TRANSPORTATION RESEARCH

RECORD

No. 1468

Highway and Facility Design

Recent Research on Roadside Safety Features

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
WASHINGTON, D.C. 1994

Transportation Research Record 1468
ISSN 0361-1981
ISBN 0-309-06102-4
Price: $26.00

Subscriber Category
IIA highway and facility design

Printed in the United States of America

Sponsorship of Transportation Research Record 1468

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Foreword

The papers in this volume were presented at the 1994 Annual Meeting of the Transportation Research Board in sessions sponsored by the Committee on Roadside Safety Features. Michie and Bronstad refute the claim that highway guardrail accidents are a likely possible cause of injuries or fatalities by examining both reported and unreported accidents. Viner et al. address the question of whether there are differences in the relative safety experiences in crashes with roadside safety hardware by vehicle type. Mak and Sicking investigate the frequency and severity of accidents involving trucks penetrating or rolling over bridge railings, which appear to dominate the selection criteria for railings along bridges. Mak et al. present the results of full-scale crash tests and evaluate the impact performances of single-slope concrete bridge rails. Ray describes a software program that can be used by highway designers to assist in assessing the safety of various roadside design alternatives. Bullard et al. present two bridge railing designs, consisting of metal railings on top of concrete parapets, for use along sidewalks on urban bridges. Hall et al. discuss the results of a survey supporting the need for continued development of the AASHTO roadside safety analysis model, particularly in the area of severity indexes. Sicking et al. describe a crashworthy telescoping tube terminal developed for box-beam guardrails. Mak and Hille present the results of full-scale crash tests of a swing-away mailbox support designed by the Minnesota Department of Transportation for use in locales where snow and ice removal during the winter presents a problem. Mak et al. identify, through the use of computer simulation, potential problems with the performance of guardrail to bridge rail transition designs currently in use by the Tennessee Department of Transportation. Alberson and Ivey present a new design of a breakaway utility pole that improves performance and reduces both initial costs and maintenance costs.

Highway Guardrails: Safety Feature or Roadside Hazard?

JARVIS D. MICHIE AND MAURICE E. BRONSTAD

On the basis of reported accident data, from 50 to 60 percent of guardrail accidents involve an injury or a fatality. From this highway engineers have concluded that guardrail installations are a roadside hazard and should be used only when absolutely necessary. On the other hand, by using a more in-depth study of accident data and estimates of the frequency of unreported accidents, a more positive view of guardrail performance is projected. Specifically, unreported guardrail impacts represent approximately 90 percent of the total impacts, with the other 10 percent being reported. Assuming no injuries or fatalities in the unreported drive-away accidents, only 6 percent of all guardrail impacts involve any injury or fatality. Furthermore, analysis reveals that terminals, as opposed to segments of typical lengths, are overrepresented in the accident data, comprising up to 40 percent of the guardrail accidents resulting in fatalities or injuries. Also, clinical data indicate that many of the 6 percentile accidents resulting in injuries or fatalities involve (*a*) guardrail installations that are obsolete, improperly constructed, or inadequately maintained, (*b*) noncrashworthy ends, or (*c*) collisions that are outside the practical design range of modern guardrail systems. It is concluded that properly installed and maintained longitudinal barriers may be successfully performing in 97 to 98 percent of all design range length-of-need impacts, with only 2 to 3 percent of the impacts causing occupant injuries or fatalities, a stark contrast to the erroneous 50 to 60 percent based on only reported accidents.

A cursory examination of fixed objects involved in ran-off-the-road single-vehicle accidents (Table 1) (*1*) reveals that guardrails rank as the third most frequent roadside object struck in fatal accidents. About 50 percent (*2*) to 60 percent (*3*) of reported guardrail accidents involve an injury or fatality.

In 1964 the authors of HRB *Special Report 81* (*4*), although not discussing the relative hazard of guardrails, advised engineers with the following statement: "As a basic principle, the highway should be designed through judicious arrangement and balance of geometric features, to preclude or minimize the need for guardrail." In 1968 the authors of *NCHRP Report 54* (*5*) cautioned engineers as to the relative hazard represented by guardrails: "Even properly designed guardrail and median barrier installations are formidable roadside hazards and provide errant vehicles with only a relative degree of protection." This statement was slightly modified in the 1971 *NCHRP Report 118* (*6*): "the longitudinal barrier affords only a relative degree of protection to vehicle occupants as a collision with this type of barrier can result in a severe accident."

Again, in 1977 according to the AASHTO Barrier Guide (*7*): "it cannot be overemphasized that a traffic barrier is itself a hazard." From the 1989 AASHTO *Roadside Design Guide* (*1*) the reader is advised that

> 60 percent of the fatal accidents . . . either overturned or collided with a fixed object. Some of these fixed objects were manmade and included . . . traffic barriers. Barrier warrants are based on the premise that a barrier should be installed only if it reduces the severity of potential accidents."

One researcher (*8*) concluded the following: "Barriers are unsafe. *When in doubt, leave them out!*"

From these statements, a reader might conclude that guardrails or longitudinal traffic barriers are not only a roadside hazard but that the perceived safety benefit, if any, is decreasing with time. It is the authors' opinion that this perception of guardrail performance is based on incomplete and misleading accident data and that the conclusions are invalid.

The purpose of this paper is to examine and assess the conventional wisdom of guardrail performance.

PURPOSE OF GUARDRAILS

In the incorporation of forgiving roadside technology into the national highway and street network, a first step for highway agencies is to establish an appropriate clear-zone width that is commensurate with the type of highway, local conditions, and funding. A minimum clear-zone width of 9.2 m (30 ft) has been FHWA policy since the mid-1960s for Interstate highways and for other roads when it is economically feasible. For lower-speed, less-traveled roads with restricted rights-of-way, a clear-zone width of less than 9.2 m (30 ft) is acceptable. Once a clear-zone width is established the hierarchy of safety treatment is the following: (*a*) remove all fixed objects and hazards that can cause abrupt decelerations or upset an errant vehicle and make the roadside area as smooth and level as possible; (*b*) if certain fixed objects such as sign and luminaire supports cannot be removed, then they should be converted to breakaway designs; and (*c*) if fixed objects and hazards cannot be removed or converted to breakaway designs, then the fixed objects or hazards should be shielded by a longitudinal barrier such as a guardrail. The purpose of a guardrail is to redirect an errant vehicle away from a roadside fixed object or hazard located in the clear zone that otherwise cannot be safety treated.

By necessity, the guardrail is located closer to the traveled way than the hazard or object that it is shielding and thus is exposed to a greater frequency of impact. Moreover, the length of a guardrail installation properly shielding a point hazard such as a pole increases the target exposure and the potential number of vehicle impacts; for longer roadside hazards such as steep embankments, the exposure of added guardrail installation length becomes insignificant.

Objective criteria for identifying roadside conditions needing guardrail shielding are specific for fixed objects and steep embankments located within the clear zone. For moderately sloped embankments of about 4:1 and steeper, an equal severity curve was

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TABLE 1 Fatalities from Impacts with Fixed Objects by Object Type

Fixed Object	1983	1984	1985	1986	1987
Tree/shrub	2841	3021	2989	3444	3299
Utility pole	1377	1426	1298	1495	1406
Guardrail	1310	1446	1258	1374	1326
Embankment	1288	1264	1211	1332	1396
Culvert/ditch	1259	1198	1337	1472	1393
Curb/wall	865	899	982	960	861
Bridge/overpass	803	738	628	577	571
Concrete barrier	263	240	225	197	203
Sign or light support	488	480	508	551	538
Other pole/support	495	434	481	518	495
Fence	434	455	431	478	484
Building	110	105	101	100	108
Impact attenuator	16	10	14	9	18
Other fixed object	565	629	630	699	729
TOTALS	12114	12345	12093	13206	12827

Source: Fatal Accident Reporting System (FARS), NHTSA

developed by Glennon and Tamburri in 1967 (*9*) and was subsequently revised by Ross in 1977 (*7*). On many secondary roads with low traffic volumes, guardrail installations have been used only at locations with adverse accident histories, even though they may be warranted in many other locations. Hence, many installations are placed at locations with high degrees of impact exposure with an attendant large number of reported accidents.

Because of practical limits, guardrails are typically developed to accommodate a large majority but not all vehicle impacts. For instance, guardrails are designed to perform with passenger sedans with masses in the 815- to 2040-kg (1,800- to 4,500-lb) range striking the barrier at 0 to 97 km/hr (60 mph) and at an angle of 0 to 25 degrees. Most guardrails will perform with less certainty for vehicles with masses greater than 2040 kg (4,500 lb) unless the speed and angle of approach are significantly reduced from 97 km/hr (60 mph) and 25 degrees. Also, guardrails are not specifically designed to handle motorcycles. The authors know that a number of guardrail failures occur when the vehicle or the impact conditions are beyond the design capacity. Classification of guardrail performance as unsatisfactory if failure occurs under these conditions would be akin to judging the performance of a collapsed 10-ton-capacity bridge brought down by a tractor trailer weighing 36,000 kg (40 tons).

Guardrail performance is dependent on the condition at impact; this includes both proper installation and maintenance. Evidence abounds that many guardrail installations are improperly installed or modified in critical details such as improperly flaring the guardrail ends or installing guardrails that are not maintained to the proper height and alignment or that are of insufficient length to properly shield the hazard. Such nonconformance is rarely detected or reported by investigating officers, and the fatality is attributed to a guardrail impact, reinforcing the notion that guardrails are hazardous.

IN-SERVICE PERFORMANCE

In essence highway agencies do not know the degree to which traffic barriers perform in service or specifically how well a specific guardrail design compares with another type. A procedure to perform in-service evaluation of safety appurtenances was recommended in *NCHRP Report 230* (*10*) and was then refined in *NCHRP Report 350* (*11*). The procedure reflects the magnitude of the task necessary to quantify the safety performances of roadside features. Such studies would include the following items:

- Exposure data that would include all impacts and not just those typically reported by police officers. Some method of identifying nonreported accidents is required, such as periodic inspection of barrier scrapes or documentation of tire marks on soft shoulders by maintenance forces. It is noted that the threshold damage cost for reporting property damage-only (PDO) accidents varies greatly among agencies, which introduces uncertainties in current data bases.
- The design feature and the actual condition of the safety feature struck. For example, there are a number of guardrail designs, such as cable or metal beam systems, each with unique performance characteristics. Importantly, the condition of the installation at the time of impact can directly affect the collision outcome. Low barrier height, improperly tensioned anchors, or an uneven approach terrain such as curbs can also reduce the effectiveness of an installation. Finally, a number of obsolete guardrail installations are still in existence today (1993), and these have little capacity to redirect modern automobiles.
- Reconstruction of reported accidents. Reported accidents need to be reconstructed to the extent that the impact velocity and angle of approach are determined, and these parameters need to be related to occupant injuries by means of an anthropometric model such as the flail space model. The trajectory of the vehicle after impact with the guardrails should be delineated to identify other harm-producing events.

Although some items of the recommended in-service evaluation procedures have been used in specific projects, the authors are unaware of any comprehensive use of the procedure.

The conventional wisdom that guardrails are hazards and offer a minimum degree of protection for errant motorists is based on incomplete and in many cases faulty data. Deficiencies in these data are attributed to several sources:

- Only severe impacts that include injuries or a disabled vehicle are generally reported; brush hits in which the vehicle is not severely damaged or occupant injuries do not occur are not generally recorded. Hence, only the most severe impacts can be analyzed, and little is known about the number and extent of the drive-aways. For this reason, the total number of impacts or even the failure rate (i.e., number of failures as a percentage of total impacts) cannot be calculated. If the number of reported accidents make up 90 percent or more of all impacts, then the reported accident would be fairly representative of all impacts. On the other hand, if the number of reported accidents make up less than 50 percent of all accidents, such inferences to their being representative would be weak or even nonexistent.
- Seldom is the type of guardrail indicated in the accident report because most officers are untrained in this technology. The guardrail could be one of the many modern systems or it could be an obsolete design such as the Tuthill system, which has not been built in more than 20 years. Moreover, sufficient information to document the condition of the guardrail at the time of the impact or whether it was properly installed is nearly always lacking. Barrier

failures (i.e., accidents resulting in serious injuries or fatalities) may be caused by an obsolete, improperly maintained installation rather than a generic guardrail in good condition.

- Accidents involving guardrails and resulting in fatalities and injuries are generally grouped according to the first harmful event, even though there may be several harmful events in an accident scenario and the guardrail impact may not be the most severe or even injury producing. For instance, studies have shown that the redirected vehicle can strike other unshielded fixed objects, overturn, or even interact with following or adjacent traffic. In some cases the vehicle may penetrate or vault an obsolete system and strike the shielded fixed object.
- Guardrail failures generally include all reported impacts, even events well beyond the design envelope. Combinations of vehicle mass, speed, and impact angles that exceed the crash test values may result in barrier failure (e.g., excessive deflection or penetration or severe injuries). However, it is arguable whether the occurrence of such accidents should in any way suggest that the installation is a hazard.

The intent of this section was to point out some of the inadequacies of current data systems that might lead to false conclusions regarding the efficacies of guardrail systems.

PERTINENT STUDIES

In evaluating the efficiencies of guardrails from existing data, four factors have been explored by researchers. The first factor is the magnitude of unreported accidents. The second factor is the effect of accident classification by first harmful event rather than most harmful event. The third factor is the significance of accidents in which impact conditions exceed the barrier capability. Finally, the fourth factor is whether the impact occurred within the length of need or on the end. These are discussed in more detail in the following paragraphs.

Unreported Accidents

Historically, only a part of all vehicle collisions are reported to police; the unreported collisions generally involve only minor property damage, although there may be a few exceptions in which even accidents resulting in fatalities and serious injuries fail to get into the reporting system. Accidents are broadly grouped according to descending severity by those involving fatalities (F accidents), injuries (I accidents), and PDO accidents. The reporting rate varies among states and locales within a state owing to several factors:

- The threshold dollar limit on PDO accidents varies. In some areas reports are prepared for PDO accidents in which \$200 damage has occurred, whereas other agencies have established higher limits such as \$500, \$1,000, or even \$2,000. When the investigating officer judges that the damage does not satisfy the threshold, a report is not prepared.
- The degree of reporting can vary with the proximity and availability of investigating officers. For remote sections of highway, motorists can make arrangements to leave the accident scene before the officer's arrival. Also, in urban areas during adverse weather conditions, a large number of fender benders can inundate the local reporting agency, encouraging the involved motorists to make other arrangements unless the collision was serious.
- In several areas the state aggressively pursues motorists who have damaged public property in a collision to get full reimbursement for the repair of any damage. For this reason many motorists will depart a site when a breakaway sign or luminaire support is knocked down without reporting the incident.
- Motorists with invalid licenses or inadequate property damage insurance or motorists possibly driving under the influence of alcohol are motivated to drive away.

There are probably other reasons that certain collisions go unreported, but these are believed to be the major ones.

To investigate the magnitude of unreported accidents with longitudinal barriers, two studies have been performed, one by Galati (*12*) and one by Carlson et al. (*13*). In 1969 Galati (*12*) investigated unreported accidents on the Schuylkill freeway median barrier. For the study the barrier was painted white. Once a month both sides of the median barrier were filmed, and the scuff marks and other damage were immediately repaired. Galati then correlated the scuff marks and damage areas with police accident reports that had been processed through the system. Using the premise that each scuff mark represented a collision or accident, he found that only one of eight collisions was reported, or about 13 percent.

In a like manner, Carlson et al. (*13*), using maintenance forces in New York, found that almost 90 percent of longitudinal barrier impacts are hit-and-run impacts and are never reported.

From these two studies it can be readily concluded that accident data relating to longitudinal barriers represent only about 10 to 13 percent of all barrier collisions and are probably skewed to the most severe type of accidents.

In 1986 Bryden and Fortuniewicz (*3*) reported on a detailed analysis of 3,302 reported accidents in which a roadside barrier was the first harmful event. Their tabular data have been modified by the authors of this paper to incorporate the estimated 90 percent unreported accidents in Table 2. It is assumed that the unreported accidents did not involve any injury or fatal events. Also, the data reflect both acceptable and unacceptable barrier performances. For instance, cases involving vehicle snagging, penetration, and vaulting are included along with redirection performance. Whereas the total number of accidents involving fatalities plus injuries (44 + 312 + 853 + 741) of 1,950 represents 59 percent of the 3,302 reported accidents, it is only 5.4 percent of the 36,302 estimated total impacts. Even including all reported accidents, some with obsolete barriers, the barriers performed without occupant injuries in 95 percent of the impacts. Clearly, this is a good performance record and removes the basis for the conventional wisdom that barriers are inherently hazardous.

A further analysis of Table 2 of second events reveals that 871 (2.4 percent) of the 36,302 total impacts reported a second event such as striking a fixed object. It is noted that only six impacts (less than 0.02 percent) involved a second event with a motor vehicle.

First and Most Harmful Event

Highway accidents may involve a single event or a sequence of events. For instance, two vehicles may collide and then one rebounds into a second vehicle and then into a roadside feature such as a luminaire support or guardrail. The most significant property damage or occupant injury may occur as a result of any one of the events or may be due to the cumulative effects of all of the events. Because it may require extensive accident reconstruction to sort out

TABLE 2 Injury Severity Related to Vehicle Damage, Barrier Function, and Secondary Collisions (3)

		Injury Severity											
		Fatal		A Injury		B Injury		C Injury		No Injury		TOTAL	
		No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Reported Accidents	Barrier Function												
	Redirect	19	.85	190	8.47	576	25.68	533	23.76	925	41.24	2243	100
	Stop	1	.31	38	11.80	95	29.50	59	18.32	129	40.06	322	100
	Snag	1	5.88	2	11.76	5	29.41	3	17.65	6	35.29	17	100
	Penetrated	5	4.63	20	18.52	48	44.44	22	20.37	13	12.04	108	100
	Ran Under	0	0	3	37.50	1	12.50	4	50.00	0	0	8	100
	Broke Thru	5	5.81	11	12.79	34	39.53	19	22.09	17	19.77	86	100
	Went Over	12	5.71	43	20.48	63	30.00	70	33.33	22	10.48	210	100
	Deflect to Fx Obj	0	0	1	7.69	10	76.92	2	15.38	0	0	13	100
	Unknown	1	.34	4	1.36	21	7.12	29	9.83	240	81.36	295	100
	Total	44	1.33	312	9.45	853	25.83	741	22.44	1352	40.94	3302	100
Non Reported Impacts (Est)		-		-		-		-		33000		33000	
Total Impacts		44		312		853		741		34352		36302	
Reported Accidents	Second Event												
	Motor Vehicle	0	0	0	0	1	16.67	2	33.33	3	50.00	6	100
	Pedestrian	0	0	0	0	0	0	0	0	2	100.00	2	100
	Other Not Fixed Obj	0	0	0	0	0	0	1	50.00	1	50.00	2	100
	Light/Utility Pole	4	7.14	9	16.07	25	44.64	15	26.79	3	5.36	56	100
	Guardrail	1	1.00	11	11.00	36	36.00	25	25.00	27	27.00	100	100
	Sign Post	1	3.85	1	3.85	11	42.31	6	23.08	7	26.92	26	100
	Tree	7	6.48	21	19.44	41	37.96	23	21.30	16	14.81	108	100
	Building/Wall	0	0	0	0	3	75.00	1	25.00	0	0	4	100
	Curbing	1	14.29	3	42.86	0	0	3	42.86	0	0	7	100
	Fence	1	5.88	5	29.41	7	41.18	3	17.65	1	5.88	17	100
	Bridge Structure	3	7.69	5	12.82	15	38.46	10	25.64	6	15.38	39	100
	Culvert/Head Wall	1	7.69	7	53.85	3	23.08	2	15.38	0	0	13	100
	Median/Barrier	0	0	3	20.00	6	40.00	2	13.33	4	26.67	15	100
	Snow Embankment	0	0	0	0	4	50.00	0	0	4	50.00	8	100
	Earth Elem/RC/Ditch	2	1.09	27	14.75	62	33.88	68	37.16	24	13.11	183	100
	Fire Hydrant	0	0	1	50.00	0	0	1	50.00	0	0	2	100
	Other Fixed Object	0	0	3	60.00	0	0	0	0	2	40.00	5	100
	Overturned	15	5.81	55	21.32	107	41.47	69	26.74	12	4.65	258	100
	Fire/Explosion	0	0	0	0	3	50.00	0	0	3	50.00	6	100
	Submersion	2	50.00	0	0	1	25.00	0	0	1	25.00	4	100
	Ran Off Rdwy Only	0	0	0	0	2	66.67	1	33.33	0	0	3	100
	Other Non Collision	1	14.29	0	0	2	28.57	2	28.57	2	28.57	7	100
	Fixed Obj Sub Tot	21	3.60	96	16.47	213	36.54	159	27.27	94	16.12	583	100
	All Second Ev SubT	39	4.48	151	17.34	329	37.77	234	26.87	118	13.55	871	100
	No Second Event	5	0.21	161	6.62	524	21.55	507	20.86	1234	50.76	2431	100
	Total	44	1.33	312	9.45	853	25.83	741	22.44	1352	40.94	3302	100
Non Reported Impacts (Est)		-		-		-		-		33000		33000	
Total Impacts		44		312		853		741		34352		36302	

the severity of each of the events, accidents are generally coded according to the first harmful event, although in many cases it may not be the most harmful event. This procedure eliminates the need for sophisticated reconstruction and engineering judgment by investigating officers and promotes consistency in the data. On the other hand, this procedure can distort the severity risk of certain roadside features.

In a recent study, Viner (*14*) examined the relationship between first and most harmful events. Harmful events in ran-off-road fatalities are compared in Table 3. Note that overturn is the predominate most harmful event and hitting a tree is the second most predominate. Since the number of overturn most harmful events of 4,820 is double the first harmful events of 2,492, apparently vehicles interacting as a first harmful event with other roadside features subsequently overturned. In longitudinal barriers that would consist of guardrails, concrete traffic rails, bridge rails, and other traffic rails, the number of most harmful events is less than the number of first harmful events in all cases. Although one cannot be certain from these data, it is believed that a number of vehicles that are redirected in the first event subsequently roll over, producing the most harmful event. As shown in Table 2, 15 of 39 fatal second events (or 38 percent) involved an overturn, some of which may have been induced by atypical barrier conditions. In any case it appears that the first event with a longitudinal barrier is not causing the number of fatalities that were once thought to be the case. What may be most important here is that the stability of the vehicle as it departs from the collision with the traffic barrier in the first event is more significant than the injury-causing dynamics of the barrier collision.

Substandard Barriers and Excessive Impact Conditions

Most longitudinal barrier accident statistics are composites of both modern and obsolete systems, both properly and improperly con-

TABLE 3 Harmful Events in Ran-off-Road Fatalities

Harmful Event	First Harmful Event	Most Harmful Event
Tree	2,870	3,246
Overturn	2,492	4,820
Utility pole	1,235	1,298
Embankment	1,187	601
Guardrail	1,101	456
Ditch	750	302
Other	565	613
Culvert	537	281
Curb	506	117
Other fixed object	461	219
Other post	457	237
Fence	421	156
Sign post	295	99
Bridge pier	211	255
Concrete traffic barrier	211	83
Bridge rail	194	118
Luminaire support	148	146
Wall	143	127
Boulder	133	76
Bridge end	122	95
Building	101	143
Immersion	98	354
Shrubbery	66	13
Other noncollision	53	40
Other traffic rail	33	16
Fire hydrant	28	9
Impact attenuator	7	3
Overhead sign post	6	11
Unknown	4	272
Fire/explosion	0	229
Totals	14,435	14,435

structed and maintained systems, and collisions that are within and beyond the typical design performance range. To fairly appraise longitudinal barrier performance, it seems appropriate to eliminate those accident data involving defective barrier installations and those accidents in which the vehicle type, mass, impact speed, or orientation are outside typical crash test conditions.

In a New York Department of Transportation study, Bryden and Fortuniewicz (*3*) produced an analysis of traffic barrier accidents as shown in Table 4; using those data the authors have rearranged the format into Table 5. Although the data set is quite extensive, it represents conditions in only one state and may not be representative of national statistics. Nevertheless, the data set certainly illustrates the nature of substandard barriers and excessive impact conditions and may suggest the order of magnitude of these factors. In Table 4 the total number of barrier accidents of 3,302 is the same as that reported in Table 2. Of the 3,302 accidents, Bryden and Fortuniewicz (*3*) reported that 811 involved obsolete systems, which involved the highest proportion of fatal accidents (i.e., 2.22

TABLE 4 Traffic Accident Injury Severity (*3*)

Accident Category	No. of Accidents	Percent Injury Severity				
		Fatal	A	B	C	None
All	270,688	0.71		63.5		35.8
All Roadside	40,163	1.50		74.2		24.3
All Barrier	3,302	1.33	9.45	25.8	22.4	40.9
Obsolete Barrier[a]	811	2.22	13.19	30.6	23.4	30.6
Current Barriers[b]	2,071	1.16	9.37	27.0	24.6	37.9
Ideal Barrier[c]	1,313	0.53	7.31	25.1	24.7	42.4

[a]Non-standard, older systems.

[b]Current New York standard systems; includes ends, some impacts beyond typical performance range, and some barriers in need of repair/maintenance.

[c]Current New York standard systems in proper condition; impacts within typical performance range - no ends.

TABLE 5 Analysis of Barrier Performance Based on Reported Accident Data

		PDO		Fatal/Injury	
	Total No.	No.	%	Number	Percentage
Current Barrier					
Ideal	1313	557	41	756	39
Atypical*	758	227	17	531	27
	2071	784	58	1287	66
Obsolete Barrier	811	248	18	563	29
Undefined (Barrier/ Impact)	420	320	24	100	5
All Reported Barrier Impacts	3302	1352	100	1950	100

*Atypical - impacts involving ends, barriers in need of repairs, and/or impacts beyond performance range (e.g. motorcycles, heavy trucks, speeds or angles clearly beyond design range).

percent). The category Current Barriers represents accidents with modern New York standard barriers including ends and installations in need of repair or maintenance and impacts that were beyond the design performance range of the installations. After excluding these anomalies, 1,313 ideal barrier impact accidents remained.

These data were reformulated in Table 5 to assess the severity of the accidents in terms of the injury plus fatality level (I + K accidents). As shown in Table 5, 29 percent of the I + K accidents are attributed to obsolete barrier installations, 27 percent of the I + K accidents are attributed to atypical impacts, and 39 percent of I + K accidents are attributed to ideal barrier impacts. From these findings it is evident that a significant part of the 6 percent barrier accidents that result in an injury or fatality may be attributed to obsolete installations and excessive impact conditions.

Length-of-Need and Terminal Sections

Generally, police-level accident data do not indicate whether the impacts occurred along a typical barrier section (i.e., length-of-need) or at the upstream end. Recognizing that modern crashworthy barrier terminals have not been universally implemented, one might surmise that barrier ends could be overrepresented in barrier accident data.

Griffin (*15*), using guardrail accident data reported in Texas in 1989, found that 20.1 percent of cases involved a guardrail end and that 79.9 percent involved something other than a guardrail end, as shown in Table 6. Although he lacked the necessary exposure data, he surmised that a greater percentage of impacts with guardrail termini are reported because of the severe nature of the collision, such as a high percentage of vehicle rollovers. Also noted in Table 6 is that the turndown guardrail terminal is involved in more than 41 percent of fatal accidents involving guardrails, in contrast to only 20.1 percent of nonfatal accidents.

It is recognized that the Texas data may not be representative of national data, in which one-fifth of barrier impacts involve a terminal. However, it is the authors' opinion that a significant number of barrier I + K accidents involve terminals, a hypothesis that has proved to be more technically challenging than accidents involving the typical barrier section.

TABLE 6 Estimated Numbers of Guardrail Accidents on Texas State-Maintained Highway System by Point of Impact (1989)

	Reported Accidents					
	Accidents on Turned Down Ends		Accidents Not on Turned Down Ends		Total	
	No.	%	No.	%	No.	%
Guardrail Accidents:						
Non-Fatal Accidents	700	95.1	2784	98.2	3484	97.6
Fatal Accidents	36	4.9	51	1.8	87	2.4
	736	100.0	2835	100.0	3571	100.0
	Nonfatal Accidents		Fatal Accidents		Total Accidents	
	No.	%	No.	%	No.	%
Guardrail Accidents:						
On Turndown Ends	700	20.1	36	41.4	736	20.6
Not on Turndown Ends	2784	79.9	51	58.6	2835	79.4
	3484	100.0	87	100.0	3571	100.0

DISCUSSION OF RESULTS

Unreported Accidents

Historically, unreported accidents have been considered nonevents, particularly to barrier developers. To improve a device the developer examined the failures or reported accidents to understand the mechanisms of the failures and then investigated how the device could be modified to eliminate or at least reduce the number of failures. It was thought that little if any worthwhile information could be gleaned from the successes or unreported accidents. Using only the reported accidents on longitudinal barriers or about 10 to 13 percent of all collisions, researchers have come to false conclusions that may have adversely affected guardrail use.

First, researchers have reported that about half of guardrail accidents result in an injury or fatality and have concluded that these devices are a roadside hazard and should be used only when absolutely necessary. On the other hand, when using the full 100 percent of guardrail collisions, the percentage of impacts involving injuries or fatalities drops to about 6 percent, or a 94 percent success rate. Moreover, researchers have analyzed only reported accident data to estimate typical impact conditions and have statistically developed some surprising typical impact angles and speeds of cars. Using the reported accident data, conclusions have been reached that 45 percent of all barrier impacts involve a nontracking vehicle (*16*). Clearly, these statistics may be important in characterizing impact conditions involving reported barrier impacts, but they are not representative of the complete spectrum of guardrail collisions.

The unreported accident problem may have significance in equally severe embankment warrant curves. In the original 1966 research, Glennon and Tamburri (*9*) compared the severity of a vehicle striking a guardrail with that of permitting the vehicle access to the slope. If the percentage of unreported accidents is the same for the two situations, then the procedure is valid. On the other hand, if the drive-away incidences of guardrail impacts are different from those in which the vehicle accesses the embankment slope, then the curve is in error. It is unlikely that the rate of unreported accidents is the same for both cases.

A second problem deals with the effect of unreported accidents on benefit-to-cost models used to justify guardrail placement. Currently, the typical guardrail impact is characterized as 3.0 on a scale of 10, with typical costs of $10,295 (*1*). This is excessively high when considering that 90 percent of impacts go unreported and certainly result in less than $500 in property damage. Using data from Table 2, the average barrier impact cost is computed to be about $2,500, with a severity index of 1.6.

It is noteworthy that the severity indexes of roadside hazards are typically estimated only from reported accident data. In some cases such as accidents with fixed objects, a high percentage of vehicles are disabled and cannot leave the site, and therefore a large percentage of impacts are reported. Although it is unknown, it is suspected that the reporting rate varies with hazard type, among other factors.

Effects of First and Most Harmful Events

The reporting of roadside accidents based on the first harmful event can be misleading and can misdirect needed barrier performance improvement. As shown in Table 3, the number of fatal accidents in which the guardrail is the most harmful event is less than 50 percent of the accidents in which the guardrail is listed as the first harmful event. This is also true for other barrier types such as concrete safety shapes and bridge rails. Historically, researchers have concentrated on the vehicle-barrier dynamics, assuming that the most harmful event occurred at this point. Using Viner's analysis (*14*), it is becoming clear that many injury-producing events are occurring after the barrier impact, such as rollover of the vehicle. Whereas both *NCHRP Report 230* (*10*) and *NCHRP Report 350* (*11*) have performance objectives of maintaining the vehicle in an upright attitude, the postimpact trajectory criterion of test vehicles has been a secondary assessment factor to date. To further improve the 94 percent performance rate of guardrails, it would seem that more attention is needed in improving the stability and trajectories of vehicles after collision with longitudinal barriers along with ensuring consistent and proper layout procedures.

Length of Need and Terminals

Engineers should be aware that a significant number of reported longitudinal barrier accidents involve upstream terminals and are overrepresented when compared with hazard length exposure and injury severity.

This is a fortunate finding in one respect: safety upgrading funds can be more specifically targeted to substandard barrier terminals with a relatively high benefit-to-cost ratio.

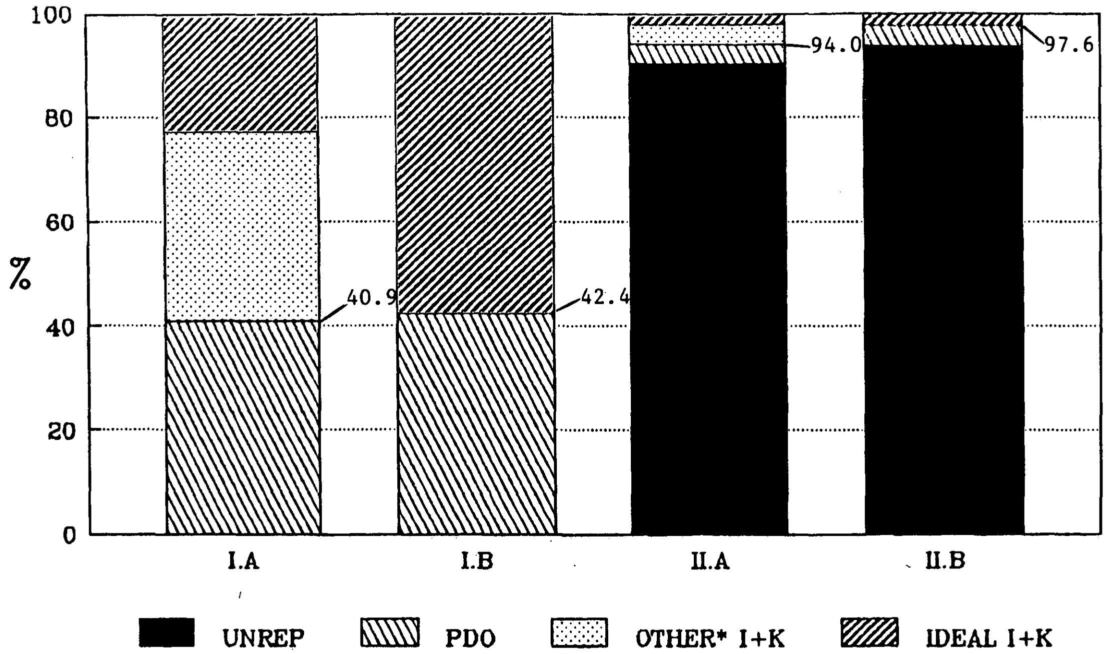
Condition and Design of Barriers

Bryden and Fortuniewicz (*3*) explored the magnitudes of (*a*) obsolete installations, (*b*) improperly laid out or constructed installations, and (*c*) improperly maintained installations as well as collision conditions beyond the device's design capabilities on the outcomes of barrier accidents. As shown in Table 5, roughly one-half (i.e., 56 percent) of 1,950 I + K accidents involved ideal barrier impacts. Hence, only 2.3 percent of all length-of-need barrier impacts in which the barrier is at standard conditions and the collision conditions are within the expected performance envelope results in an accident of I + K severity. Conversely, between 97 and 98 percent of all of these impacts involve at most PDO.

The significance of the unreported accident data is evident in Figure 1. Bar graph I.A indicates that barriers are performing without any injury or fatalities in about 41 percent of reported accidents. When other I + K accidents are screened out, the success ratio increases slightly to 42 percent. Although this difference in PDO accidents is not large, a review of injury severities and fatal accident rates does indicate improved success rates. More important, the success ratios increase to 94 percent for the all-barrier impacts, including estimated unreported accidents, and to 97.6 percent for ideal barrier impacts, including estimated unreported accidents.

CONCLUSIONS AND RECOMMENDATIONS

Longitudinal barriers have been improperly given poor performance ratings based only on reported accident data. Using estimates of unreported accidents, the success rate of longitudinal barriers is at least 94 percent, considering all types of barriers in all kinds of conditions during impacts that are within and outside the normal



I Reported Accidents Only

A. All barrier impacts
B. Ideal barrier impacts

II Reported Plus Estimated Unreported Accidents

A. All barrier impacts
B. Ideal barrier impacts

* Other - includes obsolete barriers, barrier ends, impacts outside barrier performance range, and current barriers not properly maintained

FIGURE 1 Barrier accident data analysis.

performance range. When I + K accidents involving obsolete, improperly constructed, or improperly maintained barriers and atypical impact conditions are eliminated, the success rate is at least 97 percent.

Traditional language in AASHTO barrier guides indicating "that longitudinal barriers are hazardous and should be used only if absolutely necessary" should be softened to reflect a more realistic appraisal of their performance.

Severity indexes for barrier impacts used in benefit-to-cost models may be excessively severe, resulting in understating the benefit of installing a guardrail. These severity indexes should be carefully approached in light of the estimated number of unreported accidents. Severity indexes for barrier ends should distinguish whether the end is one of the newer crashworthy ends meeting the criteria outlined in *NCHRP Report 230* (*10*) or one of the older designs that does not meet these criteria.

Embankment warrant curves may be incorrect if the reporting rate of guardrail impacts is different from that of vehicles going down embankments. These warrant curves should be evaluated using estimates of unreported accidents.

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Frequency and Severity of Crashes Involving Roadside Safety Hardware by Vehicle Type

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FHWA has issued a final rule in response to Section 1073 of the Intermodal Surface Transportation and Efficiency Act of 1991 legislation requiring rule-making on revised guidelines and standards for acceptable roadside barriers and other safety appurtenances to accommodate vans, minivans, pickup trucks, and four-wheel-drive vehicles. This rule lists *NCHRP Report 350* for guidance in determining the acceptability of roadside safety devices. *NCHRP Report 350* recommends use of a ¾-ton pickup as a replacement for the previously used 4,500-lb car in roadside safety device crash tests. The question of whether there are differences in the relative safety experiences in crashes with roadside safety hardware by vehicle body type is addressed by using data from North Carolina, Michigan, and the Fatal Accident Reporting System, General Estimates System, and R. L. Polk vehicle registration files. The data suggest that the practical worst-case test philosophy of current roadside safety device evaluation procedures has provided about the same level of protection to drivers of pickups, light vans, and utility vehicles as to passenger car drivers if the measure of safety is to be the likelihood of serious (fatal plus incapacitating) injuries. If, on the other hand, the measure of safety is to be the likelihood of fatalities, this does not appear to be the case: drivers of pickups were found to be at greater risk. The likely reason for the greater risk of fatalities found for pickup drivers is ejection in rollovers. Programs to increase seat belt use and other measures to reduce ejection rates in rollovers of pickups should be considered to reduce this risk.

Most roadside hardware acceptance test programs have used the minimum crash test matrix of *NCHRP Report 230* since its publication in 1981 (*1*). This minimum crash test matrix consists of tests using passenger cars in the 1,800- to 4,500-lb range. Section 1073 of the Intermodal Surface Transportation and Efficiency Act of 1991 legislation required the Secretary of Transportation to initiate rule-making on revised guidelines and standards for acceptable roadside barriers and other safety appurtenances to accommodate vans, minivans, pickup trucks, and four-wheel-drive vehicles. On July 16, 1993, FHWA published the final rule in response to this requirement in the *Federal Register* (*2*), listing *NCHRP Report 350* (*3*) "for guidance in determining the acceptability of roadside barriers and other safety appurtenances for use on National Highway System (NHS) projects." In particular, the testing requirements of *NCHRP Report 350* "uses a ¾-ton pickup truck as the standard test vehicle in place of the no-longer available 4,500-lb passenger car to reflect the fact that almost one-quarter of the passenger vehicles on U.S. roads are in the 'light truck' category." This paper examines the relative safety experiences in crashes with roadside safety hardware by these vehicle body types.

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North Carolina and Michigan state accident data were used to compare the relative severities of roadside safety hardware crashes involving these vehicle types. National counts of driver fatalities from Fatal Accident Reporting System (FARS) data were used both to define the size of the problem by vehicle type and to identify the vehicle types that appear to be overrepresented in hardware-related fatal crashes when compared with the estimated numbers of nationwide crashes into hardware from the General Estimates System (GES) files and with national numbers of registered vehicles from R. L. Polk vehicle registration files.

METHODOLOGY

Potential differences in driver injury by vehicle body type—passenger cars, pickup trucks, utility vehicles, vans, and other light trucks—were examined in crashes involving roadside safety hardware, which included guardrails, median barriers, bridge rails, impact attenuators, sign supports, and luminaire supports. Both state (North Carolina and Michigan) and national (FARS and GES) crash data were analyzed. Polk registration data were also compared with FARS data. The actual objects examined from these different files varied somewhat because of differing data element definitions. For example, luminaire supports were excluded from the Michigan analyses since they are combined with much larger counts of utility poles, which are not breakaway, in the same data element.

State data were used to compare driver injury severities by vehicle body type by using statistical analyses of two-way and multiway contingency tables. To further investigate the relationships between injury and vehicle type while taking into account interactions with the object struck and highway class (as a surrogate for roadside design), a series of logistic categorical data models was analyzed.

To identify vehicle types that are overrepresented in fatal hardware-related crashes, driver fatalities from the nationwide FARS (fatality) data were compared with both the proportion of body types involved in similar crashes estimated by GES data and the proportion of body types among all nationally registered vehicles obtained from Polk vehicle registration data. Again, contingency tables were used. In addition, counts of driver fatalities (FARS) were used to examine the size of the roadside hardware crash problem by vehicle body type.

There are known limitations to the use of both the state files and the FARS, GES, and Polk files in these analyses. State accident files cannot account for the differences in the percentage of crashes that are unreported that may exist between vehicle types, and thus may distort differences in crash severity comparisons by vehicle type. For example, if a given vehicle type was less likely to sustain major property damage in a low-speed impact or if drivers of a certain vehicle type fail to call police in more crashes involving no or minor injuries, then the severity of crashes in the police-reported accident file would be biased toward the severe end of the injury distribution for that vehicle type. In addition, accident findings in one state may not accurately reflect either the accident experience in another state or the average national experience. Indeed, data from two states were used in this analysis as a check on the consistency of findings.

Vehicle registration data as a surrogate for exposure to crashes into hardware cannot account for differences in driving patterns such as urban and rural usage. Vehicles driven proportionally more in rural locations than in urban locations will have underestimated exposures to rural crashes on the basis of registration numbers and would thus produce inflated rural fatality rates.

GES data can control for driving pattern differences such as rural and urban exposure; however, they are themselves estimates of national numbers of crashes, which are based on samples of police-reported accident files. Thus, numbers derived from GES data should be used in conjunction with their standard deviations to account for sampling error, and GES data are subject to the same underreporting bias as the state data.

FARS data as a census of fatal crashes do not suffer from missing data, national representativeness, or sampling bias problems. However, fatal crashes may have characteristics different from those of nonfatal crashes. For example, the percentage of fatalities from FARS data for certain vehicle–object-struck combinations may differ from the percentage of fatalities for all crash severity levels.

The authors are not aware of any data files that do not suffer from these known defects. However, it is the authors' opinion—even in consideration of these limitations—that these files can provide valuable insight on both the relative risk to occupants in crashes with roadside hardware and the magnitude of the problem by vehicle body type.

THE DATA

State Data

Driver injury observations from North Carolina and Michigan for rural single-vehicle crashes involving the specified vehicle types with selected fixed objects for 6 years (1985 to 1990) were used. Although the North Carolina data were obtained directly from the state, Michigan data were available from FHWA's Highway Information System. Since current crash test procedures call for mostly high-speed (60-mph) testing, the use of only rural data was intended to examine crashes at sites with a greater likelihood of high-speed crashes. Driver injury severity in both cases was taken from the reporting police officer's estimate of injury, using the KABCO injury scale (i.e., with *K* indicating killed, *A*, *B*, and *C* denoting progressively less severe injuries, and *O* representing a property damage-only crash).

The North Carolina data were limited to rural, single-vehicle ran-off-road or fixed-object crashes in which one of the following objects was struck: luminaire support, official highway sign, guardrail face, guardrail end, barrier face, barrier end, or crash cushion. (In cases in which more than one fixed object is struck—e.g., a breakaway sign and then a guardrail end—the North Carolina officer is instructed to code the one causing the most damage, i.e., in this example the guardrail end.) In addition, the cases were restricted to those in which the most harmful event (MHE) was judged by the investigating police officer to be either a fixed object or an overturn. Case vehicles were restricted to 1,800- to 4,500-lb passenger cars (excluding station wagons), pickup trucks, utility vehicles, and vans. The 1,800- to 4,500-lb weight range specified for passenger cars matches the weight range of cars used in *NCHRP Report 230* (*1*) test procedures for roadside safety hardware. The resulting data contained 5,008 records.

The Michigan data were limited to rural, single-vehicle accidents in which the accident type was either an overturn or a fixed-object accident and in which one of the following objects was struck: guardrail or guard post (with ends not separated from faces), highway sign, or median barrier. The vehicle types examined were passenger car, pickup truck, utility (jeep-type) vehicle, and passenger van, as defined by the investigating officer. The resulting data contained 13,554 records.

The constraints to single-vehicle crashes and to those in which the MHE involved either a fixed object or a rollover limit the data, as close as possible, to cases in which the cause of greatest harm is the fixed object of interest. In addition, in many, perhaps most, rollovers that occur subsequent to striking a fixed object, the vehicle tripping mechanism is the impact with the object, and thus the object can be said to be the cause of greatest harm in the crash. However, some intervening impact with a feature such as a ditch could also be the cause of rollover.

Michigan data, however, unlike North Carolina data, do not allow one to identify the most harmful object in crashes in which more than one object is struck. Multiple struck objects are a rather common occurrence. For example, Illinois data show that 41 percent of 411 crashes fatal to the driver or Type A injury crashes in which the first harmful event was a highway sign also involved a second crash event with some other object. Michigan data were not used to examine crash severity by type of object struck in this study since the Michigan data are not as specific as the North Carolina data in terms of linking driver injury to a specific object.

FARS, GES, and Registration Data

The fatal crash data are 1988 driver fatalities from single-vehicle FARS crashes in which the first harmful event (FHE) involved an impact attenuator, bridge rail, guardrail, concrete traffic barrier, sign support, or luminaire support. The cases were restricted to those in which the MHE was identical to the FHE or was a rollover, under the same logic cited earlier. Unlike the state data, which covered the 6-year period between 1985 and 1990, the FARS analysis is for 1988 because later years of Polk data were not available. Unlike fatalities obtained from the state data, all fatalities (rural and urban) were examined to look at the national extent of the problem by body type. FARS comparisons with GES data and Polk registration data were limited to rural crashes for consistency with analyses of the state data.

The GES data are from the 1988 file and are as similar as possible to the FARS data and state data cited earlier. The GES data base is a companion to the FARS data and represents a probability sample of all severities of police-reported traffic crashes in the United States. The data are captured from state crash files by NHTSA-contracted coders and are reviewed and checked for quality control by NHTSA. The FHEs used in the GES (and in the FARS data when they are compared with GES data) were limited to guardrails, median barriers, and impact attenuators. Bridge rails were eliminated because they are combined in the same GES data element as bridge piers and abutments, which are not roadside safety hardware. Similarly, sign and luminaire supports were excluded because they are combined in the same GES data element with (the much more numerous) utility poles.

The 1988 Polk data used were extracted from vehicle registration data files compiled by Polk for every year since 1975. The data are counts of registered vehicles in each state in the United States as of July 1 of each year. Polk data include vehicle body type, manufacturer, model, model year, curb weight, and wheel base.

RESULTS

State Data

North Carolina Data

Initial analysis involved examination of two-way contingency tables to examine the relationship of driver injury to vehicle type, object struck, highway class, and rollover presence and the relationship of each of these variables to each other. The purpose was to define the variables that are important to the issue of differences in injuries due to vehicle type and to define the other control variables that must be included in a more detailed examination. These analyses indicated the following.

First, when all roadside objects are grouped together, the data and associated statistical tests (i.e., χ^2 statistic) suggest that overall, without taking into account any potentially intervening variables (although the pickups, utility vehicles, and vans appear to have slightly more severe serious injury distributions), injury severity does not vary significantly among vehicle types (Table 1).

As expected, the data indicated a strong association between rollover and driver injury when all vehicles are grouped, with rollover resulting in more severe injuries and the likelihood of rollover differing by vehicle type. Utility vehicles are most likely to roll over (43.1 percent of the impacts), passenger cars are least likely to roll over (13.5 percent of impacts), and the likelihood of rollover for the other two vehicle types is about midway between those for utility vehicles and passenger cars (i.e., pickups, 25.7 percent; vans, 24.7 percent).

The results presented to this point may seem puzzling. In particular, pickups, vans, and other light trucks are more likely to roll over, and rollover results in more severe injuries. However, no differences in overall injury severity by vehicle type are found in these collisions. This issue is addressed in Table 2. In Table 2, pickup trucks, utility vehicles, and vans are combined (PUVs) for comparison with passenger cars. (In a second set of analyses the pickup trucks alone were compared with the passenger cars.) The injury severity variable was also dichotomized as no injury, Type *C* injury, or Type *B* injury versus Type *A* injury or *K* (minor to moderate injury versus incapacitating and fatal injuries). The latter combined category will be referred to as serious or $A + K$ injuries in the remainder of the paper.

Table 2 is a three-way breakdown of injury severity by vehicle class for rollover crashes and non-rollover crashes, separately. The 3,481 observations in Table 2 reflect the fact that approximately 28 percent of the rollover variable observations were uncoded. In the data that were coded, PUVs were again found to have significantly higher rollover rates (26.8 percent) than cars (13.2 percent). Note that crashes with cars resulted in slightly (but not significantly) higher serious injury rates in both subtables.

Expressions for probabilities of injuries in cars and PUVs can be written in terms of conditional probabilities of rollovers as

$$P(\text{injury/car}) = P(\text{injury/rollover, car})\, P(\text{rollover/car}) + P(\text{injury/no rollover, car})\, P(\text{no rollover/car})$$

$$P(\text{injury/PUV}) = P(\text{injury/rollover, PUV})\, P(\text{rollover/PUV}) + + P(\text{injury/no rollover, PUV})\, P(\text{no rollover/PUV}).$$

Use of proportions from the generated tables as estimates of the conditional probabilities gives

$$P(\text{injury/car}) = (0.2167)\,(0.1322) + (0.0601)\,(0.8678) = 0.0808$$

$$P(\text{injury/PUV}) = (0.1970)\,(0.2678) + (0.0486)\,(0.7322) = 0.0883$$

TABLE 1 Driver Injury Severity (KABCO) by Vehicle Type (North Carolina data)

Vehicle Type	Injury Severity					
Frequency Row Pct	None	C	B	A	K	Total
Car	2293 62.19	631 17.11	485 13.15	254 6.89	24 0.65	3687
Pickup Truck	562 63.36	145 16.35	107 12.06	62 6.99	11 1.24	887
Utility Vehicle	68 62.39	20 18.35	10 9.17	9 8.26	2 1.83	109
Van	89 63.12	19 13.48	22 15.60	10 7.09	1 0.71	141
Total	3012	815	624	335	38	4824

χ^2(12 d.f.) = 9.2 p = .69

TABLE 2 Driver Injury Level by Rollover and Vehicle Class (North Carolina data)

Rollover Status	Vehicle Class	Injury Severity Level		
		None, C, B	A, K	Total
Rollover	Car	282 78.33	78 21.67	360
	PUV	163 80.30	40 19.70	203
	Total	445	118	563
		$\chi^2_1 = 0.304$	p = .582	
No Rollover	Car	2221 93.99	142 6.01	2363
	PUV	528 95.14	27 4.86	555
	Total	2749	169	2918
		$\chi^2_1 = 1.124$	p = .289	
Total		3194	287	3481

Thus, although PUVs have significantly higher rollover rates and significantly higher injury rates are associated with rollovers, PUVs have slightly lower serious injury rates given rollover and given non-rollover crashes. These two effects tend to cancel each other to yield roughly similar overall injury rates, the result being about a 9 percent higher rate for crashes involving PUVs than for those involving cars (8.8 versus 8.1 percent).

To see if this canceling effect continues to hold when other factors are taken into account, a series of logistic categorical data models were analyzed. Although the details of this analysis are not presented here, models examining injury with and without rollover as a predictor variable indicated the same findings as those obtained in the analyses presented earlier. First, the driver injury proportions vary significantly with object struck, but not with vehicle type. Second, although predicted rollover rates were higher for PUVs than for cars in every subpopulation examined and although rollover is a very powerful predictor of injury, injury rates for the PUVs were not higher than the injury rates for cars.

Michigan Data

As noted earlier the basic data file was created to be as similar as possible to the North Carolina data file, using similar accident years and crash-type restrictions. The resulting data file contained records for 13,554 vehicles involved in crashes.

As was the case with North Carolina data, the overall distributions of driver injury severity do not differ significantly by vehicle type (Table 3). The injury distributions for cars and pickup trucks are virtually identical, as are those for vans and utility vehicles, with the latter groups having less severe injuries.

Rollover rates again differ significantly by vehicle type, with the rank order for Michigan (from low to high) of car (1.88 percent),

TABLE 3 Driver Injury Severity by Vehicle Type in Michigan Fixed-Object Crashes

Vehicle Type	Injury Severity					
Frequency Row Pct	No Injury	Possible Injury	Non-incap. Injury	Incapac. Injury	Fatal	Total
Car	8469 78.92	1088 10.14	757 7.05	371 3.46	46 0.43	10731
Pickup Truck	1867 78.18	248 10.39	185 7.75	76 3.18	12 0.50	2388
Utility Vehicle	202 82.79	21 8.61	14 5.74	5 2.05	2 0.82	244
Van	161 84.29	16 8.38	10 5.24	4 2.09	0 0.00	191
Total	10699	1373	966	456	60	13554

$X^2_{12} = 11.34$ p = .500

pickup truck (3.76 percent), vans (3.80 percent), and utility vehicles (5.46 percent) being the same as that for North Carolina. North Carolina rollover rates, however, were much higher (by a factor of about 6) than the Michigan rollover rates. This difference may be due mostly to varying accident type classifications. Although there is a separate variable for rollover in the North Carolina file (which measures rollover cases in combination with any accident type), rollover is noted as an accident type only in Michigan. This could mean that vehicles that strike fixed objects and that then overturn (the cases of interest) may be classified as fixed-object crashes rather than overturn crashes. Since the Michigan rollover variable is of questionable validity for the purpose of this study, no further analysis of rollovers was done.

