 40 fps into a late-model vehicle windshield will produce an HIC of about 1000. This is also shown graphically in Figure 7. Thus the data are within the range of expected values, and the abbreviated test procedures used in this exploratory research appear to be both adequate and appropriate.

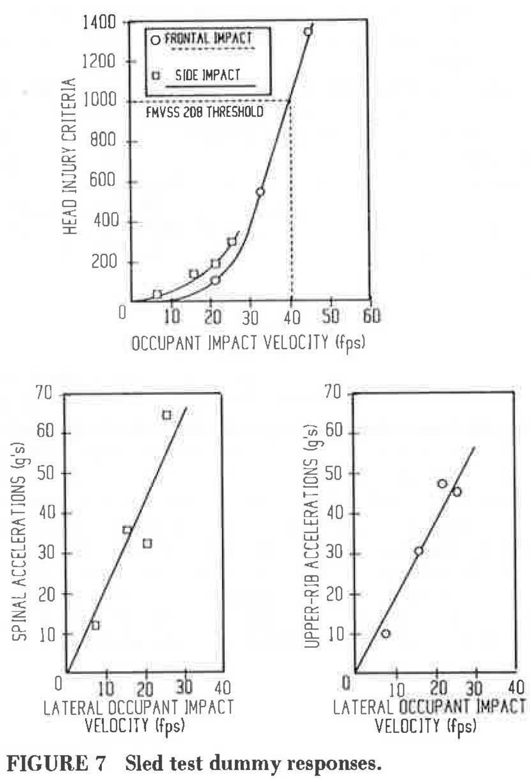

FIGURE 7 Sled test dummy responses.

The discussant apparently misunderstands the purpose of these sled tests: to examine the actual dummy response at various levels of occupant impact velocity. A flail distance of 6.5 in. was used in the sled tests because that was the actual distance measured between the head of the 50th percentile male side-impact dummy and the driver's-side door window. One of the basic assumptions of the flail space model is that human response to a collision is best quantified as a function of the occupant impact velocity. If two physiologically similar occupants experience identical occupant impact velocities, their responses should be similar regardless of the interior geometry or acceleration history of the vehicle. The NCHRP Report 230 lateral flail distance of 12 in. is used in evaluating full-scale crash tests to provide the worst case impact velocity given a particular acceleration history. In contrast, the purpose of these sled tests was to measure actual dummy responses at the occupant impact velocities actually experienced.

ACCEPTANCE INJURY THRESHOLD

Another key point of contention appears to be the question, "What should the upper bound for occupant injury severity be: AIS 2, 3, or 4?" Michie (12), in the original formulation of the flail space and occupant risk concept, suggested that:

> In line with current Federal Motor Vehicle Safety Standard (FMVSS) 208, an upper design limit for occupant protection falls between Codes 3 and 4.

This approach was restated in NCHRP Report 230 (1, p.30):

> An attempt has been made to set threshold values at a level equivalent to the American Association of Automotive Medicine Abbreviated Injury Scale (AIS) of 3 or less (3). AIS-3 classifies the resulting injury as severe but not life threatening.

Contrary to the discussant's understanding that NCHRP Report 230 specifies that the occupant has a 0.50 probability of receiving AIS-3 injuries, that report states in the passage quoted that all injuries of AIS 3 or less are acceptable though hardware developers should always strive to minimize occupant injury. Hence, the intention of NCHRP Report 230 is primarily to eliminate life-threatening injuries, that is, injuries of AIS 4 or greater.

When the acceptable injury range of AIS of 3 or less had been established, appropriate occupant impact velocities corresponding to the AIS-3 severity level were set based on the limited accident and research studies available to the author of NCHRP Report 230. A nominal 40-fps velocity was selected for occupants striking the windshield or instrument panel, and 30-fps velocity was selected for occupants striking the door. The 40-fps velocity threshold was well supported by research experience, in contrast to quite limited knowledge of human tolerance to side impacts. It was assumed in NCHRP Report 230 that occupant injury severity is a function of occupant impact velocity and that this injury severity would be lessened by reducing these impact velocities. Accordingly, reduction factors were applied to the 40- and 30-fps threshold velocities to arrive at design values of 30 and 20 fps for longitudinal and lateral impacts, respectively.

One of the objectives of this research program was to explore the relationship assumed in NCHRP Report 230 between lateral occupant impact velocity and injury in real-world accidents. As shown graphically in Figure 5, there were no occupant injuries during the first vehicle impact that were greater than AIS 3 even for occupant risk values of nearly 50 fps. From these data points, it appears to the authors that the lateral impact threshold limit of 30 fps may be too conservative and could be increased to 35 or 38 fps without adversely affecting occupant injury level. Simply stated, the design value of 20 fps may be unnecessarily restrictive, especially for more rigid longitudinal barrier systems, and could be relaxed to a design value of 25 or 30 fps.

CONCLUSION

The development of roadside safety hardware has been an active field of research for more than 25 years. Many of the attitudes and assumptions of the earlier years have become solidly cast into our present thinking about occupant protection with little regard to the validity of those assumptions today. The taxonomist Steven J. Gould has said that "Good science is self-correcting" (4); good engineering should also be self-correcting.

This study has suggested that the current 20-fps design limit for the lateral occupant impact velocity is not as crucial in mitigating injuries in redirectional collisions as was once believed. The effort spent by hardware developers in meeting this overly restrictive measure might better be spent in effecting improvements in other phases of the collision, namely the postimpact trajectory.

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Low-Maintenance End Treatment for Concrete Barriers

DEAN L. SICKING and HAYES E. ROSS, Jr.

ABSTRACT

The development of a low-maintenance crash cushion end treatment for concrete barriers is described. Features of the cushion include (a) no sacrificial energy-absorbing elements, (b) sufficient strength to withstand most impacts without damage to any components, (c) width approximately the same as that of the standard concrete safety shaped barrier, and (d) compliance with NCHRP Report 230 safety standards after only minor modifications. Results of six full-scale crash tests on the cushion are described.

Maintenance activities on heavily traveled urban freeways have become a major problem for most transportation agencies. Metal beam barriers on these freeways are frequently struck and must be repaired after most accidents. In recognition of these problems, highway engineers have begun to replace metal beam barriers with the almost maintenance-free concrete safety shaped barrier. However, the ends of these rigid concrete barriers pose both safety and maintenance problems. When left exposed or sloped to the ground, the rigid barrier end is a severe hazard. Efforts to mitigate this hazard include the use of crash cushion end treatments, flared ends, ends buried in earth berms or back slopes, and transition to a W-beam that is then terminated with a guardrail terminal. All of these safety treatments present some safety or maintenance problems, or both.

The crash cushion is probably the safest concrete barrier end treatment in use. However, crash cushion maintenance can be costly. All existing crash cushions use expendable energy-absorbing elements to attenuate head-on impacts, which destroy one or more of these energy-absorbing elements. Replacement of the damaged elements is costly, and for those end

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treatments that are struck frequently, repair costs during the life of an end treatment can be greater than initial costs. In an effort to reduce maintenance requirements associated with the use of concrete barriers on the roadside, a study was undertaken to develop a low-maintenance crash cushion end treatment for the concrete safety shaped barrier.

In this paper are described the findings of a research study funded by the Texas State Department of Highways and Public Transportation (1). The reader should refer to the cited report for more information about this study.

DESIGN CONSIDERATIONS

A large portion of crash cushion repair costs is associated with the repair or replacement of damaged components. The most effective method of cutting repair costs is to limit component damage by eliminating sacrificial energy-absorbing elements and strengthening other components. Maintenance costs can be cut further by reducing the size of the end treatment to cut the number of impacts.

Many concrete barrier end treatments must be placed very close to the traveled way. If a cushion is to have application at such sites, it must be narrow--ideally no wider than the standard concrete barrier. Narrow crash cushion end treatments must perform as a crash cushion when struck head-on and as a longitudinal barrier when struck downstream. Therefore the objective of this research was to design a low-maintenance crash cushion end treatment for concrete barriers that would (a) not have any sacrificial energy-absorbing elements, (b) have sufficient strength to withstand most impacts without damage to any components, (c) be approximately the same width as the standard concrete safety shaped barrier, and (d) meet nationally recognized safety standards (2).

ENERGY-ABSORBING ELEMENTS

The initial phase of crash cushion development involved a search for a material or device that could absorb large amounts of energy at high strain rates without sustaining any damage. Numerous chemical, plastic, and rubber companies were contacted during the search, and a large number of potential energy-absorbing materials were located. Samples were obtained of all materials that had the basic properties of interest, including Norsorex, Sorbothane, open- and closed-cell polyurethane and polyethelene foams, and several natural and synthetic rubber compounds. Spring manufacturers were also contacted regarding the potential use of steel springs as energy-absorbing devices.

Each of the candidate materials was evaluated to determine durability, response to static and dynamic loading, and cost and energy absorption per unit weight. Ultraviolet radiation and freeze-thaw tests were conducted to determine material durability, and high-speed (75-fps) and low-speed compression tests at several different temperatures were conducted to determine response to loading. Several rubber compounds were found to have the necessary durability and loading response for use in a crash cushion end treatment. The rubber cylinder, when used as ship and dock fenders, has been shown to absorb large amounts of energy and to be resistant to damage during impact loadings (3,4). Therefore a cylindrical rubber element was chosen for the energy-absorbing cartridge in the low-maintenance crash cushion end treatment.

The response of rubber cylinders to static transverse loadings has been thoroughly studied both empirically and theoretically (3-5). These studies have shown that for any particular rubber compound the static stiffness of a rubber cylinder is a function of the ratio between the outer diameter (D) and the wall thickness (t). Therefore the static stiffness of large-scale rubber cylinders can be determined by measuring the stiffness of scale-model cylinders with similar D/t ratios.

However, study of the dynamic response of rubber cylinders to transverse compression has been quite limited. The nonlinear characteristics of rubber and the large strains associated with the collapse of a cylinder make dynamic analysis virtually impossible. Therefore an empirical study of the dynamic force deflection characteristics of rubber cylinders was undertaken. One-fifth-scale-model cylinders, made from several different rubber compounds, were obtained in a variety of wall thicknesses. The scale-model cylinders were then tested statically and at three different impact speeds (5, 30, and 75 fps) to determine their force deflection characteristics. Figure 1 shows a sketch of the test setup used in the dynamic tests. Note that the test configuration allowed the sample to be compressed fully at a constant velocity.

The energy absorbed during a dynamic test has three sources: (a) inertia, (b) elastic stiffness, and (c) damping. As shown in Figure 1, when the bore

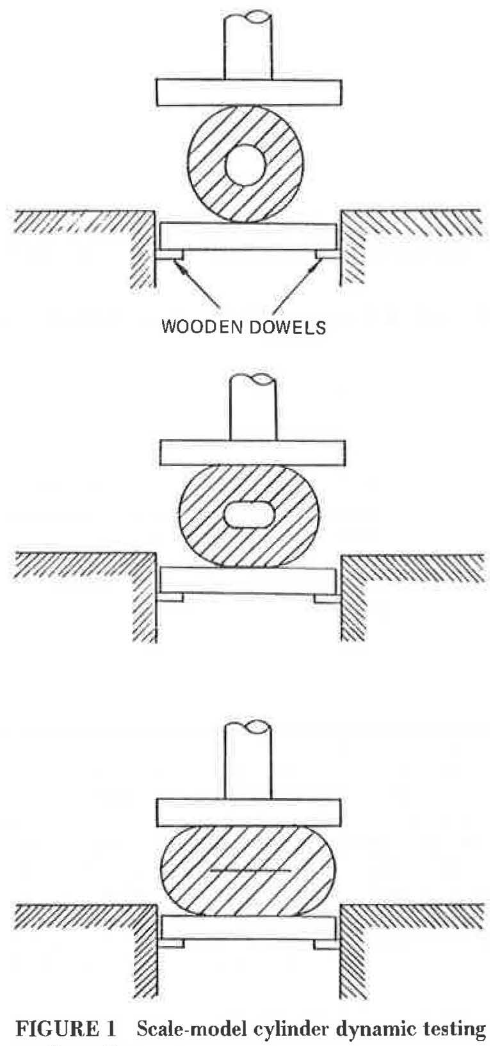

FIGURE 1 Scale-model cylinder dynamic testing configuration.

of a test specimen was completely collapsed, approximately one-half of the specimen had been accelerated to the speed of the impact plate, while the other half was virtually stationary. The energy absorbed by the inertia of the specimen was then estimated from the impact velocity and the mass of the specimen as

$$E_i = 1/2\ m\ V^2$$

where

E_i = energy transferred to cylinder due to inertia (in.-lb),
m = mass of cylinder (lb-sec^2/in.), and
V = velocity of impact plate (in./sec).

Energy absorbed due to the elastic stiffness of the specimen was measured from static testing. Energy attributable to internal damping within the specimen was then estimated from results of dynamic tests as

$$E_d = E_t - E_i - E_e$$

where

E_d = energy attributable to internal damping (in.-lb),
E_t = total energy absorbed during dynamic test (in.-lb), and
E_e = energy absorbed during static testing (in.-lb).

The energy absorbed by internal damping was found to be approximately the same for the tests at 30 and 75 fps. It can be concluded that damping within the tested rubber materials is of a hysteretic nature. Therefore energy absorbed by the rubber cylinders, with the exception of momentum transfer, is largely independent of impact velocity.

For purposes of estimating the energy absorbed by a full-scale cylinder, it was assumed that the ratio of elastic energy absorbed to damping energy absorbed was constant for each rubber compound and was unrelated to cylinder size or wall thickness. Static force deflection characteristics of large-scale rubber cylinders can be estimated directly from tests of scale-model specimens as mentioned earlier. Analysis of the results of dynamic scale-model tests indicated that thin-walled rubber cylinders do not absorb significant amounts of energy. Therefore the crash cushion design would have to use relatively thick-walled cylinders, which weigh in excess of 300 lb, and the front of the cushion would rely on momentum transfer to slow a colliding vehicle. The hardest rubber compound included in the study was selected for use in the cushion in an effort to reduce the total amount of rubber required. The selected compound is an 80-durometer natural rubber material.

Two 28-in.-diameter rubber cylinders, with wall thicknesses of 1.75 and 4.5 in., made from the selected compound were then fabricated and tested statically and dynamically to verify the loading response of large cylinders. Table 1 gives the estimated and the measured energy absorption characteristics of the full-scale rubber cylinders. As shown in the table, predicted values based on scale-model testing were quite close to measured values.

The cushion was then designed to attenuate head-on impacts with a single row of rubber cylinder energy-absorbing cartridges. The cushion was modeled for head-on impacts as a series of lumped masses and springs. The principles of conservation of energy and momentum were then employed to determine the impact severity of various sizes of vehicles as discussed in Ivey et al. (6). This analytic procedure is based on the assumption that the rubber cylinders will collapse one at a time such that one cylinder is almost completely collapsed before the next cylinder begins to be crushed. The final cushion design contained six thin-walled (1.75-in.) cylinders at the front of the cushion and seven thick-walled (4.5-in.) cylinders at the rear. Head-on impact severity measures predicted by the conservation of energy and momentum analysis are given in Table 2.

Scale-model cylinders of the selected compound were tested dynamically at -20°F and 120°F to determine the effects of temperature variation on the stiffness of the rubber. The variation in the energy absorbed, given in Table 3, was found to be less than 35 percent from the lowest test temperature to the highest. Because the front of the terminal behaves as an inertial cushion, it was possible to design the end treatment to perform acceptably at both temperature extremes.

END TREATMENT DESIGN

The final end treatment design, shown in Figure 2, consists of a single row of rubber cylinder energy-absorbing cartridges separated by steel diaphragms. A rubber cylinder is placed vertically in front of the end treatment to capture colliding vehicles and prevent override or underride of the cushion. The remaining rubber cylinders are placed horizontally to allow unrestrained collapse of the cylinders. Thrie-beam fender panels attached to the diaphragms and four 5/8-in. longitudinal cables provide redirectional capabilities. Fender panels are attached to the diaphragms with hinges to allow the thrie-beams to open outward without damaging the panels. Steel springs are used to prevent the fender panels from opening under wind loadings.

The rubber cartridges do not have sufficient elastic stiffness to completely restore the system after it has been struck. Four lightweight cables are attached between the diaphragms to allow the cushion to be pulled back into place after an impact. The end treatment is designed to sustain most impacts without replacement of any parts and to be restored to its original configuration in less than an hour.

TABLE 1 Full-Scale Test Results and Scale-Modeling Predictions

Sample (in.)			Static Energy (in.-lb)		Dynamic Energy (in.-lb)	
Wall Thickness	Outside Diameter	Length	Measured	Predicted	Measured	Predicted
1.75	28	24	23,940	22,510		74,280
1.75	28	24	23,880	22,510		74,280
4.50	28	24	180,360	134,640	231,600	215,400

TABLE 2 Predicted Occupant Impact Velocities for 60-mph Head-On Impacts

Vehicle Weight (lb)	Longitudinal Occupant Impact Velocity (ft/sec)
1,800	32
2,250	31
3,000	30
4,500	28

TABLE 3 Summary of Frozen Sample Testing

Ratio of Wall Diameter to Thickness of Sample	Energy Absorbed (in.-lb)		Change (%)
	Unfrozen	Frozen	
0.06	152	182	19.7
0.09	330	443	34.2
0.13	616	837	35.9

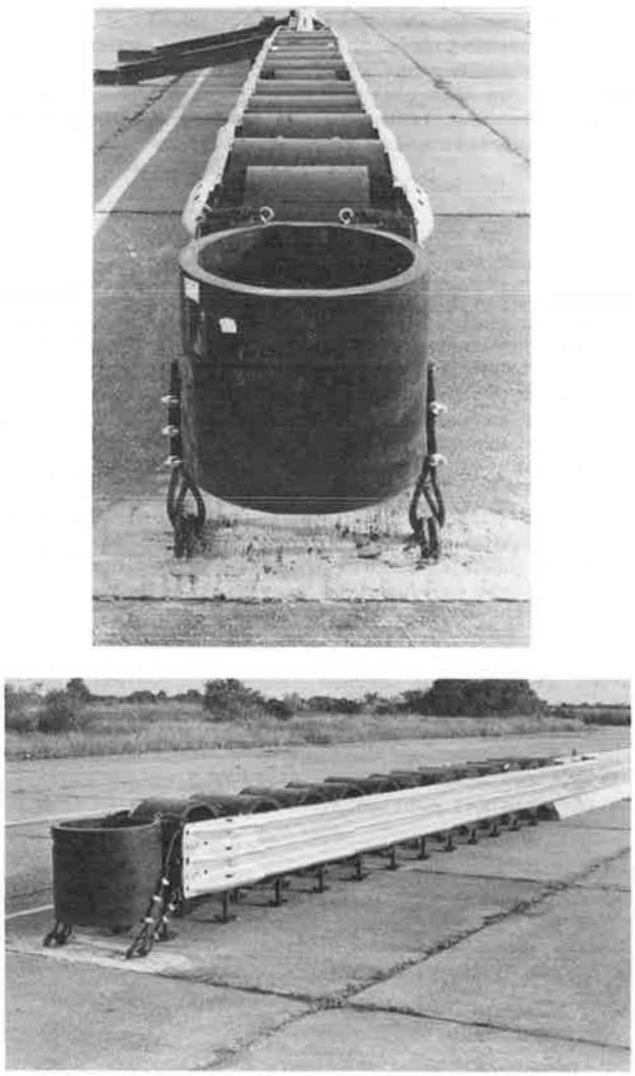

FIGURE 2 Low-maintenance end treatment.

PRELIMINARY TESTS

Three preliminary full-scale crash tests were conducted in an effort to find any design flaws before compliance testing. All three tests involved a 4,390-lb 1975 Ford Torino striking the cushion head-on.

Tests 1 and 2

The first test was conducted at 30 mph with an uninstrumented vehicle. The cushion performed well and stopped the vehicle in approximately 15 ft. The test vehicle exhibited no tendency to vault over or underride the cushion. The vehicle rebounded off the cushion at approximately 5 mph. As shown in Figure 3, the test vehicle was only lightly damaged and cushion damage was limited to minor bending of some of the skid shoes under the steel diaphragms.

FIGURE 3 Test vehicle and low-maintenance end treatment after 30-mph impact.

The cushion was pulled back into place in less than an hour and a second test was conducted at 40 mph. The end treatment smoothly decelerated the test vehicle over a distance of 17.5 ft and vehicle damage was light. The vehicle again rebound off the cushion at approximately 5 mph. Some of the hinges supporting the thrie-beam fender panels were damaged and the legs under the leading diaphragm were bent when they contacted the legs under the second diaphragm. Figure 4 shows the end treatment and test vehicle after the second test.

FIGURE 4 Vehicle and end treatment after 40-mph impact.

Test 3

The hinges on the front of the cushion were replaced with larger hinges and the method of attachment to the diaphragms was improved to reduce the possibility of damage. The legs on the first diaphragm were removed and replaced with a single leg in the center such that it would not contact the legs on the second diaphragm during impact. The test vehicle was then instrumented and a third test was conducted at 51 mph. The test vehicle was smoothly decelerated and was pushed back out of the cushion at approximately 7 mph. The vehicle was only moderately damaged, as shown in Figure 5. All occupant risk values, given in Table 4, were well below recommended limits (2). The end treatment was pulled back into place in less than an hour and, with the exception of some of the new hinges, was undamaged.

FIGURE 5 Vehicle and end treatment after Test 3.

COMPLIANCE TESTING

NCHRP Report 230 (2) calls for four full-scale crash tests of barrier end treatments. One of these tests calls for a 1,800-lb automobile to strike the middle of the end treatment at 60 mph and 15 degrees. Standard thrie-beam barriers and cable-supported narrow end treatments with thrie-beam fender panels have performed well under these tests conditions (6,7). It was therefore decided that this test would be eliminated from the matrix and the remaining three crash tests would be conducted.

Test 4

After completion of the third test, the hinges were redesigned to withstand an impact load of more than 200 g's. The new hinges were fabricated from 3/4-in. steel pipe and rod and 1/8-in. steel plate. Compliance testing was then begun with a 1979 Honda that weighed 1,810 lb striking the cushion at 58 mph and zero degrees. The center of the test vehicle was offset 16 in. from the center of the cushion. The small automobile was smoothly decelerated to a stop over a distance of approximately 17 ft. As the front of the vehicle came to a stop, the rear began to spin out. As shown in Figure 6, the vehicle was yawed approximately 90 degrees from its original direction of travel when it stopped.

The modified hinges contacted the next fender panels and prevented the front five cells from col-

TABLE 4 Summary of Crash Test Results

Test No.	Vehicle Weight (lb)	Impact Speed (mph)	Angle of Impact (degrees)	Point of Impact	Vehicle Stopping Distance (ft)	Occupant Impact Velocity (ft/sec)		Occupant Ridedown Accelerations (10 msec avg g's)	
						Longitudinal	Lateral	Longitudinal	Lateral
1	4,390	30	0	Nose	15	–[a]	–[a]	–[a]	–[a]
2	4,390	40	0	Nose	17.5	–[a]	–[a]	–[a]	–[a]
3	4,390	51	0	Nose	22.5	22.0	–[a]	7.7	–[a]
4	1,810	58	0	Nose at 16-in. eccentricity	17.5	35.5	4.2	9.0	1.5
5	4,500	57	0	Nose	23.5	26.4	NA	14.1	NA
6	4,420	61	25	8th fender panel	NA	32.7	18.9	20.9	32.5

Note: NA = occupant did not strike side of vehicle.

[a]Not measured.

FIGURE 6 Vehicle and end treatment after Test 4.

lapsing completely. As a result, the longitudinal occupant impact velocity was somewhat high at 35.7 fps, whereas the recommended value is 30 fps and the maximum allowable impact speed is 40 fps. If the hinges had not prevented the front cells from collapsing completely, the test vehicle would have traveled approximately 2.5 in. further between impacts with each diaphragm and therefore the occupant impact velocity would have been much lower. There would have been little additional speed reduction between diaphragms because the front cells do not absorb a significant amount of energy. The occupant impact velocity can be estimated for this condition by integrating the accelerometer curve from the test and adding 2.5 in. of free travel (no acceleration) after collapsing each cell. The predicted occupant impact velocity from this type of analysis is approximately 31 fps.

As the data in Table 4 indicate, all other severity measures with within recommended limits (2). No components of the crash cushion were damaged, and it was restored with less than 4 man-hours of labor. After the fourth test the hinges were notched to prevent contact between hinges and the downstream fender panels.

Test 5

The fifth test involved a 1978 Mercury Grand Marquis that weighed 4,500 lb striking the cushion head-on at 57 mph. The cushion performed well and brought the vehicle to rest over a distance of approximately 23 ft. All measures of occupant risk were below recommended limits as the data in Table 4 indicate. The vehicle rebounded off the cushion at 10.5 mph.

The cushion and test vehicle were damaged moderately, as shown in Figure 7. One of the redirectional cables snagged on a diaphragm and was broken, and one of the lightweight restoration cables between the diaphragms was cut. As a result, the cushion could not be pulled back into position as in previous tests. In addition, there was minor damage to several of the hinges and the legs under the diaphragms. There was still some minor contact between the 3/4-in. rods on the hinges and the fender panels. Therefore it is recommended that the hinges be replaced with a flat plate design. This design should be slightly stronger than those used in the tested design.

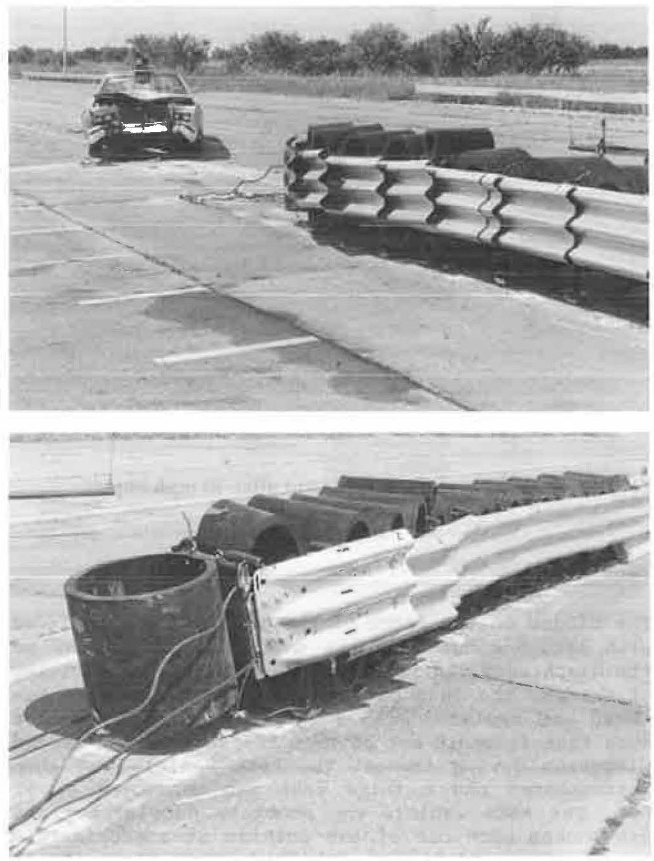

FIGURE 7 Vehicle and end treatment after Test 5.

Cushion repair was accomplished by replacing two 5/8-in.-diameter lateral restraint cables and two 1/4-in.-diameter restoration cables. It should be noted that the damaged lateral restraint cables were old and may have been frayed or damaged during previous research. However, it is recommended that all lateral restraint cables be visually inspected after every accident.

Analysis of test films indicated that all of the energy required to push the vehicle out of the cushion originated from the large-diameter cylinders at the rear of the treatment. If the 10.5-mph exit velocity is a significant concern, vehicle rebound can be virtually eliminated by placing displacement limitation devices on the redirectional cables at the sixth diaphragm. These devices would allow the diaphragm to be freely pushed backward but would limit any forward motion of the diaphragm after the vehicle was stopped.

Test 6

The final test on the end treatment involved a 4,420-lb Ford LTD striking the cushion at 61 mph and 25 degrees. The center of the test vehicle was directed at the center of the barrier end to maximize the possibility of the vehicle snagging on the end of the barrier. The test vehicle was redirected and exited the barrier at a very low angle.

During the test the lateral support element at the front of the concrete barrier gave way and the front of the barrier deflected 4 in. As the barrier was deflected, it rolled away from the impacting vehicle thereby extending the lower curb face beyond the edge of the treatment. The wheels of the test vehicle snagged somewhat on the exposed lower face and generated relatively high longitudinal and lateral forces on the automobile. Although barrier anchorage for field installations would likely be more substantial and limit this problem, it is recommended that the barrier end be transitioned to a vertical wall to further reduce the likelihood of such an occurrence.

The end treatment was not damaged heavily for a test of this severity, as shown in Figure 8. Repair would have been limited to the replacement of the last diaphragm, two thrie-beam fender panels, one wood block-out on the face of the concrete barrier, and one redirectional cable. No rubber cells showed any sign of damage. As in most impacts of this severity, the test vehicle sustained considerable damage.

FIGURE 8 Vehicle and end treatment after Test 6.

CONCLUSIONS

A low-maintenance crash cushion end treatment for concrete barriers has been successfully designed and crash tested. The cushion (a) does not have any sacrificial energy-absorbing elements, (b) has sufficient strength to withstand most impacts without damage to any components, (c) is not significantly wider than the standard concrete safety shaped barrier, and (d) has been shown to meet nationally recognized safety standards (2). Rubber cylinder energy-absorbing cells used in the cushion have withstood six relatively severe crash tests and show no signs of significant damage.

The crash cushion end treatment described here represents a significant step toward reducing maintenance costs associated with such devices. The cushion can withstand relatively severe head-on impacts--small automobiles traveling at speeds of up to 60 mph and large automobiles traveling at speeds of up to 50 mph--without sustaining damage to any components. These impact conditions include more than 95 percent of expected head-on accidents (8). After these accidents the cushion can be repaired in less than an hour and total repair cost should be less than $100. Further, even high-energy head-on and relatively severe side impacts do not cause a great deal of damage to the system. Finally, the design concepts proven in this study could be easily adapted to other types of cushions with a potential for similar reductions in maintenance costs.

ACKNOWLEDGMENTS

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Development of Condition Surveys and Inventories for Guide Rail and Drainage Facilities

G. J. MALASHESKIE et al.

ABSTRACT

In 1983 the Pennsylvania Department of Transportation implemented a systematic technique to analyze and manage Pennsylvania pavements (STAMPP), its first complete pavement management system. With STAMPP as the foundation, the department has embarked on the development of a total roadway management information system (RMIS), which to become functional requires the addition of guide rail and drainage conditions. A special task force, comprised of district, county, and central office personnel developed the techniques with which to inventory and collect the condition data for guide rail and drainage facilities, assigned treatment strategies and related costs to deficient conditions, and proposed methods for implementing identified survey results. The result is a more complete roadway management system, which allows department management to more effectively manage approximately 43,000 mi of highway pavements, shoulders, and appurtenances.

In 1983 the Pennsylvania Department of Transportation implemented its first complete pavement management system, a systematic technique to analyze and manage Pennsylvania pavements (STAMPP). This methodology provides department personnel with an objective, useful tool to more effectively manage approximately 43,000 mi of highway pavements and shoulders. STAMPP is now being used as the foundation of a roadway management information system (RMIS), currently under development by the department. In the development of the RMIS, the STAMPP task force identified a need for inclusion of data on guide rail, median barrier, and drainage facilities, along with the pavement and shoulder data being collected in STAMPP. This would result in a more complete roadway management system.

To identify the information required, and to develop a methodology to acquire that information, a special task force comprised of district, county, and central office personnel was formed. The task force members represented a variety of engineering and managerial disciplines: highway maintenance, design, pavement management, planning, and highway safety. These individuals were charged to evaluate and recommend techniques by which to systematically inventory and collect the condition data for guide rail and drainage, to assign appropriate treatment strategies to deficient conditions, and to implement these results as enhancements to STAMPP.

In their initial sessions, the task force adopted the following working objectives:

1. Develop a uniform 100 percent inventory and survey of statewide guide rail and drainage facility conditions;
2. Identify appropriate condition criteria, recommend treatments, and estimate associated costs; and
3. Develop factors relative to the inventoried guide rail and drainage items for use in allocating maintenance monies to the counties.

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The task force also adopted the following co-objectives from the STAMPP Pavement Condition Survey Field Manual (1):

1. To provide a uniform statewide condition evaluation that would improve decision making;
2. To provide management with the information and tools to monitor the condition of the network, assess future needs, establish county condition rankings, and optimize investments;
3. To provide condition information to fulfill the requirements of Pennsylvania Act 68 (1980), which requires the allocation of maintenance funds to the individual counties based on need;
4. To provide information for monitoring the performance of various pavement (guide rail and drainage) designs, materials, rehabilitation, and maintenance techniques; and
5. To provide information for identifying candidate projects for maintenance and betterment programs.

Over a period of several weeks, the task force developed the criteria and survey input forms to conduct the inventories of guide rail, median barrier, and drainage facilities on the state's highways. These criteria and forms underwent several stages of revision as a result of field surveys conducted by the task force members and meetings held with personnel in the more urban districts. The result is an inventory that satisfies the previously stated objectives and yields the following benefits to the department:

1. It provides condition information to fulfill the requirements of Act 68, enabling the department to modify the RPQI portion of the formula to allocate maintenance funds to individual counties. Overall needs will be better defined as a result of the information that will be gathered.
2. The information collected on the condition of barrier and drainage systems can be used to generate work plans for county maintenance operations.
3. In conjunction with the current STAMPP data, guide rail and drainage condition data can be used

in the development of the department's annual Highway Restoration Program.

4. Overall system management is enhanced. All managers, whether at the district, county, or central office level, will have the necessary information to better assess their future needs.

5. The survey forms that were developed will facilitate the collection of information that can be readily integrated into the roadway information data base (RIDB). Ultimately, this information will be available to develop automated straight line diagrams (SLDs), which indicate, among other things, the type and location of all traffic barriers and drainage facilities along each section of highway.

6. It improves the department's ability to address tort claims associated with guide rail or drainage conditions. The additional information that will be available to all managers will enable them to assess their needs and better establish logical priorities for improvements. This is true at the county level, in the development of annual maintenance work plans; at the district level, in the development of the annual Highway Restoration Program; and at the central office level, in the establishment of overall program guidelines.

GENERAL INFORMATION AND SURVEY DEVELOPMENT

Both the guide rail and the drainage surveys were developed to be compatible with STAMPP methodology for conducting visual condition surveys. The existing STAMPP segments and offset distances are used to locate barriers and drainage items.

The initial surveys for both guide rail and drainage will be inventories as well as condition surveys. The guide rail survey will be done on 100 percent of the highway network system on an annual basis. In the case of drainage, the complete survey of the system will be phased in over a period of 4 years, by doing 25 percent of each of the Primary Commercial Network (PCN) and the Off-PCN roadways each year. It is recognized that periodic updates will be required to quantify the condition of the drainage or barrier elements; however, this will not necessarily have to be done on an annual basis. To keep the inventory data current for both barriers and drainage, a means will be established to automatically update the information whenever work is accomplished by department forces or under contract. This will require some software development and an interface with other department recordation systems. The result will be an up-to-date inventory that will require less resurveying and will therefore be less expensive to maintain.

GUIDE RAIL SURVEY

The task force considered existing department design criteria and maintenance techniques when developing the guide rail inventory and condition survey format. Some of those considerations included the Standard Roadway and Bridge Construction Drawings, the Highway Design Manual, Part 2, Chapter 12, on Guide Rail and Median Barrier, the Maintenance Manual criteria on guide rail maintenance and replacement, the Highway Features Inventory-System, and other existing planning criteria used for I-4R, 3R, and betterment project development.

After discussions with district personnel, primarily in the commonwealth's two major urban areas, it was decided that the end treatments and actual systems currently found on roadways would be specified on the survey forms and used in the inventory, as opposed to a previous plan to merely include systems broken down by cable or panel, and strong or weak posts. The additional time required to do a more detailed inventory was considered a good tradeoff for getting a more complete inventory and accurate cost estimate of needs for maintenance and repairs.

Guide rail and median barrier system conditions are observed for extent and severity of post deflection, cable sag, system deterioration, hardware condition, and system height. Treatments and associated costs were identified for the various conditions or combinations of conditions. These are used in estimating the total needs relative to barrier condition and also in the development of STAMPP project cost estimates and are based on actual repair and construction costs currently being quoted on Pennsylvania contracts.

The condition of the system end treatments is identified as functional or nonfunctional on the basis of the ability of the treatment to perform its intended function. For bridge connections, only a current strong-post W-beam rail system with appropriate connection hardware and reduced post spacing in the vicinity of the structure is considered functional.

An important aspect of the survey is the identification of potentially unneeded guide rail and median barrier. The task force recommended that removal of existing nonfunctional systems be stressed, with update only where truly needed. The task force's sentiments were strongly reinforced by district personnel, and a reassessment of warrants and standards has been mandated by department management. It was also recognized that the department did not fully use the cost-effectiveness approach outlined in the 1977 AASHTO "Guide for Selecting, Locating, and Designing Traffic Barriers" and subsequent publications.

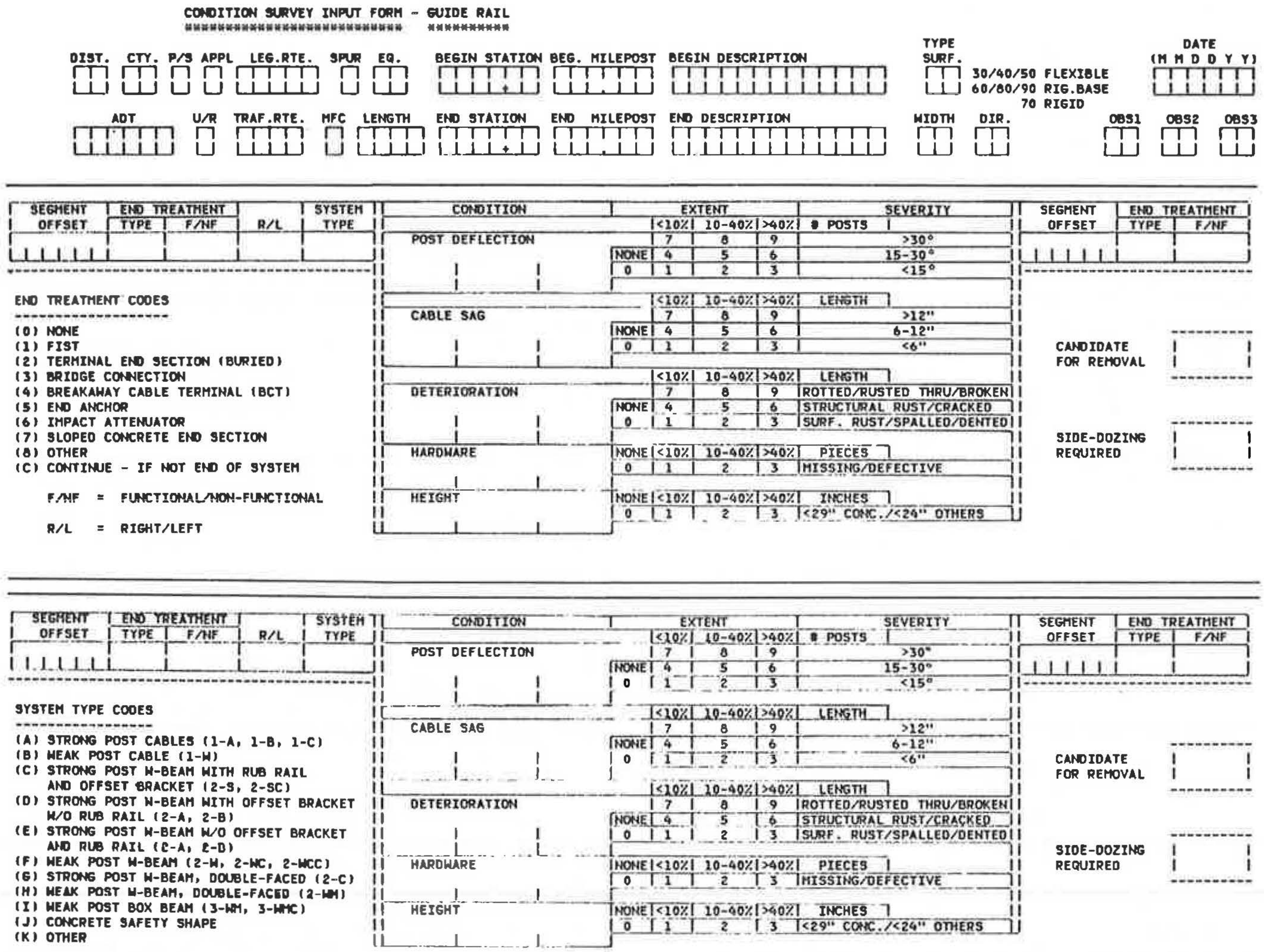
A reevaluation of placement options for typical existing conditions was recommended in accordance with the AASHTO cost-effectiveness analysis. This will include guide rail along cut-and-fill slopes, along tree-lined rural roadways, in areas where speeds have been reduced, and so forth.

Subsequent to development of the survey format, a second task group was convened to consider existing national and Pennsylvania warrants for barrier use and to evaluate revisions to design criteria in accordance with the AASHTO barrier guide Chapter 7 cost-effectiveness approach. Easy-to-use criteria were developed for use as a guideline for checking the "candidate for removal" block and revised standard criteria were recommended for consideration. These criteria are more liberal than previous criteria in that they recognize motorists' ability to safely negotiate certain slopes and fill heights. Moreover, they take into account the probability of a motorist losing control and encountering the slope. If these criteria are adopted, significant cost savings can be realized through reduced maintenance needs where barrier is removed and lesser construction costs on 3R projects. Removal of old and unwarranted barriers and a program to update warranted barriers will provide the motoring public with the most cost-effective and safe system practicable. Moreover, by reassessing barrier warrants, the department will be able to upgrade the truly needed barrier systems without spending limited funds on questionable or unneeded installations.

The guide rail survey form is shown in Figure 1, and treatment strategies and costs are given in Tables 1-4.

DRAINAGE SURVEY

Development of a drainage survey form, as in the case of guide rail, started with identification of appropriate inventory items. It was recognized that



CONDITION SURVEY INPUT FORM - GUIDE RAIL

DIST. CTY. P/S APPL LEG.RTE. SPUR EQ. BEGIN STATION BEG. MILEPOST BEGIN DESCRIPTION

TYPE SURF. 30/40/50 FLEXIBLE 60/80/90 RIG.BASE 70 RIGID

DATE (M M D D Y Y)

ADT U/R TRAF.RTE. MFC LENGTH END STATION END MILEPOST END DESCRIPTION WIDTH DIR. OBS1 OBS2 OBS3

SEGMENT OFFSET | END TREATMENT TYPE F/NF | R/L | SYSTEM TYPE

CONDITION	EXTENT				SEVERITY
POST DEFLECTION		<10%	10-40%	>40%	# POSTS
		7	8	9	>30°
	NONE	4	5	6	15-30°
	0	1	2	3	<15°
CABLE SAG		<10%	10-40%	>40%	LENGTH
		7	8	9	>12"
	NONE	4	5	6	6-12"
	0	1	2	3	<6"
DETERIORATION		<10%	10-40%	>40%	LENGTH
		7	8	9	ROTTED/RUSTED THRU/BROKEN
	NONE	4	5	6	STRUCTURAL RUST/CRACKED
	0	1	2	3	SURF. RUST/SPALLED/DENTED
HARDWARE	NONE	<10%	10-40%	>40%	PIECES
	0	1	2	3	MISSING/DEFECTIVE
HEIGHT	NONE	<10%	10-40%	>40%	INCHES
	0	1	2	3	<29" CONC./<24" OTHERS

SEGMENT OFFSET | END TREATMENT TYPE F/NF

CANDIDATE FOR REMOVAL

SIDE-DOZING REQUIRED

END TREATMENT CODES

(0) NONE
(1) FIST
(2) TERMINAL END SECTION (BURIED)
(3) BRIDGE CONNECTION
(4) BREAKAWAY CABLE TERMINAL (BCT)
(5) END ANCHOR
(6) IMPACT ATTENUATOR
(7) SLOPED CONCRETE END SECTION
(8) OTHER
(C) CONTINUE - IF NOT END OF SYSTEM

F/NF = FUNCTIONAL/NON-FUNCTIONAL

R/L = RIGHT/LEFT

SYSTEM TYPE CODES

(A) STRONG POST CABLES (1-A, 1-B, 1-C)
(B) WEAK POST CABLE (1-W)
(C) STRONG POST W-BEAM WITH RUB RAIL AND OFFSET BRACKET (2-S, 2-SC)
(D) STRONG POST W-BEAM WITH OFFSET BRACKET W/O RUB RAIL (2-A, 2-B)
(E) STRONG POST W-BEAM W/O OFFSET BRACKET AND RUB RAIL (2-A, 2-D)
(F) WEAK POST W-BEAM (2-W, 2-WC, 2-WCC)
(G) STRONG POST W-BEAM, DOUBLE-FACED (2-C)
(H) WEAK POST W-BEAM, DOUBLE-FACED (2-WM)
(I) WEAK POST BOX BEAM (3-WM, 3-WMC)
(J) CONCRETE SAFETY SHAPE
(K) OTHER

FIGURE 1 Guide rail survey form.

the drainage inventory needs would be much different in urban, curbed areas than on the rural roadway system. After much discussion within the task force and comments from the urban districts, inventory items were agreed on that included pipe and other structures less than 8 ft in width measured along the roadway centerline (structures 8 ft or greater are included in the structures inventory), inlet control, outlet control, outlet ditches, and parallel ditches. Treatments and related costs are based on normal maintenance treatments wherever possible and action is indicated based on flow conditions, structural conditions, and physical condition of the pipe, structure, inlet or outlet, ditch, and any apparent roadway distress caused by structure failure.

