HIGHWAY FACTORS GROUP CHAIRMAN FACTUAL REPORT

(61 pages)

NATIONAL TRANSPORTATION SAFETY BOARD
OFFICE OF HIGHWAY SAFETY
WASHINGTON, DC 20594

HIGHWAY FACTORS GROUP CHAIRMAN FACTUAL REPORT

A. ACCIDENT

LOCATION:	Interstate 95 (I-95) New England Thruway, at Mile Marker 3.2, in New York City, Bronx County, New York
VEHICLE 1:	1999 Prevost Motorcoach
OPERATOR:	World Wide Travel of Greater New York Ltd.
DATE:	March 12, 2011
TIME:	Approximately 5:37 a.m. EST

NTSB #: **HWY-11-MH-005**

B. HIGHWAY FACTORS GROUP

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C. DETAILS OF THE FACTUAL REPORT

The Highway Factors Group Chairman Factual Report provides the reader with a factual record of the highway conditions that existed at the time of the accident. For a better understanding of all the circumstances and facts of the accident, readers are encouraged to also examine the Vehicle Factors Group Chairman Factual Report, the Human Performance Group Chairman Factual Report, the Survival Factors Group Chairman Factual Report, and the Motor Carrier Operations Group Chairman Factual Report.

The Highway Factors Group Chairman Factual Report is organized by the following topic areas and page numbers:

1. PREFATORY DATA

1.1 ACCIDENT LOCATION

The accident occurred on the southbound lanes of I-95 (New England Thruway) at Mile Marker 3.2, approximately 11 miles northeast of Manhattan. Figure 1 is an accident location map that illustrates the accident occurred within New York City in Bronx County, New York, immediately south of the Westchester County and Bronx County line.

Figure 1 – Accident location map

1.2 NEW YORK STATE (NYS) THRUWAY AUTHORITY

The Thruway Authority is a public corporation that was created in 1950 for the purpose of financing, constructing, reconstructing, improving, developing, maintaining and operating a highway system known as the Governor Thomas E. Dewey Thruway. The powers of the Authority are vested in and exercised by a seven-member Board appointed by the Governor and confirmed by the State Senate.

The Authority maintains and operates the 570-mile superhighway system known as the New York State Thruway, one of the longest toll roads in the United States. The Thruway's 426-mile mainline connects New York City and Buffalo, the state's two largest cities. The Thruway route from New York to the Pennsylvania line at Ripley is 496 miles long. Other Thruway sections make direct connections with the Connecticut and Massachusetts turnpikes, New Jersey's Garden State Parkway and Interstate 287, and other major expressways that lead to New England, Canada, the Midwest and the South. Specifically, the 570 mile Thruway System is comprised of the following:

THE MAINLINE (I-87/I-90): New York to Buffalo (426 miles)
ERIE SECTION (I-90): Buffalo to Pennsylvania line (70 miles)
NIAGARA SECTION (I-190): Buffalo to Niagara Falls (21 miles)
BERKSHIRE SECTION (I-90): Selkirk to Massachusetts line (24 miles)
NEW ENGLAND SECTION (I-95): Bronx to Connecticut line (15 miles)
GARDEN STATE PARKWAY CONNECTION: Spring Valley to New Jersey (3 miles)
CROSS WESTCHESTER EXPRESSWAY (I-287): Mainline I-87 in Tarrytown to I-95 in Rye (11 miles)

In addition to the roadway, the Thruway Authority has jurisdiction over 808 bridges, including the Tappan Zee Bridge, and its subsidiary, the New York State Canal Corporation, which oversees 524-miles of canals and 57 locks and 16 lift bridges. The Authority has an annual budget of over $1 billion, including $400 million in operating and $600 million capital and equipment expenses. Approximately 271 million vehicles travel more than 8 billion miles on the Thruway each year.

1.3 ROADWAY MAINTENANCE AND CONSTRUCTION HISTORY

The NYS Thruway Authority was given jurisdiction of the New England Thruway (I-95) between Pelham Parkway (MP[1] 0.0) and the Connecticut State Line (MP 15.0) by the New York State Legislature and authorized start of construction in the 1950's. The construction of the roadway was completed and opened in 1958. The New York City Police Department has patrol jurisdiction of I-95 south of the New York City/Westchester County Line (MP 3.54) and the New York State Police has patrol jurisdiction of I-95 north of the New York City/Westchester County Line (MP 3.54). The New York State Thruway Authority has roadway maintenance jurisdiction of the entire New England Thruway (MP 0.0 to MP 15.0)

The Highway Factors Group Chairman Factual Report includes a number of references and quotes from more recent reports, design standards, and New York State manuals as it relates to the highway element and features along this section of I-95. Although it is footnoted in the Highway Factors Group Chairman Factual Report, it should be emphasized that this section of I-95 was constructed in the 1950's and reconstructed in 1984 to the "standards of the day".

I-95 in the vicinity of the accident location was constructed in the 1950's[2] with 9 inches of portland cement concrete over an 8 inch base.

In 1984, the existing concrete pavement along I-95 was reconstructed. Listed below are the major items included in the 1984 reconstruction project[3]:

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[1]Milepost (MP) is synonymous with Mile Marker.
[2]Construction contract FANETC 54-13.
[3]Construction contract TANE 84-25.

- Existing pavement reconstructed with 10 inches of cement concrete,
- Construction of new shoulders on right and left sides with 8 inches of asphalt concrete,
- Existing drainage inlets adjusted and relocated,
- Existing highway lighting removed and new highway lighting installed in median,
- Existing mountable (sloped) concrete curb removed at the edge of travel lane and new mountable (sloped) concrete curb installed at the back edge of shoulder,
- Construction of new overhead sign structure supports, and
- Construction of new strong post blocked-out W-beam guiderail. In the vicinity of the accident, the new strong post blocked-out W-beam guiderail only protected the vertical poles supporting the overhead sign structure support and was not continuous along the right side.

In 1998, a noise barrier was constructed along the right side of the southbound lanes of I-95. As part of the noise barrier project[4], new strong post blocked-out W-beam guiderail was installed that provided a continuous guiderail along the right side of the southbound lanes.

The shoulders along I-95 in the vicinity of the accident location were rehabilitated in 1997[5] and 2006[6]. The latest rehabilitation project in 2006 involved milling and resurfacing the right shoulder, and constructing grooved rumble strips (commonly referred to as star - shoulder treatment for accident reduction).

1.4 ANNUAL AVERAGE DAILY TRAFFIC

Table 1 summarizes the annual average daily traffic (AADT) on I-95 from Hutchinson Parkway (Exit 14) to the Westchester county line from 2005 to 2009.

Table 1 – Annual Average Daily Traffic (AADT) on I-95 from Hutchinson Parkway to the Westchester County Line

Year	Annual Average Daily Traffic (AADT)
2005	101,440
2006	112,770
2007	110,500
2008	108,490
2009	106,990

[4]Construction contract TANE 98-70.
[5]Construction contract TANE 97-29.
[6]Construction contract TANE 06-21.

1.5 VEHICLE CLASSIFICATION DATA

Table 2 summarizes the vehicle classification data at the New Rochelle Toll Plaza for the northbound lanes of I-95 near MP 6.9 for the month of October 2010.

Table 2 – Vehicle classification data

Classification of Vehicle	Traffic Volume	Percent
Passenger Cars (Class 2)	1,522,963	88.6%
Light Trucks (Class 3)	1,566	0.1%
Buses (Class 4)	1,908	0.1%
Single-Unit Trucks and Single-Trailer Trucks (Class 5 through 9)	191,297	11.1%
Multi-Trailer Trucks	410	0.1%
Total	**1,718,144**	**100%**

1.6 TRAFFIC ACCIDENT SUMMARY

Table 3 summarizes the traffic accident summary on the southbound lanes of I-95 within a 2 mile radius of the accident location from 2006 to 2011.