From the tables presented, it seems clear that the likelihood of A + K driver injury in roadside appurtenance impacts does not differ appreciably between passenger cars, pickup trucks, vans, and utility vehicles. The relationships between vehicle type and driver injury found in the Michigan data are in very good agreement with those found in the North Carolina data. The major differences in the two data sets are the generally more severe crashes (in terms of driver injury) and the much higher rollover rates for North Carolina (likely due to coding differences).

Pickup Trucks Versus Passenger Cars

Taken together the (*a*) large proportion of pickup trucks in the non-passenger car PUV groups, (*b*) results of FARS analysis presented later in this paper, and (*c*) selection of a pickup truck as the replacement test vehicle for the 4,500-lb car in *NCHRP Report 350* (*3*) indicate a need to compare pickup trucks with cars by using state data. Object struck by vehicle type and by driver injury for North Carolina data is examined in Table 4. Again, driver injury is dichotomized as A + K (serious) injury versus lesser or no injury.

For the North Carolina data there are no statistically significant differences between injury within any of the five object types. A Cochran–Mantel–Haenzel statistic summarizing across all objects likewise shows no statistical significance ($p = .318$), indicating no overall difference in serious injuries between the two vehicle types.

Guardrail face crashes were explored further with regard to rollover since the table for guardrail faces had the largest χ^2 value and the largest frequencies and the pickup $A + K$ injury percentage was higher (8.2 versus 6.1 percent).

When striking a guardrail face, pickup trucks were three times more likely to roll over than cars (24.1 versus 8.0 percent). However, the percentage of serious $A + K$ injuries in rollovers in pickup trucks

TABLE 4 Object Struck by Vehicle Type and Driver Injury (North Carolina data)

Object		Not Serious	Serious	Total		
Luminaire	Car	87 (91.58)	8 (8.42)	95	$X^2 = .448$	p = .503
	P.T.	12 (85.71)	2 (14.29)	14		
Signs	Car	1224 (94.44)	72 (5.56)	1296	$X^2 = .182$	p = .670
	P.T.	288 (93.81)	19 (6.19)	307		
G.R. End	Car	386 (81.26)	89 (18.74)	475	$X^2 = 1.934$	p = .164
	P.T.	81 (87.10)	12 (12.90)	93		
G.R. Face	Car	1524 (93.90)	99 (6.10)	1623	$X^2 = 2.238$	p = .135
	P.T.	394 (91.84)	35 (8.16)	429		
Barrier	Car	175 (94.59)	10 (5.41)	185	$X^2 = 2.026$	p = .155
	P.T.	37 (88.10)	5 (11.90)	42		
Combined (pooled)	Car	3409 (92.46)	278 (7.54)	3687	$X^2 = .473$	p = .683
	P.T.	814 (91.77)	73 (8.23)	887		

Mantel-Haenszel $X^2 = .998$ p = .318 (across objects) is less than half that of passenger cars (13.9 versus 31.2 percent). For nonrollovers the percentage of serious injuries was slightly (but not significantly) higher for pickups than for cars (5.3 versus 4.9 percent). Putting all of this together seems to show that the higher serious injury rates for pickup trucks hitting guardrail faces are primarily due to their much higher rollover rates, even though the chances of serious injury are considerably lower for the driver of a pickup truck that rolls over than for the driver of a car that rolls over. When no rollover occurs, the serious injury rates are very similar.

For Michigan data a Cochran–Mantel–Haenszel statistic summarizing across all objects was not significant ($p = .600$), again indicating no overall difference in serious injury rates between the two vehicle types.

In summary, this analysis of pickup trucks and passenger cars also indicates no significant differences in serious injury to the driver.

National Data

Size of Problem by Body Type

Driver fatalities by FHE (1988 FARS data) are given in Table 5 to examine the national size of the roadside safety hardware problem by body type. Unlike the other analyses in this paper, Table 5 shows urban as well as rural fatalities and fatalities by all vehicle body types, not just cars and light truck types. Twenty percent of these fatalities by FHE involve light trucks: pickups, 15 percent; vans, 3 percent; utility vehicles, 2 percent. Motorcycles account for about as many fatalities as pickups (161 versus 167). Medium and heavy trucks together account for 6 percent of the fatalities (70 fatalities). These 1,101 fatalities were split almost evenly between rural (54 percent) and urban (46 percent) crashes.

For crashes involving roadside safety hardware impacts, guardrail, sign support, and bridge rail FHE impacts are the types resulting in the most fatalities. The category Other in Table 5 provides data on fatalities in crashes involving concrete traffic barriers, luminaire supports, and impact attenuators. In 58 percent of the cases shown in Table 5 the cause of death (MHE) was the indicated object struck (FHE); in the remaining 42 percent the cause of death was overturn. Overturn was the cause of death in 63 percent of crashes involving sign supports, 42 percent involving guardrails, and 31 percent involving bridge rails. Little change in speed occurs in impacts with breakaway supports. Thus, for many, and perhaps most, overturns involving breakaway sign supports, the cause of vehicle tripping and thus the ultimate cause of death could well be subsequent vehicle involvement with other roadside features such as slopes and ditches. Accordingly, the size of the sign support problem is most likely overstated in Table 5.

Crash Severity by Body Type

In these analyses (*a*) national counts of rural driver fatalities in crashes with roadside safety devices in cars and light trucks (FARS) were compared with vehicle registration data (Polk) as one measure of exposure, and (*b*) national counts of fatalities in the guardrail, median barrier, and impact attenuator impact subset (FARS) were compared with national estimates of all such crashes (GES) as a second measure of exposure (Table 6). Only rural cases are shown to more closely parallel the original state-based analysis. The proportion of pickup driver fatalities in the roadside safety hardware impact group (25 percent) is substantially greater than the proportion of registered pickups (15.6 percent). In addition, for the guardrail, median barrier, and impact attenuator impact subgroup, the proportion of pickup driver fatalities (24 percent) is much higher than the estimated proportion of pickups involved in all such crashes (an estimated 9 percent). The 95 percent confidence limits of the GES estimate of crash involvement is 4.4 to 13.3 percent (8.7 ± 4.3 percent). Thus, the percentage of fatalities of pickup truck drivers in roadside safety hardware crashes is significantly in excess of what would be expected from an examination of all such crashes.

TABLE 5 Driver Fatalities (Rural plus Urban) by Body Type and FHE (1988 FARS data)

	First Harmful Event									
Body Type	Guardrail		Sign Support		Bridge Rail		Other		Totals	
	Number	%	Number	%	Number	%	Number	%	Number	%
Automobile	357	54%	92	50%	52	55%	123	73%	624	57%
Pickup	98	15%	34	19%	20	21%	15	9%	167	15%
Motorcycle	99	15%	36	20%	7	7%	19	11%	161	15%
Medium/Heavy Truck	47	7%	9	5%	10	11%	4	2%	70	6%
Van	20	3%	4	2%	0	0%	4	2%	28	3%
Other	20	3%	6	3%	2	2%	0	0%	28	3%
Truck Based Utility	15	2%	2	1%	3	3%	3	2%	23	2%
Totals	656	100%	183	100%	94	100%	168	100%	1,101	100 %

TABLE 6 Rural Roadside Safety Hardware Driver Fatalities (FARS), Involved Vehicles (GES), and Total Registered Vehicles (Polk)

Body Type	Driver Fatalities (FARS)				Involved Vehicles		Registered Vehicles (Polk)
	All		Guardrail, median barrier, impact atten.		Guardrail, Median Barrier, Impact Atten. (GES)		
	Number	%	Number	%	Number	%	%
Auto	315	69%	223	70%	46,600	88%	77.4%
Pickup	116	25%	75	24%	4,600	9%	15.6%
Van	15	3%	12	4%	1,500	3%	4.7%
Utility	13	3%	9	3%	500	1%	2.3%
Total	459		319		53,200		

DISCUSSION OF RESULTS

The analyses presented earlier appear to indicate contrasting findings between national data (FARS, GES, and Polk data) and state-based data in crashes with roadside safety hardware. The significant overrepresentation of drivers of pickups in the rural FARS data—only 9 percent of crashes and 15.6 percent of registrations but 25 percent of fatalities (Table 6)—was not found from the analyses conducted with data from either of the state files. The overall distributions of driver injury severity across objects did not differ significantly by vehicle type in the state data (Tables 1 and 3). Overall differences in the percentage of serious driver injuries found between drivers of pickup trucks and drivers of passenger cars were nearly identical in both states (Table 4 and earlier text).

Because of this contrast in findings, the proportions of fatalities alone for cars and PUVs in the North Carolina and Michigan data were compared. Because of small sample sizes of driver fatalities, no meaningful rollover–nonrollover analyses could be made with the North Carolina data [i.e., 9 fatalities of car drivers in 360 rollovers (2.5 percent) versus 7 fatalities of PUV drivers in 203 rollovers (3.45 percent)]. When all objects are combined without respect to rollover for the North Carolina data, 1.2 percent of the PUV crashes resulted in fatalities (14 fatalities), whereas 0.65 percent of the car crashes resulted in fatalities (24 fatalities), a difference nearing statistical significance ($p = .066$). Examination of the combined data for Michigan indicated no significant difference in fatality rates [0.51 percent for PUVs (14 fatalities) versus 0.44 percent for cars (45 fatalities); $p = .626$]. Thus, there is a suggestion of increased fatalities among drivers of PUVs involved in accidents for North Carolina but not for Michigan, at least in an overall sense.

A likely reason that pickup drivers were found to be overrepresented in FARS fatality data but not in the analyses of serious injuries conducted with state data is ejection of unbelted drivers in rollovers. Counts of driver fatalities by rollover and ejection outcome for the 315 car and 116 pickup cases in Table 6 are given in Table 7. The numbers of rollovers that occurred before contact with the roadside safety device (first event) and after contact (subsequent event) are given in Table 7, as are the numbers of total and partial ejections. Rollovers occurred in 82 percent of the fatal pickup and 62 percent of the fatal automobile crashes given in Table 7. In contrast, in the North Carolina data covering the full injury distribution, rollovers occurred in only 26 percent of the pickups and 13 percent of the passenger car crashes. In the FARS data, both rollover and ejection (total and partial) occurred in 62 percent of all pickup fatalities (72 of 116 cases) and 41 percent of all car fatalities (196 of 315 cases). Thus, the ejection–rollover combination is seen to be associated with a large percentage of these fatalities. Terhune (*4*) examined rollover cases in National Accident Sampling System data and concluded that ejection accounted for about half of all $A + K$ injuries in car rollovers, and on the basis of limited data, ejection appeared to be the predominant factor in light truck A + K injuries. Kahane's (*5*) review of the literature concludes that ejection increases the risk of fatality of passenger car occupants by 380 percent. Although no hard data are available, it is also possible that ejection would increase the probability of fatality more than the probability of incapacitating injury, since a crash sequence violent enough to result in ejection would present a high risk of incapacitating injury even if the occupant remained in the car. Thus, the number of incapacitating (i.e., Type *A*) injuries might be expected to increase less than the number of fatal injuries when ejection occurs.

Ejections are greatly reduced by seat belt use. Seat belt observations in North Carolina indicate belt-wearing rates for drivers of pickup trucks and utility vehicles approximately 20 percent lower than the usage rate for drivers of passenger cars (*6*). Thus, the greater ejection risk for pickup drivers than for car drivers because of lower seat belt use rates and higher rollover rates, coupled with the likely differential increase in fatalities over serious injuries in ejections, could result in greater pickup overrepresentation in the FARS data than in the state data (serious injuries plus fatalities). This is particularly true since the state data have a relatively low percentage of fatalities in the severe injury groupings examined. Examination of fatality data for North Carolina also suggests such a possible overrepresentation for PUVs.

The analysis of national data (and to a limited extent the North Carolina data) presented here supports the decision made in

TABLE 7 Rural Fatally Injured Drivers (FARS) by Rollover and Ejection for Pickups and Cars

PICKUPS

Rollover	Ejection: No	Total	Partial	Unk	Totals	
No	14	5	2	0	21	18%
First Event	3	1	1	0	5	4%
Subs. Event	20	60	10	0	90	78%
Totals	37	66	13	0	116	100
Percent	32%	57%	11%	0%	100%	

CARS

Rollover	Ejection: No	Total	Partial	Unk	Totals	
No	91	23	4	1	119	38%
First Event	1	5	1	0	7	2%
Subs. Event	64	107	17	1	189	60%
Totals	156	135	22	2	315	100%
Percent	50%	43%	7%	1%	100%	

NCHRP Report 350 (*3*) to use a pickup truck as the standard test vehicle in crash testing. FARS data demonstrate the importance of pickups in crashes with roadside safety devices. Pickups dominate the light truck driver fatality totals: 15 percent of all body types compared with 3 percent for vans and 2 percent for utility vehicles. Pickups accounted for 25 percent of the rural car–light truck group fatalities, even though they were involved only in an estimated 10 percent of such rural crashes. No attempt was made in this paper, however, to address the question of the representativeness of a pickup truck as a substitute for the previously used 4,500-lb car in crash testing as recommended in *NCHRP Report 350* (*3*).

In summary, with respect to the major question of interest, neither set of state data indicates differences in serious driver injury severity (A + K percentages) by vehicle type either when the special vehicles are grouped or when the pickup trucks were analyzed separately. On the other hand, the FARS analysis indicates an overrepresentation of pickup truck fatalities when compared with both GES-based national estimates of hardware-related crashes and Polk registration data. Known differences in seat belt use rates and risks of fatality by ejection in rollovers suggest that these findings may not be in conflict.

The higher rollover risk found for pickups compared with that found for cars in roadside hardware crashes is consistent with findings for ran-off-road crashes in other studies (*7,8*). Thus, differences in rollover outcome in crash testing may be experienced when using a ³⁄₄-ton pickup as a substitute for the previously used 4,500-lb car as recommended in *NCHRP Report 350* (*3*).

In short, the data suggest that the practical worst-case test philosophy of current roadside safety device evaluation procedures has provided about the same level of protection to drivers of pickups, light vans, and utility vehicles as to drivers of passenger cars if the measure of safety is to be the likelihood of serious (A + K) injuries.

If, on the other hand, the measure of safety is to be the likelihood of fatalities, then drivers of pickups are at greater risk. Thus, although the use of pickups in crash tests appears to be warranted, it may be the case that redesigning roadside safety hardware to reduce the rollover risk of pickups is not the most cost-effective solution to this problem. Programs to increase pickup stability, to increase seat belt use, and other measures to reduce ejection rates in rollovers of pickups also need to be considered in this regard. Such programs would affect not only injuries in crashes related to safety devices but also the larger number of injuries and fatalities seen in other pickup crashes.

ACKNOWLEDGMENTS

The North Carolina and Michigan analyses were funded in part by FHWA Contract DTFH61-92-C-00086 concerning the development and use of the Highway Safety Information System and in part by the University of North Carolina Injury Prevention Research Center under grant R49/CCR402444 from the Centers for Disease Control and Prevention and FHWA aimed at developing severity indexes. The FARS and Polk data computer runs made for this analysis were done by Carol Conley of AEPCO.

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The findings and opinions expressed in this paper are those of the authors and do not necessarily reflect the views of the sponsor.

Analysis of Bridge Railing Accidents

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As part of a study to evaluate and validate the performance-level selection criteria contained in the 1989 AASHTO *Guide Specifications for Bridge Railings*, it was found that the Benefit Cost Analysis Program (BCAP), which was used to develop the performance-level selection criteria, was dominated by the frequency and severity of accidents involving trucks penetrating or rolling over the bridge railings. Contrary to results of previous accident studies, BCAP predicted a very high incidence of penetration or rolling over the bridge railings. In an effort to better estimate the extent of bridge railing accidents in which impacting vehicles penetrated or went over the bridge railings, Texas accident data for the years 1988 through 1990 were analyzed. Also, the dates of construction or latest reconstruction were determined for a sample of the bridge accidents so that the performances of bridge railings designed to the current specifications and older bridge railings might be differentiated. Finally, hard copies of accident reports for those bridge railing accidents involving trucks penetrating or going over the bridge railings were manually reviewed. Results of the analysis indicate that passenger cars and light trucks accounted for more than three-quarters of the accidents in which vehicles went through or over the bridge railings. Also, the incidence of going through or over the bridge railing happened mostly on rural highways. For bridge railings constructed to current specifications, the proportion of accidents involving single-unit and combination trucks going through or over bridge railings was found to be 4.4 percent, which is in line with that found in previous studies. There is a significant difference in performance between bridge railings constructed after 1965 that met current design specifications and those constructed before 1965. A review of hard copies of the accident reports of the accidents involving heavy trucks going through or over bridge railings indicated that the magnitude of the problem with trucks going through or over bridge railings is much smaller than that indicated by the accident data. Only 6 of the 53 accidents actually involved heavy trucks going through or over bridge railings, and only 1 of the 6 accidents involved a bridge railing constructed after 1965. The remaining accidents were miscodes on object struck, vehicle type, or bridge railing performance.

In 1989 AASHTO adopted the *Guide Specifications for Bridge Railings* (hereinafter referred to as the Guide Specifications) (*1*). Two of the key new features of the Guide Specifications were the incorporation of the multiple performance-level concept and the requirement that future bridge railing designs be crash tested to confirm impact performance. Although the concept of multiple performance levels is very appealing and worthy of implementation, it is important to make sure that the procedures and selection criteria promulgated in the Guide Specifications are appropriate and valid. A study was sponsored by NCHRP and conducted by the Texas Transportation Institute to evaluate the performance-level selection criteria for bridge railings (*2*).

In the course of evaluating the multiple performance levels and the selection procedures contained in the Guide Specifications, it was found that the performance-level selection criteria were dominated by the frequency and severity of accidents involving trucks that either went through or rolled over the bridge railings. The Benefit Cost Analysis Program (BCAP) used to generate the performance-level selection criteria predicted a very high incidence of bridge railing penetration by trucks. For example, for a Performance Level 2 bridge railing, such as the widely used concrete safety-shaped bridge railing, BCAP predicted that 28.6 percent of the truck impacts would result in bridge railing penetrations but no trucks rolling over a bridge railing (*2*). Based on a previous study by the California Department of Transportation and preliminary investigations, it was anticipated that the rate at which trucks go through or over bridge railings would be on the order of 3 to 4 percent. There was clearly a large discrepancy between what BCAP predicts and observations made from real-world accident data.

The objectives of this analysis of bridge railing accidents were therefore to obtain better estimates of the extent of accidents involving penetration or rolling over bridge railings and to validate BCAP and the performance-level selection guidelines contained in the Guide Specifications. A summary of the accident analysis and the results are presented in this paper.

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STUDY APPROACH

Accident data from the state of Texas for the 3 years from 1988 to 1990 were used in the analysis. Accidents involving bridge railings were identified from the accident data file by keying on the variable "object struck," which has "side of bridge" as one of the codes for object struck. The performance of the bridge railing was identified from another variable, "bridge detail," which indicates if the vehicle was retained on the bridge, went through the bridge railing, or went over the bridge railing. The following screening criteria were used to select bridge railing accidents for study:

1. Only accidents on state-maintained highways were included (i.e., no city streets or county roads). The variable bridge detail was not coded for city streets or county roads, and therefore, bridge railing accidents on these roadways had to be eliminated.
2. Only single-vehicle accidents were included. When more than one vehicle was involved in an accident, it is not possible to determine which vehicle struck the bridge railing without reviewing the hard copy of the accident report. This criterion excluded all multivehicle accidents to eliminate any question as to which vehicle struck the bridge railing.
3. Only accidents with object struck coded as side of bridge were included.
4. Only accidents with the variable bridge detail coded as vehicle retained on bridge, vehicle went through bridge rail, or vehicle went over bridge rail were included.

These screening criteria reduced the data set to include only single-vehicle accidents occurring on state-maintained highways with the object struck coded as side of bridge and bridge detail coded as vehicle retained on bridge, vehicle went through bridge rail, or vehicle went over bridge rail. For the years 1988 to 1990, a total of 4,552 accidents were identified as meeting the screening criteria and were included in the analysis. The breakdown of the sample accidents by year is as follows: 1,217 in 1988, 1,754 in 1989, and 1,581 in 1990. Of the 4,552 vehicles that struck bridge railings, 4,323 (95.0 percent) were retained on the bridge, 70 (1.5 percent) went through the bridge railing, and 159 (3.5 percent) went over the bridge railing, as shown in Table 1.

The design specifications for bridge railings were substantially revised in 1964. Current bridge railings are required to meet specific geometric criteria and must be capable of resisting applied static loads without exceeding allowable stresses in any of their component members. Bridge railings constructed before 1964 were not designed to specific geometric criteria or loading capacities and are less likely to contain and redirect the impacting vehicles. To differentiate the performances of bridge railings constructed to current specifications from those of the older bridge railings, the date of construction or the latest date of reconstruction for all 229 (70 + 159) bridges with vehicles going through or over the bridge railings and a 10 percent sample (432 bridges) of the bridges with vehicles retained by the bridge railings were manually determined by matching the accidents to the individual bridges by using the bridge inventory file. Of the 661 (229 + 432) bridges checked, only 541 (81.9 percent) were successfully matched to bridges in the bridge inventory file, including 171 bridges with vehicles going through or over the bridge railings and 370 bridges with vehicles retained on the bridges. The discrepancy can be attributed to errors in the coding of the object struck (i.e., bridge railing was incorrectly coded) and in the reported locations of the accidents. This accident sample was analyzed separately in an effort to determine the difference in impact performances of bridge railings constructed to current specifications and those of the older bridge railings.

Finally, hard copies of accident reports for all 53 accidents involving single-unit trucks (24 accidents) or combination trucks (29 accidents) going through or over bridge railings were acquired from the Texas Department of Public Safety. These accident reports were manually reviewed to obtain some insights into these accidents.

ANALYSIS RESULTS

Slightly less than half (2,148 of 4,552, or 47.2 percent) of the bridge railing accidents occurred on rural highways (including towns with populations of less than 5,000), as shown in Table 2. The most significant result is that most of the accidents involving vehicles going through or over bridge railings occurred on rural highways. Of the 229 accidents involving vehicles going through (70 accidents) or over (159 accidents) bridge railings, 188 (82.1 percent) occurred on rural highways and only 41 (17.9 percent) occurred on urban highways. In comparison, slightly more than half (2,404 of 4,552, or 52.8 percent) of the bridge railing accidents occurred on urban highways. The highest proportion of accidents in which vehicles (15.1 percent) went through or over bridge railings occurred on rural farm-to-market-type roadways, which are typically two-lane, two-way highways with low traffic volumes. The bridge railings on these farm-to-market highways are likely to be constructed before 1965 and are not up to current design specifications. For rural Interstate and U.S. and state highways, the proportions of vehicles going through or over bridge railings are 7.4 and 7.3 percent, respectively. In comparison, only 1.7 percent of the bridge railing accidents on urban highways resulted in vehicles going through or over the bridge railings.

Approximately half (48.1 percent) of the bridge railing accidents occurred during the hours of darkness, and the proportion remained similar regardless of the bridge railing performance, as shown in Table 3. It is interesting to note that for accidents involving vehicles going through or over bridge railings that occurred during the hours of darkness, the overwhelming majority (97 of 115 accidents, or 84.4 percent) occurred in unlighted areas. In comparison, only 60

TABLE 1 Distribution of Accidents by Bridge Railing Performance

Bridge Railing Performance	Number	%
Vehicle Retained on Bridge	4,323	95.0
Vehicle Went Through Bridge Railing	70	1.5
Vehicle Went Over Bridge Railing	159	3.5
Total	4,552	100.0

TABLE 2 Bridge Railing Performance by Highway Type

Highway Type	Vehicle Retained on Bridge No.	%	Vehicle Went Through or Over Bridge Railing No.	%	Total No.	%
RURAL						
Rural Interstate	587	92.6	47	7.4	634	100.0
Rural US & State	1,036	92.7	81	7.3	1,117	100.0
Rural Farm to Market	337	84.9	60	15.1	397	100.0
Rural Subtotal	1,960	91.3	188	8.7	2,148	100.0
URBAN						
Urban Interstate	1,303	98.1	25	1.9	1,328	100.0
Urban US & State	972	98.5	15	1.5	987	100.0
Urban Farm to Market	88	98.9	1	1.1	89	100.0
Urban Subtotal	2,363	98.3	41	1.7	2,404	100.0
Total	4,323	95.0	229	5.0	4,552	100.0

TABLE 3 Bridge Railing Performance by Light Condition

Light Condition	Vehicle Retained on Bridge No.	%	Vehicle Went Through or Over Bridge Railing No.	%	Total No.	%
Daylight	2,075	48.0	105	45.8	2,180	47.9
Dawn	124	2.9	6	2.6	130	2.9
Dark, Not Lighted	1,236	28.6	97	42.4	1,333	29.3
Dark, Lighted	836	19.3	18	7.9	854	18.8
Dusk	52	1.2	3	1.3	55	1.2
Total	4,323	100.0	229	100.0	4,552	100.0

percent (1,236 of 2,072) of those accidents in which the vehicles were contained by the bridge railings occurred in unlighted areas. This again reflects the fact that most of the accidents involving vehicles going through or over bridge railings occurred on rural highways.

Bridge railing performance by surface conditions is shown in Table 4. Slightly more than half (53.3 percent) of the bridge railing accidents occurred on dry pavements. Another 27.9 percent of the bridge railing accidents occurred under snowy conditions, which is not surprising because bridges tend to freeze more readily. The percentage of accidents involving vehicles going through or over bridge railings was lower under wet or snowy pavement surface conditions, probably the result of lower traffic speeds during adverse weather and under adverse surface conditions.

Bridge railing performance by vehicle type is shown in Table 5. Vehicle types are categorized as passenger car, pickup truck, single-unit truck, combination truck, and other. The pickup truck category includes all light trucks (i.e., pickup trucks, vans, and utility vehicles). Single-unit trucks are medium-size trucks in which the beds or cargo-carrying areas are rigidly attached to the frames of the trucks. Combination trucks are commonly referred to as tractor-trailers. The other vehicle type is mostly motorcycles. Single-unit trucks and combination trucks (i.e., tractor-trailers) accounted for 516 (11.3 percent) and 184 (4.0 percent) of the 4,552 bridge railing accidents, respectively. As may be expected, combination trucks had the highest proportion (15.8 percent) of accidents in which the vehicle went through or over the bridge railing. However, it is somewhat surprising that the proportion of vehicles going through or over the bridge railing for single-unit trucks (4.6 percent) was actually lower than that for pickup trucks (6.7 percent), although it was higher than that for passenger cars (3.6 percent). A possible explanation is that single-unit trucks are operated under totally different conditions than the other vehicle types. For example, single-unit trucks are mostly used for local transport of goods during business hours, which would reduce their exposure to single-vehicle-type accidents and would keep operating speeds relatively low, thereby reducing the potential for penetrating a bridge railing.

As already mentioned, bridge railings constructed up through 1964 were not designed to current specifications and are less likely to contain and redirect impacting vehicles. Note that even though the bridge railing specifications changed in 1964, most bridge railings completed in 1965 would have been designed under the old specifications. Therefore, bridge railings completed in 1965 were considered to have been designed under the old specifications, whereas railings completed in 1966 and later were considered to be designed to meet the modern criteria.

To differentiate the performances of bridge railings designed to current standards from those of the older bridge railings, the sample of 541 accidents in which the date of construction or the latest date of reconstruction for the bridges was determined was analyzed further. Bridge railing performance by vehicle type and the data are broken down by date of construction or latest reconstruction is shown in Table 6. It is evident that bridges constructed after 1965 had a lower incidence of vehicles going through or over bridge railings than bridges constructed in 1965 or earlier. For all vehicle types, the proportion of vehicles going through or over bridge railings dropped from 5.9 percent for bridge railings constructed in 1965 or earlier to 3.0 percent (49.1 percent reduction) for bridge railings constructed after 1965. The differences are even more pronounced for combination trucks. The proportion of combination trucks going through or over bridge railings for railings constructed in 1965 or earlier is 24.5 percent. The corresponding proportion for bridge railings constructed after 1965 dropped to 7.7 percent (68.6 percent reduction).

Shown in Table 7 is a breakdown of highway type by date of bridge construction or latest reconstruction. It is interesting to note that rural highways have a higher proportion of bridge railings con-

TABLE 4 Bridge Railing Performance by Surface Condition

Surface Condition	Vehicle Retained on Bridge No.	%	Vehicle Went Through or Over Bridge Railing No.	%	Total No.	%
Dry	2,270	52.5	155	67.7	2,425	53.3
Wet	826	19.1	33	14.4	859	18.9
Snowy	1,227	28.4	41	17.9	1,268	27.9
Total	4,323	100.0	229	100.0	4,552	100.0

TABLE 5 Bridge Railing Performance by Vehicle Type

Vehicle Type	Vehicle Retained on Bridge No.	%	Vehicle Went Through or Over Bridge Railing No.	%	Total No.	%
Passenger Car	2,518	96.4	94	3.6	2,612	100.0
Pickup Truck	1,136	93.3	81	6.7	1,217	100.0
Single Unit Truck	492	95.4	24	4.6	516	100.0
Combination Truck	155	84.2	29	15.8	184	100.0
Other	22	95.7	1	4.3	23	100.0
Total	4,323	95.0	229	5.0	4,552	100.0

TABLE 6 Bridge Railing Performance by Vehicle Type and Year of Construction

Vehicle Type	Vehicle Retained on Bridge No.	%	Vehicle Went Through or Over Bridge Railing No.	%	Total No.	%
1965 AND EARLIER						
Passenger Car	1,070	95.6	49	4.4	1,119	100.0
Pickup Truck	470	92.3	39	7.7	509	100.0
Single Unit Truck	210	94.6	12	5.4	222	100.0
Combination Truck	40	75.5	13	24.5	53	100.0
Total	1,790	94.1	113	5.9	1,903	100.0
AFTER 1965						
Passenger Car	1,230	97.9	27	2.1	1,257	100.0
Pickup Truck	350	95.4	17	4.6	367	100.0
Single Unit Truck	210	97.7	5	2.3	215	100.0
Combination Truck	120	92.3	10	7.7	130	100.0
Total	1,910	97.0	59	3.0	1,969	100.0

For accidents in which vehicles were retained on the bridges, only a 10 percent sample was checked for year of construction or latest reconstruction.

TABLE 7 Highway Type by Year of Construction

Highway Type	Year of Construction: 1965 and Earlier No.	%	After 1965 No.	%	Total No.	%
			RURAL			
Rural Interstate	240	42.9	319	57.1	559	100.0
Rural US & State	447	54.1	379	45.9	826	100.0
Rural Farm to Market	320	87.9	44	12.1	364	100.0
Rural Subtotal	1,007	57.6	742	42.4	1,749	100.0
			URBAN			
Urban Interstate	460	38.6	731	61.4	1,191	100.0
Urban US & State	365	42.4	496	57.6	861	100.0
Urban Farm to Market	71	100.0	0	0.0	71	100.0
Urban Subtotal	896	42.2	1,227	57.8	2,123	100.0
Total	1,903	49.2	1,969	50.8	3,872	100.0

For accidents in which vehicles were retained on the bridges, only a 10 percent sample was checked for year of construction or latest reconstruction.

TABLE 8 Accident Severity by Vehicle Type and Bridge Railing Performance

Vehicle Type	No Injury No.	%	Minor to Moderate Injury No.	%	Severe to Fatal Injury No.	%	Total No.	%
			VEHICLE RETAINED ON BRIDGE					
Passenger Car	1,390	55.2	931	37.0	197	7.8	2,518	100.0
Pickup Truck	659	58.0	375	33.0	102	9.0	1,136	100.0
Single Unit Truck	281	57.1	169	34.4	42	8.5	492	100.0
Combination Truck	90	58.1	53	34.2	12	7.7	155	100.0
Other	4	18.2	6	27.3	12	54.6	22	100.0
Total	2,424	56.1	1,534	35.5	365	8.4	4,323	100.0
			VEHICLE WENT THROUGH OR OVER BRIDGE RAILING					
Passenger Car	26	27.7	37	39.3	31	33.0	94	100.0
Pickup Truck	24	29.6	35	43.2	22	27.2	81	100.0
Single Unit Truck	8	33.3	3	12.5	13	54.2	24	100.0
Combination Truck	8	27.6	9	31.0	12	41.4	29	100.0
Other	0	0.0	1	100.0	0	0.0	1	100.0
Total	66	28.8	85	37.1	78	34.1	229	100.0

structed in 1965 or earlier (57.6 percent), whereas the opposite is true for the urban highways (42.2 percent). This finding is a further explanation for the dramatic differences between the rates that vehicles go through or over bridge railings in urban and rural areas. For bridge railings on farm-to-market-type highways, only 44 of 435 (360 + 71), or 10.1 percent, were constructed after 1965. This confirms the earlier contention that bridge railings on farm-to-market-type highways are likely to be constructed before 1965 and are not up to current design specifications, thereby explaining the high percentage of vehicles going through or over bridge railings on these roads.

The severity of accidents by vehicle type by bridge railing performance is shown in Table 8. As may be expected, the severity of accidents involving vehicles going through or over bridge railings was very high. The proportion of severe to fatal (percent $A + K$) injury accidents increased from 8.4 percent for vehicles retained on bridges to 34.1 percent for vehicles that went through or over the bridge railings. The severities of the accidents in which the vehicles were contained were similar for all vehicle types except for the "other" vehicle type, which was mostly motorcycles. For vehicles that went through or over the bridge railings, the proportion of $A + K$ injury accidents was highest for single-unit trucks (54.2 percent), followed closely by combination trucks (41.4 percent), and both of these proportions were considerably higher than those for passenger cars and pickup trucks (33.0 percent and 27.2 percent, respectively).

Finally, manual review of hard copies of the 53 accident reports involving single-unit or combination trucks going through or over bridge railings revealed significant problems with the police-reported accident data. As shown in Table 9, only 17 of the 53 (32.1 percent) accidents actually involved bridge railings. The other 36 accidents were miscoded and involved approach guardrails or ends of guardrails and bridge railings. As for vehicle type, all 29 accidents involving combination trucks were coded correctly. However, for the 24 accidents involving single-unit trucks, only 6 (25 percent) were coded correctly. Of the remaining 18 accidents miscoded as involving single-unit trucks, 16 involved pickup trucks or utility vehicles and 2 involved combination trucks. Also, of the 17 accidents involving bridge railings, only 10 (58.8 percent) involved vehicles actually going through or over the bridge railings. In the other seven accidents the vehicles were actually retained on the bridges. Of the six accidents involving combination trucks going through or over bridge railings, only one involved a bridge railing constructed after 1965. These findings clearly indicate that the magnitude of the problem with trucks going through or over bridge railings is much smaller than that indicated by the accident data.

SUMMARY OF FINDINGS

A summary of the findings from this accident analysis is presented as follows.

- Passenger cars and light trucks (i.e., pickup trucks, vans, and utility vehicles) accounted for 175 of the 229 accidents (76.4 percent) in which vehicles went through or over the bridge railings.
- The accident data indicated a very low incidence of going through or over the bridge railing on urban highways. Whereas more than half (52.8 percent) of the bridge railing accidents occurred on urban highways, only 41 of 229 (17.9 percent) accidents resulting in the vehicle going through or over the bridge railing occurred on urban highways.
- The accident data from this study indicate a higher incidence of trucks (4.6 percent of single-unit trucks and 15.8 percent of combination trucks) going through or over bridge railings than was previously believed, which is on the order of 3 to 4 percent. However, when only bridge railings constructed after 1965 are considered, the proportion dropped to 2.3 percent for single-unit trucks and 7.7 percent for combination trucks, for a combined percentage of 4.4 per-

TABLE 9 Breakdown of 53 Accidents Coded as Single-Unit or Combination Trucks Going Through or Over Bridge Railings

OBJECT STRUCK

Object Struck	Number	%
Bridge Railing	17	32.1
Bridge Railing End	3	5.7
Guardrail	26	49.1
Guardrail End	5	9.4
Other	2	3.8
Total	53	100.0

VEHICLE TYPE

	Actual Vehicle Type			
Coded Vehicle Type	Combination Truck	Single Unit Truck	Pickup Truck	Total
Combination Truck	29	0	0	29
Single Unit Truck	2	6	16	24
Total	31	6	16	53

BRIDGE RAILING PERFORMANCE

	Combination Truck		Single Unit Truck		Pickup Truck		Total	
Bridge Railing Performance	No.	%	No.	%	No.	%	No.	%
Vehicle Retained on Bridge	4	40.0	1	100.0	2	33.3	7	41.2
Vehicle Went Through or Over Bridge Railing	6	60.0	0	0.0	4	66.7	10	58.8
Total	10	100.0	1	100.0	6	100.0	17	100.0

cent. These percentages are more in line with those found in previous studies.

- There is a significant difference in performance between bridge railings constructed after 1965 that met current design specifications and those constructed before 1965.
- A review of hard copies of the accident reports of the 53 accidents involving heavy trucks going through or over bridge railings indicated that the magnitude of the problem with trucks going through or over bridge railings is much smaller than that indicated by the accident data. Only 6 of the 53 accidents actually involved heavy trucks going through or over bridge railings, and only 1 of the 6 accidents involved a bridge railing constructed after 1965. The remaining accidents were miscodes on object struck, vehicle type, or bridge railing performance.

DISCUSSION OF RESULTS

Results from this accident analysis bring out a number of interesting points, which are discussed as follows.

- This analysis of bridge railing accident data confirmed previous findings that the incidence of heavy trucks going through or over bridge railings constituted at most 3 or 4 percent of reported bridge railing accidents. It should be borne in mind that this analysis was biased toward the more severe impacts, thus representing the upper-bound values. The screening criteria would favor the more severe impacts by including only single-vehicle accidents on state-maintained highways. Furthermore, there is the problem with accidents that, for whatever reason, were not reported to police agencies. Even if the ratio of reported to unreported accidents is assumed conservatively to be 1 to 1, the proportion of trucks going through or over bridge railings for bridge railings constructed after 1965 would drop to only 2.2 percent, or half of 4.4 percent.
- The results of this analysis indicated that BCAP, which was used to develop the performance-level selection guidelines contained in the 1989 AASHTO *Guide Specifications for Bridge Railings*, was in error in overpredicting the incidence of trucks penetrating bridge railings while not predicting any occurrence of trucks rolling over bridge railings. Further review of BCAP identified serious problems with its penetration and rollover algorithms. First, the structural capacities of the bridge railings used to develop the performance-level selection guidelines were severely understated, which led to the high predicted penetrations rates. Second, the rollover algorithm in BCAP was found to have grossly overpredicted the speed at which a truck would roll over a bridge railing, thus resulting in the program not predicting any occurrence of rolling over a bridge railing. Third, BCAP totally ignores the possibility that passenger cars and light trucks (i.e., pickup trucks, vans, and utility vehicles) will roll over bridge railings. Accidents involving these vehicles were found to comprise more than 75 percent of the accidents in which the impacting vehicles went through or over bridge railings. The penetration and rollover algorithms of BCAP were then modified, and the modified BCAP was used to

revise the performance-level selection guidelines under NCHRP Project 22-8 (*2*).

- The results of this analysis confirmed some of the problems associated with the use of police-level accident data (i.e., incorrect coding of accident location, object struck, accident outcome, and vehicle type). For example, only 81.9 percent of the bridge railing accidents were successfully matched to bridges, which suggests that there are errors in the coding of object struck or in the reported locations of the accidents. Results of the manual review of hard copies of accident reports are even more alarming. Only 17 of 53 (32.1 percent) reported bridge railing accidents actually involved bridge railings. Of these 17 accidents, only 10 (58.8 percent) were coded correctly in terms of the vehicle going through or over the bridge railings. Furthermore, only 6 of the 24 (25 percent) single-unit trucks were identified correctly, whereas all 29 combination trucks were correctly coded. In light of these coding problems, great care should be taken in using the accident statistics presented in this paper.

REFERENCES

1. *Guide Specifications for Bridge Railings*. AASHTO, Washington, D.C., 1989.
2. Mak, K. K., and D. L. Sicking. *Evaluation of Performance Level Selection Criteria for Bridge Railings*. Final Report. NCHRP Project 22-8. Transportation Research Board, Washington, D.C., 1992.

Single-Slope Concrete Bridge Rail

KING K. MAK, DON J. GRIPNE, AND CHARLES F. MCDEVITT

A single-slope concrete median barrier was previously designed, developed, and successfully crash tested at the Texas Transportation Institute in accordance with guidelines set forth in *NCHRP Report 230*. The Washington State Department of Transportation is interested in adapting this single-slope barrier design for use as a bridge rail. The results of full-scale crash tests conducted on this single-slope concrete bridge rail and the evaluation of its impact performance are presented. The single-slope concrete bridge rail was judged to have successfully met all evaluation criteria set forth in *NCHRP Report 350* and the 1989 AASHTO *Guide Specifications for Bridge Railings* and is recommended for field implementation.

A single-slope concrete median barrier was previously designed, developed, and successfully crash tested at the Texas Transportation Institute (TTI) in accordance with guidelines set forth by Beason et al. (*1*) and in *NCHRP Report 230* (*2*). The barrier was approved by FHWA and was adopted by many states for field implementation. As implied by its name, this single-slope barrier has a single sloped face at 79 degrees (or 11 degrees to the vertical) and is 42 in. (1.07 m) high. The single-slope barrier has several advantages over the New Jersey safety-shaped barrier. First, the single-slope barrier has a lower propensity for causing rollover than the New Jersey safety-shaped barrier without greatly increasing the damage and lateral acceleration to impacting vehicles.

Second, although the initial construction cost of the single-slope barrier is comparable to that of standard safety-shaped barrier, the maintenance costs and life cycle costs of the single-slope barrier should be substantially lower than those of the standard safety-shaped barrier. To maintain the shape and height of the barrier for the standard safety-shaped barrier, the pavement surface must first be lowered before any overlay can be applied to provide a new wearing surface. This is an expensive outlay over the life of the pavement and the barrier. On the other hand, a single-slope barrier can accommodate overlays without any concern for the shape of the barrier. Also, with an initial height of 42 in. (1.07 m), the barrier can accommodate up to 10 in. (254 mm) of overlay, e.g., five overlays of 2 in. (51 mm) each over the years, and still has a height of 32 in. (0.81 m), which is the height of the standard safety-shaped barrier.

Third, the single-slope barrier can be advantageous in situations in which there are differences in elevation between the two sides of divided highways, such as at superelevated curves. Because there is only a single sloped face, the height of the barrier can be different on the two faces to accommodate the difference in elevation without concern over the shape of the barrier. This can simplify the construction of the barrier, especially when slip-forming is used.

The Washington State Department of Transportation (WaDOT) is interested in adapting this single-slope barrier design for use as a bridge rail. Although the single-slope barrier design was originally intended for use as a median barrier, there is no reason why it could not be used as a bridge rail. The key difference between the median barrier and the bridge rail applications would be the height of the barrier, which is 42 in. (1.07 m) for the median barrier and 32 in. (0.81 m) for the bridge rail. On the basis of results of previous crash tests, the impact performance of the barrier should not be adversely affected when the barrier height is lowered from 42 to 32 in. (1.07 to 0.81 m). The other difference is that the bridge rail is tied into the bridge deck, whereas the median barrier is keyed in place with an asphalt overlay. However, because the barrier remains essentially rigid in both applications, there should not be any effect on its impact performance. It should be noted that although the bridge rail was tested at a rail height of 32 in. (0.81 m), the bridge rail would perform satisfactorily at greater rail heights.

Presented in this paper are the results of full-scale crash tests conducted on this single-slope concrete bridge rail and the evaluation of its impact performance.

TEST INSTALLATION

A schematic of the test installation is shown in Figure 1. Precast single-slope median barrier sections, previously fabricated by TTI in another study, were used for the test installation. The use of the precast single-slope median barrier sections saved the expenses of building a simulated bridge deck and the bridge rail. The rationale for this approach was that as long as the barrier remained rigid it really would not matter whether the bridge rail was tied into a simulated bridge deck. The concern was more with the shape and geometrics of the single-slope bridge rail and not the strength of the rail or its tie-in to the bridge deck. A schematic showing the rebar and connection details planned for use with the single-slope bridge rail by WaDOT and approved by FHWA is shown in Figure 2. Note that the connection details are adopted from the standard safety-shaped concrete bridge rail, which has been successfully crash tested in previous studies and was therefore not evaluated in the present study.

Four 30-ft (9.14-m) precast barrier sections were used for a total installation length of 120 ft (36.6 m). The barrier sections were connected with channel connectors at the bottom, and rebar grids were placed in the grid slots and were grouted in place. A ditch 10 in. (254 mm) deep was dug for placement of the barrier sections so that the height of the barrier above ground level was reduced from 42 to 32 in. (1.07 to 0.81 m). The bottom of the ditch was lined with base materials to ensure that the foundation for the barrier was level and smooth. To ensure that the barrier sections would remain rigid during the impacts, the barrier section within which the impacts would occur was doweled into the existing concrete pavement with No. 5 rebars spaced at 3 ft (0.91 m) center to center. Also, after the barrier was installed in the ditch, the back of the barrier was keyed with a concrete overlay, 24 in. (0.61 m) wide and 4 in. (102 mm) thick, and the area between the existing concrete pavement and the front of the barrier was backfilled with grout to make sure that the barrier

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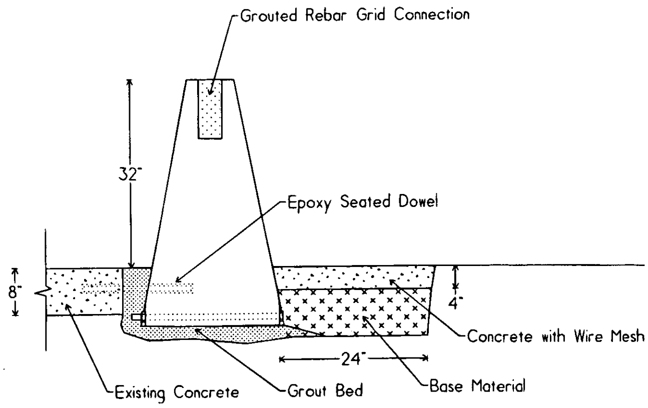


FIGURE 1 Single-slope concrete bridge rail test installation.

would remain rigid on impact. Photographs of the test installation are shown in Figure 3.

WaDOT plans to use a bridge rail height of 34 in. (0.86 m), which allows for a future overlay of 2 in. (51 mm). However, the crash tests were conducted with a bridge rail height of 32 in. (0.81 m) since the lower rail height was considered a more critical test condition. The schematic in Figure 2 showing the rebar and connection details is for a rail height of 32 in. (0.81 m), which was the rail height evaluated in the crash tests. However, the details for a 34-in. (0.86-m) single-slope bridge rail should be very similar.

CRASH TEST MATRIX

In accordance with requirements set forth in the 1989 AASHTO *Guide Specifications for Bridge Railings* (*3*) (hereinafter referred to

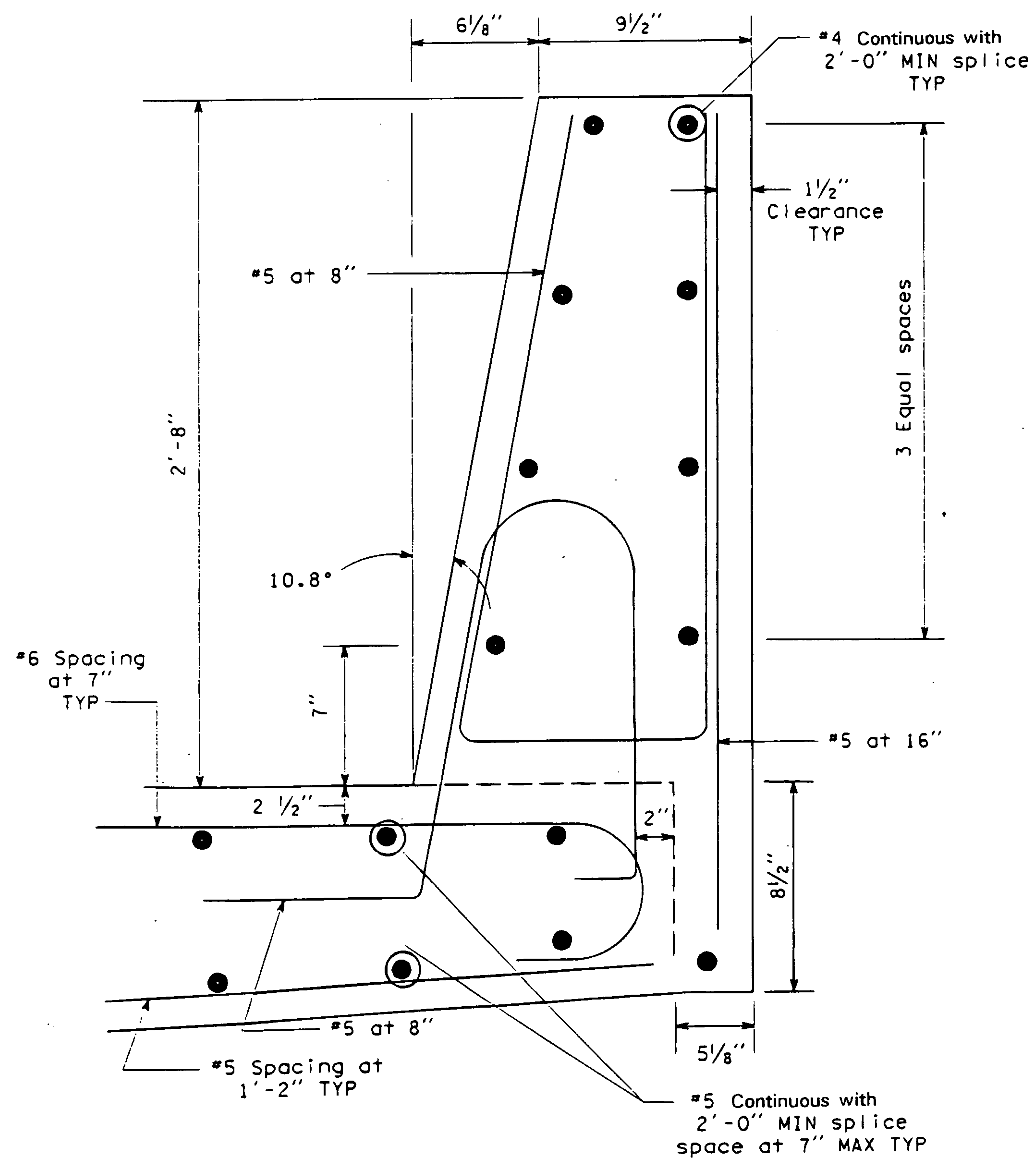


FIGURE 2 Reinforcement and connection details planned for single-slope concrete bridge rail.

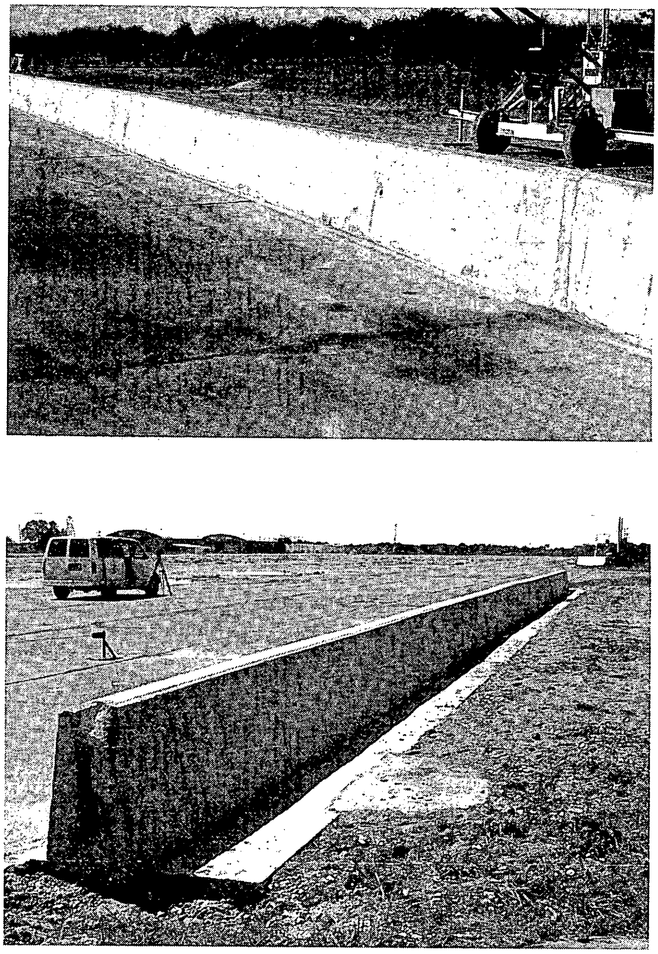


FIGURE 3 Single-slope concrete bridge rail test installation.

as the Guide Specifications) for a Performance Level 2 bridge rail, the following three crash tests are required:

1. An 1,800-lb (817-kg) passenger car striking the bridge rail at a nominal speed and angle of 60 mph (96.6 km/hr) and 20 degrees,
2. A 5,400-lb (2,449-kg) pickup truck striking the bridge rail at a nominal speed and angle of 60 mph (96.6 km/hr) and 20 degrees, and
3. An 18,000-lb (8,170-kg) single-unit truck striking the bridge rail at 50 mph (80.5 km/hr) and 15 degrees.