Figure 2 is the sample drainage survey form; Tables 5 and 6 give treatment strategies based on drainage conditions and associated treatment costs.

CONDUCTING SURVEYS

It is the intent of the task force that the initial guide rail and drainage surveys be inventories as well as condition surveys. To properly indicate present and future needs, a 100 percent survey of

TABLE 1 Guide Rail Treatment Strategies

	Extent and Severity								
	Low			Medium			High		
Condition	1 (<10%)	2 (10-40%)	3 (>40%)	4 (<10%)	5 (10-40%)	6 (>40%)	7 (<10%)	8 (10-40%)	9 (>40%)
Post deflection	X	X	X	2	2	2	3	3	4
Cable sag	X	X	1	1	2	2	3	3	3
Deterioration	X	X	X	1	1	3	3	3	4
Hardware	1	3	3						
Height	2	2	2						

Note: 1 = routine maintenance, 2 = reset (repair in place), 3 = replace in kind, and 4 = update only if system does not meet current standards. Combinations for update if system does not meet current standards: A6 + B9, B9 + C8, and A8 + B9.

TABLE 2 Treatment Strategies—Nonfunctional End Treatments

Type	Cable System	Panel System	Concrete Barrier
0	3,4	3	3
1		3,4	
2		3	
3	4	3,4	
4		3	
5	3,4	3	
6	3	3	3
7			3
8	4	4	4
C			

Note: 1 = routine maintenance, 2 = reset (repair in place), 3 = replace in kind, and 4 = update only if system does not meet current standards.

TABLE 3 Treatment Costs for Guide Rail Systems

System	1. Routine Maintenance	2. Reset/Repair in Place	3. Replace in Kind	4. Update
A	1.50	4.38	6.20	11.50
B	1.50	4.38	6.20	6.20
C	1.50	5.75	16.50	16.50
D	1.50	5.75	11.50	11.50
E	1.50	5.75	10.00	11.50
F	1.50	5.75	10.00	10.00
G	1.50	8.00	19.00	19.00
H	1.50	5.75	16.80	16.80
I	1.50	12.78	33.00	33.00
J	1.50	3.50	23.50	23.50
K				11.50

Note: costs in dollars per linear foot.

the identified inventory items is essential. Proper updating of the system, to include newly constructed features, repair or replacement of existing features, and elimination of features, is necessary in order to make the system functional.

The task force assessed various options for conducting each survey and assigned relative costs to the various options. For the guide rail survey, four options were presented for consideration:

- Option 1: Conduct the survey annually in conjunction with the present STAMPP survey by the addition of a third person in the STAMPP vehicle. It was anticipated that the third person would be able to do the guide rail survey and the STAMPP shoulder condition survey. This method would eliminate the need for additional survey vehicles and other associated equipment. One drawback to this option is the anticipated initial reduction of approximately 2 mi per day in the STAMPP survey production. This is expected to happen only the first time the survey is made, because this will be both a condition survey and an inventory. Long-term production should actually increase. Estimated annual cost is $363,000 compared with $215,000 for the STAMPP condition survey alone.
- Option 2: Use of separate survey crews to conduct each of the guide rail and STAMPP condition surveys. The advantage is that the efficiency of each survey will not be affected; however, additional personnel, vehicles, and equipment are required. The cost of two separate sets of survey crews is estimated at $356,000.
- Option 3: Have the two-man STAMPP survey crew perform a second pass on each roadway segment to pick up the guide rail survey. This would significantly reduce survey efficiency and may be prohibitive in terms of time required; however, advantages include reduced personnel and equipment needs. Estimated cost is the same as Option 2.
- Option 4: Conducting the survey by engineering consultant contract was discussed at length and judged to be cost prohibitive, although no actual cost estimate was derived. Advantages are reduced department personnel and equipment needs and noninterference with the STAMPP survey.

The task force recommendation was to use Option 1 because the estimated survey costs were in line with the other options presented and would make available a yearly update of guide rail needs for development of the counties' annual work plans and allocation of maintenance monies.

The drainage survey will have to be conducted separate from the STAMPP and guide rail surveys because it will entail considerable "walking" of each segment to assess conditions of drainage items and measure extents of some conditions. Regardless of how the survey is conducted, it is imperative that as many drainage locations as possible be identified before going into the field by checking as-built plans, when available. This will increase efficiency of the field survey as well as provide a check to assure that as many of the drainage items as possible are located and inventoried.

To replace the current trained observer survey (TOS) cycles in the maintenance allocation formula it was recommended that the drainage survey initially be conducted on the Off-PCN roads, with emphasis on those roads scheduled for surface improvement or on the 4-year plan because these are considered to generally have more urgent drainage needs. One option for conducting the survey was to have the assistant county maintenance managers responsible for inventorying and evaluating drainage conditions over approximately an 8-year period in advance of

TABLE 4 Treatment Costs for Nonfunctional End Treatments

	3. Replace in Kind			4. Update		
Type	Cable System	Panel System	Concrete	Cable System	Panel System	Concrete
0	240	750	140	750	750	
1		750			750	
2		750				
3		144		550	550	
4		1,200			1,200	
5	240	500		750		
6	1,000	1,000	1,000			
7			140			
8				240	750	140
C						

Note: cost in dollars each.

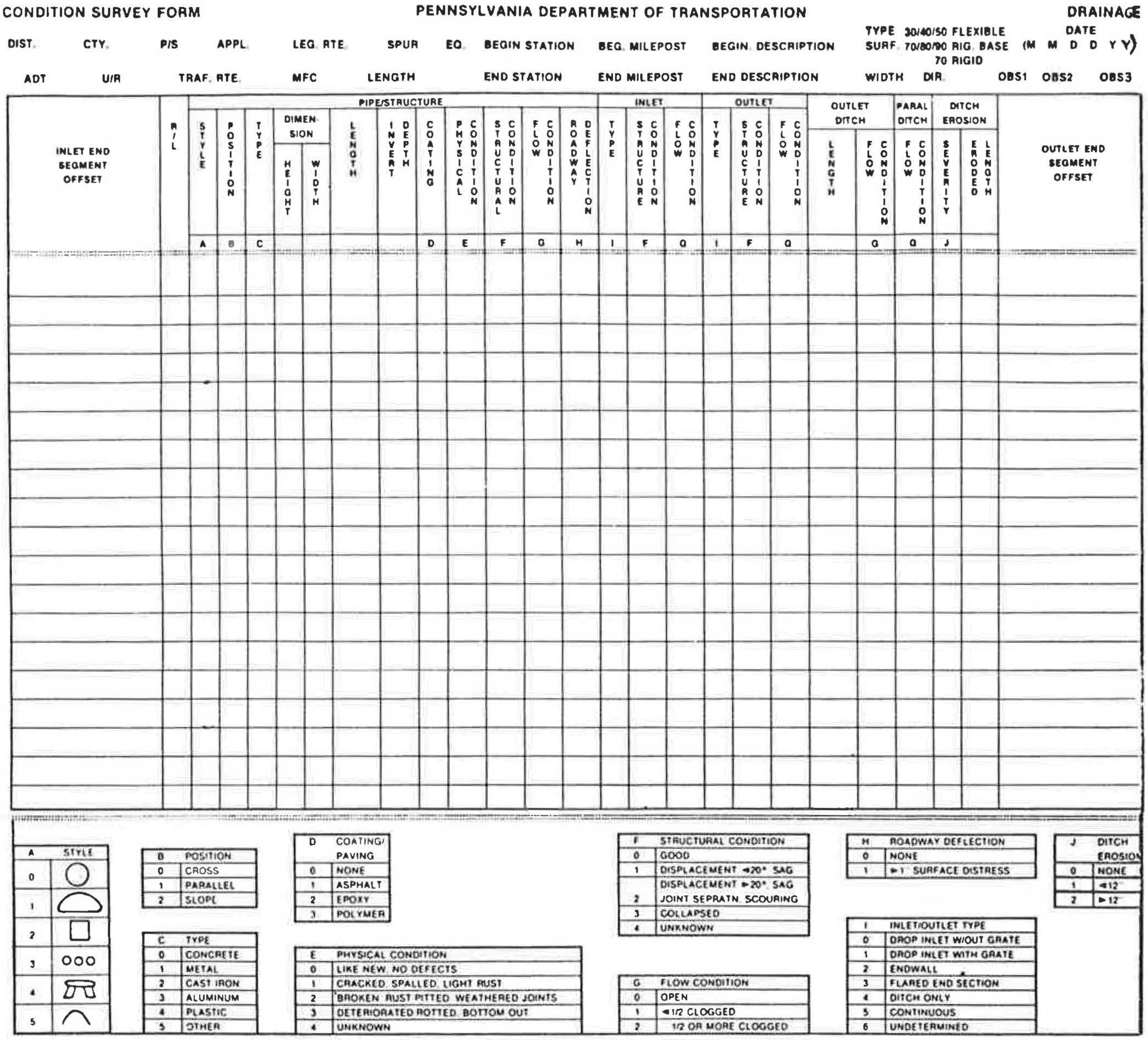


FIGURE 2 Condition survey input form—drainage.

TABLE 5 Drainage Treatment Strategies

Condition	Extent	Pipe Style (0-3)	Structure Style (4&5)
Physical, E	0	1	1
	1	1	1
	2	2	7
	3	4	7
	4	1 or 2	1 or 2
Structural, F	0	1	
	1	1	
	2	3	
	3	4	
	4	1 or 2	
Flow, G	0	1	
	1	2	
	2	5	
Ditch erosion, J	0	1	
	1	3	
	2	6	

Note: 1 = do nothing, reinspect in 5 years; 2 = do nothing, reinspect in 2 years; 3 = repair/reset; 4 = replace; 5 = flush/clean and inspect; 6 = reline; and 7 = requires inspection by bridge inspection personnel (SIRS). Combination for replacement (4): Flow condition G = 2 + roadway deflection H = 1.

their yearly surface improvement program or 4-year plan.

Another option was creation of a maintenance technician position as an assistant to each assistant county manager. This technician would be responsible for coordination of the survey with county forces over a 2-year period. A general option was to mandate conduct of the survey, starting with the Off-PCN roads, to be completed over a 2-year period, allowing the district engineer to decide who will conduct the survey. Some available personnel options included consultant contract, coordination by district staff, county staff, temporary or part-time personnel, and workfare people. It is estimated that at 5 mi per day for a two-man survey team, 17,600 man-days will be required to conduct the drainage survey.

The task force could not agree on a single, most favorable option. Thus no specific recommendation was made about who should conduct the drainage survey. The option of allowing the district engineers the choice of personnel appeared to make the most

TABLE 6 Treatment Costs—Drainage Items

Treatment	Pipe and Pipe-Arch 0 1	Box Culvert 2	Multiple Pipes 3	Small Structures 4 5	Inlets and Outlets	End Walls	Pipe and Culvert End Sections	Parallel and Outlet Ditches	Erosion Control
1									
2									
3	≤36 in., \$23.00/LF >36 in., \$46.00/LF	\$100.00/LF	\$75.00/LF	\$35.00/ft²	\$400.00 each	\$100.00 each	\$100.00 each		
4	≤36 in.,\$62.00/LF				\$1,200.00 each		≤36 in., \$500.00 each		
	>36 in., \$122.00/LF	\$125.00/ft²	\$105.00/ft²	\$135.00/ft²	\$1,500 each with grate	\$250.00 each	>36 in., \$1,000.00 each		
5	\$2.50/LF	\$36.00/LF	\$7.50/LF	\$36.00/LF	\$25.00 each	\$25.00 each	\$25.00 each	\$1.00/LF	
6									\$15.00/LF
7				\$300.00 each					

Note: LF = linear foot. Circled numbers represent style (see Figure 2).

sense because they would be better aware of their district and county personnel commitments and needs and any budgetary constraints.

The drainage survey has built-in cycles of reinspection based on existing condition, inability to assess conditions because of present survey limitations, and the degree of inspection expertise required on small structures. However, it was generally recommended that, after the initial inventory and condition survey, reinspection should be performed at 5-year intervals unless a shorter period is deemed more suitable.

STAMPP ENHANCEMENTS

It was recommended that side-dozing and swale grading be included in the STAMPP survey; however, only side-dozing was considered appropriate because swale grading is generally included in the shoulder cutting treatment for a buildup condition in the shoulder portion of the STAMPP survey. Several other RMIS system enhancements were also recommended:

1. Add an assistant county manager designation to each STAMPP segment to expedite county data acquisition for development of annual work plans. This can be built into the RMIS currently under development by the department.
2. In developing project cost estimates (2), drainage costs, based on the condition survey, should be added to the normal project assessment software programming. Guide rail costs, again based on the condition survey, should be provided as an option for project cost estimate development in a manner similar to that in which maintenance and protection of traffic, mobilization, and engineering are currently handled.
3. Include the guide rail and drainage inventory items in the development of the automated straight line diagrams.

OUTPUT AND SOFTWARE NEEDS

The formats used for each of the guide rail and drainage survey forms require some specific software programming to assure that output needs are properly addressed.

For the guide rail form, programming will be required to account for a continuous string of guide rail that extends onto an adjacent STAMPP segment, continues onto a ramp, or continues onto an intersecting road (centerline route or local). The system must be able to output an indication of need for an update to end treatments and systems identified as "other" or "none." Bridge connections identified as "nonfunctional" should also have an automatic update treatment indicated. The cost of this update should include the standard bridge terminal section and 25 ft of 2-SC guide rail, for estimation of needs or development of a project cost estimate.

On the drainage form, multiple pipes will require programming to determine the number of pipes at the location. Inlets and outlets identified as "undetermined" for programming purposes should be included with drop inlets without grates, although no costs for treatment are to be specified. Similarly, the physical or structural condition "unknown" must automatically be programmed to indicate the need for a more detailed reinspection.

Continuous parallel ditches and storm sewers with drop inlets acting as junction boxes (identified as "continuous") require programming to connect ditches and storm sewer systems from one STAMPP segment to the other. As in the case of continuous guide rail systems, this will be more important when implemented on the automated straight line diagrams.

The input of data from both surveys will need some program edits to control faulty information. Summary treatment screens as well as condition summaries by segment and other information generally output in the existing STAMPP data analysis programming (2) should be made available. The ability to have preprinted forms for subsequent surveys must be built into the program because this will significantly increase survey efficiency and thereby reduce survey costs.

It was recognized that the counties will generally want to use the data available from the surveys in developing their annual work plans and preparing guide rail and pipe repair or replacement contracts. Computer program formatting for generation of these reports should also be made available.

The districts, counties, and central office will be able to obtain the following typical information from each survey:

Drainage

1. Number of feet of pipe by size and condition per county,
2. Number of feet of pipe by size and condition per Legislative Route (LR),
3. Inlets needing repairs by LR,
4. Inlets needing reconstruction or replacement by LR or county,
5. Ditch cleaning needs (footage),
6. Pipe footage requiring flushing,
7. Inlets needing cleaning by LR and county,
8. Outlet ditches needing cleaning (footage) by LR and county,
9. Ditches needing repairs or material placement, and

10. Pipe survey needs per year by assistant manager section.

Guide Rail

1. Total amount of guide rail not within current standard (location);
2. Amount of guide rail requiring repair (location);
3. Amount of guide rail requiring replacement or updating (location);
4. Amount of guide rail for suggested removal (location);
5. End treatments needing repair;
6. End treatments needing replacement or updating;
7. Costs for treatments, replacements, and updating;
8. Percentage of "candidate for removal" in the system; and
9. Inventory of needs by guide rail type.

SURVEY IMPLEMENTATION

To effectively implement the proposed guide rail and drainage inventories and condition surveys, several steps were considered important:

1. Review by the existing STAMPP task force and upper department management for concurrence on proposed inventory and condition surveys. This was done in December 1984.
2. Development of condition survey manuals for each survey. Draft condition survey manuals were developed in early 1985 for immediate use and preliminary evaluation of survey techniques.
3. Pilot surveys to assess adequacy of survey forms and developed condition survey manuals were conducted in April and May 1985 on a sample of all state roadway classifications in York County, Pennsylvania, and changes were subsequently made to the condition survey manuals (3,4). The pilot survey consisted of preliminary condition surveys of guide rail and drainage conditions on approximately 60 mi of Legislative Routes, conducted by an in-house survey team, whose results were compared with those obtained independently on the same roadway sections by a separate quality assurance survey team. For both surveys, a one-to-one agreement was obtained for the condition items being evaluated within ±1 deviation in excess of 90 percent of the time (Figures 3 and 4).
4. Develop a training program for survey personnel. Training was given to department personnel coordinating conduct of the surveys and to all survey personnel in May 1985 for the guide rail condition survey. Training for the drainage condition survey has been developed and is to be given in August and September 1985.
5. Development of a quality assurance (QA) plan to monitor survey effectiveness and accuracy. This plan will be developed by the Roadway Management Division of the department's Bureau of Bridge and Roadway Technology, whose responsibility it will be to conduct survey QA.
6. Develop appropriate systems needs for use directly on the mainframe computer, with modifications to the STAMPP programs made as appropriate.
7. Conduct surveys and perform required QA.
8. Interface the guide rail and drainage surveys with STAMPP and include in the RIDB.

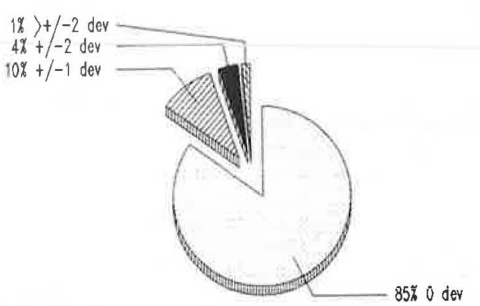


FIGURE 3 Guide rail survey—total deviation from quality assurance.

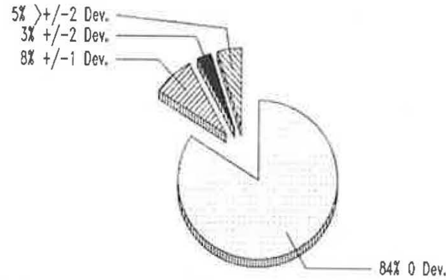


FIGURE 4 Drainage survey—total deviation from quality assurance.

The Pennsylvania Department of Transportation is committed to the use and implementation of the surveys discussed in this paper and described more completely in the condition survey manuals. By properly using the information obtained from conducting these surveys, the department will be in a better position to cost-effectively manage state tax revenues for the construction, maintenance, and general operation of the 43,000-mi state roadway system.

The department stands ready to share its survey systems with other interested governmental agencies that wish to adopt similar management tools for their roadway systems, but cautions that conditions, extents and severities, and treatments and associated costs contained herein have been selected specifically for Pennsylvania. Other systems may have to be modified accordingly.

By the end of this year, the department will have more experience with the operation of the guide rail and drainage condition surveys because the initial surveys will have been performed. Again, the department is most willing to share the results of these initial surveys with interested parties.

ACKNOWLEDGMENTS

This effort was made under the auspices of the Honorable Thomas D. Larson, Secretary, Pennsylvania Department of Transportation, with direction and review provided by the department's STAMPP task force. Appreciation is given to the many people who provided input to the surveys' development, the bureau directors and district engineers who provided the personnel to serve on the Guide Rail and Drainage Task Force, and particularly Thomas Ertwine for the clerical support necessary to complete the survey manuals.

Special appreciation and acknowledgment are given to the Guide Rail and Drainage Task Force members who, in addition to the author, were Louis Schultz, Center for Program Development and Management; Charles Wilson, Bureau of Maintenance and Operations; Paul McWilliams, Engineering District 1-0;

Ronald Stahl, Maintenance District 3-5; Atul Kantawala, Engineering District 11-0; Raymond Radomski, Maintenance Districts 4-5 and 4-7; and Ricardo Robbibaro, Maintenance District 11-1.

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This paper was prepared for internal use by the Pennsylvania Department of Transportation. The contents of this paper reflect the views of the Guide Rail and Drainage Task Force, which is solely responsible for the facts and the data presented herein. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation, nor does this paper constitute a standard, specification, or regulation.

Real-World Impact Conditions for Run-Off-the-Road Accidents

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ABSTRACT

Information is presented on real-world impact conditions for accidents involving roadside objects and features based on in-depth accident data. Of particular interest are the distributions of impact speed and angle for various functional classes. Other considerations relating to impact conditions, such as vehicle orientation at impact, are also discussed. The potential applications of the information presented in this paper are illustrated with two examples, one involving the full-scale crash test matrix and the other involving benefit-cost procedures.

In the design of roadside safety appurtenances and features, it is desirable to have information on the real-world impact conditions to ensure that the appurtenances and features will be effective in serving the intended purpose of mitigating the consequences of impacts by errant vehicles. The impact conditions refer primarily to impact speed and angle, but there are also other considerations, such as the orientation of the vehicle at impact and the area of impact on the vehicle.

To obtain such detailed information, in-depth investigation and reconstruction of accidents are required. Police-level accident data do not provide sufficient detail for this purpose. Also, the accidents have to be either a census or a statistically representative sample in order to establish the distributions of impact conditions. Unfortunately, the costs associated with in-depth accident investigation and reconstruction are high and few programs of this nature have been undertaken.

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Two such data sources (1,2) were identified and analyzed as part of a study conducted for the FHWA on severity measures for roadside objects and features (3). The first source provides data on a statistically representative sample of pole accidents collected over a 20-month period from two study areas: Bexar County (including the city of San Antonio), Texas, and a nine-county area around Lexington, Kentucky. The second source includes a census of accidents involving bridge rails, bridge or parapet ends, and approach guardrails in a 15-county area around San Antonio, Texas, over a 21-month period.

After screening for nonapplicable cases, 472 pole accident cases and 124 bridge accident cases were merged for use in the study. Note that the actual sample size available for analysis is slightly less than 596 because some of the cases have unknown impact speed or angle. Also, the pole accident cases are weighted in accordance with the statistical sampling scheme. The results of the analysis are summarized in this paper, followed by discussions of two example applications of the information under real-world accident conditions for roadside objects and features.

REAL-WORLD IMPACT CONDITIONS

For the purpose of this paper, the impact conditions are defined by impact speed for point objects (e.g., pole supports) and by both impact speed and angle for longitudinal objects (e.g., guardrails and median barriers). The emphasis of this paper is on the distributions of impact speed and impact angle. However, there are other considerations relating to impact conditions, such as the orientation of the vehicle at impact, the area of impact on the vehicle, and postimpact trajectory of the vehicle. Brief discussions of these other considerations will also be presented.

Impact Speed and Angle Distributions

Using the in-depth accident data from the two previously mentioned sources, the distributions of impact speed and angle are first determined individually (i.e., univariate distributions). A number of theoretical distributions, such as normal, exponential, and negative binomial, were fitted to the data and it was found that a gamma function provides the best fit for both univariate impact speed and impact angle distributions. Mathematically, the gamma distribution function is expressed as

$$c(x_i) = \int_0^{x_i} \{1/[\Gamma(\alpha)\ \beta^\alpha]\}\ t^{\alpha-1}E^{-t/\beta}dt$$

where

x_i = impact speed or angle,

$c(x_i)$ = cumulative probability of x,

t = dummy variable for integration, and

α,β = estimated coefficients.

Note that the gamma function is uniquely defined by the two coefficients, α and β. The cumulative gamma distribution functions for impact speed and angle for the combined data are graphically shown in Figures 1 and 2, respectively.

The process involved in determining these distributions is briefly described as follows. The empirical cumulative distribution function for impact speed or angle based on the observed data is first determined:

$c(x_i)$ = Number of accidents with $x \leq x_i$/Total number of accidents

where x_i is impact speed or angle and $c(x_i)$ is cumulative probability for x_i.

Different distribution functions are then fitted to the empirical distribution function using nonlinear least square regression. The gamma function is found to provide the best fit to the data and is therefore selected. The empirical cumulative percentages are also shown in Figures 1 and 2, and it is evident that a good fit is provided by the gamma distribution.

Because the impact conditions for longitudinal objects are defined by both impact speed and angle, it is necessary to determine the joint distribution for impact speed and angle. The actual data are arbitrarily divided into a 6 x 6 matrix and various known joint (bivariate) distributions are fitted to the data with little success. This is not surprising because the univariate impact speed and angle distributions are best estimated by gamma functions and there is no known means of mathematically expressing a joint gamma distribution.

The alternative is to assume that the impact speed and angle distributions are independent of each other so that the cell probability is simply the product of their marginal probabilities. The concern is of course with the validity of the independency assumption.

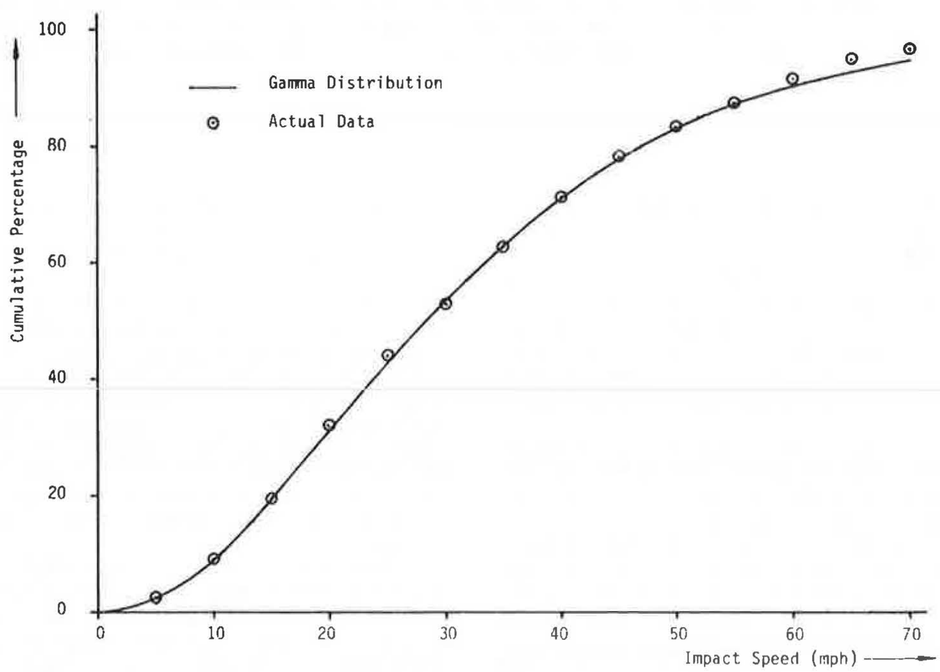

FIGURE 1 Impact speed distribution for combined data.

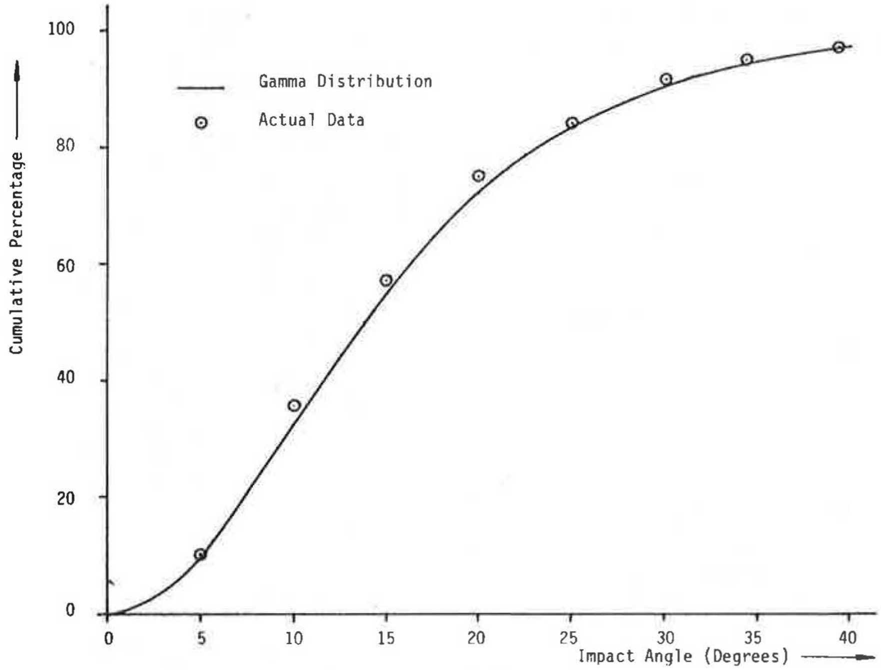

FIGURE 2 Impact angle distribution for combined data.

The data indicate that there is a weak negative correlation (-0.153) between impact speed and angle (i.e., higher impact speeds are associated with slightly lower impact angles). However, the correlation is so weak that any error introduced would likely be minor. A chi-square goodness-of-fit test was used to evaluate this hypothesis and the results indicate a reasonably good fit between the expected and the observed values, as indicated by the data in Table 1. In other words, it may be argued that the errors introduced by the independency assumption are fairly minor and acceptable for estimation purposes.

It should be pointed out that the goodness-of-fit test is very sensitive to the outlying cells (i.e., those cells with either very low or very high impact speeds or angles). Variations in the intervals of the matrix in these outlying areas could alter the results of the goodness-of-fit test. However, the probabilities associated with the outlying cells are very low and the errors introduced would therefore be relatively small.

It should be borne in mind that the impact speed and angle distributions are influenced by various roadway, roadside, and traffic characteristics. It

TABLE 1 Results of Goodness-of-Fit Test

Encroachment Speed (mph) \ Encroachment Angle (°)	≤ 5	6 - 10	11 - 15	16 - 20	21 - 30	> 30	Total
0 - 10.0	0.0088 (4.4) / 0.004 (2)	0.0206 (10.03) / 0.018 (9)	0.0204 (10.2) / 0.022 (11)	0.0155 (7.8) / 0.022 (11)	0.0168 (8.4) / 0.014 (7)	0.0084 (4.2) / 0.008 (4)	0.0905 (45.3) / 0.008 (44)
10.1 -.20.0	0.0216 (10.8) / 0.014 (7)	0.0505 (25.3) / 0.048 (24)	0.0502 (25.1) / 0.034 (17)	0.0381 (19.1) / 0.040 (20)	0.0412 (20.6) / 0.052 (26)	0.0206 (10.3) / 0.032 (16)	0.2222 (111.1) / 0.220 (110)
20.1 - 30.0	0.0219 (11.0) / 0.020 (10)	0.0514 (25.7) / 0.058 (29)	0.0511 (25.6) / 0.052 (26)	0.0388 (19.4) / 0.038 (19)	0.0419 (21.0) / 0.022 (11)	0.0210 (10.5) / 0.016 (8)	0.2261 (113.1) / 0.206 (103)
30.1 - 40.0	0.0169 (8.5) / 0.012 (6)	0.0396 (19.8) / 0.054 (27)	0.0394 (19.7) / 0.040 (20)	0.0299 (15.0) / 0.026 (13)	0.0323 (16.2) / 0.038 (19)	0.0162 (8.1) / 0.014 (7)	0.1743 (87.2) / 0.184 (92)
40.1 - 50.0	0.0114 (5.7) / 0.016 (8)	0.0267 (13.4) / 0.030 (15)	0.0265 (13.3) / 0.038 (19)	0.0201 (10.1) / 0.016 (8)	0.0218 (10.9) / 0.018 (9)	0.0109 (5.5) / 0.008 (4)	0.1174 (58.7) / 0.126 (63)
> 50	0.0164 (8.2) / 0.032 (16)	0.0386 (19.3) / 0.052 (26)	0.0383 (19.2) / 0.024 (12)	0.0291 (14.6) / 0.032 (16)	0.0314 (15.7) / 0.026 (13)	0.0157 (7.9) / 0.010 (5)	0.1695 (84.8) / 0.176 (88)
Total	0.0970 (48.5) / 0.098 (49)	0.2274 (113.7) / 0.260 (130)	0.2259 (113.0) / 0.174 (87)	0.1715 (85.8) / 0.174 (87)	0.1854 (92.7) / 0.170 (85)	0.0928 (46.4) / 0.088 (44)	1.0000 (500) / 1.000 (500)

Legend: Expected Percentage (Expected Freq.) / Observed Percentage (Observed Freq.)

Chi Square = 38.2
Degree of Freedom = 31
pval = 0.175 (Reasonable Fit)
Correlation = -0.153

TABLE 2 Coefficients of Gamma Distribution Functions for Speed and Angle by Functional Class

Functional Class	Impact Speed α	Impact Speed β	Impact Angle α	Impact Angle β
Freeway	5.878	9.789	2.560	6.037
Urban arterial	3.293	7.687	2.241	6.992
Urban collector/local	2.940	7.061	3.319	4.973
Rural arterial	2.367	15.817	1.715	8.749
Rural collector/local	4.165	6.986	1.884	8.172
Combined (all data)	2.542	12.693	2.482	6.393

Note: Gamma distribution function $c(x_i) = \int_0^{x_i} \left\{1/[\Gamma(\alpha)\beta^\alpha]\right\} t^{\alpha-1} e^{-t/\beta} dt$

would not be possible to account for all of these factors, so highway type is used as a gross surrogate measure for all such characteristics.

The data are stratified by functional class and the impact speed and angle distributions are determined for each of the functional classes. The sample sizes for some of the functional classes are too small and thus these classes are grouped together for analysis purpose (e.g., major and minor arterials and collectors and local streets). Also, the sample size for rural freeways is too small for any meaningful analysis. It was therefore decided that the impact speed and angle distributions for rural freeways would be approximated by those of urban freeways and expressways. Thus only five functional classes are included in the analysis.

Functional Class	Sample Size
Freeway and expressway	191
Urban arterial	148
Urban collector or local	134
Rural arterial	65
Rural collector or local	58
Total	596

Given that the combined data (i.e., all functional classes combined) are best fitted by the gamma distribution, it is logical to assume that the gamma distribution would also apply to the individual functional classes. On the basis of this assumption, the impact speed and angle distributions for the individual functional classes are estimated. The fits for the individual functional classes are, as expected, not as good as that for the combined data because of the smaller sample sizes. This is the reason for making the assumption that the gamma distribution function applies to the individual functional classes.

Table 2 gives a summary of the coefficients of the univariate gamma distribution functions for impact speed and angle for the five functional classes and the combined data. The probabilities of various ranges of impact speed and angle for the five functional classes and the combined data are given in Tables 3 and 4, respectively.

Again, assuming that the independency assumption is valid for the individual functional classes as well as for the entire data set, the cell probabilities for each of the five functional classes can be computed easily (Tables 5-10).

Other Considerations

There are other considerations, in addition to impact speed and angle, that relate to real-world impact conditions. Even though the emphasis of this paper is on impact speed and angle, these other considerations should also be taken into account in defining the real-world impact conditions for accidents involving roadside objects and features. These other considerations are also addressed in the two studies (1,2) used for determining the impact speed and angle distributions, highlights of which are summarized next.

For pole impacts, the front of the vehicle is the most frequent area of impact (72.9 percent). Impacts with the back of the vehicle are extremely rare, accounting for only 1.9 percent of pole accidents. Side impacts are involved in approximately 25 percent of pole accidents (13 percent for the right side and 12.2 percent for the left side), and they result in much higher injury severity than frontal or rear impacts.

For impacts with longitudinal barriers, more than

TABLE 3 Impact Speed Probability Distribution by Functional Class

Impact Speed (mph)	Freeway	Urban Arterial	Urban Collector/ Local	Rural Arterial	Rural Collector/ Local	Combined
≤10	0.0020	0.1030	0.1810	0.0763	0.0468	0.0904
11-20	0.0507	0.3086	0.3718	0.1829	0.2439	0.2222
21-30	0.1548	0.2796	0.2529	0.1983	0.2989	0.2261
31-40	0.2208	0.1678	0.1203	0.1681	0.2115	0.1743
41-50	0.2100	0.0823	0.0481	0.1264	0.1136	0.1174
51-60	0.1560	0.0358	0.0174	0.0886	0.0518	0.0730
>60	0.2057	0.0229	0.0086	0.1594	0.0335	0.0965

TABLE 4 Impact Angle Probability Distribution by Functional Class

Impact Angle (degrees)	Freeway	Urban Arterial	Urban Collector/ Local	Rural Arterial	Rural Collector/ Local	Combined
≤5	0.0974	0.1155	0.0526	0.1723	0.1491	0.0970
6-10	0.2351	0.2313	0.2046	0.2354	0.2330	0.2274
11-15	0.2322	0.2169	0.2484	0.1936	0.2011	0.2258
16-20	0.1731	0.1623	0.2007	0.1397	0.1477	0.1716
21-25	0.1125	0.1089	0.1326	0.0946	0.1003	0.1145
26-30	0.0675	0.0685	0.0777	0.0618	0.0651	0.0708
>30	0.0822	0.0965	0.0833	0.1026	0.1037	0.0928

TABLE 5 Impact Speed and Angle Distributions for Freeway

Impact Speed (mph)	Impact Angle (degrees)							
	≤5	6-10	11-15	16-20	21-25	26-30	>30	Total
≤10	0.0002	0.0005	0.0005	0.0003	0.0002	0.0001	0.0002	0.0020
11-20	0.0049	0.0119	0.0118	0.0088	0.0057	0.0034	0.0042	0.0507
21-30	0.0151	0.0364	0.0359	0.0268	0.0174	0.0104	0.0127	0.1548
31-40	0.0215	0.0519	0.0513	0.0382	0.0248	0.0149	0.0181	0.2208
41-50	0.0205	0.0494	0.0488	0.0364	0.0236	0.0142	0.0173	0.2100
51-60	0.0152	0.0367	0.0362	0.0270	0.0176	0.0105	0.0128	0.1560
>60	0.0200	0.0484	0.0478	0.0356	0.0231	0.0139	0.0169	0.2057
Total	0.0974	0.2351	0.2322	0.1731	0.1125	0.0675	0.8222	1.0000

TABLE 6 Impact Speed and Angle Distributions for Urban Arterial

Impact Speed (mph)	Impact Angle (degrees)							
	≤5	6-10	11-15	16-20	21-25	26-30	>30	Total
≤10	0.0119	0.0238	0.0223	0.0167	0.0112	0.0071	0.0099	0.1030
11-20	0.0356	0.0714	0.0669	0.0501	0.0336	0.0211	0.0298	0.3086
21-30	0.0323	0.0647	0.0606	0.0454	0.0304	0.0192	0.0270	0.2796
31-40	0.0194	0.0388	0.0364	0.0272	0.0183	0.0115	0.0162	0.1678
41-50	0.0095	0.0190	0.0179	0.0134	0.0090	0.0056	0.0079	0.0823
51-60	0.0041	0.0083	0.0078	0.0058	0.0039	0.0025	0.0035	0.0358
>60	0.0026	0.0053	0.0050	0.0037	0.0025	0.0016	0.0022	0.0229
Total	0.1155	0.2313	0.2169	0.1623	0.1089	0.0685	0.0965	1.0000

TABLE 7 Impact Speed and Angle Distributions for Urban Collector/Local

Impact Speed (mph)	Impact Angle (degrees)							
	≤5	6-10	11-15	16-20	21-25	26-30	>30	Total
≤10	0.0095	0.0370	0.0450	0.0363	0.0240	0.0141	0.0151	0.1810
11-20	0.0196	0.0761	0.0924	0.0746	0.0493	0.0289	0.0310	0.3718
21-30	0.0133	0.0517	0.0628	0.0508	0.0335	0.0197	0.0211	0.2529
31-40	0.0063	0.0246	0.0299	0.0241	0.0160	0.0093	0.0100	0.1203
41-50	0.0025	0.0098	0.0119	0.0097	0.0064	0.0037	0.0040	0.0481
51-60	0.0009	0.0036	0.0043	0.0035	0.0023	0.0014	0.0014	0.0174
>60	0.0005	0.0018	0.0021	0.0017	0.0011	0.0007	0.0007	0.0086
Total	0.0526	0.2046	0.2484	0.2007	0.1326	0.0777	0.0833	1.0001

TABLE 8 Impact Speed and Angle Distributions for Rural Arterial

Impact Speed (mph)	Impact Angle (degrees)							
	≤5	6-10	11-15	16-20	21-25	26-30	>30	Total
≤10	0.0131	0.0180	0.0148	0.0107	0.0072	0.0047	0.0078	0.0763
11-20	0.0315	0.0431	0.0354	0.0256	0.0173	0.0113	0.0188	0.1829
21-30	0.0342	0.0467	0.0384	0.0277	0.0188	0.0123	0.0203	0.1983
31-40	0.0290	0.0396	0.0325	0.0235	0.0159	0.0104	0.0172	0.1681
41-50	0.0218	0.0298	0.0245	0.0177	0.0120	0.0078	0.0130	0.1264
51-60	0.0153	0.0209	0.0172	0.0124	0.0084	0.0055	0.0091	0.0886
>60	0.0275	0.0375	0.0309	0.0223	0.0151	0.0099	0.0164	0.1594
Total	0.1723	0.2354	0.1936	0.1397	0.0946	0.0618	0.1026	1.0000

TABLE 9 Impact Speed and Angle Distributions for Rural Collector/Local

Impact Speed (mph)	Impact Angle (degrees)							
	≤5	6-10	11-15	16-20	21-25	26-30	>30	Total
≤10	0.0070	0.0109	0.0094	0.0069	0.0047	0.0030	0.0049	0.0468
11-20	0.0364	0.0568	0.0490	0.0360	0.0245	0.0159	0.0253	0.2439
21-30	0.0446	0.0696	0.0601	0.0441	0.0300	0.0195	0.0310	0.2989
31-40	0.0315	0.0493	0.0425	0.0312	0.0212	0.0138	0.0219	0.2115
41-50	0.0169	0.0265	0.0228	0.0168	0.0114	0.0074	0.0118	0.1136
51-60	0.0077	0.0121	0.0104	0.0077	0.0052	0.0034	0.0054	0.0518
>60	0.0050	0.0078	0.0067	0.0049	0.0034	0.0022	0.0035	0.0335
Total	0.1491	0.2330	0.2011	0.1477	0.1003	0.0651	0.1037	1.0000

TABLE 10 Impact Speed and Angle Distributions for Combined Data

Impact Speed (mph)	Impact Angle (degrees) ≤5	6-10	11-15	16-20	21-25	26-30	>30	Total
≤10	0.0088	0.0206	0.0204	0.0155	0.0104	0.0064	0.0084	0.0904
11-20	0.0216	0.0505	0.0502	0.0381	0.0254	0.0157	0.0206	0.2222
21-30	0.0219	0.0514	0.0511	0.0388	0.0259	0.0160	0.0210	0.2261
31-40	0.0169	0.0396	0.0394	0.0299	0.0200	0.0123	0.0162	0.1743
41-50	0.0114	0.0267	0.0265	0.0201	0.0134	0.0083	0.0109	0.1174
51-60	0.0071	0.0166	0.0165	0.0125	0.0084	0.0052	0.0068	0.0730
>60	0.0085	0.0200	0.0198	0.0151	0.0101	0.0062	0.0082	0.0879
Total	0.0970	0.2274	0.2258	0.1716	0.1145	0.0708	0.0928	0.9913

three-quarters (77.4 percent) of accidents involve more than one impact, and half of these accidents involve three or more impacts. The injury severity of the accidents increases with the number of impacts. This clearly illustrates the importance of the postimpact trajectory of the vehicles.

For the first barrier impact, only slightly more than half (51.2 percent) of the vehicles are tracking at impact whereas more than one-quarter (26.0 percent) of the vehicles are yawing at more than 30 degrees at impact. For subsequent barrier impacts, the impact speeds are lower, but the impact angles are higher than for first barrier impacts. The percentage of vehicles yawing at greater than 30 degrees increases from 26 percent for the first barrier impact to more than 40 percent for subsequent barrier impacts. Similarly, the percentage of side and back impacts doubles from less than 25 percent to more than 50 percent. This indicates that, for subsequent impacts, the vehicle trajectories are more abrupt although the impact speeds are lower.

Discussion

Caution should be exercised in using the results presented in this paper. It should be recognized that there are limitations associated with the data sources and the analyses. The results presented should be viewed only as an intermediate step in the effort to better define the distributions of impact conditions based on the best data currently available. As new and better data become available, the distributions should be updated and improved as appropriate. A brief discussion of some of the limitations associated with the two data sources used in the study follows.

First, the impact conditions refer only to reported accidents. It is well known that some accidents are not brought to the attention of law enforcement agencies or are not reported by the police for a variety of reasons. The impact conditions of these unreported accidents could be significantly different from those of reported accidents. For example, the majority of these unreported accidents might be at low impact speeds and angles, which would drastically alter the distributions. Unfortunately, the extent of such unreported accidents is not known and it is not possible to estimate the effects of such unreported accidents on the distributions of impact conditions as presented in this paper.

Second, accidents involving pole supports and appurtenances at bridge sites are not necessarily representative of all run-off-the-road accidents. For example, pole supports and appurtenances at bridge sites are likely to be placed relatively close to the roadway. This reduced extent of lateral offset may have some, albeit unknown, effect on the distributions of impact conditions. Similarly, the sites where the data were collected in the two studies are not necessarily geographically representative.

Third, functional class is used as a gross surrogate measure for the various roadway, roadside, and traffic characteristics that could influence the distributions of the impact conditions. Some examples of such influencing characteristics are lane and shoulder width, horizontal and vertical alignment, lateral offset, roadside slope, and traffic volume and speed. It would be desirable to evaluate the effect of each characteristic individually, but the sample size is too small for such detailed analysis.