Table 3 – Traffic accident summary on the southbound lanes of I-95 within a 2 mile radius of the accident location

Year	Total	Injuries	Injury Accidents
2006	55	33	24
2007	116	43	35
2008	85	37	26
2009	81	36	22
2010[7]	42	28	19
2011[8]	6	2	2
Totals	**385**	**179**	**128**

[7] The 2010 data is incomplete due to a lag time for some NYC police accident report entries to the NYS DOT statewide accident database.
[8] The 2011 data is low because it only covers from January to March.

Table 4 summarizes the traffic accident summary on the southbound lanes of I-95 broken down by cause factor from 2006 to 2011.

Table 4 – Traffic accident summary on the southbound lanes of I-95 broken down by cause factor from 2006 to 2011

Cause Factor	Accidents	Percent (%)
Unsafe lane change	81	21.0%
Following too closely	78	20.3%
Unsafe speed	37	9.6%
Passing/lane usage improper	30	7.8%
Obstruction/debris	26	6.8%
Reaction to other uninvolved	10	2.6%
Other vehicular	9	2.3%
Alcohol involvement	5	1.3%
Failure to yield right of way	4	1.0%
Pavement slippery	4	1.0%
Driver inexperience	3	0.8%
Brakes defective	2	0.5%
Driver inattention	2	0.5%
Failure to keep right	2	0.5%
Steering failure	2	0.5%
Fell asleep	1	0.3%
Turning improperly	1	0.3%
Tire failure/inadequate	1	0.3%
View obstructed/limited	1	0.3%
Backing unsafely	1	0.3%
Accelerator defective	1	0.3%
Drugs (illegal)	1	0.3%
Multiple causes	63	16.4%
Cause not reported	20	5.2%
Totals	**385**	**100%**

1.7 HISTORY OF GUIDERAIL DAMAGE REPAIR

Table 5 summarizes the history of guiderail damage repair on the southbound lanes of I-95 in the vicinity of the accident location from 2006 to 2011.

Table 5 – History of guiderail damage repair on the southbound lanes of I-95 in the vicinity of the accident location from 2006 to 2011

Date of Incident	Location	Damage Repaired	Guiderail Damage
11/23/2006	MP 3.5	3/26/2007	2 Sections (24 feet)
3/09/2007	MP 3.3	3/28/2007	3 Sections (36 feet)
Hit and Run	MP 3.4	5/24/2010	4 Sections (48 feet)
12/14/2010	MP 3.4	12/14-16/2010	32 Sections (384 feet)
Hit and Run	MP 3.3	3/03/2011	4 Sections (48 feet)

For the November 23, 2006 incident (see Attachment 1 – New York State Department of Motor Vehicles Police Accident Report dated November 23, 2006), a four door sedan lost control due to tire failure and struck the right side guiderail. Only one vehicle was involved with no injuries reported. For the March 9, 2007 incident, the NYS Thruway Authority could not find any accident report in the statewide accident database. For the December 14, 2010 incident (see Attachment 2 – New York State Department of Motor Vehicles Police Accident Report dated December 14, 2010), a tractor/semi trailer truck (3 axles) swerved onto the right shoulder due to unsafe speed and struck the guiderail. Only one vehicle was involved with no injuries reported.

In general, the NYS Thruway Authority maintenance forces adhere to the principles outlined in the NYS DOT Highway Maintenance Guidelines. The NYS DOT Highway Maintenance Guidelines[9] indicated the following:

> *"**3.6.1.2 Standard:** Guide rail should be maintained as near as possible to the original construction condition. Guide rail which is out of alignment should be straightened. Posts should also be plumb. A neat, clean, uniform and aligned appearance is desirable. Installations should be inspected regularly and bent or damaged rail should be repaired immediately. Guide rail should be inspected and reconditioned every spring as soon as weather conditions permit.*
>
> *Before any major guide rail repair or rehabilitation is undertaken, current design standards should be checked to determine if the installation is still warranted, if*

[9]New York State Department of Transportation Highway Maintenance Guidelines Roadside and Drainage Maintenance, Section 3.6 Guide Rail, Median Barriers, and Impact Attenuators; revised April 1, 1978, page 3-14.

an old type guide rail should be replaced by a newer type guide rail or if a more economical type of guide rail may be used."

The NYS Thruway Authority Maintenance Directive – Guidelines for the Repair and Upgrade of Guide Rail and Bridge Rail[10] (see Attachment 3 – NYS Thruway Authority Maintenance Directive (MD) 2002-4, Guidelines for the Repair and Upgrade of Guide Rail and Bridge Rail) indicated the following:

"5. When a substantial portion of guide rail (over 250 feet long or one-third (1/3) of the existing guide rail length) is damaged, the Section Supervisor must notify the Division Highway Engineer, who in turn must contact the Roadside Safety Unit. The Roadside Safety Unit and Division Highway Engineer will review the site for conformance to current standards and will reach consensus on whether to replace or redesign the guide rail. When redesign of the guide rail is warranted, the field layout should be developed or approved by the Roadside Safety Unit within five (5) working days. Subsequent documentation of the field layout shall be developed by the Roadside Safety Unit as soon as practicable.

6. When the conditions in item 5. (above) do not apply, repairs are to be made "in kind," except that steel block-outs should not be reset or reused for the repair of damaged heavy post blocked-out corrugated beam guide rail/median barrier. A solid block-out (wood, plastic, or synthetic) should be substituted in lieu of the steel block-outs when repairing the damaged portion of a heavy post blocked-out system."

1.8 WEATHER REPORT

The closest weather reporting facility was from LaGuardia Airport (KLGA) located approximately 8 miles southwest of the accident site. LaGuardia weather at 551 EDT was wind from 240 degrees at 11 knots gusting to 21 knots, visibility unrestricted at 10 statute miles, ceiling broken at 4,700 feet above ground level, temperature 39 degrees Fahrenheit. A review of the last 24 hours of weather indicated 0.71 inches of rain had fallen and ended by 900 EDT on March 11, 2011. The conditions at the time of the accident were considered dry.

The astronomical conditions from the United States Naval Observatory indicated the beginning of civil twilight at 545 EDT and sunrise at 613 EDT. At 545 EDT the sun was 6.1 degrees below the horizon at an azimuth of 89 degrees or almost due east. The moon was more than 15 degrees below the horizon and provided no illumination.

[10]New York State Thruway Authority Maintenance Directive (MD) 2002-4, Guidelines for the Repair and Upgrade of Guide Rail and Bridge Rail, July 15, 2002, page 2.

2. **HIGHWAY DATA**

2.1 FUNCTIONAL CLASSIFICATION

I-95 in the vicinity of the accident location was classified as an urban principal arterial. The American Association of State Highway and Transportation Officials (AASHTO) classified urban principal arterials[11] as follows:

> *"The urban principal arterial system serves the major centers of activity of urbanized areas, the highest traffic volume corridors, and the longest trip desires and carries a high proportion of the total urban area travel even though it constitutes a relatively small percentage of the total roadway network."*

2.2 HIGHWAY DESIGN

The southbound lanes of I-95 in the vicinity of the accident consisted of three travel lanes (see Highway Factors Photograph (HWY Photo-01) illustrating the three southbound travel lanes of I-95). The measured width of the travel lanes was approximately 36 feet. The travel lanes consisted of three 12-foot wide lanes. The typical section of the southbound lanes of I-95 also consisted of a 10-foot wide paved right and left shoulder.

Grooved rumble strips existed on the right and left shoulder of the southbound lanes of I-95. A sloped curb existed at the edge of the right and left shoulder.

Figure 2 illustrates a typical section of the southbound lanes of I-95 in the vicinity of the accident.

[11]A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials (AASHTO), 2004, Fifth Edition, page 11.