The crash test matrix was modified for the testing of the single-slope concrete bridge railing. The 1,800-lb (817-kg) passenger car severity test was considered unnecessary and was deleted from the crash test matrix. As mentioned previously, the single-slope concrete median barrier successfully passed the large car structural adequacy test and the small car severity test in accordance with guidelines set forth in *NCHRP Report 230* (*2*). On the basis of the results of those crash tests, it was believed that the barrier height would have little or no effect on the small car severity test and there was therefore no need to repeat the test.

As for the 5,400-lb (2,449-kg) pickup truck structural adequacy test at 60 mph (96.6 km/hr) and 20 degrees, it should also perform similarly to the 4,500-lb (2,043-kg) passenger car test at 60 mph (96.6 km/hr) and 25 degrees. However, although the large car successfully met the guidelines set forth in *NCHRP Report 230* (*2*), the vehicle exhibited a tendency to climb up on the barrier. There was therefore concern that vehicle stability might become a problem with the pickup truck when the barrier height was reduced from 42 to 32 in. (1.07 to 0.81 m). Thus, the crash test with the pickup truck was included. It was also decided that the test conditions for the pickup truck test would be in accordance with Test Level 4 of the new *NCHRP Report 350* (*4*) requirements [i.e., a 2,000-kg (4,409-lb) pickup truck striking the bridge rail at a nominal speed and angle of 100 km/hr (62.2 mph) and 25 degrees].

In summary, the actual crash test matrix used to evaluate the impact performance of this single-slope concrete bridge rail design included only the pickup truck and the single-unit-truck crash tests. Testing and evaluation were performed in accordance with guidelines outlined in *NCHRP Report 350* (*4*) and the 1989 AASHTO Guide Specifications (*3*).

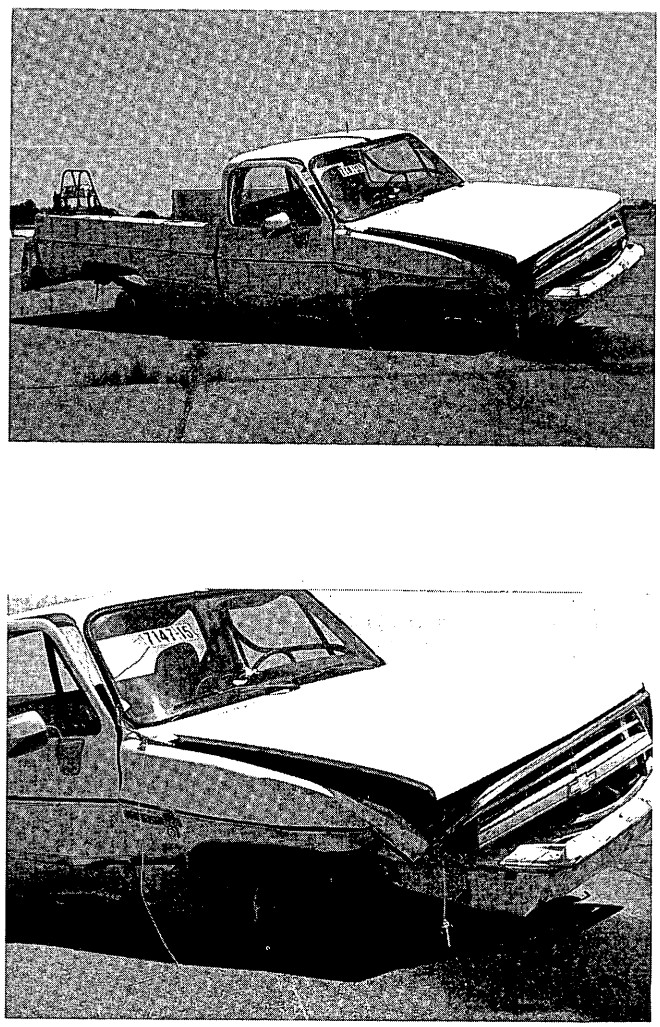
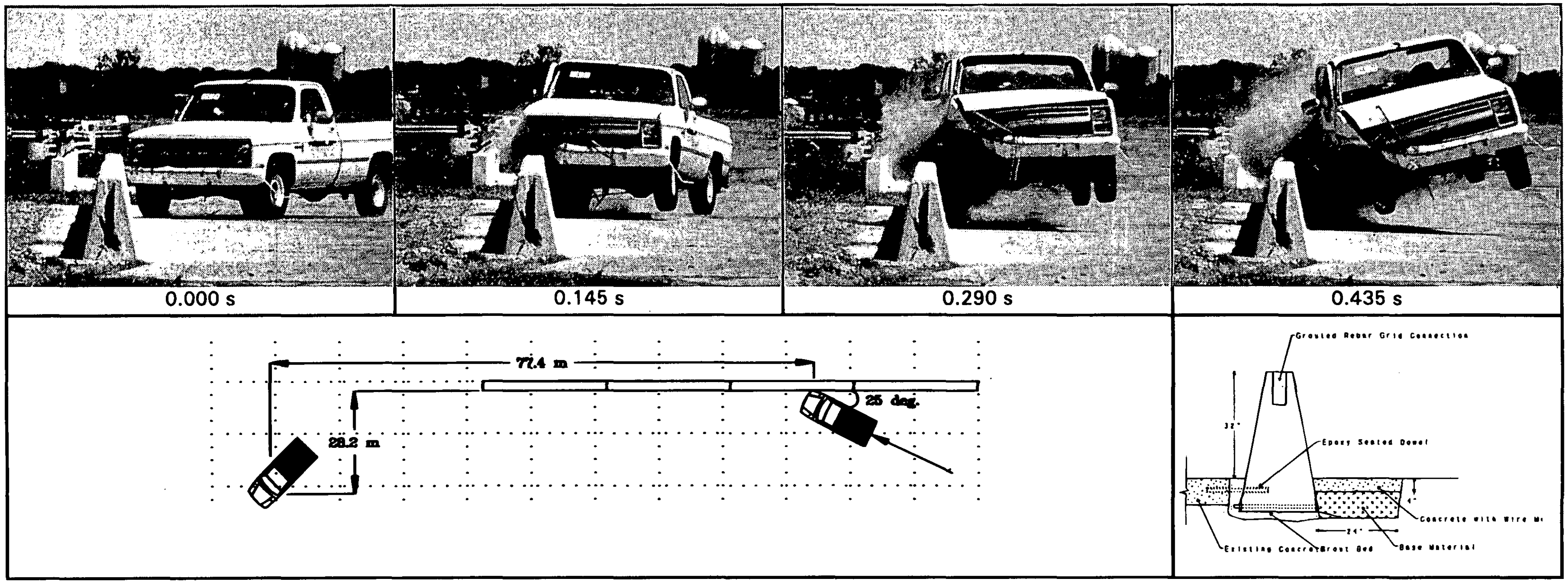


FIGURE 4 Vehicle after Test 7147-15.



General Information	
Test Agency	Texas Transportation Institute
Test No.	7147-15
Date	05/03/93
Test Article	
Type	Bridge Rail
	Single Slope Concrete
Installation Length (m)	36.6 (120 ft)
Size and/or dimension and material of key elements	81 cm (32 in) high concrete
Soil Type and Condition	N/A
Test Vehicle	
Type	Production Model
Designation	2000 P
Model	1985 Chevrolet Custom
Mass (kg) Curb	1,993 (4,390 lb)
Test Inertial	2,000 (4,405 lb)
Dummy	76 (167 lb)
Gross Static	2,076 (4,573 lb)

Impact Conditions	
Speed (km/h)	97.2 (60.4 mi/h)
Angle (deg)	25.5
Exit Conditions	
Speed (km/h)	76.6 (47.6 mi/h)
Angle (deg)	3.3
Occupant Risk Values	
Impact Velocity (m/s)	
x-direction	5.4 (17.7 ft/s)
y-direction	7.8 (25.6 ft/s)
THIV (optional)	
Ridedown Accelerations (g's)	
x-direction	-6.1
y-direction	-12.6
PHD (optional)	
ASI (optional)	
Max. 0.050-sec Averages (g's)	
x-direction	-7.3
y-direction	-13.3
z-direction	-5.6

Test Article Deflections (cm)	
Dynamic	N/A
Permanent	0.6 (0.3 in)
Vehicle Damage	
Exterior	
VDS	01RD5
CDC	01FREK2 & 01RDEW2
Interior	
OCDI	RF0020000
Maximum Exterior Vehicle Crush (cm)	41 (16.1 in)
Max. Occ. Compart. Deformation (cm)	13.9 (5.5 in)
Post-Impact Behavior	
Max. Roll Angle (deg)	30
Max. Pitch Angle (deg)	7
Max. Yaw Angle (deg)	40

FIGURE 5 Results for Test 7147-15.

RESULTS OF CRASH TESTS

Pickup Truck Crash Test (Test 7147-15)

A 1985 Chevrolet C-20 Custom Deluxe pickup truck was used for the crash test. The test inertia weight of the vehicle was 2,000 kg (4,409 lb) and its gross static weight was 2,076 kg (4,577 lb), including an uninstrumented 50th percentile male anthropometric dummy restrained in the driver's seat with lap and shoulder belts.

The vehicle struck the bridge rail 12.2 m (40.0 ft) from the upstream end at a speed of 97.2 km/hr (60.4 mph) and an angle of 25.5 degrees. The right front tire of the vehicle began to climb the face of the barrier on impact. Shortly thereafter the left front tire became airborne as the vehicle began to redirect. The rear of the vehicle then contacted the barrier, and shortly thereafter the rear wheels became airborne. The vehicle exited the barrier airborne, traveling at a speed of 76.5 km/hr (47.6 mph) and an angle of 3.3 degrees. The right front tire came back into contact with the pavement, and the tire and rim separated from the wheel hub subsequent to the impact with the pavement. The right rear tire and rim also separated from the wheel hub when the tire came back into contact with the pavement. The vehicle came to rest 77.4 m (254.0 ft) downstream and 21.2 m (69.5 ft) to the traffic side of the point of impact.

The barrier received only cosmetic damage (i.e., scrapes and tire marks), and there were two small cracks on the barrier. The vehicle was in contact with the barrier for 4.2 m (13.9 ft). The entire vehicle sustained extensive damage, as shown in Figure 4. Maximum deformation into the occupant compartment was 139 mm (5.5 in.) at the firewall area, and maximum exterior crush at the right front corner at bumper height of the vehicle was 410 mm (16.1 in). The right front wheel was pushed rearward 120 mm (4.7 in.), and the frame was bent.

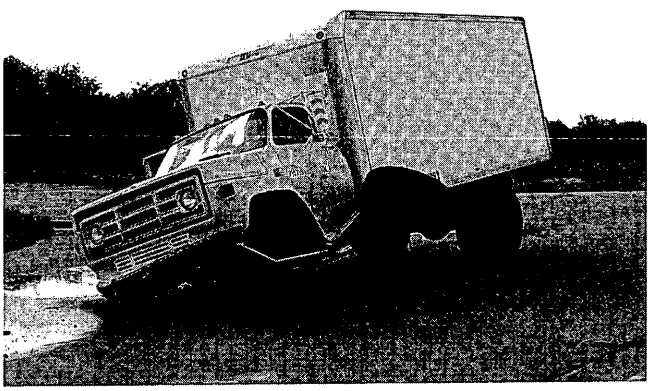
The occupant risk factors were well within the preferred limits set forth in *NCHRP Report 350* (*4*) and the Guide Specifications (*3*). In the longitudinal direction, occupant impact velocity was 5.4 m/sec (17.7 ft/sec), and the highest 0.010-sec average ridedown acceleration was −6.1 *g*. Lateral occupant impact velocity was 7.8 m/sec (25.6 ft/sec), and the highest 0.010-sec occupant ridedown acceleration was −12.6 *g*. A summary of the test results is provided in Figure 5.

Single-Unit-Truck Crash Tests (Tests 7147-16 and 7147-17)

A 1982 GMC single-unit truck with an empty weight of 5,262 kg (11,590 lb) was used for the second crash test (Test 7147-16). The vehicle was ballasted with sandbags to a test inertia and gross static weight of 8,172 kg (18,000 lb). The vehicle struck the bridge rail 13.7 m (45.0 ft) from the upstream end at a speed of 82.1 km/hr (51.0 mph) and at an angle of 10 degrees. Shortly after impact with the bridge rail the front axle separated from the vehicle. The right lower corner and edge of the box van then set down on top of the rail and rode along in this fashion until the vehicle rode off the end of the bridge rail test installation. The box van reached a maximum roll angle of 23 degrees, and the cab reached a maximum roll angle of 25 degrees. The box van then began to right itself and came to rest upright 65.4 m (214.5 ft) downstream and 2.6 m (8.5 ft) to the left of the point of impact (i.e., to the traffic side of the bridge rail).

The barrier received only cosmetic damage (i.e., gouges, scrapes, and tire marks). The vehicle was in contact with the barrier for 15.6 m (51.2 ft). The vehicle sustained extensive damage to the front suspension, as shown in Figure 6. Maximum crush at the right front corner of the vehicle was 178 mm (7.0 in.). The front axle was separated from the vehicle, and the spring shackles, U-bolts, shocks, mounts, tie rods, and steering arm were damaged. In addition, damage was sustained by the front bumper, the right front quarter-panel, and the right and left running boards. The windshield was cracked, and the fuel tank was scraped.

The occupant risk factors were well within the limits set forth in the Guide Specifications (*3*). In the longitudinal direction, occupant impact velocity was 2.3 m/sec (7.5 ft/sec), and the highest 0.010-sec average ridedown acceleration was −1.3 *g*. Lateral occupant impact velocity was 3.5 m/sec (11.5 ft/sec), and the highest 0.010-sec occupant ridedown acceleration was −2.6 *g*. A summary of the test results is given in Figure 7.



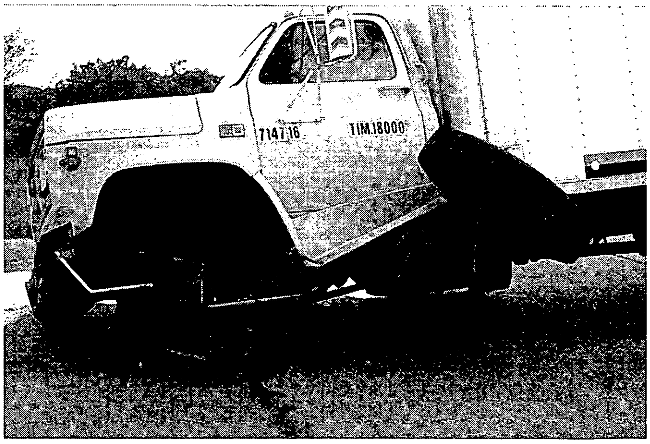
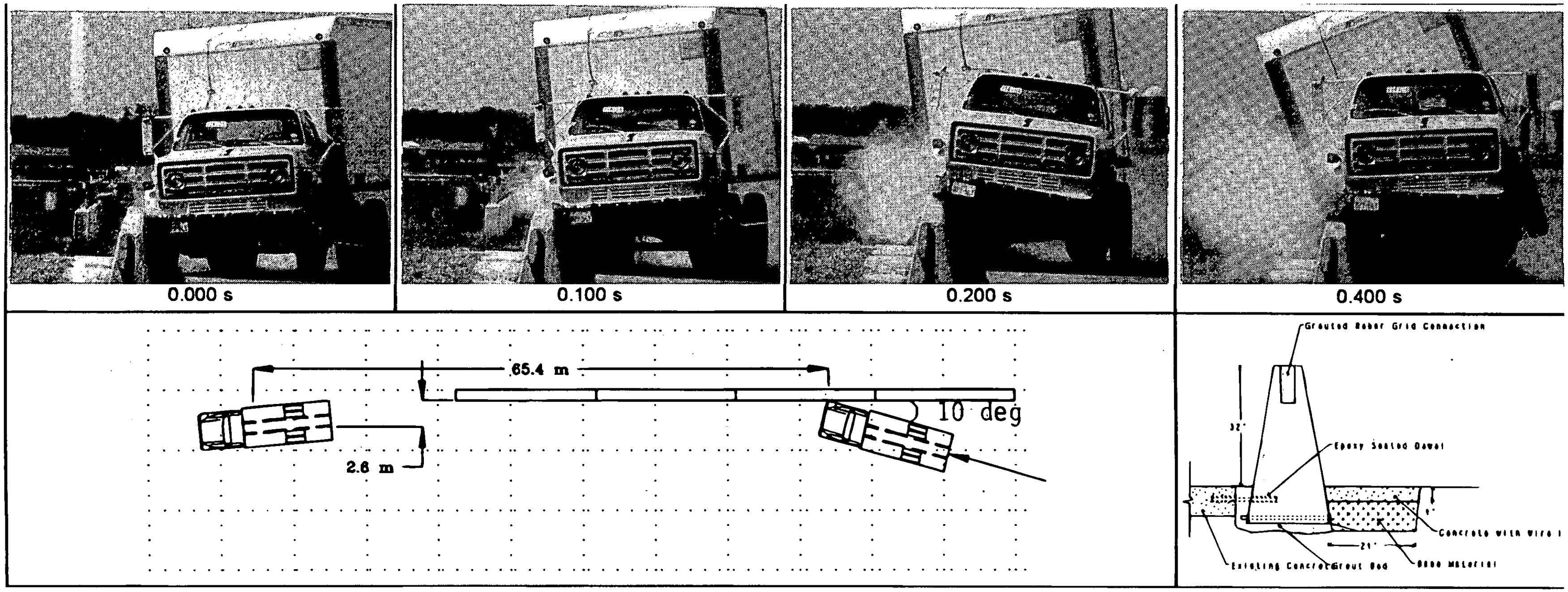


FIGURE 6 Vehicle after Test 7147-16.



General Information	
Test Agency	Texas Transportation Institute
Test No.	7147-16
Date	05/06/93
Test Article	
Type	Bridge Rail
	Single Slope Concrete
Installation Length (m)	36.6 (120 ft)
Size and/or dimension and material of key elements	81 cm (32 in) high concrete
Soil Type and Condition	N/A
Test Vehicle	
Type	Production Model
Designation	8000 S
Model	1982 GMC Single-Unit Truck
Mass (kg) Curb	5,262 (11,590 lb)
Test Inertial	8,172 (18,000 lb)
Dummy	N/A
Gross Static	8,172 (18,000 lb)

Impact Conditions	
Speed (km/h)	82.1 (51.0 mi/h)
Angle (deg)	10.0
Exit Conditions	
Speed (km/h)	N/A
Angle (deg)	0
Occupant Risk Values	
Impact Velocity (m/s)	
x-direction	2.3 (7.5 ft/s)
y-direction	3.5 (11.5 ft/s)
THIV (optional)	
Ridedown Accelerations (g's)	
x-direction	-1.3
y-direction	-2.6
PHD (optional)	
ASI (optional)	
Max. 0.050-sec Averages (g's)	
x-direction	-1.3
y-direction	-2.7
z-direction	2.4

Test Article Deflections (cm)	
Dynamic	N/A
Permanent	0.2 (0.1 in)
Vehicle Damage	
Exterior	
VDS	
CDC	
Interior	
OCDI	RF0000000
Maximum Exterior Vehicle Crush (cm)	17.8 (7.0 in)
Max. Occ. Compart. Deformation (cm)	0
Post-Impact Behavior	
Max. Roll Angle (deg)	19.8
Max. Pitch Angle (deg)	-2.3
Max. Yaw Angle (deg)	-8.2

FIGURE 7 Results for Test 7147-16.

The vehicle struck the bridge rail at an angle of 10 degrees instead of the required 15 degrees. An extensive investigation revealed that the guidance cable release mechanism did not function properly, causing the front tires to turn to the left abruptly, which in turn caused the truck to yaw counterclockwise.

Because of the lower-than-required impact angle in the first single-unit-truck test, the single-unit-truck test was repeated (Test 7147-17). A 1985 GMC single-unit truck was used for this test. The vehicle struck the bridge rail 13.1 m (43.0 ft) from the upstream end at a speed of 82.5 km/hr (51.3 mph) and an angle of 17.9 degrees. Shortly after impact, the right front tire began to climb the face of the bridge rail and the front axle became partially separated from the vehicle. The box van began to roll to the right, reaching a maximum roll angle of 53 degrees. The lower right corner and edge of the box van set down on top of the bridge rail and rode along in this fashion until the vehicle rode off the end of the bridge rail test installation. After the vehicle rode off the end of the bridge rail test installation, the front axle separated from the vehicle as the front end contacted the pavement and the rear tires of the vehicle dug into the dirt. The vehicle began to roll to the left and eventually rolled onto its left side. The vehicle came to rest 49.7 m (163.0 ft) downstream and 2.9 m (9.5 ft) behind the point of impact.

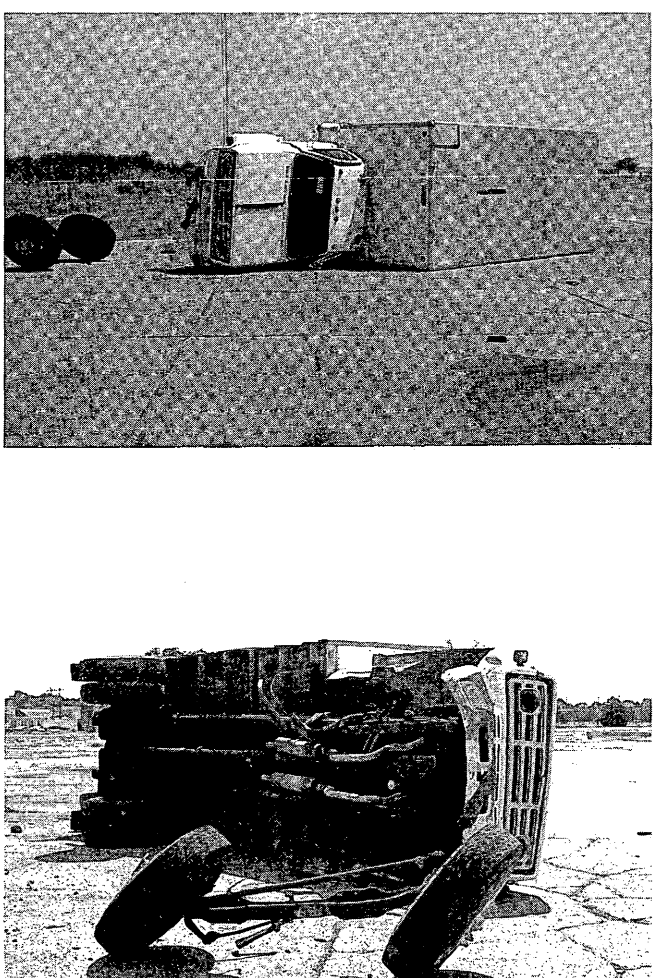


FIGURE 8 **Vehicle after Test 7147-17.**

The barrier again received only cosmetic damage (i.e., gouges, scrapes, and tire marks). The vehicle was in contact with the barrier for 23.5 m (77.0 ft). The vehicle sustained extensive damage, as shown in Figure 8. Maximum crush at the right front corner of the vehicle was 22.9 cm (9.0 in.). The front axle was separated from the vehicle, and the front suspension was damaged extensively. In addition, damage was sustained by the front bumper and grill and the right front fender, door, and running board. The entire left side of the vehicle sustained dents and scrapes due to rollover on the left side. The fuel tanks were scraped on both sides.

The occupant risk factors were again well within the limits set forth in the Guide Specifications (*3*). In the longitudinal direction, occupant impact velocity was 2.9 m/sec (9.7 ft/sec), and the highest 0.010-sec average ridedown acceleration was $-2.7\ g$. Lateral occupant impact velocity was 2.8 m/sec (9.3 ft/sec), and the highest 0.010-sec occupant ridedown acceleration was $-10.2\ g$. A summary of the test results is given in Figure 9.

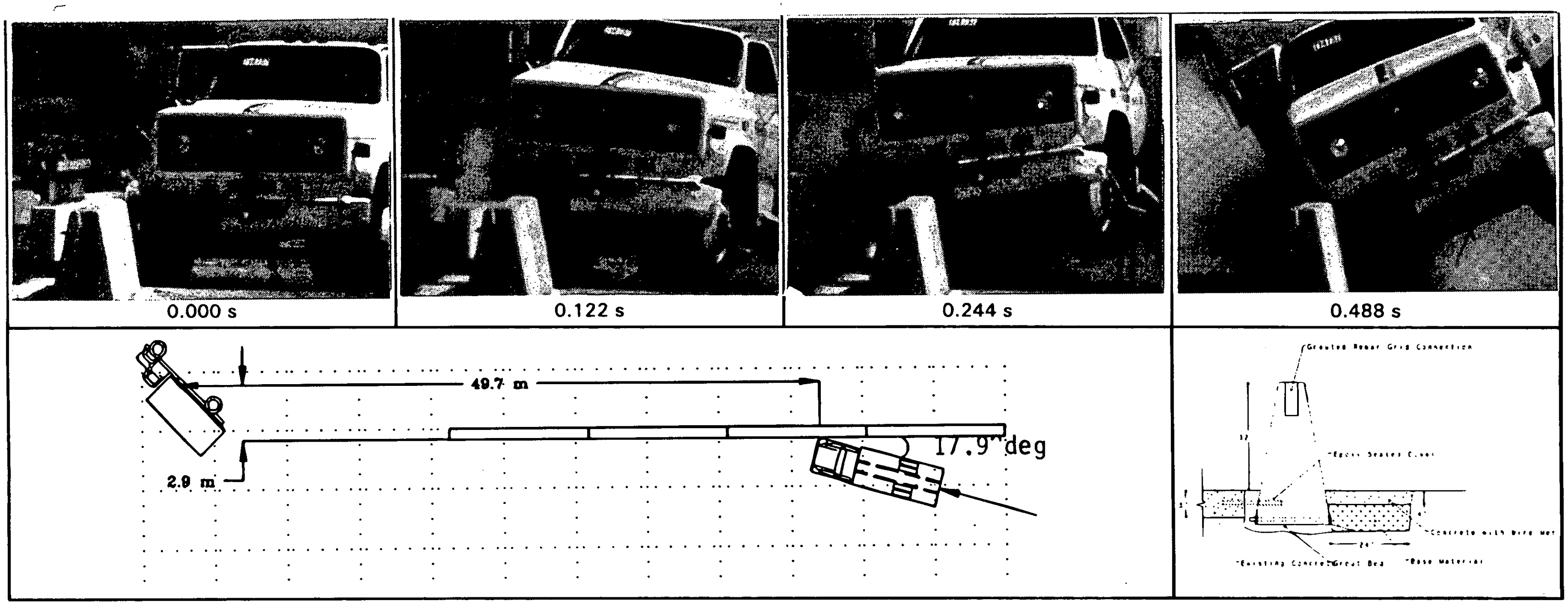
The impact angle of 17.9 degrees was greater than the required angle of 15 degrees. Extensive investigation, including detailed analysis of photographic and electronic data, failed to reveal any potential problems that could have caused this higher-than-required impact angle.

SUMMARY OF FINDINGS

The single-slope concrete bridge rail was judged to have successfully met all evaluation criteria set forth in *NCHRP Report 350* (*4*) and the 1989 AASHTO Guide Specifications (*3*) and is recommended for field applications.

For the pickup truck test (Test 7147-15), the single-slope concrete bridge rail contained and smoothly redirected the vehicle. There were no detached elements or debris to cause undue hazard to the occupants of the vehicle or to adjacent traffic. The vehicle sustained moderate damage with minor deformation into the occupant compartment. The vehicle remained upright and relatively stable during the collision period; however, there were some moderate pitching and yawing after the vehicle exited from the bridge rail. Although the vehicle came to rest 21.2 m (67.5 ft) from the traffic side of the bridge rail, the trajectory of the vehicle was judged to pose minimal potential hazard to adjacent traffic. Part of the vehicle trajectory could be attributed to the separation of the tires and rims from the wheel hubs for the two right-side tires. Also, the exit angle of 3.3 degrees was substantially less than 60 percent of the impact angle. The occupant impact velocities and ridedown accelerations were well within the limits set forth in *NCHRP Report 350* (*4*) and the 1989 AASHTO Guide Specifications (*3*).

For the two single-unit-truck tests (Tests 7147-16 and 7147-17), the single-slope concrete bridge rail contained and redirected the test vehicles and did not allow the vehicles to penetrate or go over the bridge rail. There were no detached elements or debris from the bridge rail to present undue hazard to occupants in the vehicles or other adjacent traffic. The integrity of the occupant compartment was maintained. In Test 7147-16 the vehicle remained upright and relatively stable during and after the collision. In Test 7147-17 the vehicle remained upright during collision with the bridge rail, but then rolled over onto its left side (nonimpact side) after exiting from the bridge rail test installation. The rollover occurred on the traffic side of the bridge rail, which is considered acceptable under the evaluation criteria set forth in the 1989 AASHTO Guide Specifica-



General Information

Test Agency	Texas Transportation Institute
Test No.	7147-17
Date	05/06/93
Test Article	
Type	Bridge Rail
	Single Slope Concrete
Installation Length (m)	36.6 (120 ft)
Size and/or dimension and material of key elements	81 cm (32 in) high concrete
Soil Type and Condition	N/A
Test Vehicle	
Type	Production Model
Designation	8000 S
Model	1985 GMC Single-Unit Truck
Mass (kg) Curb	5,207 (11,470 lb)
Test Inertial	8,172 (18,000 lb)
Dummy	N/A
Gross Static	8,172 (18,000 lb)

Impact Conditions

Speed (km/h)	82.5 (51.3 mi/h)
Angle (deg)	17.9
Exit Conditions	
Speed (km/h)	N/A
Angle (deg)	0
Occupant Risk Values	
Impact Velocity (m/s)	
x-direction	2.9 (9.7 ft/s)
y-direction	2.8 (9.3 ft/s)
THIV (optional)	
Ridedown Accelerations (g's)	
x-direction	-2.7
y-direction	-10.2
PHD (optional)	
ASI (optional)	
Max. 0.050-sec Averages (g's)	
x-direction	-2.0
y-direction	-5.6
z-direction	-1.4

Test Article Deflections (cm)

Dynamic	N/A
Permanent	0.2 (0.1 in)
Vehicle Damage	
Exterior	
VDS	
CDC	
Interior	
OCDI	RF0000000
Maximum Exterior Vehicle Crush (cm)	
Max. Occ. Compart. Deformation (cm)	
Post-Impact Behavior	
Max. Roll Angle (deg)	53.0
Max. Pitch Angle (deg)	4.3
Max. Yaw Angle (deg)	-18.9

FIGURE 9 Results for Test 7147-17.

tions (*3*). The vehicle trajectory did not pose any potential hazard to adjacent traffic in either test.

The impact angles for the two single-unit-truck tests were too low (10 degrees) in the first test and too high (17.9 degrees) in the second test. However, because both tests successfully met all evaluation criteria, it is reasonable to argue that the single-slope concrete bridge rail would have performed satisfactorily had the impact angle been at the required 15 degrees. A review of the two tests showed that, for the test with the higher impact angle, the vehicle was less stable with a much higher roll angle toward the barrier and a slightly higher climb on the barrier during impact with the bridge rail. Of course, the vehicle rolled over after exiting from the bridge rail in the test with the higher impact angle.

REFERENCES

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2. Michie, J. D. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances.* TRB, National Research Council, Washington, D.C., March 1981.
3. *Guide Specifications for Bridge Railings.* AASHTO, Washington, D.C., 1989.
4. Ross, H. E., Jr., D. L. Sicking, R. A. Zimmer, and J. D. Michie. *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features.* TRB, National Research Council, Washington, D.C., 1993.

SafetyAdvisor: Framework for Performing Roadside Safety Assessments

MALCOLM H. RAY

A software tool for assisting roadway designers in assessing the safeness of roadside designs is described. The SafetyAdvisor is an interactive Windows 3.0 program that graphically displays the roadway and hazardous objects along it. The probability of observing no severe or fatal accidents in a year is used as a measure of the safety of the roadway. This measure is calculated using user-definable probablistic expressions. The SafetyAdvisor provides a convenient computational framework for estimating the effectiveness of different roadway design alternatives.

A prototype safety assessment software tool, the SafetyAdvisor, was developed to aid engineers in assessing the safeness of roadway sections and alternative designs (*1*). The SafetyAdvisor was developed for 386 and better-compatible personal computers using C++ (*2*) running under Windows 3.0 (*3*). Input is accomplished through ASCII text files that describe the roadway geometry, the operational conditions, and the locations of hazards. Output is displayed graphically to the user. The assessment tool calculates the probability of not observing a fatal or severe accident within the next year on the basis of the roadway characteristics that the user supplies. The safety scale, displayed in the view along with a representation of the roadway, is calculated on the basis of probability models that are stored in ASCII text files that can be modified using a standard text editor. The SafetyAdvisor automatically finds the appropriate model and calculates the safety scale on the basis of the observable characteristics of each roadway segment.

MATHEMATICAL MODEL

The safety scale measures the probability that an accident of a particular severity will not be observed within a 1-year period. This scale is based on the assumption that accidents are a random Poisson process as described by the following equation (*4,5*):

$$P(x) = \frac{e^{-\lambda t} (\lambda t)^x}{x!} \tag{1}$$

In the context of safety assessment, x is the number of accidents, λ is the accident rate, and t is the period of time being considered. The condition $x = 0$, $t = 1$ represents the probability of observing no accidents in a period of 1 year. Recalling that $x!$ of zero is 1 and λt to the zero power is 1, the following expression is obtained:

$$S = e^{-\lambda} = e^{-P(I) \cdot \text{ADT}} \tag{2}$$

Momentum Engineering, Inc., 6935 Birch Street, Falls Church, Va. 22046-2202.

where λ, the accident rate, has been replaced by $P(I) \cdot$ ADT. $P(I)$ is the probability that any one trip through the segment will result in an accident of severity I or greater. Severity, I, could be defined in several ways: a fatal accident, an injury accident, a tow-away accident, or any accident at all. ADT is the average daily traffic volume in vehicles per day. The units of $P(I)$ are accidents per vehicle per day per year, so the units of $P(I) \cdot$ ADT are accidents of severity I per year.

If the accident rate [i.e., λ or $P(I) \cdot$ ADT] were zero, the safety scale would be 1 (i.e., no chance of an accident of severity I occurring). For a given traffic volume, the safety scale decreases as the probability of an accident, $P(I)$, increases. Similarly, for a given probability of an accident, the safety scale decreases as the traffic volume increases.

Although the safety scale, S, is a physically meaningful measure of the absolute safeness of the roadway, it does not show how safe a particular site is with respect to other similar sites. If the absolute safety scale, S, of each segment of roadway in a jurisdiction were measured and recorded, the standard deviation of S could be obtained. A relative safety scale, z_s, could be defined as the difference between the observed and the mean values of S divided by the standard deviation of S:

$$z_s = \frac{(S_{\text{obs}} - S_{\text{avg}})}{\sigma_s} \tag{3}$$

where

S_{obs} = observable absolute safety scale of a particular roadway segment;
S_{avg} = mean absolute safety scale of roadways with similar functional classifications; and
σ_s = standard deviation of the safety scale for similar roadways.

The relative safety scale represents the number of standard deviations that a particular observed absolute safety scale is above or below the mean safety scale for that functional class. This is exactly how most states define a hazardous location using accident rates.

Estimating the number of injury accidents involves summing the effects of all of the potentially hazardous events that a vehicle could encounter while traversing the segment. The probability of an accident of severity I involving a particular hazard is given by (*6*)

$$P(I)_i = P(E)_i \, P(C \mid E)_i \, P(I \mid C)_i \tag{4}$$

where

$P(E)$ = probability of encroaching onto the roadside,
$P(C \mid E)$ = probability of colliding with an object given that an encroachment has occurred, and
$P(I \mid C)$ = probability of a severity I injury given that a collision has occurred.

The probability of experiencing an injury accident on a road segment could be estimated by combining the $P(I)_i$ of each hazard in the segment. The subscript i denotes a particular scenario like running off the road and striking a tree or overturning on a steep side slope. Equation 4 can be used to combine all of the possible hazards the vehicle occupant is likely to encounter along the highway.

Encroachments initiate a sequence of events that sometimes results in an accident. The collision model, $P(C \mid E)$, describes the probability that an encroachment will progress into an accident. The severity model, $P(I \mid C)$, is the probability that if a collision occurs it will have severity I. A general form for the three conditional probabilities in Equation 4 can be written as

$$P_j(E) = \prod_{k=1}^{l} a_k b_k^{c_k}$$

$$P_j(C \mid E) = \prod_{k=1}^{l} d_k e_k^{f_k}$$

$$P_j(I \mid C) = \prod_{k=1}^{l} g_k h_k^{i_k} \qquad (5)$$

The symbol Π indicates that each term is multiplied by the next term or, more generally, that each term is functionally connected to the last term. The values for a_k through i_k are characteristics of the roadway or constants. The values could come from analytical methods, statistical analyses, or experience. The important concept is that the encroachment, collision, and severity are predicted by some set of measurable characteristics of the highway. The mathematical basis for the SafetyAdvisor software is more fully described in other publications (*1,7,8*).

DESCRIPTION OF SAFETYADVISOR

Solving a safety assessment problem by hand or with computer software involves three types of information: data, mathematical models, and procedures. In the context of safety assessment, data are the observable characteristics of the roadway. The mathematical models transform the basic data into abstract quantities that describe the effect of each hazard. The safety assessment procedure takes these abstract quantities and transforms them into a single measure of the safety of the whole roadway. Typical roadside safety software tools like Roadside or the Benefit Cost Analysis Program (BCAP) represent the mathematical models and procedures in computer code; the data are input by the user. The SafetyAdvisor is structured differently since only the procedure is represented as computer source code. The data and the mathematical models are provided as inputs. The advantage of this technique is that specific mathematical models can be easily changed and updated as better models become available. Changing the encroachment model in Roadside, for example, would require changing the source code, recompiling, and redistributing the executable version. Changing the encroachment model in the SafetyAdvisor requires changing an input file with a text editor; no software needs to be modified.

Data Files

Input data files are all standard ASCII text files that can be manipulated with any standard ASCII text editor like the Windows Notepad editor (*3*). Table 1 shows the typical input file format. A quotation mark in any of the files delimits a comment, and all key words are terminated with a colon. The user must provide a project file and three characteristics files to perform a safety assessment using the SafetyAdvisor:

- Project file—contains the names of the hazard, operational conditions, and roadway geometrics characteristics files as well as other data needed to start the assessment procedure. The upper left example in Table 1 shows a project file for the SafetyAdvisor.
- Roadway Geometric file—contains information about roadway characteristics that change from location to location. The degree of curvature and grade are examples of the type of information that can be stored in this file. The lower left example in Table 1 shows an example of a simple geometric characteristics file.

TABLE 1 Example Roadway Characteristics File

Project File	*Operational Conditions File*
TITLE: An example road	"File name: ex_op.txt
NAME: example.prj	"Operation conditions
HAZARD DATA: ex_haz	ADT: 2000
OPCOND DATA: ex_op	Speed: 35
ALIGNMENT DATA: ex_road	Lane_width: 11
STARTING SEGMENT: 100	Clearzone_width: 10
SEGMENT INCREMENT: 50	Shoulder_width: 2
	Grade: -5
	Mean_Safety_Scale: 0.9669
	Stnd_Dev_Safety_Scl: 0.022
Roadway Geometric File	*Hazard Location File*
"File name: ex_road.txt	"File name: ex_haz.txt
"Roadway characteristic file	"Hazardous location file
Location: 100	"steep side slope
Curvature: 0	Location: 101 slope
Location: 150	offset: 8,
Curvature: 30	length: 200,
Location: 200	slope: -0.75
Curvature: 30	Location: 300 slope
Location: 250	.
Curvature: 30	.

- Operational Conditions file—contains information about characteristics that are constant over the entire roadway. Information like the posted speed, lane widths, shoulder types, and traffic volume could be stored in this file. The upper right example in Table 1 shows an operational conditions file for the SafetyAdvisor.
- Hazard Location file—contains a list of hazards, their locations, and their characteristics. Each hazard name must correspond to the file name of a hazard severity model (discussed in the next section). For example, using the hazard name SLOPE will cause the program to search the current directory for a model file named "slope.mdl." This file has the information required to calculate the severity of the collision. The lower right example in Table 1 shows a simple example of a hazardous location file. This file only has one hazard, a steep side slope. The values after the hazard name (slope) are characteristics of that particular hazard. A more typical hazardous location file would contain the locations and characteristics of trees, guardrails, utility poles, slopes, and any other roadside objects for which the user has mathematical severity models.

The program does not require any particular set of characteristics or key words to perform an analysis. The program reads in a line of text and searches for a character string terminated by a colon. This string becomes the name of a characteristic. For example, Grade and Speed in Table 1 are defined as characteristics of the whole road when this file is read into the software. The value following the colon is the value associated with the new characteristic name. Grade has the value −5 and Speed has the value 35. Slope is defined as a characteristic of the segment from station 101 to station 300 since it appears in the hazard location file (Table 1, lower right example). If a new characteristic is needed, all the user needs to do is type it into either the geometric, operational conditions, or hazardous locations input files with the corresponding values.

Input Model Files

Model files are required to calculate the values for the probabilities of encroachment, collision, and severity needed to evaluate Equation 4. Although they may, users need not ever interact with these files. They can be developed and distributed by the user's agency or national organizations such as FHWA, NCHRP, or AASHTO. Three types of model files correspond to the three conditional probabilities in Equation 4:

- Encroachment Model files, like the first example shown in Table 2, specify the details of the encroachment model to be used in evaluating the hazard.
- Collision Model files, like the second example shown in Table 2, estimate the probability that a collision will occur with the hazard given that an encroachment occurs.
- Severity Model files, like the third example shown in Table 2, estimate the probability that a collision with the specified type of hazard will result in an accident of severity *I*.

All model files have the structure and format shown in Table 2. The key words Name and Type identify comments about the source and type of the model. The next group of lines identifies the algebraic terms of the model. Each new term is identified with the key word Term. There is no limit on the number of terms that can be in a mathematical model. Each term is composed of four parts: operator, coefficient, variable, and exponent. These parts are stored as strings so the values can be either numbers or strings of text characters.

TABLE 2 Example Probability Model Files

```
Encroachment Model File
Name: Roadside Design Guide
Type: Run-off-road   encroachments
Term: *{0.0005*0.8909^GRADE[-6,-2]}
Term: *{1.0*1.25^CURVATURE[-6,6]}
Term: *{LENGTH[6,100]*0.00018939^1.0}

Collision Model File
Name: Roadside Design Guide
Type: Run-off-road collision model
Term: *{0.1520*1.0435^SPEED}
Term: *{1.0*0.9036^OFFSET}

Severity Model File -- Steep Slope
NAME: Steep Slopes -- Roadside Design Guide
TYPE: Severity
Term: *{0.001286*0.0007412^SLOPE[-0.1,-0.7]}
Term: *{1.0*0.9800^SPEED[40.,70.]}

Severity Model File -- G4 Guardrail
NAME: G4 Guardrail -- Roadside Design Guide
TYPE: Severity
Term: *{3.065e-08*SPEED[40.,70.]^3.}
```

When the safety scale is being calculated the SafetyAdvisor searches the Hazard file (e.g., Table 1) for hazards in the active segment. Each time the SafetyAdvisor encountered the word "slope" in the hazard characteristics file (Table 1) it would search the current directory for a model file named "slope.mdl." The SafetyAdvisor would read in this model file and interpret each value as either a numerical value or a string. For the first term in the hazard model file (Table 2, third example), the values 0.001286 and 0.0007412 would be recognized as numerical values and assigned to the coefficient and variable of the term. The next group of characters is the string Slope. This will represent the "exponent" of the term. Since this is not a numerical value the SafetyAdvisor looks through all the characteristics files (Table 1) for the character string Slope in the currently selected segment. If Slope is found, the number following the word is used in the equation. The value for the Slope in Table 1 is −0.75. If a value cannot be found the user is asked for one. Square brackets are optional limits on the values that can be taken by the parameter. Slope, for example, must have a value of between −0.1 and −0.7, as shown in Table 2; if the value found in the characteristics file is outside that range the closest boundary value is used and a warning message is printed in the log file.

This method makes the SafetyAdvisor flexible since the program itself makes no assumptions about the data needed to perform a safety assessment. Slope is just an arbitrary string to the software; the information in the model files and the input files gives it meaning. If future research indicates that a variable Surface-Type is an important component of a predictive probablistic model, this new variable can be added to the model and input files with a text editor; no coding changes would be required in the software.

Scenario Files

Hazardous events can be grouped into common hazardous scenarios. Run-off-road accidents, for example, represent a variety of cases all involving the vehicle leaving the traveled way and entering the roadside. The user identifies the hazardous objects and characteristics along the side of the road in the hazard file, but there must be a mechanism for correctly associating a particular hazard with the correct encroachment and collision models. The scenario list defines the encroachment and collision model that should be used in conjunction with the hazard model. A scenario list for run-off-road hazards would have the following form:

Run_off_Road: rdge, rdgc

[tree, pole, wall, g4, slope]

"Run-off-road" is the arbitrary scenario name, and "rdge" and "rdgc" are the names of the encroachment and collision model files that should be found on the disk (Table 2). The square brackets contain all of the names of the hazards that belong to the run-off-road hazardous scenario. Collisions with trees, poles, walls, and guardrails are all members of the run-off-road scenario group, so the same encroachment and collision models will be used when these hazards are detected in the hazard file. This list can be added to, modified, and changed to suit the user's needs.

Program Output

Several graphical views are available to the user in the prototype software for viewing the input data and the analysis results. The plan view, the only view discussed in this paper, is a graphical representation of the data in the input file, as shown in Figure 1. (The SafetyAdvisor displays much more information on a color VGA monitor than is possible to show in black and white figures.) Figure 1 is based on information in the characteristics files (e.g., Table 1). Changing the width of the clear zone or the degree of horizontal curvature will cause the screen to be redrawn with the new values. Functions for drawing some hazards like trees, guardrails, fences, and slopes have been included.

The text in the upper left corner of the view identifies the segment, the safety scale on that segment, the relative safety scale, and

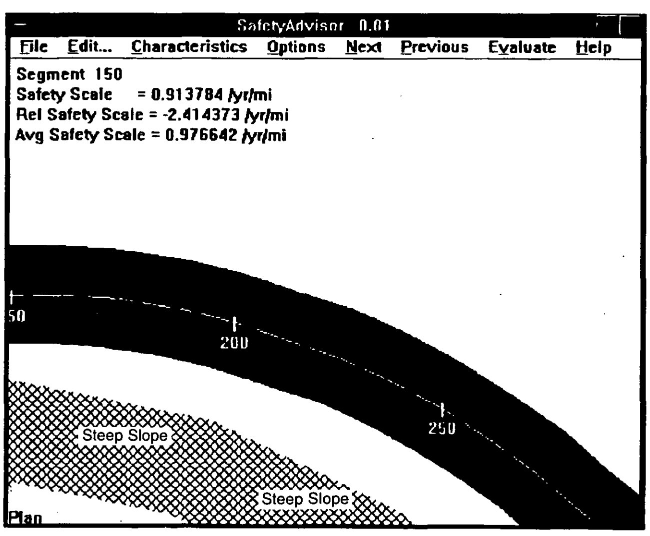

FIGURE 1 SafetyAdvisor view at Station 1 + 50: unshielded slope alternative.

the current running average safety scale on the roadway. Once the input files are assembled, the analysis proceeds by simply moving up and down the roadway by pressing the Next or Previous buttons. Each time the user presses the Evaluate, the Next, or the Previous buttons, the software calculates the safety scale and relative safety scale and displays it on the screen. The user may add, remove, or change hazardous objects and instantly see the effect on the safety scale. For example, the user could remove a tree, widen a lane, install a guardrail, or flatten a side slope and see how much the absolute and relative safety scales change. This feature makes what-if analyses easy to perform and provides a tool that the engineer can use to explore alternatives quickly.

There are a number of limitations to this prototype software related to programming and run-time efficiency. Many more features could be added to the code to make it even more flexible and easy to use. The purpose of this research, however, was to demonstrate how the safety scale could be used to assess roadway safeness. More detail on limits on roadway lengths, processing time, and input restrictions can be found in the program documentation (*1*).

EXAMPLE MODELS

The 1988 AASHTO *Roadside Design Guide* (*9*) contains the most widely disseminated guidelines for designing roadsides. Appendix A of the guide presents a cost-effectiveness approach to making roadside design decisions. The *Roadside Design Guide* contains models for encroachment, collision, and severity that could be transformed into the formats described in Equation 5.

Models based on the *Roadside Design Guide* (*9*) are used in the next section to present an example problem. The derivation of these models is not presented here but can be found in the documentation of the program (*1*). Although the *Roadside Design Guide* models are used throughout this paper to illustrate the use of the SafetyAdvisor, they should not automatically be considered authoritative or recommended. The *Roadside Design Guide* models are simply the first steps in developing models of encroachment, collision, and severity. Much research will be required to develop better, more realistic probablistic models, but the models are sufficient to illustrate how this type of probablistic method could be used.

There must be one severity model for each type of hazard found along the roadside. A mathematical severity model of a collision with a tree will be much different from a model of a guardrail collision. Finding a mathematical approximation of the probability of an injury I in an accident scenario involves two steps: a measure of severity must be selected, and the severity measure must be formulated in terms of the probability $P(I \mid C)$.

The first step is to choose a severity measure. There are several choices: all accidents, all tow-away accidents, all injury accidents, and all fatal accidents are measures of severity that have been used in the past. The societal cost of each of these severity levels and the weighted cost of distributions of these levels have also been used. The most costly accidents are the severe and fatal injury accidents, the so-called A + K accidents. One reasonable measure of severity is the probability of observing an A + K accident in a particular collision scenario; this is the measure of severity that will be used in this example.

The assumed percentage of accident type (severity) as a function of severity index is given in the 1989 AASHTO *Guide Specifications for Bridge Railings* (*10*). If the severe (A) and fatal (K) accidents are summed together and plotted against the severity index, a linear regression of these values will yield (assuming a cubic function) the following expression ($R^2 = 0.98$) (*1*):

$$P\,(\mathrm{A + K}) = 0.001286\,(\mathrm{SI}^3) \tag{6}$$

where SI is the 1977 barrier guide severity index (*11*).

This expression provides a reasonable way to map SIs to the probability of sustaining an A + K injury. This relationship is presented as a method for linking the probability of experiencing an A + K injury with the widely used severity indexes used in the barrier guide and the *Roadside Design Guide*.

Steep cross slopes are a common roadside hazard that are often shielded using guardrails. Table 3 shows the SIs recommended by Clinger (*12*) for side slopes on embankments as a function of slope and travel speed. These values, like most values associated with the *Roadside Design Guide*, are subjective estimates of the severity of accidents on side slopes. The severity of the accident is presumed to be a function of the magnitude of the slope (all of these slopes are negative, i.e., downhill) and the departure speed of the vehicle. A linear regression of the natural log of SI with the two independent variables (speed and slope) yields the following:

$$\mathrm{SI} = (0.0905C)\,(0.9933^V) \tag{7}$$

where C is the cross slope of the roadside and V is the assumed mean travel speed. Equation 8 can now be used to transform these SIs to the probability of a slope-related accident resulting in an A + K injury:

$$P\,(\mathrm{A + K} \mid C) = 0.001286\,(\mathrm{SI})^3$$

$$P\,(\mathrm{A + K} \mid C) = 0.001286\,[(0.0905^C)\,(0.9933^V)]^3$$

$$P\,(\mathrm{A + K} \mid C) = 0.001286\,(0.0007412^C)\,(0.9800^V) \tag{8}$$

This form can be used directly by the SafetyAdvisor as shown in Table 2. If a vehicle becomes involved in an accident on a steep side slope, this equation provides a method for estimating the probability that the accident will result in an A + K injury. A severity model for collisions with a G4 (1S) guardrail is shown in Table 4.

EXAMPLE PROBLEM

Site Characteristics

The following example is presented to show how the safety scale can be used to rank different roadway sites, perform benefit–cost analyses, and explore design alternatives as well as to illustrate the use of the SafetyAdvisor software.

TABLE 3 Average Severity Indexes of Accidents on Side Slopes

	Speed (mph)			
Slope	40	50	60	70
10:1	0.4	1.1	1.8	2.5
6:1	1.2	1.7	2.6	3.1
4:1	2.0	2.7	3.6	4.5
3:1	2.3	3.1	4.0	4.9
2:1	3.4	4.3	5.4	6.8

TABLE 4 Severity Indexes for G4(1S) Guardrails

Speed (mph)	Severity Index
40	2.6
50	3.1
60	3.6
70	4.3

The example road has a traffic volume of 2,000 vehicles per day with a downgrade of 5 percent and horizontal curvature of 30 degrees. A −3:4 (rise:run) side slope is on a 15-ft embankment on the right side of the roadway (going in the direction of increasing station numbers), and a tall cut is on the left. The side slope on the fill embankment is not shielded by a guardrail. The objective of this analysis will be to determine whether adding a guardrail will have a significant effect on the safeness of the roadway.

The characteristics are summarized in Table 1 for the geometrics, operational conditions, and hazard characteristics files, respectively. Only the run-off-road hazardous scenario will be considered. The encroachment and collision models for the run-off-road scenario are shown in Table 2. Only two hazard models are needed for this model—the steep side slope model and the G4 guardrail model shown in Table 2. Figure 1 shows the plan view of the unshielded example road at Station 1 + 50. The absolute and relative safety scales of this segment (from 1 + 50 to 2 + 00) are 0.9138 and −2.41 (shown in the upper left corner of the screen in Figure 1). As the engineer moves along the roadway by pushing the Next button, the safety scale values along the length of the roadway can be observed. The lowest (i.e., least safe) segment for the example roadway is between stations 1 + 50 and 2 + 00. If the engineer would like to see the effect of placing a guardrail along the road, the Edit menu selection could be chosen and a guardrail could be added to the hazard file, and the SafetyAdvisor view would be updated to include the new guardrail as shown in Figure 2.