EXAMPLE APPLICATIONS

Information on real-world impact conditions can be helpful in the design and evaluation of roadside safety appurtenances and features. Two example applications are illustrated. The first example is a comparison of the full-scale crash test matrix currently in use and real-world impact conditions. The second example involves the use of the information in a benefit-cost model for evaluating highway safety improvements.

Full-Scale Crash Test Matrix

The full-scale crash test matrix for performance evaluation of roadside safety appurtenances has evolved over the years (4-7) with little consideration given to real-world impact conditions. It would be interesting to see how the full-scale crash test matrix currently in use would compare with real-world impact conditions.

Tables 11 and 12 are reproduced from Tables 3 and 4 of NCHRP Report 230 (4) and give the current recommended minimum and supplemental full-scale crash test matrix for roadside safety appurtenances, respectively. Tests that involve large vehicles are excluded from this comparison because the accident data pertain only to passenger vehicles.

The comparisons are divided into two parts: those for point objects, such as breakaway or yielding supports, crash cushions, and barrier ends, in which only impact speed is considered; and those for longitudinal barriers in which both impact speed and impact angle are included.

For point objects, the crash test speeds are either 20 or 60 mph except for one supplemental test at 40 mph for yielding or base-bending supports. Table 13 gives a summary of the percentage of impacting vehicles with speeds of up to 20 mph, greater than 40 mph, and greater than 60 mph for various highway types. It is evident from the table that there are major differences in speed distributions among the various highway types.

As may be expected, freeways have the highest impact speed distribution, followed by rural ar-

TABLE 11 Crash Test Conditions for Minimum Matrix (4)

Appurtenance	Test Designation	Vehicle Type[d]	Impact Speed (mph)	Impact Angle[e] (deg)	Target Impact Severity[f] (ft-kips)	Impact Point[g]	Evaluation Criteria[h]
Longitudinal Barrier[a] Length-of-Need	10	4500S	60	25[i]	$97^{-9,+17}$	For post and beam systems, midway between posts in span contianing railing splice	A,D,E,H,I
	11	2250S	60	15[i]	$18^{-2,+3}$	For post and beam systems, vehicle should contact railing splice	A,D,E,F,(G),H,I
	12	1800S	60	15[i]	$14^{-2,+2}$	For post and beam system, vehicle should contact railing splice	A,D,E,F,(G),H,I
Transition	30	4500S	60	25[i]	$97^{-9,+17}$	15 ft upstream from second system	A,D,E,H,I
Terminal	40	4500S	60	25[i]	$97^{-9,+17}$	At beginning of lenth-of-need	A,D,E,H,I
	41	4500S	60	0[i]	$541^{-53,+94}$	Center nose of device	C,D,E,F,(G),H,J
	42	2250S	60	15[i]	$18^{-2,+3}$	Midway between nose and lenth-of-need	C,D,E,F,(G),H,I,J
	43	2250S	60[o]	0[i]	$270^{-26,+47}$	Offset 1.25 ft from center nose of device	C,D,E,F,(G),H,J
	44	1800S	60	15[i]	$14^{-2,+2}$	Midway between nose and length-of-need	C,D,E,F,(G),H,I,J
	45	1800S	60[o]	0[i]	$216^{-21,+37}$	Offset 1.25 ft from center nose of device	C,D,E,F,(G),H,J
Crash Cushion[b]	50	4500S	60	0[j]	$541^{-53,+94}$	Center nose of device	C,D,E,F,(G),H,J
	51	2250S	60[o]	0[j]	$270^{-26,+47}$	Center nose of device	C,D,E,F,(G),H,J
	52	1800S	60[o]	0[j]	$216^{-21,+37}$	Center nose of device	C,D,E,F,(G),H,J
	53[l]	4500S	60	20[j]	$63^{-6,+11}$	Alongside, midlength	C,D,E,H,I,J
	54	4500S	60	10-15[j]	$541^{-53,+94}$	0-3 ft offset from center of nose of device	C,D,E,F,(G),H,J
Breakaway or Yielding Support[c]	60	2250S	20	[k]	$30^{-4,+4}$	Center of bumper[m,n]	B,D,E, F,(G),H,J
	61	2250S	60	[k]	$270^{-26,+47}$	At quarter point of bumper[n]	B,D,E,F,(G),H,J
	62	1800S	20	[k]	$24^{-3,+3}$	Center of bumper[m,n]	B,D,E,F,(G),H,J
	63	1800S	60	[k]	$216^{-21,+37}$	At quarter point of bumper[n]	B,D,E,F,(G),H,J

(a) Includes guardrail, bridgerail, median and construction barriers.
(b) Includes devices such as water cells, sand containers, steel drums, etc.
(c) Includes sign, luminaire, and signal box supports.
(d) See Table 2 for description.
(e) + 2 degrees
(f) IS = $1/2\ m\ (v \sin \theta)^2$ where m is vehicle test inertial mass, slugs; v is impact speed, fps; and θ is impact angle for redirectional impacts or 90 deg for frontal impacts, deg.
(g) Point on appurtenance where initial vehicle contact is made.
(h) See Table 6 for performance evaluation factors; ( ) denotes supplementary status.
(i) From centerline of highway.
(j) From line of symmetry of device.
(k) Test article shall be oriented with respect to the vehicle approach path to a position that will theoretically produce the maximum vehicle velocity change; the orientation shall be consistent with reasonably expected traffic situations.
(l) See Commentary, Chapter 4 Test Conditions for devices which are not intended to redirect vehicle when impacted on the side of the device.
(m) For base bending devices, the impact point should be at the quarter point of the bumper.
(n) For multiple supports, align vehicle so that the maximum number of supports are contacted assuming the vehicle departs from the highway with an angle from 0 to 30 deg.
(o) For devices that produce fairly constant or slowly varying vehicle accelerations; an additional test at 20 mph (32 kph) is recommended for staged devices, those devices that produce a sequence of individual vehicle deceleration pulses (i.e. "lumpy" device) and/or those devices comprised of massive components that are displaced during dynamic performance (see commentary).

terials, and urban collectors and local streets have the lowest. The percentage of impacting vehicles with speeds of up to 20 mph ranges from a low of 5 percent for freeways to a high of 30.9 percent for urban arterials and 37.2 percent for urban collectors and local streets. Freeways and rural arterials have substantial percentages of accidents with impact speeds above 60 mph (20.6 and 15.9 percent, respectively), and those for the other highway types are quite low, ranging from 0.9 to 3.4 percent. The percentages of impact speeds above 40 mph are again highest for freeways (57.2 percent) and lowest for urban collectors and local streets (7.4 percent).

For longitudinal barriers, the two major test conditions are at impact speeds of 60 mph with impact angles at 15 or 25 degrees. Table 14 gives a summary of the percentages of accidents with impact conditions that exceed one or both of these criteria. It is interesting to note that, unlike those of impact speed, the distributions of impact angles vary little among the various highway types. This supports the assumption of independency between impact speed and angle. The 15-degree impact angle is slightly above the median (55th percentile) and the 25-degree impact angle represents roughly the 85th percentile.

When both impact speed and angle criteria are taken into consideration, the percentage of accidents that exceed both criteria is actually quite small. For instance, even for freeways, only 3 percent of the accidents have impact speeds of more than 60 mph and impact angles greater than 25 degrees, and 9 percent of the accidents have impact speeds of more than 60 mph and impact angles greater that 15 degrees. This suggests that the current full-scale crash test conditions for longitudinal barriers are actually rather stringent.

The results of the comparison of the crash test matrix and real-world impact conditions point to the desirability of the multiple service level concept (8). Currently, appurtenances are designed under one set of test conditions regardless of the application. As a result, appurtenances may be underde-

TABLE 12 Typical Supolementary Crash Test Conditions (4)

Appurtenance	Test Designation	Vehicle Type[d]	Impact Speed (mph)	Impact Angle[e] (deg)	Target Impact Severity[f] (ft-kips)	Impact Point[g]	Evaluation Criteria[h]
Longitudinal Barrier[a] Length-of-Need	S13	1800S	60	20[i]	$25^{-2,+4}$	For post and beam system, at mid span.	A,D,E,H,I
	S14[p]	4500S	60	15[i]	$36^{-4,+6}$	For post and beam system, vehicle should contact railing splice.	A,D,E,H,I
	S15[q]	40,000P	60	15[i]	$237^{-23,+41}$	For post and beam system, vehicle should contact railing splice.	A,D,E
	S16[r]	20,000P	45	7[i]	$14^{-2,+3}$	For post and beam system, vehicle should contact railing splice.	A,D,E
	S17[r]	20,000P	50	15[i]	$77^{-9,+16}$	For post and beam system, vehicle should contact railing splice.	A,D,E
	S18[r]	20,000P	60	15[i]	$111^{-11,+19}$	For post and beam system, vehicle should contact railing splice.	A,D,E
	S19	32,000P	60	15[i]	$97^{-9,+17}$	For post and beam system, vehicle should contact railing splice.	A,D,E
	S20[s]	80,000A	50	15[i]	[t]	For post and beam sytem, vehicle should contact railing splice.	A,D[s]
	S21[s]	80,000F	50	15[i]	[t]	For post and beam system, vehicle should contact railing splice.	A,D[s]
Transition	S31[p]	4500S	60	15[i]	$36^{-4,+6}$	15 ft upstream from second system	A,D,E,H
	S32[q]	40,000P	60	15[i]	$237^{-23,+41}$	15 ft upstream from second system	A,D,E
Terminals	S46[p]	4500S	60	15[i]	$36^{-4,+6}$	At beginning of length-of-need	A,D,E,H
	S47[q]	40,000P	60	15[i]	$237^{-27,+41}$	At beginning of length-of-need	A,D,E
Crash Cushion[b]	(NONE)						
Breakaway or Yielding Support[c]	S64	1800S	40	[k]	$96^{-14,+15}$	Center of bumper[m,n]	B,D,E,F,(G),H,J

For notes (a) through (o), see Table 3.

(p) Multiple Service Level 1 structural adequacy test; see Commentary, Chapter 4.

(q) Multiple Service Level 3 structural adequacy test; see Commentary, Chapter 4.

(r) Utility bus stability test; S16 for Multiple Service Level 1 appurtenance; S17 for Multiple Service Level 2 appurtenance; S18 specified for Multiple Service Level 3 appurtenance.

(s) Cargo/debris containment test; vehicle, cargo, and debris shall be contained on traffic side of barrier.

(t) Not appropriate for articulated vehicles.

TABLE 13 Percentage of Accidents by Impact Speed and Highway Type for Point Objects

Highway Type	Percentage at ≤20 mph	>40 mph	>60 mph
Freeway	5.1	57.2	20.6
Urban arterial	30.9	14.1	2.3
Urban collector/local	37.2	7.4	0.9
Rural arterial	18.3	37.4	15.9
Rural collector/local	24.4	19.9	3.4
Combined	22.2	28.7	9.7

signed for certain conditions and overdesigned for others. It may be desirable to establish different performance standards or guidelines for use with different applications.

One possible approach is to select the test conditions at a given percentile of real-world impact conditions. Table 15 gives impact speeds, rounded off to the nearest 5 mph, for the various highway types at different percentiles. It is evident from the data in the table that, for a given percentile, the impact speed varies greatly among the various highway types. For example, the current test speed of 60 mph corresponds to the 90th percentile impact speed for all highway types. However, the 90th percentile impact speeds for individual highway types range from a low of 40 mph for urban collectors and local streets to a high of 70 mph for freeways and rural arterials.

TABLE 14 Percentage of Accidents by Impact Speed, Angle, and Highway Type for Longitudinal Barriers

Highway Type	>60 mph	>15°	>25°	>60 mph and >15°	>60 mph and >25°
Freeway	20.6	43.5	15.0	8.95	3.08
Urban arterial	2.3	43.6	16.5	1.00	0.39
Urban collector/local	0.9	49.4	16.1	0.42	0.14
Rural arterial	15.9	39.9	16.4	6.36	2.62
Rural collector/local	3.4	41.7	16.9	1.40	0.35
Combined	9.7	45.0	16.4	4.34	1.58

TABLE 15 Percentile Impact Speed by Highway Type

Highway Type	Percentile Impact Speed (mph) 85th	90th	95th
Freeway	65	70	80
Urban arterial	40	45	50
Urban collector/local	35	40	45
Rural arterial	60	70	80
Rural collector/local	45	50	60
Combined	50	60	70

An appurtenance designed for freeway use could be overdesigned for applications on urban streets and vice versa. It appears logical and perhaps more cost-effective to have different performance standards or guidelines for testing appurtenances intended for different applications. For example, a lower test speed of 45 mph may be sufficient for guardrails designed for use on urban streets, which might allow for reduced post sizes or increased post spacing. This in turn could result in lower costs for the appurtenances and still allow a reasonable level of performance to be maintained.

Information other than impact speed and angle may also be useful in assessing the crash test matrix. For example, side impacts account for nearly 25 percent of point object accidents with much higher resultant injury severity. It is also known that the breakaway mechanism for pole supports may not function properly in some side impacts. An additional side impact test in the current test matrix may be desirable.

The postimpact trajectory of impacting vehicles and subsequent impacts are other areas of concern about impacts with longitudinal barriers. The potential hazard with postimpact trajectory is recognized in the current testing procedures, and evaluation criteria based on exit speed and angle and redirection into the traffic lanes have been established. Nevertheless, a closer examination of the postimpact trajectory of the vehicle may be desirable.

Little attention has been given to vehicle yawing at impact, and its effect on the performance of appurtenances is virtually unknown except that it increases the probability of nonfrontal impacts. Given the high proportion of nontracking vehicles at impact for reported accidents, it may be desirable to study the effect of vehicle yawing on the performance of appurtenances.

Benefit-Cost Model

Benefit-cost (B-C) procedures are used to determine if the benefits from a safety improvement justify the associated costs and to rank improvements in priority order so as to maximize the benefits for a given funding level. Inputs to the B-C model include the angles at which vehicles depart from the travelway for the determination of the number of expected accidents at a given site, and impact speeds and angles for the estimation of the severity of the accidents, the costs for repairing roadside facilities, and the performance of safety devices.

Accident prediction algorithms are frequently based on an encroachment probability model. The model assumes that inadvertent encroachments are randomly distributed along the roadway and that these errant vehicles travel along a relatively straight path after leaving the travelway. The path of an encroaching vehicle and the probability of an accident are therefore directly related to the angle of encroachment. However, only limited data on the distribution of encroachment angles are available from a few encroachment and special accident studies (9-12) that do not distinguish among encroachment characteristics on different classes of highways.

The severity of accidents involving roadside objects and features is strongly related to the impact speed and, for longitudinal objects, also the angle of impact. Repair costs for roadside appurtenances and the performance of safety devices have been shown to be related to the kinetic energy and lateral momentum of impacting vehicles (4,9,13). The performance of safety devices is especially important when trying to determine the appropriate performance level at a specific site.

Joint impact speed and angle distributions have not been available directly from accident data. A point-mass cornering model has therefore been used to relate impact speed distributions to impact angle distributions. Furthermore, the impact speed data are based on estimates by police officers (11,12), which are highly unreliable.

The impact speed and angle distributions described herein have been incorporated into revised B-C procedures (see paper by Sicking and Ross in this Record) in an effort to improve the accuracy of encroachment probability B-C algorithms. This in turn could provide better estimates of the probability of an accident occurring, the severity of an accident when it does occur, the likelihood that an appurtenance would perform satisfactorily, and the repair cost for the appurtenance. All of the aforementioned probabilities and costs are important to the overall B-C analysis.

Other information related to impact conditions may also contribute to the B-C analysis even though it is not incorporated in the current procedures. For example, vehicle orientation at impact, such as side impacts into pole supports, may have a significant influence on accident severity. These potential effects have not been evaluated, in part because of the lack of information on impact conditions. Some of the information presented in this paper may be suitable for incorporation into the B-C procedures in the future.

SUMMARY

In this paper is presented information on the real-world impact conditions of accidents involving roadside objects and features based on in-depth accident data. Of particular interest are the distributions of impact speed and angle for various functional classes. The potential applications of the information presented herein are illustrated with two examples, one involving the full-scale crash test matrix and the other involving B-C procedures.

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Discussion

J. D. Michie*

The authors address an important topic and attempt to develop impact angles and speeds for vehicles in roadside collisions. Although the authors caution the readers about the limitations of the data, their presentation of findings to four significant figures (i.e., Tables 2-10) suggests that the results are extremely precise. I not only question the inferred precision of the angle and speed models, I also question the representativeness for all roadside collisions of the accident data. The last point is most important because it bears directly on basis assumptions for cost-benefit analyses of roadside safety.

The paper is based on police-reported accident cases that were subsequently investigated and reconstructed. Police-reported accidents represent only about one-third of the 18 million highway accidents that are annually reported to all sources (National Safety Council, 1980-1982 data). Moreover, it has been determined by Galati (1) and by Bryden (NYDOT Proposal for Project 180-1, June 1983) that as few as 10 percent of longitudinal barrier collisions may be reported. For obsolete longitudinal barriers located on older, lower traffic volume roads, the percentage of unreported driveaway collisions is believed to decrease to approximately 60 percent. Thus the data base used by the authors reflects only a part (i.e., 10 to 40 percent) of roadside collisions. This would not be a problem if the reported accident data base were representative of all roadside collisions. Indeed, the authors recognized that the less severe collisions are underrepresented, especially the low-speed and low-angle impacts with longitudinal barriers. Obviously, the models are thus skewed to the more severe impacts. Although I question the validity of the impact speed model, my greatest concern is with the impact angle model. Although a gamma function certainly provides the best fit for the reported accident data set, it is opined that an exponential function (which differs greatly from the gamma function) would have resulted if a more representative sample had been available.

A secondary concern is the generalization of model application from only bridge rail data to longitudinal barriers in general. Bridge rails are peculiar to the longitudinal barriers set in that they are (a) generally located closer to the traveled way and (b) generally more rigid. Barrier offset distance affects the maximum potential angle at which a vehicle can turn into a barrier and attendantly affects the spectrum of impact angles. Barrier rigidness may affect vehicle damage and the number of unreported driveaway collisions. To illustrate the difference between bridge rail and longitudinal barriers, the authors report that 77.4 percent of impacts resulted in one or more subsequent impacts; because bridge rails are rigid and located near the traveled way, they readily redirect the errant vehicles across the highway and often into another bridge rail or fixed object. In contrast, Bryden and Fortuniewicz (see their paper in this Record) showed that multiple impacts occur in only 26 percent of the reported cases. Clearly, bridge rail accident data are not representative of longitudinal barrier collisions, at least with regard to the propensity for secondary impacts.

The data sets used by the authors represent the most complete description of a group of roadside collisions, but the data suffer from (a) lack of exposure information such as traffic volume, operating speed distribution, vehicle types and distribution, and density of roadside features and (b) measurement or estimate of unreported accidents from continuous monitoring techniques (very expensive) to highway damage repair records or periodic photologging of scuff marks on barriers. The approach suggested by Cirillo (2) appears to address these limitations.

The authors are to be commended for addressing a most important aspect of roadside safety. Having accurate speed and angle impact models is crucial to effecting a more rational crash test matrix and providing more realistic cost-benefit analysis programs.

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Authors' Closure

The authors would like to thank J.D. Michie for his thoughtful comments on the paper. We agree with the comments in general but would differ on some of the specific points. First, some cell probabilities, especially those for joint impact speed and angle distributions, are very small and require four decimal places to provide one significant figure. For example, the cell probabilities for impact speed of

10 mph and less on freeways are on the order of 0.0001 to 0.0005 for the various impact angle ranges (see Table 4 of the paper). The use of four decimal places is maintained throughout all the tables for uniformity and does not imply extreme precision.

Second, the authors recognize the importance of exposure and attempt to control for exposure by using highway functional class as a surrogate measure. The sample size is not large enough for more detailed breakdown, as suggested by Michie, to include exposure measures such as traffic volume, operating speed, and vehicle type.

The authors recognize and agree in principle with Michie on the limitations of the accident data used in developing the impact angle and speed distributions. There is no question that a certain percentage of accidents involving roadside objects is not reported to the law enforcement agencies for a variety of reasons. A number of studies, some of which are cited by Michie, attempted to determine the extent of unreported barrier accidents by comparing the number of scuff marks, scrapes, and dents on barriers with reported barrier accidents (first harmful event only). The results vary greatly among the studies, and there is no consistent trend.

The authors have some doubts as to how meaningful and accurate these estimates of unreported accidents are. It is the opinion of the authors that many of these barrier scuff marks, scrapes, and dents are caused by vehicles, such as large trucks, and maintenance and farm equipment that are on the shoulder intentionally and are thus not unreported accidents. Also, damage to barriers can be caused by secondary impacts that would not be identified when only first harmful events are considered.

For instance, in an ongoing study conducted by the Texas Transportation Institute (TTI) for the FHWA, all accident reports for 1982 were manually reviewed for two sections of freeways in San Antonio, Texas, in an effort to identify concrete median barrier (CMB) accidents. It was found that 40 percent of the CMB impacts were not the first harmful event but were subsequent to vehicle-to-vehicle impacts. Also, multiple impacts with the barrier were noted in many of the accidents. A simple comparison of scuff marks, scrapes, and dents on barriers and reported barrier impacts as first harmful events would have incorrectly identified these subsequent impacts as unreported accidents.

This discussion does not imply that there are no unreported barrier accidents but simply that we have pitifully little information about these "unreported accidents." This brings us to a more fundamental concern: whether and how we should account for these unreported accidents in the design and performance evaluation of roadside appurtenances. There is no available information on these unreported accidents and it is unlikely that such data will become available in the foreseeable future. Assumptions and conjectures could be made about the characteristics of these unreported accidents, such as an exponential distribution for impact angles, mentioned by Michie. However, the fact remains that we simply do not know. The authors would argue that it is better and more practical to use available data from reported accidents than to depend on unsubstantiated assumptions and conjectures about unreported accidents. Furthermore, it can be argued that it is better to err on the side of overstating the impact severity because this will generally result in greater use of improved safety features.

The discussions presented in the paper on impact conditions other than speed and angle, such as areas of impact on the vehicle, vehicle yawing at impact, and subsequent impacts, are direct excerpts from the two referenced studies and are included in the paper for information purposes. It is certainly not the authors' intention to suggest that bridge rail accidents are representative of other longitudinal barrier collisions. However, the authors believe that the issues raised with the bridge rail accidents would also apply to other longitudinal barrier accidents, though the magnitude of the problems may be different. For example, subsequent impacts may be more frequent for bridge rail accidents than for other longitudinal barriers as pointed out by Michie, but this should not negate the concern for subsequent impacts.

Another point raised by Michie is the effect of barrier offset distance on the impact angle at which a vehicle strikes an object. The authors agree that the potential for higher impact angles increases as offset distance increases. However, the potential for reduced impact angle (or no contact at all) also increases with greater offset distance because drivers, if in control of steering or braking, or both, will typically try to steer back to the roadway or stop, or both, before striking the object. Indeed, the data reported in the paper suggest that impact angle is somewhat independent of offset distance.

In summary, though the authors differ with Michie's comments on specific points, the comments are well founded and reflect the general lack of available information in this area. The authors recognize the limitations of the materials presented in the paper but hope that the information will be of some utility to researchers in the roadside safety area.

A Low-Maintenance, Energy-Absorbing Bridge Rail

W. LYNN BEASON, T. J. HIRSCH, and JOHN C. CAIN

ABSTRACT

A low-maintenance, energy-absorbing bridge rail has been developed for use in high traffic volume situations where the cost of repairing conventional bridge rails has become prohibitively expensive. The new bridge rail is designed to meet or exceed current bridge rail design guidelines. It incorporates railings and posts made of steel tubing and rubber energy absorbers and is designed to be installed on new or existing standard bridge decks. Results of crash tests show that the bridge rail can smoothly redirect a 4,500-lb (2043-kg) automobile impacting at a velocity of 60 mph (96.6 km/hr) and an angle of 25 degrees and remain in service with no maintenance. If exposed to a more severe impact, the bridge rail may have to be repaired, but the bridge deck will remain undamaged. Finally, the new energy-absorbing rail occupies less bridge deck area than do conventional bridge rails.

Bridge rails currently in use are capable of smoothly redirecting automobiles that strike them. However, virtually all types of bridge rails require some type of repair when they are subjected to moderate to severe impacts. The types of damage normally incurred include damage to the bridge rail, bridge rail posts, and bridge deck. The damage is more prevalent with metal bridge rails, but even concrete parapet bridge rails are susceptible to damage when exposed to severe impacts. In many cases the costs associated with bridge rail repair can be greater than the original installation costs. Repair and maintenance costs can become overwhelming on high-volume, multilane expressways where bridge rails are subjected to a greatly increased risk of impact. There is a need for an alternative bridge rail that can redirect errant automobiles without being damaged.

The research reported in this paper was directed toward development of a low-maintenance, energy-absorbing bridge rail that meets or exceeds current bridge rail design criteria. The bridge rail developed incorporates structural steel tube railing and post members and rubber energy absorbers. Further, the bridge rail is designed to be installed on standard Texas State Department of Highways and Public Transportation (SDHPT) bridge decks. No special deck reinforcement is required. Therefore the bridge rail can be installed on either new or existing bridge decks. This paper is a discussion of the development and testing of the new bridge rail.

DEVELOPMENT OF THE ENERGY-ABSORBING BRIDGE RAIL

The objective of the research presented in this paper was to develop an energy-absorbing bridge rail that conforms to current bridge rail design standards and that can withstand the impact of a 4,500-lb (2043-kg) automobile traveling at a velocity of 60 mph (96.6 km/hr) and impacting at an angle of 25 degrees with no damage. Further, it was desired to develop a bridge rail that can be installed on either new or existing bridge decks. Development of the energy-absorbing bridge rail involved a study of related bridge rail test results, a conceptual design of the bridge rail, and static testing of critical components.

Previous Research

Results of crash tests on different types of conventional bridge rails show that current deck-to-post and deck-to-concrete parapet connections are not capable of transferring the loads associated with severe automobile impacts into the bridge deck without significant damage to either the bridge rail or the bridge deck (1,2). This was found to be the case with both steel and concrete bridge rails. Further, it was found that the accelerations associated with vehicles striking many conventional bridge rails exceed the limits set forth in NCHRP Report 230 (3).

Results of the previous research have shown that the performance of bridge rails can be improved by incorporating an energy-absorbing mechanism. These results show both that vehicular accelerations can be reduced and that the magnitudes of the forces transferred to the bridge slab can be attenuated through the use of an energy-absorbing bridge rail (4-7). However, the initial costs associated with the different types of energy-absorbing bridge rails surveyed are much higher than the initial costs associated with conventional bridge rails. In addition, none of the energy-absorbing bridge rails surveyed was maintenance free following the large automobile crash test. Further, none of the energy-absorbing rails surveyed can be attached to standard bridge decks. Therefore the previously developed energy-absorbing bridge rails have not gained widespread acceptance.

New Bridge Rail

The decision was made early in this project to develop an energy-absorbing bridge rail that employs a stiff rail supported at regular intervals by flexible energy-absorbing supports. Figure 1 shows an idealized section of the new energy-absorbing bridge rail. This arrangement allows impact forces to be spread over a greater distance along the length of the bridge rail than is the case for conventional bridge rail systems that employ flexible rail sections and stiff posts. Consequently, more of the

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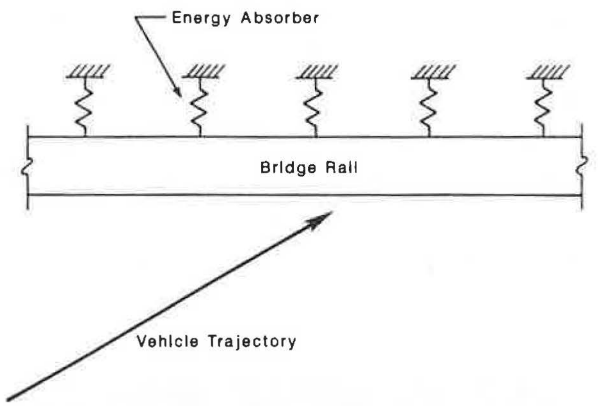


FIGURE 1 Idealized energy-absorbing bridge rail.

bridge deck is brought into action to resist impact forces.

Conceptually, the bridge rail could be made of either concrete or steel. The authors opted to use a bridge rail made of two square steel tubes that are stacked one on top of the other and skip welded along their length. This type of bridge rail is not susceptible to local crushing or buckling problems before development of full plastic flexural capacity. Similar rails have been used in two other recent Texas Transportation Institute (TTI) projects (8,9). Further, the steel tubes needed to fabricate the rail are commonly available in a wide range of sizes.

In previously developed energy-absorbing bridge rails, the energy-absorbing element has been a steel member that absorbs energy by either crushing or deforming (4-7). The authors chose to use rubber energy absorbers in the development of the bridge rail presented herein. The rubber energy absorbers used are primarily manufactured for use in marine dock-fendering systems. Rubber energy absorbers of this type are available from a variety of manufacturers. The rubber used is highly resilient, it remains elastic when subjected to large strains, and it is resistant to the elements of nature. Further, it is readily available in a wide range of different geometries. A cylindrical rubber energy absorber was chosen for the current application.

To complete the system, the energy absorbers needed to be supported in a manner that allowed the impact loads to be transferred into the bridge deck. There are several different ways in which this could be accomplished. One way would be to mount the energy absorbers to the face of a concrete parapet. This option would be acceptable if the rail were to be mounted on a new bridge, but this approach would be prohibitively expensive for a retrofit operation. Therefore the authors chose to support the rubber energy absorbers with steel posts.

Conventional steel bridge posts are welded to base plates that are attached to the bridge deck with anchor bolts. Previous tests on conventional bridge posts show that the bridge deck is severely cracked and spalled before the post reaches its full potential (1). As a result, severe damage is often done to the bridge deck in even moderate impacts. As stated earlier, one of the major objectives of this project was to prevent damage to the bridge deck. To accomplish this, a new bridge post design was developed.

Figure 2 is a sketch of the new bridge post developed for this project. The bridge post is attached to the deck with three bolts that pass through the deck. The mounting holes in the bridge deck can be cast during construction or they can be drilled after construction. When the post is subjected to a lateral force, both a shear force and a moment must be transferred into the bridge deck. The post is designed such that the bolt farthest from the edge of the slab transfers the shear into the deck. This is accomplished by control of the mounting hole tolerances. The moment is transferred into the deck through a couple that develops between the inboard contact force and the tensile forces in the two bolts near the edge of the deck. The inboard force is transferred to the bottom of the deck through a neoprene bearing pad. The outboard force is transferred to the top of the deck through base plates that rest on neoprene bearing pads. In both cases the load experienced by the bridge deck is a compressive load as shown in Figure 3. The magnitudes of the contact stresses are controlled by the sizes of the bearing areas.

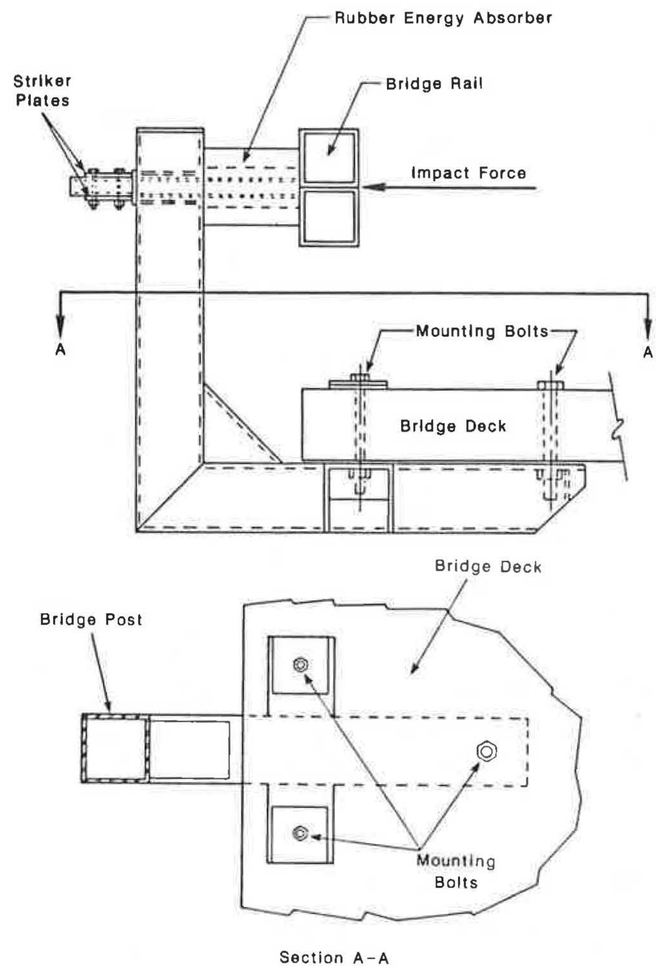


FIGURE 2 Energy-absorbing bridge post.

The weight of the rail is supported by a square steel tube that passes through the center of the cylindrical energy absorber and through a sleeved opening in the post, as shown in Figure 2. During installation of the bridge rail the energy absorber is compressed slightly and striker plates are attached to the back side of the support tube with bolts. The entire assembly is then held firmly in place by the compressive force locked into the energy absorber. The sleeved opening is larger than the support tube so that when the rail is subjected to a lateral force the impact force is transferred to the post through the energy absorber as the support tube passes freely through the post.

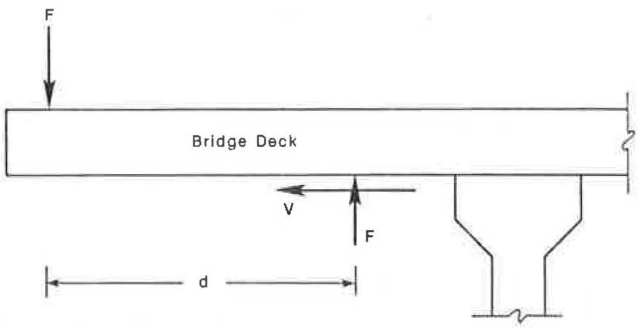


FIGURE 3 Forces exerted on bridge deck.

In selecting the final member sizes for the energy-absorbing bridge rail, the authors relied on structural analysis techniques for beams on elastic foundations, results generated using the BARRIER VII crash simulation program (10), results of selected static tests, and engineering judgment. As a result of these considerations, the bridge rail was made of 6- x 6- x 1/4-in. (15.2- x 15.2- x 0.64-cm) steel tubing and the bridge posts were fabricated using 7- x 7- x 1/4-in. (17.8- x 17.8- x 0.64-cm) steel tubing. The cylindrical rubber energy absorbers chosen had 8-in. (20.3-cm) outer diameters, 4-in. (10.2-cm) inner diameters, and were 10 1/2 in. (26.7 cm) long. Complete fabrication details of the final energy-absorbing bridge rail are available elsewhere (11).

Static Bridge Post Tests

Before construction of the prototype bridge rail, a series of static tests was conducted to verify the combined performance of the post, energy absorber, and bridge deck. These tests were conducted using energy-absorbing bridge posts that were mounted on a short section of bridge deck overhang 7.5 in. (19.1 cm) thick. This bridge deck section was constructed using standard details (11). Mounting holes for the bridge posts were cast into the bridge deck section. Load was applied to the bridge post with a horizontal hydraulic cylinder mounted so that the line of action of the applied load was 21 in. (53.3 cm) above the bridge deck. Results of the tests show that

1. The rubber energy absorber-plunger mechanism operates smoothly even when the lateral load contains a significant longitudinal component;
2. The onset of major yielding in the post occurs at a lateral load of 25,000 lb (115.6 kN);
3. The ultimate strength of the post is 29,000 lb (129.0 kN);
4. Failure of the post was the result of multiple plastic hinges that formed at different points on the post; and
5. There was no cracking in the bridge deck section at the ultimate load.

These results verified that the new bridge post performed as designed.

FULL-SCALE CRASH TEST RESULTS

Full-scale testing of the energy-absorbing bridge rail was conducted at the TTI proving grounds in Bryan, Texas. All tests were run in accordance with criteria presented in NCHRP Report 230 (3). The purpose of the tests was to evaluate the performance of the energy-absorbing bridge rail in terms of structural adequacy, occupant risk, and vehicle exit trajectory.

The tests were conducted using the 59-ft (18-m) section of the energy-absorbing bridge rail shown in Figure 4. NCHRP Report 230 specifies that a 75-ft (22.9-m) section of the bridge rail should be tested; however, it is the opinion of the authors that the performance of the bridge rail is not affected by this deviation. Further, the acceptance of the shorter section allowed the use of an existing standard SDHPT bridge deck.

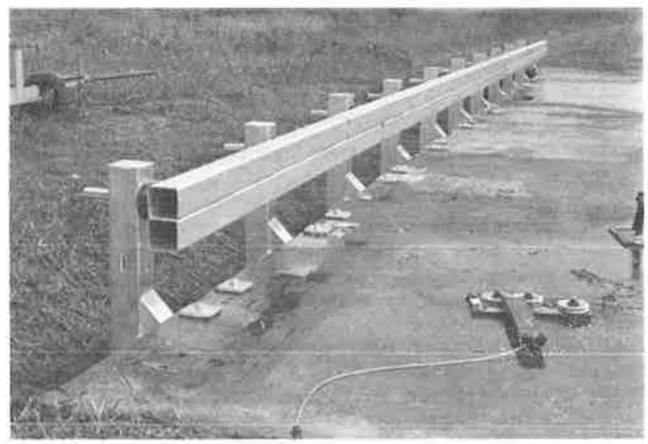


FIGURE 4 59-ft (18-m) section of energy-absorbing bridge rail.

The bridge deck used is approximately 15 years old and has been used in at least three other TTI bridge rail tests. As a result, the bridge deck has accumulated a significant amount of cracking and spalling, which is typical of actual bridge deck damage. Figure 5 shows an example of the worst bridge deck damage before testing. The energy-absorbing bridge rail was mounted on the existing deck so that this worst area of spalling was located between two posts. No attempt was made to repair any of the cracked or spalled areas in the bridge deck. The necessary mounting holes were drilled in the deck using a coring machine without regard for the place-

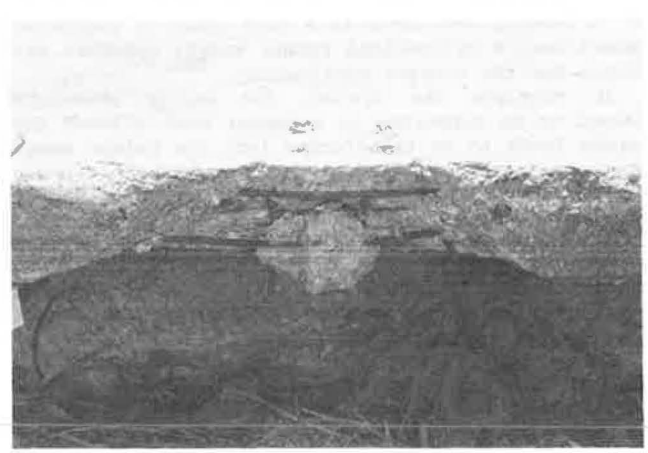


FIGURE 5 Example of worst bridge deck damage before testing.

FIGURE 6 Test vehicle after Test 1.

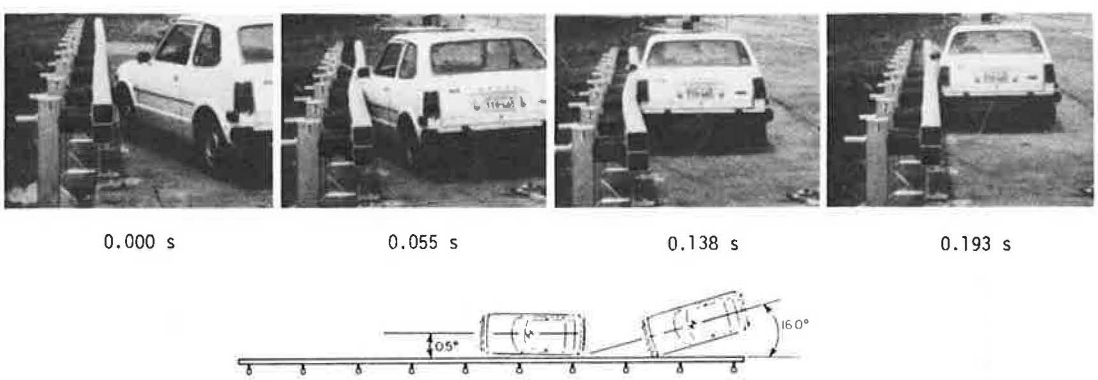
FIGURE 7 Bridge rail after Test 1.

ment of internal reinforcement. This procedure would be typical of a retrofit operation.

Two tests involving a full-sized automobile and a subcompact automobile were conducted on the bridge rail. The tests were conducted in order of increasing severity using the same bridge rail. Complete photographic and accelerometer data are available elsewhere (11). Short discussions of the test results are presented next.

In Test 1 a 1,802-lb (818-kg) Honda Civic struck the energy-absorbing bridge rail at a velocity of 62.6 mph (101 km/hr) and an angle of 16 degrees. Figures 6 and 7 show the test vehicle and bridge rail after the test. Figure 8 shows a summary of the test results. The test vehicle was smoothly redirected with an exit angle of only 0.5 degrees. The damage to the impacting automobile was considered moderate given the severity of the impact. The maximum dynamic deflection of the bridge rail was 4.6 in. (11.7 cm) and the permanent deflection of the face of the rail was 0.6 in. (1.52 cm). This permanent deflection was the result of slack in the post-to-deck connections. The bridge deck experienced no cracking or spalling as a result of this test.

In the second test, a 4,500-lb (2043-kg) Oldsmobile Delta 98 struck the bridge rail at a velocity of 61.0 mph (98.1 km/hr) and an impact angle of 25.5 degrees. The same bridge rail used in Test 1 was used in Test 2. Figures 9 and 10 show the test vehicle and bridge rail after the test. Results of this test are summarized in Figure 11. In this test the automobile was smoothly redirected with an exit angle of only 2.0 degrees. In the opinion of the authors, the damage done to the vehicle was significantly less than would be expected if the automobile struck a rigid bridge rail such as a concrete parapet. The maximum dynamic deflection of the energy-absorbing bridge rail was 7.2 in. (18.3 cm) and the permanent deflection relative to the original face of the rail



Test No.	2417-1
Date	5/21/85
Rail	Low-maintenance, energy-absorbing bridge rail
Post	7x7x¼ in Structural Steel Tube
Post Spacing	6.25 ft (1.91 m)
Length of Installation .	59 ft (18 m)
Rail Deflection	
Maximum	0.38 ft (0.12 m)
Permanent.	0.05 ft (0.02 m)
Vehicle.	1979 Honda Civic
Vehicle Weight	
Test inertia	1,802 lb (818 kg)
Gross static	1,972 lb (895 kg)

Impact Speed.	62.56 mi/h (100.7 km/h)
Impact Angle.	16.0 degrees
Exit Angle.	0.5 degrees
Change in Velocity. . . .	7.79 mi/h (12.5 km/h)
Change in Momentum. . . .	639 lb-s
Occupant Impact Velocity	
Longitudinal.	11.7 fps (3.6 m/s)
Lateral	20.4 fps (6.2 m/s)
Occupant Ridedown Acceleration	
Longitudinal.	-0.6 g
Lateral	8.7 g
Vehicle Damage Classification	
TAD	11LD4
VDI	11LDES2

FIGURE 8 Summary of test results of Test 1.

FIGURE 9 Test vehicle after Test 2.

FIGURE 10 Bridge rail after Test 2.

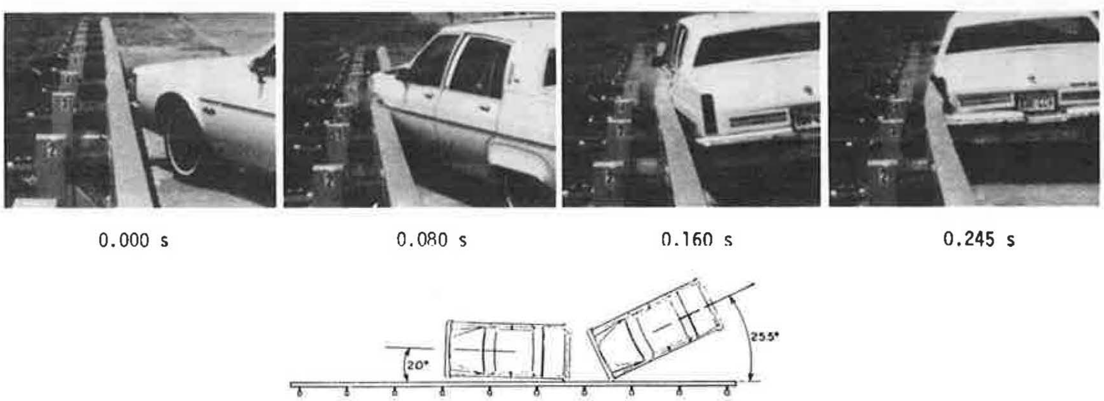
was 0.96 in. (2.4 cm). This permanent deflection was the result of connection slack coupled with a slight amount of yielding in the bridge rail. The bridge deck sustained no damage or cracking during the second test. No maintenance would have been required to keep the bridge rail in service following this impact.

CONCLUSIONS

A low-maintenance, energy-absorbing bridge rail has been developed for use in high traffic volume situations where the cost of repairing conventional bridge rails has become prohibitively expensive. The new bridge rail is designed to meet or exceed all current bridge rail design guidelines for safety and to smoothly redirect a 4,500-lb (2043-kg) automobile traveling at 60 mph (96.6 km/hr) and an impact angle of 25 degrees with no damage done to either the bridge rail or the bridge deck.