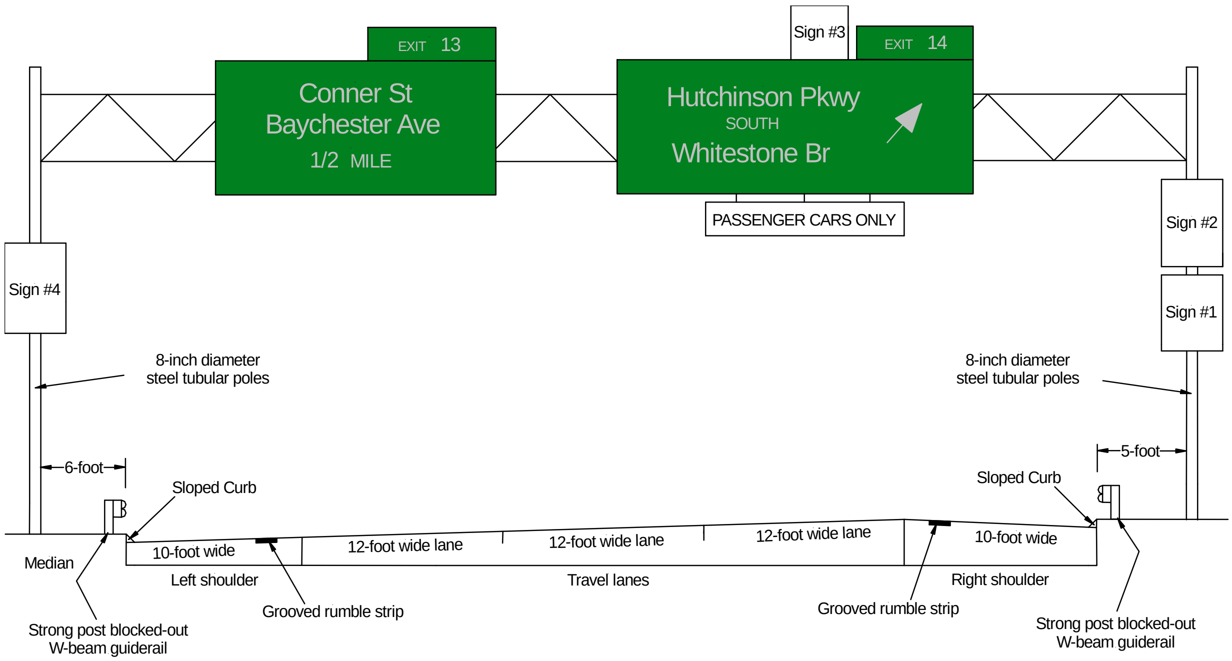

Figure 2 – Typical section of the southbound lanes of I-95 in the vicinity of the accident

The sign numbers shown in Figure 2 consisted of the following:

Sign #1 – Within NYC Limits Red Lights Photo Enforced
Sign #2 – Truck Restriction Use Only Designated Truck Routes Exit Only At Point Closest To Destination
Sign #3 – No Truck Symbol
Sign #4 – NYC Law No Turn On Red Except Where Posted

2.3 SPEED LIMIT

The posted speed limit for the southbound lanes of I-95 in the vicinity of the accident was 50 miles per hour (mph) for trucks and passenger cars (see Attachment 4 – NYS Thruway Authority Regulatory Speed Limits). The accident occurred in Bronx County, New York in which the posted speed limit for the southbound lanes of I-95 is 50 mph for all vehicles. The posted speed limit sign was located approximately 1,000 feet prior to the accident. In Westchester County, New York, immediately north of the accident, the posted truck speed limit for the southbound lanes of I-95 was 50 mph and the posted state speed limit was 55 mph for passenger cars.

2.4 EXISTING SIGNAGE FOR THE SOUTHBOUND LANES OF I-95

Table 6 summarizes the existing signage for the southbound lanes of I-95 preceding the accident.

Table 6 – Existing signage for the southbound lanes of I-95 preceding the accident

Existing Sign	Distance from Accident	Location of Sign
NO TRUCKS BUSES TRAILERS LEFT LANE	2.5 miles prior to accident	Sign located beyond right shoulder
STATE SPEED LIMIT 55	1 mile prior to Accident	Sign located beyond right shoulder
TRUCK SPEED LIMIT 50	1 mile prior to Accident	Sign located beyond right shoulder
NO TRUCKS BUSES TRAILERS LEFT LANE	2,640 feet prior to accident	Sign located beyond left shoulder in median
SPEED LIMIT 50	1,000 feet prior to accident	Sign located beyond right shoulder
WITHIN N.Y.C. LIMITS HAZARDOUS LIQUEFIED GAS TANK OR EXPLOSIVES TRUCKS PROHIBITED UNLESS AUTHORIZED BY NYC FIRE DEPT PH # 718-999-2094 HAZARDOUS CARGO THRU SHIPMENT MUST FOLLOW DESIGNATED ROUTES & TIMES PENALTY: $10,000 & 30 DAYS	453 feet prior to Accident	Sign located beyond right shoulder
TRUCK ROUTE TO I-678 SOUTH WHITESTONE BR USE EXIT 6A	217 feet prior to Accident	Sign located beyond right shoulder

WELCOME TO THE BRONX MAYOR MICHAEL R. BLOOMBERG BORO PRES. RUBEN DIAZ JR.	130 feet prior to Accident	Sign located beyond right shoulder

2.5 HIGHWAY MARKINGS

The southbound lanes of I-95 contained a 6-inch wide solid yellow line separating the left travel lane from the left shoulder and a 6-inch wide solid white line separating the right travel lane from the right shoulder. The left travel lane, middle travel lane, and right travel lane were separated by 6-inch wide broken white lines. The 6-inch wide broken white lines were each 10 feet long and had 30 foot spaces between them. Both the yellow and white lines were retro-reflective[12] and met the requirements of the 2009 Manual on Uniform Traffic Control Devices (MUTCD) for width and spacing.[13]

2.6 HORIZONTAL AND VERTICAL CURVATURE OF I-95

The horizontal curvature[14] on the southbound lanes of I-95 prior to the accident consisted of a 1,600 foot radius curve that turned to the right in the direction of travel (see Highway Factors Photograph (HWY Photo-02) illustrating the 1,600 foot radius and 4,500 foot radius horizontal curves). The 1,600 foot radius horizontal curve ended approximately 770 feet prior to the accident location. The horizontal curvature transitioned to a 4,500 foot radius curve that turned to the left in the direction of travel. The accident location was located within the 4,500 foot radius curve (see Attachment 5 – NYS Thruway Authority Horizontal and Vertical Plans for I-95, Construction Contract TANE 84-25).

The accident location was located on a crest vertical curve. The vertical grade on the southbound lanes of I-95 prior to the accident location consisted of a positive +2.4% (percent) upgrade slope and a negative -3.0% (percent) downgrade slope.

[12]Retro-reflectivity is the property of a surface that allows a large portion of the light coming from a point source to be returned directly back to a point near its origin.
[13]2009 Manual on Uniform Traffic Control Devices for Streets and Highways, U.S. Department of Transportation, Federal Highway Administration, page 348.
[14]The information on the roadway geometrics was based on design standards/criteria from 1984 when I-95 was reconstructed.

2.7 DESIGN SPEED OF HORIZONTAL CURVE

The design speed for the 1,600 foot radius horizontal curve can be calculated from the following formula[15] that requires the minimum radius, maximum rate of super-elevation (e_{max}), and maximum side friction factor (f_{max}).

$$R_{min} = \frac{V^2}{15(0.01e_{max} + f_{max})}$$

where:

R_{min} = minimum radius; 1,600 feet
V = design speed; miles per hour
e_{max} = maximum rate of super-elevation; 6.25% (percent)
f_{max} = maximum side friction factor; 0.11

$$1{,}600 = \frac{V^2}{15(0.01(6.25) + 0.11)}$$

Design speed = **64 miles per hour**

The design speed for the 1,600 foot radius horizontal curve was calculated to be 64 miles per hour (mph).