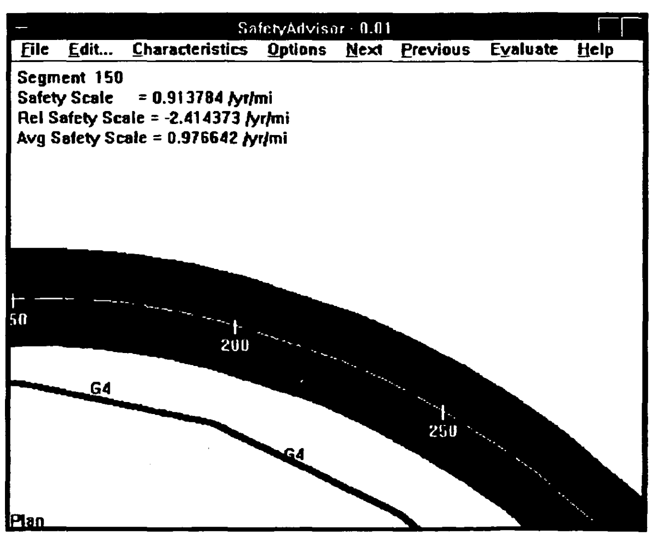

FIGURE 2 SafetyAdvisor view at Station 1 + 50: guardrail alternative.

The safety scales for the unshielded side slope alternative and the shielded side slope alternative (i.e., with a guardrail) are shown in Table 5. The right portion of the table shows the safety scale and the average safety scale for each homogeneous segment of roadway for the unshielded site. The left portion of the table shows the results for the site with a guardrail installed. The maximum difference between the safety scale on the improved roadway and the original unimproved roadway is 0.0665 on Segments 1 + 50 to 2 + 00 (e.g., $S_{guardrail} - S_{slope} = 0.9803 - 0.9138 = 0.0665$). Installing the guardrail reduced the probability of observing a serious accident by 0.0665.

Economic Analysis

Guardrail installation costs approximately $15 per foot, or $79,200 per mile. The *Roadside Design Guide* (*9*) recommends values of $110,000 and $500,000 for severe and fatal injuries, respectively. In 1984 and 1985 there were almost 25 times more injury accidents than fatal accidents on rural primary roads like this example roadway, so the weighted average A + K accident cost is (*13*)

$$\frac{(110{,}000 \cdot 25) + (500{,}000 \cdot 1)}{26} = \$125{,}000 \tag{9}$$

Assuming a 20-year design life and a 4 percent rate of return and assuming that the societal cost of a typical severe accident is $125,000 (*9*), the present worth (*PW*) of the accident cost reduction is

$$PW_{\text{acc. cost reduction}} = 13.59 \cdot 0.0665 \cdot 125{,}000$$

$$PW_{\text{acc. cost reduction}} = \$112{,}967 \tag{10}$$

The benefit–cost ratio for this improvement is the $112,967 accident cost reduction divided by the $79,200 cost of installing the guardrail, or 1.4. According to this analysis, the project is cost-beneficial since the present worth of the cost is less than the present worth of the accident reduction.

The benefit-cost approach, however, is dependent on the values chosen for each injury category and the rate of return. The analysis presented in the previous paragraph used the *Roadside Design Guide* cost values (*9*), but a recent FHWA technical advisory (*14*) advises using $11,000 and $1,500,000 for values of an injury and a fatal injury, respectively. The average weighted A + K accident cost using these values would be

$$\frac{(1{,}500{,}000 \cdot 1) + (11{,}000 \cdot 25)}{26} = \$68{,}269 \tag{11}$$

If $68,269 is substituted for $125,000, a benefit–cost ratio of 0.86 is obtained, indicating that the project is not cost-beneficial. This example shows one of the problems with decision criteria based on economic factors alone: the answer is dependent on the economic values chosen for the value of a severe or fatal injury and economic values like the rate of return. This is a valid method for allocating monetary resources but may not be adequate for measuring safeness. A method that does not rely on economic quantities would help engineers in establishing an absolute ranking that is only a function of the characteristics of the roadway and not the economic values currently in vogue. Calculating the safety scale provides such a method.

Safety Assessment

An alternative to an economic analysis would be to use the safety scale directly. The lowest safety scale in the uncorrected section of roadway shown in Table 5 was 0.9138. Does this represent a safe, a typical, or an unsafe roadway? Data from the Highway Safety Information System (HSIS) for Maine between 1986 and 1988 indicate that on average there were 242 fatal and severe roadside-related accidents on rural two-lane roadways, similar to the example roadway. The HSIS data also indicate that there are 7,191 mi of two-lane undivided roadway in the state, so the A + K accident rate is the 242 A + K accidents divided by the 7,191 mi of roadway, or 0.0337 A + K roadside accidents per mile of two-lane rural undivided roadway per year. Assuming that these values can be used for the example roadway, this value can be inserted directly into Equation 2 to calculate the safety scale:

$$S = e^{-\lambda} = e^{-0.0337} = 0.9669 \tag{12}$$

The least-safe road segment of the example road had a safety scale of 0.9138 (Table 5), making it less safe than other similar roads. If the county ranked all of its potential improvements by the observed safety scale, the county engineer could simply correct sites starting with the road with the lowest value on the safety scale and work up the list until the year's funding was exhausted or until a certain minimum safety scale was attained on all roads, say 0.95.

Another approach would be to use the relative safety scale defined in Equation 3. The average safety scale for this type of roadway (at least in Maine) was found to be 0.9669. If the standard devi-

TABLE 5 Results of SafetyAdvisor Analysis: South Berry Chapel Road

	G4(1S) Guardrail		-3:4 Slope	
Station	Safety Scale	Relative Safety Scale	Safety Scale	Relative Safety Scale
100	0.9948	1.27	0.9766	0.44
150	0.9803	0.61	0.9138	-2.41
200	0.9803	0.61	0.9138	-2.41
250	0.9803	0.61	0.9138	-2.41
300	0.9803	0.61	0.9138	-2.41

ation were known the relative safety scale could be calculated directly from Equation 3. In this case the standard deviation is not known but it can be estimated since the safety scale is an exponential distribution. The standard deviation for Equation 12 would be the square root of 1/ADT, in this case 0.022 (*5*). The relative safety scale for this unimproved roadway is therefore

$$z_s = \frac{S - \overline{S}}{\sqrt{1/\text{ADT}}}$$

$$z_s = \frac{0.9138 - 0.9669}{\sqrt{1/2{,}000}}$$

$$z_s = -2.41 \tag{13}$$

The safety scale for this segment of the roadway is estimated to be more than two standard deviations below those for other similar roadways, making it a poor segment. This value is independent of any subjective cost estimates, and it allows the engineer to assess this particular site solely with respect to its geometric, operational, and hazard characteristics.

After the addition of the guardrail the lowest safety scale is estimated to be 0.9803 (left column of Table 5 for Station 1 + 50). The relative safety scale of the improved segment of roadway would be

$$z_s = \frac{S - \overline{S}}{\sqrt{1/\text{ADT}}}$$

$$z_s = \frac{0.9803 - 0.9669}{\sqrt{1/2{,}000}}$$

$$z_s = 0.61 \tag{14}$$

The roadway is slightly better than average, so it is performing at least as well as other roadways in the jurisdiction.

This simple example problem has demonstrated the following:

1. Traditional cost-benefit analyses can be performed by using the safety scale.
2. The safety scale and the relative safety scale can be used to evaluate the effectiveness of proposed countermeasures.
3. Use of the relative safety scale provides a means of determining how unsafe a particular site is compared with other similar roadways without the need for resorting to subjective measures like severity indexes, the value of a life or serious injury, and the assumed rate of return.

CONCLUSIONS

The safety assessment method described in this paper is a useful technique for (*a*) ranking problem sites, (*b*) evaluating alternative designs, and (*c*) allocating scarce highway improvement resources. The SafetyAdvisor provides engineers with a quick, reliable, and easy-to-use tool for performing this type of safety assessment. The software tool separates the process of performing safety analysis from the details of the probablistic models. More improved probablistic models can easily be incorporated into the procedure without having to change the source code of the SafetyAdvisor. There are many probablistic models that need to be developed and validated to provide confidence in the SafetyAdvisor's assessments, but these efforts can easily be merged with the existing models by using software tools like the SafetyAdvisor. The SafetyAdvisor establishes a rational methodology for performing safety assessments without the need for having all of the best probablistic models up front before useful computer software can be developed.

ACKNOWLEDGMENT

The portion of this work concerning mathematical modeling was performed as part of an FHWA contract. The author would like to thank Joe Bared and Justin True of the Design Concepts Research Division of FHWA for their insight and comments during the research. Yusef Mohamedshah provided the HSIS data referred to in the example problem.

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Development of Combination Pedestrian-Traffic Bridge Railings

D. LANCE BULLARD, JR., WANDA L. MENGES, AND C. EUGENE BUTH

Two bridge railing designs have been developed for use in urban areas. The railings consist of concrete parapets with metal railings mounted on top of the parapet. The parapets facilitate transfer of post loads into the bridge deck and the metal railing portion permits visibility through the railing. The railings were designed by ultimate-strength methods of analysis. Prototypes of each design were subjected to full-scale crash tests when they were mounted on 8-in. (20.3-cm)-high, 5-ft (1.5-m)-wide sidewalks and when they were mounted flush on simulated bridge decks. Acceptable performance was obtained in all tests.

FHWA's requirement that new bridge railing designs be proven through full-scale crash tests has generated a need to develop proven designs that are acceptable and that meet the diverse needs of individual states. Reported herein is a portion of work done in a recent study to develop new bridge railing and transition designs (*1*). The railing designs are intended for use in urban areas where truck traffic is minimal. Two different, although similar, railing designs were developed (*2,3*). Ultimate-strength methods of analysis were used to design the railings. Prototypes of the railings were subjected to full-scale crash tests specified in the 1989 AASHTO *Guide Specifications for Bridge Railings* (*4*), and acceptable performance was obtained in all tests. One railing design was tested to Performance Level 1, and the other design was tested to Performance Level 2. Both railing designs were crash tested, first in a configuration with a raised sidewalk and again later with a flush roadway approach surface.

DESCRIPTION OF BR27D AND BR27C BRIDGE RAILINGS

BR27D Bridge Railing

The BR27D railing was constructed of two A500 rails (grade B, TS 4 × 3 × 1/4 in.) attached to posts (A500 grade B, TS 4 × 4 × 3/16 × 24 in.) mounted atop an 18.0-in. (0.5-m) reinforced concrete parapet. Longitudinal post spacing was 6.7 ft (2.0 m). The vertical clear space between each of the two rail elements and the lower rail element and the concrete parapet was 8.0 in. (0.2 m). The railing installation was constructed on the bridge deck surface and mounted atop a 5.0-ft (1.5-m)-wide sidewalk with an 8-in. (0.2-m)-high curb at the face of the sidewalk. The length of the bridge railing installations was 100 ft (30.5 m). Detailed elevations of the bridge railings are shown in Figures 1 and 2, and photographs of the completed bridge railing installations are shown in Figure 3.

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BR27C Bridge Railing

The BR27C railing was constructed of rails (A500 grade B, TS 4 × 3 × 1/4 in.) attached to posts (A500 grade B, TS 4 × 4 × 3/16 × 18 in.) mounted atop a 24.0-in. (0.6-m) reinforced concrete parapet. Longitudinal post spacing was 6.7 ft (2.0 m), and the vertical clear space between the parapet and the bottom of the rail was 14.0 in. (0.4 m). The railing installation was constructed on the bridge deck surface and mounted atop a 5.0-ft (1.5-m)-wide sidewalk with an 8-in. (0.2-m)-high curb at the face of the sidewalk. The length of the bridge railing installations was 100.0 ft (30.5 m). Detailed elevations of the bridge railings are shown in Figures 4 and 5. Photographs of the completed bridge railing installations are shown in Figure 6.

DESIGN OF RAILINGS

The BR27D railing was designed to meet Performance Level 1 (PL1) of the 1989 *Guide Specifications for Bridge Railings* (*4*). The design force used for this level was 26 kips (115.6 kN) at 32 in. (0.8 m) above the road surface for installations in which a raised sidewalk was not present. A raised sidewalk serves to lift and partially redirect a vehicle and influences the magnitude and location of the collision force.

Ultimate-strength methods of analysis were used to evaluate the strength of the railing (*5*). For the metal upper portion of the railing, plastic hinge failure mechanisms were evaluated. If the failure mechanism occurs between adjacent posts, plastic hinges would form in the rail elements near midspan and at each adjacent post. The strength of such a mechanism in this railing was computed to be 41.2 kips (183.3 kN). If the failure mechanism extends over two spans of the railing, plastic hinges would form in the rail elements at the central post and at the far ends of adjacent spans. A plastic hinge would also form in the central post. The computed strength for such a mechanism is 26.4 kips (117.4 kN). For a plastic mechanism extending over three spans, the computed strength is 28.9 kips (128.5 kN). The mechanism that would form is the one that gives the lowest strength. For the metal portion of this railing, the computed strength would be 26.4 kips (117.4 kN) at 34 in. (0.9 m) above the top of the sidewalk.

The strength of the concrete parapet portion of the railing was evaluated by the yieldline analysis presented by Hirsch (*5*). The computed strength for load applied at the top of the parapet is 122.4 kips (544.4 kN). A portion of the parapet strength is used to support the metal post [8.9 kips (39.6 kN) for this design].

The combined maximum strength of the parapet and metal railing would be 122.4 minus 8.9 plus 26.4 equals 139.9 kips (622.3 kN) at 21 in. (0.5 m) above the sidewalk. If the parapet were only partially loaded, lower strengths at greater heights would be obtained.

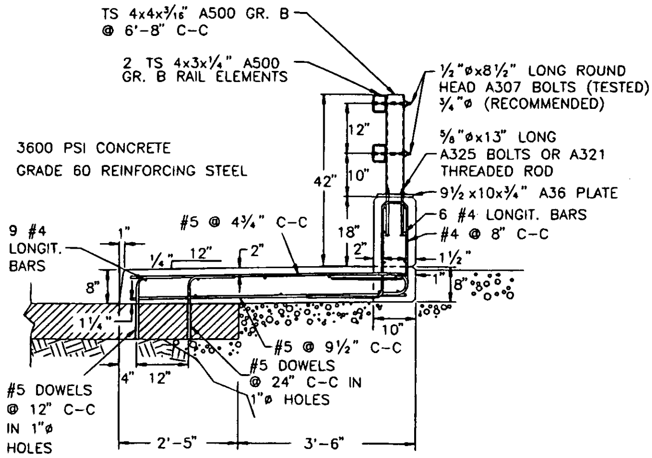


FIGURE 1 Cross section of BR27D bridge railing mounted on sidewalk.

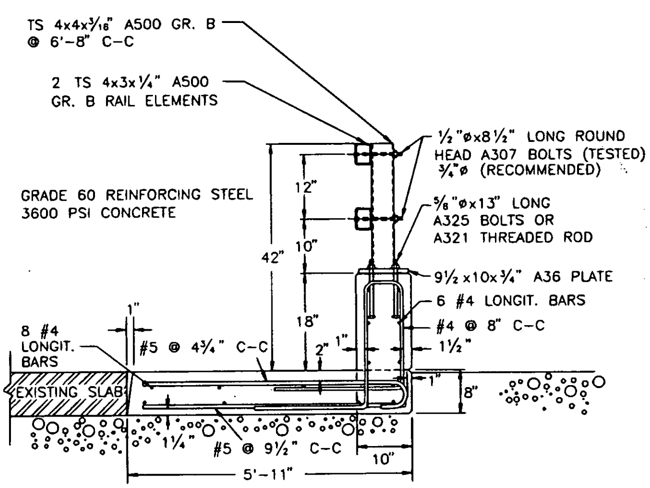


FIGURE 2 Cross section of BR27D mounted flush on deck.

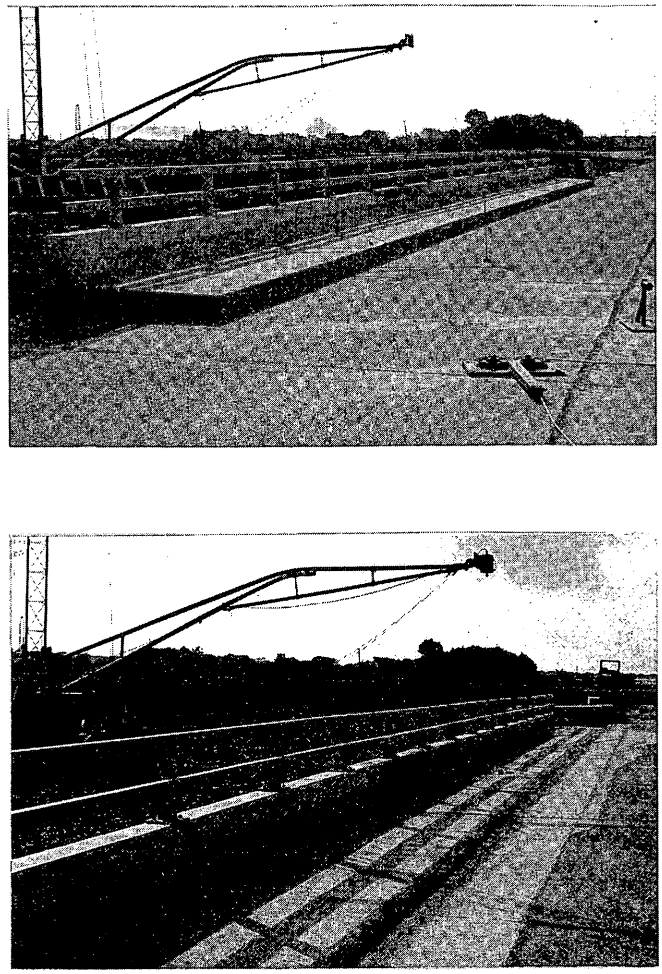


FIGURE 3 BR27D mounted on sidewalk (*top*) and flush on deck (*bottom*).

The BR27C railing was designed to meet PL1 requirements, but it was later tested to Performance Level 2 (PL2) requirements. The design force for the PL2 railings is 56 kips (249.1 kN) at 32 in. (0.8 m) above the road surface for installations in which a raised sidewalk is not present. Ultimate-strength methods of analysis similar to those used for the BR27D railing were used for the BR27C railing. For only the metal railing, a two-span mechanism is the control, and the computed strength is 18.9 kips (84.1 kN) at 40 in. (1.0 m) above the sidewalk. The computed strength of the concrete parapet with force applied at its top edge is 73.3 kips (326.0 kN). The maximum combined strength of the parapet and metal railing is 73.3 minus 10.2 plus 18.9 equals 82 kips (364.7 kN) at 27.7 in. (0.7 m) above the sidewalk. If the parapet were only partially loaded, lower strengths at greater heights would be obtained.

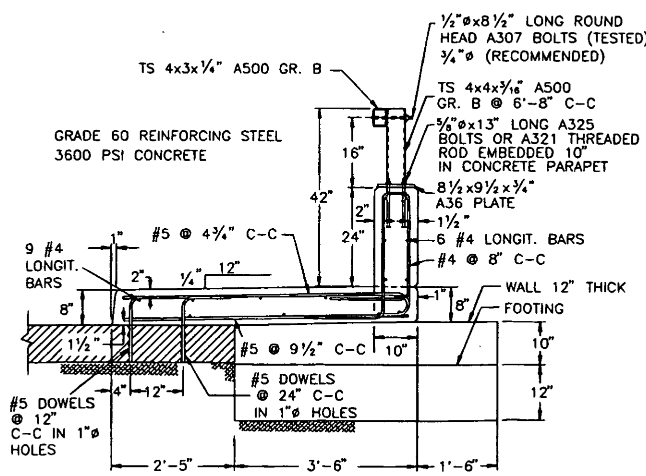


FIGURE 4 Cross section of BR27C mounted on sidewalk.

FULL-SCALE CRASH TESTS

The BR27C and BR27D railings were designed for use in urban areas where truck traffic is minimal. The BR27D railing was tested

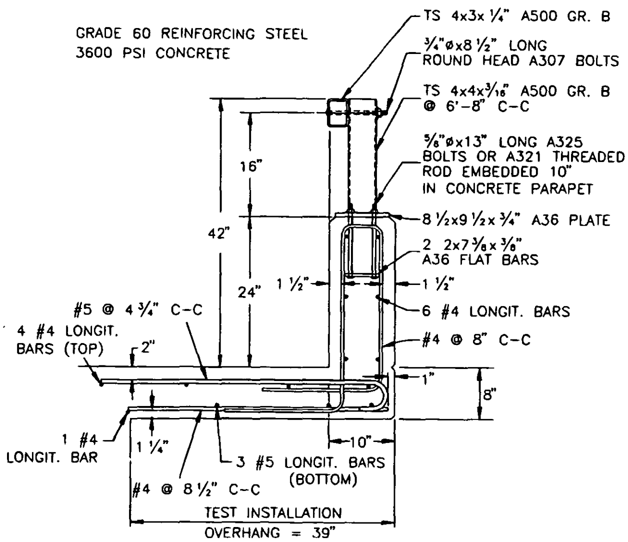


FIGURE 5 Cross section of BR27C mounted flush on deck.

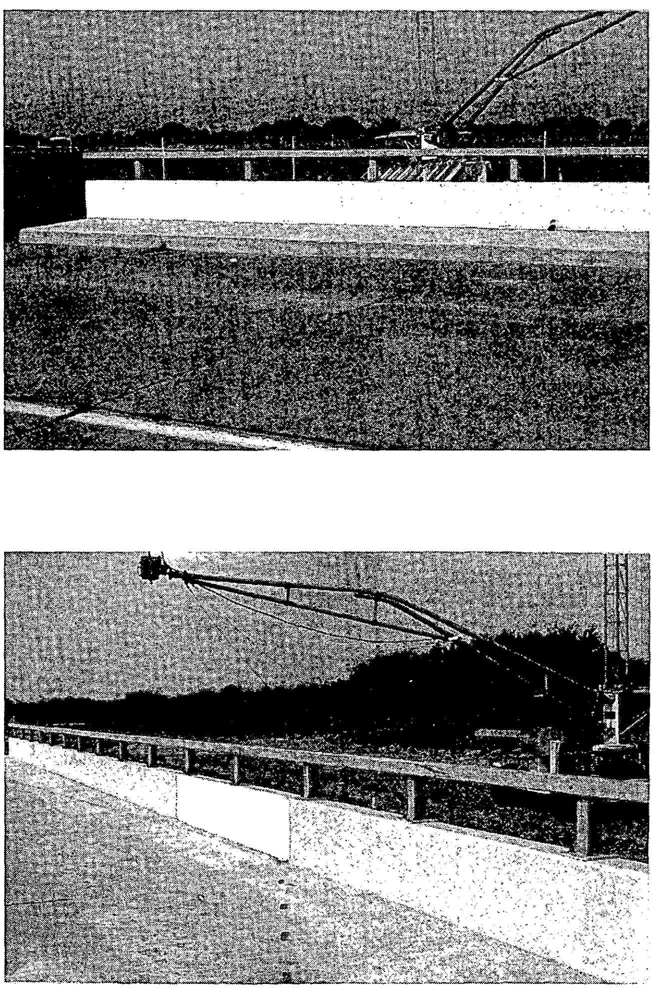


FIGURE 6 BR27C mounted on sidewalk (*top*) and flush on deck (*bottom*).

to PL1 both on the sidewalk (Tests 7069-22 and 7069-23) and on the deck (Tests 7069-30 and 7069-31). The BR27C railing was tested to PL2 both on the sidewalk (Tests 7069-24, 7069-25, and 7069-26) and on the deck (Tests 7069-32, 7069-33, and 7069-34). The sidewalk for both designs was 5 ft (1.5 m) wide, and its face formed an 8-in. (0.2-m)-high curb. All testing was performed in accordance with the test procedures specified in *NCHRP Report 230* (*6*), and the results were evaluated according to the requirements of the AASHTO specifications displayed in Figure 7.

Test Results for BR27D

The BR27D railing designs performed acceptably according to PL1 requirements in both series of tests. Generally, the railing functioned as a "rigid" railing, with only a small amount of permanent deformation in the metal railing in the more severe tests.

BR27D Mounted on Sidewalk

Test 7069-22 Impact with the curb slowed the vehicle to 46.6 mph (75.0 km/hr) and partially redirected the vehicle to 13.4 degrees before it contacted the railing at Post 5. Redirection of the vehicle was relatively smooth, with only minimal intrusion of the bumper between rail elements. There was minimal damage to the bridge railing system, with no measurable permanent deformation to the rail elements. According to the AASHTO specifications for PL1 tests with 1,800-lb (817-kg) vehicles the bridge railing performed acceptably, as shown in Figure 8 and Table 1.

Test 7069-23 As in the first test, impact with the curb partially redirected and slowed the vehicle. The vehicle struck the railing 3 ft from Post 5 (between Posts 4 and 5) traveling at a speed of 43.8 mph (70.5 km/hr) and at an angle of 19.7 degrees. Smooth redirection occurred, with minimal intrusion of the bumper between the lower metal rail element and the concrete parapet. The railing system received minimal damage, and maximum permanent deformation to the rail element was 0.5 in. (13 mm) between Posts 5 and 6. Posts 5 and 6 were displaced rearward approximately ³⁄₁₆ in. (5 mm) at the anchor bolt holes. The railing performed acceptably according to AASHTO requirements for PL1 tests with 5,400-lb (2452-kg) vehicles (Figure 9 and Table 1).

BR27D Mounted Flush on Deck

Test 7069-30 The vehicle struck the railing system approximately 25.5 ft (7.8 m) from the end of the bridge railing. The railing contained and smoothly redirected the vehicle, with no measurable permanent deformation to the rail elements. As shown in Figure 10 and Table 2, the railing performed acceptably according to PL1 requirements.

Test 7069-31 The pickup struck the railing system approximately 1 ft (0.3 m) downstream of Post 5. Redirection of the vehicle was relatively smooth, with no snagging and minimal lateral movement of the rail element. The railing system received minimal dam-

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

Notes:

1. Except as noted, all full-scale tests shall be conducted and reported in accordance with the requirements in NCHRP Report No. 230. In addition, the maximum loads that can be transmitted from the bridge railing to the bridge deck are to be determined from static force measurements or ultimate strength analysis and reported.
2. Permissible tolerances on the test speeds and angles are as follows:

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

   Tests that indicate acceptable railing performance but that exceed the allowable upper tolerances will be accepted.

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

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

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

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

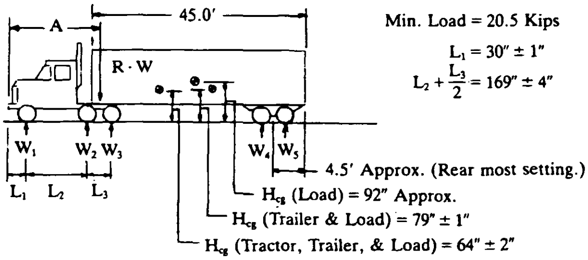
Occupant Impact Velocity–fps	
Longitudinal	Lateral
30	25

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

Occupant Ridedown Acceleration—g's	
Longitudinal	Lateral
15	15

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

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



Min. Load = 20.5 Kips

$L_1 = 30'' \pm 1''$

$L_2 + \frac{L_3}{2} = 169'' \pm 4''$

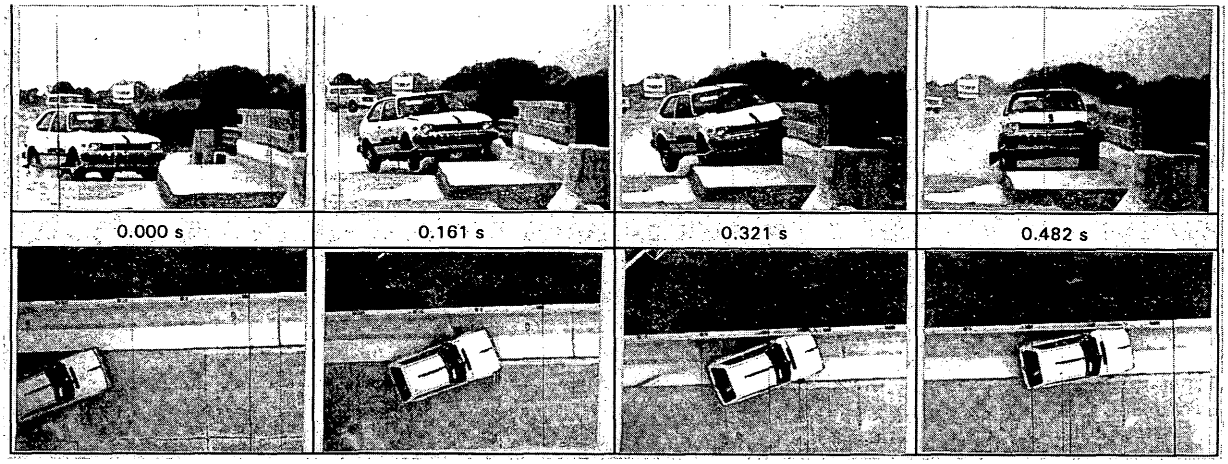
$$A = L_1 + \frac{W_2L_2 + W_3(L_2 + L_3)}{W_1 + W_2 + W_3}$$

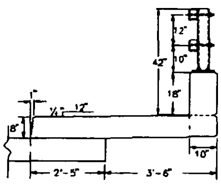
$$R = \frac{W_1 + W_2 + W_3}{W}$$

$W = W_1 + W_2 + W_3 + W_4 + W_5$ = total vehicle weight.

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

FIGURE 7 Bridge railing performance levels and crash test criteria (*4*).





1 in = 25.4 mm
1 ft = 0.305 m

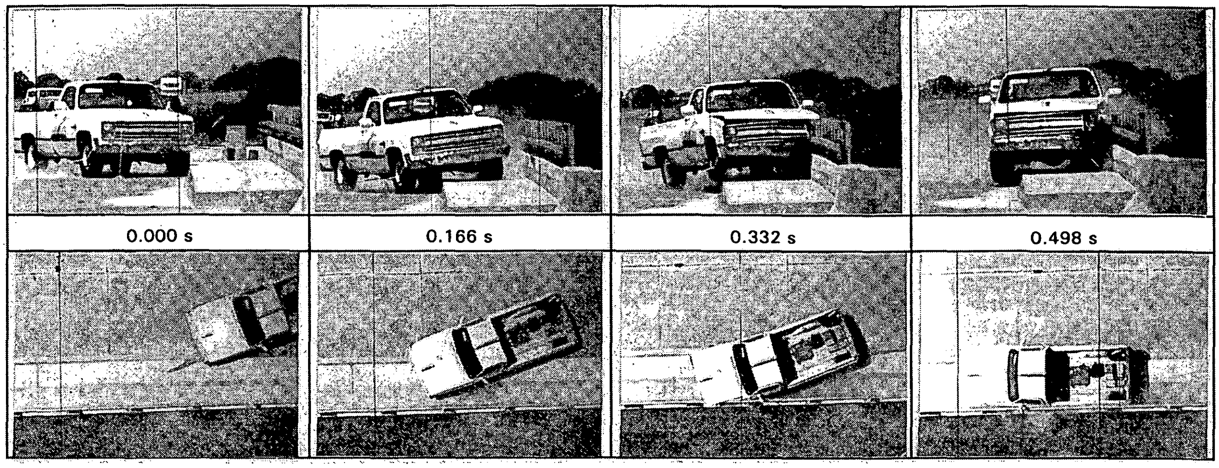
Test No.	7069-22
Date	03/24/92
Test Installation	BR27D Bridge Railing on sidewalk
Installation Length	100 ft (30 m)
Test Vehicle	1983 Honda Civic
Vehicle Weight	
Test Inertia	1800 lb (817 kg)
Gross Static	1967 lb (893 kg)
Vehicle Damage Classification	
TAD	11LFQ3
CDC	11FLEK2 & 11LFES2
Maximum Vehicle Crush	6.0 in (152 mm)

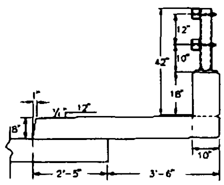
Impact Speed	51.7 mi/h (83.2 km/h)
Impact Angle	20.8 degrees
Speed at Parallel	41.0 mi/h (66.0 km/h)
Exit Speed	40.8 mi/h (65.6 km/h)
Exit Trajectory	6.1 degrees
Vehicle Accelerations (Max 50-ms Avg)	
Longitudinal	-4.4 g
Lateral	-6.8 g
Occ. Impact Velocity	
Longitudinal	12.2 ft/s (3.7 m/s)
Lateral	6.3 ft/s (1.9 m/s)
Occ. Ridedown Acc.	
Longitudinal	-4.7 g
Lateral	-13.3 g

FIGURE 8 Results for Test 7069-22.

TABLE 1 Evaluation of Tests on BR27D Mounted on Sidewalk

EVALUATION CRITERIA	TEST 7069-22	TEST 7069-23	PASS/ FAIL
A. Must contain vehicle	Vehicle contained	Vehicle contained	Pass
B. Debris shall not penetrate occupant compartment	No debris penetrated	No debris penetrated	Pass
C. Occupant compartment must have essentially no deformation	No deformation	No deformation	Pass
D. Vehicle must remain upright	Remained upright	Remained upright	Pass
E. Smooth redirection of vehicle	Relatively smooth redirection	Relatively smooth redirection	Pass
F. Effective coefficient of friction	Marginal	Good	Pass
G. Occupant Impact Velocity (30/25) Occupant Ridedown (15/15)	12.2 ft/s Long 6.3 ft/s Lat -4.7 g Long -13.3 g Lat	13.2 ft/s Long 14.0 ft/s Lat -2.3 g Long -10.6 g Lat	Pass
H. Exit angle less than 12 degrees	Exit angle 6.1 degrees	Exit angle 5.3 degrees	Pass

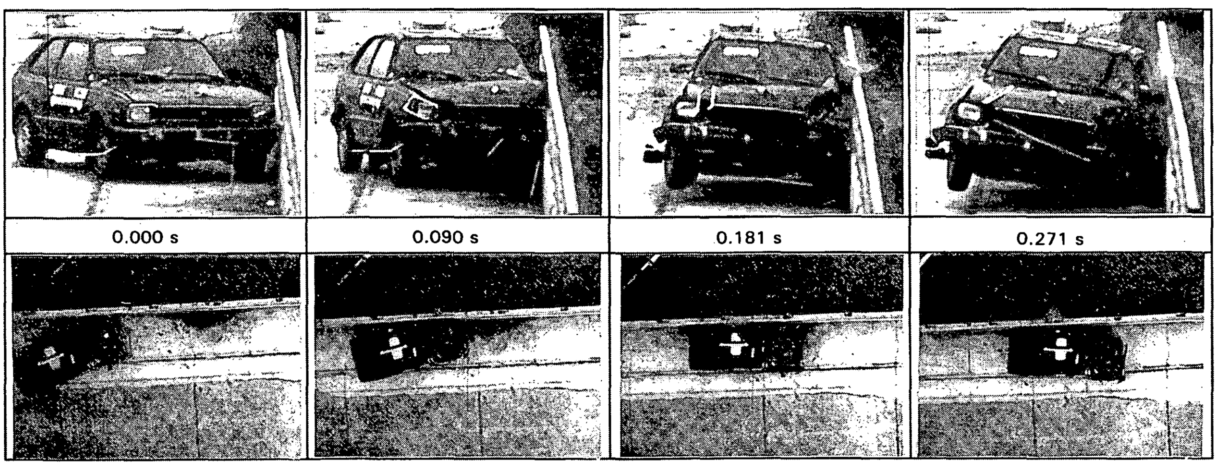


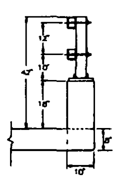


1 in = 25.4 mm
1 ft = 0.305 m

Test No.	7069-23	Impact Speed	45.3 mi/h (72.9 km/h)
Date	03/26/92	Impact Angle	20.2 degrees
		Speed at Parallel	40.3 mi/h (64.8 km/h)
Test Installation	BR27D Bridge Railing on sidewalk	Exit Speed	37.2 mi/h (59.9 km/h)
		Exit Trajectory	5.3 degrees
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1984 Chevrolet Custom Pickup	Longitudinal	-3.7 g
Vehicle Weight		Lateral	-7.8 g
Test Inertia	5400 lb (2452 kg)	Occ. Impact Velocity	
Gross Static	5565 lb (2527 kg)	Longitudinal	13.2 ft/s (4.0 m/s)
Vehicle Damage Classification		Lateral	14.0 ft/s (4.3 m/s)
TAD	11LFQ4 & 11LD4	Occ. Ridedown Acc.	
CDC	11FLEK2 & 11LFEW3	Longitudinal	-2.3 g
Maximum Vehicle Crush	12.5 in (318 mm)	Lateral	-10.6 g

FIGURE 9 **Results for Test 7069-23.**





1 in = 25.4 mm

Test No.	7069-30	Impact Speed	51.2 mi/h (82.4 km/h)
Date	05/19/92	Impact Angle	20.5 degrees
		Speed at Parallel	43.6 mi/h (70.2 km/h)
Test Installation	BR27D Bridge Railing on deck	Exit Speed	43.0 mi/h (69.2 km/h)
		Exit Trajectory	6.8 degrees
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1983 Honda Civic	Longitudinal	-7.5 g
Vehicle Weight		Lateral	-12.8 g
Test Inertia	1800 lb (817 kg)	Occ. Impact Velocity	
Gross Static	1970 lb (894 kg)	Longitudinal	16.0 ft/s (4.9 m/s)
Vehicle Damage Classification		Lateral	21.5 ft/s (6.6 m/s)
TAD	11LFQ3	Occ. Ridedown Acc.	
CDC	11FLEK2 & 11LFES2	Longitudinal	-3.6 g
Maximum Vehicle Crush	7.0 in (178 mm)	Lateral	-6.1 g

FIGURE 10 **Results for Test 7069-30.**

TABLE 2 Evaluation of Tests on BR27D Mounted Flush on Deck

EVALUATION CRITERIA	TEST 7069-30	TEST 7069-31	PASS/ FAIL
A. Must contain vehicle	Vehicle contained	Vehicle contained	Pass
B. Debris shall not penetrate occupant compartment	No debris penetrated	No debris penetrated	Pass
C. Occupant compartment must have essentially no deformation	No deformation	No deformation	Pass
D. Vehicle must remain upright	Remained upright	Remained upright	Pass
E. Smooth redirection of vehicle	Smooth redirection	Relatively smooth redirection	Pass
F. Effective coefficient of friction	Good	Good	Pass
G. Occupant Impact Velocity (30/25) Occupant Ridedown (15/15)	16.0 ft/s Long 21.5 ft/s Lat -3.6 g Long -6.1 g Lat	11.7 ft/s Long 12.3 ft/s Lat 2.2 g Long -8.2 g Lat	Pass
H. Exit angle less than 12 degrees	Exit angle 6.8 degrees	Exit angle 6.2 degrees	Pass

age, with a maximum permanent deformation of 0.5 in. (13 mm) to the metal rail element between Posts 5 and 6. Figure 11 and Table 2 present the results showing that the railing performed acceptably according to the PL1 requirements of the AASHTO specifications.

Test Results for BR27C

After testing of the BR27C railing on sidewalk, two details were changed before testing the BR27C railing mounted flush on deck. The rail-to-post connection bolts were changed from ½ in. (13 mm) in diameter to ¾ in. (19 mm) in diameter, and an anchorage assembly was added at the end of the anchor bolts. These modifications are recommended for both versions of the railing. Both designs of the BR27C railing performed acceptably according to PL2 requirements.

BR27C Mounted on Sidewalk

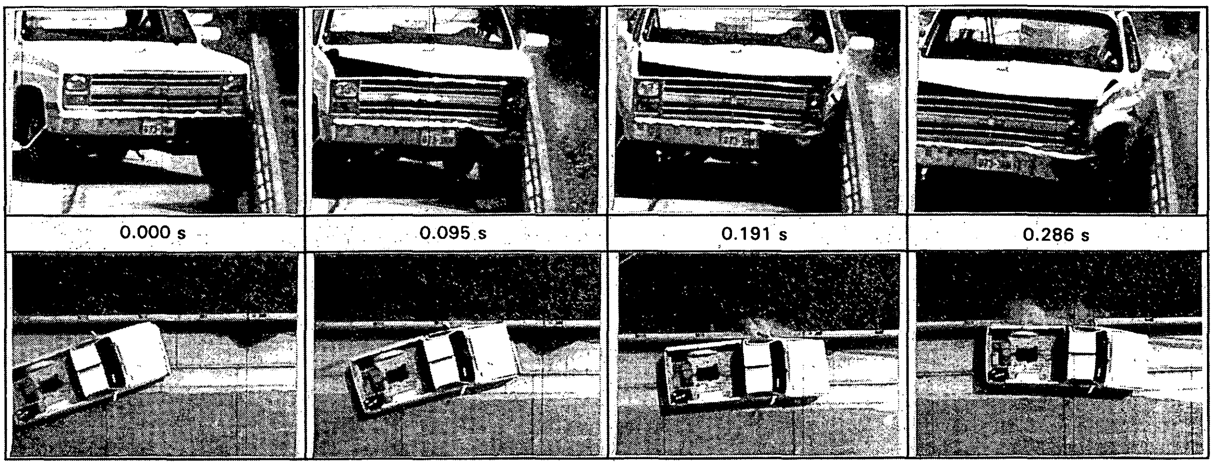
Test 7069-24 Partial redirection and slowing of the vehicle occurred as the vehicle traversed the curb of the sidewalk. The vehicle struck the railing traveling at 55.5 mph (89.3 km/hr) and an angle of 18.1 degrees. Redirection of the vehicle by the railing was relatively smooth. The railing system received minimal damage, with no measurable permanent deformation to the metal rail elements. However, the left corner of the bumper snagged Post 6 (leaving plastic trim), and Posts 5 and 6 were pulled up such that the washers rotated freely under the nuts on the front side of the railing. Although the lateral ridedown acceleration of 17.2 *g* was slightly above AASHTO's recommended 15-*g* limit for the 1,800-lb (817-kg) vehicle, the test was judged acceptable for this category because it was well within the limits of the other three occupant risk factors. See Figure 12 and Table 3 for detailed results.

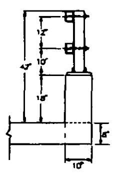
Test 7069-25 Impact with the curb caused minimal redirection and slowing of the vehicle during this test. The vehicle bumper struck the railing near Post 4 at a speed of 59.8 mph (96.2 km/hr) and an angle of 17.9 degrees. Redirection of the vehicle was relatively smooth, with minimal intrusion of the bumper between the concrete parapet and the lower rail element. The railing system received minimal damage, with no measurable permanent deformation to the metal rail elements. However, as in the test with the 1,800-lb (817-kg) vehicle, the left corner of the bumper had snagged Post 5 and pulled it up such that the washer rotated freely under the nut on the left front side of the railing. According to the PL2 limits specified by AASHTO for tests with 5,400-lb (2,452-kg) pickups, the railing performed acceptably. Results are presented in Figure 13 and Table 3.

Test 7069-26 A single-unit truck was used for the third crash test on the BR27C railing on sidewalk. Shortly after impact with the curb the vehicle began a slight counterclockwise yaw and the vehicle bumper struck the railing [3 ft (1 m) downstream of Post 7] traveling at a speed of 47.9 mph (77.1 km/hr) and an angle of 14.4 degrees. During the collision the right front wheel and part of the hub broke loose from the axle, and as the vehicle continued forward the lower edge of the vehicle's cargo box pulled the metal rail off of Posts 10 through 14. The railing system contained the test vehicle with minimal lateral movement of the bridge railing. There was no measurable permanent deformation to the metal rail elements in the immediate impact area; however, the bolts connecting the rail to the posts from Posts 10 through 14 were sheared as a result of vertical load from the cargo box. The railing performed acceptably according to AASHTO PL2 requirements, and results and evaluation are presented in Figure 14 and Table 3.

BR27C Mounted on Deck

Test 7069-32 The vehicle struck the railing system 1.1 ft (0.3 m) downstream from Post 3 [or 17.8 ft (5.4 m) from the end of the bridge railing]. The bridge railing received minimal damage, with no deformation to the metal rail element. There was no intru-

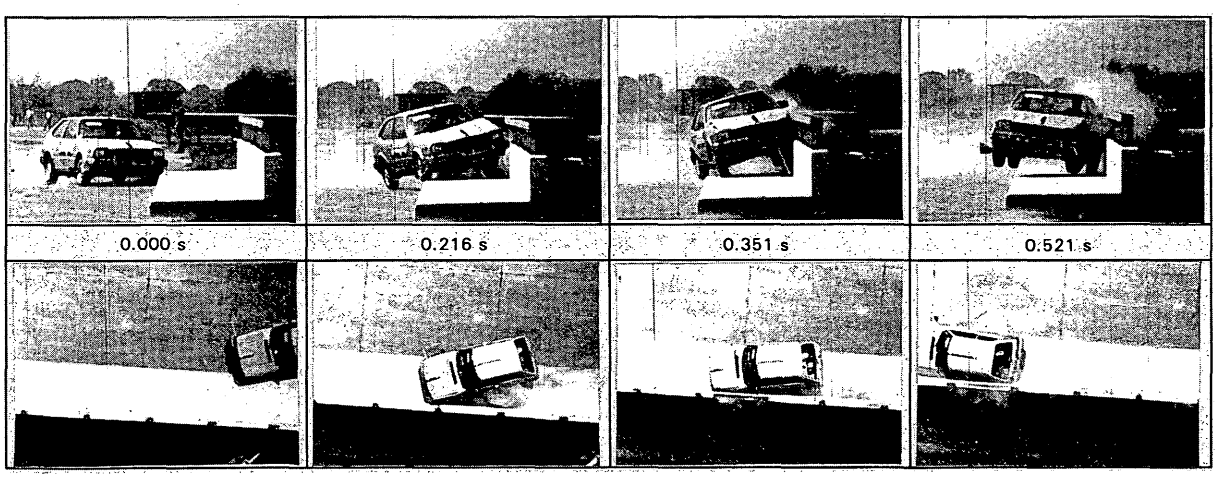


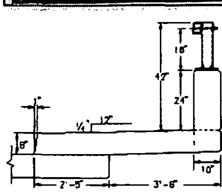


1 in = 25.4 mm

Test No.	7069-31	Impact Speed	45.6 mi/h (73.4 km/h)
Date	05/21/92	Impact Angle	18.8 degrees
		Speed at Parallel	40.8 mi/h (65.6 km/h)
Test Installation	BR27D Bridge Railing on deck	Exit Speed	38.0 mi/h (61.1 km/h)
		Exit Trajectory	6.2 degrees
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1985 Chevrolet Custom Pickup	Longitudinal	-4.1 g
Vehicle Weight		Lateral	-7.5 g
Test Inertia	5400 lb (2452 kg)	Occ. Impact Velocity	
Gross Static	5566 lb (2527 kg)	Longitudinal	11.7 ft/s (3.6 m/s)
Vehicle Damage Classification		Lateral	12.3 ft/s (3.7 m/s)
TAD	11LFQ3 & 11LD2	Occ. Ridedown Acc.	
CDC	11FLEK2 & 11LFEW2	Longitudinal	2.2 g
Maximum Vehicle Crush	6.5 in (165 mm)	Lateral	-8.2 g

FIGURE 11 Results for Test 7069-31.





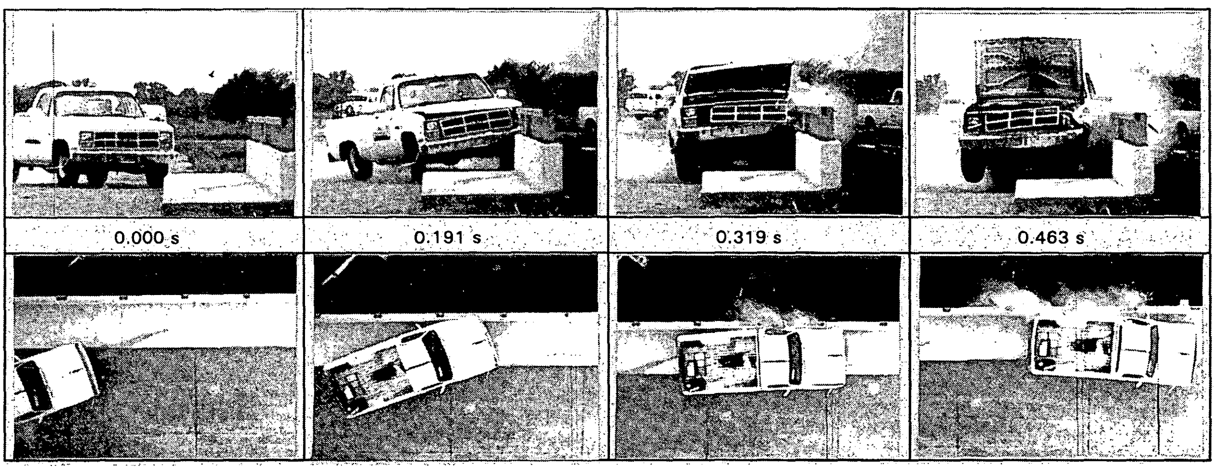
1 in = 25.4 mm
1 ft = 0.305 m

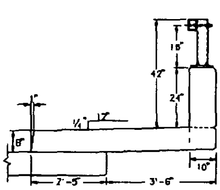
Test No.	7069-24	Impact Speed	61.7 mi/h (99.3 km/h)
Date	03/31/92	Impact Angle	18.7 degrees
		Speed at Parallel	50.9 mi/h (81.9 km/h)
Test Installation	BR27C Bridge Railing on sidewalk	Exit Speed	50.3 mi/h (80.9 km/h)
		Exit Trajectory	1.0 degree
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1982 Honda Civic	Longitudinal	-5.6 g
Vehicle Weight		Lateral	-9.3 g
Test Inertia	1800 lb (817 kg)	Occ. Impact Velocity	
Gross Static	1965 lb (892 kg)	Longitudinal	15.3 ft/s (4.7 m/s)
Vehicle Damage Classification		Lateral	6.5 ft/s (2.0 m/s)
TAD	11LFQ3	Occ. Ridedown Acc.	
CDC	11FLEK2 & 11LFES2	Longitudinal	-3.8 g
Maximum Vehicle Crush	7.5 in (191 mm)	Lateral	-17.2 g

FIGURE 12 Results for Test 7069-24.

TABLE 3 Evaluation of Tests on BR27C Mounted on Sidewalk

EVALUATION CRITERIA	TEST 7069-24	TEST 7069-25	TEST 7069-26	PASS/ FAIL
A. Must contain vehicle	Vehicle contained	Vehicle contained	Vehicle contained	Pass
B. Debris shall not penetrate occupant compartment	No debris penetrated	No debris penetrated	No debris penetrated	Pass
C. Occupant compartment must have essentially no deformation	No deformation	No deformation	No deformation	Pass
D. Vehicle must remain upright	Remained upright	Remained upright	Remained upright	Pass
E. Smooth redirection of vehicle	Relatively smooth redirection	Relatively smooth redirection	Relatively smooth redirection	Pass
F. Effective coefficient of friction	Marginal to good	Good	Marginal to good	Pass
G. Occupant Impact Velocity (30/25) Occupant Ridedown (15/15)	15.3 ft/s Long 6.5 ft/s Lat -3.8 g Long -17.2 g Lat	12.9 ft/s Long 19.9 ft/s Lat -4.4 g Long -10.8 g Lat	8.2 ft/s Long 9.4 ft/s Lat -2.9 g Long -6.9 g Lat	Pass
H. Exit angle less than 12 degrees	Exit angle 1.0 degrees	Exit angle 5.4 degrees	Exit angle 0 degrees	Pass



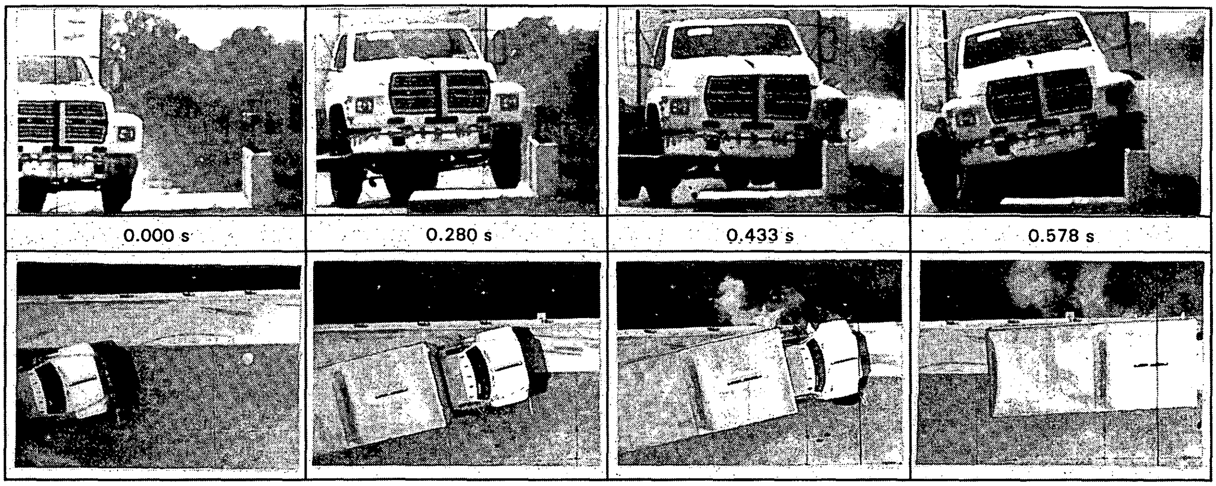


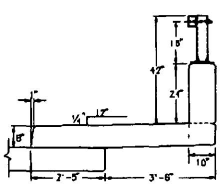
1 in = 25.4 mm
1 ft = 0.305 m

Test No.	7069-25
Date	04/02/92
Test Installation	BR27C Bridge Railing on sidewalk
Installation Length	100 ft (30 m)
Test Vehicle	1984 GMC Sierra Pickup
Vehicle Weight	
Test Inertia	5400 lb (2452 kg)
Gross Static	5568 lb (2528 kg)
Vehicle Damage Classification	
TAD	11LFQ4 & 11LD4
CDC	11FLEK2 & 11LFEW2
Maximum Vehicle Crush	12.0 in (305 mm)

Impact Speed	62.6 mi/h (100.7 km/h)
Impact Angle	19.4 degrees
Speed at Parallel	56.7 mi/h (91.2 km/h)
Exit Speed	53.5 mi/h (86.1 km/h)
Exit Trajectory	5.4 degrees
Vehicle Accelerations (Max 50-ms Avg)	
Longitudinal	-4.6 g
Lateral	-9.3 g
Occ. Impact Velocity	
Longitudinal	12.9 ft/s (3.9 m/s)
Lateral	19.9 ft/s (6.1 m/s)
Occ. Ridedown Acc.	
Longitudinal	-4.4 g
Lateral	-10.8 g

FIGURE 13 Results for Test 7069-25.