A prototype bridge rail has been subjected to two full-scale crash tests involving a 1,800-lb (817-kg) automobile and a 4,500-lb (2043-kg) automobile as prescribed in NCHRP Report 230 (3). Results from both of these tests were within the acceptable limits for roll, pitch, yaw, acceleration, and velocity changes. The vehicles were smoothly redirected throughout the collisions with extremely shallow exit angles. The final vehicle trajectory after impact was parallel to the barrier face. Following the large automobile impact the bridge rail had less than 1 in. (2.54 cm) of permanent lateral deformation, the bridge deck was undamaged, and no maintenance would have been required to keep the bridge rail in service.

Although the new energy-absorbing bridge rail system is a significant departure from conventional bridge rails, it has many advantages. Static data show that even if the new bridge post is taken to



Test No. 2417-2
Date 5/24/85
Rail . . . . Low-maintenance, energy-absorbing bridge rail
Post . . . . 7x7x¼ in Structural Steel Tube
Post Spacing . . . . . . 6.25 ft (1.91 m)
Length of Installation . 59 ft (18 m)
Rail Deflection
Maximum . . . . . . 0.60 ft (0.18 m)
Permanent. . . . . . 0.08 ft (0.02 m)
Vehicle. . . . . 1980 Oldsmobile Ninety-eight
Vehicle Weight . . . . . 4,500 lb (2,043 kg)

Impact Speed. . . . . . . 61.0 mi/h (98.1 km/h)
Impact Angle. . . . . . . 25.5 degrees
Exit Angle. . . . . 2.0 degrees
Change in Velocity. . . . 14.36 mi/h (23.1 km/h)
Change in Momentum. . . . 2,943 lb-s
Occupant Impact Velocity
Longitudinal. . . . . 18.9 fps (5.8 m/s)
Lateral . . . . 28.5 fps (8.7 m/s)
Occupant Ridedown Acceleration
Longitudinal. . . . . -2.5 g
Lateral . . . . 10.0 g
Vehicle Damage Classification
TAD 10LD7
VDI 10LDES3

FIGURE 11 Summary of results of Test 2.

failure the bridge deck will not be damaged. This means that regardless of the impact severity, no bridge deck repair will be required. In addition, no special deck reinforcements or modifications are required so that the bridge rail can easily be retrofitted onto an existing bridge deck. Finally, because of the unique design of the bridge post, less bridge deck space is required for the new energy-absorbing bridge rail than is required for conventional bridge rails. This could be of major importance in retrofit operations where additional lane width is desirable.

ACKNOWLEDGMENTS

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Crash Test Evaluation of Eccentric Loader Guardrail Terminals

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ABSTRACT

The test and evaluation of two W-beam guardrail terminal systems are described. The two terminals, though quite similar, are characterized by a 4.0-ft (1.5-m) flare offset and a 1.5-ft (0.5-m) flare offset. Both designs were subjected to the four-test terminal matrix of NCHRP Report 230 with the 1,800-lb minicar and successful results are reported. The basis for the terminal designs is the breakaway cable terminal (BCT) that has been used in this country for more than 10 years. Improvements to the BCT were necessary because of 1,800-lb (800-kg) automobile impact considerations. The improvements include a nose section attached to the beam end to promote beam buckling for end-on impacts. The nose section is enclosed by a length of standard culvert material.

Traffic barrier end treatments have been a troublesome detail since the implementation of these devices. Upright terminals have spearded vehicles striking them end-on, and turned-down terminals have launched vehicles into obstacles or multiple rollovers. The guardrail breakaway cable terminal (BCT) was designed and developed in NCHRP projects (1-6) and use of this device has been widespread since the first installation in the mid-1970s. As reported in a recent survey (7) of guardrail end treatments, 40 states specify the W-beam guardrail BCT, and 24 states use a version of the turned-down terminal.

Accident data from the field have indicated some unsatisfactory performance of the guardrail BCT (8-10). Detailed examination of these data indicates that a significant percentage of the guardrail BCTs are being installed without the recommended 4-ft (1.2-m) offset parabolic flare. It is apparent that many of the sites where the guardrail BCT has been installed will not accommodate the full flare. Accordingly, many have been installed straight or offset less than 4 ft (1.2 m). Another installation problem noted was the use of a straight taper instead of the parabolic flare to offset the beam end from the rail tangent line. This tapered section represents essentially the same spearing hazard as the straight BCT.

Recent changes in the testing criteria for terminals, found in NCHRP Report 230 (11), have produced a most demanding test condition for terminals. Test-44 conditions, which call for a 1,800-lb (800-kg) vehicle striking end-on at 60 mph (94 km/hr) with a 15-in. (0.4-m) offset from vehicle centerline to terminal centerline, have resulted in violent reactions of the test vehicle to a properly installed BCT. Results of the test using both wood and steel end posts included violent spinning of the vehicle and either rollover or spearing as reported by Kimball et al. (12).

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The FHWA awarded a contract to Southwest Research Institute (SwRI) to produce at least one innovative safe terminal for W-beam guardrails that meets the criteria of NCHRP Report 230 including the 1,800-lb (800-kg) vehicle and, it was hoped, a lower weight vehicle.

The scope of the project included the formulation of design concepts to satisfy the objectives of the contract. On the basis of a critique of these concepts, an uncompleted guardrail concept formulated in a previous FHWA contract at SwRI (13) was selected as a promising solution. This concept used the 4-ft (1.2-m) flare offset geometry of the BCT and was similar in design to the BCT. Another 1.5-ft (0.5-m) flare offset version was also developed.

Development of these terminal designs included detailed design and full-scale crash test evaluations according to the terminal test matrix of NCHRP Report 230 using the 1,800-lb minicar.

TERMINAL DESIGN

General

The guardrail BCT provided the basis for a new terminal design called the eccentric loader BCT. The name is derived from a design feature that introduces a bending moment on the beam end through the use of an eccentric connection.

Development of both the 4-ft (1.2-m) and the 1.5-ft (0.5-m) flare offset designs was completed. The initial design and development work was accomplished in another FHWA contract at SwRI and the results of the 4-ft (1.2-m) flare work were also reported in the final report of the project (13). A recent FHWA technical advisory (14) summarizes the work of this project and includes design drawings.

Terminal Description

The eccentric loader design as shown in Figure 1 is similar to a BCT with the nose section removed and replaced with a fabricated structural steel lever surrounded by a vertical section of corrugated steel

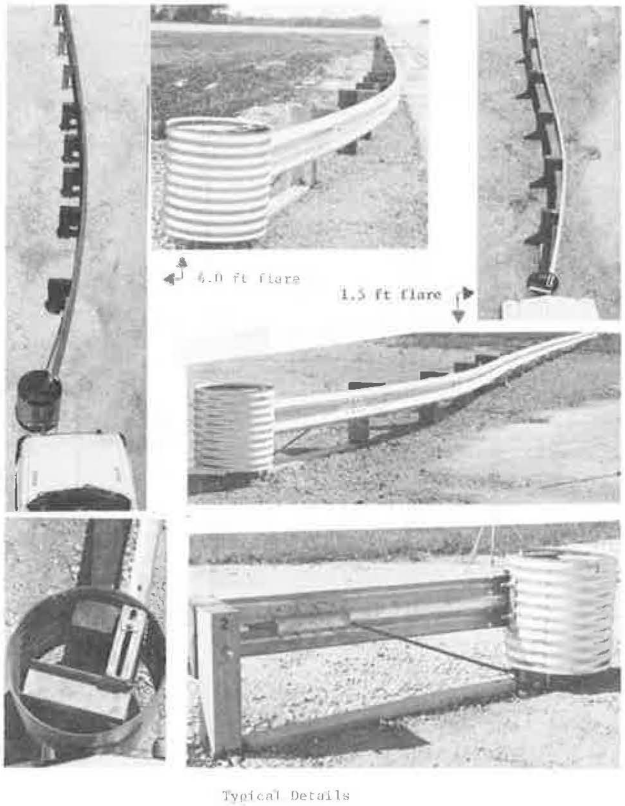


FIGURE 1 Eccentric loader terminal.

pipe. Positive connection of the nose to the end of the W-beam is accomplished by a bolt through the last row of splice holes in the W-beam and long slots in the eccentric loader as shown in Figure 2. The purpose of the bolt is to hold the assembly together after impact; the slot allows longitudinal translation of the end without W-beam resistance. Because the anchor cable provides beam anchorage for the system, no tensile force transfer from the beam to the eccentric loader is necessary or desired.

The eccentric loader has three functions:

- During end-on impacts it transfers the force to the end post, which results in release of the anchor cable before any longitudinal force reaches the W-beam rail element.
- The corrugated steel pipe that encases the W-beam end provides an impenetrable barrier to the end and distributes the resisting force of the W-beam rail element over a large area of the impacting vehicle.
- The off-center attachment of the eccentric loader to the W-beam induces a moment at the W-beam end and thus greatly reduces the buckling strength of the beam.

Further reduction of the beam column strength is accomplished by omission of post-to-rail attachment in the flared area. Position of the beam is maintained by its connection to the eccentric loader; intermediate vertical support for the end beam and anchor cable vertical force component is provided by a shelf angle at the second post.

The first two breakaway posts are installed in steel tube foundations with soil bearing plates as introduced in NCHRP Results Digest 124 (6) and currently used by many states. Because of the additional force transmitted to the anchor cable as a result of omitting the post-to-rail attachment, a strut is placed between the tube foundations to "couple" the two foundations for maximum resistance in the soil during downstream impacts.

During the development of this design, it was observed that contact with posts beyond the first two breakaway posts during end-on impacts increased the potential for vehicle rollover. To minimize this problem, the next four posts (4-ft flare) and three posts (1.5-ft flare) were replaced by breakaway wood posts with drilled holes at and below grade. These posts, which have been extensively used in other designs, were developed in another FHWA project (15). Because the lateral strength was also reduced by the drilled holes, the post spacing was reduced for the 4-ft (1.2-m) flare design because of the localized increased impact angle of this geometry.

Another feature that differs from the original BCT is the use of a block-out between the second post and beam while maintaining approximately the same post alignment. The increased beam curvature at the end required to clear the block-out further reduces the beam column strength.

FULL-SCALE CRASH TESTS

Crash tests conforming to Tests 45, 40, 41, and 44 from the terminal test matrix of NCHRP Report 230 were conducted and successful results obtained. These tests are summarized in Tables 1 and 2. Results of the successful tests are briefly described.

4-ft (1.2-m) Flare Offset Tests

Test RBCT-13

The terminal was evaluated for 60-mph (95-km/hr), end-on impact with the 1,800-lb (800-kg) vehicle with a 15-in. (0.4-m) offset. This test condition has been particularly troublesome because of the weight and stability of the small automobile.

The test vehicle struck the system as shown in Figure 3 and was redirected behind the barrier as designed. Although there was some intrusion into the right side door, no evidence of spearing or potential spearing was noted. The test values measured were in compliance with the criteria of NCHRP Report 230.

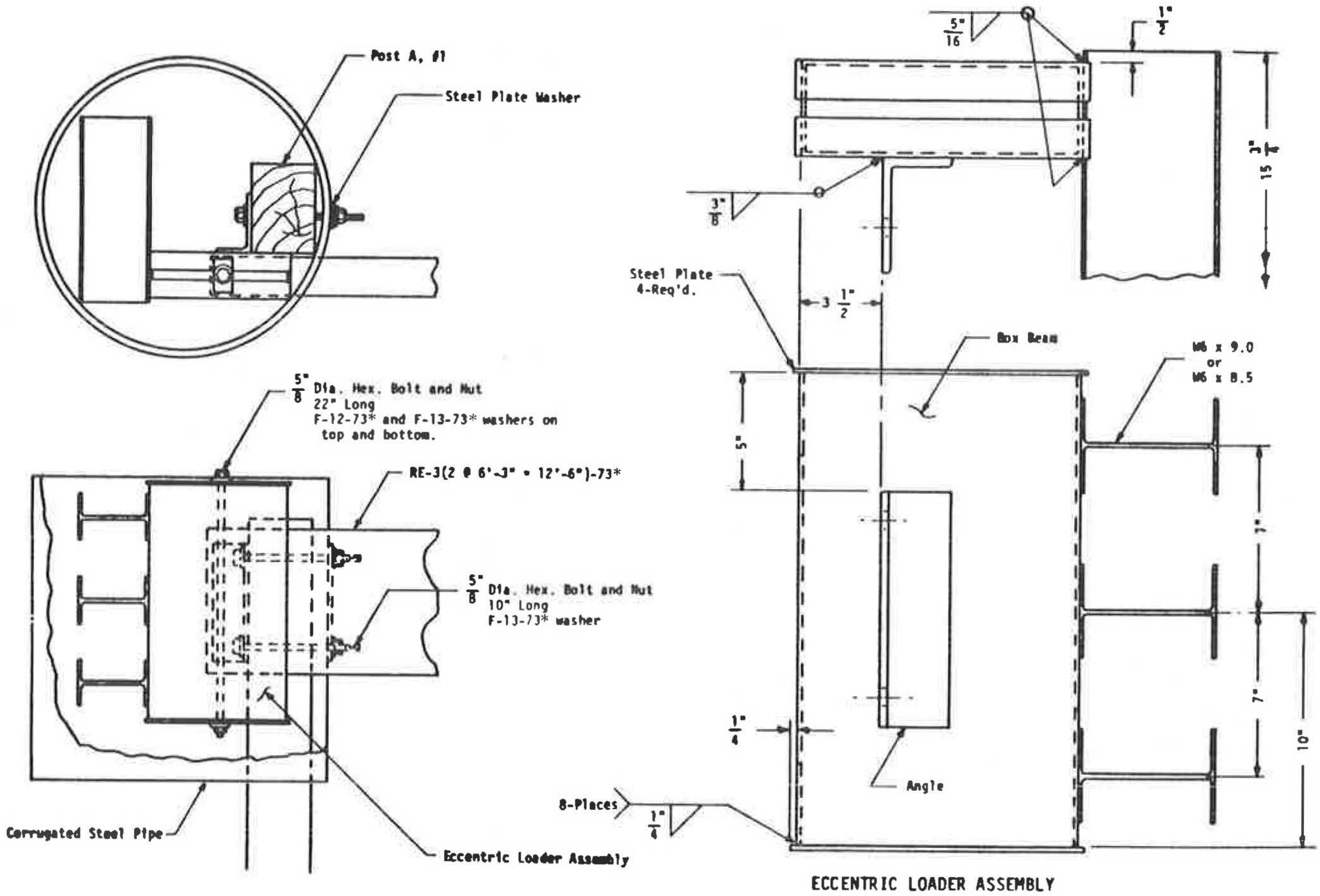
Test RBCT-17

Some difficulty was encountered in achieving the desired results for the length-of-need strength test. Problems attributed to foundation movement at the end post were corrected by adding a strut between the first and second posts. In addition, the slot in the box-beam section of the nose was modified to eliminate loading of the end post by the tension force of the beam, which had caused premature end post failure. Neither of these changes are considered significant for end-on Test RBCT-13 results.

The 4,500-lb (2000-kg) test vehicle struck the terminal downstream of the third post at 58.2 mph (93.6 km/hr) and 24.2 degrees (as measured from the travel way). The vehicle was smoothly redirected as shown in Figure 4 and results indicate compliance with NCHRP Report 230.

Test RBCT-18

This test evaluated the eccentric loader for 60-mph (95-km/hr) end-on performance with the 4,500-lb (2000-kg) vehicle. The vehicle struck the nose and



*See report by AASHTO-AGC-ARTBA·Joint Cooperative Committee, "A Guide to Standardized Barrier Rail Hardware."

FIGURE 2 Eccentric loader.

TABLE 1 Summary of 4-ft Flare Crash Test Results

	Test No.			
	RBCT-13	RBCT-17	RBCT-18	RBCT-19
NCHRP Report 230 test no.	45	40	41	44
Test vehicle	1979 Honda	1978 Plymouth	1978 Plymouth	1978 Honda
Vehicle inertial weight, lb	1,821	4,389	4,423	1,740
Vehicle gross weight, lb	1,986	4,719	4,753	1,905
Impact speed (film), mph	60.5	58.2	59.0	58.9
Impact angle (film), degrees	0.2	24.2	0.6	15.0
Exit angle (film), degrees		14 7		12.6
Maximum 50-msec avg acceleration (accelerometer/film)				
Longitudinal	4.1/6.8	-3.3 film	-2.6/-5.8	-4.8 film
Lateral	5.9/3.4	4.8 film	3.2/4.3	-8.7/-7.4
Occupant risk, NCHRP Report 230[a] (accelerometer/film)				
ΔV longitudinal, fps (30)	29.1/26.7	16.7 film	16.6/13.5	17.1/-7.6
ΔV lateral, fps (20)	12.4/12.0	11.5 film	-13.2/-10.1	19.2/21.0
Ridedown acceleration, g's (accelerometer)				
Longitudinal (15)	7.4	3.4	11.2	
Lateral (15)	7.5	4.8	13.4	13.7
NCHRP Report 230 evaluation				
Structural adequacy (A,D)	NA	Passed	NA	NA
Occupant risk (E,F,G)	Passed	NA	Passed	Passed
Vehicle trajectory (H,I)	Passed	Passed	Passed	Exit angle 12.6° > 0.6(15°)

Note: multiply lb by 0.454 to obtain kg; multiply ft by 0.305 to obtain m; multiply mph by 1.609 to obtain km/hr; and multiple fps by 0.305 to obtain mps. NA = not applicable.

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

TABLE 2 Summary of 1.5-ft Flare Crash Test Results

	Test No.			
	EN-3	EN-5	EN-4	EN-6
NCHRP Report 230 test no.	45	40	41	44
Test vehicle	1979 Honda	1978 Dodge	1978 Dodge	1979 Honda
Vehicle inertial weight, lb	1,815	4,319	4,370	1,785
Vehicle gross weight, lb	1,980	4,649	4,700	1,950
Impact speed (film), mph	59.1	62.9	60.1	58.4
Impact angle (film), degrees	0.5	24.9	0.1	16.4
Exit angle (film), degrees				6.3
Maximum 50-msec avg acceleration (accelerometer/film)				
Longitudinal	-13.8/-8.6	-3.9/-3.0	-5.9/-4.1	-4.4/-3.7
Lateral	4.1/2.9	-7.7/-6.2	2.3/2.1	-8.9/-6.3
Occupant risk, NCHRP Report 230[a] (accelerometer/film)				
ΔV longitudinal, fps (30)	25.8/27.6	6.5/8.1	9.2/15.3	15.5/13.0
ΔV lateral, fps (20)	4.6/10.6	17.3/14.5	-6.1/-11.5	19.0/20.2
Ridedown acceleration, g's (accelerometer)				
Longitudinal (15)	8.7		7.5	1.2
Lateral (15)	10.4	10.6	5.7	10.5
NCHRP Report 230 evaluation				
Structural adequacy (A,D)	NA	Passed	NA	NA
Occupant risk (E,F,G)	Passed	NA	Passed	Passed
Vehicle trajectory (H,I)	Passed	Passed	Passed	Passed

Note: Multiply lb by 0.454 to obtain kg; multiply ft by 0.305 to obtain m; multiply mph by 1.609 to obtain km/hr; and multiply fps by 0.305 to obtain mps. NA = not applicable.

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

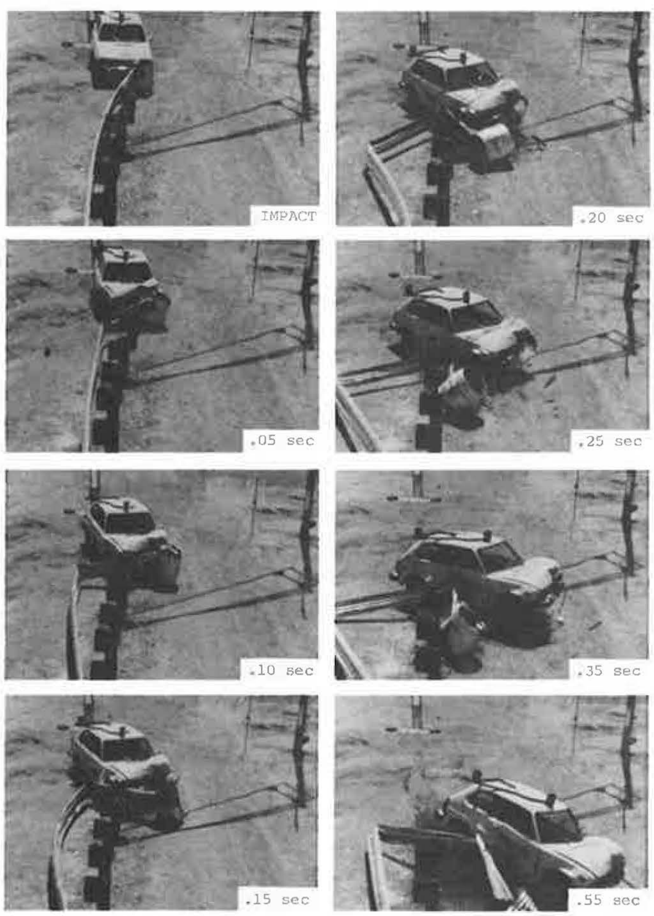


FIGURE 3 Sequential photographs, Test RBCT-13.

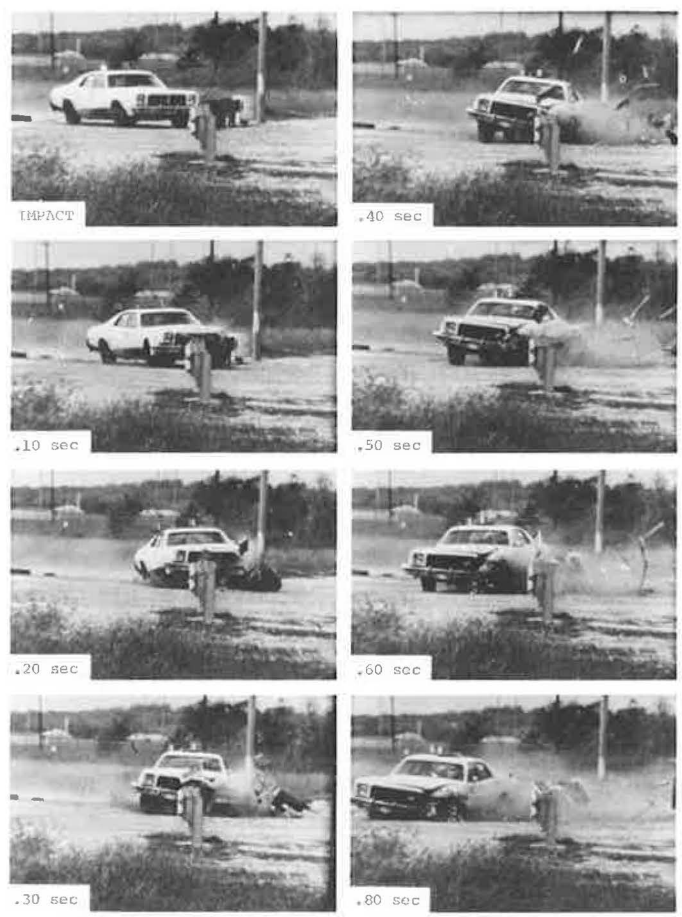


FIGURE 4 Sequential photographs, Test RBCT-17.

was smoothly redirected behind the barrier as shown in Figure 5. Test values indicated compliance with NCHRP Report 230.

Test RBCT-19

This test evaluates a 60-mph (95-km/hr), 15-degree angle impact at a point midway between the nose and length-of-need with a 1,8000-lb (800-kg) vehicle. As shown in Figure 6, the vehicle was smoothly redirected. All values of NCHRP Report 230 were met with the exception of the vehicle trajectory requirement of the exit angle not exceeding 60 percent of the impact angle. Although the exit angle at loss of barrier contact exceeded the 60 percent value, the heading angle of the vehicle began to decrease soon after it left the barrier, and the overall vehicle postimpact trajectory is considered excellent.

1.5-ft (0.5-m) Flare Offset Tests

Test EN-3

The terminal was evaluated for the 60-mph (95-km/hr), end-on impact with the 1,800-lb (800-kg) vehicle with a 15-in. (0.4-m) offset. The test vehicle struck the system as shown in Figure 7 and was redirected behind the barrier as designed. Although considerable vehicle roll and pitch were observed during the test, the vehicle remained upright and came to rest 50 ft (15 m) downstream and 18 ft (5 m) behind the initial impact point. Measured test values indicated compliance with the requirements of NCHRP Report 230.

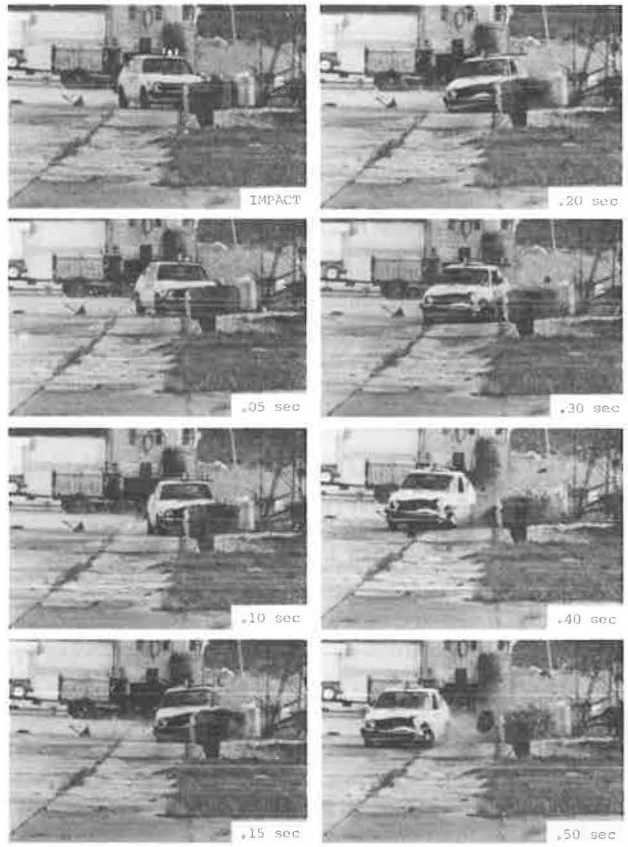


FIGURE 6 Sequential photographs, Test RBCT-19.

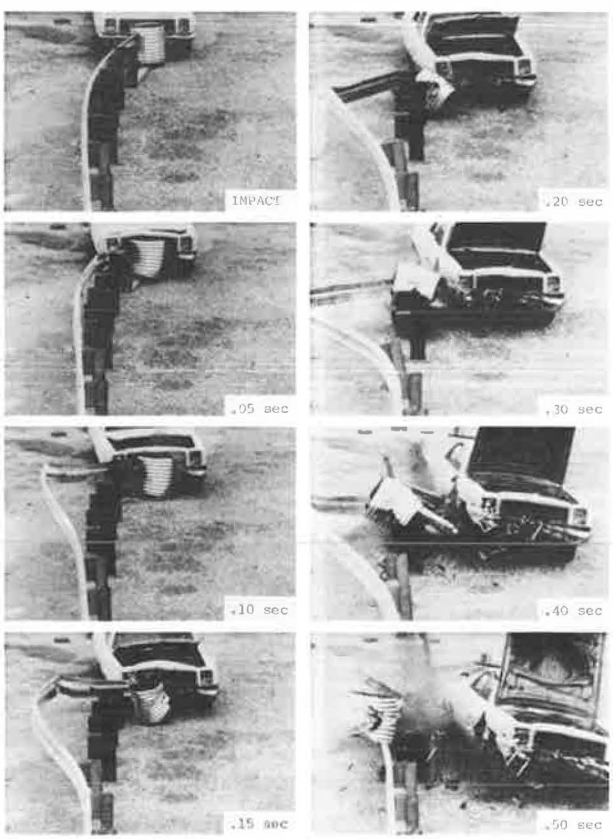


FIGURE 5 Sequential photographs, Test RBCT-18.

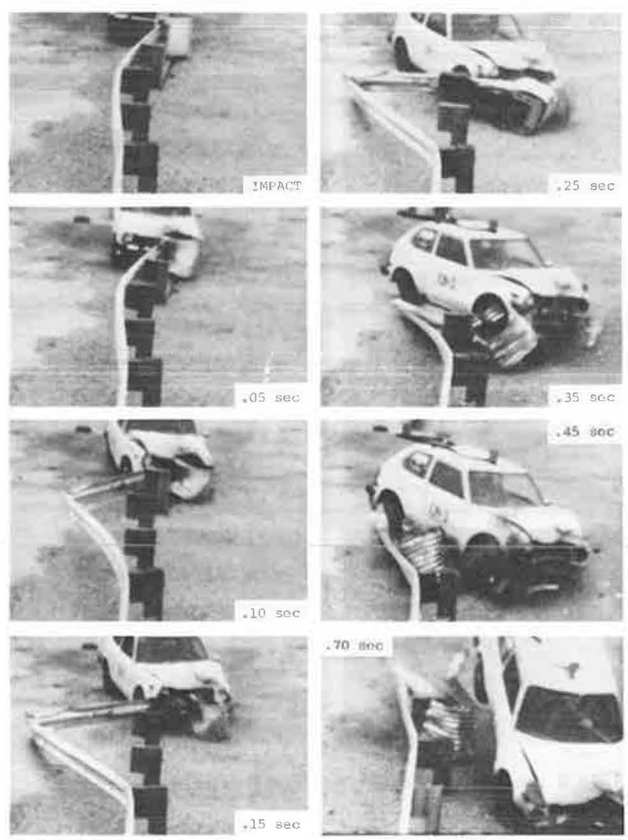


FIGURE 7 Sequential photographs, Test EN-3.

Test EN-4

The purpose of this test was to evaluate the terminal for a center-on impact with the 4,500-lb (1800-kg) vehicle at 60 mph (95 km/hr). As shown in Figure 8, the test vehicle was redirected behind the system although there was considerable vehicle roll. Compliance with the requirements of NCHRP Report 230 was determined.

Test EN-5

This test evaluated the anchor strength of the terminal when struck at the length-of-need by a 4,500-lb (1800-kg) vehicle at 60 mph (95 km/hr) and a 25-degree angle. The test vehicle was smoothly redirected after striking the barrier at the third post as shown in Figure 9. Compliance with the requirements of NCHRP Report 230 was obtained.

Test EN-6

This test was conducted with a 1,800-lb (800-kg) vehicle striking at 60 mph (95 km/hr) and a 15-degree angle with the initial impact point midway between the length-of-need (Post 3) and the end post. As shown in Figure 10, the vehicle was smoothly redirected and the test requirements of NCHRP Report 230 were met.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. On the basis of the results of the test series discussed in this paper, both the 4-ft (1.2-m) and the 1.5-ft (0.5-m) flare offset eccentric loader BCT

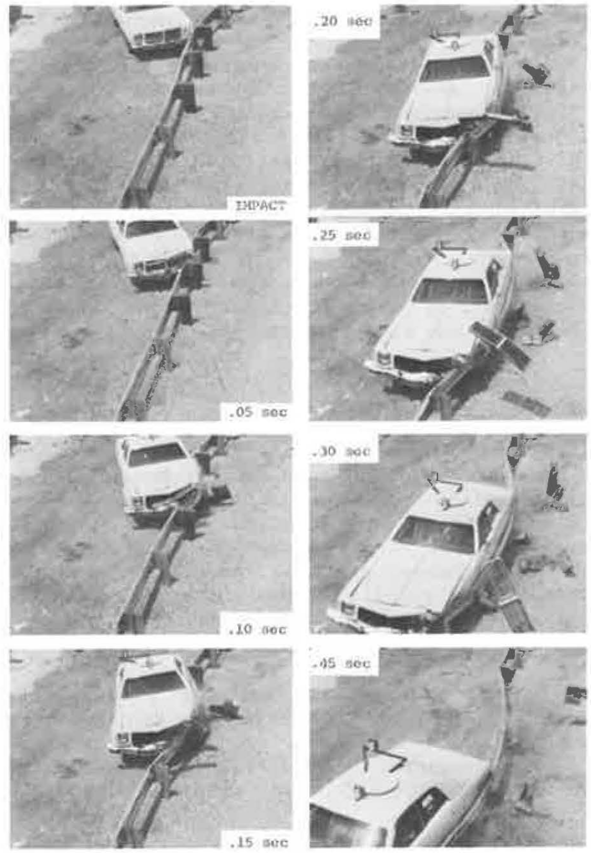


FIGURE 9 Sequential photographs, Test EN-5.

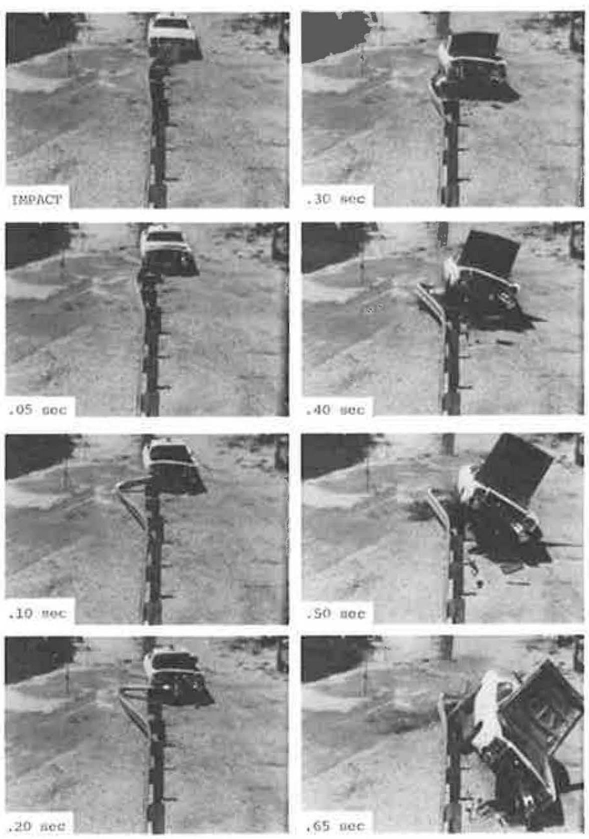


FIGURE 8 Sequential photographs, Test EN-4.

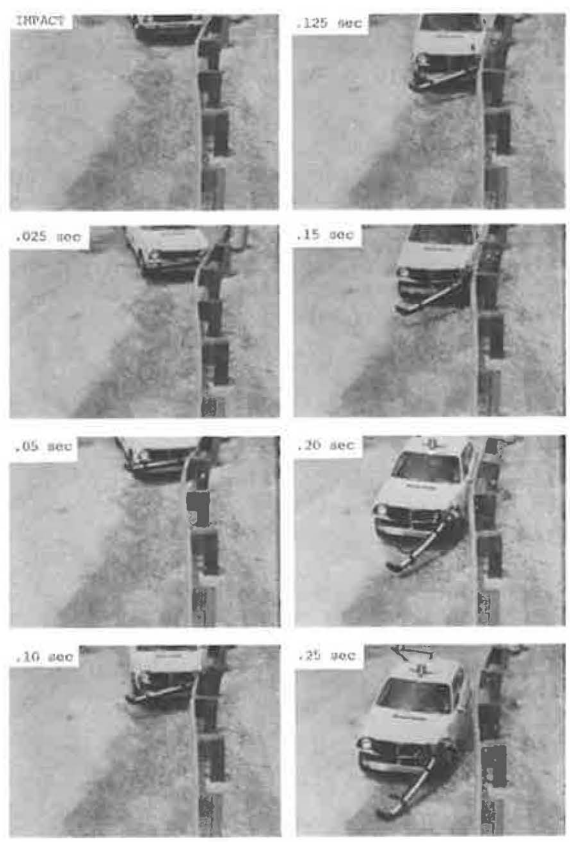


FIGURE 10 Sequential photographs, Test EN-6.

terminal designs satisfy the requirements of NCHRP Report 230.

2. The designs are considered suitable for retrofit or new construction applications.

3. The terminals are considered appropriate for all guardrail systems that are either W-beam systems or have a satisfactory transition to a W-beam system.

4. The 4-ft (1.2-m) flare offset design is considered superior to the 1.5-ft (0.5-m) flare offset design because of the more stable vehicle behavior during end-on impacts.

5. Expected additional costs over the current BCT guardrail terminal for the eccentric loader designs are in the $300 to $400 range.

Recommendations

1. The eccentric loader terminals are recommended for immediate implementation as experimental devices. Design drawings are available from FHWA.

2. How changes in the design drawings will affect the performance of the system should be carefully considered. Changes are not recommended unless cost advantages are realized without compromising performance or improved performance is realized.

3. Where space permits, the 4-ft (1.2-m) flare offset design is recommended. The 1.5-ft (0.5-m) flare should be used at sites with limited space; this is preferable to installing the larger flare on the sideslope.

4. Consideration should be given to distance traveled beyond the end during end-on impacts.

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Traffic Barrier Performance Related to Vehicle Size and Type

JAMES E. BRYDEN and JAN S. FORTUNIEWICZ

ABSTRACT

Field investigations were completed at 3,302 traffic barrier accident sites in New York State to determine the effects of various parameters on barrier performance. Information gathered includes vehicle size and type, barrier type and rail height, and highway parameters. Performance was assessed in terms of occupant injuries, vehicle containment, and secondary collisions. New York's traffic barriers resulted in lower occupant injury rates than do roadside accidents in general, with modern barrier types resulting in fewer injuries than older barriers. Satisfactory vehicle containment was achieved in about 75 percent of the reported barrier accidents. Secondary collisions resulted in about 25 percent of all barrier accidents, primarily when the vehicle was not contained by the barrier. Secondary collisions with fixed objects were most common, followed by rollovers, but other vehicles or pedestrians were rarely involved. Injury rates were much higher when satisfactory containment was not achieved or secondary collisions resulted. Traffic barriers performed best for passenger automobiles and had somewhat reduced performance for vans and light trucks. Heavy trucks experienced about the same severe injury rates as passenger automobiles, but they also frequently penetrated traffic barriers and were involved in secondary collisions. Injury rates in motorcycle accidents were extremely high. Traffic barriers performed best in collisions with midsized passenger automobiles, followed by the smallest and then the largest passenger automobiles. The lower protection provided large automobiles appears to be related to more frequent barrier penetration and secondary collisions.

In-service evaluation is recognized as a final stage of development for new or extensively modified highway safety appurtenances (1). New York State's light-post traffic barriers were developed and perfected during the 1960s. Field performance evaluations conducted in the 1960s and early 1970s confirmed that these barriers provide excellent protection to errant vehicles (2,3). However, during the past few years, substantial changes in vehicle design have occurred and smaller, lighter vehicles are now a large portion of the vehicle fleet. In addition, many highways along which these barriers were installed have been overlaid resulting in changes in effective barrier height. Finally, other barrier types are in service--both early designs that may be reaching the end of their useful life and new designs used selectively for special situations. Thus information was needed to relate the severity of barrier accidents to vehicle size and type, barrier type and mounting height, and roadway features.

PURPOSE AND SCOPE

This investigation is based on traffic accidents on state highways in New York State. Information was compiled on personal injuries, vehicle damage and characteristics, barrier and highway characteristics, and various impact and vehicle trajectory parameters. These data were then analyzed to determine how barrier performance was affected by vehicle size and weight, barrier type and mounting height, and roadway features. In this paper barrier performance in general and the effects of vehicle size and type are examined. Further analysis of accident records will be complete in 1986, and those results will be included in subsequent reports.

METHODOLOGY

New York State law requires an accident report on any traffic accident resulting in personal injury, property damage exceeding $400, or damage to property other than the vehicles involved. These reports are generally filed by the motorist for minor accidents and by a police officer for more severe accidents. Although the law requires an accident report for any accident resulting in damage to a traffic barrier, most minor barrier accidents do not generate a report. Reports are more likely in cases that result in personal injury or vehicle damage sufficient to require towing.

Accident reports provide information on accident time and location, roadway and weather parameters, personal injury and vehicle damage, vehicle registration data, and a brief narrative and sketch describing the accident. These reports are coded by Department of Motor Vehicles (DMV) personnel for computer storage and analysis. For this project, DMV provided a computer tape covering the 12-month period from July 1, 1982, through June 30, 1983, listing all accidents on state-maintained highways in which the first harmful event was impact with a guardrail or median barrier. Because it is difficult or impossible to determine the effect of the barrier on personal injuries, vehicle damage, and other performance indicators for secondary barrier collisions, only accidents in which collision with a barrier was the first harmful event were included in this project.

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Each accident in this investigation was classified according to the most severe injury in the vehicle. Injury severity for each vehicle occupant involved in the accident was contained in the record, with the injury classification for each accident based on the most severe injury level. The most severe nonfatal injuries, A injuries, include severe lacerations, broken or distorted limbs, skull fractures, and other serious injuries. Abrasions, lacerations, and lumps to the head are classed as B injuries, and C injuries are limited to momentary unconsciousness, limping, nausea, hysteria, and complaint of pain with no visible injury.

No injury level was designated on nearly one-third of the records received from DMV. Because injuries are required by state law to be reported, and because most of the accident reports were filed by police agencies, it appears to be reasonable to assume that those records with no specific report of injuries actually represented accidents with no injuries. Although a few minor injuries may have gone undetected, it does not appear likely that many severe injuries would have been unreported.

Using vehicle registration data from another DMV file, vehicle identification numbers (VINs) were added to the accident file for vehicles registered in New York State. The Vindicator Program developed by NHTSA was used to decode the VIN number and add specific vehicle data--make, model, series, weight, wheelbase--to the accident file. The resulting file contained accident description--date, location, impact conditions and factors--as well as personal injury data and detailed vehicle descriptions for about two-thirds of the records. New York's 16,000-mi state highway system includes more than 4,200 mi of traffic barrier. The initial accident file provided by DMV contained 4,698 records, which agreed well with the number expected on the basis of historical records. Subsequent elimination of accidents in New York City and on the NYS Thruway plus invalid traffic barrier records reduced the actual sample to 3,302 accidents.

Although the computer file contained some of the data needed for this investigation, the hard-copy accident reports contained more vital data in the narratives and sketches. That information was necessary to pinpoint accident sites to specific runs of barrier because the coded location was based on reference markers at tenth-mile intervals. In addition, valuable data on impact conditions, vehicle damage, and postimpact vehicle trajectories could only be obtained from the narratives and sketches. In all, hard-copy reports were reviewed for nearly 4,000 of the original 4,698 accidents.

The primary measure of barrier performance is personal injury, but vehicle damage provides a secondary measure. Vehicle damage is important from a financial standpoint, and lower damage is desirable from the standpoint of reduced cost to the motorist. More important, vehicle damage is a surrogate measure of impact severity and injury potential. Vehicle damage was therefore examined in this investigation as a secondary measure of barrier performance. Damage data on individual accident records also provided information about impact conditions. By using the data listed on the accident reports plus the accident sketches and narratives, damage ratings were made for all but two records in the primary accident file. In many cases, although it was possible to determine that some damage had occurred, the exact extent was unknown. When severity ratings were made by research staff, they were made on the conservative side. That is, damage was at least as severe as the rating assigned.

Another important measure of barrier performance is its ability to contain and gradually redirect a vehicle parallel to the roadway. Undesirable responses include barrier penetration (vaulting, submarining, breakthrough), abrupt stops or snags, or deflecting the barrier to contact an object behind it. Barrier response for most of the records was classified into one of eight categories using the narrative descriptions in the hard-copy accident reports. Those categories were redirected, stopped in contact with the barrier, snagged, penetrated, ran under, broke through, went over, and deflected to a fixed object. Redirection accidents were generally quite obvious from the narrative descriptions, but the stopped and snagged categories were more difficult to classify. Definite snags were apparent in only a small number of accidents, but it is possible that some of those classified as "stopped" actually involved a degree of snagging or pocketing. Likewise, it was sometimes difficult to determine the means by which penetration occurred. Therefore, in addition to the three specific classifications of under, over, and through, a fourth general penetration category was included for cases in which a specific determination was impossible.

Another measure of barrier performance in this study is secondary collisions. Following impact with a barrier, the desirable vehicle reaction is to redirect smoothly parallel to the barrier or to stop adjacent to it. Secondary responses--collisions with other fixed objects or vehicles and rollovers--are highly undesirable because they increase the risk of injury to vehicle occupants as well as to those in other vehicles. Secondary impacts were categorized on the DMV records from information contained on the accident report. In this investigation research staff validated the second event codes using the hard-copy narratives and sketches.

The DMV computer records were printed out on special forms with each record on a separate page. These forms were designed to make it possible to add additional roadway and barrier data in coded form. Before proceeding, however, each of the hard-copy reports was reviewed to eliminate incorrectly coded records that did not involve traffic barriers or that were otherwise invalid. Data coding on the forms was accomplished through examination of department photolog files to obtain barrier and roadway parameters, and field inspections were made to determine traffic barrier height and to confirm barrier and roadway parameters.

At every site where roadway or barrier conditions indicated that recent changes may have been made, data obtained during the field visit were compared with the photolog files and construction records. In this way highway changes were detected, and the data entered for each record were correct, with a high degree of reliability, for the time of the accident.

Following completion of the field investigation, the additional data were added to the DMV accident file. The resulting file contained 3,302 records, all on the state highway system outside New York City and all screened to ensure that they described valid barrier accidents. Not every file was complete because in some cases vehicle data were missing. In other cases the accident site could not be located precisely, and some or all of the roadway or barrier data were thus missing. However, ensuring that all the data on the file were reliable meant that the conclusions drawn from this study could be accepted with a high level of confidence.