2.8 SLOPED CURB

A sloped curb[16] (or mountable concrete curb) was located at the edge of the 10-foot wide paved right and left shoulder (see Figure 3 – View of sloped curb). The height of the sloped curb was approximately 5 inches, measured from the pavement surface (see Attachment 6 – NYS DOT Mountable Concrete Curb (Type AB).

[15] A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials (AASHTO), 2004, Fifth Edition, page 146.
[16] The information on the sloped curb was based on design standards/criteria from 1984 when I-95 was reconstructed, using a mountable concrete curb (Type AB).

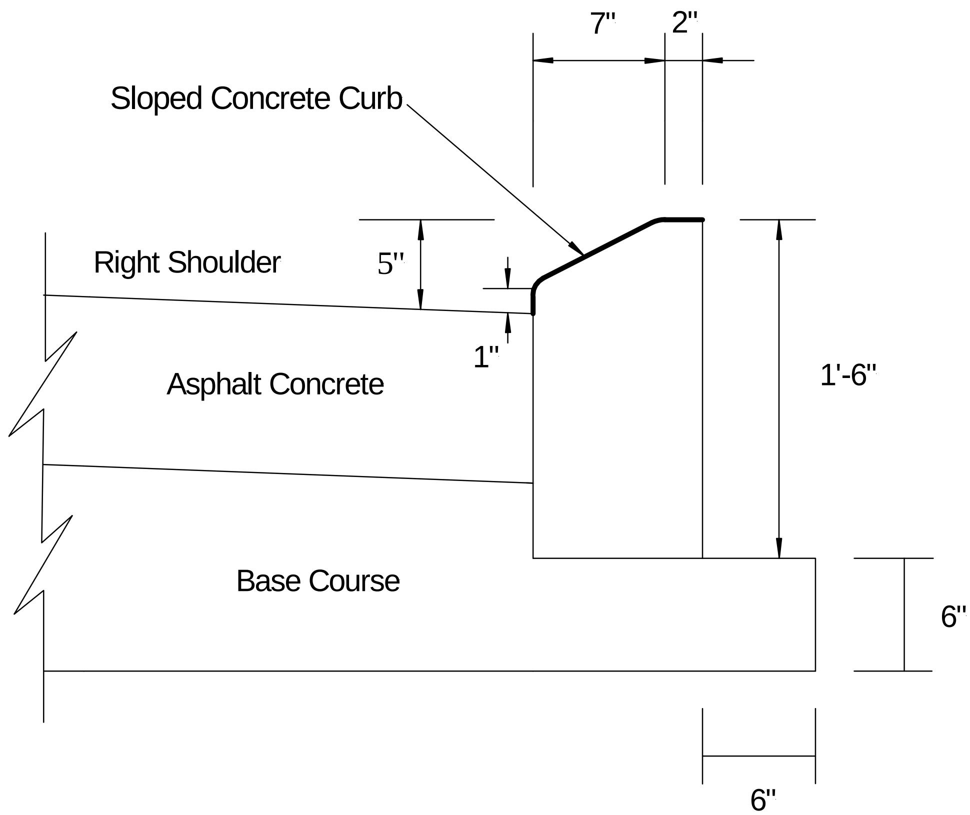

Figure 3 – View of sloped curb (mountable concrete curb, Type AB)

The primary purpose of the mountable curb, when originally constructed in the 1950's, was to convey storm water runoff from the travel lanes of I-95 to a closed drainage system. Closed drainage systems employ catch basins located at the left and right pavement edges that connect to storm drain pipes located underneath the roadway which empty into existing wetlands or drainage channels. Closed drainage systems are typically designed for developed urban areas due to the built up nature of the area and limited space to drain storm water runoff into an open drainage system of roadside ditches. In contrast, open drainage systems are generally employed in rural and suburban areas, due to the ample space available for constructing open drainage ditches adjacent to the roadway.

When the mountable curb was constructed in the vicinity of the accident as part of the original construction of I-95 in the 1950's (construction contract FANETC 54-13), the mountable curb was located at the edge of the travel lane (between the travel lane and shoulder). After a period of time, the pavement area adjacent to the catch basins located at periodic locations along the curb began to fail due to repeated loadings of heavy vehicular traffic. The design report for construction contract TANE 84-25[17] (see Attachment 7 - Design Report for Construction Contract TANE 84-25) indicated the following:

[17]Design Report for Safety Improvements including Rehabilitation of Roadway and Structures, New England Thruway Route I-95, Vollmer Associates, Inc., February 9, 1983, page 21.

"c. <u>Drainage System</u>

The existing drainage system appears to be in generally good condition except for the pavement area adjacent to catch basins. At many locations this pavement is badly cracked and broken up. The apparent cause of this condition is the failure of the masonry course which sits atop the catch basin top slab and supports the catch basin frame and grate. This masonry course, constructed originally of common (red) brick, has disintegrated over the years, leaving the frame and grate unsupported and creating a void beneath the roadway pavement. This void eventually caused the roadway pavement to fail."

To fix this problem, the 1984 reconstruction project (construction contract TANE 84-25) constructed new shoulders flush with the travel lanes and relocated the mountable curb and catch basins to the back edge of the shoulder. The mountable curb was in compliance with the guidance and standard sheet details in force at that time. The AASHTO A Policy on Geometric Design of Highways and Streets – 1984 manual stated *"Barrier curbs should not be used on freeways, but if provided in special cases, the curb should not be closer than the outer edge of shoulder. Mountable curbs, if used, should also be placed at the outer edge of shoulder."* The November 1974 revision of the NYS DOT Highway Design Manual indicated *"Curbs greater than 3 inches high are not to be placed over 1 foot or less than 10 foot in front of a guide rail system to avoid the possibility of vehicles vaulting the rail."* The design report for construction contract TANE 84-25[18] (see Attachment 7 - Design Report for Construction Contract TANE 84-25) indicated the following:

"b. <u>Drainage</u>

As noted in the discussion on the existing drainage system, a major portion of roadway failures occurred at existing drainage structures and it is felt that this is due in large part to repeated loadings of heavy vehicles on these structures. Often, the masonry course, which is used to set the drainage frame to its proper elevation, is a primary location of these failures. This situation will be in part remedied via the construction of flush roadway shoulders and the accompanying relocation of drainage structures to the back of this shoulder."

The 1984 reconstruction project reused the existing storm drain pipes underneath the roadway and followed the same patterns of collecting and discharging storm water runoff as used in the original construction contract.

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[18] Design Report for Safety Improvements including Rehabilitation of Roadway and Structures, New England Thruway Route I-95, Vollmer Associates, Inc., February 9, 1983, page 112.

The American Association of State Highway and Transportation Officials (AASHTO) A Policy on Geometric Design of Highways and Streets indicated the following regarding curb placement[19]:

> *"**Curb Placement***
>
> *…Vertical curbs should not be used along freeways or other high-speed arterials, but if a curb is needed, it should be of the sloping type and should not be located closer to the traveled way than the outer edge of the shoulder…*
>
> *…If a curb is used in conjunction with a traffic barrier, the height of a vertical curb should be limited to 100 mm [4 in] or it should be of the sloping type, ideally, located flush with or behind the face of the barrier…"*

The New York State Department of Transportation Highway Design Manual indicated the following regarding curbs and curb/barrier combinations on high-speed (50 mph or greater) highways[20]:

> *"…Mountable curbing of any height is not to be installed on new or reconstruction projects, except that, when curbing is necessary for drainage control on high-speed roads, mountable curbs with a maximum height of 4 inches may be used at the outside edge of shoulder where the shoulder is of the minimum width specified in Chapter 2 of this manual…*
>
> *…Because of the vaulting concerns mentioned above, when it is necessary to use guide rail adjacent to mountable curbs on high-speed highways, the preferred location is within one foot of the face of the curb…"*

The American Association of State Highway and Transportation Officials (AASHTO) Roadside Design Guide (RDG) is not considered a standard, nor is it a design policy. It is intended for use as a resource document from which individual highway agencies can develop standards and policies. The AASHTO Roadside Design Guide (RDG) indicated the following regarding curbs[21]:

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[19]A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials (AASHTO), 2004, Fifth Edition, pages 322 and 323.