1 in = 25.4 mm
1 ft = 0.305 m

Test No.	7069-26	Impact Speed	51.0 mi/h (82.0 km/h)
Date	04/08/92	Impact Angle	13.7 degrees
		Speed at Parallel	44.8 mi/h (72.1 km/h)
Test Installation	BR27C Bridge Railing on sidewalk	Exit Speed	N/A
		Exit Trajectory	0 degree
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1980 Ford Single-Unit Truck	Longitudinal	-1.9 g
		Lateral	-2.9 g
Vehicle Weight		Occ. Impact Velocity	
Empty Weight	10,550 lb (4790 kg)	Longitudinal	8.2 ft/s (2.5 m/s)
Test Inertia	18,000 lb (8172 kg)	Lateral	9.4 ft/s (2.9 m/s)
		Occ. Ridedown Acc.	
Maximum Vehicle Crush	12.0 in (305 mm)	Longitudinal	-2.9 g
		Lateral	-6.9 g

FIGURE 14 Results for Test 7069-26.

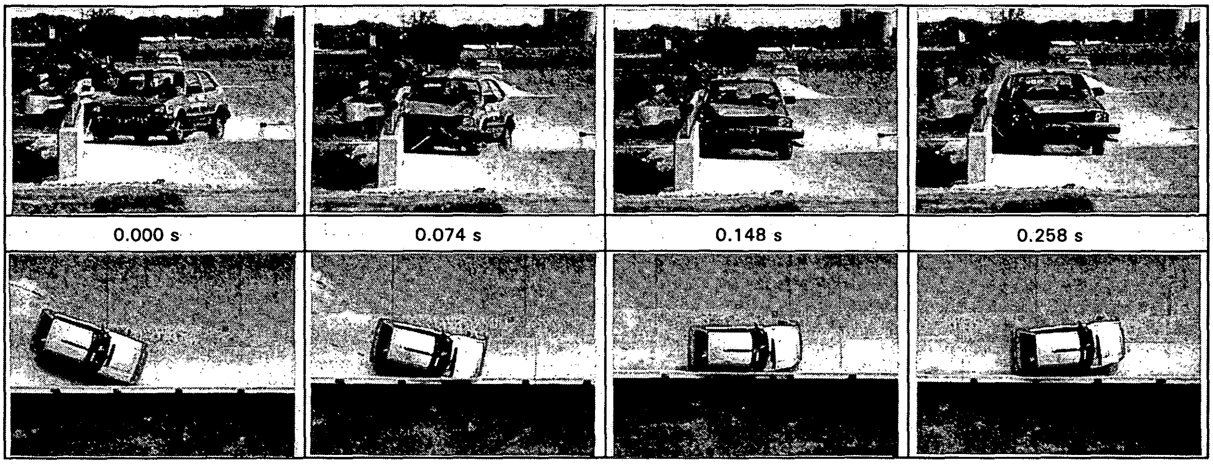
sion of railing components into the occupant compartment, although there was a 1-in. (25-mm) dent into the occupant compartment at the firewall. This deformation into the occupant compartment was deemed as not life-threatening, and therefore the test was judged acceptable for this category. As shown in Figure 15 and Table 4, the railing performed acceptably according to AASHTO PL2 requirements.

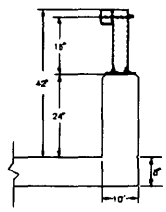
Test 7069-33 The pickup struck the railing 1.9 ft (0.6 m) downstream from Post 3 [or 18.6 ft (5.7 m) from the end of the bridge railing]. Redirection of the vehicle was relatively smooth, with minimal intrusion of the bumper between the parapet and lower metal rail element and slight contact with Post 4. There was 0.5 in. (13 mm) of deformation to the lower metal rail element, and there was a hairline crack in the concrete parapet 17.5 in. (0.4 m) down from Post 3. There was no intrusion of railing components into the occupant compartment, although there was a 0.5-in. (13-mm) dent into the occupant compartment at the firewall. As in the test with the 1,800-lb (817-kg) vehicle, this deformation into the occupant compartment was not considered life-threatening. The railing was judged acceptable according to PL2 requirements, and results and evaluation of the test are shown in Figure 16 and Table 4.

Test 7069-34 A single-unit truck vehicle struck the railing 1.0 ft (0.3 m) downstream from Post 5. As the vehicle struck the railing the bumper rode up the concrete parapet, went between the concrete parapet and lower metal rail element, made contact with Post 6, and then contacted Post 7. The bridge railing received minimal damage, with most being contained within the area around Posts 4, 5, and 6. Cracking occurred in Post 4 and 5 in the heat-affected zone in the post at the post-to-base plate connection. The crack occurred at the corners on the traffic side of the tubular steel element (corner of maximum tensile stress) and extended approximately 1 in. in both directions. There was a hairline crack in the concrete parapet in line with the rear post bolts at Post 4. There was 1.5 in. (38 mm) of deformation to the metal rail element between Posts 4 and 5. As shown in Figure 17 and Table 4, the railing performed acceptably according to the PL2 requirements.

SUMMARY AND CONCLUSION

Two 42-in. (1.1-m)-tall bridge railing designs for use in urban areas were designed and tested. Both designs consisted of concrete parapets with metal railings mounted on top of the parapet. The parapet aids in distributing post loads into the bridge deck and the metal portion of the railing permits visibility through the railing. Ultimate-strength, plastic mechanism methods of analysis were used to design the railings. Prototypes of each railing design were subjected to full-scale crash tests when they were mounted on 8-in. (0.2-m)-high, 5-ft (1.5-m)-wide sidewalks and when they were mounted





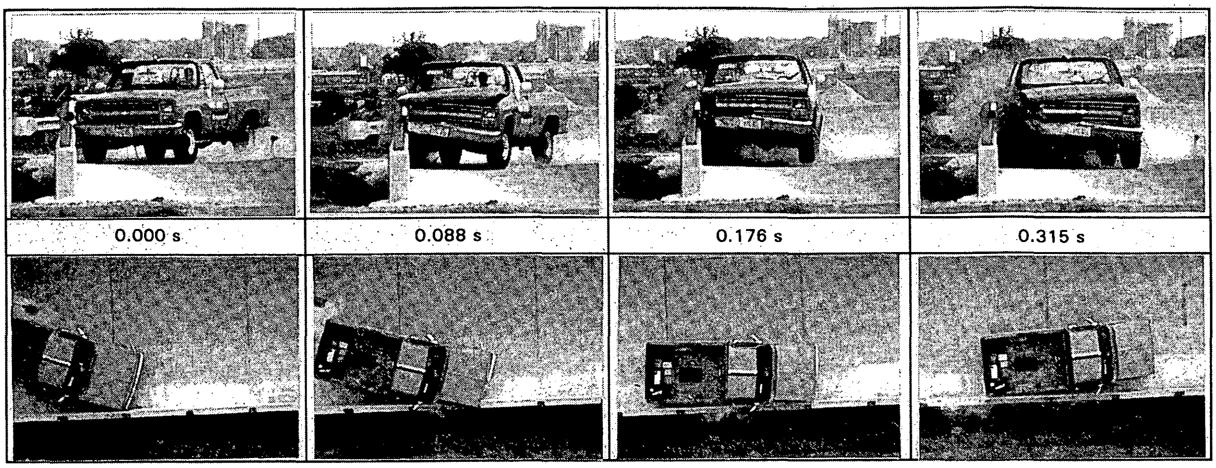
1 in = 25.4 mm

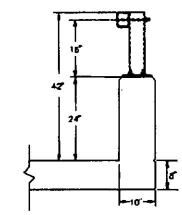
Test No.	7069-32	Impact Speed	60.3 mi/h (97.0 km/h)
Date	07/14/92	Impact Angle	19.8 degrees
		Speed at Parallel	53.6 mi/h (86.2 km/h)
Test Installation	BR27C Bridge Railing on deck	Exit Speed	50.6 mi/h (81.4 km/h)
		Exit Trajectory	6.6 degrees
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1983 Honda Civic	Longitudinal	-5.7 g
Vehicle Weight		Lateral	12.2 g
Test Inertia	1800 lb (817 kg)	Occ. Impact Velocity	
Gross Static	1970 lb (894 kg)	Longitudinal	14.5 ft/s (4.4 m/s)
Vehicle Damage Classification		Lateral	24.6 ft/s (7.5 m/s)
TAD	01RFQ5	Occ. Ridedown Acc.	
CDC	01FREK3 & 01RYEW4	Longitudinal	-1.2 g
Maximum Vehicle Crush	6.5 in (165 mm)	Lateral	12.7 g

FIGURE 15 Results for Test 7069-32.

TABLE 4 Evaluation of Tests on BR27C Mounted Flush on Deck

EVALUATION CRITERIA	TEST 7069-32	TEST 7069-33	TEST 7069-34	PASS/ FAIL
A. Must contain vehicle	Vehicle contained	Vehicle contained	Vehicle contained	Pass
B. Debris shall not penetrate occupant compartment	No debris penetrated	No debris penetrated	No debris penetrated	Pass
C. Occupant compartment must have essentially no deformation	Minimal deformation (1 in)	Minimal deformation (0.5 in)	No deformation	Pass
D. Vehicle must remain upright	Remained upright	Remained upright	Remained upright during test period	Pass
E. Smooth redirection of vehicle	Relatively smooth redirection	Relatively smooth redirection	Relatively smooth redirection	Pass
F. Effective coefficient of friction	Good	Good	Marginal	Pass
G. Occupant Impact Velocity (30/25) Occupant Ridedown (15/15)	14.5 ft/s Long 24.6 ft/s Lat -1.2 g Long -12.7 g Lat	11.6 ft/s Long 20.1 ft/s Lat -2.2 g Long 8.1 g Lat	8.2 ft/s Long 13.1 ft/s Lat -1.1 g Long 4.3 g Lat	Pass
H. Exit angle less than 12 degrees	Exit angle 6.6 degrees	Exit angle 6.5 degrees	Exit angle 3.5 degrees	Pass

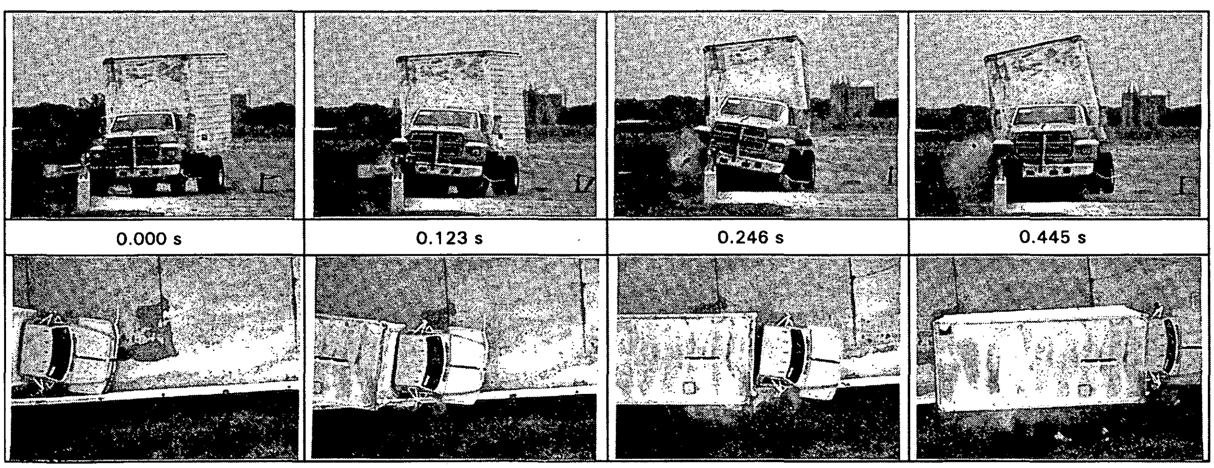


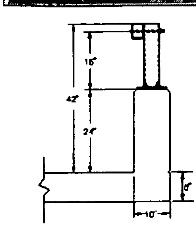


1 in = 25.4 mm

Test No.	7069-33	Impact Speed	55.3 mi/h (89.0 km/h)
Date	07/16/92	Impact Angle	19.6 degrees
		Speed at Parallel	47.9 mi/h (77.1 km/h)
Test Installation	BR27C Bridge Railing on deck	Exit Speed	44.8 mi/h (72.1 km/h)
		Exit Trajectory	6.5 degrees
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1985 Chevrolet Pickup	Longitudinal	-4.9 g
Vehicle Weight		Lateral	9.3 g
Test Inertia	5400 lb (2452 kg)	Occ. Impact Velocity	
Gross Static	5570 lb (2529 kg)	Longitudinal	11.6 ft/s (3.5 m/s)
Vehicle Damage Classification		Lateral	20.1 ft/s (6.1 m/s)
TAD	01RFQ4 & 01RD2	Occ. Ridedown Acc.	
CDC	01FREK2 & 01RDEW2	Longitudinal	-2.2 g
Maximum Vehicle Crush	9.0 in (229 mm)	Lateral	8.1 g

FIGURE 16 Results for Test 7069-33.





1 in = 25.4 mm

Test No.	7069-34	Impact Speed	52.5 mi/h (84.5 km/h)
Date	08/12/92	Impact Angle	12.8 degrees
		Speed at Parallel	46.8 mi/h (75.3 km/h)
Test Installation	BR27C Bridge Railing on deck	Exit Speed	44.6 mi/h (71.8 km/h)
		Exit Trajectory	3.5 degrees
Installation Length	100 ft (30 m)	Vehicle Accelerations (Max 50-ms Avg)	
Test Vehicle	1981 Ford Single-Unit Truck	Longitudinal	-1.9 g
		Lateral	4.3 g
Vehicle Weight		Occ. Impact Velocity	
Test Inertia	10,490 lb (4762 kg)	Longitudinal	8.2 ft/s (2.5 m/s)
Gross Static	18,000 lb (8172 kg)	Lateral	13.1 ft/s (4.0 m/s)
Maximum Vehicle Crush	9.0 in (229 mm)	Occ. Ridedown Acc.	
		Longitudinal	-1.1 g
Max. Perm. Rail Deform.	1.5 in (38 mm)	Lateral	5.2 g

FIGURE 17 Results for Test 7069-34.

flush on a simulated bridge deck. Design BR27D was tested to PL1 requirements of the 1989 AASHTO *Guide Specifications for Bridge Railings* (*4*), and BR27C was tested to PL2. Acceptable performances were obtained in all tests.

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Concerns About Use of Severity Indexes in Roadside Safety Evaluations

J. W. HALL, D. S. TURNER, AND L. E. HALL

Severity indexes, which serve as indicators of the expected injury consequences of a crash, are an integral part of the analysis of proposed roadside safety improvements. Although research since the 1960s has sought to quantify severity indexes for a range of object types and impact conditions, wide variations remain in the values from which analysts may choose when performing cost-effectiveness evaluations. To clarify the current state of the practice in understanding and using severity indexes, a survey of state highway agencies was conducted. Among the 11 primary parameters used in the AASHTO roadside safety analysis model, respondents expressed the least level of confidence in severity indexes; indeed, more than 70 percent indicated that they encountered problems in selecting and justifying these values. Numerous respondents asserted a need for the validation of the severity indexes used in the model. General support was expressed for the inclusion of more object types and impact conditions in tabulations of severity indexes, although opinions were divided on the merits of providing a range of severity indexes as opposed to specific values. Survey results also supported the need for continued development of the roadside safety method, better documentation of the procedures, user-friendly computer programs, and additional training.

During the past 30 years significant progress has been made in reducing the number of highway fatalities that occur in run-off-the-road accidents. Improvements are most evident on Interstate freeways, where obstacle-free roadsides and the judicious use of barrier systems provide a restrained motorist in an errant vehicle a good chance of surviving an excursion onto the roadside. Similar treatments have been effective on arterial, collector, and even local roads, but the expense of implementing corrective action has limited the extent of improvements on these facilities.

As part of the economic evaluation of alternative roadside safety improvements, the analyst compares the incremental benefits resulting from a treatment with the additional costs required to build and maintain it. In these cases the expected benefits arise from a reduction in the frequency or severity of collisions with roadside obstacles. A critical element in the projection of benefits is the severity of those crashes that are expected to occur with and without a particular treatment. These benefits are currently estimated in a multistep process that relies in part on severity indexes.

Alternative definitions have been suggested, but most early researchers defined severity indexes on a scale of 0 to 1; for specific objects the severity index represented the proportion of reported accidents that resulted in a fatality or injury. Although there were points of agreement, results from studies often differed, possibly because of variations in object design and placement, impact speed, vehicle characteristics, and similar factors. By the mid-1970s a refined procedure and an enlarged scale of 0 (no damage) to 10 (fatality) were used to describe severity. Some indexes were based more on professional judgment and expert opinion than on the results of accident studies and were inherently difficult or impossible to validate by traditional methods. During the past 15 years, serious efforts have been made to develop justifiable severity indexes by both traditional and innovative techniques, including expert opinion, analyses of large accident data bases, in-depth studies of particular objects, evaluation of vehicle damage, application of accident cost models, simulation, and the results of crash testing. In most cases these studies have increased the level of understanding of severity indexes, although the perplexing variations in values recommended by different studies have not been eliminated. The development of severity indexes continues today with a number of ongoing initiatives that may help clarify some of the long-standing concerns.

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The evolution of severity indexes is partially evidenced by a comparison of the values for a sample of objects from a 1974 NCHRP report (*1*) with 1991 values given by FHWA (*2*). The older values, calculated on a scale of from 0 to 1, represented the average proportion of reported accidents that resulted in a fatality or injury. The more recent data from FHWA are expressed on a scale of from 0 to 10, but they are based more on judgment than on actual accident data. They also include a much greater range of object types and impact speeds. A sample of severity indexes from the NCHRP report is compared with similar objects evaluated by FHWA for a 97-km/hr (60-mph) design in Table 1. Although the values clearly differ, the general pattern of more severe objects remains relatively consistent.

Despite continual improvements in severity indexes during the past three decades, inconsistencies and difficulties remain. Questions exist about many factors such as the roles of impact angle and speed, whether accident data can yield accurate severity indexes, whether average severity indexes are appropriate for circumstances of individual accidents, and whether users have an adequate understanding of such indexes. Highway safety managers are aware of these problems, and major efforts are under way by FHWA and NCHRP to improve severity indexes. One such NCHRP project was conducted by the authors to prepare a report on severity indexes.

Throughout the history of severity indexes, it has been assumed that roadside safety analysts understood the concept and possessed sufficient judgment to choose appropriate severity indexes for cost-effectiveness determinations. Unfortunately, this has not always been the case, since highway agency safety analysts, public works managers, and others have not always kept abreast of the relevant technical developments. By the early 1990s FHWA had invested extensive efforts in the development and promulgation of severity indexes, but the degree of understanding among users and the extent of use for off-road accident analyses varied considerably.

TABLE 1 Comparison of Severity Indexes, 1974 Versus 1991

Object	NCHRP 148[a]	FHWA[b]
Sign Support		
Breakaway	0.22	1.7
Rigid (steel)	0.53	5.3
Luminaire Support		
Breakaway	0.22	2.8
Rigid	0.53	5.5
Guardrail Face	0.33	3.6
Tree (medium-size)	0.50	5.5
Embankment		
6:1 slope	0.22	2.6
3:1 slope	0.53	4.0
Utility Pole	0.53	5.5
Bridge Pier	0.70	5.5

[a] Represents the portion of accidents resulting in a fatality or injury.
[b] Represents the average severity, on a scale of 0-10, for 97 km/h (60 mph) design.

PURPOSE OF THIS PAPER

The understanding and use of severity indexes by design and safety personnel are examined in this paper. It is based on a survey conducted as part of an NCHRP project. A questionnaire distributed to state highway and transportation departments identified several areas in which these agencies were experiencing difficulty in evaluating alternative roadside safety improvements. Several findings from this survey could affect future research on severity indexes and roadside safety cost-effectiveness procedures.

SURVEY OF STATE HIGHWAY AGENCIES

The technical literature, supplemented by information from recent telephone interviews with recognized experts in the applied and research communities, confirms that numerous research teams have examined various aspects of the severity index issue. The most recent AASHTO standards for roadside safety design (*3*) incorporate inputs from multiple contributors and provide a limited set of severity indexes as a function of speed, object type, and impact point. However, qualified observers have expressed concern regarding the validity of the severity indexes cited in current AASHTO procedures and have noted the sensitivity of the economic analyses of roadside safety improvements to rather small changes in assumed severity indexes, especially at the upper end of the severity scale. The expanded level of detail in the more recent supplemental information on severity indexes (*2*) may have partially offset these concerns, although interviews with severity index users conducted as part of this study provide ample evidence that neither researchers nor practitioners are comfortable with the current values.

In an effort to determine if and how the individual state highway and transportation departments had resolved their concerns, a survey was developed and distributed to safety and traffic engineers in these agencies. The survey, which was intentionally kept short to encourage responses, was distributed in August 1992. It was sent to prominent, upper-level highway and traffic engineers at each agency, and those who had not responded were recontacted in October. Overall, individuals representing 38 states (76 percent) responded to the survey; although input from the remaining states would have been welcome, there is no reason to believe that additional responses would have affected the primary findings from the survey. In some cases the original recipient of the survey passed it along to others in the organization who worked more closely with the day-to-day task of assessing roadside safety. These people may be in a better position to address the technical issues raised by the survey, although they may lack the background to respond to policy issues. In about 10 cases answers given in the survey required further clarification; respondents were contacted by telephone and were asked to expand on their replies.

The following sections indicate the questions presented on the survey and summarize the responses. Not every respondent provided a reply to each question, so responses do not always total to 38.

Resources for Roadside Safety Analysis

The respondents were first asked what resources the "agency routinely use(s) to assist with roadside safety analyses." The 1989 AASHTO *Roadside Design Guide* (*3*) was reportedly the most widely used resource, with 32 (84 percent) of the respondents indicating that they use it. In addition, respondents from 13 states (34 percent) reported that they used other technical references, including the 1977 AASHTO Barrier Guide (*4*), the *Supplemental Information for Use with the ROADSIDE Computer Program* (*2*), and locally developed design or traffic engineering manuals.

The ROADSIDE computer program was cited as a resource by 15 (39 percent) respondents. This statistic probably overstates the program's use, however, since many of the affirmative responses were accompanied by qualifiers such as "occasionally," "not routinely," or "optional." The limited use of the ROADSIDE computer program is somewhat surprising, since it clearly simplifies the computational aspects, especially when multiple alternatives are being considered. Twelve states use other computer software in their roadside safety analyses. On the basis of comments provided by the respondents and several follow-up telephone interviews, many of the software packages were developed in-house to satisfy particular conditions. For example, some were developed to select projects for the federal-aid safety program. Other agencies reported using specialized software to analyze accident records, and two used special software to calculate the length of need for guardrails.

Several survey responses offered alternative methods for the identification of problem locations and the development of corrective actions. These are typically designed to reflect local characteristics. For example, Indiana has developed its own *Roadside Design Guide* (*5*), combining elements of AASHTO's publication (*3*), Indiana Department of Transportation (DOT) clear zone policy, and the severity indexes in FHWA's supplemental information (*2*). However, Indiana DOT used existing data and made several assumptions to estimate the severity indexes for certain proprietary guardrail end treatments. The Indiana guide will be limited to applications for new, non-Interstate construction and resurfacing, restoration, and rehabilitation (RRR) work.

Nevada DOT developed and uses a personal computer Basic program called Potential, which calculates hazard indexes for roadside features (*6*). The program helps Nevada DOT perform "what-if" analyses for proposed treatments based on the design and operating features of the road. Nevada does not use the ROADSIDE computer program but rather relies on AASHTO's 1977 Barrier Guide (*4*).

Parameter Selection

Survey respondents who indicated that they use the *Roadside Design Guide* (*3*) or ROADSIDE software (*2*) in conducting evaluations and making decisions regarding roadside safety were asked if they "have problems in selecting or justifying values for 11 parameters" necessary for applying these procedures. This question was potentially the most fruitful in the survey, since it addresses the serviceability of the most commonly used resources for evaluation and decision making related to roadside safety.

The items enumerated in this question represent the minimum data requirements (or assumptions) an analyst needs to conduct roadside safety evaluations using the AASHTO methodology. Several of the data parameters (e.g., roadway gradient and traffic volume) are clearly within the purview of the highway agency; if the agency does not have the information, it cannot expect to find it in secondary sources. On the other hand there are few highway agencies that routinely develop several other parameters (e.g., encroachment rates and angles) required in the model. Regardless of the source of the information, the question sought to establish the ease with which the respondent could obtain justifiable values for these parameters and the respondent's confidence in the selected values.

Of the respondents from 33 agencies that reported using either the *Roadside Design Guide* (*3*) or the ROADSIDE computer program (*2*), between 28 and 31 rated each of the parameters; the last two columns in Table 2 indicate the percentage of respondents encountering difficulty in selecting or justifying parameter values while performing roadside safety analyses.

As expected, the responses reflect a high level of confidence in the site-specific parameters such as roadway alignment, traffic volume, and object dimensions and placement. This was not true for other types of parameters. As suggested by Figure 1, about 40 percent of the respondents encounter problems in establishing the encroachment rate and lateral extent either often or occasionally. More than half experienced difficulty with values for the angle of encroachment and the cost of a single-vehicle run-off-road accident. Figure 1 clearly demonstrates that highway agencies have the least degree of confidence in severity indexes for fixed-object impacts. More than 70 percent of the respondents report difficulty in selecting and justifying these values. The severity index and accident costs, two of the most problematic parameters, are believed by most knowledgeable analysts to have the most significant effect on the outcome of a roadside safety analysis.

Solutions

Respondents citing problems with quantifying key parameters were then asked "what would be required to reduce or eliminate these problems and improve your confidence in assessing the safety effectiveness of roadside improvements." The 24 states that responded to this question offered a variety of suggestions, but by far the most common, given by 12 (50 percent) of those responding, dealt with severity indexes. Representative comments called for a "well-documented set of severity indexes" for a "wider variety of objects." Some respondents expressed their general frustration with the apparent subjectivity of the values presented in the *Roadside Design Guide* (*3*) by calling for better field data, not only for severity indexes but also for encroachment parameters and accident costs.

Two respondents expressed dissatisfaction with the substantial amount of engineering judgment required by the current methods of roadside safety analysis. Conversely, others believed that the rigidity of the *Roadside Design Guide* (*3*) and ROADSIDE software (*2*) stifled their exercise of engineering judgment. "Informed engineering judgment" is a fundamental component of the profession. What differentiates the informed, educated opinion of an engineer from the guess of a typical citizen is a readily available, credible, and comprehensive set of evidence that the engineer can apply to the problem at hand. Survey responses indicate that many engineers believe that this necessary informational base is missing or inadequate in the case of severity indexes.

Questions of this type permitted the responding engineers to mention issues that have created recent difficulties for them. Isolated points mentioned by one or two persons who make significant use of the AASHTO procedures could reflect real problems and might lend themselves to simple correction. Responses in this category include:

- Clarify the proper use of design versus operating speed,
- Provide for traffic volumes greater than 20,000,
- Give more information on encroachment angle-runout length relationship, and
- Include data on barrier repair costs.

Severity Indexes

Appendix A of the AASHTO *Roadside Design Guide* (*3*) provides a limited set of severity indexes as a function of object type, impact

TABLE 2 Respondents Experiencing Problems in Selecting Parameters

Encounter Problems With:	Rarely	Occasionally	Often
Design Traffic Volume	90%	7%	3%
Roadway Curvature	90%	3%	7%
Roadway Gradient	93%	0%	7%
Design Speed	77%	19%	3%
Baseline Encroachment Rate	61%	21%	18%
Encroachment Angle	45%	28%	28%
Hazard Offset	79%	21%	0%
Dimensions of the Hazard	83%	17%	0%
Lateral Extent of Encroachment	62%	28%	10%
Severity Indices	27%	30%	43%
Expected Accident Costs	47%	23%	30%

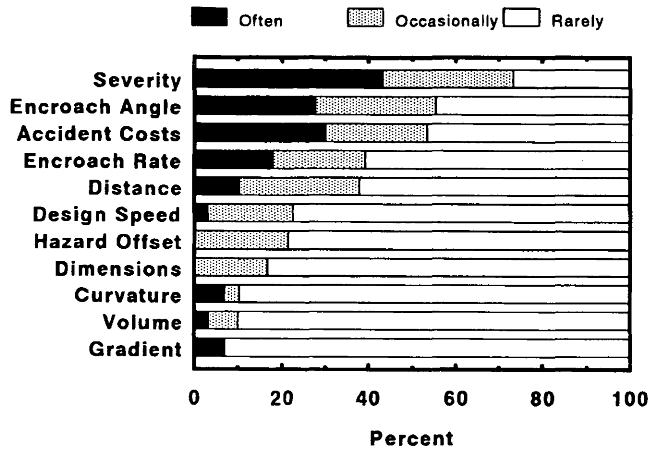

FIGURE 1 Respondents encountering problems in roadside safety analysis.

location, and design speed. The survey asked whether the respondents use this information, whether they are highly confident in the information, and whether they have developed alternate information in which they have greater confidence.

In contrast to the respondents from 33 states that had previously indicated that they used the *Roadside Design Guide* (*3*) or ROADSIDE software (*2*), only 15 of the 36 respondents (42 percent) to this question claimed to use the severity index information in Appendix A of the *Roadside Design Guide*. The states have little confidence in the quality of the severity index information; only 5 of the 27 respondents (19 percent) indicated a high degree of confidence.

Only 23 percent of the respondents indicated that they had developed information on severity indexes that they believed was more reliable than the *Roadside Design Guide*. On the basis of supplemental comments provided by the states, it appears that most had not actually developed alternate values for severity indexes; rather, they were aware of or were using alternate severity indexes, such as those contained in FHWA's supplemental information.

Pennsylvania was the only responding state to describe internal efforts to develop alternative severity index information. Pennsylvania DOT uses its accident record system together with assumed unit costs of crashes (ranging from $1,994 for property damage only to $1,259,544 for a fatal crash) to estimate the average cost of impacts with nine different object types in both urban and rural areas. Although the unit costs have not been revised recently, average crash costs are updated annually to reflect the actual severities of reported crashes. The resultant costs, which serve as surrogates for severity, were formerly used in benefit-cost analyses; Pennsylvania DOT only implemented treatments with a benefit-cost ratio of >2. Pennsylvania now emphasizes safety improvements along corridors according to problem locations identified by cluster parameters [e.g., five hit tree accidents per 0.3 km (0.2 mi) per year]; corridors with multiple accident clusters are reviewed in the field by safety engineers.

Desired Revisions

Anticipating that there might be some level of dissatisfaction with existing severity indexes, the survey asked "what specific revisions are needed to make fixed-object severity indexes more useful in the analysis of roadside safety improvements." Although the responses to this question were highly varied, the central theme of the most common request was for severity indexes to encompass a more extensive set of objects. Several respondents mentioned specific objects, including trees by diameter, different barrier designs, and the combined effects of embankment height and slope.

Related topics of interest included more information on severity indexes as a function of speed, angle of impact, and the roadside slope between the traveled way and a rigid object. Several respondents volunteered that the greater number of objects included in the FHWA supplemental information was quite helpful, although it still had significant gaps.

The lack of severity index credibility evident in replies to the previous question was also obvious in these responses, in which the need for reliable, justifiable values that reflect real-world conditions was mentioned by several respondents. The concept of using a more scientific approach to determining severity indexes and carefully explaining the process and the results to the end user was also recommended by several respondents. Numerous individuals believed that the whole methodology in the *Roadside Design Guide* (*3*) needs to be better explained.

Divergent opinions were offered on the issue of providing discrete severity indexes versus a range of values. One respondent argued convincingly that the presentation of single severity indexes, as in Table A.3 of the *Roadside Design Guide* (*3*), gives a designer the false impression that severity indexes are absolute. Another respondent contends that ranges of values, as given in FHWA's supplemental information, create an undue burden for the typical user who has insufficient expertise to make a choice among the severity indexes in a range. These differences of opinion are simply diverse perspectives on how well (or poorly) the AASHTO guidelines accommodate "informed engineering judgment."

Four additional recommendations were offered by several respondents. These suggestions appear to deserve consideration in any effort to enhance either the roadside safety analysis procedures or parameters.

- The existing process is too vague. Two competent engineers using the AASHTO methodology and the FHWA supplemental information to evaluate a particular situation can arrive at dramatically different results.
- The *Roadside Design Guide* (*3*) should be clearer on the proper method for evaluating multiple roadside obstacles at a location.
- Application of the roadside safety evaluation procedures over an extended section of highway is extremely time-consuming.
- A tabulation of cost-effective treatments as a function of design speeds would be a useful addition.

Alternative Tabulations

The survey asked if respondents are "aware of any severity index tabulations for roadside obstacles that could be used to supplement or corroborate those presented in the AASHTO *Roadside Design Guide*" (*3*). Those responding affirmatively were asked to indicate the source of the information.

Of the 37 responses to this question, 13 (35 percent) indicated an awareness of supplemental severity index information. Most of these identified FHWA's *Supplemental Information for Use with the ROADSIDE Computer Program* (*2*), but it was clear that some respondents were not aware of the most recent version of this document. Other respondents mentioned a computer program developed by the University of Kansas, some research results from Vanderbilt University, the New York DOT accident reduction factors, and AASHTO's 1977 Barrier Guide (*4*). One state noted that its own accident records included cost and casualty information that could be used for this purpose.

Ongoing Studies

Information was solicited about any ongoing projects or studies that are attempting to improve the understanding, usefulness, or quality of roadside severity index information. Only three of the respondents indicated an awareness of such activities; they referred to some research at Vanderbilt University and NCHRP Projects 22-8 and 22-9.

Carney is directing research at Vanderbilt University toward the use of comprehensive federal traffic accident data systems. Data from the Fatal Accident Reporting System (FARS) and the National Accident Sampling System (NASS) are being used for these efforts. The FARS and NASS data are being subjected to multiple statistical treatments in an attempt to develop more meaningful severity indexes.

NCHRP Project 22-8 (*7*), on the evaluation of performance-level selection criteria for bridge railings, included a detailed examination of the Benefit-Cost Analysis Program (BCAP), a computer program (*8*) developed to facilitate the evaluation of alternative roadside safety improvements. As its name suggests, BCAP compares an improvement's incremental benefits accruing to road users with the additional costs for construction and maintenance incurred by the highway agency (*9*). NCHRP Project 22-8 researchers analyzed 4,552 accidents involving Texas bridges for the period from 1988 to 1990 (*10*) and found that the proportion of severe to fatal (percent A + K) injury accidents differed markedly among vehicles retained on the bridges, those that went through the bridge railings, and those that went over the bridge railings. The study was unable to establish the reasons for the difference in severity between vaulting and penetration.

Mak at the Texas Transportation Institute and Sicking at the University of Nebraska are currently conducting NCHRP Project 22-9 to develop improved microcomputer software for cost-effectiveness analysis procedures. The proposed software is intended for two primary uses:

- To access alternate roadside safety treatments for either point locations or sections of roadway, and
- To develop warrants and guidelines, including those which consider the performance levels of safety features.

At the time that the survey was undertaken, Mak was also conducting an FHWA project to develop techniques and plans for future accident research studies to improve benefit–cost models (such as the models being developed in NCHRP Project 22-9). The now-completed study examined the potential for the development of severity indexes through the collection of in-depth accident data.

Although only three ongoing projects were mentioned by respondents, they are prominent examples of the types of research efforts necessary to significantly improve current severity index values. An additional study by Council at the University of North Carolina is developing techniques to account for items such as the effects of airbags and unreported accidents. The work is especially timely since the adoption of new technologies, such as airbags, could make current severity indexes obsolete.

New Research Suggestions

The next question challenged respondents to identify potential improvements to existing resources. Specifically, they were asked, "If you were given the authority to define the next major research project addressing the weaknesses of existing severity index and/or roadside safety information, what would be the primary focus of the research?" Finally, they were asked for any other comments or suggestions related to severity indexes, roadside safety, establishing priorities, or cost-effective treatments.

The responses to both questions tended to offer suggestions in which additional improvements could be made in the roadside safety analysis process. Some of the respondents' interests require research for their resolution, whereas others might be resolved through administrative or educational initiatives.

Five respondents suggested that the primary focus of a new research project should be verification of projected severity indexes, preferably through an evaluation of actual improvements that were selected on the basis of the *Roadside Design Guide* (*3*) methods. The skepticism expressed by many could potentially be resolved through a validation project. Four respondents recommended efforts to simplify the analysis methods. A similar number of respondents proposed studies to develop severity indexes for objects that are not included in the current guidelines. Two states suggested that the primary need was to establish more credible information on encroachment rates and angles, whereas two others believed that improved accident cost estimates should be a priority topic. Other issues recommended for additional study included methods and data for speeds of less than 64 km/hr (40 mph), determination of cost-effective clear roadside widths, and the redirection capabilities of back slopes.

Although it was not a research topic, several states mentioned a need to improve the user interface and operation of the ROADSIDE computer program (*2*). In addition, some respondents expressed concern that engineers within their agencies did not have a good understanding of the factors associated with roadside safety; the simplicity of the ROADSIDE program could lead the unwary to erroneous conclusions. In other words, in the absence of "informed engineering judgment," ROADSIDE simply allows the analyst to make mistakes faster.

Summary

Responses of state traffic and highway safety engineers to the survey described here provide a reasonably representative picture of the roadside safety analysis methods used by highway agencies. Survey responses indicate that AASHTO's 1989 *Roadside Design Guide* (*3*) and the companion ROADSIDE computer program (*2*) are the authoritative, most commonly used technical references on roadside safety issues. Respondents expressed relatively high degrees of confidence in the values of those analysis parameters, such as traffic volume and roadway alignment, that they can readily determine for their own road systems. On the other hand they expressed concerns about those parameters that are not specific to a particular study site; prime examples include severity indexes, roadside encroachment characteristics, and accident costs. The responses from the extended population of practitioners, who attempt to implement research recommendations on a daily basis, may differ from the perceptions of researchers, who may be more familiar with the technical difficulties involved in developing severity index values.

CONCLUSIONS

In spite of previous studies to define, develop, and test severity indexes, the present research found that the severity index has not reached a mature stage of development. Currently, the most widely used values for severity indexes are those presented in the *Roadside Design Guide* (*3*) along with those in the *Supplemental Information for Use with the ROADSIDE Computer Program* (*2*). The developers of these indexes based them on expert opinion tempered with an understanding of general accident study methodologies and results. To date no research effort has confirmed these severity index values as accurate, authoritative, or representative of crashes that actually occur on U.S. roadsides. Despite some shortcomings, the AASHTO procedures, together with supplemental information developed by FHWA, represent the best guidance available today; they should certainly be used as a starting point for the beginning user.

Although local engineers and consultants also conduct roadside safety analyses, state highway safety analysts and designers are the most frequent users of severity indexes. National survey results show that these individuals have greater problems with severity indexes than with any other aspect of roadside cost-effectiveness studies. In addition, their responses indicated uncertainty, and in some cases confusion and frustration, about cost-effectiveness studies and the ROADSIDE computer model. Clearly, there is a need for improvement in the understanding and use of these safety tools.

The survey found that state highway safety analysts and designers had an extremely difficult time selecting and justifying their choice of severity indexes, accident costs, and encroachment parameters. When roadside safety calculations produce nonintuitive results or support treatments with excessive costs, the skeptical analyst may simply be inclined to blame severity indexes or other parameters that are difficult or impossible to validate.

Despite concerns with severity index accuracy, there was considerable sentiment among survey respondents for an expanded severity index list. As long as severity indexes are not tied directly to crash experience, it should be possible to incorporate additional objects, different object designs, other speeds, and similar parameters into such a list.

The findings of this project offer several important opportunities for additional research. First, it is obvious that many users of current roadside safety evaluation methods lack confidence in the results of their analyses; an effort is needed to correct any deficiencies and bolster the confidence of the users. Second, the inventory of objects and conditions included in a list of severity indexes should be expanded and annotated to facilitate proper analysis, especially by those with limited engineering experience. Third, the software commonly employed to simplify the analyses should be made more user friendly; modifications should also limit the opportunity for serious errors due to the unwary acceptance of default values within the program. Fourth, the levels of understanding of roadside cost-effectiveness methodology vary considerably with the training and experience of the analyst; consequently, there is a real need for expanded training in this area, especially for young engineers.

Finally, a major effort is required to significantly improve the quality and accuracy of severity indexes. The endeavor must be comprehensive in terms of the obstacles and conditions addressed and must recognize the dynamic aspects of both vehicle and roadway technologies that will continue to influence crash severity. The optimal method for undertaking this type of study is not certain. A meaningful study based on accident and roadway data would require extensive, high-quality data bases and would need to account for unreported accidents. Alternative study procedures employing some of the innovative techniques used on a smaller scale in several recent studies might provide a better opportunity for resolving the severity index dilemma.

ACKNOWLEDGMENTS

Materials in this paper were developed during preparation of TRB's NCHRP Project 20-5, Synthesis Topic 23-09. The authors gratefully acknowledge the contributions of the state highway agency personnel who participated in the survey of understanding and use of severity indexes for cost-effectiveness determinations for off-road accidents.

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Box-Beam Guardrail Terminal

DEAN L. SICKING, KING K. MAK, AND WILLIAM B. WILSON

A crashworthy terminal for box-beam guardrails was developed and successfully crash tested. The terminal incorporated a telescoping tube configuration with a 7-in. × 7-in. × 1/8-in. (178-mm × 178-mm × 3.2-mm) outer tube placed over the standard 6-in. × 6-in. × 3/16-in. (152-mm × 152-mm × 4.8-mm) box-beam rail element. A breakaway post and cable mechanism, similar to that used with the breakaway cable terminal, was used at the end of the terminal to provide anchorage for downstream impacts. An impact head attached to a short segment of a 6-in. × 6-in. × 3/16-in. (152-mm × 152-mm × 4.8-mm) box beam was inserted into the upstream end of the outer tube. The impact head serves to capture impacting vehicles, and the short tubular element slides back into the outer tube to allow the lead wood post to break away. A breakaway tensile connector similar to that used in the ET-2000 terminal was incorporated to transmit tension between the two telescoping tubes without adversely affecting system compression. Pultruded glass/polyester fiber-reinforced plastic tubes were inserted inside the telescoping steel tubes to provide energy dissipation. Full-scale crash testing demonstrated that this telescoping tube terminal for box-beam guardrails met safety standards set forth in *NCHRP Report 230.*

Guardrails are often used to protect the motoring public from serious roadside hazards such as bridge piers and steep roadside slopes. Even though guardrail installation is considered a safety improvement at these sites, the barrier is a hazard in itself. In fact guardrails are the third leading object struck in fatal ran-off-road accidents, behind only trees and utility poles (*1*). A large portion of these fatalities can be directly attributed to accidents involving guardrail terminals. A recent study of guardrail accidents in Texas indicated that terminals accounted for 41 percent of all fatal guardrail accidents, whereas they constituted only 20 percent of nonfatal guardrail accidents (*2*).

The severity associated with guardrail terminal accidents has prompted recent development of improved guardrail end treatments for the widely used W-beam guardrail (*3–6*). However, less widely used barriers, such as box-beam guardrails, have been neglected. The only terminal currently available for box-beam guardrails involves tapering the rail element down to the ground. This sloped-end design has been shown to have the potential for causing impacting vehicles to vault and roll over under certain impact conditions, particularly for small vehicles traveling at high speeds (*7*).

The lack of a crashworthy terminal for box-beam guardrails has caused the Wyoming Department of Transportation (WyDOT) and other highway agencies to begin flaring box-beam guardrail ends out of the clear zone. This practice requires additional lengths of guardrail beyond the length of need, resulting in higher barrier costs and increased frequencies of barrier accidents. Furthermore, this practice cannot be implemented at some sites because of roadside slopes that restrict the ability to flare the guardrail ends. Thus, highway agencies are faced with a choice of using a different type of barrier that may result in severe snow-drifting problems, installing expensive crash cushions to shield the barrier end, or installing an unsafe terminal within the clear zone.

In recognition of the safety problems posed by existing box-beam guardrail terminal designs, WyDOT sponsored a research study at the Texas Transportation Institute to develop a safer end treatment for this barrier (*8*). The objective of the research was to develop a crashworthy terminal for box-beam guardrails that are relatively inexpensive to construct and maintain. The remainder of this paper describes the development and full-scale crash testing of this new telescoping tube terminal for box-beam guardrails.

DESIGN CRITERIA

In accordance with *NCHRP Report 230* (*9*), a guardrail terminal is required to provide safe deceleration or controlled barrier penetration for vehicles striking upstream from the beginning of the length of need (LON) and barrier anchorage for redirecting vehicles striking beyond the LON. Controlled penetration of a barrier end at a high rate of speed could still lead to secondary collisions with serious consequences. Thus, it is desirable for a barrier terminal to provide some level of impact attenuation. Attenuating terminals capture vehicles striking head-on or at low angles and provide safe deceleration until the vehicle comes to a stop. Although attenuating terminals cannot capture vehicles striking at very high angles, the vehicles are slowed significantly and the severity of any secondary impact is minimized. Field experience has shown that roadside slopes and other site constraints often restrict the use of flared barrier terminals. Thus, it is desirable to design the guardrail terminal so that it can be used on a tangent.

Costs associated with the terminal are also a major consideration. Most guardrail installations are rarely, if ever, struck, and the benefits of even greatly improved impact performance are often not sufficient to justify the higher terminal costs (*10*). Past experience has shown that high construction and maintenance costs have prevented widespread implementation of a number of crashworthy barrier terminals.

In view of the information just presented, the primary objective of the research described here was to develop a box-beam guardrail terminal that could offer the following features:

- Meet nationally recognized safety standards (*9*),
- Provide attenuation for vehicles striking the barrier end,
- Provide safe impact performance when installed on a tangent section of guardrail,
- Be inexpensive to install and maintain, and
- Be simple to construct.

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TELESCOPING TUBE TERMINAL CONCEPT

Box-beam guardrails are weak-post barrier systems that are commonly used in regions that receive heavy snow. The barrier uses 6-in. × 6-in. × 3⁄16-in. (152-mm × 152-mm × 4.8-mm) structural steel tubing as a longitudinal rail element. The rail is mounted on S3×5.7 steel posts spaced 6 ft (1.83 m) apart. The structural steel tube gives the rail element a relatively high bending strength, whereas the weak steel posts allow large lateral deflections. The telescoping tube terminal concept involves placing an oversized outer tube on the end of the standard box-beam rail element. An impact head is placed in front of the outer tube to capture the striking vehicles. During end-on or low-angle impacts, the impact head would capture the vehicle and the outer tube will telescope back over the box-beam rail element. Energy absorbers, placed inside the outer tube, are crushed as the system telescopes down and thereby provide a controlled deceleration for vehicles. For head-on impacts at higher angles, the system would initially capture the vehicle. However, as the terminal telescopes back, the vehicle would push the barrier end to the side. Eventually, lateral loads in the terminal would become sufficient to bend the outer rail element and allow the vehicle to pass behind the barrier. Also, the terminal must be designed to provide adequate tensile capacity for the box-beam guardrail to successfully redirect vehicles impacting the side of the barrier. Thus, the terminal system must be capable of transmitting high tensile loads while having low compressive strength.

ENERGY ABSORBERS

The energy absorber must be efficient in terms of both the energy dissipated per unit volume of material and the ratio between initial and crushed absorber length. First, there is very limited space inside the telescoping outer tube. Second, there is a practical limit to the length of the telescoping outer tube before the weight would become prohibitive to safely accommodate impacts of small passenger cars. If the energy dissipation per unit volume of material is too low, the terminal would require too long a crush distance to bring large passenger cars to a safe stop. Similarly, if the ratio between initial and crushed absorber lengths is too low, the required length of the telescoping tube would also be excessive. Theoretically, an energy-absorbing terminal must deflect at least 16 ft (4.88 m) during head-on impacts to meet current crash test standards. The energy absorber must be capable of nearly 100 percent compression to avoid an excessive length for the telescoping tube.

After an extensive search for available energy-absorbing materials, the research team decided on pultruded fiber-reinforced plastic (FRP) as the energy absorber for use with the telescoping tube terminal. Numerous pultruded FRP structural shapes were obtained in different sizes and were tested to identify the most efficient energy-absorbing configurations. Static crush testing was used as a preliminary screening process. A dynamic testing program was then undertaken to identify both the dynamic crush characteristics of the FRP material and the dynamic buckling characteristics of the structural shapes.

Testing results indicated that round tubes provided greater energy dissipation per unit volume of material than any other shape, and thus, round tubes were selected for use in the telescoping tube terminal. It was also found that buckling of the tube became a problem for small-diameter tubes. It was therefore decided to use the largest-diameter tube possible, that is, 6 in. (152 mm) in diameter, and to control for the energy-absorbing characteristics by using different wall thicknesses for the tubes. Also, it was found that the FRP material is capable of developing very high compressive stresses before crush initiates. Tulip-shaped ends were incorporated as crush initiation mechanisms to eliminate these high initial crush forces, as shown in Figure 1.

TERMINAL DESIGN

Schematic drawings of the telescoping tube terminal are shown in Figure 1. A specially fabricated 7-in. × 7-in. × 1⁄8-in. (178 × 178 × 3.2-mm) A36 steel outer tube weighing approximately 310 lb (141 kg) is incorporated into the design. The tube is manufactured from two bent plates and is welded along two corners with a series of 3-in. (76-mm) welds spaced 6 in. (152 mm) center to center. Each end of the tube is strengthened with continuous welds and outer collars to limit terminal damage during low-speed impacts. The upstream end of the outer tube incorporates 24-in. (0.61-m) continuous welds and a 6-in. (152-mm)-wide, 1⁄4-in. (6.4-mm)-thick A36 steel collar, whereas the downstream end is constructed with a similar collar that is only 2 in. (51 mm) wide.

The impact head is designed along the lines of the ET-2000 impact head and weighs approximately 125 lb (57 kg). The impact plate, as shown in Figure 1, is constructed with a 20-in. × 20-in × 3⁄8-in. (508-mm × 508-mm × 9.5-mm) A36 steel and incorporates 1½-in. × 1⁄4-in. (38.1-mm × 6.4-mm) A36 steel straps welded on the perimeter of the plate to provide a mechanical interlock with impacting vehicles. The impact plate is attached with 3⁄8-in. (9.5-mm)-thick A36 steel gussets to a 3-ft (0.91-m)-long section of standard TS 6-in. × 6-in. × 3⁄16-in. (152-mm × 152-mm × 4.8-mm) A500 grade B steel tube normally used in box-beam guardrails. An end cap made from a 1⁄8-in. (3.2-mm)-thick steel plate is welded to the end of the box-beam section. The end of the 6-in. × 6-in. × 3⁄16-in. (152-mm × 152-mm × 4.8-mm) tube is enlarged to provide a closer fit inside the outer tube by welding 1⁄4-in. (6.4-mm) steel straps to all four sides. This reduces the clearance between the inner and outer tubes to approximately 1⁄8 in. (3.2 mm) on all sides. If the inner tube is inserted 1 ft (0.31 m) into the outer tube, this level of tolerance would allow only a 1.2-degree misalignment between the two tubes. The upstream end of the box-beam rail and both ends of the intermediate spacer blocks are treated in a similar fashion to minimize the possibility of rotation within the outer tube.

Preliminary testing indicated that the gusset plates on the impact head could cut through the end of the outer tube, causing severe damage, even under moderate impact conditions. Therefore, steel angles with 1½-in. (38-mm)-thick rubber pads are welded to the sides of the TS 6-in. × 6-in. × 3⁄16-in. (152-mm × 152-mm × 4.8-mm) tube to prevent direct contact between the gusset plates and the outer tube. The rubber pads reduce both the impact forces transmitted to the vehicle when the impact head contacts the outer tube and the damage to the outer tube during low- and moderate-speed impacts.