TRAFFIC BARRIERS ENCOUNTERED

New York State's standard traffic barriers consist of cable, W-beam, and box-beam rail on S 3 x 5.7 steel posts (light posts); W-beam on W 6 x 9 steel posts with block outs (heavy posts); and concrete safety shape barrier. Occasionally, W-beam on wood posts with block outs is used on parkways as well as limited quantities of other barrier types including thrie-beam on heavy posts and self-restoring barrier (SERB). These systems are shown as standards in the 1977 AASHTO barrier guide (4) or have recently been developed and standardized through FHWA-sponsored research. In addition to barriers now specified, various types of previously specified barriers remain in service. These include various combinations of cable and W-beam rail on wood, concrete and steel posts, as well as other types of posts, rails, and concrete walls.

Barriers encountered in this investigation were as follows:

Barrier Type	No. of Accidents
Light-post traffic barriers	1,887
Heavy-post blocked-out W-beam	94
Concrete safety shape	90
Obsolete barriers	810
Others, unknown	421
Total	3,302

RESULTS

General Barrier Performance

The primary purpose of traffic barrier is to prevent vehicles from contacting features along the highway that are potentially more hazardous than the barrier itself. If a system performs well, barrier accidents should be less severe than other roadside accidents. Barrier accidents examined in this study are compared with other accident types in Table 1.

TABLE 1 Comparison of Traffic Barrier Accident Severity and Other Accident Types

			PERCENT OF TOTAL ACCIDENTS				
ACCIDENT TYPE (a)	LOCATION (b)	TOTAL ACCIDENTS	FATAL	A INJURY	B INJURY	C INJURY	NO INJURY
ALL BAR	ST.WIDE	3,302	1.33	9.45	25.83	22.44	40.94
CUR BAR	ST.WIDE	2,071	1.16	9.37	26.99	24.58	37.90
ALL	ST.WIDE	270,688	.71		63.49		35.80
ALL R S	ST.WIDE	40,163	1.50		74.22		24.29

(a) ALL = ALL ACCIDENTS, ALL BAR = ALL BARRIER ACCIDENTS, CUR BAR = CURRENT BARRIER ACCIDENTS, ALL R S = ALL ROAD SIDE ACC

(b) ST.WIDE = STATE MAINTAINED HIGHWAYS UPSTATE AND LONG ISLAND

The data for all accidents and roadside accidents are taken from DMV reports (5) for calendar year 1983. However, injuries are not broken down by severity class. The DMV report lists the total number of injured persons in each severity class not the total number of accidents. Because some vehicles have more than one occupant, often with different injury severities, the injury distribution by occupant is less severe than by accident. Therefore only the total number of injury accidents was examined. Injury classification for the barrier study, on the other hand, is based on the most severe injury in each accident. Because the same injury reporting system was used for the general data in Table 1, it is subject to the same assumptions regarding unreported injuries as is the traffic barrier data base.

The data in Table 1 reveal that guide rail accident statewide are more severe in terms of fatalities and total injuries than are all New York State accidents. However, fixed object accidents are normally more severe than vehicle-to-vehicle accidents, and when barrier accidents from this study are compared with roadside accidents in general, this becomes apparent. Statewide barrier accidents were significantly less severe than all statewide roadside accidents. Considering only currently specified barriers, performance is even better.

Table 2 relates injury severity to vehicle damage, barrier function, and secondary collisions. About 60 percent of the vehicles were rated as lightly damaged, and less than 7 percent were demolished. These data show a pronounced relationship between vehicle damage and injury severity: as damage increased, so did the likelihood of injury. More than half of the fatalities occurred in vehicles that were demolished, and 80 percent of the accidents with no reported injury had vehicle damage rated light. Stated differently, the risk of serious injury or death was less than 7 percent in vehicles with damage rated light but increased to nearly 39 percent if the vehicle was demolished. These relationships are shown graphically in Figure 1.

This information may hold importance for evaluating the results of full-scale crash tests for which injury data are not available. Although vehicle damage in this study was not rated very precisely, a clear-cut relationship is apparent between vehicle damage and injury severity. It therefore appears that more precise scales of vehicle damage, such as those used in full-scale crash tests, may provide an excellent surrogate measure of injury potential.

Barrier function, including injury severity for each category, is also summarized in Table 2. Responses with acceptable containment--redirection and stopped adjacent to the barrier--have much lower severity rates than the unsatisfactory responses of snagging and noncontainment. Comparing those two groups of responses results in a highly significant difference. Accidents in which the vehicle was contained had less than 10 percent severe injuries (fatal and A) and more than 40 percent with no injuries compared with nearly 25 percent severe injuries and only 13.5 percent without injuries when containment was not achieved. Fortunately, in nearly 80 percent of the cases the vehicle was satisfactorily contained. Snagging was noted in only 0.5 percent of the accidents, and there were various types of penetration in 12.5 percent. Because light-post barrier deflects a substantial amount on impact, adequate distance must be provided behind the barrier to accommodate that deflection. Only 13 cases were noted in this analysis--less than 0.5 percent--in which deflection was sufficient to permit the vehicle to contact a fixed object behind the barrier. Underrunning the barrier, a concern with modern vehicles with low frontal geometry, was the least common and occurred in less than 1/4 of 1 percent of all cases.

Secondary collisions, also summarized in Table 2, occurred in just over one-fourth of the cases. Secondary collisions with another motor vehicle were extremely rare--only six were recorded in the primary data file--and secondary collisions with pedestrians were even more rare, with two cases recorded. Most common was collision with a fixed object, which accounted for 583 of 871 second events. Overturning was the next most common and accounted for nearly one-third of the secondary collisions. The data in Table 2 indicate clearly the increased injury severity associated with second events. Although second events were recorded in only 26 percent of the total accidents, they accounted for nearly 90 percent of the fatal accidents and half of the A injuries. Less than 14 percent of the second event accidents had no

TABLE 2 Injury Severity Related to Vehicle Damage, Barrier Function, and Secondary Collisions

	INJURY SEVERITY											
	FATAL		A INJURY		B INJURY		C INJURY		NONE REPORTED		TOTAL	
VEHICLE DAMAGE (NOTE 1)												
DEMOLISHED	23	52.27%	61	19.55%	77	9.03%	47	6.34%	10	.74%	218	6.60%
SEVERE	8	18.18%	46	14.74%	123	14.42%	101	13.63%	53	3.92%	331	10.02%
MODERATE	6	13.64%	82	26.28%	265	31.07%	216	29.15%	206	15.24%	775	23.47%
LIGHT	7	15.91%	122	39.10%	386	45.25%	377	50.88%	1081	79.96%	1973	59.75%
NONE	0	.00%	1	.32%	0	.00%	0	.00%	2	.15%	3	.09%
UNKNOWN	0	.00%	0	.00%	2	.23%	0	.00%	0	.00%	2	.06%
TOTAL	44	100.00%	312	100.00%	853	100.00%	741	100.00%	1352	100.00%	3302	100.00%
VEHICLE DAMAGE (NOTE 2)												
DEMOLISHED	23	10.55%	61	27.98%	77	35.32%	47	21.56%	10	4.59%	218	100.00%
SEVERE	8	2.42%	46	13.90%	123	37.16%	101	30.51%	53	16.01%	331	100.00%
MODERATE	6	.77%	82	10.58%	265	34.19%	216	27.87%	206	26.58%	775	100.00%
LIGHT	7	.35%	122	6.18%	386	19.56%	377	19.11%	1081	54.79%	1973	100.00%
NONE	0	.00%	1	33.33%	0	.00%	0	.00%	2	66.67%	3	100.00%
UNKNOWN	0	.00%	0	.00%	2	100.00%	0	.00%	0	.00%	2	100.00%
TOTAL	44	1.33%	312	9.45%	853	25.83%	741	22.44%	1352	40.94%	3302	100.00%
BARRIER FUNCTION (NOTE 3)												
REDIRECT	19	43.18%	190	60.90%	576	67.53%	533	71.93%	925	68.42%	2243	67.93%
STOP	1	2.27%	38	12.18%	95	11.14%	59	7.96%	129	9.54%	322	9.75%
SNAG	1	2.27%	2	.64%	5	.59%	3	.40%	6	.44%	17	.51%
PENETRATED	5	11.36%	20	6.41%	48	5.63%	22	2.97%	13	.96%	108	3.27%
RAN UNDER	0	.00%	3	.96%	1	.12%	4	.54%	0	.00%	8	.24%
BROKETHR	5	11.36%	11	3.53%	34	3.99%	19	2.56%	17	1.26%	86	2.60%
WENT OVER	12	27.27%	43	13.78%	63	7.39%	70	9.45%	22	1.63%	210	6.36%
DEFLECT TOFIX	0	.00%	1	.32%	10	1.17%	2	.27%	0	.00%	13	.39%
UNKNOWN	1	2.27%	4	1.28%	21	2.46%	29	3.91%	240	17.75%	295	8.93%
TOTAL	44	100.00%	312	100.00%	853	100.00%	741	100.00%	1352	100.00%	3302	100.00%
BARRIER FUNCTION (NOTE 4)												
REDIRECT	19	.85%	190	8.47%	576	25.68%	533	23.76%	925	41.24%	2243	100.00%
STOP	1	.31%	38	11.80%	95	29.50%	59	18.32%	129	40.06%	322	100.00%
SNAG	1	5.88%	2	11.76%	5	29.41%	3	17.65%	6	35.29%	17	100.00%
PENETRATED	5	4.63%	20	18.52%	48	44.44%	22	20.37%	13	12.04%	108	100.00%
RAN UNDER	0	.00%	3	37.50%	1	12.50%	4	50.00%	0	.00%	8	100.00%
BROKETHR	5	5.81%	11	12.79%	34	39.53%	19	22.09%	17	19.77%	86	100.00%
WENT OVER	12	5.71%	43	20.48%	63	30.00%	70	33.33%	22	10.48%	210	100.00%
DEFLECT TOFIX	0	.00%	1	7.69%	10	76.92%	2	15.38%	0	.00%	13	100.00%
UNKNOWN	1	.34%	4	1.36%	21	7.12%	29	9.83%	240	81.36%	295	100.00%
TOTAL	44	1.33%	312	9.45%	853	25.83%	741	22.44%	1352	40.94%	3302	100.00%
SECOND EVENT												
MOTOR VEHICLE	0	.00%	0	.00%	1	16.67%	2	33.33%	3	50.00%	6	100.00%
PEDESTRIAN	0	.00%	0	.00%	0	.00%	0	.00%	2	100.00%	2	100.00%
OTHER NOT FIXED OBJ	0	.00%	0	.00%	0	.00%	1	50.00%	1	50.00%	2	100.00%
LIGHT/UTILITY POLE	4	7.14%	9	16.07%	25	44.64%	15	26.79%	3	5.36%	56	100.00%
GUIDERAIL	1	1.00%	11	11.00%	36	36.00%	25	25.00%	27	27.00%	100	100.00%
SIGN POST	1	3.85%	1	3.85%	11	42.31%	6	23.08%	7	26.92%	26	100.00%
TREE	7	6.48%	21	19.44%	41	37.96%	23	21.30%	16	14.81%	108	100.00%
BUILDING/WALL	0	.00%	0	.00%	3	75.00%	1	25.00%	0	.00%	4	100.00%
CURBING	1	14.29%	3	42.86%	0	.00%	3	42.86%	0	.00%	7	100.00%
FENCE	1	5.88%	5	29.41%	7	41.18%	3	17.65%	1	5.88%	17	100.00%
BRIDGE STRUCTURE	3	7.69%	5	12.82%	15	38.46%	10	25.64%	6	15.38%	39	100.00%
CULVERT/HEAD WALL	1	7.69%	7	53.85%	3	23.08%	2	15.38%	0	.00%	13	100.00%
MEDIAN/BARRIER	0	.00%	3	20.00%	6	40.00%	2	13.33%	4	26.67%	15	100.00%
SNOW EMBANKMENT	0	.00%	0	.00%	4	50.00%	0	.00%	4	50.00%	8	100.00%
EARTH ELEM/RC/DITCH	2	1.09%	27	14.75%	62	33.88%	68	37.16%	24	13.11%	183	100.00%
FIRE HYDRANT	0	.00%	1	50.00%	0	.00%	1	50.00%	0	.00%	2	100.00%
OTHER FIXED OBJECT	0	.00%	3	60.00%	0	.00%	0	.00%	2	40.00%	5	100.00%
OVERTURNED	15	5.81%	55	21.32%	107	41.47%	69	26.74%	12	4.65%	258	100.00%
FIRE/EXPLOSION	0	.00%	0	.00%	3	50.00%	0	.00%	3	50.00%	6	100.00%
SUBMERSION	2	50.00%	0	.00%	1	25.00%	0	.00%	1	25.00%	4	100.00%
RAN OFF ROADWY ONLY	0	.00%	0	.00%	2	66.67%	1	33.33%	0	.00%	3	100.00%
OTHER NON COLLISION	1	14.29%	0	.00%	2	28.57%	2	28.57%	2	28.57%	7	100.00%
FIXED OBJECT SUB TOT	21	3.60%	96	16.47%	213	36.54%	159	27.27%	94	16.12%	583	100.00%
ALL SECOND EV SUB T	39	4.48%	151	17.34%	329	37.77%	234	26.87%	118	13.55%	871	100.00%
NO SECOND EVENT	5	.21%	161	6.62%	524	21.55%	507	20.86%	1234	50.76%	2431	100.00%
TOTAL	44	1.33%	312	9.45%	853	25.83%	741	22.44%	1352	40.94%	3302	100.00%

NOTES: FATAL + A INJURIES = SEVERE INJURIES
1 - PERCENT OF EACH INJURY SEVERITY OCCURRING IN EACH DAMAGE CATEGORY
2 - PERCENT OF EACH DAMAGE CATEGORY OCCURRING IN EACH SEVERITY CATEGORY
3 - PERCENT OF EACH INJURY SEVERITY OCCURRING IN EACH BARRIER FUNCTION CATEGORY
4 - PERCENT OF EACH BARRIER FUNCTION CATEGORY OCCURRING IN EACH SEVERITY CATEGORY

reported injuries compared with more than half of the accidents without second events. Especially harmful second event types were collisions with utility poles with 23 percent severe injuries, collisions with trees with 26 percent severe injuries, and overturns with 27 percent severe injuries.

Injury severity was shown in the previous paragraphs to be higher when a second event occurred and in those cases in which the vehicle was not properly contained by the barrier. It is logical to expect that if a vehicle is satisfactorily contained, second events will be less likely. This relationship is examined in Table 3, which provides a comparison of the occurrence of second events for the various categories of barrier function. When the vehicle was redirected or stopped adjacent to the barrier, second events were relatively rare--less than 20 percent of all accidents. However, when the vehicle was not properly contained, the vehicle overturned in 28

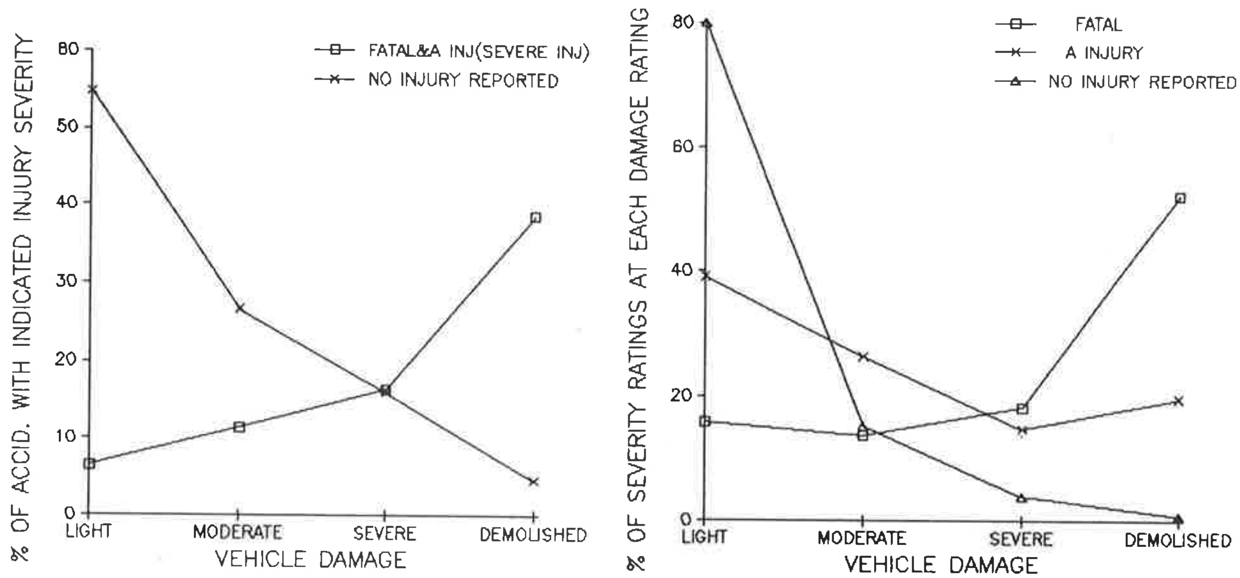

FIGURE 1 Injury severity related to vehicle damage.

percent of the cases and struck a fixed object in 52 percent of the cases. This clearly points out the desirability of smooth containment and redirection in vehicle-barrier collisions.

Effects of Vehicle Type on Barrier Performance

By using information from the accident report, vehicle type was classified for all but 7 of the 3,302 accidents in the primary file. Table 4 gives injury severity, barrier function, and secondary collision data for each vehicle type. Passenger automobiles had by far the lowest overall injury rates. Vans and light trucks had B and C injury rates nearly identical to those of passenger automobiles, but the rate of fatal and A injuries was significantly higher. Based on a small number of fatal and A injury accidents, the rate for tractor-trailers is not much

TABLE 3 Secondary Collisions Related to Barrier Function

SECOND EVENT[1]

BARRIER FUNCT	OVERTURNED		FIXED OBJECT		OTHER		NONE		TOTAL	
REDIRECT	125	48.45%	328	56.16%	15	50.00%	1775	73.05%	2243	67.93%
STOP	11	4.26%	18	3.08%	2	6.67%	291	11.98%	322	9.75%
CONTAINED S T	136	52.71%	346	59.25%	17	56.67%	2066	85.02%	2565	77.68%
SNAG	2	.78%	3	.51%	0	.00%	12	.49%	17	.51%
PENETRATED	28	10.85%	63	10.79%	4	13.33%	13	.53%	108	3.27%
RAN UNDER	0	.00%	1	.17%	0	.00%	7	.29%	8	.24%
BROKETHR	18	6.98%	51	8.73%	2	6.67%	15	.62%	86	2.60%
WENT OVER	72	27.91%	94	16.10%	7	23.33%	37	1.52%	210	6.36%
PENETRATED ST	118	45.74%	209	35.79%	13	43.33%	72	2.96%	412	12.48%
DEFLECT TOFIX	0	.00%	12	2.05%	0	.00%	1	.04%	13	.39%
UNKNOWN	2	.78%	14	2.40%	0	.00%	279	11.48%	295	8.93%
TOTAL	258	100.00%	584	100.00%	30	100.00%	2430	100.00%	3302	100.00%

SECOND EVENT[2]

BARRIER FUNCT	OVERTURNED		FIXED OBJECT		OTHER		NONE		TOTAL	
REDIRECT	125	5.57%	328	14.62%	15	.67%	1775	79.14%	2243	100.00%
STOP	11	3.42%	18	5.59%	2	.62%	291	90.37%	322	100.00%
CONTAINED S T	136	5.30%	346	13.49%	17	.66%	2066	80.55%	2565	100.00%
SNAG	2	11.76%	3	17.65%	0	.00%	12	70.59%	17	100.00%
PENETRATED	28	25.93%	63	58.33%	4	3.70%	13	12.04%	108	100.00%
RAN UNDER	0	.00%	1	12.50%	0	.00%	7	87.50%	8	100.00%
BROKETHR	18	20.93%	51	59.30%	2	2.33%	15	17.44%	86	100.00%
WENT OVER	72	34.29%	94	44.76%	7	3.33%	37	17.62%	210	100.00%
PENETRATED ST	118	28.64%	209	50.73%	13	3.16%	72	17.48%	412	100.00%
DEFLECT TOFIX	0	.00%	12	92.31%	0	.00%	1	7.69%	13	100.00%
UNKNOWN	2	.68%	14	4.75%	0	.00%	279	94.58%	295	100.00%
TOTAL	258	7.81%	584	17.69%	30	.91%	2430	73.59%	3302	100.00%

1 - PERCENT OF SECOND EVENT CATEGORY OCCURRING IN EACH BARRIER FUNCTION CATEGO
2 - PERCENT OF BARRIER FUNCTION CATEGORY OCCURRING IN EACH SECOND EVENT CATEGO

TABLE 4 Vehicle Type Related to Injury Severity, Barrier Function, and Secondary Events

INJURY LEVEL	PASSENGER CAR		MOTORCYCLE		VAN/LT TRUCK		HEAVY TRUCK		SEMI		BUS		UNKNOWN		TOTAL	
FATAL	27	.96%	7	8.43%	7	2.17%	1	6.67%	2	3.64%	0	.00%	0	.00%	44	1.33%
A INJURY	229	8.13%	33	39.76%	42	13.04%	0	.00%	4	7.27%	1	50.00%	3	42.86%	312	9.45%
B INJURY	708	25.12%	31	37.35%	84	26.09%	6	40.00%	24	43.64%	0	.00%	0	.00%	853	25.83%
C INJURY	631	22.39%	12	14.46%	74	22.98%	7	46.67%	16	29.09%	1	50.00%	0	.00%	741	22.44%
NONE	1223	43.40%	0	.00%	115	35.71%	1	6.67%	9	16.36%	0	.00%	4	57.14%	1352	40.94%
TOTAL	2818	100.00%	83	100.00%	322	100.00%	15	100.00%	55	100.00%	2	100.00%	7	100.00%	3302	100.00%
BARRIER FUNCTION																
REDIRECTED	1959	69.52%	50	60.24%	201	62.42%	5	33.33%	21	38.18%	2	100.00%	0	.00%	2238	67.78%
STOP	264	9.37%	22	26.51%	32	9.94%	1	6.67%	3	5.45%	0	.00%	0	.00%	322	9.75%
CONTAINED SUB TOTAL	2223	78.89%	72	86.75%	233	72.36%	6	40.00%	24	43.64%	2	100.00%	0	.00%	2560	77.53%
SNAGGED	14	.50%	0	.00%	3	.93%	0	.00%	0	.00%	0	.00%	0	.00%	17	.51%
PENETRATED	76	2.70%	1	1.20%	20	6.21%	3	20.00%	8	14.55%	0	.00%	0	.00%	108	3.27%
RAN UNDER	4	.14%	4	4.82%	0	.00%	0	.00%	0	.00%	0	.00%	0	.00%	8	.24%
BROKE THROUGH	66	2.34%	1	1.20%	7	2.17%	4	26.67%	8	14.55%	0	.00%	0	.00%	86	2.60%
WENT OVER	159	5.64%	4	4.82%	33	10.25%	2	13.33%	12	21.82%	0	.00%	0	.00%	210	6.36%
PENETRATED SUB TOTAL	305	10.82%	10	12.05%	60	18.63%	9	60.00%	28	50.91%	0	.00%	0	.00%	412	12.48%
DEFLECTED TO FIX OBJ	9	.32%	0	.00%	3	.93%	0	.00%	1	1.82%	0	.00%	0	.00%	13	.39%
UNKNOWN	267	9.47%	1	1.20%	23	7.14%	0	.00%	2	3.64%	0	.00%	7	100.00%	300	9.09%
TOTAL	2818	100.00%	83	100.00%	322	100.00%	15	100.00%	55	100.00%	2	100.00%	7	100.00%	3302	100.00%
SECOND EVENT																
FIXED OBJECT	465	16.50%	8	9.64%	86	26.71%	7	46.67%	14	25.45%	1	50.00%	0	.00%	581	17.60%
OVERTURNED	172	6.10%	13	15.66%	52	16.15%	6	40.00%	15	27.27%	0	.00%	0	.00%	258	7.81%
OTHER SECOND EVENT	24	.85%	1	1.20%	5	1.55%	0	.00%	0	.00%	0	.00%	0	.00%	30	.91%
SECOND EVENT SUB T	661	23.46%	22	26.51%	143	44.41%	13	86.67%	29	52.73%	1	50.00%	0	.00%	869	26.32%
NO SECOND EVENT	2157	76.54%	61	73.49%	179	55.59%	2	13.33%	26	47.27%	1	50.00%	0	.00%	2426	73.47%
UNKNOWN	0	.00%	0	.00%	0	.00%	0	.00%	0	.00%	0	.00%	7	100.00%	7	.21%
TOTAL	2818	100.00%	83	100.00%	322	100.00%	15	100.00%	55	100.00%	2	100.00%	7	100.00%	3302	100.00%

higher than for passenger automobiles, but the rate of B and C injuries appears to be substantially higher. Motorcycle accidents had by far the highest severity rate, with all the reported accidents resulting in personal injury and nearly half resulting in a fatality or an A injury. Only 15 accidents involving heavy trucks were recorded, but it appears that the total injury rate was substantially higher than for passenger automobiles. Only one accident each was reported involving an intercity bus and a school bus; these resulted in an A injury and a C injury, respectively.

Considering barrier function, satisfactory containment--redirected or stopped--resulted in 79 percent of the passenger automobile impacts and 72 percent of the light truck impacts, but in just over 40 percent of the impacts involving heavy trucks and tractor-trailers. Considering only midsection collisions (no collisions on barrier terminals), 88 percent of the passenger automobiles and 83 percent of the vans and light trucks were contained. Containment of heavy trucks and tractor-trailers changed only slightly from the total sample because of the low occurrence of terminal accidents for those vehicle types.

Passenger automobiles and motorcycles experienced secondary impacts in only about one-quarter of all collisions compared with 45 percent of the van and light truck accidents and 60 percent of the heavy truck and tractor-trailer collisions combined. Overturning was relatively rare for passenger automobiles: only 6 percent of all collisions resulted in this type of second event. However, overturning occurred in 16 percent of the van and light truck collisions and in 30 percent of the heavy truck and tractor-trailer accidents. Nearly all the remaining second events were impacts on fixed objects for each of the vehicle types.

Effects of Passenger Automobile Size and Weight on Barrier Performance

In addition to classifying vehicle type from information on the accident form, VINs obtained from registration files provided detailed vehicle information. Passenger automobiles were further sorted by wheelbase, using categories suggested by NHTSA (6), and by weight, using categories from earlier reports by New York State (7) and General Motors (8).

Size classes used in the analysis are as follows:

Vehicle Class	Description	Wheelbase (in.)	Weight (lb)
1	Small subcompact	<96	<2,000
2	Subcompact	96-101	2,000-2,499
3	Compact	102-111	2,500-3,249
4	Intermediate	112-120	3,250-3,999
5	Full size	>120	≥4,000

Passenger automobile injury severity is given in Table 5 by wheelbase and by weight. Because of the small numbers of fatal accidents, severity was regrouped into three categories: fatal and A, termed severe injuries; B and C, termed nonsevere injuries; and none. A chi-square analysis was performed to determine whether severity differed by vehicle size. In terms of weight, the differences among the five classes are highly significant: the lowest severe and total injury rates were for subcompacts, and small subcompacts and compacts had only slightly higher rates. Intermediate and full-sized cars had substantially higher rates. By wheelbase, the results were quite similar except that small subcompacts had the lowest severe injury rate followed by compacts and subcompacts. Larger automobiles again had substantially higher rates, although the differences among classes were not highly significant.

TABLE 5 Injury Severity Related to Passenger Automobile Wheelbase and Weight

WHEELBASE, INCHES

INJURY LEVEL	< 96 TOTAL		96-101 TOTAL		102-111 TOTAL		112-120 TOTAL		> 120 TOTAL		UNKNOWN TOTAL		TOTAL	
FATAL	1	.20%	5	.93%	8	1.04%	7	1.50%	0	.00%	6	1.48%	27	.96%
A INJURY	37	7.25%	39	7.25%	50	6.52%	42	8.99%	15	11.45%	46	11.36%	229	8.13%
B INJURY	127	24.90%	133	24.72%	176	22.95%	104	22.27%	36	27.48%	132	32.59%	708	25.12%
C INJURY	108	21.18%	101	18.77%	194	25.29%	102	21.84%	29	22.14%	97	23.95%	631	22.39%
NONE	237	46.47%	260	48.33%	339	44.20%	212	45.40%	51	38.93%	124	30.62%	1223	43.40%
FATAL &A INJURY	38	7.45%	44	8.18%	58	7.56%	49	10.49%	15	11.45%	52	12.84%	256	9.08%
B & C INJURY	235	46.08%	234	43.49%	370	48.24%	206	44.11%	65	49.62%	229	56.54%	1339	47.52%
TOTAL	510	100.00%	538	100.00%	767	100.00%	467	100.00%	131	100.00%	405	100.00%	2818	100.00%

WEIGHT, lb.

INJURY LEVEL	< 2000 TOTAL		2000-2499 TOTAL		2500-3249 TOTAL		3250-3999 TOTAL		> 4000 TOTAL		TOTAL TOTAL	
FATAL	1	.40%	2	.34%	7	.87%	10	1.52%	4	1.65%	27	.96%
A INJURY	19	7.69%	44	7.53%	60	7.48%	50	7.60%	24	9.88%	229	8.13%
B INJURY	57	23.08%	148	25.34%	196	24.44%	151	22.95%	60	24.69%	708	25.12%
C INJURY	54	21.86%	109	18.66%	181	22.57%	163	24.77%	53	21.81%	631	22.39%
NONE	116	46.96%	281	48.12%	358	44.64%	284	43.16%	102	41.98%	1223	43.40%
FATAL & A INJ	20	8.10%	46	7.88%	67	8.35%	60	9.12%	28	11.52%	256	9.08%
B & C INJURY	111	44.94%	257	44.01%	377	47.01%	314	47.72%	113	46.50%	1339	47.52%
TOTAL	247	100.00%	584	100.00%	802	100.00%	658	100.00%	243	100.00%	2818	100.00%

Table 6 gives possible causes of the differences in injury rates among vehicle sizes. It was shown previously that accidents that resulted in a secondary collision or lack of barrier containment had higher severity rates. Therefore these two parameters were examined in terms of vehicle size and weight. Larger automobiles were involved in more second events and satisfactorily contained less often than smaller cars. When classified by vehicle weight, the differences among vehicles are highly significant for second event and significant for containment. By wheelbase, the differences are still apparent but not statistically significant. The types of second events also show a clear-cut difference among vehicle classes. Overturning was most frequent for small--either by wheelbase or weight--automobiles and least frequent for large automobiles. The opposite was true of fixed object collisions: small automobiles experienced the fewest and large automobiles the most. These differences are shown graphically in Figures 2 and 3. The overturn rate for the smallest automobiles was about double that of the largest automobiles, and the fixed object involvement was about half. However, because fixed object involvement was at least double the overturn rate overall, the net result was that large vehicles had a higher secondary collision rate than small vehicles. This higher involvement in secondary collisions and lower containment rate appear to explain the higher injury rates for larger automobiles.

DISCUSSION AND FINDINGS

Accidents involving roadside and fixed objects tend to be more severe than other accident types. However, data from this investigation show that collisions with traffic barriers are less severe than roadside accidents in general, and, if only modern barrier types are considered, the reported injury rate is about 20 percent less than for all roadside accidents. Traffic barriers currently installed in New York State resulted in fatal injuries in about 1 percent of the reported accidents and other serious injuries in an additional 9 percent. Older types of barriers had about twice as many fatalities and 50 percent more serious injuries than modern barriers. Vehicle damage as determined from information provided on accident reports correlated closely with personal injuries. These data indicate that vehicle damage from full-scale crash tests may provide a good surrogate measure of personal injury potential.

Two aspects of barrier accidents were closely examined in an attempt to explain differences in performance. Barrier function, as defined by the postimpact trajectory of the vehicle, described how the barrier either met or failed to meet its primary purpose of preventing contact with the roadside hazard. Secondary collisions provided a second measure of how well the barrier performed this function. These results clearly showed that injuries were lowest when the barrier performed as intended (i.e., the vehicle was properly contained by the barrier and no secondary collision resulted). Overall, the vehicle was contained by the barrier in more than 75 percent of the accidents, and secondary collisions--primarily fixed objects or rollovers--occurred in only about 25 percent of the impacts. Secondary collisions with other vehicles or pedestrians were extremely rare; they occurred in less than 1/4 percent of all accidents. It was also shown that secondary collisions were much more likely when proper containment was not achieved. Less than 20 percent of the containment accidents resulted in secondary collisions compared with about 80 percent of the noncontainment accidents.

Traffic barriers are designed specifically to contain and protect passenger automobiles. As expected, results of this investigation confirm that barriers performed best for passenger automobiles in terms of injury severity as well as vehicle containment and secondary collisions. Very little protection

TABLE 6 Secondary Collisions and Barrier Function Related to Passenger Automobile Size

WHEELBASE, INCHES

SECOND EVENTS	< 96		96-101		102-111		112-120		> 120		TOTAL	
OVERTURNED	38	7.45%	31	5.76%	36	4.69%	26	5.57%	4	3.05%	172	6.10%
FIXED OBJECT	66	12.94%	94	17.47%	119	15.51%	86	18.42%	26	19.85%	465	16.50%
OTHER SECOND EVENT	4	.78%	1	.19%	5	.65%	4	.86%	4	3.05%	24	.85%
SECOND EVENT SUB T	108	21.18%	126	23.42%	160	20.86%	116	24.84%	34	25.95%	661	23.46%
NO SECOND EVENT	402	78.82%	412	76.58%	607	79.14%	351	75.16%	97	74.05%	2157	76.54%
TOTAL	510	100.00%	538	100.00%	767	100.00%	467	100.00%	131	100.00%	2818	100.00%

WEIGHT, lb.

SECOND EVENTS	< 2000		2000-2499		2500-3249		3250-3999		> 4000		TOTAL	
OVERTURNED	18	7.29%	37	6.34%	42	5.24%	43	6.53%	10	4.12%	172	6.10%
FIXED OBJECT	29	11.74%	85	14.55%	131	16.33%	114	17.33%	49	20.16%	465	16.50%
OTHER SECOND EVENT	2	.81%	3	.51%	3	.37%	9	1.37%	5	2.06%	24	.85%
SECOND EVENT SUB T	49	19.84%	125	21.40%	176	21.95%	166	25.23%	64	26.34%	661	23.46%
NO SECOND EVENT	198	80.16%	459	78.60%	626	78.05%	492	74.77%	179	73.66%	2157	76.54%
TOTAL	247	100.00%	584	100.00%	802	100.00%	658	100.00%	243	100.00%	2818	100.00%

WHEELBASE, INCHES

BARRIER FUNCTION	< 96		96-101		102-111		112-120		> 120		TOTAL	
REDIRECT	362	70.98%	387	71.80%	535	69.75%	303	64.88%	95	72.52%	1959	69.52%
STOP	42	8.24%	47	8.72%	71	9.26%	48	10.28%	7	5.34%	264	9.37%
CONTAINED SUB T	404	79.22%	434	80.52%	606	79.01%	351	75.16%	102	77.86%	2223	78.89%
SNAG	5	.98%	0	.00%	2	.26%	5	1.07%	0	.00%	14	.50%
PENETRATED	10	1.96%	17	3.15%	15	1.96%	16	3.43%	6	4.58%	76	2.70%
RAN UNDER	1	.20%	2	.37%	1	.13%	0	.00%	0	.00%	4	.14%
BROKE THROUGH	9	1.76%	12	2.23%	19	2.48%	16	3.43%	4	3.05%	66	2.34%
WENT OVER	27	5.29%	27	5.01%	35	4.56%	23	4.93%	12	9.16%	159	5.64%
PENETRATED SUB T	47	9.22%	58	10.76%	70	9.13%	55	11.78%	22	16.79%	305	10.82%
DEFLECTED TO F O	1	.20%	2	.37%	2	.26%	3	.64%	0	.00%	9	.32%
UNKNOWN	53	10.39%	45	8.35%	87	11.34%	53	11.35%	7	5.34%	267	9.47%
TOTAL	510	100.00%	539	100.00%	767	100.00%	467	100.00%	131	100.00%	2818	100.00%

WEIGHT, lb.

BARRIER FUNCTION	< 2000		2000-2499		2500-3249		3250-3999		> 4000		TOTAL	
REDIRECT	174	70.45%	427	73.12%	568	70.82%	435	66.11%	164	67.49%	1959	69.52%
STOP	23	9.31%	42	7.19%	81	10.10%	62	9.42%	19	7.82%	264	9.37%
CONTAINED SUB T	197	79.76%	469	80.31%	649	80.92%	497	75.53%	183	75.31%	2223	78.89%
SNAG	2	.81%	3	.51%	2	.25%	5	.76%	0	.00%	14	.50%
PENETRATED	6	2.43%	12	2.05%	21	2.62%	16	2.43%	10	4.12%	76	2.70%
RAN UNDER	1	.40%	0	.00%	2	.25%	1	.15%	0	.00%	4	.14%
BROKE THROUGH	2	.81%	15	2.57%	16	2.00%	21	3.19%	8	3.29%	66	2.34%
WENT OVER	15	6.07%	27	4.62%	36	4.49%	39	5.93%	23	9.47%	159	5.64%
PENETRATED SUB T	24	9.72%	54	9.25%	75	9.35%	77	11.70%	41	16.87%	305	10.82%
DEFLECTED TO F O	0	.00%	2	.34%	1	.12%	5	.76%	0	.00%	9	.32%
UNKNOWN	24	9.72%	56	9.59%	75	9.35%	74	11.25%	19	7.82%	267	9.47%
TOTAL	247	100.00%	584	100.00%	802	100.00%	658	100.00%	243	100.00%	2818	100.00%

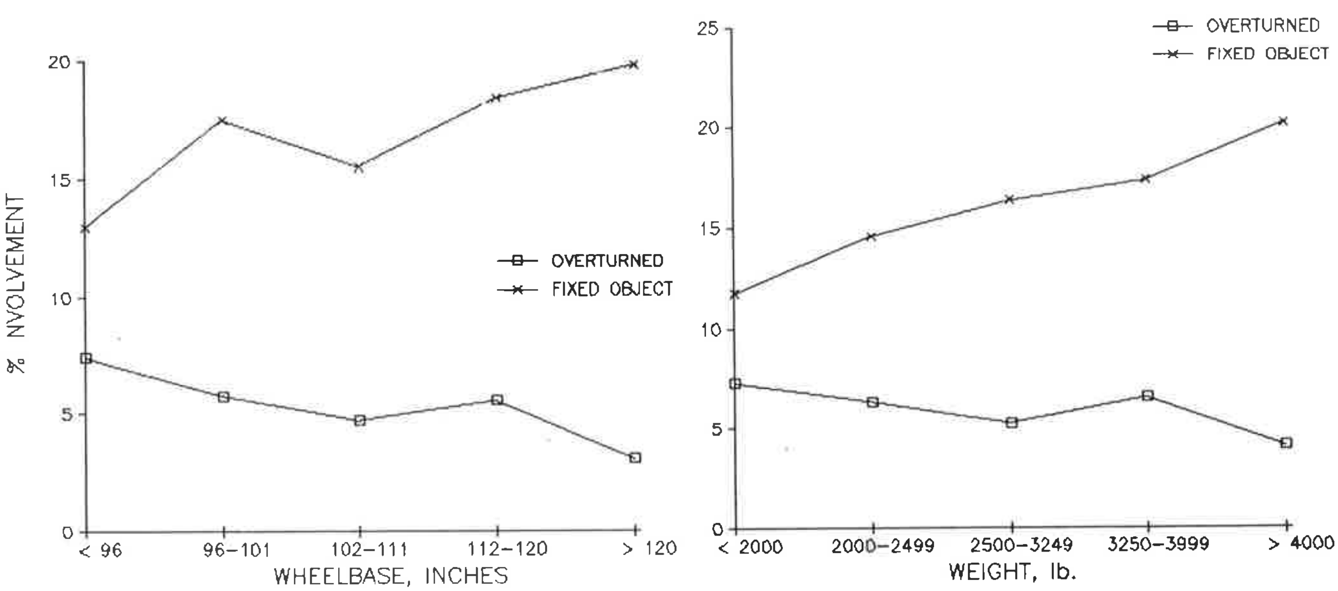

FIGURE 2 Effects of automobile size on secondary event involvement.

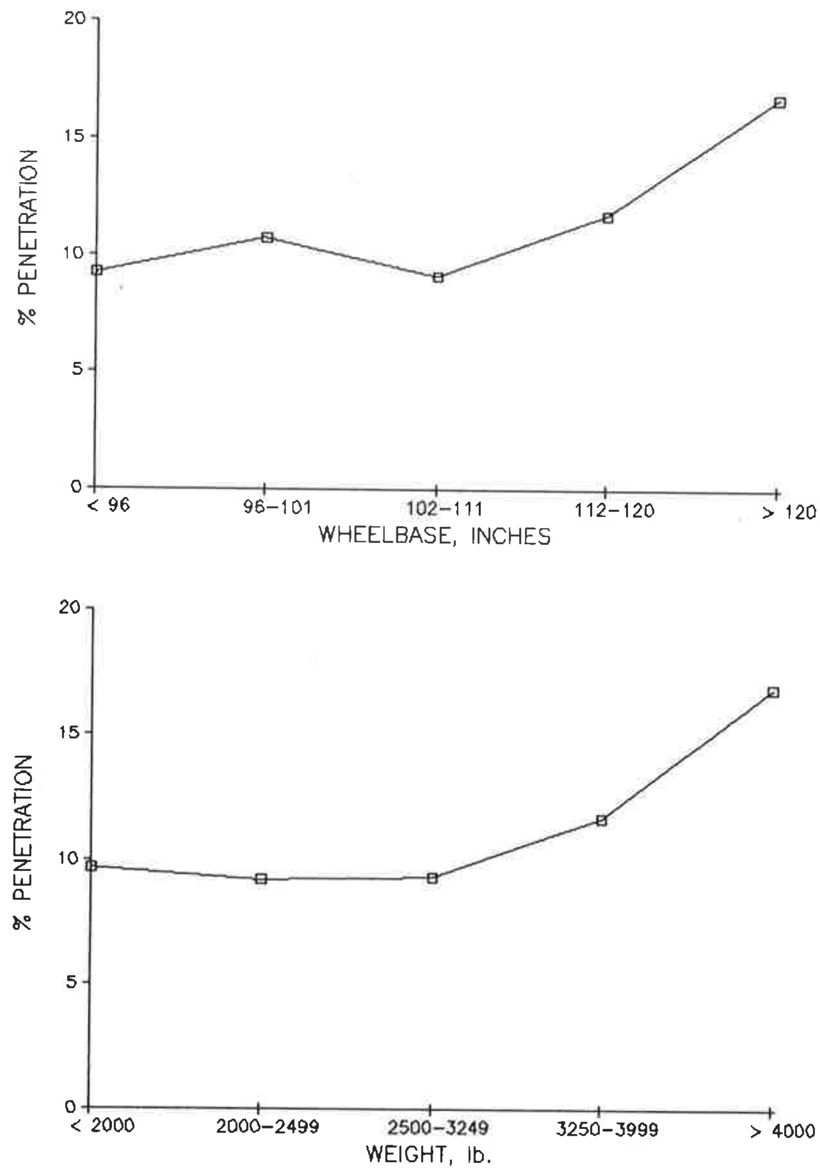

FIGURE 3 Effects of automobile size on barrier penetration.

was afforded motorcyclists, however, and nearly half the reported collisions resulted in severe injuries. Performance with vans and light trucks was likewise not as good as with passenger automobiles. Injury rates were higher, containment was achieved less often, and secondary collisions were more frequent. After passenger automobiles, vans and light trucks were the most common vehicle types involved; they accounted for nearly 10 percent of the accidents. Further examination of these results indicates that the better results experienced with passenger automobiles compared with vans and light trucks may relate primarily to vehicle weight, with center of gravity related to a lesser degree. Vans and light trucks are generally heavier than passenger automobiles and have a higher center of gravity. When results for these vehicles are compared with those for the heaviest passenger automobiles, the difference in performance is much less than when they are compared with results for all passenger automobiles. Injury rates are still higher than those for the heaviest passenger automobiles, but the differences are considerably less. Containment was nearly as good as for the heavy passenger automobiles, and the rate of secondary collisions with fixed objects was much closer. However, secondary rollovers are still quite high for vans and light trucks compared with the heaviest passenger automobiles, which indicates that the higher centers of gravity of these vehicles, compared with passenger automobiles, may be an important consideration in collisions with traffic barriers.

As expected, traffic barriers did not contain large trucks nearly as well as passenger automobiles and light trucks; fewer than half of these vehicles were contained compared with more than 80 percent of the passenger automobiles and three-fourths of the light trucks. Secondary collisions were reported in 60 percent of the heavy truck accidents. However, in spite of the low containment rate and frequent secondary collisions, the severe injury rate was similar to that for passenger automobiles, although the nonsevere injury rate was much higher. It appears that the large mass and relatively strong passenger compartments of these heavy vehicles may help to alleviate severe injuries, even though the collision event itself is more violent than for smaller vehicles.