[20]Highway Design Manual, New York State Department of Transportation, Chapter 10 – Roadside Design, Guide Rail, and Appurtenances, Revision 57, June 28, 2010, page 10-37.

[21]Roadside Design Guide, American Association of State Highway and Transportation Officials (AASHTO), 2011 Ballot Version, Chapter 5.

"5.6.2.1 CURBS

…Crash tests have shown that use of most guardrail/curb combinations where high-speed, high-angle impacts are likely should be discouraged... Where there are no feasible alternatives, the designer should consider using a sloping curb no higher than 100 mm [4 in.] and consider stiffening the guardrail to reduce potential deflection. Other measures that may improve performance are bolting a W-beam to the back of the posts, reducing post spacing, nesting the rail, or adding a rubrail…

5.6.2.1.1 Curb/Guardrail Combinations for Strong-Post W-Beam Guardrail

…For design speeds above 80 km/h [50 mph], a 100 mm [4 in.] or shorter sloping curb is recommended for installations where the face of the curb is flush with the face of the guardrail…

Greater than 80 km/h [50 mph]
For design speeds above 80 km/h [50 mph], guardrails should be used with 100 mm [4 in.] or shorter sloping-face curbs, and the face of the curbs should be flush with the face of the guardrail. Above operating speeds of 100 km/h [60 mph], the sloping face should be 1V:3H or flatter and no taller than 100 mm [4 in.] high. Refer to Figure 5.35 for additional details regarding the use of laydown style curb when these guidelines are not practical."

2.9 STRONG POST BLOCKED-OUT W-BEAM GUIDERAIL

A strong post blocked-out W-beam guiderail[22] was offset approximately 3 inches (typical) from the back face of the sloped curb (see Figure 4 – View of strong post blocked-out W-beam guiderail). The W-beam rail element was blocked-out from the posts with W 6 x 9 steel I-beam[23] block-outs. The strong post blocked-out W-beam guiderail was continuous along the right shoulder approaching the accident site and extended approximately 180 feet beyond the overhead sign structure support. A strong post blocked-out W-beam guiderail was also located along the left shoulder approaching the accident site and extended for approximately 135 feet.

The W-beam rail element was raised approximately 12 inches from the ground surface. The height of the W-beam rail element was 12 inches. The total height from the pavement surface to the top of the W-beam rail element was approximately 29 inches[24].

[22]The information on the W-beam guiderail was based on design standards/criteria from 1984 when I-95 was reconstructed.
[23]The W 6 x 9 steel I-beam shape consisted of a 6 inch - height, 4 inch - width, 0.17 inch – web thickness, and 9 pounds per foot – weight.
[24]The total height included a 5 inch high sloped curb.

The strong posts that connected to the W-beam rail element consisted of W 6 x 9 steel I-beam posts that were spaced 6 feet and 3 inches apart. The W 6 x 9 steel I-beam posts were embedded in the ground approximately 5 to 6 feet.

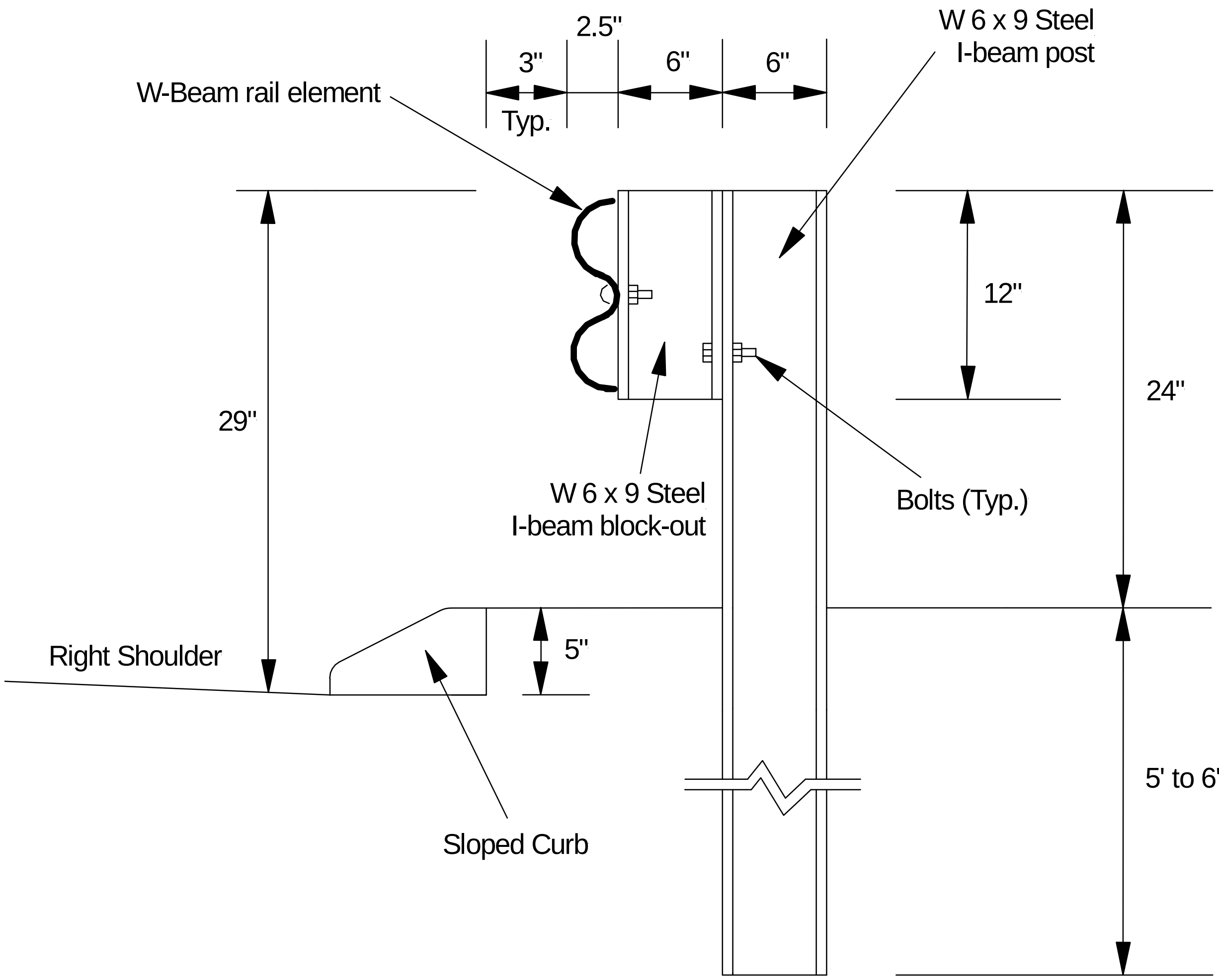

Figure 4 – View of strong post blocked-out W-beam guiderail

The strong post blocked-out W-beam guiderail was initially constructed as part of the 1984 reconstruction project (construction contract TANE 84-25). In the vicinity of the accident, the new strong post blocked-out W-beam guiderail shielded the vertical poles supporting the overhead sign structure support and was not continuous along the right side of the southbound lanes. The W-beam guiderail extended for approximately 300 feet (see Attachment 8 – NYS Thruway Authority Plans for strong post blocked-out W-beam guiderail as part of Construction Contract TANE 84-25).