The impact head is designed to be attached to a 5.5-in × 7.5-in. (140-mm × 191-mm) breakaway wood post similar to that used in breakaway cable terminals (BCTs). The wood post is weakened with a 2¾-in. (69.9-mm)-diameter hole at the base and is inserted into a 6-in. × 8-in. (152-mm × 203-mm) steel foundation tube. A BCT-type cable assembly is attached to the outer tube using a

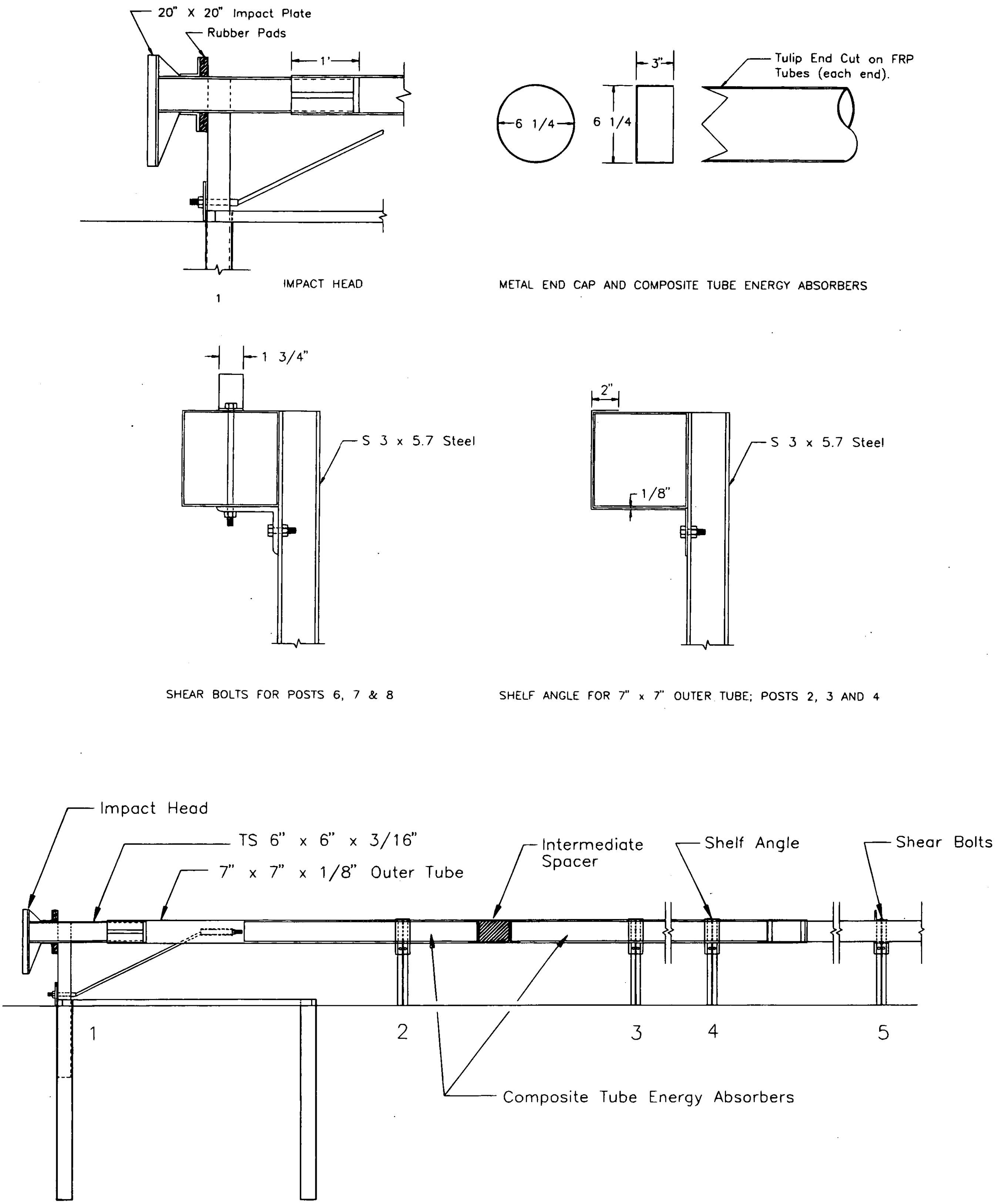


FIGURE 1 Telescoping tube terminal.

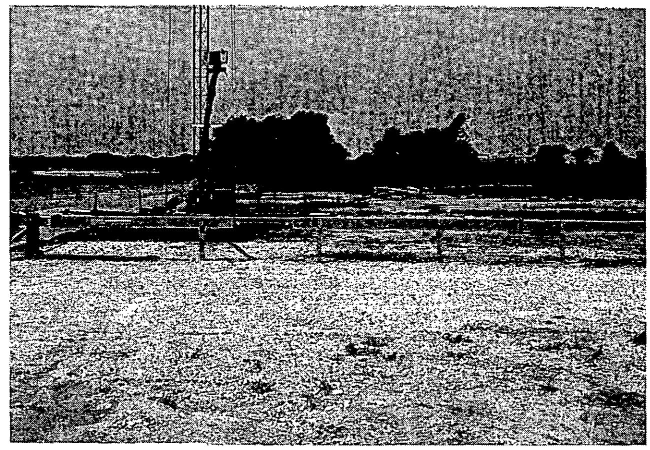
TS 2½-in. × 2½-in. × 3/16-in. (63.5-mm × 63.5-mm × 4.8-mm) steel tube welded to the outer surface. The BCT cable is anchored through the hole in the base of the leading wood post. A second steel foundation tube with ground channel strut is incorporated to reinforce the foundation tube under the first post.

The outer tube transmits tension to the downstream box-beam rail through a breakaway tensile connector, similar in design to that used with the ET-2000 guardrail terminal. Six lugs with teeth in one direction and sloped surfaces in the other direction are welded to the top of a 3-in. × 2-in. × 3/16-in. (76.2-mm × 50.8-mm × 4.8-mm) × 16¾-in. (435-mm)-long structural tubing. Corresponding holes were cut on the bottom of the TS 6-in. × 6-in. × 3/16-in. (152-mm × 152-mm × 4.8-mm) box beam for the lugs to engage. The detachable anchor mechanism is then attached to the outer tube with a 1¼-in. (32-mm)-diameter grade 5 all-thread rod and a 6-in. (152-mm)-long 2-in. × 2-in. × 3/16-in. (50.8-mm × 50.8-mm × 4.8-mm) structural tube welded to the bottom of the outer tube. The lugs on the detachable anchor mechanism are designed to release from the holes in the box-beam rail when the device is loaded in compression during end-on impacts. During side impacts, the anchor mechanism is loaded in tension and the steel lugs do not release from the box-beam rail, thereby preventing the two telescoping beams from becoming separated.

Except for the initial wooden breakaway post, all other posts are constructed from the S3 × 5.7 structural steel normally used with box-beam guardrails. However, in order to facilitate telescoping of the outer tube over the standard box-beam rail, special shelf angles are used with the outer tube, as shown in Figure 1. The shelf angles provide some constraint of the outer tube without the need for passing a bolt though the beam. Also, the first post downstream from the outer tube (Post 5) is not bolted to the standard box-beam rail. The next three posts (Posts 6, 7, and 8) in the system incorporate a 5/16-in. (7.9-mm)-diameter A307 bolt and a small clip angle at the top of the beam, as shown in Figure 1, to facilitate consistent shearing of the bolted connections during head-on impacts.

Energy dissipation elements were selected and configured, using a combined conservation of energy and momentum approach, to provide optimum safety performance for the terminal during head-on impacts. The final design incorporates a 6-ft (1.83-m)-long, 6-in. (152-mm)-diameter, 1/8-in. (3.2-mm)-wall-thickness tube at the front of the cushion to provide low-energy dissipation during small-car impacts and a 12 ft 8 in. (3.86 m)-long, 6-in. (152-mm)-diameter, ¼-in. (6.4-mm)-wall-thickness tube at the back of the terminal to provide sufficient energy-absorbing capability to handle large-car impacts.

This configuration provides for approximately 2 ft 4 in. (0.71 m) of empty space within the telescoping outer tube. This empty space provides for a low deceleration period during the time that the impact head and outer tubes are being accelerated to the speed of the vehicle. The thin-walled energy absorber also is provided with a 6-in. (152-mm)-long tulip crush initiator at each end to further delay the onset of high decelerations associated with crushing the full FRP section. Photographs of the completed terminal are shown in Figure 2.



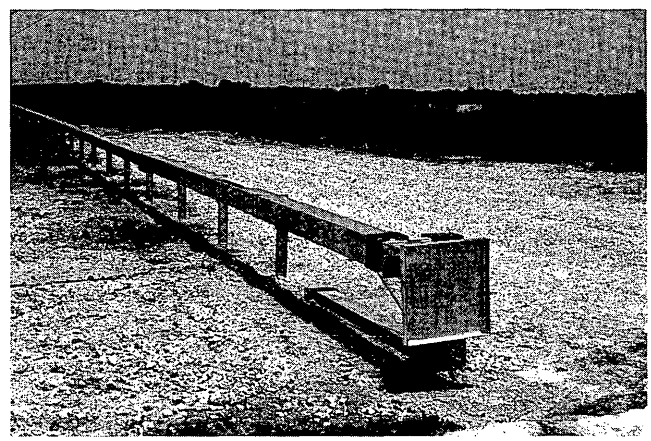


FIGURE 2 Completed telescoping tube terminal.

COMPLIANCE TESTING

NCHRP Report 230 (*9*) requires four full-scale crash tests of barrier terminals. Two of the tests are designed to study the head-on impact performance of the end treatment, and the remaining two tests investigate the redirective capacity of the barrier near the end of the terminal. The telescoping tube terminal successfully passed all four of the recommended crash tests, as summarized in Table 1 and described in the following sections.

Small-Car Head-On Test

The first compliance test involved an 1,800-lb (817-kg) passenger car striking the terminal head-on at a speed of 58.1 mph (93.5 km/hr). The vehicle was offset from the barrier centerline approximately 15 in. (381 mm) from the center of the terminal away from the roadway. This orientation will cause the vehicle to rotate counterclockwise toward the back of the rail and allow the telescoping tubes to buckle outward away from the barrier posts. Thus, offsetting the vehicle to the backside of the rail should maximize the potential for rail buckling and test failure. On impact the leading wooden post fractured and the cable anchor mechanism released as designed. The impact head was then pushed back until it contacted the outer tube. The vehicle was smoothly decelerated until it was virtually stopped. The vehicle then began to yaw counterclockwise as expected, and the outer tube began to bend at the point where the impact head section terminated. The vehicle was slowed to almost a complete stop by the time the vehicle released from the terminal. Note that the impact conditions for this test are designed to cause the

TABLE 1 Full-Scale Crash Test Results

Vehicle Weight, lb (kg)	Impact Conditions			Test Results				
	Speed, mph (kph)	Angle, (degrees)	Offset, in. (mm)	Maximum Deflection, ft (m)	Occupant Impact Velocity		Ridedown Acceleration	
					Long. ft/s (m/s)	Lateral ft/s (m/s)	Long. (g's)	Lateral (g's)
1,800 (817)	58.1 (93.5)	0	15 (381)	8.9 (2.7) (long.)	32.5 (9.9)	4.0 (1.2)	15.3	2.4
4,500 (2,041)	58.0 (93.3)	0	0	15 (4.6) (long.)	25.5 (7.8)	5.5 (1.7)	11.1	1.1
1,800 (817)	62.3 (100)	20.7	0	2.2 (0.7) (lat.)	21.6 (6.6)	17.7 (5.4)	7.1	9.7
4,500 (2,041)	61.7 (99.3)	25.3	0	6.5 (2.0) (lat.)	13.6 (4.2)	11.4 (3.5)	4.5	8.1

vehicle to spin out and possibly roll over. Since the telescoping tube terminal effectively attenuated virtually all of the impact energy, the vehicle remained stable and upright after leaving the terminal.

As shown in Table 1, all occupant risk values for this test were well below maximum allowable levels. Damage to the test vehicle and the telescoping tube terminal was relatively severe, as shown in Figure 3. Vehicle damage was localized to the grill and engine compartment, with no deformation of the occupant compartment. Terminal repair would have required replacement of the outer tube, FRP energy absorbers, and the first five posts.

Large-Car Head-on Test

The second compliance test was designed to evaluate terminal performance during high-speed, head-on impacts with full-size automobiles. This test involved a 4,500-lb (2,041-kg) vehicle striking the terminal head-on at a speed of 58 mph (93.3 km/hr). The terminal again performed as designed, and the test vehicle was smoothly decelerated to a stop over a distance of 15 ft (4.57 m). The test vehicle was slightly offset toward the roadside, and as a result the vehicle slowly yawed counterclockwise during the test, thereby producing some eccentricity in the telescoping tube. As the vehicle was slowing to a stop, the outer tube began to buckle. This behavior did not increase the vehicle deceleration rate and is merely an indication that the FRP crush forces are high enough to allow global buckling of the outer box beam if sufficient eccentricity is introduced. The outer tube would have continued to telescope over the inner tube if the vehicle had maintained sufficient energy to crush the FRP elements. Also note that approximately 3 ft (0.91 m) of usable energy absorber remained in the telescoping tube after the test and that this section of energy absorber could have absorbed almost 100 kip-ft (135.5 kJ) of additional energy. Thus, the terminal would have been able to successfully attenuate an impact with significantly higher impact energy without a significant increase in deceleration forces, and the difference between the actual impact severity of 505 kip-ft (684 kJ) and the target value of 541 kip-ft (733 kJ) is not considered to be significant.

All occupant risk values for this test were well within the recommended limits, as shown in Table 1. Damage to the vehicle, shown if Figure 4, was again isolated to the front and engine compartment areas. Damage to the telescoping tube terminal was not significantly greater than that observed during the small-car test. The outer tube was severely damaged and would require replacement. The FRP

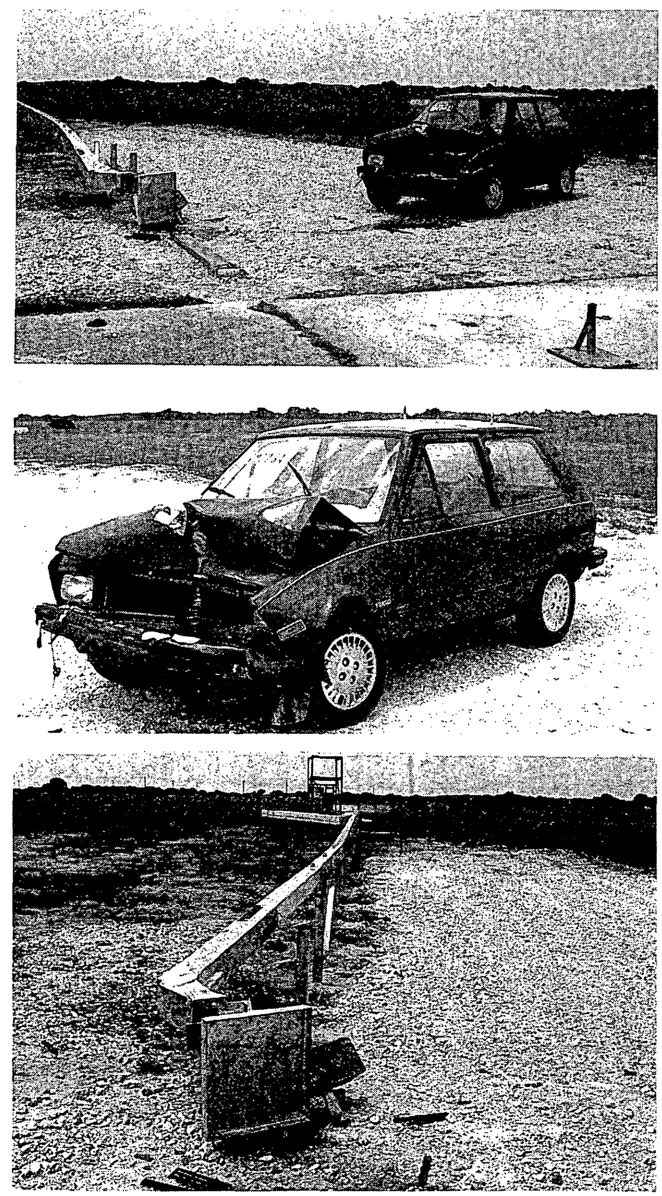


FIGURE 3 Damaged test vehicle and terminal after small-car head-on test.

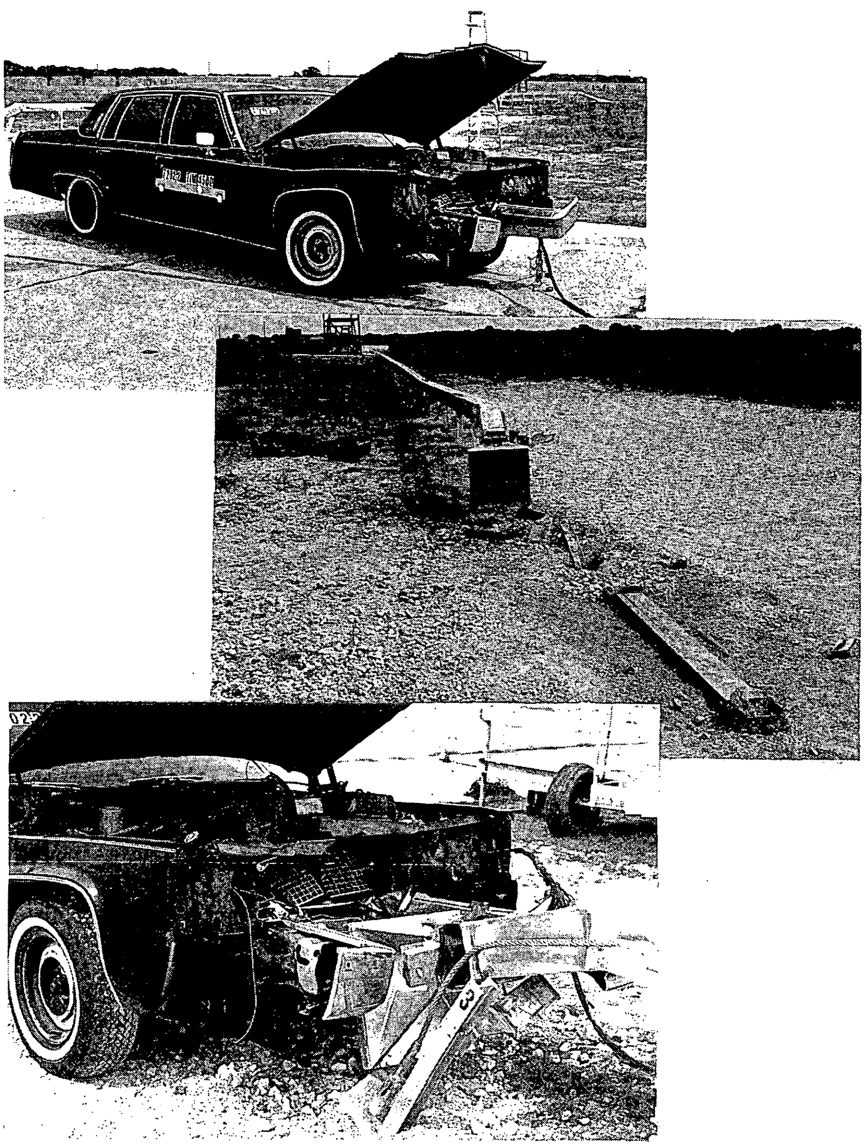


FIGURE 4 Damaged test vehicle and terminal after large-car head-on test.

energy absorbers and the first seven posts would also have required replacement.

Small-Car Redirection Test

The third compliance test was intended to evaluate the ability of the terminal to redirect small cars impacting the side of the terminal upstream from the beginning of the LON. The impact point, just upstream of Post 2, was halfway between the end of the terminal and the beginning of the LON at Post 3. The 1,800-lb (817-kg) test vehicle struck the terminal at a speed of 62.3 mph (100.3 km/hr) and an angle of 20.7 degrees. Terminal performance was very similar to that of a standard box-beam guardrail. The rail deflected sufficiently to allow the steel guardrail posts to contact the right from tire of the vehicle. The snag forces and rail redirection forces counterbalanced to cause the vehicle to slide down the rail without yawing away from the barrier. The test vehicle came to rest 150 ft (45.7 m) downstream from the original impact point and approximately 4 ft (1.2 m) in front of the rail.

All occupant risk values for this test were again within recommended limits, as shown in Table 1. Test vehicle and barrier damage were relatively minor for a test of this severity, as shown in Figure 5. Test vehicle damage was distributed along the passenger side of the vehicle, with the worst areas concentrated at the right front quarter panel. Terminal repair would again require replacement of the outer tube element, the FRP attenuation elements, and six steel guardrail posts.

Large-Car Redirection Test

The final compliance test was configured to examine the terminal's capacity for redirecting a vehicle at its designed containment limit. This test involved a 4,500-lb (2,041-kg) vehicle impacting the barrier at Post 3, the beginning of LON, at a speed of 61.7 mph

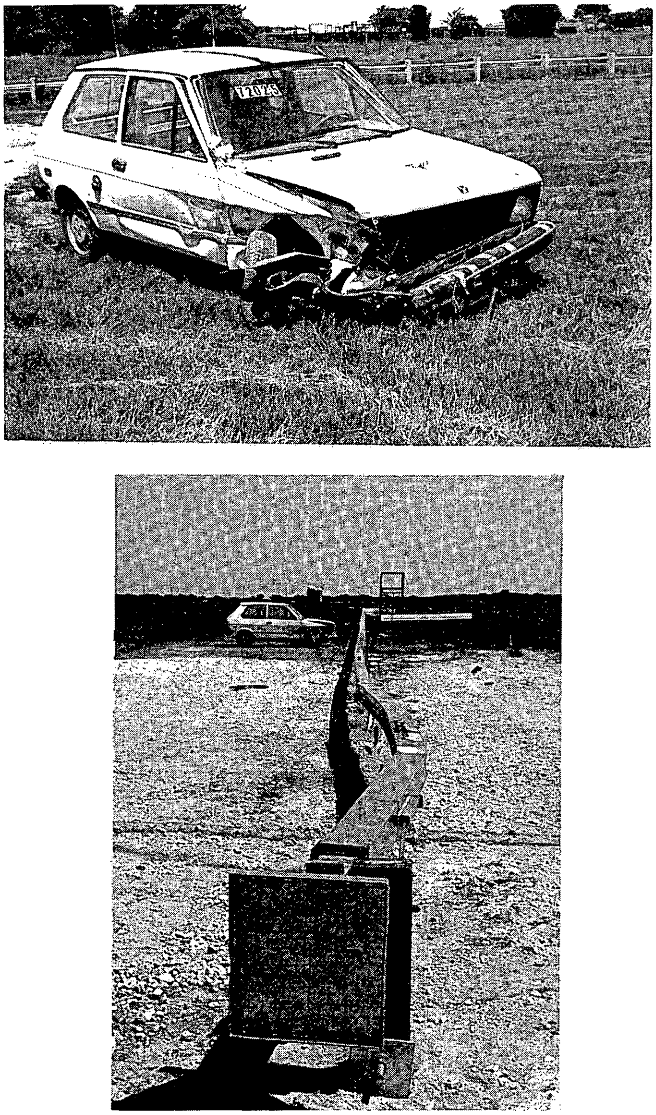


FIGURE 5 Damaged test vehicle and terminal after small-car redirection test.

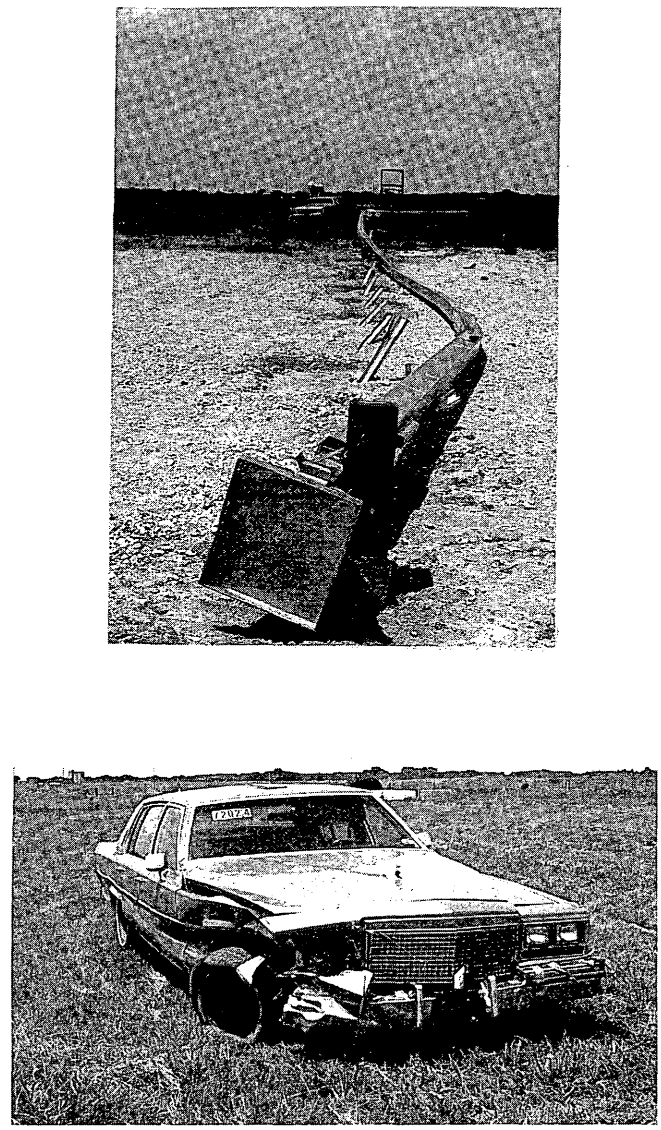


FIGURE 6 Damaged test vehicle and terminal after large-car redirection test.

(99.3 km/hr) and an angle of 25.3 degrees. The terminal again performed in a manner similar to that of a standard box-beam guardrail in smoothly redirecting the test vehicle. The test vehicle remained in contact with the rail until it came to rest approximately 145 ft (44.2 m) from the initial point of impact. Although the impact head became detached from the leading post, the breakaway cable mechanism proved to have the strength necessary to provide adequate anchorage for the barrier system.

All occupant risk values were well below recommended values, and the vehicle damage was relatively light, as shown in Figure 6. The barrier system would have required replacement of the outer tube, FRP energy absorbers, and approximately 20 guardrail posts.

CONCLUSIONS AND RECOMMENDATIONS

The telescoping tube terminal for use with box-beam guardrail has been shown to satisfy the requirements set forth in *NCHRP Report 230* (*9*). The system is designed to capture a vehicle striking the end of the terminal and to decelerate it to a safe and controlled stop rather than allowing the vehicle to penetrate behind the barrier at a high rate of speed. Furthermore, the system is designed to be installed tangent to the guardrail and can be used at sites where flared treatments are inappropriate. This terminal should perform well wherever there is sufficient space for it to be constructed either tangent or nearly tangent to the barrier system. Note that the terminal is approximately 50 ft (15.2 m) long. This section of the barrier must be installed along a straight line with no curvature.

Although production and installation costs are extremely difficult to quantify, terminal production costs are estimated to be in the range of $2,000 to $2,500 and installation costs should be less than $500. In addition, even though this terminal is somewhat more costly and more complicated to construct than existing sloped-end treatments, these factors should not be major obstacles to field implementation of the terminal. In fact, since the terminal will eliminate the need for flaring the barrier end out of the clear zone, the total cost of using the new terminal can, in some cases, be lower than the cost of using long flared sections of barrier with a conventional sloped-end treatment. Furthermore, note that although the initial cost of this terminal is comparable to that of the ET-2000 end treatment, it is much less expensive than other high-performance terminals such as the CAT, SENTRE, and BRAKEMASTER (*11*). Repair costs for this terminal, estimated from the head-on crash tests described herein, should be in the range of $1,250 to $1,500. These costs are also on the low end of the range for high-performance barrier terminals.

This terminal can easily be adapted for use as a median barrier end treatment with only minor modifications. These modifications would include (*a*) placing the first four posts (Posts 1 through 4) under the outer tube and the next four posts (Posts 5 through 8) under the standard section of 6-in. × 6-in. (152-mm × 152-mm) box-beam rail element instead of behind them and developing the appropriate attachment mechanisms, (*b*) developing a method for transitioning the 6-in. × 8-in. (152-mm × 203-mm) structural steel tube used in the box-beam median barrier to the 6-in. × 6-in. (152-mm × 152-mm) tube used for the box-beam guardrail, and (*c*) using a mechanism to accommodate reverse-direction impacts. Efforts to develop such a median barrier terminal are currently under way at the Texas Transportation Institute under the sponsorship of the WyDOT.

This telescoping tube terminal for box-beam guardrail has been approved by FHWA for field implementation, and WyDOT is in the process of incorporating this terminal design into some of its upcoming projects. Construction activities and accident histories will be monitored closely by WyDOT to identify any construction, maintenance, or safety problems associated with this terminal. Appropriate modifications to the terminal design will be incorporated, if necessary, to resolve the identified problems.

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Minnesota Swing-Away Mailbox Support

KING K. MAK AND ROGER D. HILLE

A swing-away mailbox support was designed by the Minnesota Department of Transportation (MnDOT) for use in locales where snow and ice removal during the winter presents a problem. The Minnesota swing-away mailbox support design uses a cantilevered arm for attachment of the mailbox assembly. The cantilever arm design is intended to allow for more efficient snow plowing operation without damaging the mailbox support, which presents a maintenance problem. The design allows complete snow removal beyond the shoulder or curbline, thus reducing snow-drifting on the roadway. It is easily installed with existing highway agency equipment, can be salvaged and reinstalled, and costs considerably less than current mailbox designs approved by MnDOT. The results of four full-scale crash tests conducted on this Minnesota swing-away mailbox support and the evaluation of its impact performance are presented. The mailbox support with a single mailbox assembly was judged to have successfully met all evaluation criteria outlined in *NCHRP Report 350* and the 1985 AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. However, the mailbox support with a triple mailbox assembly was judged to have failed to meet the evaluation criteria.

The Minnesota Department of Transportation (MnDOT) has designed a swing-away mailbox support suitable for use in locales where snow and ice removal during the winter presents a problem. The Minnesota swing-away mailbox support design uses a cantilevered arm for attachment of the mailbox assembly. The cantilever design is intended to allow for snow plowing operation without damaging the mailbox support, which presents a maintenance problem. The design allows complete snow removal beyond the shoulder or curbline, thus reducing snow-drifting on the roadway. It is easily installed with existing highway agency equipment, can be salvaged and reinstalled, and costs considerably less than current mailbox designs approved by MnDOT. This paper presents the results of four full-scale crash tests conducted on this Minnesota swing-away mailbox support and the evaluation of its impact performance. Testing and evaluation were performed in accordance with guidelines outlined in *NCHRP Report 350* (*1*) and the 1985 AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (*2*).

MINNESOTA SWING-AWAY MAILBOX SUPPORT DESIGN

The Minnesota swing-away mailbox support, a schematic diagram of which is shown in Figure 1, consists of four major components:

1. U-channel base post. A 3-lb/ft (4.46-kg/m), 60,000-lb/in^2 (413,685-kN/m^2) U-channel sign post is driven into the ground as a base post, leaving a stub height of approximately 18 in. (0.46 m) above ground level. The minimum specified embedment depth of the post is 4 ft (1.22 m) so that either a 6-ft (1.83-m)-long or a 7-ft (2.13-m)-long post may be used with the installation. A 7-ft (2.13-m)-long post was used in the crash tests since it was considered to be a more critical condition from a base bending standpoint. Note that the strong axis of the U-channel post is aligned with the direction of vehicle travel.
2. Vertical support. A vertical support, made from 1.66-in. (42-mm)-outsider-diameter, 1.38-in. (35-mm)-inside-diameter standard-weight pipe, is bolted to the post stub with two ⅜-in. × 2.5-in. (9.5-mm × 64-mm) bolts spaced 12 in. (0.31 m) apart. The locations of the bolts are adjustable so that the height of the mailbox above the roadway surface is between 38 and 42 in. (0.97 and 1.07 m). A midrange mailbox height of 40 in. (1.02 m) was used in the crash tests. The top 12 in. (0.31 m) of the pipe is bent at a 45-degree angle. A 16-in. (0.41-m)-long, 1.315-in. (33-mm)-outside-diameter, 1.049-in. (27-mm)-inside-diameter standard-weight pipe is inserted into the bent end of the vertical support and is welded in place. The insert pipe extends 8 in. (203 mm) beyond the end of the vertical support for attachment of the cantilever arm. A groove, ½ in. (13 mm) wide and ⅛ in. (3.2 mm) deep, is cut into the insert pipe 3 in. (76 mm) above the end of the vertical support for use with a ¼-in. (6.4-mm)-diameter set screw to attach the cantilever arm. The set screw and groove configuration renders removal of the cantilever arm more difficult, to discourage vandalism, although it still allows the cantilever arm to rotate freely about the insert pipe and to separate readily from the vertical support on impact.
3. Cantilever arm. A cantilever arm, also made from 1.66-in. (42-mm)-outside-diameter, 1.38-in. (35-mm)-inside-diameter standard-weight pipe, connects the vertical support to the mailbox assembly. The cantilever arm is 48 in. (1.22 m) in length, 12 in. (0.31 m) of which is bent at 45 degrees for attachment to the insert pipe. Two ⅛-in. (3.2-mm)-thick, 5-in. (127-mm)-long, 1-in. (25-mm)-wide metal straps, one at the end of the cantilever arm and the other spaced 12 in. (0.31 m) apart, are welded to the top of the pipe. Two 5/16-in. (7.9-mm) holes, spaced 4 in. (102 mm) center to center, are drilled in the straps for attachment of the mailbox assembly to the cantilever arm. An alternative design shortens the metal strap to only 2.5 in. (64 mm) in length with a single 5/16-in. (7.9-mm)-diameter hole drilled through the center of the pipe and strap. The purpose of the shorter strap is to minimize the potential of the straps penetrating the windshield if they should become exposed during an impact. It was decided to use the longer metal strap attachments for the test installation since that would be the more critical design from a safety standpoint.

For the triple mailbox assembly, the cantilever arm consists of standard-weight pipe for the bent portion of the arm that attaches to the insert arm and the first 5 in. (127 mm) of the horizontal arm. The remainder of the horizontal arm is constructed of thin-wall pipe (such as muffler pipe) welded to the standard-weight pipe to reduce the weight of the cantilever arm. The horizontal arm forks out into three branches, spaced 12 in. (0.31 m) apart, one for each of the three mailbox assemblies.

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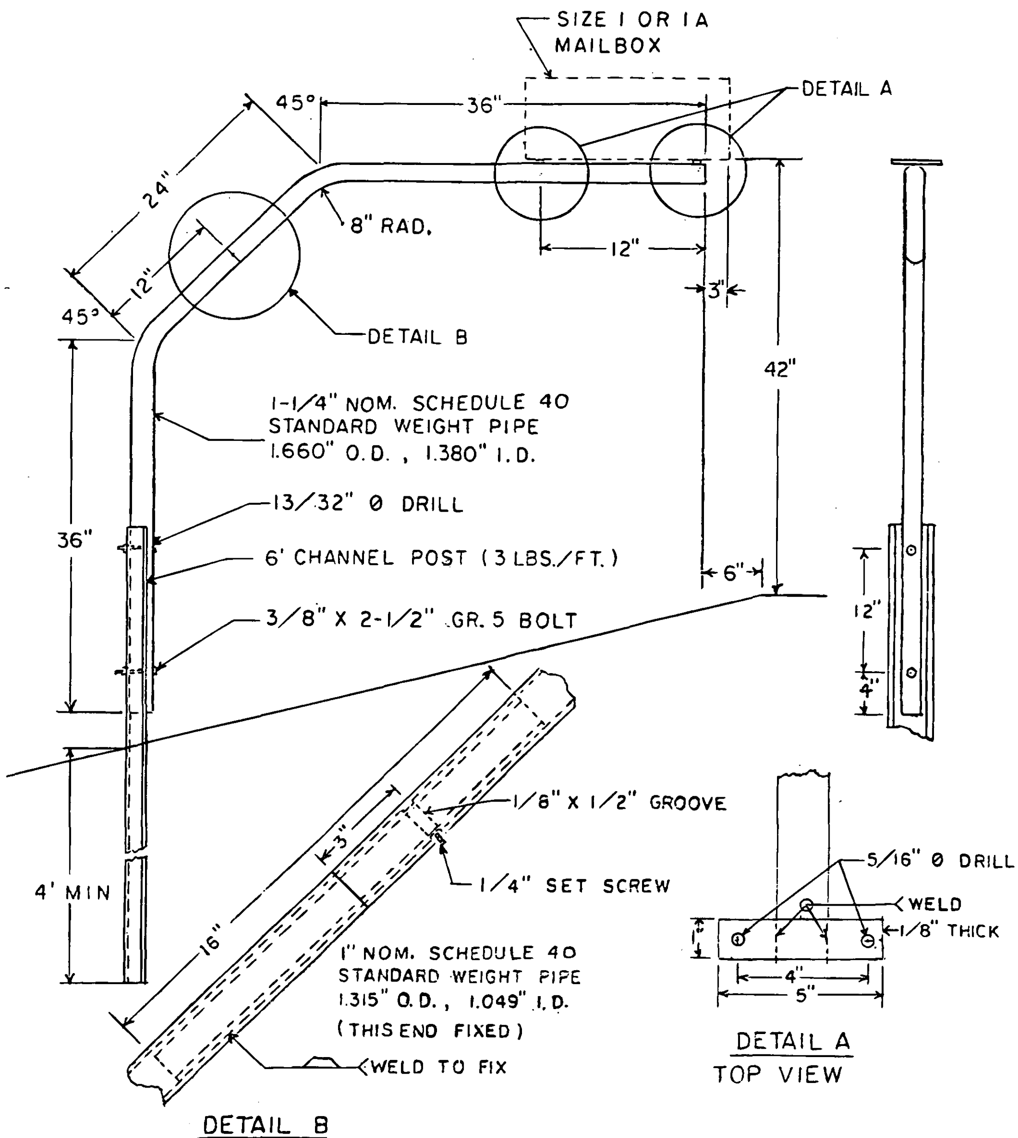


FIGURE 1 Minnesota swing-away mailbox support design.

4. Mailbox assembly. A 16-in. (0.41-m)-long, 8-in. (203-mm)-wide (nominal), 1-in. (25-mm)-thick (nominal) wood board is bolted to the straps on the cantilever arm with four ¼-in. (6.4-mm)-diameter, 1.5-in. (38-mm)-long carriage bolts. A size 1A mailbox is attached to the wood board with drywall (sheetrock) screws.

A standard plastic newspaper tube is also attached to one side of the mailbox assembly with a 16-gage metal bracket. The plastic newspaper tube is attached to the metal bracket with two ¼-in. × ½-in. (6.4-mm × 13-mm) bolts, and the metal bracket is attached to the bottom of the wood board with four 1-in. (25-mm) drywall screws. This attachment configuration has been approved by the U.S. Postal Service.

The attachment of the mailboxes to the cantilever arm for the triple mailbox assembly was similar to that of the single mailbox assembly. For each mailbox assembly, a wood board was bolted to the cantilever arm and the mailbox was attached to the wood board with drywall screws. A single plastic newspaper tube was attached to one end (nonimpact end) of the triple mailbox assembly.

Photographs of the mailbox test installation and details of the mounting bracket and the post attachment are shown in Figures 2 and 3, respectively.

CRASH TEST MATRIX

Four full-scale crash tests were conducted to evaluate the impact performance of the Minnesota swing-away mailbox support:

1. *NCHRP Report 350* (*1*) Test Designation 3-60 (Test 7147-11). An 820-kg (1,808-lb) passenger car struck the vertical mailbox support head-on at a nominal speed of 35 km/hr (21.7 mph) and 0 degrees. The mailbox support was aligned with the right front quarter point of the vehicle.
2. *NCHRP Report 350* (*1*) Test Designation 3-61 (Test 7147-12). An 820-kg (1,808-lb) passenger car struck the mailbox support head-on at a nominal speed of 100 km/hr (62.1 mph) and 0 degrees.

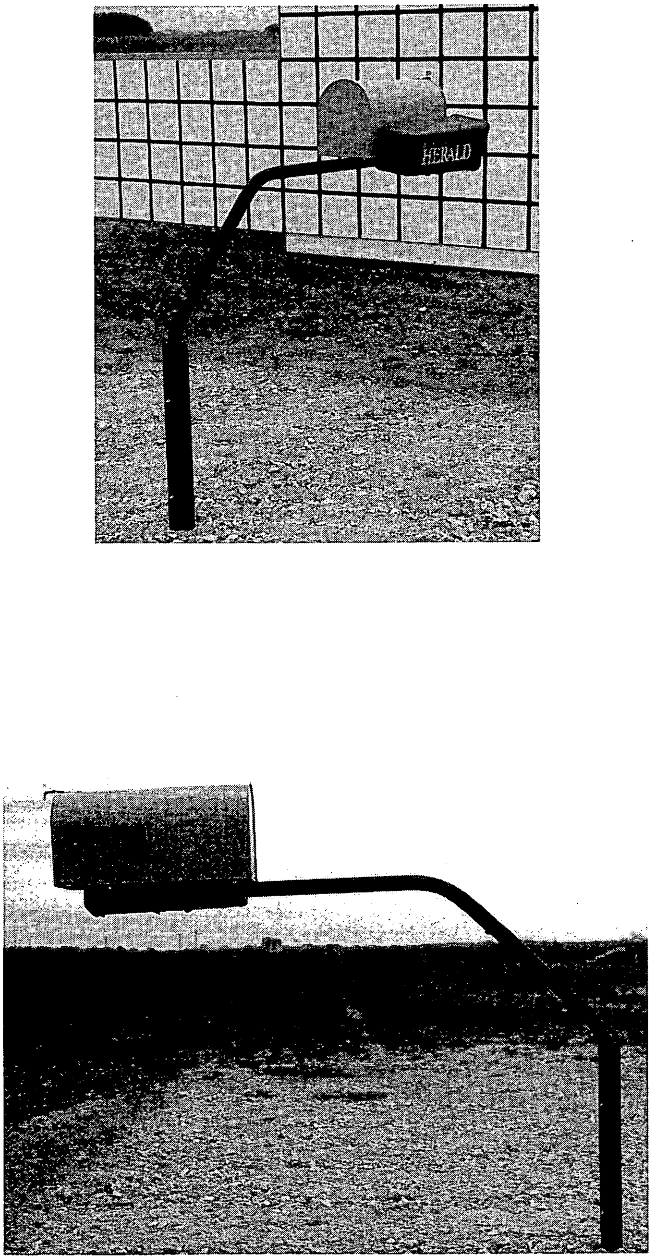


FIGURE 2 Minnesota swing-away mailbox installation.

The mailbox support was aligned with the right front quarter point of the vehicle.

3. Test 7147-13. An 820-kg (1,808-lb) passenger car struck a single mailbox assembly head-on at a nominal speed of 100 km/hr (62.1 mph) and 0 degrees. The centerline of the mailbox assembly was aligned with the centerline of the vehicle.

4. Test 7147-14. This test was identical to Test 7147-13 except for the mailbox assembly, to which three mailboxes instead of a single mailbox were attached.

In accordance with the crash test matrix for support structures outlined in *NCHRP Report 350* (*1*), two crash tests are required for evaluation of the mailbox support, which are Tests 3-60 and 3-61.

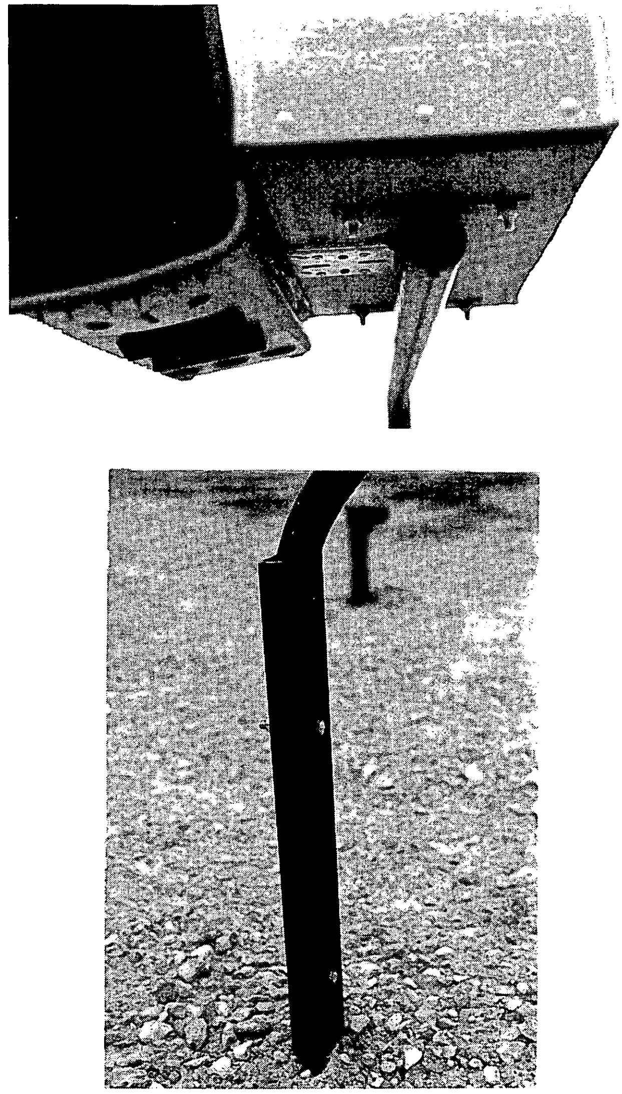


FIGURE 3 Mounting bracket and post attachment.

The objective of the low-speed test (Test 3-60) is to evaluate the breakaway, fracture, or yielding mechanism of the support, whereas the objective of the high-speed test (Test 3-61) is to evaluate the vehicle and test article trajectory. The right front quarter point of the vehicle was selected as the point of impact so that the interaction between the cantilevered arm and the mailbox assembly with the windshield of the vehicle can be evaluated.

The third (Test 7147-13) and fourth (Test 7147-14) tests in which the mailbox assembly directly impacts the windshield of the vehicle are not specifically required according to guidelines set forth in *NCHRP Report 350* (*1*) but they are included because of the cantilever design of the mailbox support. Previous crash tests have shown that the mailbox assembly has the potential of striking the windshield and intruding into the passenger compartment. This potential is minimized by designing the support structure so that the front of the vehicle will contact and engage the support structure first. This allows the mailbox assembly to be pushed forward and downward or thrown up and over the vehicle, thus avoiding impact of the mailbox assembly with the windshield.

In the case of the Minnesota swing-away mailbox support, the mailbox assembly is attached to a cantilevered arm so that the mailbox assembly could impact the windshield of the vehicle without the front of the vehicle impacting the support. Since the mailbox assembly has the potential of directly impacting the windshield of the vehicle, Crash Tests 3 and 4 were included in the crash test matrix to evaluate the potential of the mailbox assembly penetrating or intruding into the occupant compartment.

RESULTS OF CRASH TESTS

All crash tests and data analysis were conducted in accordance with guidelines contained in *NCHRP Report 350* (*1*). All four crash tests were conducted with 820-kg (1,808-lb) passenger cars at a test weight of 895 kg (1,971 lb), including an uninstrumented 50th percentile male anthropometric dummy placed in the driver's seat. Photographs of a typical test vehicle are shown in Figure 4. The vehicles were directed into the test installation using the cable reverse tow and guidance system, and was released to be free-wheeling and unrestrained just before impact. Brief descriptions of the crash test and data analysis procedures are presented as follows.

FIGURE 4 Test vehicle.

Test 7147-11

A 1986 Yugo GV was used for the first crash test. The vehicle struck the mailbox support at a speed of 35.1 km/hr (21.8 mph). On impact the vertical support and the U-channel base post began to lean forward and the cantilever arm and mailbox assembly began to rotate toward the vehicle. The cantilever arm then separated from the vertical support. The vehicle lost contact with the cantilever arm and mailbox assembly traveling at a speed of 25.9 km/hr (16.1 mph). However, the vertical support remained in contact with the undercarriage of the vehicle until the vehicle cleared the vertical support. The brakes on the vehicle were then applied, and the vehicle subsequently came to rest approximately 24 m (80 ft) downstream from the point of impact.

The cantilever arm and mailbox assembly came to rest approximately 17 m (55 ft) downstream and 5 m (15 ft) to the right of the impact point. The cantilever arm was only scraped, and the mailbox assembly was deformed, as shown in Figure 5. The vertical support was scraped, and the U-channel base post was bent and pushed back 180 mm (7 in.) at ground level.

The vehicle (also shown in Figure 5) sustained minimal damage. There was 80 mm (3.2 in.) of permanent deformation to the bumper where contact with the vertical support and U-channel base post occurred. There were dents in the oil pan and gas tank and scrape marks along the floor pan on the right side caused by contact with the vertical support of the mailbox test installation.

FIGURE 5 Mailbox installation (*top*) and vehicle (*bottom*) after Test 7147-11.

A summary of the test results is presented in Table 1. In the longitudinal direction, occupant impact velocity was 1.9 m/sec (6.1 ft/sec), and the highest 10-msec average ridedown acceleration was 0.9 *g*. No occupant contact occurred in the lateral direction. The change in vehicle velocity at the loss of contact was 9.2 km/hr (5.7 mph).

Test 7147-12

The 1986 Yugo GV used in the first test (Test 7147-11) was repaired and used for the second crash test. The vehicle struck the mailbox vertical support at a speed of 104.9 km/hr (65.2 mph). On impact the vertical support and the U-channel base post began to lean forward and the cantilever arm and mailbox assembly began to rotate toward the vehicle. The mailbox also began to separate from the wood board that was attached to the cantilever arm. The mailbox became completely detached from the wood board, and the mailbox struck the A-pillar on the driver's side of the vehicle. The mailbox lost contact with the vehicle while the vehicle was traveling at 98.0 km/hr (60.9 mph). The vertical support and U-channel base post remained in contact with the undercarriage of the vehicle until the vehicle cleared the vertical support. The brakes on the vehicle were then applied, and the vehicle subsequently came to rest 134 m (441 ft) downstream from the point of impact.

The mailbox installation separated into several pieces as shown in Figure 6. The plastic newspaper tube landed 15 m (48 ft) downstream and 8 m (25 ft) to the left of the point of impact. The deformed mailbox landed 18 m (60 ft) downstream and 5 m (18 ft) to the left of the point of impact. The cantilever arm and wood board were found 22 m (72 ft) downstream and 12 m (38 ft) to the left of the point of impact. The vertical support arm was only scraped, and the U-channel base post was bent and pushed back 150 mm (6 in.) at ground level.

The vehicle sustained minimal damage, as shown in Figure 6. There was 120 mm (4.8 in.) of permanent deformation to the bumper where contact with the vertical support and the U-channel base post occurred. The A-pillar on the driver's side was deformed from impact by the mailbox, and the windshield was cracked around the point of impact. The door post on the driver side was bent and the glass was broken out. There was also damage to the hood and grill and the right rear tire and rim. There was a dent in the gas tank, and there were scrape marks and a dent along the floor pan on the right side of the undercarriage caused by contact with the vertical support.

A summary of the test results is presented in Table 1. In the longitudinal direction, occupant impact velocity was 1.3 m/sec (4.3 ft/sec), and the highest 10-msec average ridedown acceleration was − 2.7 *g*. In the lateral direction, occupant impact velocity was 1.4 m/sec (4.5 ft/sec), and the highest 10-msec average ridedown acceleration was 4.6 *g*. The change in vehicle velocity at the loss of contact was 6.9 km/hr (4.3 mph).

Test 7147-13

The 1986 Yugo GV used in the first two tests was repaired and used for the third crash test. The vehicle struck the mailbox assembly at a speed of 103 km/hr (64.0 mph). On impact the mailbox shattered the windshield. The cantilever arm contacted the A-pillar on the passenger's side of the vehicle, and the mailbox assembly started to rotate away from the windshield and then separated from the vertical support. The mailbox assembly and the cantilever arm then went up and over the vehicle. The vehicle was traveling at 99.6 km/hr (61.9 mph) as it lost contact with the mailbox assembly. The windshield, which was held in place by a rubber grommet, separated from the vehicle. The detached windshield first went outward and upward, contacted the roof of the vehicle, and was partially on the roof of the vehicle before eventually sliding back inside the occupant compartment after the brakes on the vehicle were applied. The vehicle subsequently came to rest 100 m (327 ft) downstream from the point of impact.

The mailbox installation separated into several pieces, as shown in Figure 7. The cantilever arm and part of the wood board landed 54 m (177 ft) downstream and 1.4 m (4.5 ft) to the right of the point of impact. The severely deformed mailbox, part of the wood board, and the plastic newspaper tube came to rest 55 m (182 ft) downstream and 0.3 m (1 ft) to the left of the point of impact. The vertical support was only scraped, and the U-channel base post was not damaged or pushed back.