Traffic barrier performance with small passenger automobiles has been an area of great concern because of the low mass, reduced vehicle stability, and lesser crush resistance of these vehicles compared with larger passenger automobiles. However, results of this investigation show that even the smallest passenger automobiles--those with wheelbases less than 96 in. and curb weights less than 2,000 lb--were provided good protection by traffic barriers. The highest severe injury rates were experienced by the largest, heaviest vehicles. It appears that this trend is related to barrier strength more than vehicle properties. Heavier automobiles were contained less often than lighter ones and experienced nearly twice as many secondary collisions with fixed objects. Although smaller automobiles experienced the most rollovers, this vehicle reaction was relatively scarce for all passenger vehicles and therefore did not affect injury rates to a large degree.

On the basis of the results of 3,302 traffic barrier accidents investigated in this study, the following findings can be stated:

1. Traffic barriers accidents resulted in lower injury rates than roadside accidents in general.
2. Current traffic barriers perform much better than older barriers.
3. Severity of occupant injuries was closely related to vehicle damage.
4. Satisfactory vehicle containment resulted in more than 75 percent of the cases.
5. About 25 percent of the cases involved a secondary collision.
6. Fixed object collisions were the most common second event, occurring in less than 18 percent of all accidents, followed by rollovers with less than 8 percent. Secondary collisions with other vehicles or pedestrians were extremely rare.
7. Injury rates were much higher for accidents involving lack of containment or secondary collisions.
8. Barriers performed best for passenger automobiles and exhibited reduced performance for vans and light trucks.
9. Injury rates were extremely high for motorcycle accidents.
10. In terms of vehicles containment and secondary collisions, barriers did not perform well with heavy trucks, although severe injury rates were about the same as for passenger automobiles.
11. Traffic barriers performed best with smaller passenger automobiles and showed some reduction in performance for larger automobiles.
12. The lower protection provided the largest passenger automobiles related to reduced vehicle containment and more frequent secondary events.
13. Small automobiles experienced more rollovers following traffic barrier collisions, but this event was still relatively rare and occurred in only about 7 percent of the collisions for the smallest vehicles.
14. Large passenger automobiles experienced more secondary collisions with fixed objects than smaller ones; 20 percent of all accidents with the largest cars involved such collisions.

ACKNOWLEDGMENT

This work was conducted in cooperation with the Federal Highway Administration. Bureau engineers and technicians participating in data collection and analysis were William G. Roth, Alan W. Rowley, Wayne R. Shrome, Robert P. Murray, David J. Leininger, Peter D. Kelly, Steven L. Roden, and William D. Nolan. The contribution of the State of New York Department of Motor Vehicles in supplying accident records is gratefully acknowledged. Valuable assistance was also received from Department of Transportation employees in the Traffic and Safety Division, Facilities Design Division, Data Services Bureau Photolog Unit, Regions 1 and 2, and various highway maintenance residencies. Valuable assistance was also provided by bureau employees Richard G. Picciocca, Gerald L. Anania, Tina M. Bohl, and Nicholas J. Bruno in developing computer analysis systems used in this project.

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Concrete Safety Shape with Metal Rail on Top To Redirect 80,000-lb Trucks

T. J. HIRSCH, W. L. FAIRBANKS, and C. E. BUTH

ABSTRACT

Because the concrete safety shape 32 in. (81 cm) high is a popular median and bridge barrier, it was desirable to see if it could be modified and strengthened to make it an effective traffic rail for trucks. A metal traffic rail 18 in. (46 cm) high was mounted on top of the 32-in. (81-cm) concrete safety shape to make a bridge rail 50 in. (127 cm) high to restrain and redirect 80,000-lb (36 287-kg) van-type trucks. The bridge rail was struck by such a truck at 48.4 mph (77.9 km/hr) at an angle of 14.5 degrees. The bridge rail did restrain the truck on the simulated bridge. The truck did roll on its side. This was attributed to the 9.5-in. (24-cm) setback of the metal rail from the sloping face of the concrete safety shape, which produced a roll angle of 11.3 degrees before the vehicle contacted the metal rail. The final position of the truck was parallel to and in front of the rail.

Current bridge rails are designed to restrain and redirect passenger automobiles only. Collisions of large trucks with these bridge rails have, in the past, led to catastrophic accidents. Concern for the reduction of the severity of these accidents has led highway designers to devote more attention to the containment and redirection of large trucks at selected locations. Several bridge rails that will restrain and redirect large trucks have been designed recently (1,2). Because the concrete safety shape 32 in. (81 cm) high is a popular median and bridge barrier, it was desirable to see if it could be modified to make an effective truck traffic rail.

The factors involved in the design of bridge rails to contain and redirect large trucks are not nearly as well understood or researched as those involved in the design of passenger automobile rails. Therefore it was the objective of this project to design, build, and test a bridge rail to contain and redirect an 80,000-lb (36 287-kg) van-type tractor-trailer, as shown in Figure 1. The design was based on data presented elsewhere (1-7).

The combination rail selected was a modification of the Texas Type T5 traffic rail with a modified Texas Type C4 metal traffic rail mounted on top. The modified T5 rail consists of a concrete safety shaped parapet 32 in. (81.3 cm) high. The concrete parapet was thickened to 10.5 in. (27 cm) at the top and 20 in. (51 cm) at the bottom and contains a large amount of reinforcing steel. This provides both flexibility and strength, thus minimizing cracking of the concrete and permanent deflection of the rail when struck by heavy vehicles. The thickness of the bridge deck below the concrete parapet was increased to minimize cracking and provide greater strength.

DESIGN TECHNIQUE

Earlier tests have shown that the greatest forces generated during the redirection of tractor-trailer vehicles occur when the tandem axles of the tractor and the front of the trailer strike the bridge railing. A relatively small part of the total kinetic energy is expended in the redirection of the front axle of the tractor, and the rear tandem axles of the trailer have an even smaller impact. Given that the total loaded weight on the tandem axles of the tractor would be approximately 34,000 lb (15 436 kg) (Figure 1), it was assumed that 10,000 lb (4 540 kg) of this load would probably be transferred to the rail through the wheels and the axles. The remaining 24,000 lb (10 896 kg) would be transferred to the rail through the bed of the van trailer.

Accelerometer data from past tests indicated that the tandem axles of the tractor would be subjected to a maximum average 50-msec lateral acceleration of about 6 *g*'s. Therefore equivalent static design forces of 60,000 lb (27 240 kg) (10,000 lb x 6 *g*'s) applied at a height of 21 in. (53.3 cm) and 144,000 lb (65 376 kg) (24,000 lb x 6 *g*'s) applied at a height of 47.6 in. (120.9 cm) were used to design the rail using yield line theory for reinforced concrete. These procedures are outlined elsewhere (3).

DESCRIPTION OF BRIDGE RAIL AND DECK MODIFICATIONS

The modified T5 rail has a modified Texas Type C4 metal rail 18 in. (45.7 cm) tall mounted on top. This makes a combination bridge rail 50 in. (127 cm) tall that is designed to retain large 80,000-lb (36 287-kg) van-type trucks or tractor-trailers striking at 15 degrees and 50 mph (80.5 km/hr). Drawings of this rail are shown in Figures 2-4. Figure 5 shows photographs that compare the size of this bridge rail with the van-type tractor-trailer.

The concrete parapet was basically a standard Texas Type T5 traffic rail that was thickened to 10.5 in. (26.7 cm) at the top and 20 in. (50.8 cm) at the bottom. It was anchored to the bridge deck by No. 5 stirrups spaced at 8 in. (20 cm) as shown, and eight No. 6 longitudinal bars were used.

The metal rail mounted on top of the modified T5 concrete rail was a standard Texas Type C4 metal traffic rail with three modifications as shown in Figure 3. The first modification involved the use of one additional steel post plate 1 in. (2.54 cm) thick

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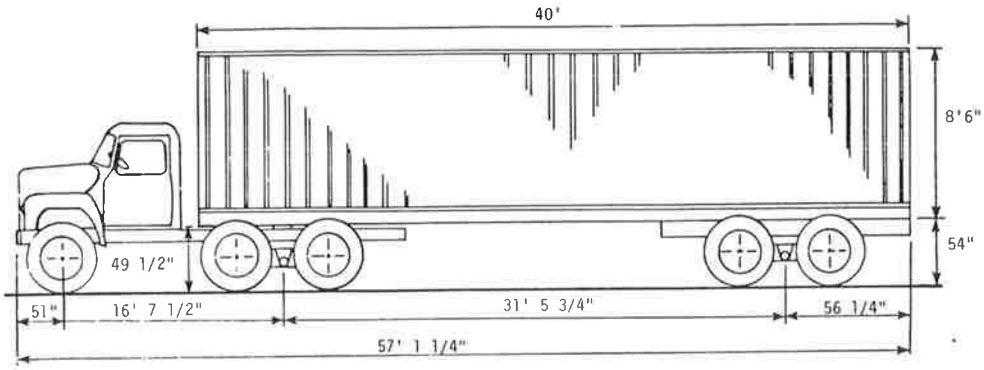


EMPTY WEIGHTS	
Tractor only	18,320 lb
Trailer only	13,760 lb
Total Empty Weight	32,080 lb
LOADED WEIGHTS	
Weight on front axle	12,020 lb
Weight on Center axles	34,170 lb
Weight on rear axles	33,890 lb
Total Loaded Weight	80,080 lb

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

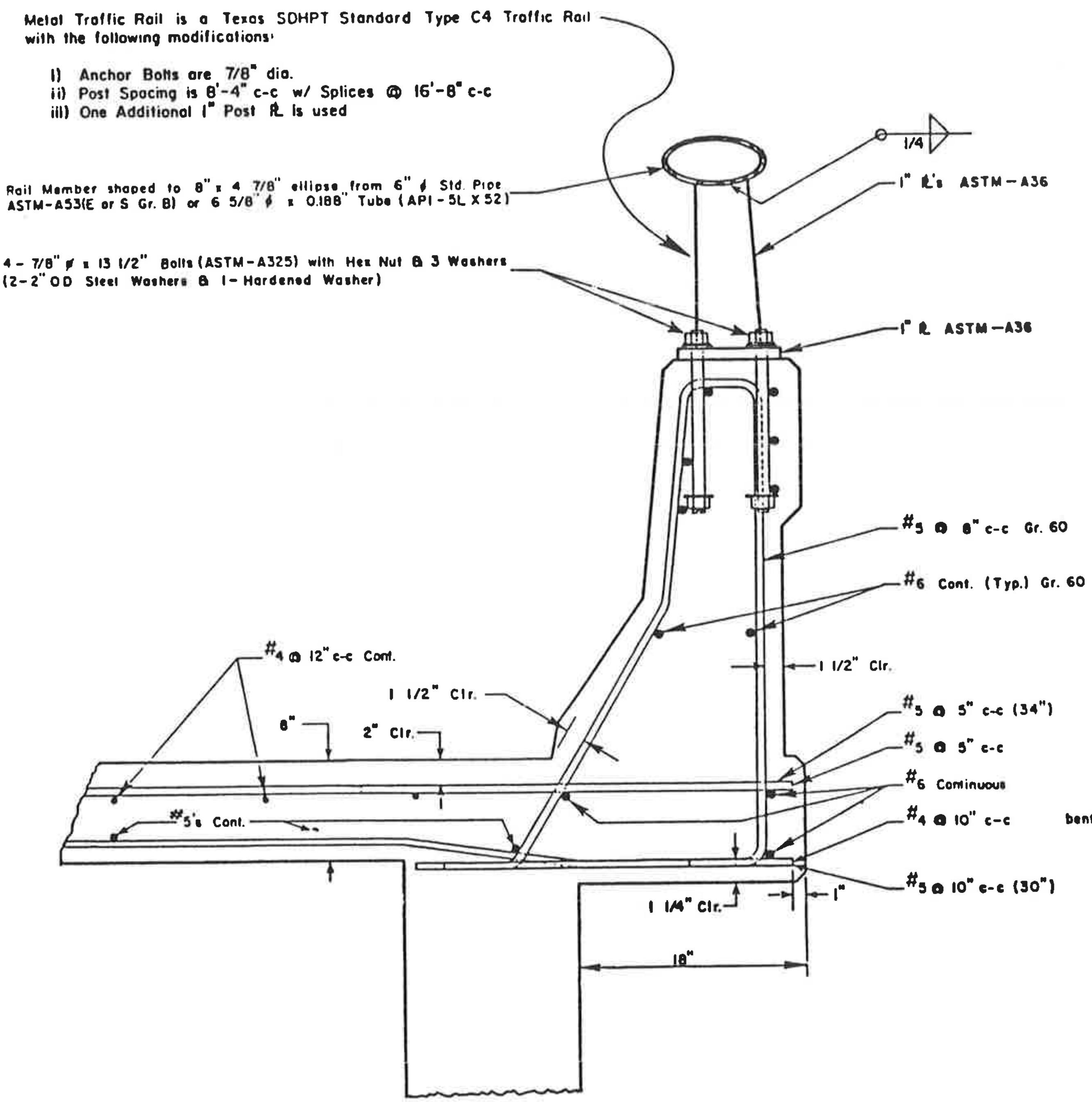


FIGURE 2 Cross section of the modified T5 bridge rail.

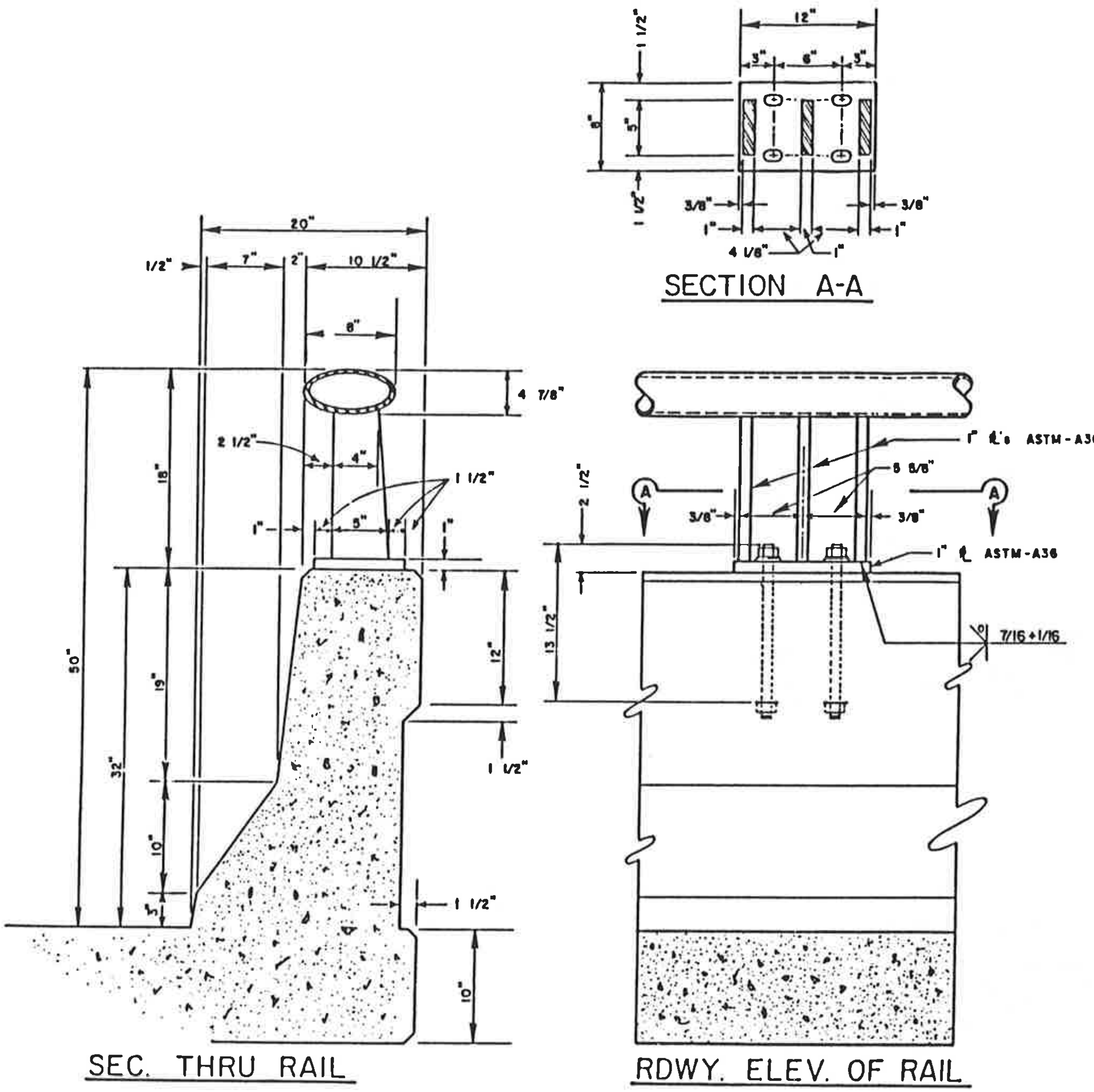


FIGURE 3 Dimensions and elevation of the modified T5 bridge rail.

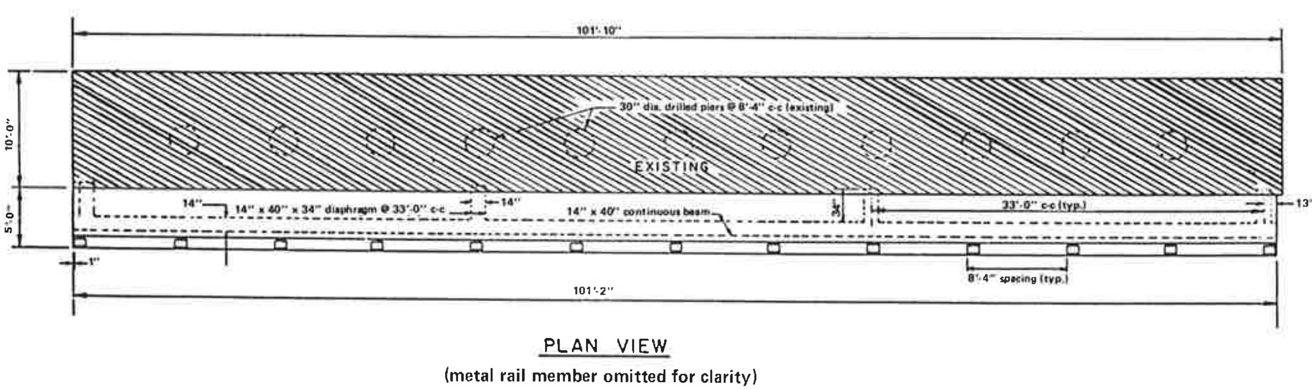


FIGURE 4 Plan view of the modified T5 bridge rail.

FIGURE 5 Comparison of 80,000-lb van-type truck and modified T5 bridge rail.

(ASTM-A36). This modification brought the total number of post plates used in each post to three. The second modification was the use of ASTM-A325 bolts 7/8 in. (2.2 cm) in diameter in place of the standard 3/4-in. (1.9-cm) bolts. The last modification was the reduction of the post spacing from 10 ft (3 m) to 8 ft 4 in. (2.5 m). These modifications were made for the purpose of increasing the strength of the metal rail so that it could provide a greater resistance to overturning by the van trailer.

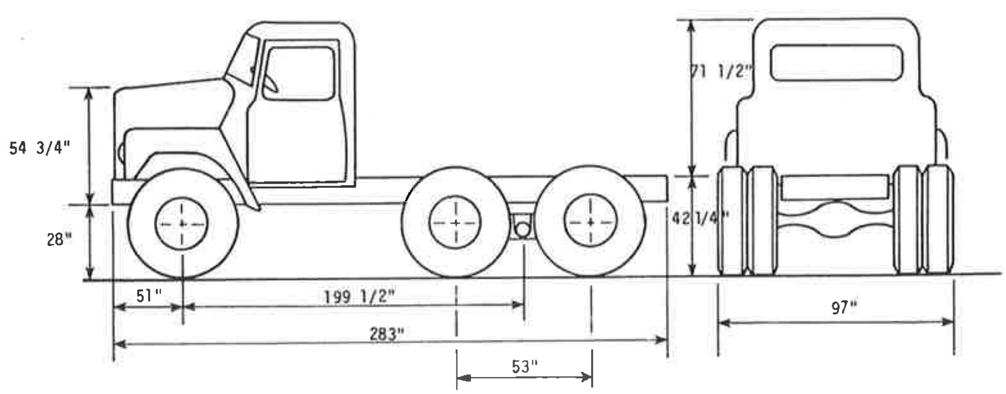
The metal rail was fabricated from standard steel pipe 6 in. (15 cm) in diameter (ASTM A53 Grade B) shaped into an ellipse 8 in. x 4 7/8 in. (20 cm x 12.4 cm) and welded to the modified post mentioned previously. These posts were in turn welded to a base plate made of steel plate 1 in. (2.54 cm) thick (ASTM-A36). The posts were anchored to the concrete rail by means of four A325 bolts 7/8 in. (2.2 cm) in diameter by 13.5 in. (34.3 cm) long. One steel washer 2 in. (5.1 cm) in diameter and one hardened steel washer were installed under each bolt nut.

The strength of the Texas standard bridge deck 7 in. (18 cm) thick was increased in many ways. The dimensions and reinforcement pattern of the standard bridge deck were essentially maintained throughout except in the cantilever portion of the deck. These changes are detailed in Figure 2. The length of the cantilever portion was decreased from 30 in. (76 cm) to 18 in. (46 cm), and the thickness was increased to 10 in. (25.4 cm). The size of the upper transverse bars was maintained at No. 5s, and the standard 5-in. (12.7-cm) spacing was decreased to 2.5 in. (6.4 cm). The lower transverse reinforcement consisted of an alternating pattern of bent No. 4s that extended into the lower portion of the bridge deck and straight No. 5s, each at a spacing of 10 in. (25.4 cm). The size of the upper and lower longitudinal bars was increased to No. 6s from No. 4s and No. 5s, and the spacing was increased from 12 in. (30.5 cm) to 16.5 in. (41.9 cm).

All reinforcing bars used in the bridge rail had a minimum yield strength of 60 ksi (413.4 MPa), and the bridge deck reinforcement had a minimum yield strength of 40 ksi (275.6 MPa). It should be noted that all of the 28-day compressive strengths were well above the minimum specified strength of 3,600 psi (24.8 MPa).

INSTRUMENTATION AND DATA ANALYSIS

The vehicle was equipped with triaxial accelerometers mounted above the tractor tandem wheels. Yaw, pitch, and roll were sensed by on-board gyroscopic instruments. The electronic signals were telemetered to a base station for recording on magnetic tape and for



EMPTY WEIGHTS

Tractor only	18,320 lb
Trailer only	13,760 lb
Total Empty Weight	32,080 lb

FIGURE 6 Empty tractor dimensions and weights.

display on a real-time strip chart. Provision was made for transmission of calibration signals before and after the test, and an accurate time reference signal was simultaneously recorded with the data.

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

Data from the electronic transducers was digitized, using a Southwest Technical Products 6800 microcomputer, for analysis and evaluation of performance. Several computer programs were used to process various types of data from the test vehicle.

Still and motion photography were used to document the test, to obtain time-displacement data, and to observe phenomena that occurred during the impact.

FIGURE 7 80,000-lb truck before and after test.

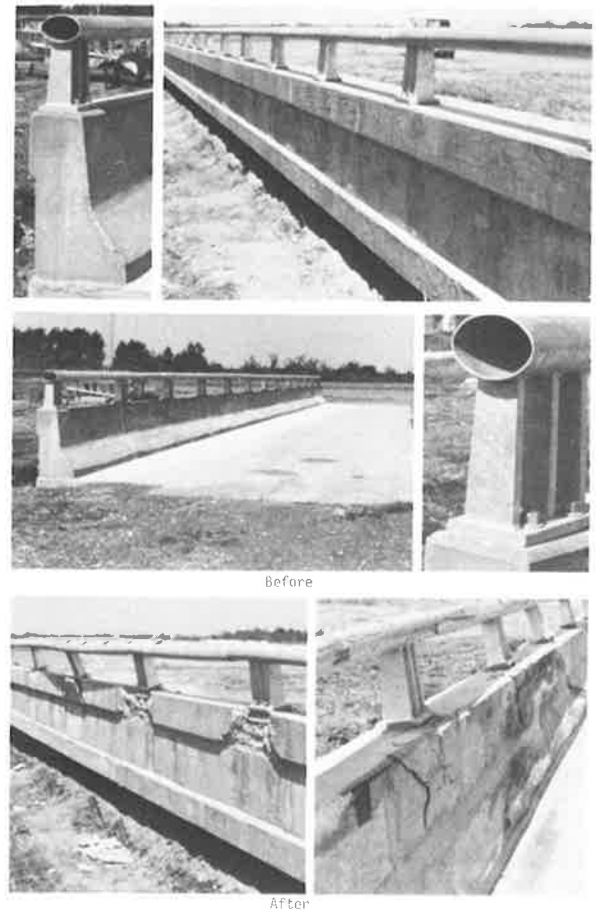


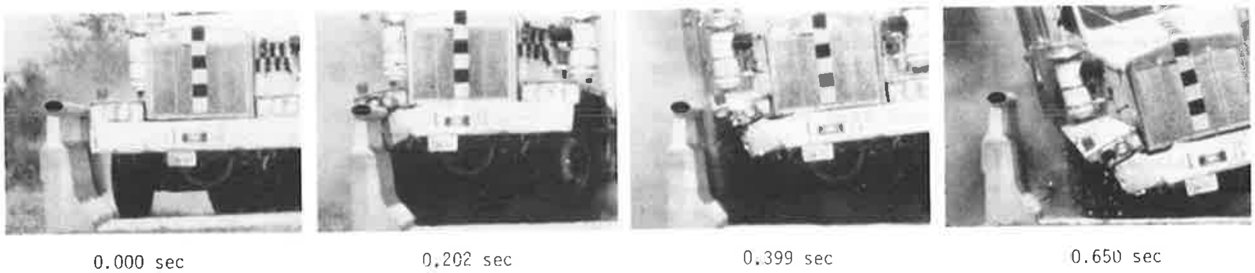
FIGURE 8 Bridge rail before and after test.

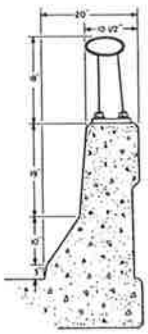
Still photography was used to record conditions of the test vehicle and bridge rail installation before and after the test. Motion photography was used to record the collision event.

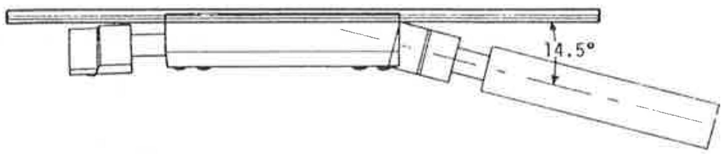
TRUCK CRASH TEST

This bridge rail system was designed to contain and redirect an 80,000-lb (36 287-kg) van-type tractor-trailer. A simulated bridge deck with this rail system was built at the Texas Transportation Institute proving grounds and tested with a 1981 Kenworth tractor-trailer ballasted with sand bags to 80,080 lb (36 356 kg). Drawings showing the dimensions of this vehicle along with loaded and unloaded weights on each axle or pair of axles are shown in Figures 1 and 6. Before-and-after test photographs of the truck are shown in Figure 7.

The truck struck the rail at 48.4 mph (77.9 km/hr) at a 14.5-degree angle. The impact point was 26 in. (66 cm) downstream from Post 5, and the truck was contained and redirected. The tractor-trailer did, however, roll 90 degrees and came to rest on its side approximately 175 ft (53 m) from the impact point. Figure 8 shows the bridge rail and test site immediately after the test. The truck sustained damage to the right front and right tandem wheels. The cab of the truck remained intact. A summary of the crash test data is shown in Figure 9.







Test No. 2416-1
Date 9/18/84
Test Installation. . . . . Mod. Texas Type T5 Bridge Rail w/Mod. Texas Type C4 Metal Rail
Length of Installation . . . . 101.2 ft (30.8 m)
Metal Rail Deflection
Permanent . . . . 0.5 ft (0.2 m)
Maximum . . . . 0.9 ft (0.3 m)
Vehicle. . . . . 1981 Kenworth Tractor with Freuhauf Van-type Trailer

Vehicle Weights
Empty Weight . . . . . . 32,080 lbs (14,564 kg)
Gross Static . . . . . . 80,080 lbs (36,356 kg)
Impact Speed . . . . 48.4 mph (77.9 km/h)
Impact Angle . . . . 14.5 deg
Tractor Accelerations at Drive Axles
(Max. 0.050 sec Avg)
Longitudinal . . . . . . -2.4 g
Lateral. . . . . 5.5 g
Vertical . . . . 3.9 g
Max. Roll Angle. . . . . 90 deg

FIGURE 9 Summary of data for Test 2416-1.

The bridge deck supporting the rail sustained no damage. The concrete parapet was not significantly damaged, but the metal rail experienced damage between Posts 5 and 8 (Figure 10). It was determined from the overhead film that the metal rail was deflected a maximum of 11 in. (27.9 cm) and sustained a permanent deflection of 6 in. (15.2 cm). The concrete rail was permanently displaced 0.5 in. (1.3 cm). The threads were stripped from the trafficside anchor nuts of Posts 5 and 6 of the metal rail. Examination revealed that the thread fit was too loose on the bolts 7/8 in. (2.2 cm) in diameter

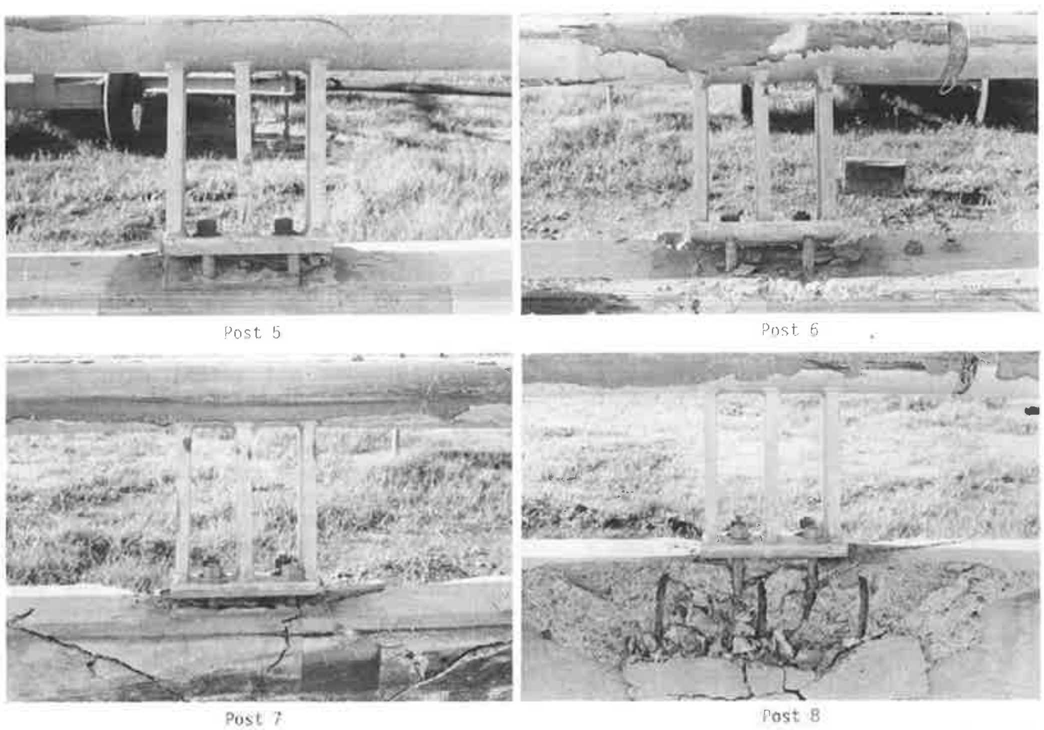


FIGURE 10 Posts 5-8 after test.

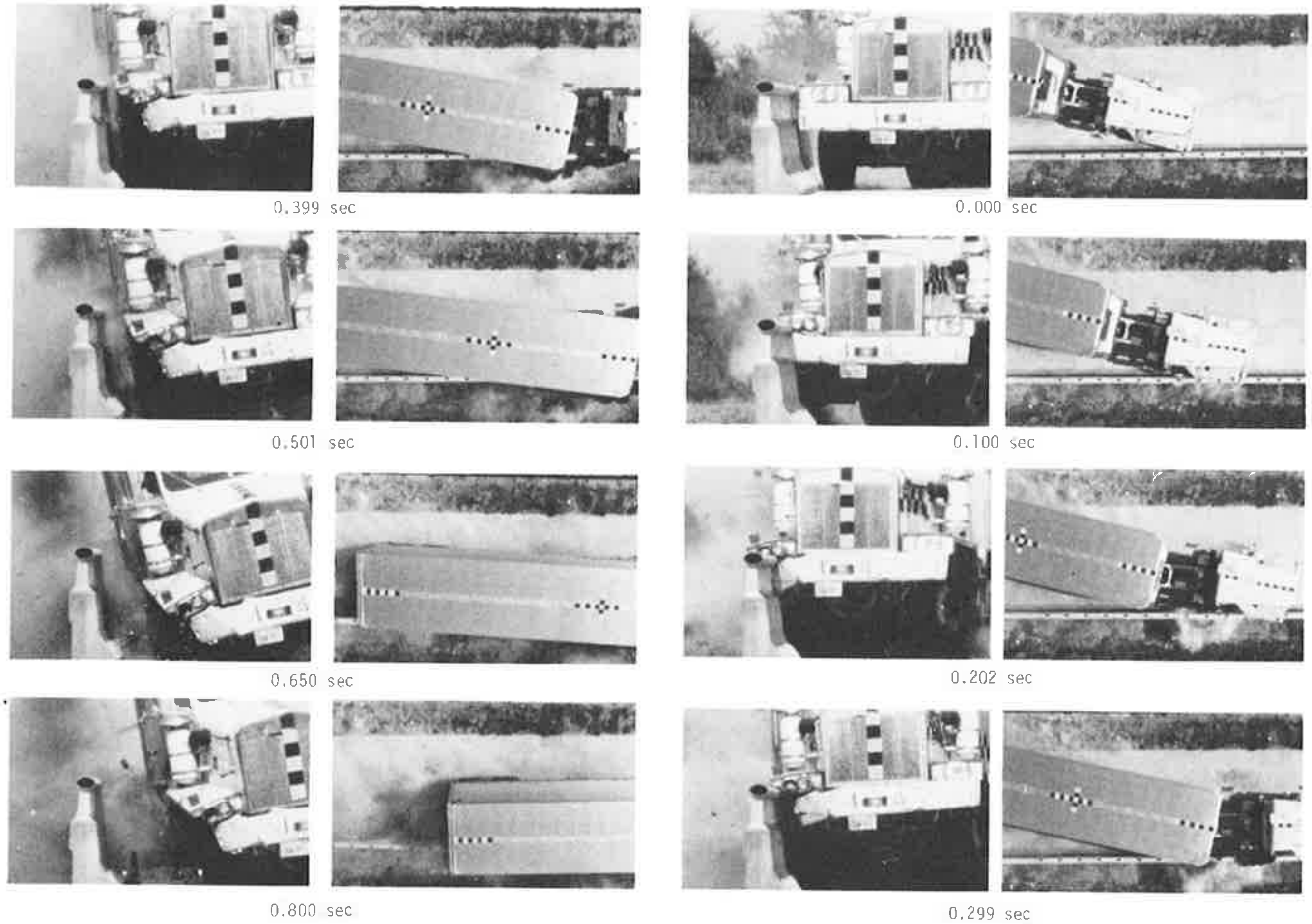


FIGURE 11 Sequential photographs of Test 2416-1.

anchoring the metal posts. This problem has occurred with some previous tests, and laboratory experiments indicated that the bolts with the improper nut fit developed only 75 percent of the ultimate tensile strength developed by those bolts with proper nut fit. The trafficside anchor bolts of Posts 6 and 7 pulled loose from the concrete parapet. Figure 11 is sequential photographs showing overhead and frontal views of the crash test.

Maximum positive roll of the tractor tandem axles and the trailer was 90 degrees. From the accelerometers, the longitudinal and lateral maximum average 0.050-sec accelerations were -2.4 g's and 5.5 g's, respectively. Graphs of the filtered data from the yaw, pitch, and roll rate gyroscopes and the x, y, and z accelerometers are shown in Figures 12-15.

DISCUSSION OF RESULTS

NCHRP Report 230 recommends the following criteria for Test S20 (80,000 lb/50 mph/15 degrees) (5,p.10):

1. Test article shall smoothly redirect the vehicle; the vehicle shall not penetrate or go over the installation.
2. Detached elements, fragments or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
3. Vehicle, cargo, and debris shall be contained on traffic side of barrier.

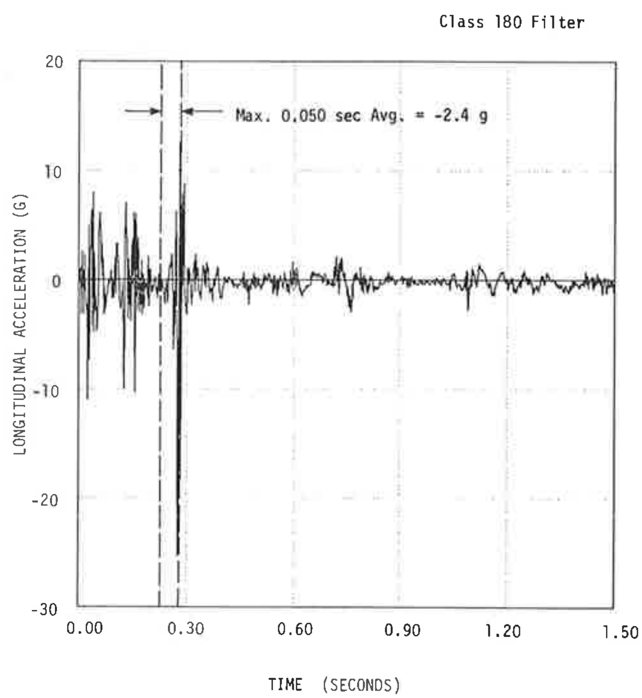


FIGURE 12 Vehicle longitudinal accelerometer trace for Test 2416-1.

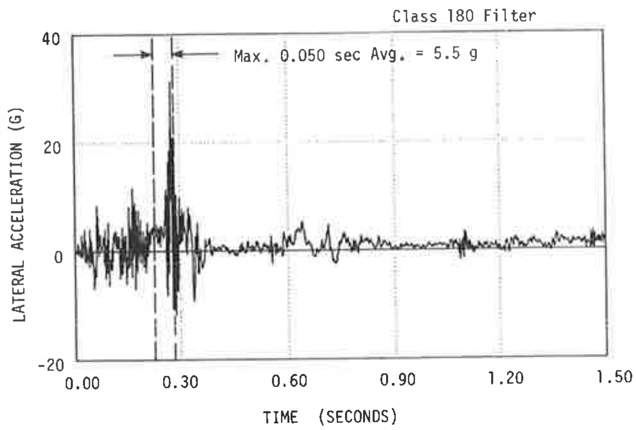


FIGURE 13 Vehicle lateral accelerometer trace for Test 2416-1.

According to these criteria the test was a success even though the truck rolled on its side. The bridge rail contained and redirected the truck and remained totally intact while doing so. The roll of the truck is attributed to the sloping face of the concrete safety shape. The metal traffic rail is set back 9 1/2 in. (24 cm) from the lower face of the concrete shape 47 1/2 in. (121 cm) below. This means the trailer undergoes a roll angle of 11.3 degrees ($\tan^{-1}$ 9.5/47.5) before it contacts the metal rail. Hirsch and Arnold (1) report that where the redirection face of the rail was vertical no roll was experienced.

Impact severity as defined by the occupant flail space approach was also computed from the accelerometer data. The recommended threshold values for the flail space evaluation of passenger automobiles are 40 and 30 fps, respectively, for the longitudinal and lateral occupant impact velocity and 20 g's for the highest 10-msec average deceleration after contact. The computed values for this test were well below these recommended values. The longitudinal occupant impact velocity was 6.59 fps, and the highest 10-msec average occupant acceleration after contact was -2.34 g's. The lateral occupant impact velocity was 15.49 fps, and the highest 10-msec average acceleration was 5.6 g's. These recommended

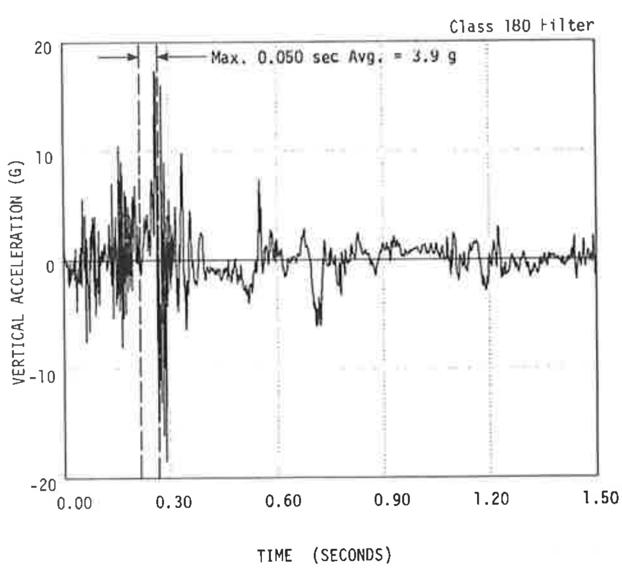


FIGURE 14 Vehicle vertical accelerometer trace for Test 2416-1.

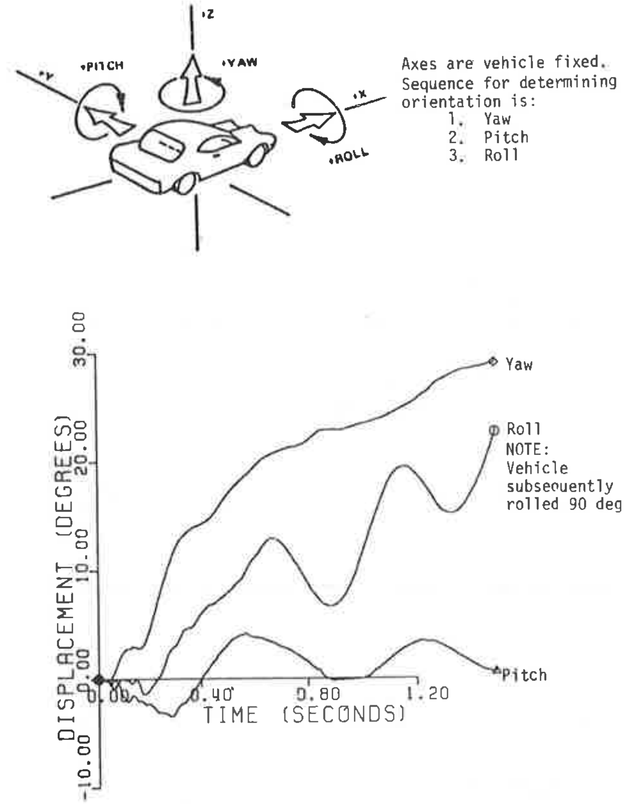


FIGURE 15 Vehicle angular displacement for Test 2416-1.

threshold values do not apply to large trucks. They are presented here only for comparison purposes.

SUMMARY AND CONCLUSIONS

A standard Texas Type T5 traffic rail was modified by increasing its strength and effective height so that it could restrain and redirect an 80,000-lb (36 287-kg) van-type truck or tractor-trailer. The concrete parapet was 32 in. (81.3 cm) tall, and total rail height was 50 in. (127 cm).

The crash test was conducted on this bridge rail with an 80,080-lb (36 356-kg) van-type tractor-trailer striking the rail at 48.4 mph (77.9 km/hr) at an impact angle of 14.5 degrees. The vehicle was restrained, redirected, and came to rest on its side approximately 175 ft. (53 m) from the impact point. Although the truck rollover was not desirable, the bridge rail did meet the S20 criteria of NCHRP Report 230 (5).

The four ASTM-A325 anchor bolts 7/8 in. (2.2 cm) in diameter by 13 1/2 in. (34.3 cm) long used at each post had two deficiencies. The threads on the bolts were cut too loose (not according to specifications) and permitted the nuts to be stripped off at two posts. The anchor bolts were not long enough to develop their strength. The length of 13 1/2 in. (34.3 cm) should be increased to at least 18 in. (46 cm) to increase the development length.

This test has shown that a bridge rail can be built with the concrete safety shape on a slightly modified Texas standard bridge deck to contain large van-type tractor-trailer trucks.

The cross-sectional area of this modified concrete safety shape is approximately 2.8 ft² (0.26 m²) compared with approximately 2.5 ft² (0.23 m²) for a standard Texas traffic rail Type T5. The cost of this modified rail would be approximately $80 per linear foot, whereas a standard Texas Type T5 traffic rail normally costs about $35 per linear foot.

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Crash Cushion Improvement Priority and Performance Evaluation

MIKE Y. HOUH, KENNETH M. EPSTEIN, and JOE LEE

ABSTRACT

Traffic impact attenuators play a vital role in highway safety. When properly engineered, located, and maintained, impact attenuators can result in the savings of numerous lives and reductions in property damage. However, there have been widespread improper application and use of impact attenuators. The study on which this paper is based focused on location, design, and maintenance of traffic impact attenuators. As a case study, the current management and operational procedures used for the Highway Safety Appurtenances Replacement program of the District of Columbia were evaluated. Traffic characteristics and roadway environmental features that contribute to roadside collisions were identified and analyzed using a multiple regression technique. The analysis revealed that street light luminance, truck percentage, radius of horizontal roadway curvature, and attenuator offset distance are the factors most correlated to roadside collision incidents.