As part of the 1998 noise barrier project (construction contract 98-70), the strong post blocked-out W-beam guiderail was extended along the entire length of the noise barrier wall. The noise barrier wall was offset approximately 21 feet from the W-beam guiderail in the vicinity of the accident and tapered to a 5 foot offset approximately 2,100 feet north of the accident location. The 1998 noise barrier project provided a continuous guiderail along the right side of the southbound lanes (see Attachment 9 – NYS Thruway Authority Plans for strong post blocked-out W-beam guiderail as part of Construction Contract TANE 98-70).

The strong post blocked-out W-beam guiderail with steel block-out was an approved crash tested barrier system when initially constructed. Crash test performance levels of barrier systems have evolved over the years.

The Transportation Research Board (TRB) National Cooperative Highway Research Program (NCHRP) Project 22-2(4) was initiated to recommend procedures for the safety performance evaluation of barrier systems. That project resulted in a 1981 report, NCHRP Report 230, Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances, which served thereafter as the primary reference for full-scale crash testing in the United States. The crash-test procedures were based on the barrier being evaluated for dynamic performance based on a minimum matrix of conditions. NCHRP Report 230 did not include site-specific guidance as to which vehicle type would be appropriate for a given location.

As knowledge about roadside safety performance evaluations continued to evolve, NCHRP Project 22-7 was formed to update NCHRP Report 230. The result was NCHRP Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features, issued in 1993. NCHRP Report 350 describes full-scale crash testing using six test levels to evaluate the structural adequacy of a barrier system. As with NCHRP Report 230, NCHRP Report 350 did not include specific guidance as to which test level would be appropriate for a given site.

Table 7 summarizes the descriptions of the full-scale crash testing using the six test levels.

Table 7 – Descriptions of the full-scale crash testing using the six test levels

Test Level	Description
TL-1 (Test Level 1)	Successful tests of an 1,800 pound car impacting a barrier at an angle of 20 degrees and a 4,400 pound pickup truck impacting a barrier at an angle of 25 degrees, at speeds of 30 mph.
TL-2 (Test Level 2)	Successful tests of an 1,800 pound car impacting a barrier at an angle of 20 degrees and a 4,400 pound pickup truck impacting a barrier at an angle of 25 degrees, at speeds of 45 mph.
TL-3 (Test Level 3)	Successful tests of an 1,800 pound car impacting a barrier at an angle of 20 degrees and a 4,400 pound pickup truck impacting a barrier at an angle of 25 degrees, at speeds of 60 mph.
TL-4 (Test Level 4)	Successful tests of a 17,600 pound single-unit truck impacting a barrier at an angle of 15 degrees, at speeds of 50 mph.
TL-5 (Test Level 5)	Successful tests of a 80,000 pound tractor-trailer van impacting a barrier at an angle of 15 degrees, at speeds of 50 mph.
TL-6 (Test Level 6)	Successful tests of a 80,000 pound tractor-trailer tanker impacting a barrier at an angle of 15 degrees, at speeds of 50 mph.

The NYS DOT Highway Design Manual[25] indicated the following:

"TL-3 is used in New York State as the normal test level for all other highways except for bridge railings and pier protection. In practice, TL-3 devices are also used for most low-speed highways, rather than using separate TL-2 or TL-1 systems."

Table 8 summarizes the minimum height requirements for bridge railings as contained in the AASHTO LRFD Bridge Design Specifications[26].

Table 8 – Minimum height requirements for bridge railings

Bridge Railing Test Levels – Minimum Height of Railing					
TL-1	**TL-2**	**TL-3**	**TL-4**	**TL-5**	**TL-6**
27 inches	27 inches	27 inches	32 inches	42 inches	90 inches

Most recently, AASHTO has issued the 2009 Manual for Assessing Safety Hardware (MASH) developed under NCHRP Project 22-14(02). MASH represents the latest evolution in barrier testing and will be used to evaluate the structural adequacy of a barrier system based on updated test vehicles and impact conditions. It contains revised criteria for evaluation of highway safety features based on changes in vehicle fleets. MASH replaced NCHRP Report 350 on January 1, 2011[27]. MASH is not a design standard and does not supersede the criteria for the design of roadside barriers contained in the AASHTO Roadside Design Guide. As with the previous NCHRP project documents, MASH utilized full-scale crash testing and did not provide site-specific guidance regarding barrier performance.

The 2006 AASHTO Roadside Design Guide recognized that most roadside barriers were developed, tested, and installed with the intention of containing and redirecting passenger vehicles with masses up to 4,400 pounds. The 2006 AASHTO Roadside Design Guide does not contain objective warrants for the use of higher performance traffic barriers to redirect larger vehicles, such as buses and trucks. However, the AASHTO Roadside Design Guide does mention three subjective factors most often considered for the use of higher performance traffic barriers that include:

- High percentage of heavy vehicles in traffic stream,
- Adverse geometrics, such as sharp curvature, which are often combined with poor sight distance, and

[25]New York State Department of Transportation Highway Design Manual, Appendix C, page 10C-3.
[26]LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials (AASHTO), 2010, Fifth Edition, page 13-17.
[27]This date only applies to hardware that was being tested when MASH was published in October 2009. All new roadside hardware crash testing has been under the requirements of MASH since October 2009.

- Severe consequences associated with penetration of a barrier by a large vehicle.

Recognizing the need for better guidance for selecting the appropriate guardrail performance level for a given site, NCHRP Project 22-12(02) was initiated that resulted in a 2009 report, NCHRP Report 638, Guidelines for Guardrail Implementation. The objectives of NCHRP Report 638 included the following: 1) Developing objective guardrail selection guidelines that would provide specific guidance for identifying the most cost beneficial guardrail performance level to be used on any given route, 2) Identifying when a more detailed analysis was warranted, and 3) Presenting procedures for conducting a more thorough evaluation of guardrail need, when necessary.

FHWA's position on all new or replacement of roadside barriers has been the following[28]: *"Except as modified below, all new or replacement safety features on the NHS covered by the guidelines in the NCHRP Report 350 that are included in projects advertised for bids or are included in work done by force-account or by State forces on or after October 1, 1998, are to have been tested and evaluated and found acceptable in accordance with the guidelines in the NCHRP Report 350."*

FHWA indicated in a letter to Division Administrators and Federal Lands Highway Division Engineers dated February 14, 2000 (see Attachment 10 – FHWA Memorandum to Division Administrators and Federal Lands Highway Division Engineers dated February 14, 2000) that the strong post blocked-out W-beam guiderail with steel block-out had been crash tested and accepted in accordance with the guidelines in NCHRP Report 350 as a TL-2 barrier. The same letter indicated that the strong post blocked-out W-beam guiderail with wood or approved plastic block-out had been accepted as a TL-3 barrier.

Most recently, new highway safety hardware not previously evaluated under NCHRP Report 350 must utilize MASH for testing and evaluation. The AASHTO LRFD Bridge Design Specifications indicated the following[29]: *"All highway safety hardware accepted prior to the adoption of AASHTO, Manual for Assessing Safety Hardware (MASH), using criteria contained in NCHRP Report 350, may remain in place and may continue to be manufactured and installed. Highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. New highway safety hardware not previously evaluated must utilize MASH for testing and evaluation."*

FHWA's position on when an existing roadside barrier should be upgraded has been the following[30]: *"The FHWA does not intend that this requirement (that new highway safety features installed on the NHS be proven crashworthy in accordance with the guidelines in the*

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[28]Federal Highway Administration, Memorandum to Regional Administrators, Federal Lands Highway Program Administrator, Division Administrators, and Federal Lands Highway Division Engineers from Director, Office of Engineering; Identifying Acceptable Highway Safety Features, July 25, 1997, page 2.
[29]LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials (AASHTO), 2010 Interim Revisions, Fifth Edition, page 121.
[30]Federal Highway Administration, Memorandum to Regional Administrators, Federal Lands Highway Program Administrator, Division Administrators, and Federal Lands Highway Division Engineers from Director, Office of Engineering; Identifying Acceptable Highway Safety Features, July 25, 1997, page 3.