The vehicle (also shown in Figure 7) sustained moderate damage. There was 30 mm (1.2 in.) of permanent deformation to the A-pillar on the passenger's side of the vehicle, and the door post on the passenger's side was deformed at the location where the cantilever arm made contact. There was also a scratch located on the left rear section of the roof from contact by the detached cantilever arm as it went over the vehicle. The windshield was broken out and was lying on the floorboard of the vehicle. However, it should be noted that the windshield actually went outward and upward after separation from the vehicle and was partially on the roof of the vehicle before falling back into the occupant compartment. The detachment of the windshield from the vehicle could be partially attributed to the poor design of the windshield, which was held in place only with a rubber grommet. Most other vehicles have a more positive

TABLE 1 Crash Test Results

Test No.	Impact Speed, km/h (mph)	Occupant Impact Velocity, m/s (ft/s)		Ridedown Acceleration, g's		Comments
		Long.	Lateral	Long.	Lateral	
7147-11	35.1 (21.8)	1.9 (6.1)	No Contact	0.9	No Contact	
7147-12	104.9 (65.2)	1.3 (4.3)	1.4 (4.5)	-2.7	4.6	
7147-13	103.0 (64.0)	No Contact	1.2 (3.9)	No Contact	1.0	Windshield cracked and separated from vehicle
7147-14	101.0 (62.8)	0.9 (2.8)	No Contact	-0.3	No Contact	Windshield penetrated by mailbox assembly

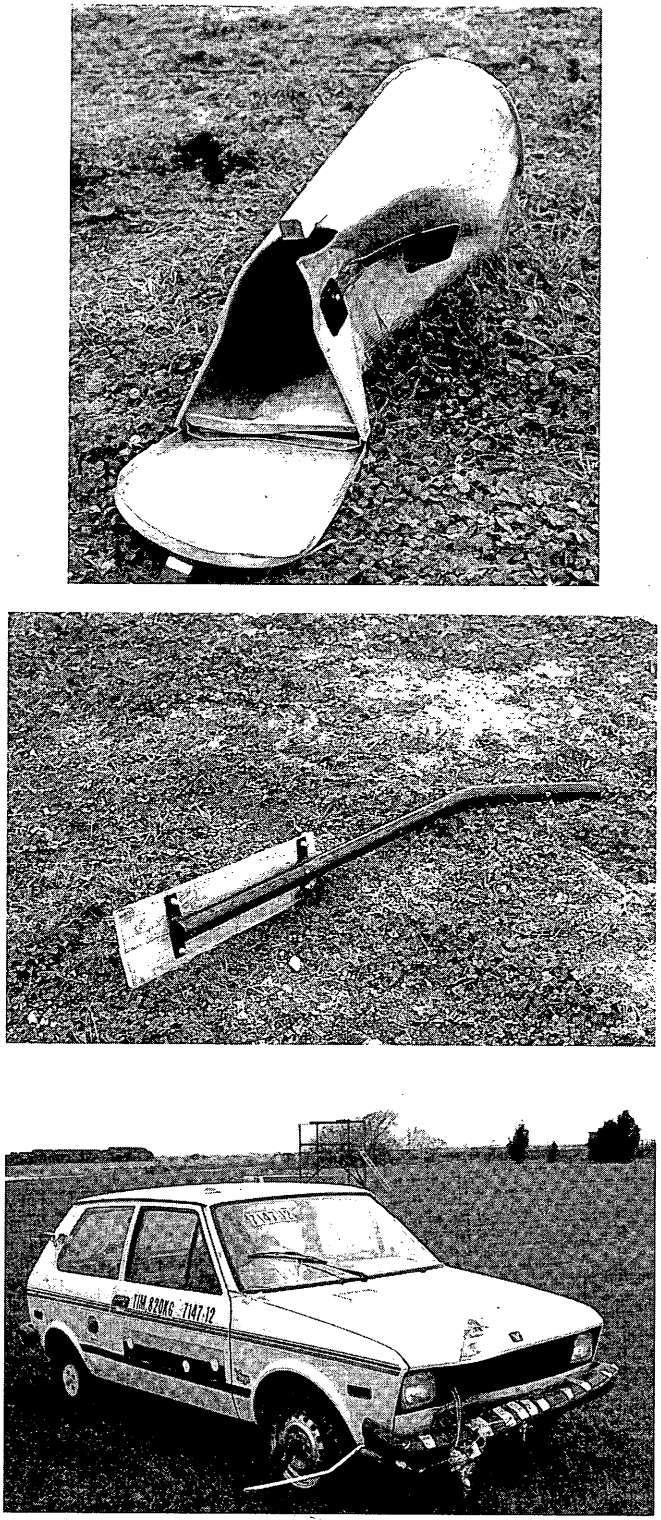


FIGURE 6 Mailbox installation (*top* and *middle*) and vehicle (*bottom*) after Test 7147-12.

mechanism for attaching the windshield to the vehicle. In additions previous crash tests caused damage to the A-pillar, which might have further weakened the attachment mechanism.

A summary of the test results is presented in Table 1. No occupant contact occurred in the longitudinal direction. In the lateral

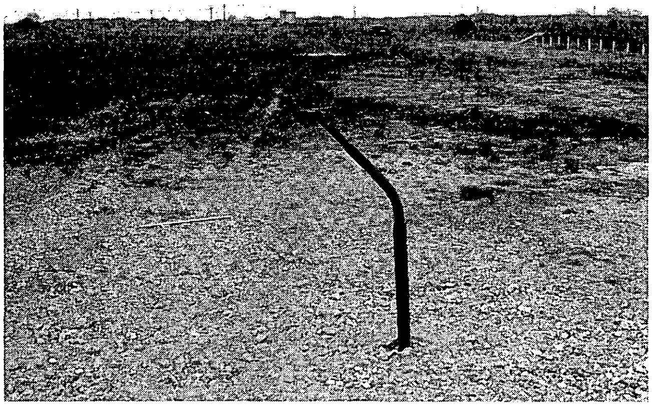
FIGURE 7 Mailbox installation (*top*) and vehicle (*bottom*) after Test 7147-13.

direction, occupant impact velocity was 1.2 m/sec (3.9 ft/sec), and the highest 10-msec average ridedown acceleration was 1.0 *g*. The change in vehicle velocity at the loss of contact was 3.4 km/hr (2.1 mph).

Test 7147-14

A 1989 Yugo GVL was used for the fourth crash test. The vehicle struck the triple mailbox assembly at a speed of 101 km/hr (62.8 mph). On impact the mailbox assembly shattered the windshield, and the first mailbox bounced up and struck the edge of the roof just above the windshield. The cantilever arm then contacted the A-pillar on the passenger's side of the vehicle, and the cantilever arm and mailbox assembly separated from the vertical support at 41 m/sec after impact. The cantilever arm and mailbox assembly intruded into the occupant compartment of the vehicle and rode along partially in the compartment and partially on the hood of the vehicle. The brakes on the vehicle were applied, and the vehicle subsequently came to rest 121 m (397 ft) downstream from the point of impact.

The test site and components of the mailbox test installation after the test are shown in Figure 8. The mailbox assembly was de-



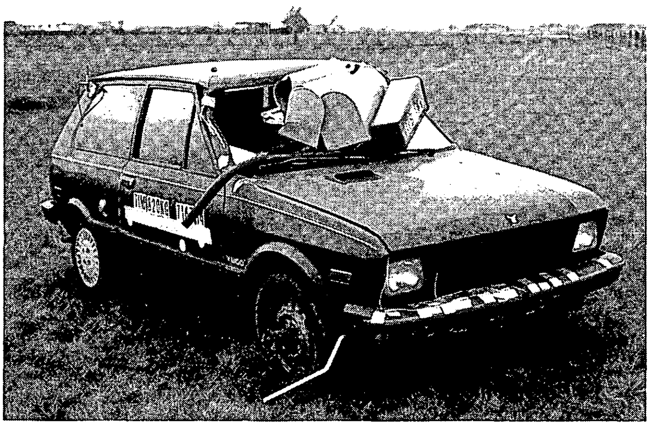


FIGURE 8 Mailbox installation (*top*) and vehicle (*bottom*) after Test 7147-14.

formed, but it remained attached to the cantilever arm and remained with the vehicle through final rest. The vertical support was only scraped, and the U-channel base post was bent slightly.

The vehicle sustained moderate damage around the windshield area, as shown in Figure 8. The mailbox assembly intruded into the occupant compartment through the windshield and remained partially in the compartment throughout the test period. The roof of the vehicle was deformed upward from the inside of the vehicle approximately 50 mm (2 in.). The passenger's side door was pushed out 40 mm (1.6 in.), and the glass was shattered. The A-pillar and door post on the passenger's side was also deformed.

In the longitudinal direction, occupant impact velocity was 0.9 m/sec (2.8 ft/sec), and the highest 10-msec average ridedown acceleration was -0.3 *g*. No occupant contact occurred in the lateral direction. The change in velocity at the loss of contact was not applicable since the mailbox assembly and the cantilever arm remained in contact with the vehicle throughout the test period.

SUMMARY OF FINDINGS AND DISCUSSION OF RESULTS

The Minnesota swing-away mailbox support with a single mailbox assembly was judged to have successfully met all evaluation criteria set forth in *NCHRP Report 350* (*1*) and the 1985 AASHTO *Standard Specification for Structural Supports for Highway Signs, Luminaries and Traffic Signals* (*2*).

The first two crash tests (Tests 7147-11 and 7147-12) involving impacts with the vertical supports of the mailbox installations with single mailbox assemblies showed occupant impact velocities and ridedown accelerations that were well below the preferred limiting values of 3 m/sec (11.8 ft/sec) and 15 *g*, respectively. No penetration or intrusion into the occupant compartment occurred. Debris from the test installation, which consisted of the cantilever arm and the mailbox assembly, remained close to the approximate path of the vehicle and did not pose any potential hazard to adjacent traffic. The vehicle remained stable during and after the impact sequence.

The third crash test (Test 7147-13) with the single mailbox assembly directly struck and damaged the windshield, but the windshield kept the mailbox assembly from intruding or penetrating into the occupant compartment. Damage to the windshield is normally not considered a desirable behavior since it could obstruct the driver's vision or otherwise cause the driver to lose control of the vehicle. However, given the need for a cantilever design because of the snow-plowing operation, damage to the windshield is considered an acceptable trade-off provided that there was no intrusion or penetration into the occupant compartment. It is recommended that the maximum size of mailbox used with the support be limited to size 1A or smaller.

The fourth crash test (Test 7147-14) with triple mailbox assembly was judged to have failed to meet the evaluation criteria set forth in *NCHRP Report 350* (*1*). The mailbox assembly shattered the windshield and substantially intruded and penetrated into the occupant compartment, which was judged to be unacceptable. It appeared that two factors contributed to the unsatisfactory performance: (*a*) the combined weight of the triple mailbox assembly and the cantilever arm was 19 kg (42 lb), which was more than double the weight of 8.8 kg (19.5 lb) for the single mailbox assembly, and (*b*) the width of the triple mailbox assembly allowed the mailbox assembly to impact and penetrate the windshield before the cantilever arm struck the A-pillar of the vehicle, which would have partially counteracted against the force of the mailbox assembly into the windshield. In light of the unsatisfactory performance of the triple mailbox assembly, the use of the swing-away mailbox support design should be limited to only a single mailbox assembly. At locations where multiple mailboxes are to be installed, it is recommended that each mailbox be installed on its own support and that they be spaced at least 36 in. (0.91 m) apart to allow for unrestricted functioning of the cantilever arm.

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Tennessee Bridge Rail to Guardrail Transition Designs

KING K. MAK, ROGER P. BLIGH, AND PETE FALKENBERG

Potential problems associated with the impact performance of guardrail to bridge rail transition designs currently used by the Tennessee Department of Transportation were identified through use of computer simulation. To alleviate these deficiencies, three alternative designs were developed for use as a retrofit to their existing transition installations and for new installations. In the first retrofit alternative, the first three 6-ft (1.83-m)-long W6 × 15 posts adjacent to the concrete parapet are replaced with 8-ft (2.44-m)-long W8 × 21 posts. The second design involves the addition of two 6-ft (1.83-m)-long W6 × 15 posts between the first three existing W6 × 15 posts to effectively reduce the post spacing to 1 ft 6.75 in. (0.48 m). The third retrofit design involves the addition of a lower C6 × 8.2 channel rub rail to the existing transition system. All three modified designs were crash tested and found to perform satisfactorily in accordance with the recommended guidelines presented in *NCHRP Report 230.* Since the impact performances of the three systems were essentially the same, the choice of which design to use as a retrofit in the field becomes a consideration of cost and site-specific requirements. Details of these three alternative transition designs and the results of the full-scale crash tests are presented.

A study was undertaken by the Texas Transportation Institute to analyze and evaluate the impact performances of various bridge rail, guardrail, transition, and end treatment designs currently in use by the Tennessee Department of Transportation (TDOT). The results of an evaluation of the existing TDOT guardrail to bridge rail transition design and the effort to design, develop, and crash test various retrofit transition designs are presented here.

TDOT currently uses a steel post design for approach guardrails at bridge ends. The standard steel post system, shown in Figure 1, consists of a 25-ft (7.62-m) section of 10-gage W-beam mounted at a height of 27 in. (68.6 cm) on six W6 × 15 structural steel posts embedded 44 in. (1.12 m) and spaced at a reduced post spacing of 3 ft 1.5 in. (0.95 m). In addition, the first three posts upstream from the end of the concrete bridge parapet have ¼-in. × 8-in. × 24-in. (0.64-cm × 20.3-cm × 61.0-cm) steel soil plates welded 5 in. (12.7 cm) below the ground surface. No W-beam backup plates are specified beyond the first W6 × 15 post in the transition, at which point the post spacing is reduced 3 ft 1.5 in. (0.95 m).

One of the most common parapets to which this transition is attached is shown as Detail A in Figure 1. This design corresponds to TDOT standard drawing K-38-151. The wing post is a vertical concrete wall 27 in. (0.69 m) high and 12 in. (30.5 cm) thick. The end of the wall tapers away from the roadway to a thickness of 3 in. (7.6 cm).

The existing transition design connected to a vertical concrete parapet was evaluated by using the Barrier VII computer simulation program (*1*). The Barrier VII computer simulation model is a two-dimensional simulation program that models vehicular impacts with deformable barriers. The program employs a sophisticated barrier model that is idealized as an assemblage of discrete structural members possessing geometric and material nonlinearities. It has been used successfully to simulate impacts with a variety of flexible barriers, including transitions from flexible to rigid barriers (*2–4*).

The simulation results indicated that this transition design would exhibit undesirable impact performance. Predicted values for maximum dynamic rail deflection and wheel overlap on the end of the flared vertical concrete wing post were 12 in. (30.5 cm) and 4.3 in. (10.9 cm), respectively. With reference to Figure 1, Detail A, it can be seen that the predicted extent of wheel contact projects beyond the back edge of the parapet. Although not confirmed with a full-scale test, contact of this magnitude was considered unacceptable because of the high probability of the wheel assembly hooking or snagging abruptly on the end of the concrete wingpost. Such behavior could lead to severe deceleration of the vehicle or other undesirable results.

In view of the deficiencies identified with the current transition system, it was necessary to investigate alternative designs for potential use by TDOT. In recent years FHWA has issued two technical advisories (TAs) on the subject of guardrail transitions. These TAs provide information on new and retrofit transition systems that have been successfully crash tested (*5,6*). In TA T5040.26 (*5*), several transition designs appropriate for attachment to a vertical parapet with a curved, flared, or tapered end were presented. TA T5040.34 (*6*) presented several additional transition designs appropriate for attachment to concrete safety-shaped bridge parapets.

The parapet commonly used by TDOT possesses some general similarities to the vertical curved-back and vertical flared-back concrete bridge rail ends detailed in TA T5040.26. However, the exposed ends of these parapets are offset 18 in. (45.7 cm) and 16 in. (40.6 cm), respectively, from the traffic face of the rail. As shown in Figure 1, the geometry of the TDOT parapet is much more severe, with the end tapered only 9 in. (22.9 cm) from the face of the rail. For this reason it was concluded that the impact performance of a system consisting of one of the transition designs in the FHWA TAs attached to the TDOT parapet could not be inferred from previous test results and that additional testing was warranted. Furthermore, although TDOT has the option of changing its standard bridge end parapet details and adopting one of the TA designs for new construction applications, it was considered essential that one or more designs be developed and tested for retrofitting the numerous installations that currently exist in the field.

A significant simulation study was undertaken in an effort to identify design modifications that would alleviate the identified deficiencies and improve the impact performance of TDOT's existing transition system. When selecting potential design modifications, several factors were considered including ease of retrofitting

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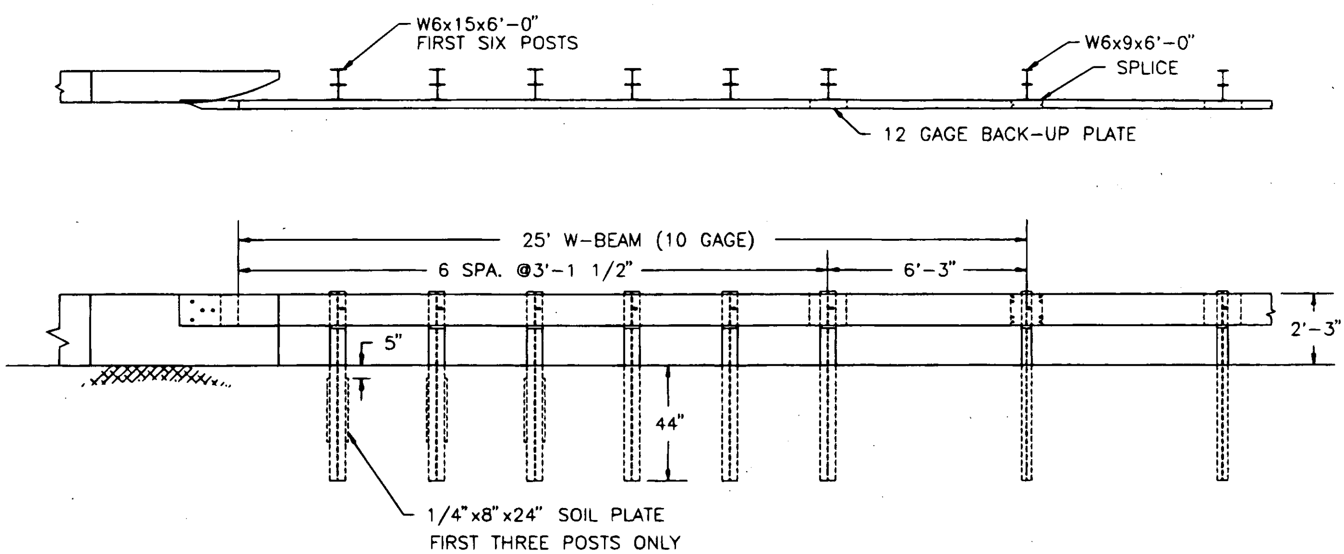

FIGURE 1 Standard TDOT steel post transition to vertical concrete parapet.

existing installations and use of standard hardware items. The objective was to increase lateral barrier stiffness and thereby reduce maximum barrier deflections and wheel snagging on the end of the bridge parapet. The key parameters that were investigated included beam strength, post size, post embedment depth, and post spacing. Although none of the designs contained in the FHWA TAs were directly adopted, many of the design details of these systems were used in developing candidate designs for use with TDOT's vertical concrete parapet.

On the basis of this analysis, three alternative retrofit transition designs were developed for consideration by TDOT. Details of these three retrofit designs are discussed in the following sections.

DESIGN DETAILS

System 1: Larger Post Size and Embedment Depth

In the first retrofit alternative, the first three 6-ft (1.83-m)-long W6 × 15 posts adjacent to the concrete parapet are replaced with 8-ft (2.44-m)-long W8 × 21 posts. These larger posts have an embedment depth of 68 in. (1.73 m), compared with the standard embedment depth of 44 in. (1.12 m). The next three posts upstream in the transition section are the standard 6-ft (1.83-m)-long W6 × 15 structural steel posts with a standard embedment depth of 44 in. (1.12 m) at the existing post spacing of 3 ft 1.5 in. (0.95 m).

The use of the W8 × 21 posts allows greater stiffness to be achieved through increased embedment depths. When the 8-ft (2.44-m)-long W8 × 21 posts were used in lieu of the standard 6-ft (1.83-m)-long W6 × 15 posts, computer simulation indicated a reduction in the amount of wheel overlap on the end of the concrete parapet of 2.5 in. (6.4 cm). Note that soil plates are not used with the W8 × 21 posts since studies have shown that soil plates contribute very little to the post stiffness (*2*).

The rail element consists of a 25-ft (7.62-cm) section of single 12-gage W-beam mounted at a height of 27 in. (68.6 cm). Although the existing standard TDOT transition uses a 10-gage W-beam, TDOT expressed an interest in testing with a 12-gage rail to reduce inventory and eliminate the possibility of construction and maintenance crews installing a rail of improper thickness in the transition. The simulation results indicated and previous testing has shown that a single 12-gage W-beam rail is capable of withstanding the severe dynamic loading that occurs during a transition impact (*7*). However, the use of a single W-beam requires that backup plates be used at all nonsplice post locations. The importance of backup plates in steel post guardrail systems has been demonstrated in full-scale crash testing (*8*).

It should also be noted that a 6-in. (15.24-cm)-inner diameter, 12-in (30.5-cm)-long Schedule 40 pipe is placed between the rail and the flared portion of the concrete parapet. The purpose of this steel pipe is to help minimize deflections and prevent local yielding of the W-beam rail around the end of the parapet by acting as a controlled, collapsible spacer. The spacer tube is connected to the W-beam rail with a single ⅝-in. (1.59-cm) button-head bolt. Other than the use of the spacer pipe, there were no changes to the connection of the W-beam rail to the bridge parapet from the original TDOT design. Details of the System 1 retrofit installation are shown in Figure 2.

System 2: Reduced Post Spacing

The second candidate retrofit transition design incorporates W6 × 15 structural steel posts with two sets of reduced post spacing. Adjacent to the concrete parapet, the post spacing is 1 ft 6.75 in. (0.48 m), which is followed by a post spacing of 3 ft 1.5 in. 0.95 m). In a retrofit situation this alternative would simply involve placing two additional W6 × 15 posts between the first three existing posts. These two additional intermediate posts are not connected to the W-beam but are simply installed with the face of the blockout adjacent to the back side of the rail. The embedment depth for all of the W6 × 15 posts is the standard 44 in. (1.12 m). Note that, although the existing W6 × 15 posts incorporate soil plates, the use of soil plates on these posts and on the additional posts is not required. Previous studies have shown that the addition of soil plates on W6 × 15 posts does little to increase either the stiffness or the maximum capacity of the soil-post system (*2*).

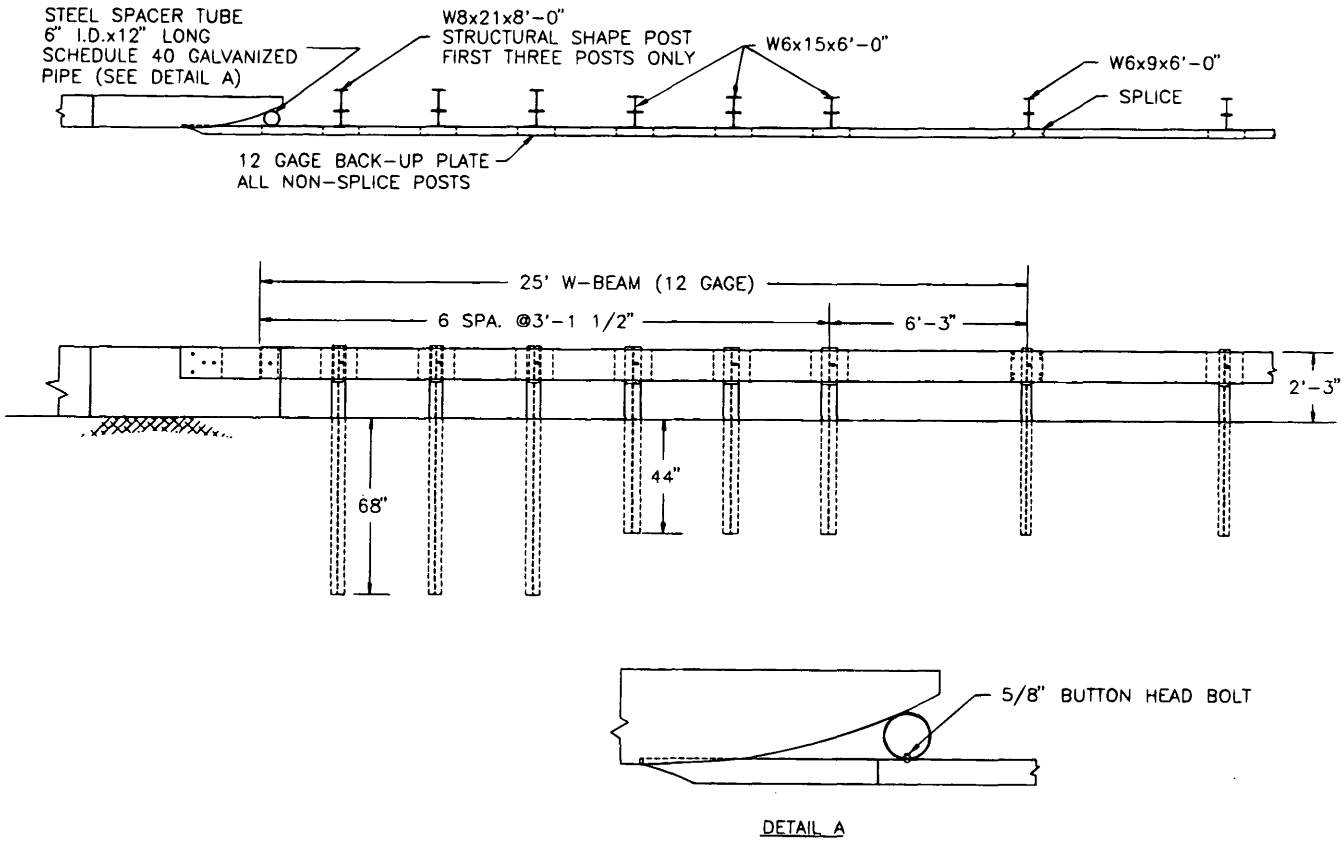

FIGURE 2 Modified steel post transition to vertical concrete parapet—larger post size and embedment depth option.

Based on the crash test results of System 1, a nested 12-gage W-beam rail was incorporated into the design of Transition Systems 2 and 3 to further enhance impact performance. Simulation results indicated that the nested rail would decrease the amount of wheel overlap on the end of the parapet by approximately 0.5 in. (1.27 cm) and, more importantly, would reduce the degree of localized yielding of the rail at the spacer pipe. No backup plates are required in the region of the nested rail.

As with the System 1 design, a 6-in. (15.24-cm)-diameter steel spacer pipe is used between the nested W-beam rail and the flared wall of the concrete parapet. Otherwise, the connection details remain unchanged and the nested W-beam rail is terminated with a standard W-beam terminal connector. Details of the System 2 design are shown in Figure 3.

System 3: Rub Rail

The third alternative transition design uses a C6 × 8.2 steel channel as a lower rub rail element to help mitigate the amount of wheel contact on the end of the concrete parapet. This rub rail is anchored to the concrete parapet and is also connected to the front flanges of the steel guardrail posts. The upstream end of the rub rail is terminated behind the fifth post in the transition to minimize the potential for spearing or wheel snagging during upstream impacts. The posts and post spacing are identical to those of the standard TDOT transition, with W6 × 15 posts embedded 44 in. (1.12 m) and spaced at 3 ft 1.5 in. (0.95 m). Once again, although soil plates will be present in the field on existing installations, their use is not required during repair or new construction applications.

Similar to System 2, this design uses a 25-ft (7.62-m) section of nested 12-gage W-beam rail adjacent to the concrete parapet and a 6-in. (15.24-cm)-diameter spacer pipe between the nested W-beam rail and the flared portion of the concrete parapet. Details of the System 3 retrofit transition installation are shown in Figure 4. Each of these three alternative retrofit designs was crash tested and evaluated, and the results are presented as follows.

CRASH TEST RESULTS

The test installation consisted of a simulated vertical concrete bridge parapet with a 9-in. (22.9-cm) flare away from the roadway. Details of the parapet conform to TDOT standard drawing K-38-151 and are shown in Figure 1. Attached to the vertical parapet is the 25-ft (7.62-m) transition section. The posts in the transition were placed by drilling and backfilling with a standard strong soil as defined in *NCHRP Report 230* (*9*). Upstream from the transition section is a standard G4(1S) guardrail consisting of a 12-gage W-beam mounted at 27 in. (68.6 cm) on W6 × 9 steel posts spaced at 6 ft 3 in. (1.91 m). The total length of the approach guardrail was 75 ft (22.9 m), with the upstream end terminated with a standard breakaway cable terminal end terminal.

Each of the three alternative transition designs was crash tested and evaluated in accordance with the test procedures and the evaluation criteria outlined in *NCHRP Report 230* (*9*). As recommended

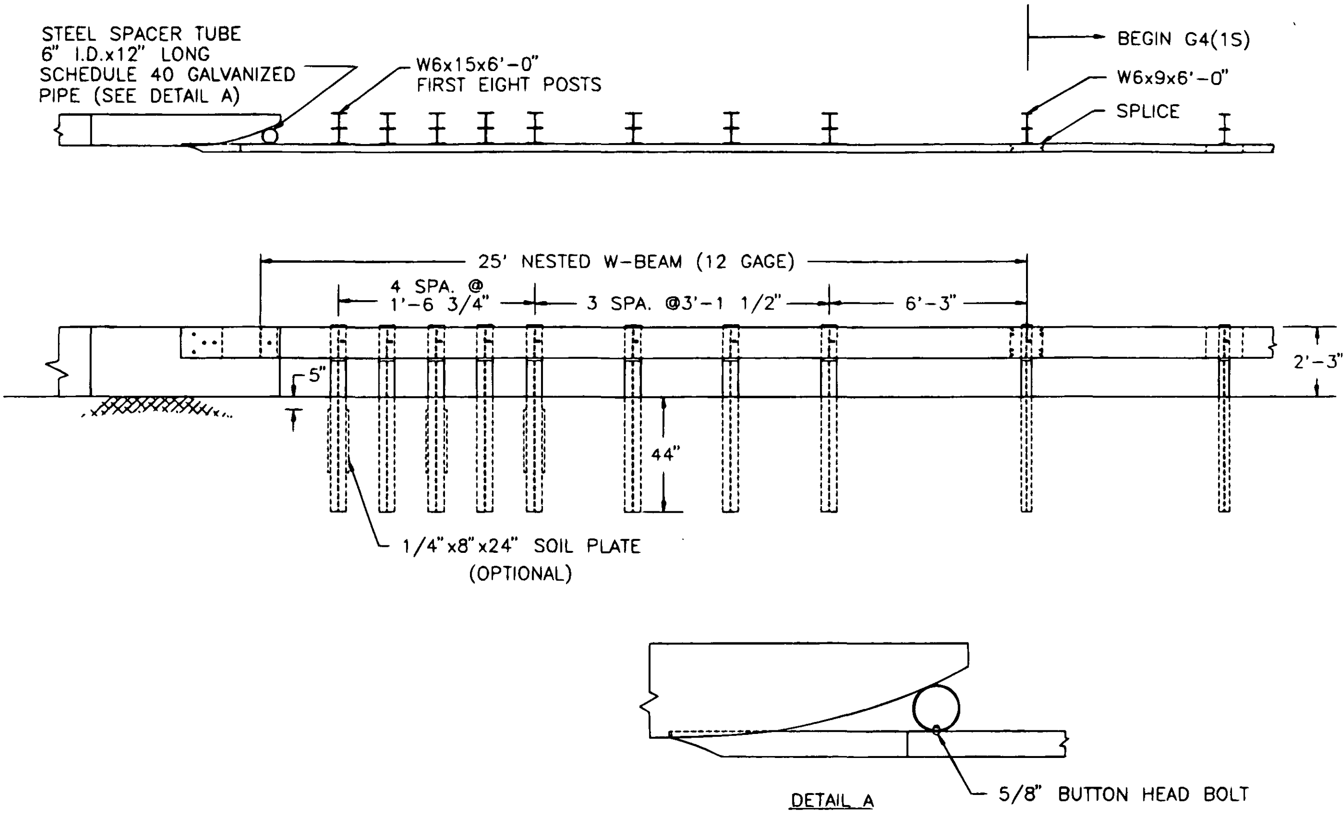

FIGURE 3 Modified steel post transition to vertical concrete parapet—reduced post spacing option.

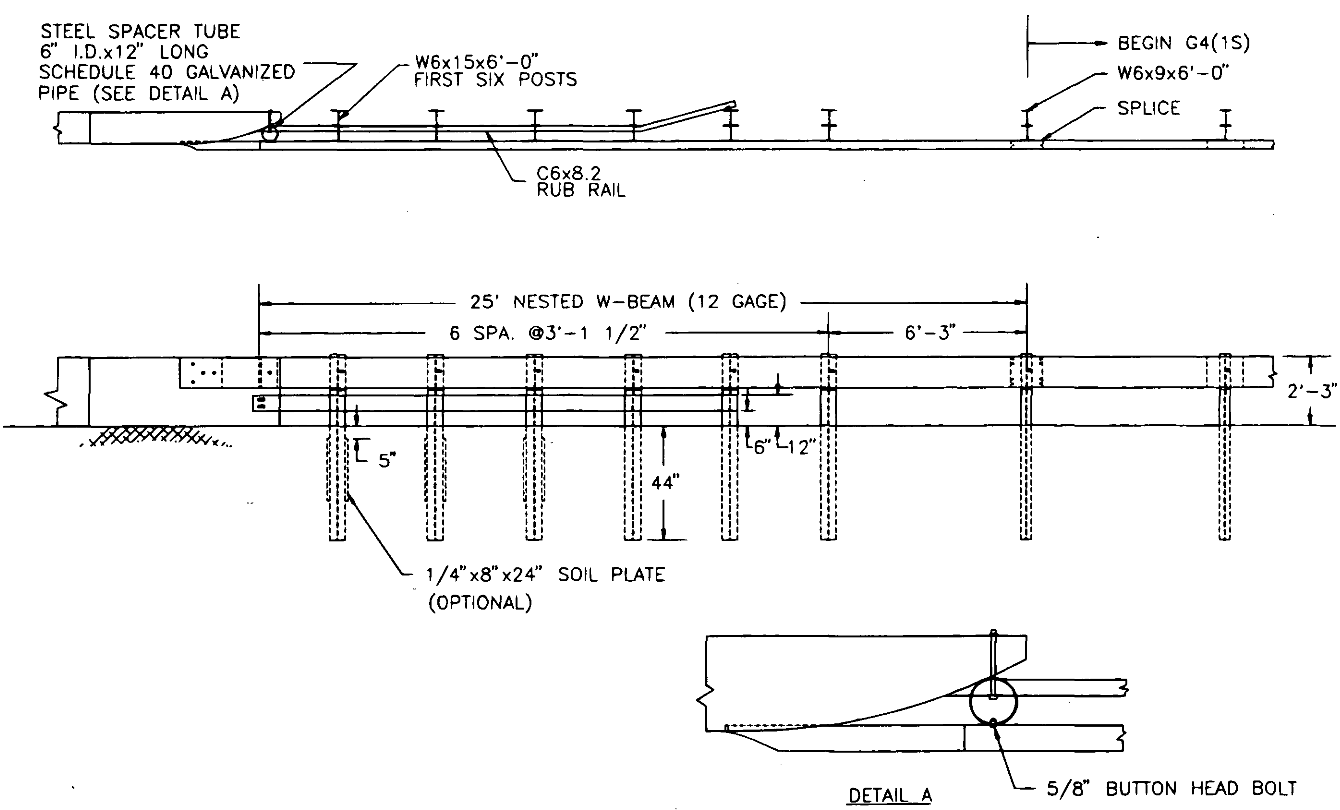

FIGURE 4 Modified steel post transition to vertical concrete parapet—rub rail option.

in *NCHRP Report 230*, each of the three alternative designs was crash tested with a 4,500-lb (2,041-kg) vehicle striking the transition section at a speed of 60 mph (96.6 km/hr) and an angle of 25 degrees. The point of impact for all three transition designs was selected at 6 ft (1.83 m) from the end of the concrete wing post, which was determined to be the critical impact location for these transition systems based on Barrier VII computer simulation results.

System 1: Larger Post Size and Embedment Depth

The System 1 test installation is shown in Figure 5. A 1982 Oldsmobile Ninety-Eight impacted the transition 6.0 ft (1.8 m) upstream from the end of the concrete parapet at 61.4 mph (98.8 km/hr) and at an angle of 25.1 degrees. Although significant wheel contact with the parapet end was observed, the vehicle was successfully redirected. The spacer pipe performed as designed, preventing excessive deflections of the W-beam along the flared portion of the parapet. Although there was some evidence of localized yielding of the W-beam around the spacer pipe, the pipe collapsed in a controlled manner before allowing any significant pocketing or snagging to occur.

As the vehicle redirected, the rocker panel at the base of the A-pillar contacted the flared section of the parapet, causing some buckling and wrinkling of the floor pan beneath the passenger seat and near the transmission housing. The vehicle lost contact with the rail at approximately 0.34 sec after impact, traveling at a speed of 45.3 mph (72.9 km/hr) and at an exit angle of 8.2 degrees.

The damage to both the test installation and the vehicle is shown in Figure 6. The transition and concrete parapet sustained only minor damage. There was residual deformation to the rail in the area of the first three posts, with the maximum permanent rail deformation being 5.0 in. (12.7 cm). The spacer pipe positioned between the W-beam rail and the flared portion of the parapet collapsed approximately 1 in. (2.54 cm).

The damage sustained by the test vehicle was substantial. The maximum crush was 16.0 in. (40.6 cm) at the right front corner of the vehicle. The right front wheel and control arm were bent and pushed rearward 15.3 in. (38.7 cm) because of contact with the end and sloped face of the concrete parapet. The front end of the vehicle was shifted to the left 3.0 in. (7.6 cm). In addition, the subframe was bent, the floor pan was buckled, and the windshield was broken.

In summary, the transition was judged to have met the performance criteria set forth in *NCHRP Report 230* (*9*). The test vehicle

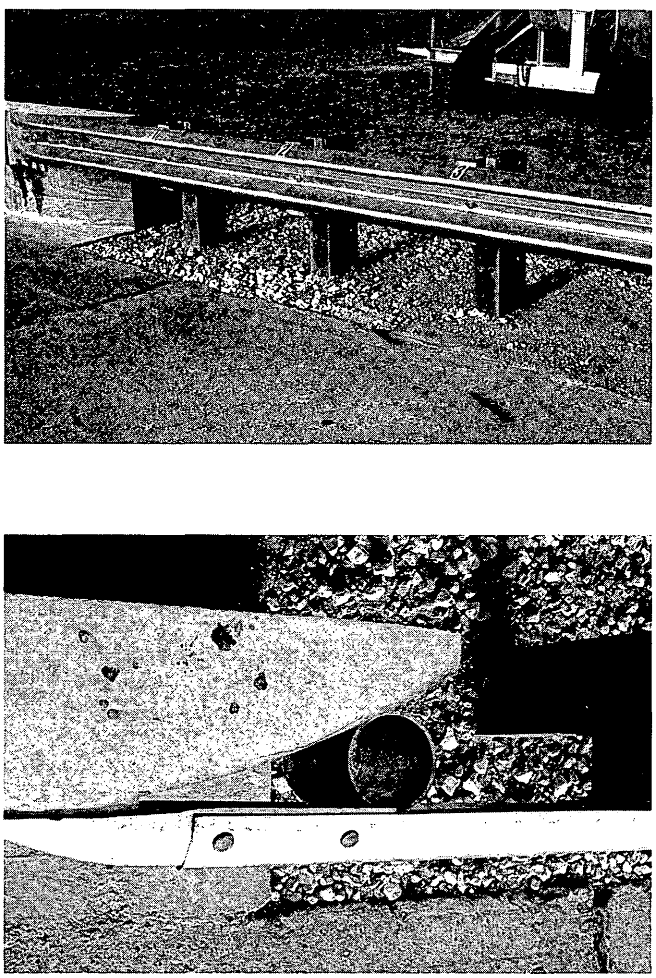

FIGURE 5 Tennessee large post transition before testing of System 1.

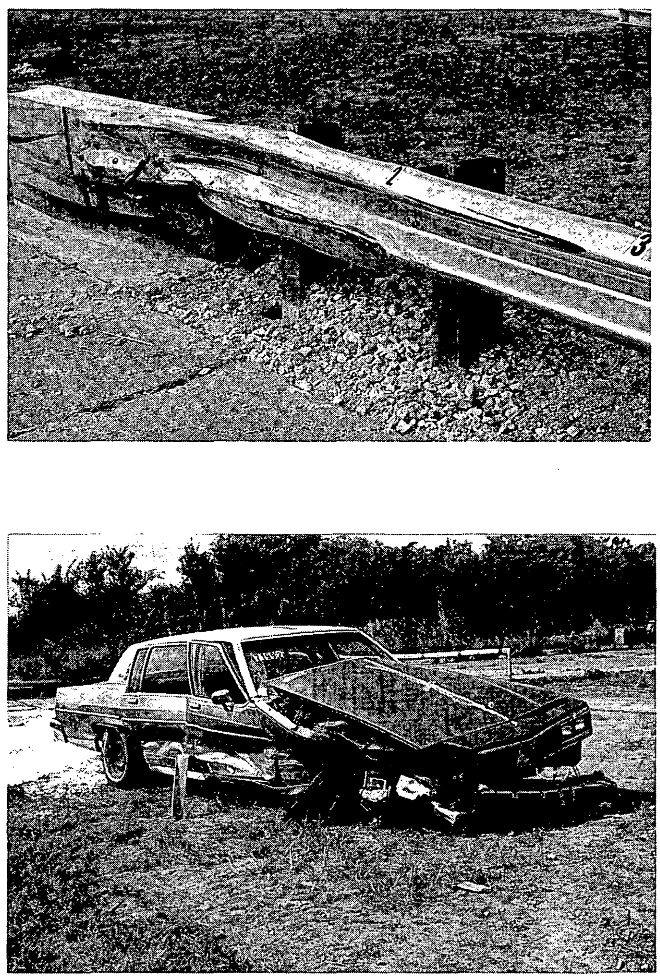

FIGURE 6 Barrier (*top*) and vehicle (*bottom*) damage after testing of System 1.

remained upright and stable during the impact period and after leaving the installation, and there was no debris from the vehicle or barrier that might present undue hazard to other traffic. Damage to the transition was relatively minor, with no apparent structural damage to the concrete bridge parapet. Although damage to the test vehicle was severe, there was minimal intrusion into the occupant compartment. Contact of the subframe with the flared wall of the concrete parapet caused the floor pan of the vehicle to buckle. However, this deformation was primarily concentrated under the front passenger's seat and was not judged to pose a significant hazard to the occupant.

System 2: Reduced Post Spacing

The System 2 test installation is shown in Figure 7. In this test a 1980 Oldsmobile Ninety-Eight struck the transition 6.0 ft (1.8 m) upstream from the end of the concrete parapet at 62.0 mph (99.8 km/hr) and at an angle of 24.4 degrees. Shortly after impact, the right front wheel rotated about the spindle assembly, allowing it to fold under the rail and contact the first two guardrail posts upstream from the end of the concrete parapet. As the vehicle progressed along the transition, the right front wheel contacted the end of the parapet and the subframe at the base of the A-pillar contacted the flared face of the parapet. Although this contact was significant, the vehicle remained stable and was successfully redirected. The vehicle lost contact with the rail approximately 0.34 sec after impact, traveling at a speed of 44.3 mph (71.2 km/hr) and at an exit angle of 13.5 degrees.

Figure 8 shows the damage to the barrier and vehicle after the test. Residual deformation of the guardrail occurred in the vicinity of the first six posts. The maximum permanent rail deformation was measured to be 4.0 in. (10.2 cm). Vehicle tire marks were noted on the outside flanges of Posts 1 and 2 and on the end of the concrete bridge parapet. The introduction of additional posts in the wheelpath of the vehicle permitted more wheel snagging to occur, which in turn damaged the wheel and resulted in contact with the end of the parapet. The spacer pipe experienced 2.5 in. (6.53 cm) of permanent deformation and performed as intended.

The test vehicle sustained extensive damage. The maximum recorded crush was 20.0 in. (50.8 cm) at the right front corner of the

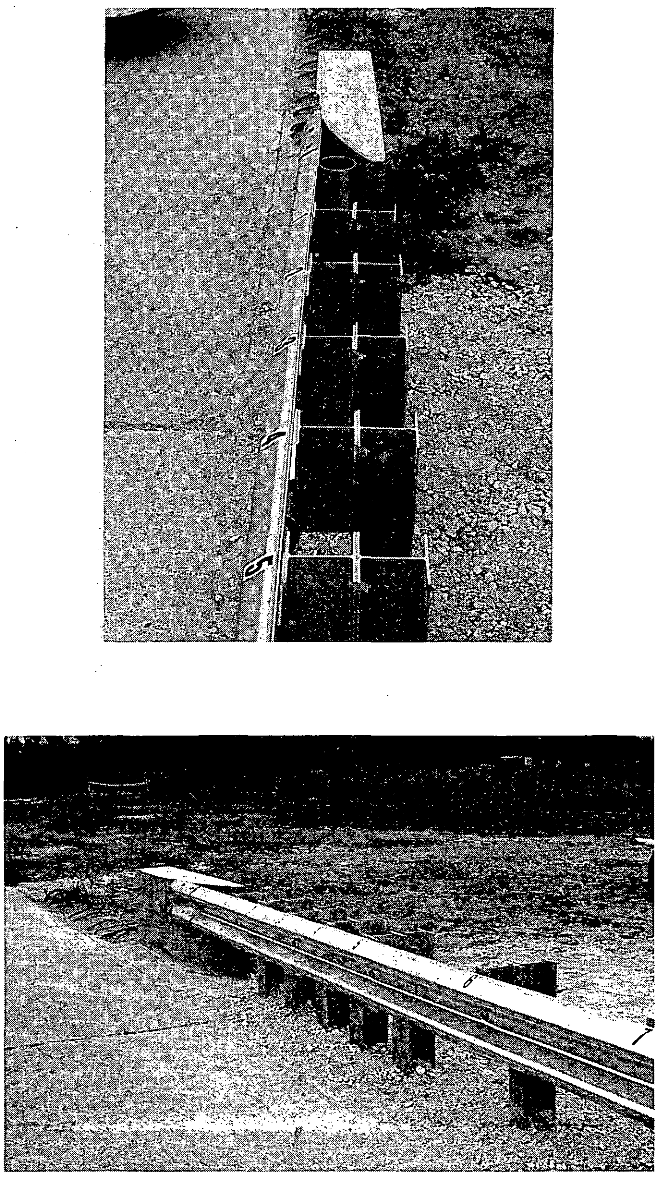

FIGURE 7 Tennessee reduced post spacing transition before testing of System 2.

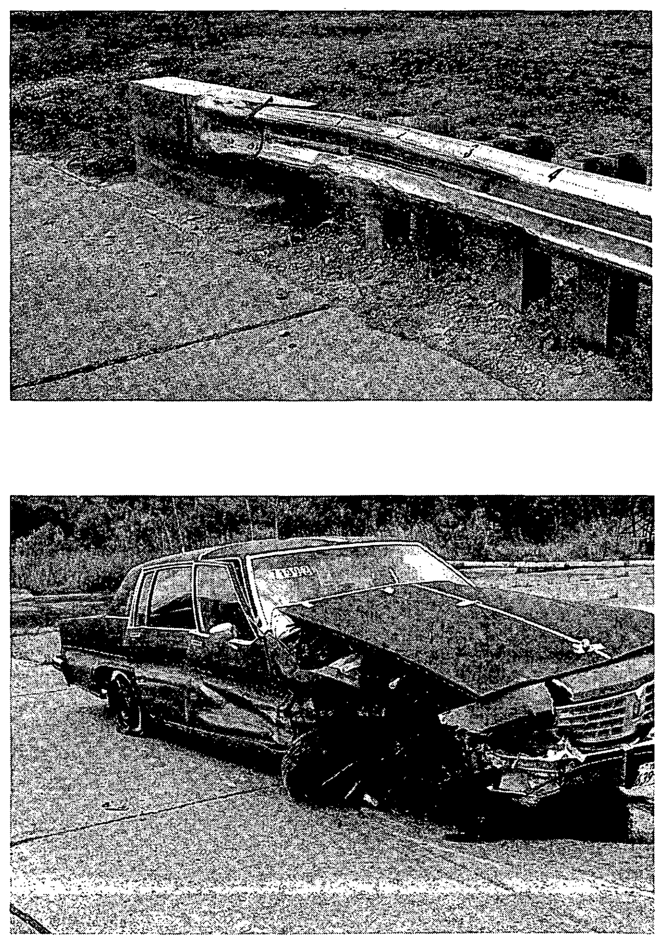

FIGURE 8 Barrier (*top*) and vehicle (*bottom*) damage after testing of System 2.

vehicle. The right front wheel and control arm were bent and pushed rearward a distance of 11.0 in. (27.9 cm). The entire front end of the vehicle was shifted to the left 2.5 in. (6.4 cm). In addition, the right front brake disc was pulled off the spindle, the subframe was bent, and the windshield was broken. Contact of the subframe with the face of the concrete parapet resulted in some minor buckling or wrinkling of the floor pan. The entire right side of the vehicle was dented and scraped by contact with the nested W-beam rail.

In summary, the results of this test were judged to be in compliance with the recommended performance criteria for transitions as presented in *NCHRP Report 230* (*9*). The installation successfully contained and redirected the impacting vehicle. Although not required in the evaluation of a strength test, all occupant risk values were within the maximum acceptable limits set forth in *NCHRP Report 230* for a survivable impact. Damage to the test installation was minor, with no apparent structural damage to the concrete bridge parapet. The test vehicle sustained severe damage, but there was no intrusion into the occupant compartment. There was some buckling of the floor pan under the passenger's seat due to the subframe contacting the side of the concrete parapet. However, this buckling was considered minor in nature and did not constitute a severe hazard for the occupant.

System 3: Rub Rail

The System 3 test installation is shown in Figure 9. A 1984 Cadillac Coupe DeVille struck the transition installation 6.0 ft (1.8 m) upstream of the end of the concrete parapet at 61.0 mph (98.2 km/hr) and at an angle of 24.7 degrees. The rub rail prevented the right front tire from snagging on the end of the concrete bridge parapet, and the vehicle was successfully redirected. However, contact of the subframe and wheel with the flared face of the parapet resulted in some minor buckling of the floor pan on the passenger side of the vehicle and extensive damage to the wheel assembly. The vehicle lost contact with the rail approximately 0.30 sec after impact, traveling at a speed of 44.8 mph (72.1 km/hr) and at an exit angle of 10.5 degrees.

Damage to the transition and vehicle after testing of System 3 is shown in Figure 10. The installation sustained relatively minor damage for an impact of this severity. There was residual deformation to the guardrail in the vicinity of the first three posts. The maximum permanent deformation along the W-beam rail was 4.5 in. (11.4 cm). Maximum permanent deformation to the rub rail was 2.0 in. (5.1 cm). The steel spacer pipe collapsed 1.5 in. (3.81 cm).

Damage to the test vehicle was considerable. The maximum crush was 19.0 in. (48.3 cm) at the right front corner of the vehicle. The right front wheel and control arm were severely bent and pushed rearward 11.0 in. (27.9 cm). The entire front end of the vehicle was shifted to the left 4.5 in. (11.4 cm). In addition, the subframe was bent, the floor pan was buckled, and the windshield was broken.

In summary, the installation successfully contained and redirected the impacting vehicle. Although not required in the evaluation of the strength test, all of the occupant risk criteria were within maximum acceptable values, further indicating that the vehicle was smoothly redirected without experiencing any severe decelerations. Damage to the transition was minor in nature, with no apparent structural damage to the concrete bridge parapet. Damage to the vehicle was severe, but acceptable for an impact of this severity. There was no intrusion into the occupant compartment, and the buckling of the floor pan that occurred did not constitute a severe hazard to the occupants.

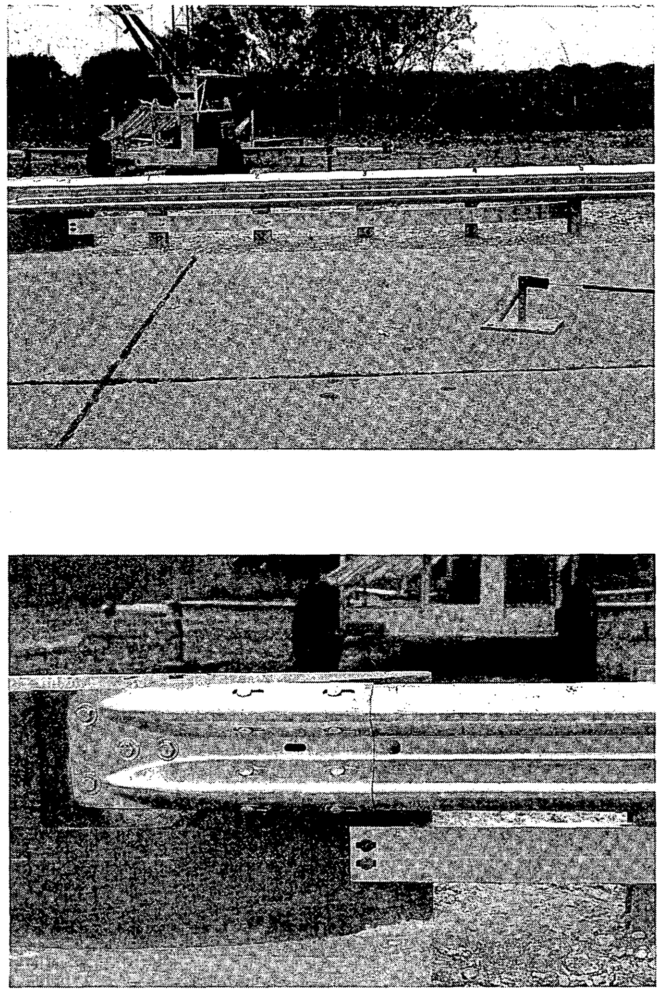

FIGURE 9 Tennessee transition with rub rail before testing of System 3.

CONCLUSIONS AND RECOMMENDATIONS

Simulation results indicated that TDOT's standard guardrail to bridge rail transition design would exhibit undesirable impact performance. Three alternative retrofit transition designs were developed to improve the impact performance of the existing system. Significant details of these systems are as follows:

- System 1: Larger post size and embedment depth. The first three 6-ft (1.83-m)-long W6 × 15 posts in the standard design are replaced with 8-ft (2.44-m)-long W8 × 21 posts.
- System 2: Reduced post spacing. Two 6-ft (1.83-m)-long W6 × 15 steel posts are added between the first three existing W6 × 15 posts to effectively reduce the post spacing adjacent to the parapet from 3 ft 1.5 in. (0.95 m) to 1 ft 6.75 in. (0.48 m).
- System 3: Addition of a lower C6 × 8.2 channel rub rail.