Impact attenuator systems are defined by AASHTO as "protective systems which prevent errant vehicles from impacting hazards by either smoothly decelerating the vehicle to a stop when hit head-on, or by redirecting it away from the hazard for glancing impacts." Many sources have shown that the installation of impact attenuators has proven to be a cost-effective means of saving lives and reducing the severity of fixed-object accidents. For example, the 1981 Highway Safety Stewardship Report (1) ranked it as the second most effective highway safety improvement, with a benefit-to-cost ratio of 3.1. Despite the effectiveness of impact attenuators, problems in location, design, field inspection, maintenance, and performance evaluation still exist.

In the past most crash cushions were installed at locations where the most obvious crash potential existed. As these obviously dangerous locations are improved, the most cost-effective locations in which to install future impact attenuators become less apparent. It can be difficult to identify these locations through the application of common sense and engineering judgment. Recent field reviews of impact

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attenuators in the District of Columbia reveal that the recently installed crash cushions have lower damage rates than do those at older locations. This finding is consistent with the nationwide trend of declining benefit-to-cost ratios for impact attenuator installations. To continue locating impact attenuators in a cost-effective manner, definite procedures are needed for determining disproportionate representations of accidents associated with roadway features and traffic conditions.

The performance of a crash cushion during impact is dependent on its precrash, on-site condition. In other words, small changes in the field installation from the original design or improper maintenance may totally destroy the intended performance. It is therefore desirable that damage or on-site defects be reported and corrected immediately. However, because the frequency with which attenuators are struck varies by season and location, a fixed-schedule field check may never meet the urgently needed fast-reporting requirement. To eliminate the risk that a damaged crash cushion remains undetected or unrepaired until another accident occurs, it is critical to determine the optimal frequency of field inspections and maintenance.

Another problem involved in the evaluation of impact attenuator performance is that a large portion of attenuator collisions go unreported because a well-designed crash cushion can reduce the severity of damage and allow the errant vehicle to be driven away after impact. In 1983 it was found that only 24 attenuator-related accidents were reported by the District of Columbia's Metropolitan Police Department. Thus reported accidents do not reflect the actual number of accidents involving impact attenuators and the extent of damage. To obtain the real picture of impact attenuator performance in the field, an attenuator data inventory was created to collect all available information for analysis.

RESEARCH OBJECTIVES

The goal of this study was to determine problems of location, design, and maintenance associated with the District of Columbia's impact attenuation systems. As a case study, the current management and operational procedures used for the Highway Safety Appurtenances Replacement (HSAR) program were evaluated.

The first objective of this study was to develop procedures for checking the performance of impact attenuators installed on D.C. highways.

The second objective was to evaluate the impact frequency and damage severity for each study location and to determine a maintenance schedule and an inspection interval for the D.C. attenuator system in order to reduce the possibility of unprotected hits.

The third objective was to identify the major descriptors that represent the relationships between crash cushion accidents and roadway environmental factors, so that a mathematical model could be developed to predict the accident potential of a specific location.

SCOPE OF STUDY AND DATA COLLECTION PROCEDURE

The study was restricted to the 88 impact attenuators located on those D.C. highways that were opened to traffic when the study began. All of the field data for each location were collected in 1,000-ft roadway sections measured upstream from the attenuators. The roadway and traffic descriptors downstream from the attenuators are considered to have less effect on attenuator accidents. Therefore this information is not recorded except for roadway illumination data, which may affect a driver's visibility. The illumination data were measured in 600- x 100-ft rectangular areas centered at the nose of each attenuator. A total of 29 data items were collected in the attenuator inventory. These data items were as follows:

A. Permanent data
   1. Basic attenuator information (BAI)
      - Assigned location number
      - Location name
      - Quadrant code
      - Maryland grid number
      - Attenuator type
      - Street classification
      - Installation and/or upgrade costs
      - Installation and/or upgrade dates
      - Field check route number
   2. Traffic control and characteristics (TCC)
      - Average daily traffic (ADT)
      - Truck percentage
      - Attenuator design speed
      - No-passing zone length (including cross hatch area)
      - Number of traffic lanes
      - Traffic control signs
      - Object signs and indications
   3. Geometric factors and pavement conditions (GFPC)
      - Curb height
      - Gradient
      - Horizontal alignment
      - Offset distance
      - Pavement conditions and skid index
      - Street light luminance
      - Object type

B. Accident and maintenance records
   1. Detailed damage condition (number of hits)
   2. Number of times that maintenance was performed
   3. Maintenance costs
   4. Reported accidents
   5. Number of injuries

During FY 1984 (October 1, 1983, to September 30, 1984), a total of 10 field inspections were conducted. It was found that the study locations had been struck 158 times (excluding brush hits). Only 19 attenuator-related accidents were reported by the various police agencies, about 12 percent of the total hits. These accidents included 11 injury accidents and 8 property-damage-only (PDO) accidents. None of these accidents resulted in fatalities, and only 13 injuries were reported. Sixty-seven repairs have been performed on the attenuators, about 43 percent of the total number that were damaged. Of the 158 hits, 96 (61 percent) occurred at locations considered fully protected and 62 (39 percent) occurred at unprotected locations.

RESEARCH PROCEDURE AND METHODOLOGY

After the data were collected, the following methods were used to gain the research objectives:

1. Before-and-after study to determine the effectiveness of the D.C. attenuator system,
2. Benefit-cost analyses to evaluate the economic effectiveness of each attenuator installation,
3. Frequency study to optimize the inspection and maintenance schedules for attenuator system operation,
4. Multiple regression and correlation analyses to determine the influences of roadway and traffic factors on attenuator accidents, and

5. Bivariate sortings to analyze the influences of nonnumerical factors on attenuator accidents.

The before-and-after study is a comparison between the expected accident rate and the actual reported accident rate. In this study, the number of attenuator hits observed in the field was used as the expected accidents. The calculations were done by a spread-sheet program. The computed results were then interpolated on the Poisson curve chart to determine whether the changes were significant at the selected level of significance (Figure 1). To convert the number of hits to accident severities, the following assumptions were used:

- Any visible damage on the attenuator, except brush hits, is treated as a property damage accident;
- Any head-on impact with a deceleration force higher than 10 g is treated as an injury accident;
- Any angular impact with a deceleration force higher than 5 g is treated as an injury accident;
- A strike near the rear end of an attenuator is treated as an injury accident;
- Brush hits are ignored because the driver may successfully avoid the fixed object if no crash cushion is installed; and
- No fatality was assumed because, in the D.C. accident rating procedure, fatalities are rated equally with injuries.

These assumptions are made on the basis of information provided by the 1975 FHWA publication, Crash Cushions: Selection Criteria and Design (2) and the 1984 report, Safer Bridge Railings (3).

The benefit-cost analysis computes the payback ratio of each dollar spent on attenuators. The reduction of accident costs (i.e., the difference between expected accident costs and reported accident costs) is the real contribution of the attenuator system. The reductions were treated as the benefit in the benefit-to-cost (B/C) ratio calculation. The 1982 motor vehicle accident costs ($8,000 for an injury and $1,090 for a PDO accident), published by the National Safety Council (NSC), were used to estimate the benefits. Only one injury was assumed for each injury accident. This is a conservative estimate given that the average vehicle occupancy in the District of Columbia is 1.41 persons per vehicle. These motor vehicle accident costs were treated as the present worth when the calculations were made. These costs were then directly converted into the equivalent uniform annual benefit (EUAB). The attenuator costs include capital costs, maintenance costs, and expenses for inspection and reporting. An annual interest rate of 8 percent and a service life of 10 (for sand barrel systems) to 12 (for other types of crash cushions) years were used to determine the capital recovery factor (CRF) for different attenuator sites. No salvage value was included in the calculation of equivalent uniform annual cost (EUAC) because the cost to remove an old attenuator is higher than its residual value.

The frequency studies involve calculations of the probability of damage occurring during a specific inspection interval and the required repair time for each attenuation system before the second hit takes place. Assuming that attenuator accidents are random events, the probability of occurrence of an accident at a particular location during a specific time period can be predicted by using the Poisson equation if the annual number of hits is known. In this case, the Poisson distribution can be expressed as follows (4):

$$P(n) = H^n * e^{-H}/n!$$

where

H = number of hits expected in t days,
n = accident occurrences of interest,
e = natural logarithm base (e = 2.71828), and
$n!$ = n factorial (1*2*3 . . . n).

For each impact attenuator location, the expected number of hits during t days (H) can be calculated as

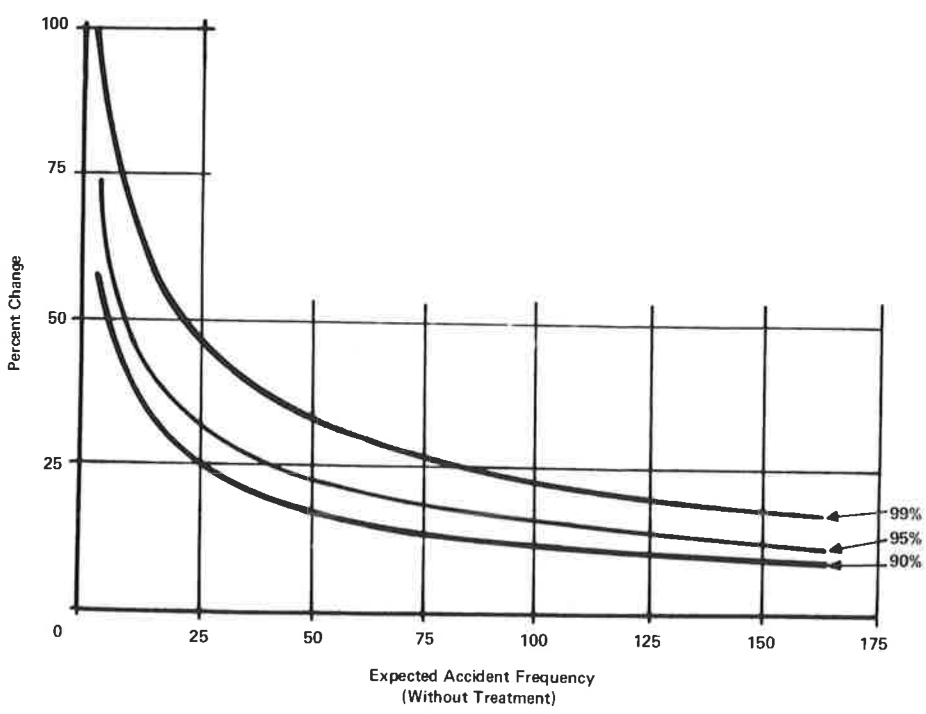

FIGURE 1 Poisson curves.

$$H = \bar{H} * t/365$$

where $\bar{H}$ is the annual number of hits and t is the time period of interest.

The attenuator maintenance interval such that no more than one hit (t_{0-1}) will occur can be determined by the following formulas:

$$t_0 = -[365 * \ln P(0)]/\bar{H}$$

$$t_1 = -[365 * \ln P(1)]/\bar{H}$$

$$t_{0-1} = t_0 + t_1$$

where t_0 is the time interval during which no hit occurs and t_1 is the time interval during which only one hit occurs.

For each location, 10 sets of data with 12 roadway and traffic variables were included in the multiple regression and correlation analyses. Two dependent variables, the number of hits and accident cost, were tested against the 12 independent variables. A brief description of these variables follows.

Dependent variables
- Y1 = Number of hits
- Y2 = Accident costs for attenuator locations

Independent variables
- X1 = Average daily traffic (1,000s)
- X2 = Truck percentage
- X3 = Attenuator design speed (average running speed + 5 mph)
- X4 = Number of approaching traffic lanes
- X5 = Pavement service index
- X6 = Curb height (in.)
- X7 = Roadway gradient variance (percentage)
- X8 = Horizontal curvature radius (100 ft)
- X9 = Street light luminance (footcandles)
- X10 = Length of no-passing zone (100 ft)
- X11 = Offset distance (ft)
- X12 = Skid index measured at 40 mph

Both correlation and regression analyses were performed on an IBM PC microcomputer using the ABstat software package (5). The major parameters of the analyses included the correlation coefficient (r_{ij}), regression coefficient (R), coefficient of determination (R^2), beta weight (b_{ij}), F-test (F), and a probability factor called "PROB" that indicates the chance of tested observations being drawn from a zero-coefficient population. Correlation coefficients will range from -1 to +1. A coefficient of -1 means perfect negative correlation, a coefficient of 0 means absolutely no correlation, and a coefficient of +1 means perfect positive correlation. In other words, the closer the coefficient is to 0, the weaker the correlation, disregarding the negative or positive relationship. R^2 shows that the proportion of variation in the predicted dependent variable can be explained by independent variables in a regression equation. R indicates how closely the change in a dependent variable is related to the change in the independent variables. The F-test is an indicator of the goodness of fit of the linear equation. The F-test shows whether the observed multiple correlation is due to sampling fluctuation or measurement error. The beta weights indicate the relationship between one independent variable and the predicted dependent variable with other variables held constant (6).

The purpose of bivariate analyses is to group the nonnumerical data on attenuator sites into analytical formats. These data included the following categories:

- C1: Street classifications (six)
- C2: Attenuator types (five)
- C3: Traffic lane distribution
- C4: Fixed-object types
- C5: Usage of object markers
- C6: Existence of traffic control devices

These data cannot be translated into numeric formats. Therefore they cannot be analyzed through numerical analyses. However, their existence can affect the performance of impact attenuators and should not be neglected in this study. These variables were tested against other variables through tabular forms.

FINDINGS AND INTERPRETATIONS

Before-and-After Analysis

The results of the before-and-after study are shown in Figure 2. After the accident numbers and traffic exposures were entered, the spread sheet automatically computed the reduction rate for different accident measures. The tested percentage changes were obtained from the Poisson curve chart (Figure 1). The 95 percent curve was selected for this calculation. If the calculated percentage is greater than the value interpolated from the curve, the change is significant. The data in Figure 2 indicate that the installation of impact attenuators has significantly reduced both the frequency and the severity of accidents at these attenuator sites. It is generally agreed by most engineers that the installation of crash cushions reduces the space in which to recover that a driver has available. Thus accident frequency may increase after a crash cushion is installed. However, the results of this study show that the installation of impact attenuators can reduce the total number of reported accidents.

Benefit-Cost Analysis

Table 1 gives the calculation process of EUAC for each study location. The results show that the D.C. attenuator system has an average service life of 6.75 years. In FY 1984 the cost to the D.C. Government to maintain each crash cushion was $1,778.36. Table 2 includes the calculations of EUAB and the B/C ratio. The mean B/C ratio for all study locations was found to be 4.97. The citywide impact attenuator B/C ratio, which is the result of total EUAB divided by total EUAC, was found to be 4.4.

Inspection and Maintenance Frequency Analysis

The probabilities of no damage P(0) or damage from only one hit P(1) can be directly calculated from Table 3. The data in the table indicate that, by using the current 36-day average inspection interval, D.C. engineers have found 98.61 percent of possible damage before the second hit occurs. For the location subject to the highest number of hits (seven hits), there is a probability of about 84.75 percent that the damage was detected before the successive strike took place. The maintenance frequencies for the 99, 95, and 90 percent levels of confidence were calculated for all study locations (Table 4). An average maintenance interval of 10.65 days was found for the D.C. impact attenuator system at the 95 percent level of confidence. Actually, the required maintenance frequency varies with location. The location that has been hit the most times (seven hits) should be repaired immediately (2.67 days).

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        PROJECT LOCATION :  THE DISTRICT'S 88 INPACT ATTENUATORS
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I           I        I        I EXPECTEDI DIFFERENCE  I           I
I FREQUENCY IBEFORE  IAFTER   I ACCIDENTI-------------I           I
I           I        I(ACTUAL)IFREQUENCYI  N   I  %   I           I
I-----------I--------I--------I---------I------I------I           I
I  TOTAL    I        I        I         I      I      I           I
I ACC. NO.  I    158 I     19 I     158 I  139 I0.87  I           I
I-----------I--------I--------I---------I------I------I           I
I  FATAL    I        I        I         I      I      I           I
I ACC. NO.  I      0 I      0 I       0 I    0 I  0   I           I
I-----------I--------I--------I---------I------I------I           I
I  INJURY   I        I        I         I      I      I           I
I ACC. NO.  I     81 I     11 I      81 I   70 I0.86  I           I
I-----------I--------I--------I---------I------I------I           I
I    PDO    I        I        I         I      I      I           I
I ACC. NO.  I     77 I      8 I      77 I   69 I0.89  I           I
I-----------I--------I--------I---------I------I------I           I
I   NO. OF  I        I        I         I      I      I           I
I INJURIES  I     81 I     13 I      81 I   68 I0.83  I           I
I-----------I--------I--------I---------I------I------I           I
I           I        I        I         I      I      I           I
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IEXPOSURE INI        I        I         I DIFFERENCE  I  POISSON  I
I1000 VOLUMEI997.545 I997.545 I 997.545 I             I   TEST    I
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I           I        I        IEXPECTED I RATE I TEST I LEVEL OF  I
I   RATE    IBEFORE  IAFTER   IACCIDENT I      I      ICONFIDENCE I
I           I        I        I  RATE   I  %   I  %   I   95%     I
I-----------I--------I--------I---------I------I------I-----------I
I  TOTAL    I        I        I         I      I      I           I
I ACC. RATE I  0.158 I  0.019 I   0.158 I0.879 I0.12  ISIGNIFICANTI
I-----------I--------I--------I---------I------I------I-----------I
I  FATAL    I        I        I         I      I      I           I
I ACC. RATE I  0.000 I  0.000 I   0.000 I0.000 I      I           I
I-----------I--------I--------I---------I------I------I-----------I
I  INJURY   I        I        I         I      I      I           I
I ACC. RATE I  0.081 I  0.011 I   0.081 I0.864 I0.18  ISIGNIFICANTI
I-----------I--------I--------I---------I------I------I-----------I
I    PDO    I        I        I         I      I      I           I
I ACC. RATE I  0.077 I  0.008 I   0.077 I0.896 I0.19  ISIGNIFICANTI
I-----------I--------I--------I---------I------I------I-----------I
I  INJURY   I        I        I         I      I      I           I
I   RATE    I  0.081 I  0.013 I   0.081 I0.839 I0.18  ISIGNIFICANTI
I-----------I--------I--------I---------I------I------I-----------I
I           I        I        I         I      I      I           I
I===========I========I========I=========I======I======I===========I
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FIGURE 2 Measure of effectiveness data comparison work sheet.

TABLE 1 D.C. Attenuation System, Annual Cost Estimation (FY 1983)

Attenuator No.	Installation Cost ($)	Age of Attenuator (years)	Age After Upgrade (years)	Degree of Freedom	Annual Capital Cost ($)	Maintenance No.	Maintenance Material Cost ($)	Maintenance Labor Cost ($)	Inspection Cost ($)	Reporting Cost ($)	EUAC ($)
1	11,900.00	16	9	0.1490	1,773.10	2	548.26	50.00	57.20	14.77	2,443.33
2	11,900.00	16	9	0.1490	1,773.10	1	42.00	25.00	57.20	14.77	1,912.07
3	7,000.00	16	9	0.1490	1,043.00	1	176.00	25.00	57.20	14.77	1,315.97
4	6,000.00	16	9	0.1490	894.00	1	0.00	25.00	57.20	14.77	990.97
5	8,000.00	14		0.0000	0.00	0	0.00	0.00	57.20	14.77	71.97
6	8,000.00	14		0.0000	0.00	0	0.00	0.00	57.20	14.77	71.97
7	10,000.00	12		0.1327	1,327.00	1	138.94	25.00	57.20	14.77	1,562.91
8	3,600.00	12	9	0.1490	536.40	2	346.00	50.00	57.20	14.77	1,004.37
10	30,000.00	12		0.1327	3,981.00	0	0.00	0.00	57.20	14.77	4,052.97
11	30,000.00	12		0.1327	3,981.00	1	0.00	50.00	57.20	14.77	4,102.97
12	30,000.00	12	8	0.1490	4,470.00	1	0.00	25.00	57.20	14.77	4,566.97
13	30,000.00	11		0.1327	3,981.00	0	0.00	0.00	57.20	14.77	4,052.97
14	30,000.00	12		0.1327	3,981.00	1	240.00	50.00	57.20	14.77	4,342.97
15	30,000.00	12		0.1327	3,981.00	1	196.00	50.00	57.20	14.77	4,298.97
16	6,000.00	11	9	0.1490	894.00	1	717.00	25.00	57.20	14.77	1,707.97
17	7,921.00	11	9	0.1490	1,180.23	2	409.36	50.00	57.20	14.77	1,711.56
18	7,921.00	11	9	0.1490	1,180.23	2	198.00	50.00	57.20	14.77	1,500.20
19	8,300.00	10		0.1327	1,101.41	1	176.72	75.00	57.20	14.77	1,425.10
20	4,000.00	10		0.1490	596.00	3	509.20	75.00	57.20	14.77	1,252.17
21	11,500.00	9		0.1327	1,526.05	0	0.00	0.00	57.20	14,77	1,598.02
22	11,500.00	9		0.1327	1,526.05	0	0.00	0.00	57.20	14.77	1,598.02
23	2,150.00	9		0.1490	320.35	1	0.00	25.00	57.20	14.77	417.32
24	4,237.00	9		0.1490	631.31	1	198.00	25.00	57.20	14.77	926.28
25	4,237.00	9		0.1490	631.31	1	163.20	25.00	57.20	14.77	891.48
26	4,237.00	9		0.1490	631.31	2	217.80	50.00	57.20	14.77	971.08
27	4,237.00	9		0.1490	631.31	2	375.52	50.00	57.20	14.77	1,128.80
28	4,237.00	9		0.1490	631.31	1	0.00	25.00	57.20	14.77	728.28
29	4,237.00	9		0.1490	631.31	2	86.60	50.00	57.20	14.77	839.88
30	4,237.00	9		0.1490	631.31	3	1,534.00	125.00	57.20	14.77	2,362.28
31	4,237.00	9		0.1490	631.31	1	0.00	25.00	57.20	14.77	728.28
32	11,393.00	10		0.1490	1,697.56	1	742.00	25.00	57.20	14.77	2,536.53

TABLE 1 Continued

Attenuator No.	Installation Cost ($)	Age of Attenuator (years)	Age After Upgrade (years)	Degree of Freedom	Annual Capital Cost ($)	Maintenance No.	Maintenance Material Cost ($)	Maintenance Labor Cost ($)	Inspection Cost ($)	Reporting Cost ($)	EUAC ($)
33	742.00	8		0.1490	110.56	1	0.00	50.00	57.20	14.77	232.53
34	18,000.00	8		0.1327	2,388.60	1	248.00	75.00	57.20	14.77	2,783.57
35	2,570.00	7		0.1490	382.93	2	1,434.00	175.00	57.20	14.77	2,063.90
36	18,000.00	7		0.1327	2,388.60	0	0.00	0.00	57.20	14.77	2,460.57
37	18,000.00	7		0.1327	2,388.60	0	0.00	0.00	57.20	14.77	2,460.57
38	7,250.00	7		0.1327	962.08	1	346.00	50.00	57.20	14.77	1,430.05
39	7,250.00	7		0.1490	1,080.25	2	1,236.00	125.00	57.20	14.77	2,513.22
40	5,000.00	6		0.1490	745.00	1	371.00	25.00	57.20	14.77	1,212.97
41	5,000.00	6		0.1490	745.00	1	173.00	25.00	57.20	14.77	1,014.97
42	9,000.00	6		0.1327	1,194.30	1	30.00	12.50	57.20	14.77	1,308.77
43	9,000.00	6		0.1327	1,194.30	1	30.00	12.50	57.20	14.77	1,308.77
48	9,000.00	7		0.1327	1,194.30	0	0.00	12.50	57.20	14.77	1,278.77
49	9,000.00	7		0.1327	1,194.30	0	0.00	12.50	57.20	14.77	1,278.77
50	9,000.00	7		0.1327	1,194.30	0	0.00	0.00	57.20	14.77	1,266.27
51	9,000.00	7		0.1327	1,194.30	1	0.00	50.00	57.20	14.77	1,316.27
52	2,150.00	5		0.1490	320.35	0	0.00	0.00	57.20	14.77	392.32
53	2,150.00	5		0.1490	320.35	0	0.00	0.00	57.20	14.77	392.32
54	2,150.00	5		0.1490	320.35	0	0.00	0.00	57.20	14.77	392.32
55	19,000.00	5		0.1327	2,521.30	1	346.00	125.00	57.20	14.77	3,064.27
56	3,700.00	5		0.1490	551.30	0	0.00	0.00	57.20	14.77	623.27
57	6,000.00	4		0.1490	894.00	0	0.00	0.00	57.20	14.77	965.97
58	4,250.00	4		0.1490	633.25	1	0.00	25.00	57.20	14.77	730.22
59	19,000.00	4		0.1327	2,521.30	0	0.00	0.00	57.20	14.77	2,593.27
60	6,500.00	4		0.1490	968.50	3	1,311.00	75.00	57.20	14.77	2,426.47
61	4,000.00	4		0.1490	596.00	1	717.00	25.00	57.20	14.77	1,409.97
62	4,250.00	4		0.1490	633.25	0	0.00	0.00	57.20	14.77	750.22
63	5,000.00	6		0.1490	745.00	1	915.00	75.00	57.20	14.77	1,806.97
64	4,250.00	4		0.1490	633.25	0	0.00	0.00	57.20	14.77	705.22
65	4,250.00	4		0.1490	633.25	0	0.00	0.00	57.20	14.77	705.22
66	4,250.00	4		0.1490	633.25	1	298.00	12.50	57.20	14.77	1,015.72
68	6,590.00	4		0.1490	981.91	1	173.00	12.50	57.20	14.77	1,239.38
69	6,700.00	4		0.1490	998.30	1	0.00	12.50	57.20	14.77	1,082.77
70	5,000.00	4		0.1490	745.00	0	0.00	0.00	57.20	14.77	816.97
71	4,250.00	4		0.1490	633.25	0	0.00	0.00	57.20	14.77	705.22
72	19,000.00	4		0.1327	2,521.30	1	594.00	75.00	57.20	14.77	3,262.27
73	19,000.00	4		0.1327	2,521.30	1	594.00	50.00	57.20	14.77	3,237.27
74	19,000.00	3		0.1327	2,521.30	0	0.00	0.00	57.20	14.77	2,593.27
75	28,000.00	3		0.1327	3,715.60	0	0.00	0.00	57.20	14.77	3,787.57
76	9,000.00	3		0.1327	1,194.30	0	0.00	0.00	57.20	14.77	1,266.27
77	9,000.00	3		0.1327	1,194.30	0	0.00	0.00	57.20	14.77	1,266.27
78	11,400.00	3		0.1327	1,512.78	1	0.00	12.50	57.20	14.77	1,597.25
79	28,000.00	3		0.1327	3,715.60	0	0.00	0.00	57.20	14.77	3,787.57
80	30,000.00	3		0.1327	3,981.00	1	0.00	12.50	57.20	14.77	4,065.47
81	28,000.00	3		0.1327	3,715.60	0	0.00	0.00	57.20	14.77	3,787.57
82	19,000.00	3		0.1327	2,521.30	0	0.00	0.00	57.20	14.77	2,593.27
83	19,000.00	3		0.1327	2,521.30	0	0.00	0.00	57.20	14.77	2,593.27
85	6,000.00	2		0.1490	894.00	0	0.00	0.00	57.20	14.77	965.97
86	15,000.00	2		0.1327	1,990.50	1	214.80	50.00	57.20	14.77	2,327.27
87	18,000.00	2		0.1327	2,388.60	0	0.00	0.00	57.20	14.77	2,460.57
88	19,000.00	2		0.1327	2,521.30	0	0.00	0.00	57.20	14.77	2,593.27
89	750.00	2		0.1490	111.75	0	0.00	0.00	57.20	14.77	183.72
90	19,000.00	2		0.1327	2,521.30	1	0.00	25.00	57.20	14.77	2,618.27
91	5,000.00	2		0.1490	745.00	2	1,903.00	100.00	57.20	14.77	2,819.97
93	19,000.00	2		0.1327	2,521.30	0	0.00	0.00	57.20	14.77	2,593.27
107	8,000.00	2		0.1327	1,061.60	0	0.00	0.00	57.20	14.77	1,133.57
108	8,000.00	2		0.1327	1,061.60	0	0.00	0.00	57.20	14.77	1,133.57
109	2,150.00	1		0.1490	320.35	1	594.00	25.00	57.20	14.77	1,011.32
Total	958,533.00				129,207.53	67	18,542.40	2,412.50	5,033.60	1,299.76	156,495.79
Average	10,892.42	6.75			1,468.27	0.76	210.71	27.41	57.20	14.77	1,778.36

TABLE 2 D.C. Impact Attenuation System, Benefit-Cost Analysis

Attenuator No.	No. of Hits	Expected Accident Number and Severity: No.	Injury	Cost ($)	Reported Accident Number and Severity: No.	Injury	Cost ($)	Estimated Benefit ($)	EUAC ($)	Benefit-to-Cost Ratio
1	4	4	1	12,360.00	1	0	1,090.00	11,270.00	2,443.33	4.61
2	0	0	0	0.00	0	0	0.00	0.00	1,912.07	0.00
3	3	3	0	3,270.00	1	0	1,090.00	2,180.00	1,315.97	1.66
4	2	2	1	10,180.00	0	0	0.00	10,180.00	990.97	10.27
5	6	6	2	22,540.00	2	1	10,180.00	12,360.00	71.97	NA
6	4	4	0	4,360.00	0	0	0.00	4,360.00	71.97	NA
7	1	1	1	9,090.00	0	0	0.00	9,090.00	1,562.91	5.82
8	5	5	2	21,450.00	0	0	0.00	21,450.00	1,004.37	21.36
10	4	4	3	28,360.00	0	0	0.00	28,360.00	4,052.97	7.00
11	2	2	1	10,180.00	0	0	0.00	10,180.00	4,102.97	2.48
12	3	3	3	27,270.00	0	0	0.00	27,270.00	4,566.97	5.97

TABLE 2 Continued

Attenuator No.	No. of Hits	Expected Accident Number and Severity			Reported Accident Number and Severity			Estimated Benefit ($)	EUAC ($)	Benefit-to-Cost Ratio
		No.	Injury	Cost ($)	No.	Injury	Cost ($)			
13	3	3	3	27,270.00	0	0	0.00	27,270.00	4,052.97	6.73
14	1	1	0	1,090.00	0	0	0.00	1,090.00	4,342.97	0.25
15	1	1	1	9,090.00	0	0	0.00	9,090.00	4,298.97	2.11
16	5	5	4	37,450.00	0	0	0.00	37,450.00	1,707.97	21.93
17	2	2	2	18,180.00	1	2	17,090.00	1,090.00	1,711.56	0.64
18	1	1	0	1,090.00	0	0	0.00	1,090.00	1,500.20	0.73
19	2	2	2	18,180.00	0	0	0.00	18,180.00	1,425.10	12.76
20	6	6	4	38,540.00	2	2	18,180.00	20,360.00	1,252.17	16.26
21	1	1	0	1,090.00	0	0	0.00	1,090.00	1,598.02	0.68
22	0	0	0	0.00	0	0	0.00	0.00	1,598.02	0.00
23	2	2	1	10,180.00	0	0	0.00	10,180.00	417.32	24.39
24	1	1	0	1,090.00	0	0	0.00	1,090.00	926.28	1.18
25	3	3	1	11,270.00	0	0	0.00	11,270.00	891.48	12.64
26	5	5	3	29,450.00	0	0	0.00	029,450.00	971.08	30.33
27	6	6	2	22,540.00	0	0	0.00	22,540.00	1,128.80	19.97
28	2	2	2	18,180.00	0	0	0.00	18,180.00	728.28	24.96
29	2	2	1	10,180.00	0	0	0.00	10,180.00	839.88	12.12
30	7	7	3	31,630.00	0	0	0.00	31,630.00	2,362.28	13.39
31	1	1	0	1,090.00	0	0	0.00	1,090.00	728.28	1.50
32	2	2	0	2,180.00	0	0	0.00	2,180.00	2,536.53	0.86
33	2	2	1	10,180.00	1	1	9,090.00	1,090.00	232.53	4.69
34	2	2	1	10,180.00	0	0	0.00	10,180.00	2,783.57	3.66
35	4	4	4	36,360.00	1	1	9,090.00	27,270.00	2,063.90	13.21
36	0	0	0	0.00	0	0	0.00	0.00	2,460.57	0.00
37	0	0	0	0.00	0	0	0.00	0.00	2,460.57	0.00
38	3	3	0	3,270.00	0	0	0.00	3,270.00	1,430.05	2.29
39	3	3	2	19,270.00	1	0	1,090.00	18,180.00	2,513.22	7.23
40	1	1	1	9,090.00	0	0	0.00	9,090.00	1,212.97	7.49
41	1	1	1	9,090.00	0	0	0.00	9,090.00	1,014.97	8.96
42	0	0	0	0.00	0	0	0.00	0.00	1,308.77	0.00
43	0	0	0	0.00	0	0	0.00	0.00	1,308.77	0.00
48	2	2	1	10,180.00	1	1	9,090.00	1,090.00	1,278.77	0.85
49	0	0	0	0.00	0	0	0.00	0.00	1,278.77	0.00
50	1	1	0	1,090.00	0	0	0.00	1,090.00	1,266.27	0.86
51	0	0	0	0.00	0	0	0.00	0.00	1,316.27	0.00
52	0	0	0	0.00	0	0	0.00	0.00	392.32	0.00
53	0	0	0	0.00	0	0	0.00	0.00	392.32	0.00
54	0	0	0	0.00	0	0	0.00	0.00	392.32	0.00
55	3	3	1	11,270.00	1	1	9,090.00	2,180.00	3,064.27	0.71
56	0	0	0	0.00	0	0	0.00	0.00	623.27	0.00
57	0	0	0	0.00	0	0	0.00	0.00	965.97	0.00
58	3	3	1	11,270.00	0	0	0.00	11,270.00	730.22	15.43
59	1	1	1	9,090.00	0	0	0.00	9,090.00	2,593.27	3.51
60	5	5	3	29,450.00	0	0	0.00	29,450.00	2,426.47	12.14
61	3	3	3	27,270.00	2	0	2,180.00	25,090.00	1,409.97	17.79
62	2	2	0	2,180.00	0	0	0.00	2,180.00	705.22	3.09
63	1	1	1	9,090.00	0	0	0.00	9,090.00	1,806.97	5.03
64	0	0	0	0.00	0	0	0.00	0.00	705.22	0.00
65	0	0	0	0.00	0	0	0.00	0.00	705.22	0.00
66	0	0	0	0.00	0	0	0.00	0.00	1,015.72	0.00
68	1	1	0	1,090.00	0	0	0.00	1,090.00	1,239.38	0.88
69	1	1	0	1,090.00	0	0	0.00	1,090.00	1,082.77	1.01
70	0	0	0	0.00	0	0	0.00	0.00	816.97	0.00
71	1	1	0	1,090.00	0	0	0.00	1,090.00	705.22	1.55
72	6	6	2	22,540.00	1	1	9,090.00	13,450.00	3,262.27	4.12
73	3	3	2	19,270.00	0	0	0.00	19,270.00	3,237.27	5.95
74	0	0	0	0.00	0	0	0.00	0.00	2,593.27	0.00
75	1	1	0	1,090.00	0	0	0.00	1,090.00	3,787.57	0.29
76	0	0	0	0.00	0	0	0.00	0.00	1,266.27	0.00
77	0	0	0	0.00	0	0	0.00	0.00	1,266.27	0.00
78	5	5	2	21,450.00	2	2	18,180.00	3,270.00	1,597.25	2.05
79	0	0	0	0.00	0	0	0.00	0.00	3,787.57	0.00
80	1	1	1	9,090.00	1	1	9,090.00	0.00	4,065.47	0.00
81	1	1	1	9,090.00	1	0	1,090.00	8,000.00	3,787.57	2.11
82	2	2	0	2,180.00	0	0	0.00	2,180.00	2,593.27	0.84
83	0	0	0	0.00	0	0	0.00	0.00	2,593.27	0.00
85	1	1	0	1,090.00	0	0	0.00	1,090.00	965.97	1.13
86	3	3	3	27,270.00	0	0	0.00	27,270.00	2,327.27	11.72
87	0	0	0	0.00	0	0	0.00	0.00	2,460.57	0.00
88	0	0	0	0.00	0	0	0.00	0.00	2,593.27	0.00
89	0	0	0	0.00	0	0	0.00	0.00	183.72	0.00
90	1	1	0	1,090.00	0	0	0.00	1,090.00	2,618.27	0.42
91	6	6	5	46,540.00	0	0	0.00	46,540.00	2,819.97	16.50
93	0	0	0	0.00	0	0	0.00	0.00	2,593.27	0.00
107	0	0	0	0.00	0	0	0.00	0.00	1,133.57	0.00
108	0	0	0	0.00	0	0	0.00	0.00	1,133.57	0.00
109	1	1	1	9,090.00	0	0	0.00	9,090.00	1,011.32	8.99
Total	158	158	81	820,220.00	19	13	124,710.00	695,510.00	156,495.78	
Average	1.80	1.80	0.92	9,320.68	0.22	0.15	1,417.16	7,903.52	1,778.36	4.97

Note: Citywide B/C ratio = 4.44.

TABLE 3 **Probability of Damage Occurring During Inspection Intervals (36 days)**

Attenuator No.	Type	No. of Hits	ADT	Period (days)	Inspection Period (days)	Expected Hits (H)	Probability of No Hit (0)	Probability of One Hit P(1)	Probability of No and One Hit	Probability of More Than One Hit
1	S16	4	64,800	365	36	0.3945	0.6740	0.2659	0.9399	9.0601
2	S17	0	64,800	365	36	0.000	1.0000	0.0000	1.0000	0.0000
3	S26	3	59,040	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
4	S15	2	23,200	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
5	HS4	6	37,380	365	36	0.5918	0.5533	0.3275	0.8808	0.1192
6	HS4	4	20,000	365	36	0.3945	0.6740	0.2659	0.9399	0.0601
7	HC	1	57,200	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
8	S9	5	34,920	365	36	0.4932	0.6107	0.3012	0.9119	0.0881
10	HSB	4	27,300	365	36	0.3945	0.6740	0.2659	0.9399	0.0601
11	HSB	2	27,000	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
12	S5	3	8,000	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
13	HSB	3	29,000	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
14	HSB	1	13,000	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
15	HSB	1	49,000	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
16	S7	5	23,200	365	36	0.4932	0.6107	0.3012	0.9119	0.0881
17	S4	2	36,600	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
18	S4	1	39,180	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
19	HC	2	64,800	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
20	S11	6	42,800	365	36	0.5918	0.5533	0.3275	0.8808	0.1192
21	HC	1	29,000	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
22	HC	0	16,980	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
23	S9	2	19,800	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
24	S10	1	25,800	365	36	0.0986	0.9061	0.0894	0.9554	0.0046
25	S11	3	34,200	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
26	S8	5	22,700	365	36	0.4932	0.6107	0.3012	0.9119	0.0881
27	S9	6	29,400	365	36	0.5918	0.5533	0.3275	0.8808	0.1192
28	S8	2	27,300	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
29	S3	2	27,300	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
30	S6	7	29,400	365	36	0.6904	0.5014	0.3462	0.8475	0.1525
31	S9	1	51,624	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
32	S5	2	15,000	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
33	S3	2	39,190	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
34	HC	2	9,780	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
35	S4	4	22,740	365	36	0.3945	0.6740	0.2659	0.9399	0.0601
36	HC	0	2,160	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
37	HC	0	2,160	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
38	G6	3	12,720	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
39	S7S7	3	17,940	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
40	S10	1	19,500	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
41	S10	1	19,500	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
42	HC	0	9,300	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
43	HC	0	9,300	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
48	HC	2	29,580	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
49	HC	0	29,380	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
50	G6	1	34,560	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
51	G6	0	34,560	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
52	S10	0	9.780	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
53	S9	0	9,780	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
54	S10	0	9,780	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
55	G6S5	3	47,220	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
56	S10	0	47,220	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
57	S12	0	45,220	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
58	S9	3	34,200	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
59	G6S2	1	34,200	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
60	S17	5	41,400	365	36	0.4932	0.6107	0.3012	0.9119	0.0881
61	S13	3	47,220	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
62	S10	2	46,200	365	36	0.1973	0.8210	0.1619	0.9829	0.0171
63	S10	1	46,200	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
64	S11	0	46,200	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
65	S10	0	46,200	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
66	S10	0	46,200	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
68	S17	1	41,400	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
69	S18	1	14,200	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
70	S11	0	47,220	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
71	S11	1	46,200	365	36	0.0986	0.9061	0.0894	0.9955	0.0045
72	B5S3	6	41,400	365	36	0.5918	0.5533	0.3275	0.8808	0.1192
73	S6S2	3	47,220	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
74	G4	0	47,280	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
75	H14	1	39,000	365	36	0.0986	0.9061	0.0894	0.9955	0.0045
76	G3	0	14,040	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
77	G3	0	14,040	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
78	G5S3	5	110,220	365	36	0.4932	0.6107	0.3012	0.9119	0.0881
79	H14	0	37,380	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
80	H14	1	12,600	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
81	H14	1	59,040	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
82	G5	2	21,000	365	36	0.1973	0.8210	0.1619	0.9640	0.0171
83	G3	0	26,000	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
85	S11	1	30,000	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
86	G3	3	11,400	365	36	0.2959	0.7439	0.2201	0.9640	0.0360
87	HC	0	9,780	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
88	G5	0	25,200	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
89	S3	0	42,800	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
90	G2	1	1,200	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
91	S11	6	34,200	365	36	0.5918	0.5533	0.3275	0.8808	0.1192
93	G3	0	30,000	365	36	0.0000	1.0000	0.0000	1.0000	0.0000

TABLE 3 Continued

Attenuator No.	Type	No. of Hits	ADT	Period (days)	Inspection Period (days)	Expected Hits (H)	Probability of No Hit (0)	Probability of One Hit P(1)	Probability of No and One Hit	Probability of More Than One Hit
107	G2S1	0	12,600	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
108	G2S1	0	12,600	365	36	0.0000	1.0000	0.0000	1.0000	0.0000
109	S4	1	30,000	365	36	0.0986	0.9061	0.0894	0.9954	0.0046
Total		158								
Average		1.795		365	36	0.17708	0.8377	0.1483	0.9861	0.0139
Citywide		158		365	36	15.5835	.0000	.0000	.0000	1.0000

TABLE 4 Frequency Table for Attenuator Maintenance

Attenuator No.	Type	No. of Hits	Period (days)	Annual Average Hits	Action Days for P(0) = 0.99	Action Days for P(0) = 0.95	Action Days for P(0) = 0.90
1	S16	4	365	4	0.92	4.68	9.61
2	S17	0	365	0	NA	NA	NA
3	S26	3	365	3	1.22	6.24	12.82
4	S15	2	365	2	1.83	9.36	19.23
5	HS4	6	365	6	0.61	3.12	6.41
6	HS4	4	365	4	0.92	4.68	9.61
7	HC	1	365	1	3.67	18.72	38.46
8	S9	5	365	5	0.73	3.74	7.69
10	HSB	4	365	4	0.92	4.68	9.61
11	HSB	2	365	2	1.83	9.36	19.23
12	S5	3	365	3	1.22	6.24	12.82
13	HSB	3	365	3	1.22	6.24	12.82
14	HSB	1	365	1	3.67	18.72	38.46
15	HSB	1	365	1	3.67	18.72	38.46
16	S7	5	365	5	0.73	3.74	7.69
17	S4	2	365	2	1.83	9.36	19.23
18	S4	1	365	1	3.67	18.72	38.46
19	HC	2	365	2	1.83	9.36	19.23
20	S11	6	365	6	0.61	3.12	6.41
21	HC	1	365	1	3.67	18.72	38.46
22	HC	0	365	0	NA	NA	NA
23	S9	2	365	2	1.83	9.36	19.23
24	S10	1	365	1	3.67	18.72	38.46
25	S11	3	365	3	1.22	6.24	12.82
26	S8	5	365	5	0.73	3.74	7.69
27	S9	6	365	6	0.61	3.12	6.41
28	S8	2	365	2	1.83	9.36	19.23
29	S3	2	365	2	1.83	9.36	19.23
30	S6	7	365	7	0.52	2.67	5.49
31	S9	1	365	1	3.67	18.72	38.46
32	S5	2	365	2	1.83	9.36	19.23
33	S3	2	365	2	1.83	9.36	19.23
34	HC	2	365	2	1.83	9.36	19.23
35	S4	4	365	4	0.92	4.68	9.61
36	HC	0	365	0	NA	NA	NA
37	HC	0	365	0	NA	NA	NA
38	G6	3	365	3	1.22	6.24	12.82
39	S7	3	365	3	1.22	6.24	12.82
40	S10	1	365	1	3.67	18.72	38.46
41	S10	1	365	1	3.67	18.72	38.46
42	HC	0	365	0	NA	NA	NA
43	HC	0	365	0	NA	NA	NA
48	HC	2	365	2	1.83	9.36	19.23
49	HC	0	365	0	NA	NA	NA
50	G6	1	365	1	3.67	18.72	38.46
51	G6	0	365	0	NA	NA	NA
52	S10	0	365	0	NA	NA	NA
53	S9	0	365	0	NA	NA	NA
54	S10	0	365	0	NA	NA	NA
55	G6S4	3	365	3	1.22	6.24	12.82
56	S10	0	365	0	NA	NA	NA
57	S12	0	365	0	NA	NA	NA
58	S9	3	365	3	1.22	6.24	12.82
59	G6S2	1	365	1	3.67	18.72	38.46
60	S17	5	365	5	0.73	3.74	7.69
61	S13	3	365	3	1.22	6.24	12.82
62	S10	2	365	2	1.83	9.36	19.23
63	S10	1	365	1	3.67	18.72	38.46
64	S11	0	365	0	NA	NA	NA
65	S10	0	365	0	NA	NA	NA
66	S10	0	365	0	NA	NA	NA
68	S17	1	365	1	3.67	18.72	38.46
69	S18	1	365	1	3.67	18.72	38.46
70	S11	0	365	0	NA	NA	NA
71	S11	1	365	1	3.67	18.72	38.46
72	G5S3	6	365	6	0.61	3.12	6.41
73	G6S2	3	365	3	1.22	6.24	12.82
74	G4	0	365	0	NA	NA	NA

TABLE 4 Continued

Attenuator No.	Type	No. of Hits	Period (days)	Annual Average Hits	Action Days for P(0) = 0.99	Action Days for P(0) = 0.95	Action Days for P(0) = 0.90
75	HI4	1	365	1	3.67	18.72	38.46
76	G3	0	365	0	NA	NA	NA
77	G3	0	365	0	NA	NA	NA
78	G5S3	5	365	5	0.73	3.74	7.69
79	HI4	0	365	0	NA	NA	NA
80	HI4	1	365	1	3.67	18.72	38.46
81	HI4	1	365	1	3.67	18.72	38.46
82	G5	2	365	2	1.83	9.36	19.23
83	G3	0	365	0	NA	NA	NA
85	S11	1	365	1	3.67	18.72	38.46
86	G3	3	365	3	1.22	6.24	12.82
87	HC	0	365	0	NA	NA	NA
88	G5	0	365	0	NA	NA	NA
89	S3	0	365	0	NA	NA	NA
90	G2	1	365	1	3.67	18.72	38.46
91	S11	6	365	6	0.61	3.12	6.41
93	G3	0	365	0	NA	NA	NA
107	G2S1	0	365	0	NA	NA	NA
108	G2S1	0	365	0	NA	NA	NA
109	S4	1	365	1	3.67	18.72	38.46
Total		158			125.25	639.22	1,313.02
Average		1.80	365		2.09	10.65	21.88

Note: NA means the data are not available; however, the damage should be repaired as soon as possible.