NCHRP Report 350) result in the replacement or upgrading of any existing installed features beyond what would normally occur with planned highway improvements. On the other hand, a State should have a rational, documented policy for determining when an existing non-standard feature should be upgraded."

The American Association of State Highway and Transportation Officials (AASHTO) Roadside Design Guide (RDG) classified strong post blocked-out W-beam[31] as the following:

> *"**5.4.1.5 Blocked-Out W-Beam (Strong Post)***
>
> *Strong-post W-beam is the most common barrier system in use today. It consists of steel posts (SGRO4a) as shown in Figure 5.9 or wood posts (SGRO4b) as shown in Figure 5.10 that support a W-beam rail element that is blocked out from the posts with routed timber, steel, or recycled plastic spacer blocks. These blocks minimize vehicle snagging on the posts and reduce the likelihood of a vehicle vaulting over the barrier by maintaining rail height during the initial stages of post deflection. Resistance in this and all strong post systems results from a combination of tensile and flexural stiffness of the rail and the bending or shearing resistance of the posts.*
>
> *Several acceptable strong post W-beam designs are in use. The spacer blocks are typically timber or recycled plastic with a 150 mm [6 in.] width to match each posts dimensions. One of the most commonly used designs, the steel post guardrail system with steel blocks, failed to meet the NCHRP Report 350 evaluation criteria at TL-3 when the pickup truck snagged on a post and subsequently overturned. However, this system remains acceptable as a TL-2 barrier. In order to provide a TL-3 barrier with steel posts, 150 mm x 200 mm (6 in. x 8 in.) routed wood or plastic blocks of similar dimensions should be used as a substitute for the steel blocks…*
>
> *The standard length for timber posts has been increased to 1830 mm [6 feet] to match the length of steel posts, however, the recent Report 350 tests used the original 1625 mm [5 ft 4 in.] posts and either length remains acceptable. The original height to the top of the rail for strong post W-beam was 685 mm [27 in.]. This was slightly modified when the height measurement was changed from the top of the rail to the center of the rail with the adoption of metric units. A 550 mm [21.5 in.] height to the center of the rail translated to a 706 mm [28 in.] top height. Either top rail height is considered acceptable…*
>
> *Impact performance: Based primarily on testing of the two common designs noted above, this system is effective at redirecting vehicles in the 820-2000 kg [1,800-4,400 lb] range. The wood post (SGR04b) system with wood blocks passed the NCHRP Report 350 TL-3 test with a 2000 kg [4,400 lb] pickup truck (24.3-degree impact angle, 100.8 km/h [62.5 mph]). The maximum lateral*

[31]Roadside Design Guide, American Association of State Highway and Transportation Officials (AASHTO), 2006, Third Edition, pages 5-12 through 5-15.

defection was 0.8 m [31.5 in.]. A steel post system with a 150 mm x 200 mm [6 in. x 8in.] routed wood block also passed the NCHRP Report 350 TL-3 test with the 2000 kg [4,400 lb] pickup truck (25.5-degree impact angle, 101.5 km/h [63 mph]). The maximum lateral deflection of this system was 1.0 m [3.3 ft]."

2.10 OVERHEAD SIGN STRUCTURE SUPPORT

The vertical poles that supported the overhead sign structure support were considered fixed-base support systems (see Highway Factors Photograph (HWY Photo-03) illustrating the overhead sign structure support above the southbound travel lanes of I-95). Fixed-base supports systems are designed to not yield or break away on impact. The overhead sign structure support was supported by four vertical poles (two located beyond the paved right shoulder and two located beyond the paved left shoulder). The vertical poles consisted of 8 inch diameter steel tubular poles[32]. The vertical poles were separated by a lateral distance of 5 feet and 2 inches (see Attachment 11 – NYS Thruway Authority Signage and Structure Plans for I-95, Construction Contract TANE 84-25).

The vertical poles were located within the clear zone and offset from the edge of traveled way by 15 feet. The clear zone concept involves providing a traversable and unobstructed roadside area beyond the traveled way for use by errant vehicles. The width of the clear zone is usually set at 30 feet[33] for freeways. If an obstacle is located within the clear zone, it generally should be removed, relocated, redesigned, or shielded by traffic barriers or crash cushions.

The height of the east vertical pole supporting the overhead sign structure support was approximately 28 feet and the height of the west vertical pole was approximately 27 feet (see Highway Factors Photograph (HWY Photo-04) illustrating the height of the east vertical pole was approximately 28 feet and the height of the west vertical pole was approximately 27 feet). Figure 5 illustrates the base plate and connection details of the vertical poles. The bolts used to anchor the base plate were 1 ½ inch diameter anchor bolts[34]. The base plate outside diameter was 17 inches and the base plate inside diameter was 8.625 inches. The base plate thickness was 1.375 inches and the vertical pole wall thickness was .3125 inches.

The NYS Thruway Authority could not find any records to indicate the vertical poles that supported the overhead sign structure support in the vicinity of the accident had been hit in the past.

[32]ASTM A53 Grade B Type E Pipe, 8 inch Schedule 40 STD.
[33]The 30 feet is measured from the edge of paved traveled way, or the intersection of the paved traveled way and shoulder.
[34]36,000 pounds per square inch (P.S.I.) minimum yield anchor bolts.