In addition, all three of these retrofit designs use a 6-in. (15.2-cm)-diameter spacer pipe between the W-beam and the flared face of the

FIGURE 10 Barrier (*top*) and vehicle (*bottom*) damage after testing of System 3.

TABLE 1 Test Results

Description	Test 1 (7199-2)	Test 2 (7199-3)	Test 3 (7199-5)
Test Vehicle	1982 Oldsmobile Ninety-Eight	1980 Oldsmobile Ninety-Eight	1984 Cadillac Coupe DeVille
Test Weight, lb (kg)	4500 (2041)	4500 (2041)	4500 (2041)
Impact Speed, mi/h (km/h)	61.4 (98.8)	62.0 (99.8)	61.0 (98.2)
Impact Angle, deg.	25.1	24.4	24.7
Exit Speed, mi/h (km/h)	45.3 (72.9)	44.3 (71.2)	44.8 (72.1)
Exit Angle, deg.	8.2	13.5	10.5
Velocity Change[a], mi/h (km/h)	16.1 (25.9)	17.7 (28.6)	16.2 (26.1)
Occupant Impact Velocity[b]			
Longitudinal, ft/s (m/s)	18.1 (5.5)	16.5 (5.0)	12.6 (3.8)
Lateral, ft/s (m/s)	-28.3 (8.6)[c]	-21.5 (6.5)[c]	-22.6 (6.9)[c]
Occupant Ridedown Acceleration[b]			
Longitudinal, g	-8.6	-13.1	8.4
Lateral, g	11.5	15.6[d]	16.2[d]
Length of Rail Contact, ft (m)	14.2 (4.3)	14.7 (4.5)	14.2 (4.3)
Maximum Permanent Rail Deflection, in (cm)	5.0 (12.7)	4.0 (10.2)	4.5 (11.4)
Maximum Vehicle Crush, in (cm)	16.0 (40.6)	20.0 (50.8)	19.0 (48.3)

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

[b] According to NCHRP Report 230 guidelines, the occupant risk criteria are not applicable for the 4500-lb passenger car crash test.

[c] Greater than recommended value of 20 ft/sec (6.1 m/sec), but less than acceptable limit of 30 ft/sec (9.1 m/sec).

[d] Greater than recommended value of 15g, but less than acceptable limit of 20 g.

concrete parapet to help reduce deflections and to minimize wheel and vehicle contact on the end of the parapet. Also, nested 12-gage W-beam rails were used for Systems 2 and 3 and are recommended for use with all three transition designs.

These three designs were evaluated through a series of full-scale crash tests, the results of which are summarized in Table 1. All three designs were judged to be in compliance with the recommended performance criteria for transitions presented in *NCHRP Report 230* (*9*). These designs provide an acceptable retrofit for the standard TDOT steel post approach guardrail attached to a tapered vertical concrete parapet. Although not required for the evaluation of a strength test, such as those conducted on transitions, occupant risk criteria are presented for information purposes and for comparison of the results with the results obtained from tests of other designs. As shown in Table 1, although some of these values are above the recommended limits, all of the values are below the maximum acceptable limits set forth in *NCHRP Report 230*.

The impact severity of System 1 (larger post size and embedment depth) was found to be slightly greater than those of the other two systems. This difference in performance could likely be attributed to the use of a single W-beam rail for this system, whereas nested W-beam rails were used with the other two systems. It is believed that the performance of System 1 would have been comparable to those of the other two systems had a nested W-beam rail been used.

The additional posts present in System 2 (reduced post spacing) allowed more wheel contact to occur, thereby slightly increasing the impact severity. System 3 (rub rail) prevented the wheels from contacting the end of the parapet and therefore provided slightly better impact performance than those provided by the other two alternative designs.

Since the impact performances of all three systems were essentially the same, the choice of which alternative design to use in the field becomes primarily a consideration of economics and site-specific requirements. The reduced post spacing option (System 2) may be the most economical retrofit design since it does not require any modification to the existing posts in the transition. However, the reduced post spacing severely decreases the clear space between posts, which may pose a problem at sites with bridge end drainage. The other systems retain the existing post spacing of 3 ft 1.5 in. (0.95 m) but require some modifications to the installation. For the large post alternative (System 1) the first three posts are replaced, and the rub rail alternative requires the drilling of holes in the concrete parapet (and in the posts if holes are not already predrilled) to accommodate the channel rub rail.

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Improved Breakaway Utility Pole, AD-IV

DEAN C. ALBERSON AND DON L. IVEY

Performance-tested breakaway utility poles have been available for almost a decade. The Texas Transportation Institute developed the Hawkins Breakaway System for FHWA under a contract completed in 1985. Design modifications to decrease the tolerance requirements of the upper hinge connection and the base connection have been completed by the Texas Transportation Institute. These modifications have reduced the amount of material used in the base connection, reduced the machining cost for the upper hinge straps, and significantly reduced the maintenance procedures for the upper connection. In turn initial costs and maintenance costs have been reduced. The new design, AD-IV, was subjected to three pendulum tests and was crash tested with a 1,800-lb automobile at 60 mph. AD-IV meets the test evaluation criteria of *NCHRP Report 230*. FHWA granted approval of the system on June 17, 1993.

The first practical structural system that can be used to convert a timber utility pole into a breakaway structure was developed by the Texas Transportation Institute for FHWA. This work was completed in 1985. The result, the FHWA Breakaway Pole System or the Hawkins Breakaway System (HBS), met both the requirements of *NCHRP Report 230* (*1*) and the requirements of utility companies (*2,3*).

With FHWA leadership, HBS has now been implemented in Kentucky and Massachusetts. Several other states, including Washington, Florida, and Texas, are planning further installations. Texas is now developing specifications to use AD-IV on 60 installations of wood poles to support luminaires and to carry the power supply for the temporary lighting in a construction zone in El Paso. The purpose of the field demonstration projects was not to verify the performance of HBS during collisions. That was clearly demonstrated by crash tests in the proving ground environment (*2*). The purpose was to evaluate the installation procedures and the performance of HBS under such environmental loads as wind and ice. The results of these field evaluations have been excellent (*4*). No serious problems have been encountered in installation or maintenance, and the modified poles have, as predicted, withstood winds up to 70 mph in Kentucky and up to 80 mph in Massachusetts.

Just as predicted by laboratory strength tests, the HBS installations are stronger than those without the breakaway modification. In the 80-mph wind event in Massachusetts, unmodified poles were broken down, whereas the HBS installations developed only small rotations in the upper parts of the poles, that part above the upper knee connections.

It was clear, however, that in spite of the excellent performance to date there are improvements in HBS that would be helpful to the utility companies and states where it will be used. In fact, it was never considered that HBS would be the final system design (*5*).

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Recognizing the value of developing an improved design, the Texas Transportation Institute continued to develop an improved breakaway pole system. The goals were simple: reduce cost and improve performance. The result is AD-IV (*6*). Costs are projected to be reduced significantly in the AD-IV design, and several other improvements have been demonstrated. The AD-IV design was approved by FHWA on June 17, 1993 (*7*). The following sections describe and illustrate these design improvements.

DESCRIPTION OF BREAKAWAY POLES

This system consists of a lower connection (slip base), an upper connection (hinge mechanism), and structural support cables. The slip base and hinge mechanism activate on impact, reducing the effect of a semirigid pole on the errant vehicle while minimizing the effect on utility service. The slip base is designed to withstand the overturning moments imposed by in-service wind loads as well as to yield appropriately to the forces of an automobile collision. The upper hinge mechanism is sized so as to adequately transmit service loads while hinging during a collision to allow the bottom segment of the pole to rotate up and out of the way. 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 overhead guys (one above the upper connection and one below the neutral conductor) stabilize the upper portion of the pole during a collision to ensure the development of the bending moment necessary to activate the hinge. If enough utility conductors are present, the upper guys may possibly be eliminated. The proper function of a breakaway utility pole is illustrated in Figure 1.

Approved breakaway designs consist of three basic modifications to existing (or new) timber poles. The modifications used are a slip base (lower connection), a plastic hinge (upper connection), and the overhead guys (structural support cables). These devices for the HBS system were previously described in detail (*3*).

DESIGN DISADVANTAGES OF HBS

Subsequent to completion of the original FHWA project (*2*), discussions were held with representatives of numerous utility companies and with several steel fabricators. The following characteristics of Federal Highway-Breakaway Pole were discussed:

1. The circular shape of the base plates along with the six machined bolt slots were considered cost factors. If these circular bases were fabricated from plate steel there would be considerable waste. A square base plate, if not a functional disadvantage, would be lower in cost, and if a four-bolt connection rather than the six-bolt connection could be designed, further cost reductions could be

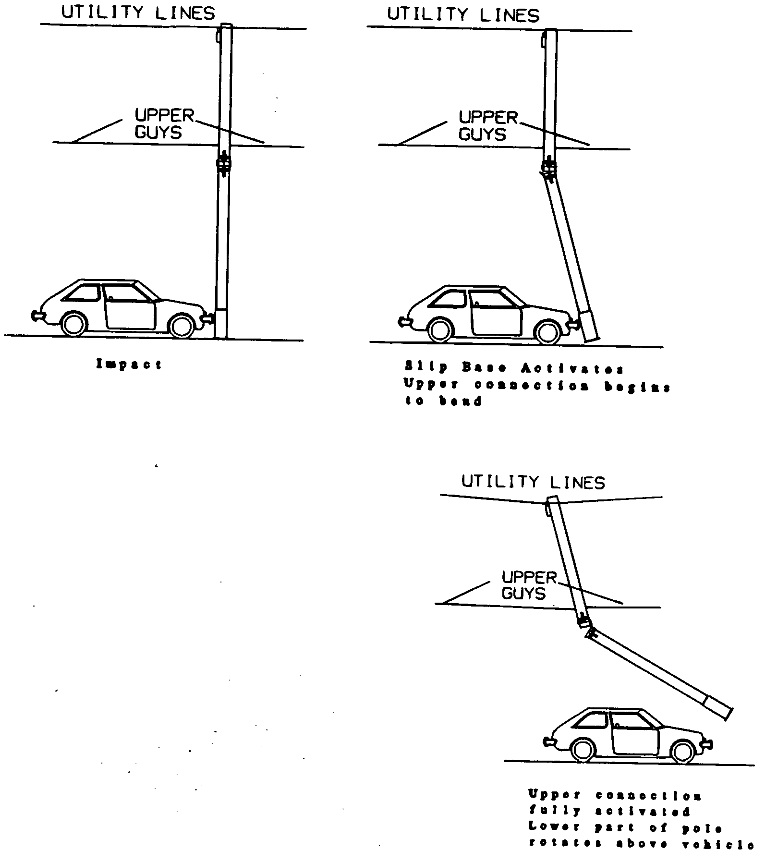


FIGURE 1 Proper function of breakaway utility pole.

achieved. Fortunately, the six-bolt connection had already been shown to be substantially overdesigned in static pullover tests (Figure 2).

2. The matching of a slot and four holes as the first yield mechanism of the steel straps in the upper connection would entail considerable fabrication costs. The tolerances of the margin between the holes needed to be accurate to within ±1/1,000 of an inch to achieve the proper yield strength. Designing the upper yielding connection to reduce these costs and make quality control less critical was considered important.

3. When an HBS modified timber pole is subjected to high repeated wind loads, an angular deflection of 5 to 10 degrees can develop in the upper pole segment. The HBS design required the upper pole to be straightened, usually by some piece of heavy equipment, and then all the bolts in at least one of the pole bands to be loosened, the bands adjusted, and the bolts retightened. A less labor- and equipment-intensive way to maintain poles in true alignment was considered of some importance.

DESCRIPTION OF AD-IV

It will be seen as AD-IV is described in detail that the three shortcomings of HBS have been overcome. AD-IV works precisely like HBS. Figure 3 shows the two components of AD-IV that are different from HBS. The overhead guys of HBS (Figure 1) are exactly the same in the AD-IV system.

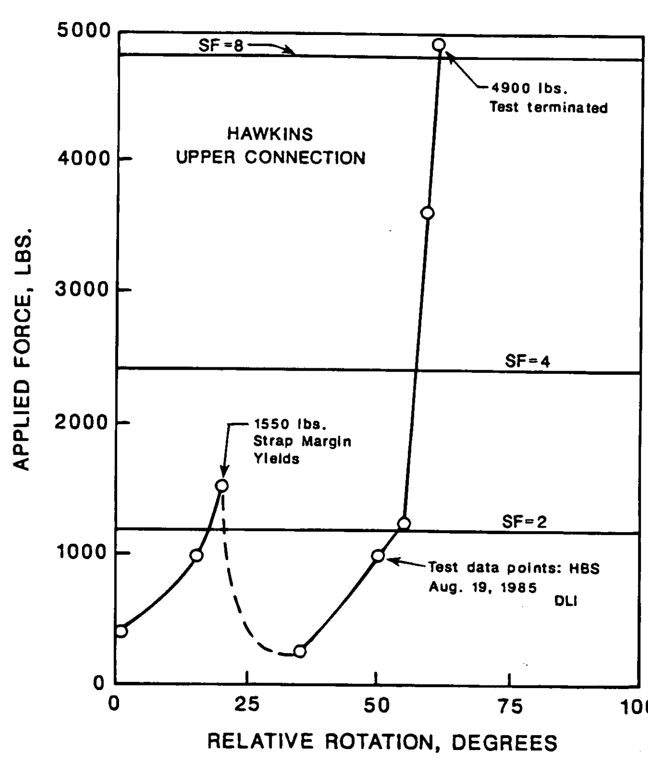


FIGURE 2 Load rotation characteristic of HBS upper connection (ultimate load safety factor is greater than eight).

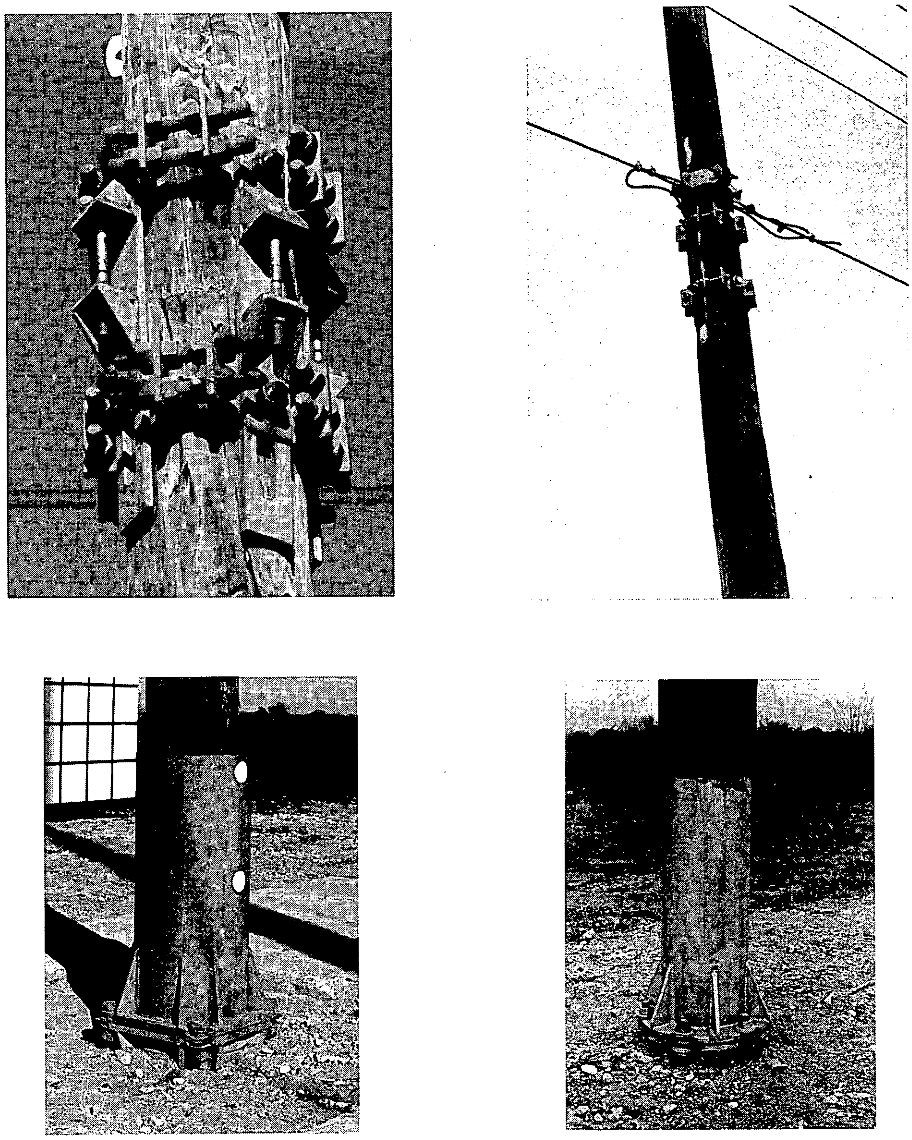


FIGURE 3 Upper and lower connections of AD-IV utility pole modification (*left*) compared with HBS (*right*).

LOWER CONNECTION: SLIP BASE

Just as in HBS, a lower shear plane is created by AD-IV through installation of a slip base at an elevation of 3 in. above grade. This shear plane consists of two $\frac{3}{4}$-in.-thick plates separated by a 26-gage keeper plate intended to maintain the bolts in the recessed corners of the 1 ft 3$\frac{3}{4}$ in. square base plate and by 2$\frac{1}{2}$-in.-diameter $\times$ $\frac{1}{8}$-in. washers. The base plates are connected to each other by four 1$\frac{1}{8}$-in.-diameter high-strength bolts, with 2$\frac{1}{2}$-in. $\times$ $\frac{1}{4}$-in. washers. These bolts are torqued to 200 ft-lb. Connection of the wooden utility pole to the slip base is through a steel pipe (Figures 3 and 4). These tubes are nominally 12 in. in diameter and 30 in. in length and are welded to the base plates. In addition, the base plates are braced by $\frac{1}{2}$-in.-thick stiffeners that are welded to both the base plate and the steel tube.

UPPER CONNECTION

Similar in basic structure, this connection consists of a pair of pole bands installed above and below a saw cut through the pole. The straps connecting the pole bands are detailed in Figure 5. The pole bands (and straps) are further secured to the pole by means of 1-in.-diameter through bolts as shown in Figure 6. At the bottom pole band, the bolt pass through the upper end of a 5$\frac{3}{4}$-in. slot. Initial bending resistance is provided by the strength of four $\frac{5}{8}$-in. gal-

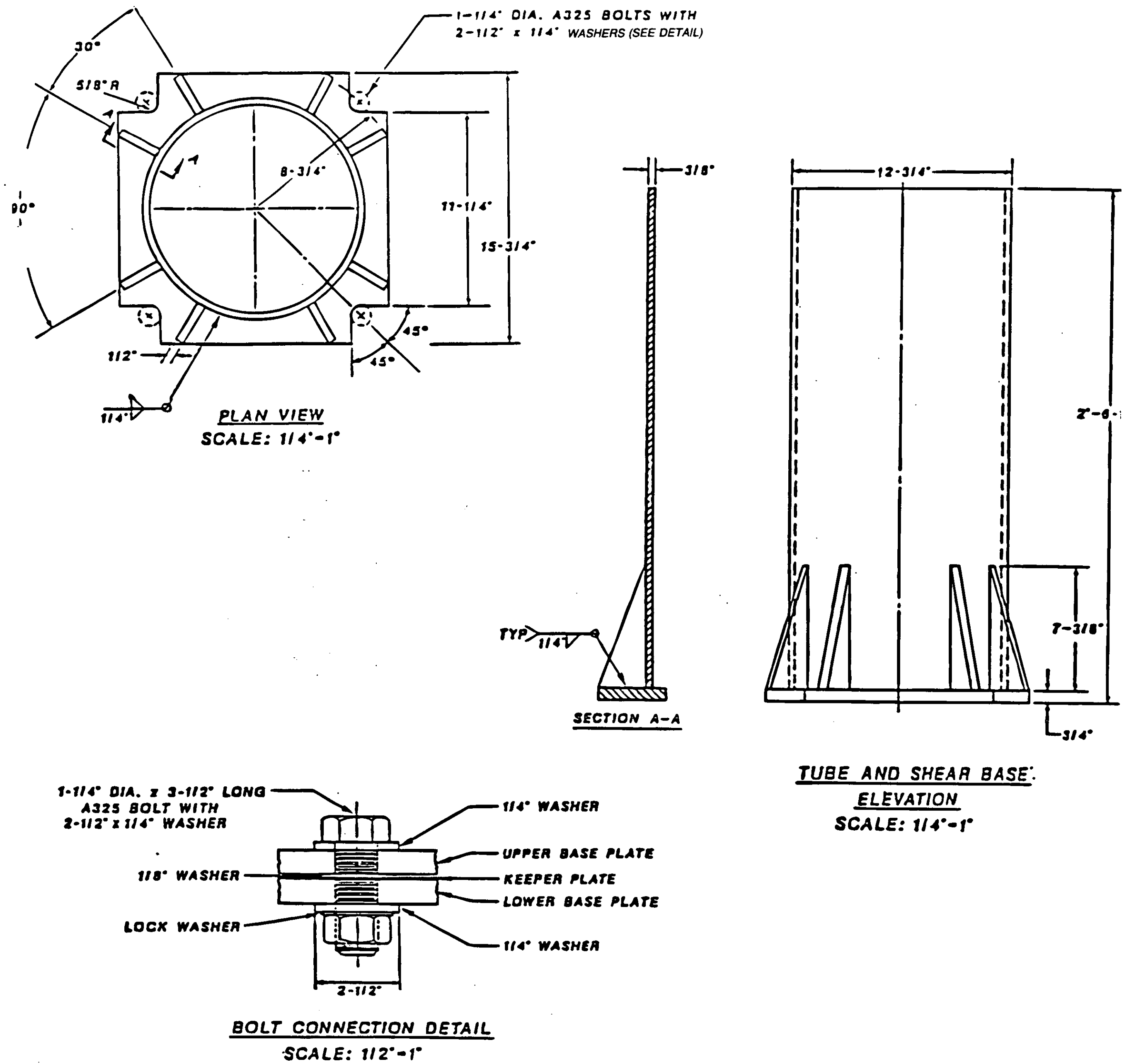


FIGURE 4 AD-IV slip base (lower connection).

vanized bolts that connect the brackets shown in Figure 6. These bolts have a turned stress riser groove 1 in. above the point where the threads start. The groove is ¼-in. wide and of sufficient depth that the remaining bolt diameter is 280/1,000 ± 5/1,000 in. Once two of the bolts fail in tension at a predetermined bending moment of 18,000 ft-lb, resistance is offered by friction between the straps and through bolts and by bending of the straps. Once significant rotation has occurred, the bolts bear on the end of the slot, thereby providing the required ultimate bending strength represented by a horizontal force approaching 4,800 lb, a safety factor of 4 for Class 4 poles. A completed installation is shown in Figure 7.

The load versus rotation curve is presented in Figure 8. This curve is similar to that of the HBS upper connection (shown in Figure 2) and achieves the same safety factors at the appropriate angular rotation levels.

MEETING *NCHRP REPORT 230* REQUIREMENTS

Use of the AD-IV upper connection does not result in any significant performance differences during automobile collisions. The advantages of the AD-IV are twofold. First, the costly machining of

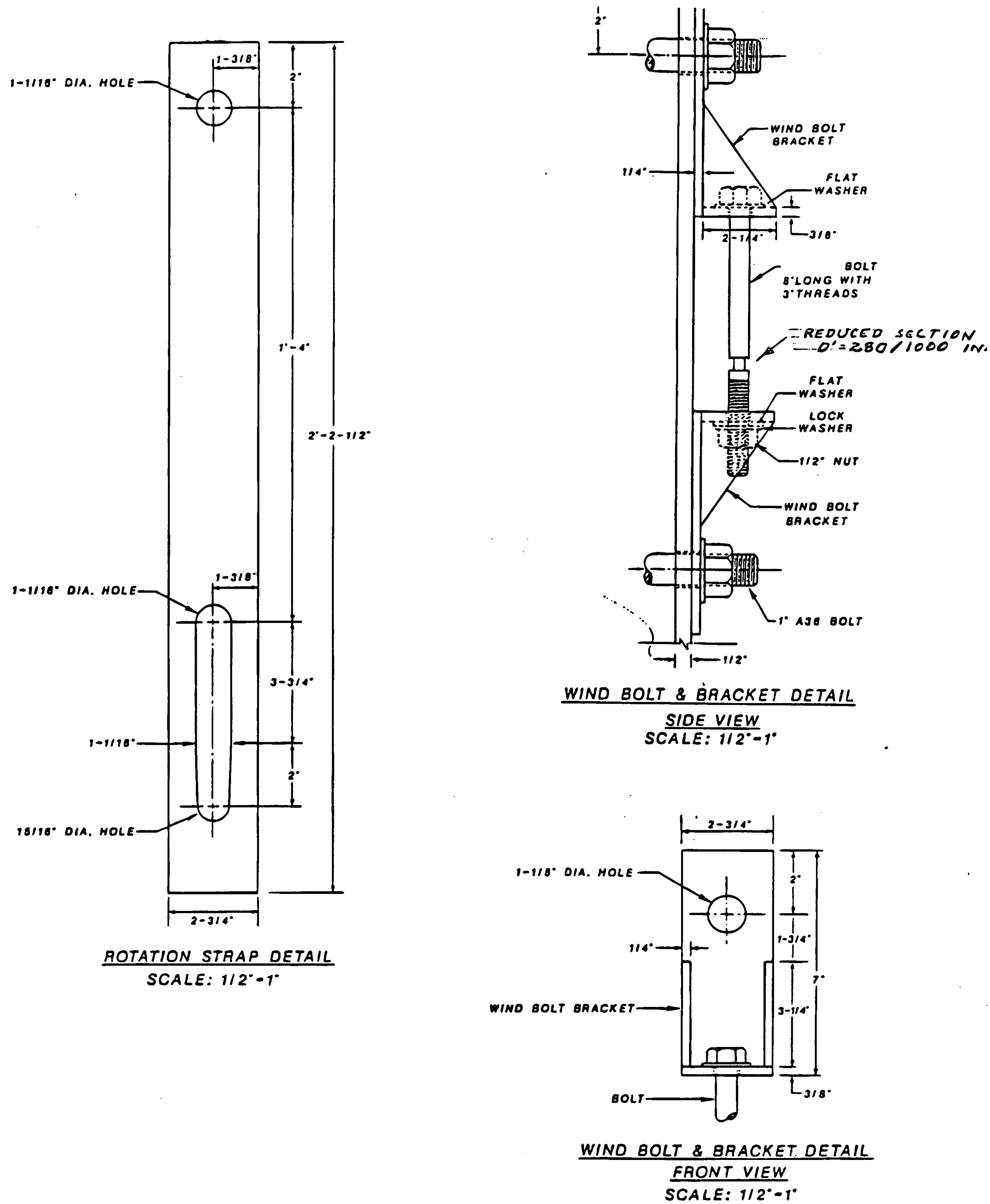
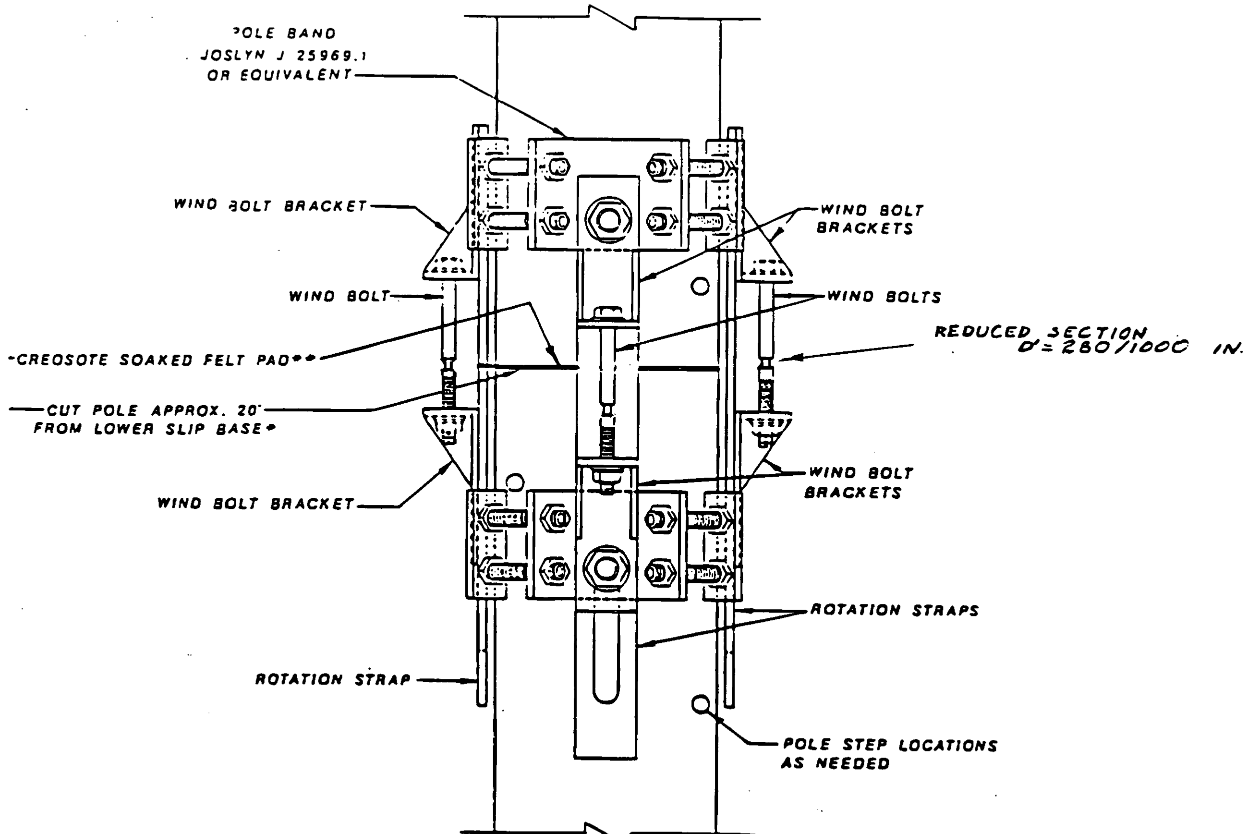
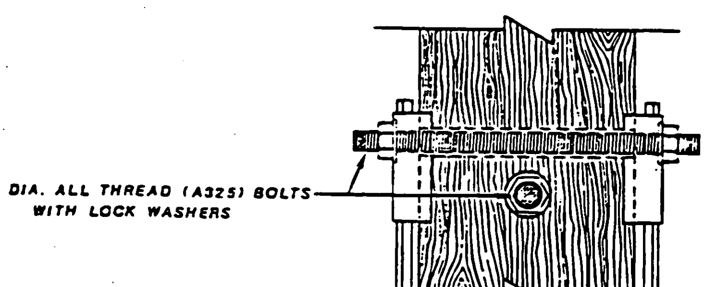


FIGURE 5 **Detail of rotation strap, wind bolt, and bracket.**



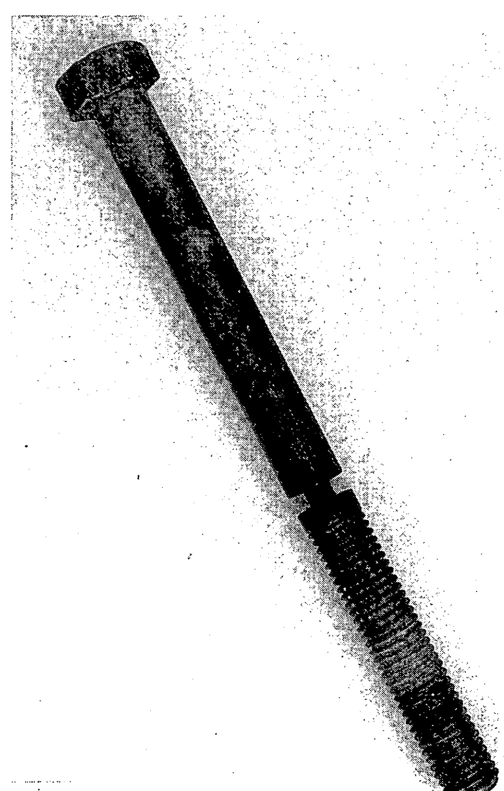
WIND BOLT & BRACKET OPTION

SCALE: 1/4"=1'



SECTION A-A'

SCALE: 1/4"=1'



wind bolt

FIGURE 6 **AD-IV hinge (upper connection).**

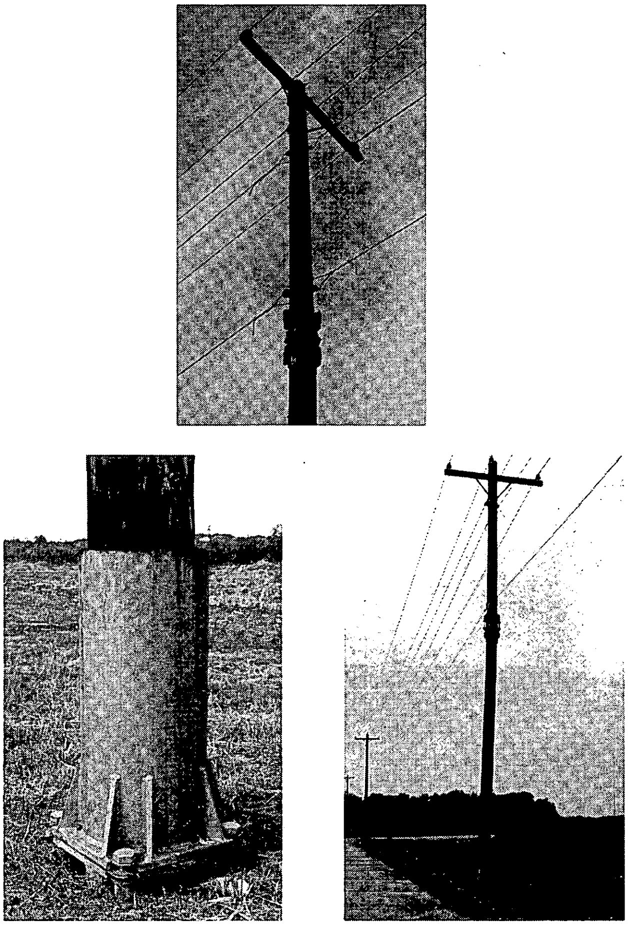


FIGURE 7 Slip base utility pole before Test 6018A-1.

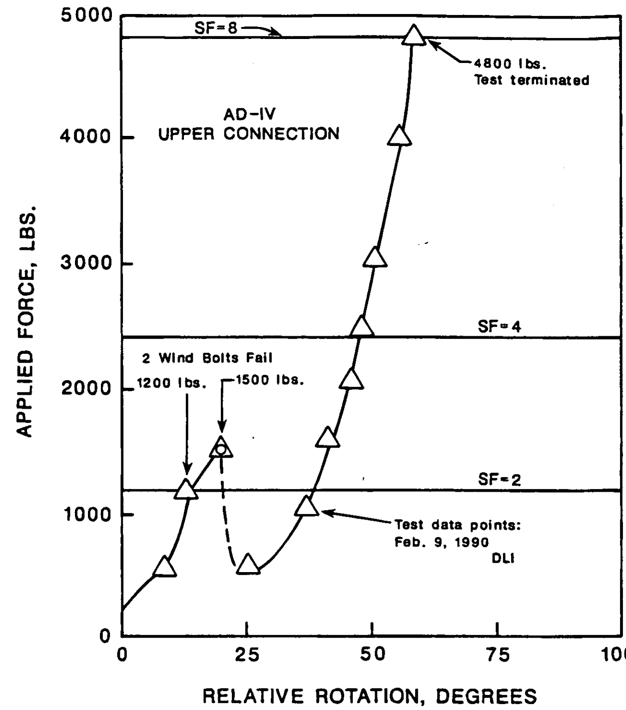


FIGURE 8 Load-rotation characteristic of AD-IV upper connection (ultimate load safety factor is greater than eight).

the wind straps for HBS has been eliminated. Second, if the AD-IV upper connection allows the upper part of the pole to lean during high winds or excessive ice, the pole can easily be straightened by simply loosening the large through bolts that clamp the wind straps, tightening or loosening the wind bolts to change the slope of the upper pole segment, and then retightening the through bolts. No heavy equipment would be required.

Use of the AD-IV lower connection should result in a slight reduction of energy absorbed in activating the slip base (*6*) owing to three factors: (*a*) the weight of the square plate is reduced, (*b*) the friction to be overcome using four bolts is approximately two-thirds the friction associated with the six-bolt HBS connection, and (*c*) the orientation of the slots in the corners of the AD-IV base is optimum for release if it is impacted from the primary traffic direction. In the case of HBS, the two bolts with slots located 90 degrees out of phase with the traffic direction must be moved laterally to allow the slip base to activate. Thus, AD-IV should perform somewhat better than HBS. Since HBS meets the requirements of *NCHRP Report 230* (*1*), AD-IV will also meet those same requirements with a slightly greater margin of safety. Although it was not considered necessary to perform all NCHRP full-scale compliance tests on AD-IV, the most critical test was run with an 1,800-lb automobile at 60 mph.

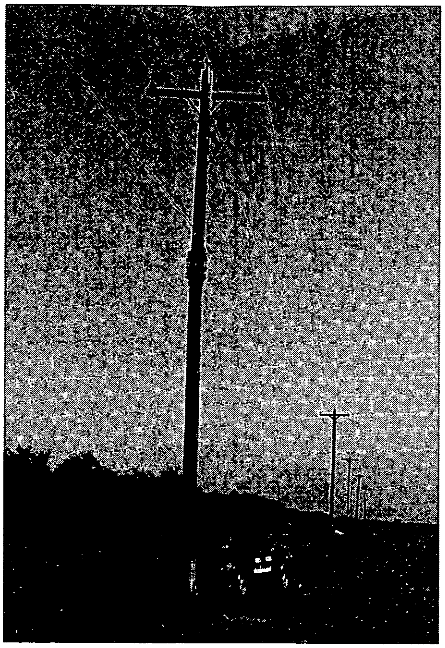
CRASH TEST ANALYSIS

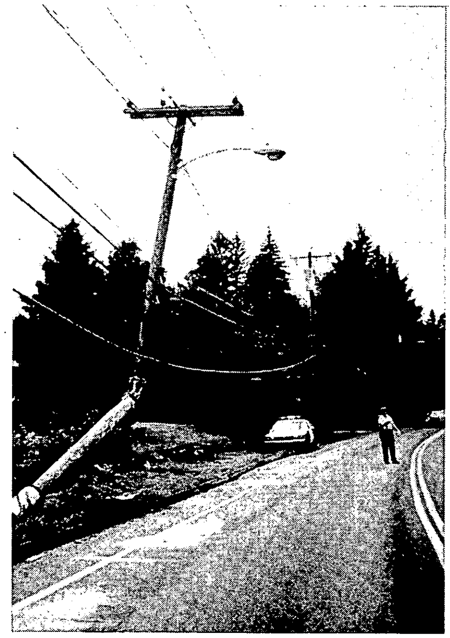
A 1980 Honda Civic (Figure 9) was used for the full scale crash test. The inertial mass of the test vehicle was 1,800 lb (816 kg), and its gross static mass was 2,130 lb (966 kg). The vehicle was directed into the utility pole by the cable reverse tow and guidance system and was released to be freewheeling and unrestrained just before impact. The vehicle impacted the pole at a speed of 59.6 mph (95.9 km/hr), and the angle of impact was 15.0 degrees relative to the strung wires.

With time zero being the point of first contact with the pole, the hinge began to flex at 0.027 sec, and there was visible space between the upper and lower sections of the pole at 0.047 sec. The hinge reached maximum extension at 0.131 sec. Contact between the pole base and vehicle was lost at 0.181 sec, and the vehicle separation speed was 42.8 mph (68.9 km/hr).

As can be seen in Figure 10, the pole received minor damage at the top cross members, the hinge deflected, and the upper guy wire broke at its connection at 0.377 sec after impact. This guy wire break would not be the normal case in a field installation because it was found that a 3/8-in. wire rope was used; it should have been 1/2 in. The normal field installation after being impacted in the August 24, 1990, hit in Grafton, Massachusetts, is shown in Figure 11. The lower section of the pole and the slip base were undamaged. Brakes were applied at 1.13 sec, and the vehicle came to rest 165.0 ft (50.3 m) from the point of first contact.

The vehicle sustained damage as shown in Figure 12. Maximum crush at the center front bumper height was 17.0 in. (43.2 cm), and





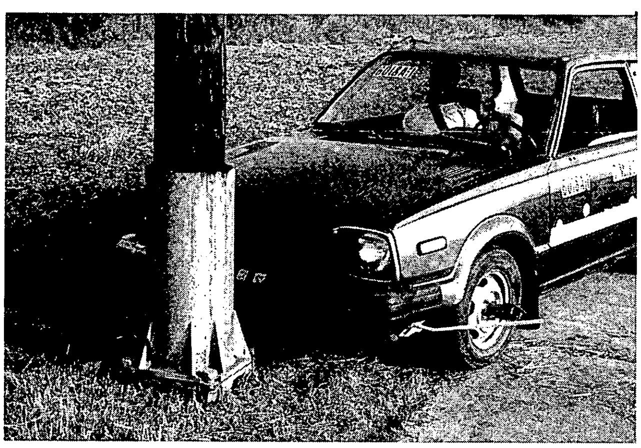


FIGURE 9 Vehicle and utility pole geometrics for Test 6018A-1.

FIGURE 10 Field installation after collision.

both the left and right front corners were pulled inward approximately 2.0 in. (5.1 cm).

Data from the accelerometer located at the center of gravity were digitized for evaluation, and occupant risk factors were computed as follows. In the longitudinal direction, occupant impact velocity was 19.7 ft/sec (6.0 m/sec) at 0.122 sec, the highest 0.010-sec average ridedown acceleration was −2.4 *g* between 0.144 and 0.154 sec, and the maximum 0.050-sec average acceleration was −13.6 *g* between 0.0 and 0.050 sec. In the lateral direction, occupant impact velocity was −3.1 ft/sec (−0.94 m/sec) at 0.944 sec, the highest 0.010-sec average ridedown acceleration was 1.4 *g* between 0.969 and 0.979 sec, and the maximum 0.050-sec average acceleration was −2.0 *g* between 0.024 and 0.074 sec. These data and other pertinent information from the test are summarized in Figure 13. Note the occupant impact velocity and the ridedown acceleration are well below the limits preferred by *NCHRP Report 230* (*1*).

The data in Table 1 indicate that the results of the test met *NCHRP Report 230* criteria. This test had not been run on the HBS during the original project for FHWA. Data are compared with data from tests with vehicles traveling at 20 and 40 mph, which were reported previously. Pendulum tests were conducted during the development process for AD-IV to verify breakaway characteristics. Accelerations from the final test, which complied with *NCHRP Report 230* guidelines, are compared with earlier pendulum tests on the HBS in Figure 14.

CONCLUSIONS

Engineers at FHWA have played a major role in conducting field trials of breakaway utility poles, and the FHWA-sponsored research was the turning point in developing practical, strong, and collision-safe utility poles. Table 2, by Buser and Buser (*4*), documents colli-

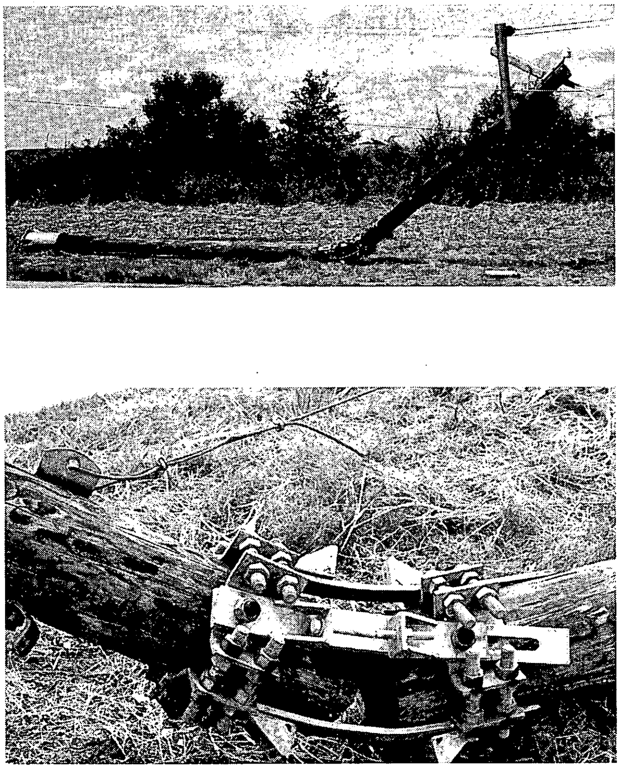
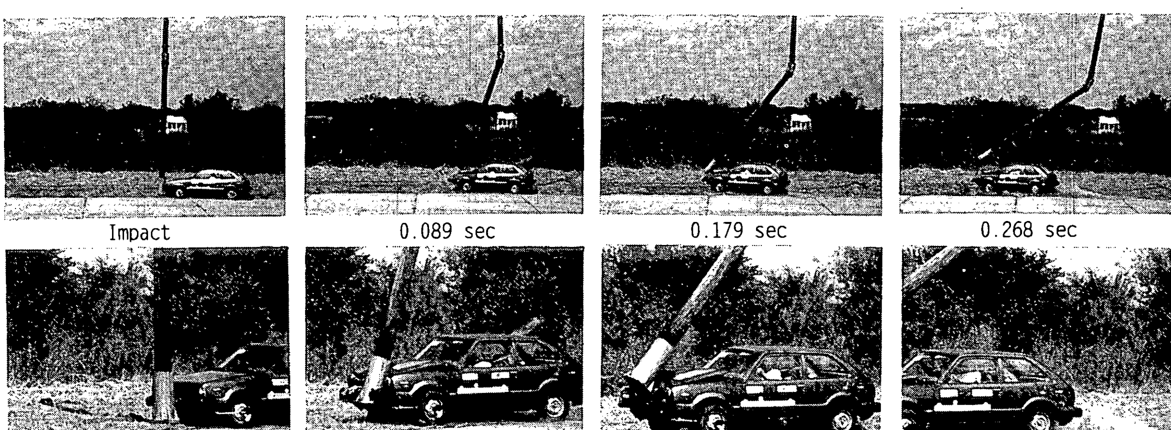


FIGURE 11 Utility pole after Test 6018A-1.

FIGURE 12 Vehicle after Test 6018A-1.



Test No. 6018A-1
Date 09/02/92
Test Installation . . . Slip-base Utility Pole
Installation Length . . N/A
Max. Dynamic Movement . Not Obtained
Max. Perm. Movement . . 27.25 ft (8.31 m)
Test Vehicle . . . . . . 1980 Honda Civic
Vehicle Weight
Test Inertia . . . . . 1,800 lb (816 kg)
Gross Static . . . . . 2,130 lb (966 kg)
Vehicle Damage Classification
TAD 12FC4
CDC 12FCEW2
Maximum Vehicle Crush. . 17.0 in (43.2 cm)

Impact Speed. . . . . 59.6 mi/h (95.9 km/h)
Impact Angle. . . . . 15.0 deg
Speed at Parallel . . N/A
Exit Speed . . . . . 42.9 mi/h (69.0 km/h)
Exit Trajectory . . . Not Obtained
Vehicle Accelerations at center-of-gravity
(Max. 0.050-sec Average)
Longitudinal. . . . -13.6 g
Lateral . . . . . . - 2.0 g
Occupant Impact Velocity at true c.g.
Longitudinal. . . . 19.7 ft/s (6.0 m/s)
Lateral . . . . . . -3.1 ft/s (0.9 m/s)
Occupant Ridedown Accelerations
Longitudinal . . . -2.4 g
Lateral . . . . . . 1.4 g

FIGURE 13 Summary of results for Test 6018A-1.

TABLE 1 Selected Test Results (*7*).

	Occupant Velocity Change ft/s	Ride Down Acceleration 10ms max g's	Highest Vehicle Acceleration 50ms max g's
Test 6018A-1 (AD-IV) 60 mph	19.7	2.4	13.6
Test 4859-16 (FHWA) 40 mph	12.0	1.0	8.0
Test 4859-12 (FHWA) 20 mph	10.1	2.1	6.7
NCHRP 230 (Guidelines)	30	15	---
FHWA Suggested Value	22*	---	---

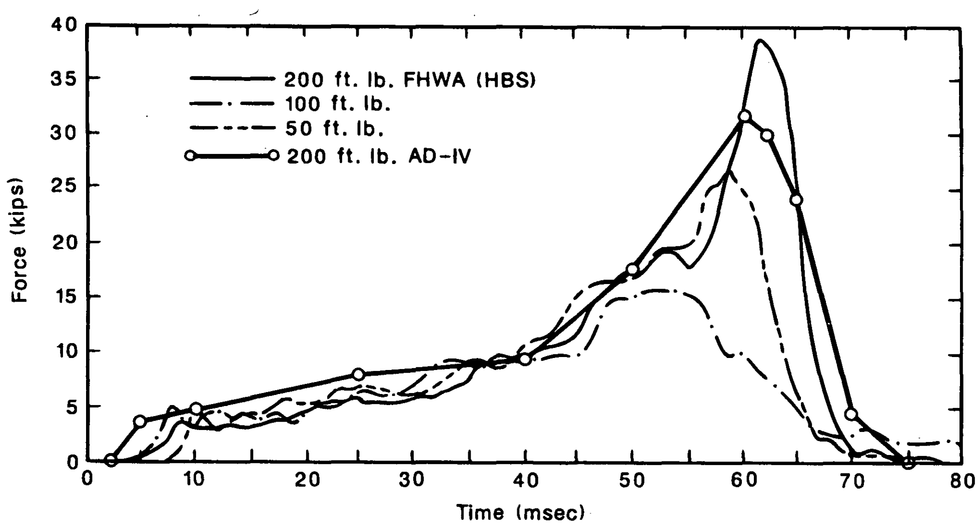


FIGURE 14 Impulse curve comparing AD-IV with original design (HBS).

TABLE 2 Breakaway Utility Pole Collisions.

DATE	PLACE	TIME TO RESTORE POLE
August 24, 1990	Grafton	3 Hours
December 12, 1990	Oxford	4 Hours
April 21, 1991	Oxford	2 Hours
May 12, 1991	Methuen	1/2 Hour
September 25, 1991	Oxford	1 Hour

- **THERE WERE NO INJURIES IN ANY COLLISION.**
- **THERE WAS NO SERVICE LOST IN ANY COLLISION.**

sions with breakaway utility poles in field installations. There were no injuries and no service was lost in any of the collisions. Table 3 summarizes the differences between HBS and AD-IV and suggests that AD-IV is the next preferred step in the evolution of practical, low-cost, high-performance structural systems that can be used to modify timber utility poles. AD-IV systems were scheduled for implementation in Fort Worth, Texas, during 1994. It may be appropriate to include AD-IV installations at new locations as other states continue or begin the implementation process. The precedents for improving roadside safety through modification of selected timber utility poles are well established (*8,9*). Additionally, noninterruption of service and short repair times enhance cost considerations. On September 15, 1993, William Quirk of Boston Edison stated, "These poles [breakaway] save money on maintenance." AD-IV joins HBS, crash cushions, and guardrail designs as one more method of treating those poles found to be a hazard to the public (*10*).

ACKNOWLEDGMENTS

The authors have received help from so many sources it is feasible only to name the individuals without reference to their broad scope of contributions. The authors thank Maurice E. Bronstad, Carol A. Buser, Richard P. Buser, James L. Cline, Don Cangelose, Kenneth R. Ewald, James A. Hatton, Teddy J. Hirsch, King K. Mak, Charles F. McDevitt, Wanda L. Menges, Jarvis D. Michie, Robert K.

TABLE 3 Specific Points of Difference Between FHWA Breakaway Pole and AD-IV

	HBS	AD-IV
1.	Upper Connection	Upper Connection
	* 4 strap connectors between upper and lower pole bands.	* 4 strap/4 bolt connection between upper and lower pole bands.
	* Complicated arrangements of slots and holes machined to rigorous tolerances.	* Connection requires no precision machining.
	* No practical means of correcting misalignment of upper and middle pole segment without heavy construction equipment.	* Connection (4 strap/4 bolt) can easily be adjusted by individual maintenance workers to replace fractured elements and correct misalignment.
2.	Lower Connection	Lower Connection
	* 6 bolt circular slip base.	* 4 bolt square slip base
	* Circular base produces much waste when fabricated from steel plate.	* Square base reduces waste to negligible amounts and reduces weight of the resultant plate. (This is a significant safety advantage in reducing inertia of pole structure.)
	* Bolt/slot geometric arrangement is not optimum relative to energy absorbed when vehicle strikes the structure.	* Bolt/slot geometric arrangement reduces activation energy to lowest feasible level for most probable impact angles. (This is a significant safety advance and is found on no other multi-directional slip base.)

Musselman, Robert M. Olson, Richard D. Powers, William Quirk, Bill D. Ray, Paul C. Scott, Claude J. Toomer, Timothy L. Tucker, Charles V. Zegeer, and Richard A. Zimmer. Finally, the authors give a special acknowledgment to Charley V. Wootan, Director Emeritus of the Texas Transportation Institute, for a classic research environment that is hospitable to both conventional and unconventional engineers and where imagination is considered a virtue.

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