Correlation and Multiple Regression Analyses

The correlation matrix (Figure 3) indicates that the relationships between the 12 independent variables and the number of hits are quite logical. For example, it shows that ADT, truck percentage, and number of lanes are highly correlated ($r > 0.5$). The interrelationship between pavement rating and skid index is also strong. The strongest negative correlation (-0.36885) was found between speed and street light luminance.

The relationships between the dependent variable, number of hits (Y1), and the 12 independent variables can be described by the following regression model:

$$
\begin{aligned}
Y1 = 0.3329 &- 0.0104\ X1 + 1.0892\ X2 + 0.0111\ X3 \\
&- 0.0911\ X4 - 0.1296\ X5 - 0.0102\ X6 \\
&- 0.1011\ X7 - 0.2030\ X8 - 1.5653\ X9 \\
&- 0.1811\ X10 - 0.1361\ X11 + 0.0612\ X12
\end{aligned}
$$

$Se = 1.3578$, $R = 0.7347$, $R^2 = 0.5398$
$F = 7.2332$, dof = [12,74], PROB = 0.0000

This multiple regression equation shows that truck percentage (X2), horizontal curvature (X8), street light luminance (X9), length of no-passing zone (X10), and attenuator offset (X11) are strongly correlated with the number of hits (Y1). However, length of no-passing zone (X10) cannot pass the F-test at the 5 percent level of significance. A comparison between the calculated F-value and the F-test table value shows that the multiple regression model passes the 5 percent level of significance. The coefficient of determination indicates that this model can explain 53.98 percent of the total variation in the number of hits. When each single independent variable was tested against Y1, it was found that the significant independent variables can be ranked according to priority as follows:

X9 = Street light luminance, negatively correlated;
X2 = Truck percentage, positively correlated;
X8 = Length of horizontal curvature radius, negatively correlated; and
X11 = Offset distance, negatively correlated.

Theoretically, a predictive model should not include the insignificant variables. The reconstruction of this model using the four significant variables at the 5 percent level of significance showed the following information:

$$
\begin{aligned}
Y1 = 2.8706 &+ 0.8129\ X2 - 0.2131\ X8 \\
&- 1.7401\ X9 - 0.1538\ X11
\end{aligned}
$$

$Se = 1.3920$, $R = 0.6812$, $R^2 = 0.4640$
$F = 17.7478$, dof = [4,82], PROB = 0.0000

	X1 ADT	X2 TRUCK	X3 SPEED	X4 LANES	X5 PVT	X6 CURB	X7 GRADIENT	X8 RADIUS	X9 LIGHT	X10 MARKING	X11 OFFSET	X12 SKID	Y1 HITS
X1	1												
X2	0.5308	1											
X3	0.4216	0.3611	1										
X4	0.6818	0.5758	0.4201	1									
X5	0.0779	-0.1032	0.0316	-0.1373	1								
X6	0.0191	0.1490	-0.1633	0.1594	-0.0955	1							
X7	-0.0395	0.0356	-0.0361	-0.0915	0.1295	-0.0786	1						
X8	0.0947	0.1292	0.2164	0.1449	-0.0510	-0.1211	-0.0458	1					
X9	-0.2057	-0.1359	-0.3689	-0.1328	0.0368	0.0590	0.0852	0.3048	1				
X10	0.3284	0.1880	0.3249	0.2049	0.0682	0.2504	-0.1829	0.1257	-0.0431	1			
X11	0.1469	0.0141	0.3327	-0.0459	0.2474	-0.2075	0.0192	0.1562	-0.0809	0.2833	1		
X12	0.2122	-0.0012	0.2140	0.0388	0.5883	-0.2415	0.0810	-0.1373	-0.1168	0.1011	0.1972	1	
Y1	0.0916	0.3329	0.0908	0.1477	-0.0953	0.0170	-0.1111	-0.4092	-0.4548	0.1917	-0.2767	0.1201	1

FIGURE 3 Correlation matrix—number of hits versus independent variables X1-X12.

Comparison with the table value of the F-distribution showed that this reconstructed model is also significant. However, the interpretation power of this model decreases to 46.4 percent because of the dropout of the insignificant variables.

The multiple regression equation for accident cost (Y2) and the other 12 independent variables can pass the F-test at the 5 percent level of significance. However, the equation can only interpret 39.22 percent of accident cost change. This low value of R cast doubt on the usefulness of the model for the purpose of predicting accident severity.

Bivariate Analyses

The bivariate analyses of variables generated the following findings:

- Locations on Interstates, freeways, and expressways, which account for about 73 percent of the total locations, took 85 percent of the total hits. This finding is due primarily to lower street light illumination.
- During the study period, the Hi-dro Cell sandwich and sand barrel locations suffered the most hits, with a ratio of 3.0 and 2.13 per location, respectively.
- Attenuators located at gore areas had 4.4 times the number of hits that were sustained by the ones located at one side of a roadway. Attenuators located on the right side of a highway were twice as likely to be hit as were those on the left side. Locations with three to five approaching lanes were more likely to be hit.
- Bridge gores, tunnel gores, and retaining walls had impact frequencies of 4, 3, and 2.4 hits, respectively, per location per year.
- Locations with object markers were hit 50 percent less often than locations without them.

CONCLUSION

For more than a decade impact attenuators have been used by many state and local transportation agencies to prevent errant vehicles from colliding with rigid roadside hazards. However, limited research has been conducted in this field because attenuators are usually placed on roadways with high speeds and volumes where the number of collisions represents a small percentage of the total traffic. The attenuator's capabilities for reducing accident frequency and severity have made a study more difficult because few accident records are available. Despite these difficulties, the District of Columbia has provided an ideal climate for conducting this study because of its limited geographic area and large number of attenuator installations.

Through extensive data collection, a comprehensive procedure was developed and is currently used by the D.C. Government to determine improvement priorities and to evaluate the performance of impact attenuators. This procedure consists of before-and-after studies, benefit-cost analyses, damage frequency studies, multiple regression and correlation analyses, and bivariate sortings.

On the basis of this procedure and the crash cushion accident and impact records of FY 1984, the following conclusions can be drawn from this study:

1. Street light illumination, truck percentage, radius of horizontal roadway curvature, and offset distance were the major factors that determined the frequency of damage to impact attenuators. This finding may imply that driver's visibility, vehicle width, and effective roadway width for travel play major roles in roadside collision incidents, given that, during the study period, 15 (79 percent) of the 19 total reported crash cushion accidents occurred during nighttime hours. The predictive regression models can be used to select and to assign priority to the candidate locations if these descriptors can be measured in the field.
2. The schedules for field inspection and maintenance should vary with impact frequency and the selected level of significance. In the District of Columbia it was found that by conducting monthly field investigations, the engineer can be 95 percent confident that about 99 percent of the damage was detected before the second hit occurred.
3. The conventional before-and-after study and benefit-cost analysis are usually conducted 3 years after a highway facility is built. These methods cannot serve the need for continuous evaluation of impact attenuators because most of the D.C. crash cushions were installed long before the study was conducted. Therefore these methods were modified to compare actual accident measures with expected measures. The before-and-after study shows that the installation of impact attenuators has significantly reduced both the frequency and the severity of accidents in the District of Columbia. The citywide benefit-to-cost ratio for the District of Columbia attenuator system was found to be 4.4, which is higher than the 3.1 national average.

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Benefit-Cost Analysis of Roadside Safety Alternatives

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ABSTRACT

In recent years, benefit-cost (B-C) analysis procedures have been widely accepted as a rational method for evaluating safety treatment alternatives. Most methods of analysis employed to date have significant limitations, overstate the severity of accidents, and are cumbersome to use. An advanced B-C analysis model that incorporates numerous modifications to enhance versatility and improve determination of accident severity is described. Basic encroachment data on which the model is based are presented, and the applications and limitations of the model are discussed. An example of the use of the model to develop general barrier use guidelines is also included.

Highway engineers have always faced the difficult problem of determining when and where safety features should be used. Until recently, safety feature use guidelines were based primarily on the relative hazard of the possible alternatives. For example, if a high-speed traversal of a roadside slope was thought to be more hazardous than a similar impact with a roadside barrier, a barrier was deemed to be necessary. No consideration was given to the probability that a high-speed accident would occur. This led highway agencies to invest large sums of money to erect guardrail at sites where there was little or no probability of the occurrence of a severe accident.

When safety improvement programs gained higher priority, safety projects began to compete with construction and other projects for highway agency funds. Therefore it became necessary to evaluate the relative merits of all projects. A benefit-cost (B-C) analysis procedure for studying safety improvements was developed to determine the benefits obtained from each dollar spent on safety improvement (1). The 1977 AASHTO barrier guide presented highway engineers with a "simplified" B-C analysis procedure (2). Accident severities were estimated by highway safety professionals including accident investigators, highway engineers, and researchers. Severities derived in this manner have been found to be representative of high-speed accidents. As a result, all predicted accidents were by default assumed to involve high impact speeds, and the procedure overstated the severity of many types of accidents. Therefore the technique frequently led to the use of safety appurtenances at sites where such devices were not warranted. In these cases accidents involving the safety treatment occur more frequently and are more severe than accidents at similar untreated sites.

Efforts to further refine the B-C analysis technique have led researchers to develop relatively sophisticated algorithms (3-5). Although these programs do a better job of properly accounting for all of the costs associated with a safety improvement, the procedures have significant limitations, generally continue to overstate the severity of most accidents that are predicted to occur, and are quite difficult to use.

In an effort to resolve some of the problems associated with existing warranting procedures, an advanced B-C analysis algorithm was developed. Major improvements have been made in the algorithm to improve the versatility of the procedure and the determination of the severity associated with predicted accidents. Further, the algorithm has been coded for use with microcomputers to reduce implementation problems.

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BENEFIT-COST METHODOLOGY

The B-C methodology compares the benefits derived from a safety improvement to the direct highway agency costs incurred as a result of the improvement. Benefits are measured in terms of reductions in societal costs due to decreases in the number or severity, or both, of accidents. Direct highway agency costs are comprised of initial, maintenance, and accident repair costs of a proposed improvement. A ratio between the benefits and costs of an improvement is used to determine if the improvement is cost beneficial:

$$BC_{2-1} = (SC_1 - SC_2)/(DC_2 - DC_1) \quad (1)$$

where

BC_{2-1} = B-C ratio of Alternative 1 compared with Alternative 2,
SC_1 = annualized societal cost of Alternative 1,
DC_1 = annualized direct cost of Alternative 1,
SC_2 = annualized societal cost of Alternative 2, and
DC_2 = annualized direct cost of Alternative 2.

For Equation 1, Alternative 2 is normally considered to be an improvement relative to Alternative 1. When the B-C ratio for a safety improvement is below 1.0, the improvement should not normally be implemented. However, budgetary limitations prevent funding of all projects that have a B-C ratio of 1.0 or more. Ideally, a highway agency can use a B-C approach to analyze all proposed projects, including safety improvements, rehabilitation, and new construction, to determine the optimum use of available funding.

ACCIDENT PREDICTION MODEL

Most benefits and some costs associated with a safety improvement are directly related to the number and severity of accidents that will occur at the site under consideration. Thus accident prediction is critical to the analysis of the need for safety improvements. Although some authors have attempted to use accident data to predict accident frequency and severity, to date these efforts have met with limited success due to poor quality or small accident data bases, or both. Currently, the best available methods for predicting accident frequency and severity are based on encroachment probability models.

An encroachment probability model is based on the concept that the number of run-off-the-road accidents that occurs at a given site can be related to the number of vehicles that inadvertently leave the roadway at that site. Further, it is assumed that the frequency and nature of uncontrolled encroachments can be related to roadway and traffic characteristics. Thus the goal of an encroachment probability model is to relate roadway and traffic characteristics to the expected accident frequency at a site.

The general approach in calculating accident frequency is to determine the region along the roadway, or hazard envelope, within which a vehicle leaving the travelway at a prescribed angle will strike the hazard. A typical hazard envelope is shown in Figure 1. Note that the hazard envelope is divided into three basic ranges. The first encroachment range corresponds with accidents involving the side of the hazard parallel to the roadway and is the same length as the hazard. The second range corresponds to impacts on the corner of the hazard between the two exposed faces and is a function of the effective width of the vehicle. Accident analysis studies have shown that many vehicles involved in roadside accidents are not tracking (6,7). Therefore the effective vehicle width used in the encroachment algorithm is the average of the vehicle width and length. The third encroachment range corresponds to vehicles striking the side of the hazard and is a function of the width of the hazard.

As shown in Figure 1, uncontrolled vehicles are assumed to encroach along a straight path. The probability that a vehicle of a particular size will leave the traveled way within a specific encroachment range at a prescribed angle and speed is merely the length of the range in miles times the probability of a vehicle encroaching under the given conditions:

$$P(E^{W,2}_{V,\theta}|E) = P(W)P(E_{V,\theta}|E)(W_e/\sin\theta)/5{,}280 \qquad (2)$$

where

$P(E^{W,2}_{V,\theta}|E)$ = probability that a vehicle of size W will encroach at speed V and angle θ into encroachment range 2, given that an encroachment has occurred;

$P(W)$ = probability that an encroaching vehicle will be of size W;

$P(E_{V,\theta}|E)$ = probability that an encroaching vehicle will be traveling at speed V and encroaching at angle θ; and

W_e = effective vehicle width (1/2 vehicle width + 1/2 vehicle length) in feet.

Note that this probability is based on the assumption that vehicles encroach randomly within the area of interest.

When a vehicle leaves the travelway within the hazard envelope, there is some probability that the vehicle will stop or steer back to the roadway before striking the hazard. Therefore the probability of entering the hazard envelope must be modified by the probability of a vehicle encroaching far enough laterally to reach the obstacle. The probability that an encroaching vehicle will strike the corner of the hazard is

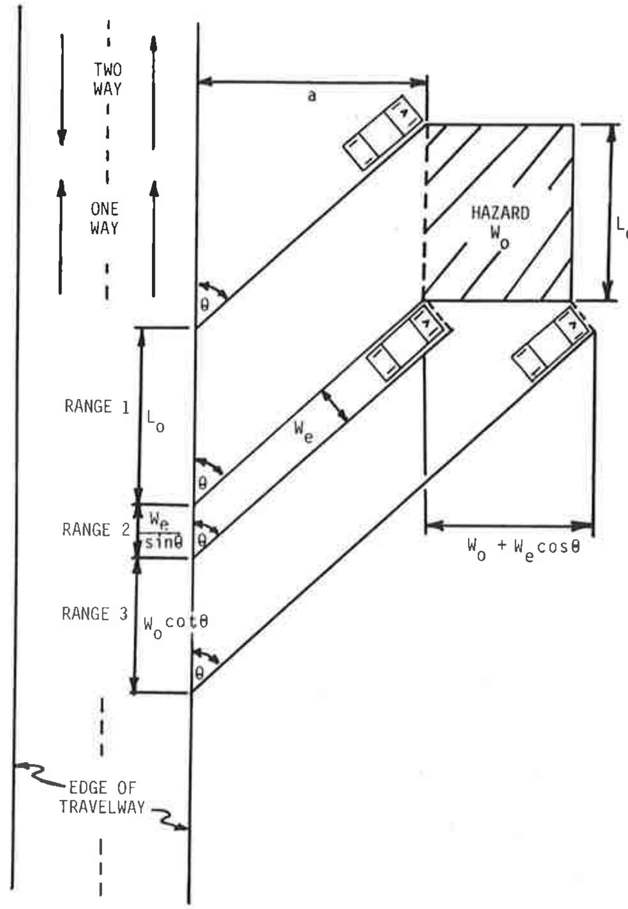

FIGURE 1 Hazard envelope for single hazard.

$$P(C^{W,2}_{V,\theta}|E) = P(W)P(E_{V,\theta})(1/5{,}280)\{\sec\theta\csc\theta\sum_{j=1}^{N} P[LE>(a+j-1/2)]\} \qquad (3)$$

where

$P(C^{W,2}_{V,\theta}|E)$ = probability that a vehicle of size W encroaching at speed V and angle θ will strike hazard within range 2, given that an encroachment has occurred;

a = distance from travelway to fixed object (ft);

$P[LE>(a+\ldots)]$ = probability that the lateral extent of encroachment is greater than or equal to a+...; and

$N = W_e \times \cos\theta$ (ft).

The probability that an encroaching vehicle will strike a single hazard is merely the sum of the probabilities of impacts within each encroachment range.

For most circumstances of interest, two or more hazards are present at one location. For these situations the hazard envelopes can overlap and create a complex geometric problem as shown in Figure 2. This figure shows a rectangular hazard shielded by guardrail. Some vehicles encroaching within this region will strike the longitudinal barrier and be redirected, and other accidents will involve vehicles going behind or through the barrier and striking the

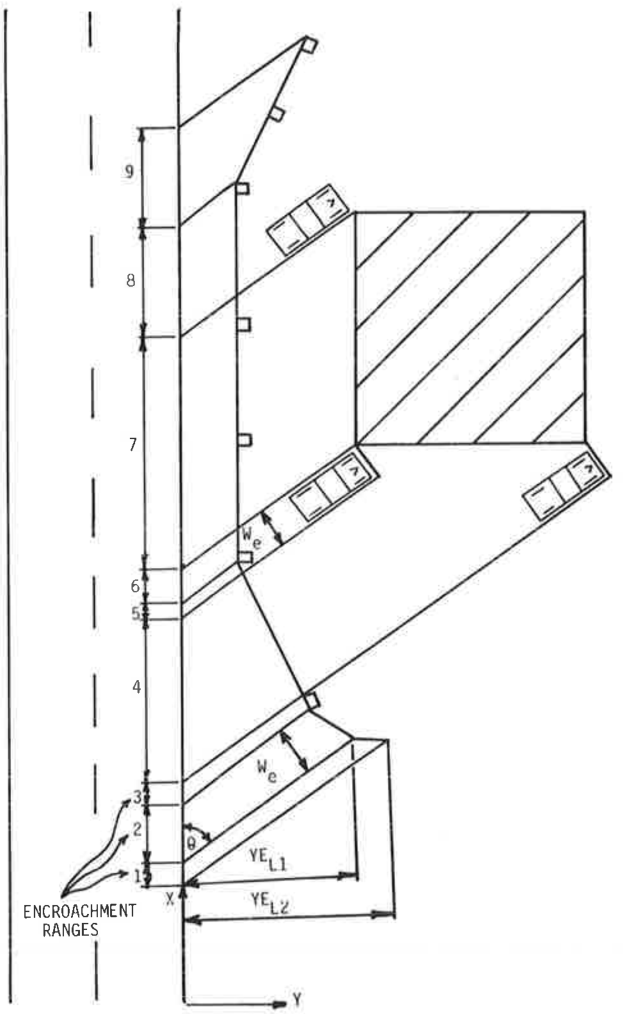

FIGURE 2 Hazard envelope for multiple hazards.

protected hazard. Hazard envelopes for multiple-hazard locations can be described if the relative locations and the geometry of all hazards are known. Figure 2 shows nine encroachment ranges comparing the overlapping hazard envelopes of the two hazards. Each encroachment range describes a unique combination of hazard faces that an encroaching vehicle would contact. For example, a vehicle with sufficient speed to penetrate the barrier, leaving the roadway within encroachment range 7, would first contact the longitudinal face of the barrier and then the longitudinal face of the hazard.

The encroachment probability model developed in this study uses hazard locations and geometry to determine the limits of all encroachment ranges and the lateral distances to each hazard within the range. The model then calculates the probability of a collision within each encroachment range in a manner analogous to that given in Equation 3:

$$P(C^{W,i}_{V,\theta} | E^{W}_{V,\theta}) = L_i/5{,}280 \sum_{j=YB_{L_i}}^{YE_{L_i}} [P(LE \geq j)]/|YE_{L_i} - YB_{L_i}| \qquad (4)$$

where

$P(C^{W,i}_{V,\theta} | E^{W}_{V,\theta})$ = probability that a vehicle of size W leaving the roadway at speed V and angle θ will strike the first hazard within encroachment range i given that an encroachment has occurred involving a vehicle of size W, speed V, and angle θ;

L_i = length of encroachment range i;

YE_{L_i} = lateral distance from end of encroachment range i to first hazard within the range; and

YB_{L_i} = lateral distance from beginning of encroachment range i to first hazard within the range.

The total accident costs for any site can then be determined by multiplying the collision probability from Equation 4 by the encroachment frequency and the accident cost of the predicted accident and summing overall possible accident types:

$$AAC = \sum_{W} \sum_{V} \sum_{\theta} \sum_{i} P(C^{W,i}_{V,\theta} E^{W}_{V,\theta})\, AC^{W,i}_{V,\theta}\, E_f \qquad (5)$$

where

AAC = annual accident costs arising from run-off-the-road traffic accidents within the region of interest ($/year),

E_f = uncontrolled encroachment frequency (encroachments per mile per year),

$\sum_W$ = summation over all encroachment vehicle sizes,

$\sum_V$ = summation over all encroachment velocities,

$\sum_\theta$ = summation over all encroachment angles,

$\sum_i$ = summation over all encroachment ranges, and

$AC^{W,i}_{V,\theta}$ = accident costs associated with an accident involving a vehicle of size W striking hazard i at speed V and angle θ.

Equation 5 is based on the probability of the encroaching vehicle striking the first hazard within encroachment range i. For some predicted accidents, the errant vehicle will penetrate the first hazard within the encroachment range. For example, longitudinal barriers have a performance level beyond which vehicle restraint cannot be assured. When a vehicle is predicted to penetrate the first hazard within the range, it is assumed that the vehicle will strike the next hazard within the range.

Accident costs shown previously were calculated for traffic moving in only one direction. A similar procedure was developed for use on two-lane, two-way highways. In this application, the accident prediction algorithm is used twice. The procedure is first used to determine the costs of accidents resulting from vehicles leaving the right side of the roadway. Then accident costs are developed in an analogous procedure for accidents involving vehicles leaving the left size of the roadway. Encroachments from the right lane have been shown to comprise approximately 65 percent of all encroachments (6,8). For two-lane roadways, the remaining encroachments must originate from the left side of the travelway.

Encroachment Characteristics

The accident prediction model described requires a knowledge of certain characteristics of uncontrolled encroachments including frequency, speed, angle, and lateral movement. Few pure encroachment data are currently available. The largest data base containing pure encroachment information was collected on Canadian highways by Cooper (9). Unlike other efforts (10), this study involved highways with operating speeds in the same range as those on most U.S. highways today. Therefore findings from Cooper (9) were used to determine both encroachment frequency and lateral movement information. Cooper collected encroachment frequency data on relatively straight, flat sections of roadways of two different classes, four-lane divided and two-lane, two-way. These data included both controlled and uncontrolled encroachments. Accident data have been used to adjust encroachment frequencies from Cooper to eliminate controlled encroachments (11,12). The adjusted encroachment frequency curves are shown in Figure 3. Accident data have also been used to develop encroachment frequency adjustment factors, given in Table 1, to account for the effects of vertical or horizontal curvature on encroachment frequency (13).

Cooper also collected information on the lateral extent of encroachment. Information on lateral extent of encroachment from other sources is considered unrepresentative of modern accident characteristics because it involves either high-speed traffic (speed limit of 70 mph) (10) or was collected from accident data (8). Distributions of lateral vehicle movement developed from Cooper's data show few vehicles encroaching less than 10 ft before returning to the roadway. Many of the highways studied had paved or graveled shoulders that tend to hide evidence of encroachments with short lateral extent. Data on lateral extent of movement from Cooper have been adjusted by curve fitting the data points beyond 12 ft (the widest shoulder width in the study) to eliminate the effects of paved shoulders. Figure 4 shows both the raw and the adjusted lateral extent of movement distributions from Cooper (9). Note that for very short encroachments, the probability of lateral encroachment is greater than 1. Thus the curve in Figure 4 serves as an adjustment for the encroachments of short lateral extent that were not detected in the encroachment study.

No pure encroachment data published to date have contained any information on encroachment speed. Encroachment velocity and angle are known to be related. Therefore encroachment angle data are be-

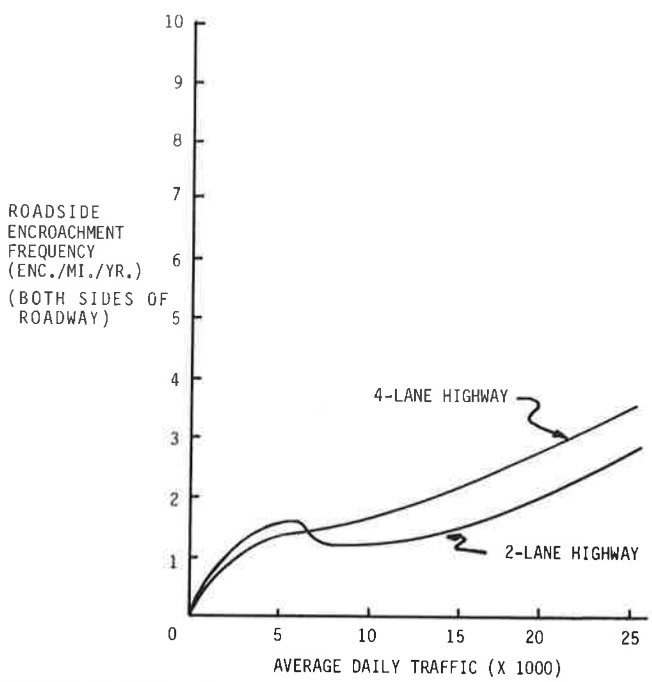

FIGURE 3 Adjusted encroachment frequencies (7).

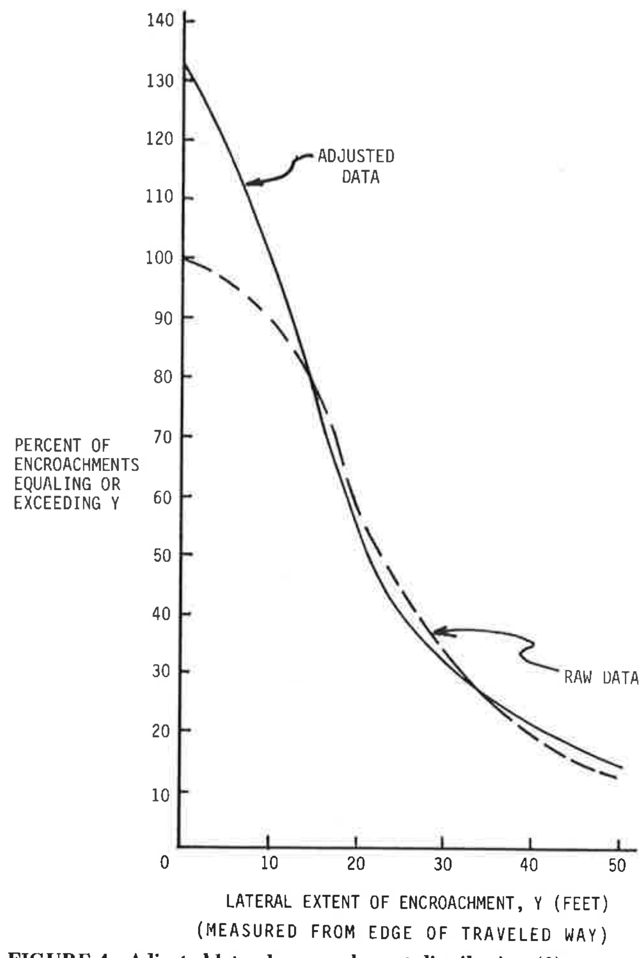

FIGURE 4 Adjusted lateral encroachment distribution (9).

TABLE 1 Encroachment Frequency Adjustment Factors for Horizontal and Vertical Alignment *(13)*

Roadway Curvature (degrees)	Encroachment Location with Respect to Curve: Inside — Uphill or Moderate Downhill Grade (>-2%)	Inside — Steeper Downhill Grade (≤-2%)	Outside — Uphill or Moderate Downhill Grade (>-2%)	Outside — Steeper Downhill Grade (≤-2%)
0-3	1.00	0.80	1.00	0.80
3.01-6	1.24	2.06	2.76	4.60
>6	1.98	4.00	4.42	9.00

lieved to be of little value without accompanying speed data. The best available method of estimating combined impact speed and angle distributions is through computer reconstruction of traffic accidents (7,14). Table 2 gives the distribution of freeway encroachment speeds and angles developed from Mak et al. (7) and Mak and Calcote (14). Although impact speed distributions developed from accident data are biased toward high impact speeds, accident severities from these distributions are more representative of real-world accidents than are severity estimates based solely on high-speed impacts. Distributions such as the ones given in Table 2 have been developed for a variety of functional classes of highways. The procedure described herein uses the appropriate distribution based on the functional class of highway under consideration.

Although small vehicles have been shown to be overrepresented in reported accident data, it is believed that much of this overrepresentation is the result of reduced crashworthiness of small automobiles rather than an increased encroachment probability. Few data are currently available to relate encroachment probability to vehicle size. Therefore it has been assumed that encroachment rates are independent of vehicle size and that the probability of an encroaching vehicle being of a particular size is equal to the decimal fraction of vehicles of that size in the traffic stream.

Accident Costs and Performance Levels

Accident costs of primary interest in a B-C analysis are the societal costs that result from occupant injury and vehicle damage and the direct highway agency costs that arise from damage to highway facilities. Societal and direct costs are strongly related to the performance of the highway appurtenance that is struck. For example, if a barrier contains and redirects an impacting vehicle, the expected societal costs will normally be well below those of an accident involving barrier penetration. Thus the performance level of a safety device must be defined before accident costs can be determined.

The impact performance of highway appurtenances is generally believed to be limited by the degree of impact loading the device can safely withstand or attenuate. For barriers, the degree of loading has been shown to be related to the impact severity (IS) (15-17):

$$IS = 1/2\ m(V \sin \theta)^2 \qquad (6)$$

where

- IS = impact severity (ft-lb),
- m = vehicle mass (lb-sec²),
- V = vehicle impact velocity (ft/sec), and
- θ = vehicle impact angle (angle between resultant velocity vector and face of barrier) (degrees).

For the B-C algorithm described herein, the performance level for barriers is measured in terms of impact severity. For other devices, such as crash cushions, performance level is measured in terms of total kinetic energy of the impacting vehicle.

Societal costs have traditionally been linked to the severity or probability of injury to vehicle occupants through a severity index scale. This scale was first developed in the mid-1970s (2) and has since been updated to reflect current cost figures (12). Table 3 gives the severity index scale from Bronstad and Michie (16).

TABLE 3 Severity Index Scale

Severity Index	PDO Accidents[a] (%)	Injury Accidents (%)	Fatal Accidents (%)	Societal Cost per Accident ($)
0	100	0	0	1,600
1	85	15	0	3,450
2	70	30	0	5,500
3	55	45	0	7,500
4	40	59	1	15,800
5	30	65	5	42,400
6	20	68	12	87,900
7	10	60	30	203,000
8	0	40	60	393,000
9	0	21	79	513,000
10	0	5	95	614,000

[a]PDO refers to those accidents in which property damage only is involved.

Crash testing and simulations have been used to estimate impact severities of many common highway hazards in terms of vehicle accelerations and damage. Vehicle accelerations have been linked to occupant injury by comparing damage to crash test vehicles and damage to vehicles involved in traffic accidents (18). Procedures from Olson et al. (18) can be used to estimate crash test injury probabilities from measured vehicle accelerations. However, crash testing is normally conducted at speeds near 60 mph. A large gap therefore exists in severity indices data for roadside features at speeds of less than 60 mph. In the absence of test data, the researchers have assumed a linear relationship between the severity index, given in Table 3, and impact speed. It should be noted that linearity is assumed between severity index and impact speed, not severity per se. As can be seen from Table 3, accident costs increase exponentially as the severity index increases. Figure 5 shows severity indices of W-beam guardrail accidents derived from measured crash test accelerations. Crash test data used in the development of Figure 5 were collected from tests involving full-sized, subcompact, and mini-sized vehicles. Note that most crash tests involve impact angles of 15 and 25 degrees. Therefore severity indices for other impact angles must be interpolated and extrapolated from the curves shown in Figure 4.

Costs that arise from damage to a highway ap-

TABLE 2 Combined Impact Velocity and Angle Distributions from Accident Studies *(9,10)*

Speed (mph)	Combined Gamma Function Probabilities for Angle (degrees) <5	5-15	15-25	25-35	35-45	>45	Total
<20	.0429	.1862	.1163	.0466	.0157	.0067	.414
20-30	.0268	.1163	.0726	.0291	.0098	.0042	.259
30-40	.0168	.0732	.0458	.0183	.0062	.0026	.163
40-50	.0093	.0392	.0245	.0098	.0033	.0014	.088
50-60	.0044	.0191	.0119	.0048	.0016	.0007	.043
>60	.0035	.0152	.0095	.0038	.0013	.0005	.034
Total	.104	.4490	.2810	.1120	.1790	.016	

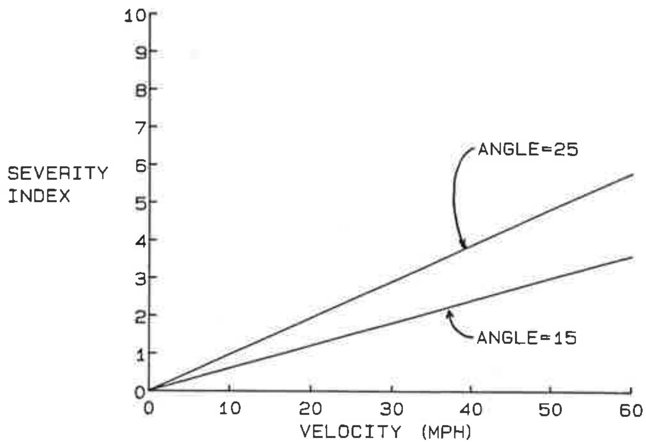

FIGURE 5 W-beam guardrail crash test severity indices.

purtenance are generally believed to be proportional to the degree of impact loading on the appurtenance. Bronstad and Michie (16) and Ivey et al. (17) have shown that IS is approximately proportional to the degree of barrier loading and that it follows that barrier repair costs should be roughly proportional to IS. Figure 6 shows repair costs for W-beam guardrail estimated from crash test results. Repair costs of other safety appurtenances are assumed to be roughly proportional to the total kinetic energy of the impacting vehicle. More detailed descriptions of performance level and accident cost determination can be found elsewhere (5).

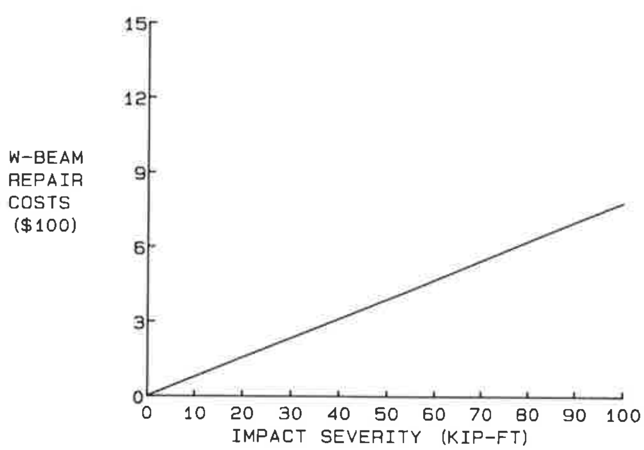

FIGURE 6 W-beam guardrail repair costs.

Improvements to the B-C Model

The B-C model described herein has incorporated most of the improvements found in all previous models. Additional modifications have been added to improve accuracy and enhance the capability of the algorithm, including shielding of one obstacle by another, accident cost and appurtenance repair as a function of accident impact conditions, use of reconstructed accidents to predict impact conditions, and relating appurtenance performance to impact conditions.

Applications

The encroachment probability model on which the B-C model is based is general in nature and can therefore be used to study a wide variety of highway conditions. These models are well suited for use in developing general safety treatment guidelines or policies (5).

For example, a common problem faced by many highway engineers is how to safely treat the slope hazard at deep fill sections. In such cases an engineer must determine whether to place the slope breakpoint away from the shoulder by increasing the amount of fill material and to use a barrier to shield the slope. Safety treatment alternatives for deep fill sections, shown in Figures 7 and 8, include increas-

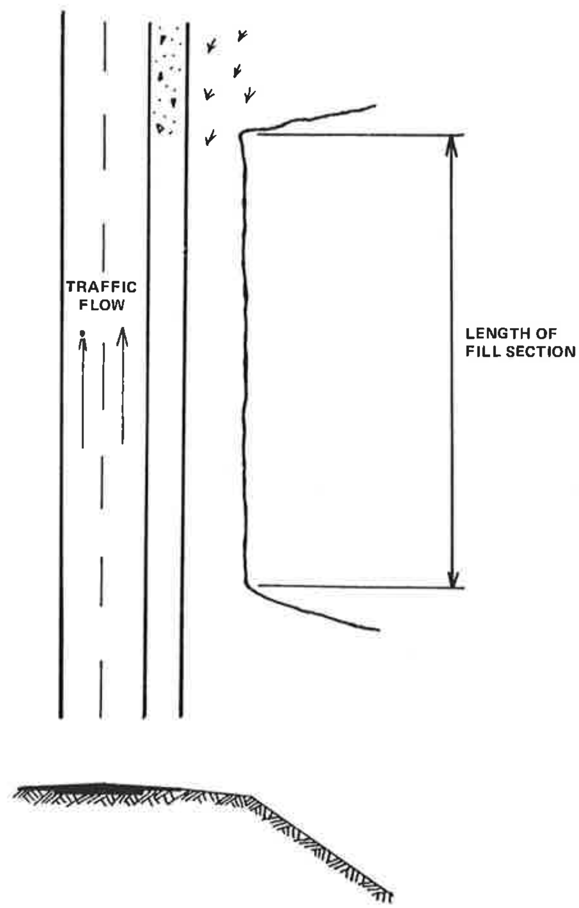

FIGURE 7 Typical barn roof fill section.

ing the available recovery area by moving the slope breakpoint away from the travelway and using W-beam guardrail to shield the slope. Typical cost and severity data for safety treatments of a 20-ft-deep fill section are presented next. (Note that for this example the severity of a 60-mph encroachment onto a deep 1 1/2:1 slope is estimated to correspond to a severity index of 8.0. Impact severities for other speeds are estimated on the basis of the assumed linear relationship between impact speed and severity index discussed previously. Further, the severity of impact with steep roadside slopes is assumed to be relatively independent of impact angle.)

1. Safety alternative costs
   - W-beam barrier, $15/ft;
   - Repair costs, $7.8/ft-kip (IS) (see Figure 6);
   - Performance level, 97 kip-ft; and
   - Cost of additional fill, $5/yd (in place).
2. Accident severity indices
   - W-beam barrier

Impacts below PL (Figure 5)
Impacts above PL SI = 7.0
• 1.5:1 slope SI = 0.133 x impact velocity (mph)

Additional input data sources and highway descriptors were assumed to be as follows:

Variable	Assumed Value or Source
Accident costs	Table 3
Discount rate	4 percent
Project duration	20 years
Roadway alignment	Straight, flat
Functional highway class	Freeway
Type of highway	Four-lane, divided
Encroachment speeds and angles	Table 2
Lateral vehicle movement	Figure 4

The B-C model was then used to determine the relative benefits and costs for barrier-protected and unprotected slopes with the slope breakpoint offset 3, 15, 30, and 45 ft from the traveled way. The most cost beneficial alternative was determined for a wide variety of fill section lengths and traffic volumes. General guidelines for safety treatment of deep fill sections were then developed as shown in Figure 9.

Another application of the B-C analysis algorithm described herein is in the study of special or new safety appurtenances and unusual sites. General guidelines, such as those shown in Figure 9, cannot be applied to all situations. Further, some safety appurtenances are designed for special sites that cannot be generalized. Highway engineers have expressed a need for a method of studying these special situations whenever they arise. Finally, this algorithm provides for the first time an objective method for determining optimum barrier flare rates and optimum barrier runout lengths in front of fixed hazards.

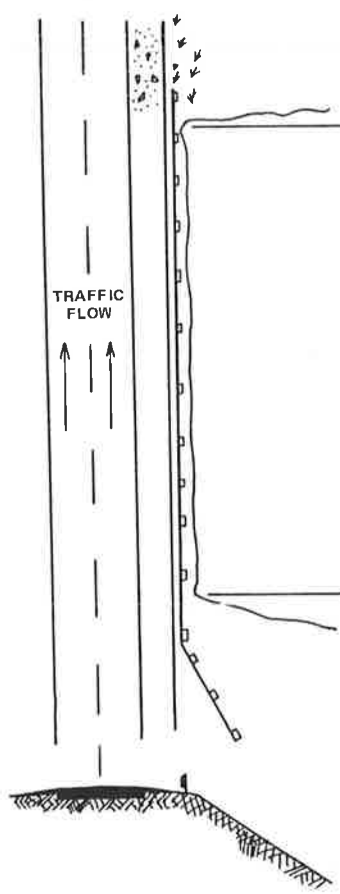

FIGURE 8 Typical guardrail placement on fill section.

Limitations

As shown in the foregoing discussion, encroachment models have been developed to study accident frequencies of roadside hazards. These models are not designed to examine other types of accidents such as multiple-vehicle accidents. Therefore this technique cannot be used to study most safety treatments at intersections or to determine warrants for median barrier applications.

Another limitation of encroachment probability models is found in the determination of accident severity based on predicted impact conditions. Accident severity is an important factor in determining the total accident costs of a safety alternative. There is still only a tenuous link between impact conditions and accident severity. Further, accident severities of some hazards, such as dropoffs and roadside slopes, are quite difficult to quantify. Thus the model has a limited value in the analysis of problems in which the severity of potential accidents cannot be estimated.

CONCLUSIONS

The B-C procedures described herein represent a significant improvement over existing procedures in the accuracy and versatility of analysis of the need for safety improvements. The technique is based on the best accident, encroachment, and impact severity information currently available. When better data become available, they should be incorporated into the procedures. The computer model can be used to develop general roadside safety appurtenance use guidelines. The FHWA has adopted the model for developing barrier use guidelines for the update to the 1977 barrier guide.

Microcomputer versions of this program should allow practicing highway engineers to apply these procedures without the difficulty associated with most other methods. Therefore this B-C model should allow more potential safety improvement projects to be analyzed in terms of the expected benefits and costs, thereby resulting in a more efficient application of available highway improvement dollars.

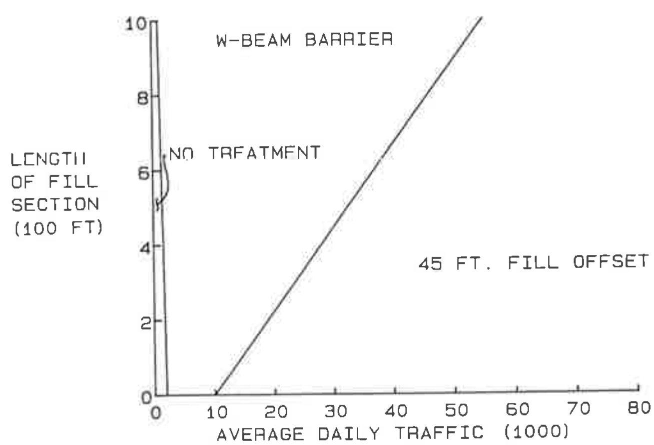

FIGURE 9 Guidelines for safety treatment of fill section 20 ft deep.

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