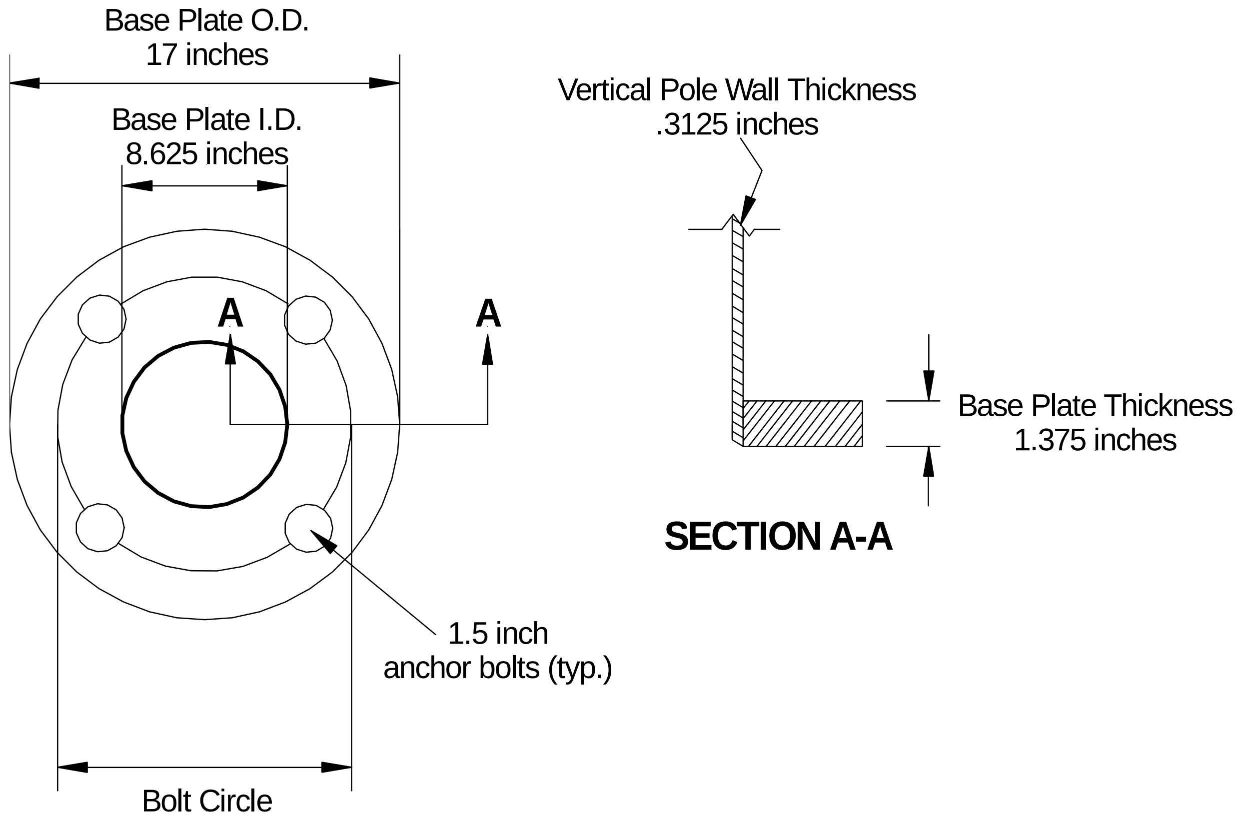

Figure 5 – Base Plate and Connection Details of the Vertical Poles

The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals indicated the following regarding overhead sign supports and high-level lighting supports[35]:

> *"**2.5.5 Overhead Sign Supports and High-Level Lighting Supports***
>
> *Overhead sign and high-level lighting structural supports should be placed outside the clear zone distance; otherwise, they should be protected with a proper guardrail or other barrier.*
>
> ***C2.5.5***
>
> *Overhead sign and high-level lighting supports are considered fixed-base support systems that do not yield or break away on impact. The large mass of these support systems and the potential safety consequences of the systems falling to the ground necessitate a fixed-base design. Fixed-base systems are rigid obstacles and should not be used in the clear zone area unless shielded by a barrier. In*

[35]Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, American Association of State Highway and Transportation Officials (AASHTO), 2009, Fifth Edition, page 2-7.

some cases, it may be cost effective to place overhead sign supports outside the clear zone with no barrier protection when the added cost of the greater span structure is compared with the long-term costs of guardrail and vegetation maintenance."

2.11 STAR (SHOULDER TREATMENT FOR ACCIDENT REDUCTION)

Grooved rumble strips[36] (commonly referred to as star - shoulder treatment for accident reduction) existed on the paved right shoulder and paved left shoulder of the southbound lanes of I-95 (see Attachment 12 – NYS Thruway Authority Plans and Specifications for Shoulder Treatment for Accident Reduction (STAR) for I-95, Construction Contract TANE 06-21). The rumble strip dimensions were approximately 15.7 inches (or 400 mm) long and 7 inches (or 180 mm) wide. The rumble strips were spaced approximately 4.8 inches apart (or 11.8 inches from centerline of rumble strip to centerline of rumble strip). The depression of the rumble strip into the shoulder was approximately 0.5 inches (or 12 mm). The rumble strips were offset from the edge of traveled way by approximately 15.7 inches (or 400 mm) (see Highway Factors Photograph (HWY Photo-05) illustrating the grooved rumble strips on the paved right shoulder of the southbound lanes of I-95 looking to the north).

2.12 SUPER-ELEVATION OF THE TRAVEL LANES OF I-95

The super-elevation of the southbound travel lanes of I-95 in the vicinity of the accident location was a positive +3% (percent) cross slope, sloped upward from left to right, in the direction of travel. The super-elevation of the right shoulder in the vicinity of the accident location was a negative -2% (percent) cross slope, sloped downward from left to right, in the direction of travel.

2.13 HIGHWAY LIGHTING

As part of the 1984 reconstruction project (construction contract TANE 84-25) the entire existing lighting system on I-95 was removed and replaced. In the vicinity of the accident, two 400 watt high pressure sodium symmetrical distribution luminaires were mounted on top of a single pole. The single pole extended approximately 45 feet high from the finished grade. The spacing of the single poles was approximately 250 feet on center (see Attachment 13 – NYS Thruway Authority Highway Lighting Plans for I-95, Construction Contract TANE 84-25).

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[36]Grooved rumble strips are also referred to as SNAP (Sonic Noise Alert Pattern), rumbles, rumble strips, and rumble stripes if a paint line is applied over them.

3. PHYSICAL EVIDENCE

NTSB investigators documented the evidence of tire marks on the paved right shoulder as a result of the bus being redirected onto the shoulder of the roadway after collision with the strong post blocked-out W-beam guiderail. The initial point of contact with the guiderail was approximately 480 feet from the final rest of the bus (see Highway Factors Photograph (HWY Photo-06) illustrating the initial point of contact with the guiderail). Figure 6 illustrates the overall scene diagram showing the tire marks on the paved right shoulder and the progression of the bus from the initial point of contact with the guiderail to final rest.

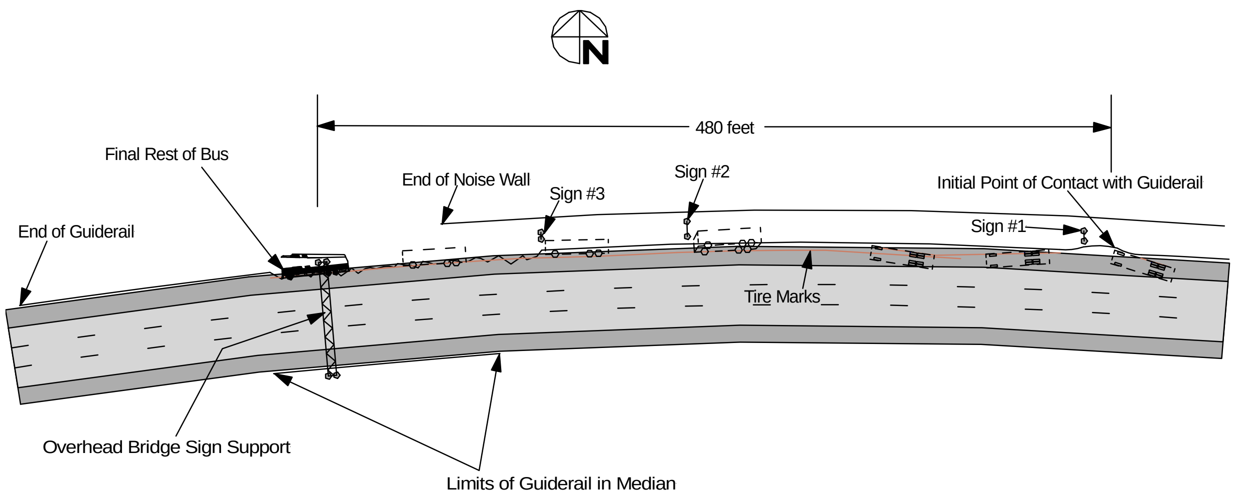

Figure 6 – Overall scene diagram

The sign numbers shown in Figure 6 consisted of the following:

Sign #1 – Within N.Y.C. Limits Hazardous Liquefied Gas Tank or Explosives Trucks Prohibited Unless Authorized by NYC Fire Dept Ph # 718-999-2094 Hazardous Cargo Thru Shipment Must Follow Designated Routes & Times Penalty: $10,000 & 30 Days (see Highway Factors Photograph (HWY Photo-07) illustrating Sign #1 located approximately 453 feet from the final rest of the bus)

Sign #2 – Truck Route to I-678 South Whitestone Br Use Exit 6A (see Highway Factors Photograph (HWY Photo-08) illustrating Sign #2 located approximately 217 feet from the final rest of the bus)

Sign #3 – Welcome to the Bronx Mayor Michael R. Bloomberg Boro Pres. Ruben Diaz Jr. (see Highway Factors Photograph (HWY Photo-09) illustrating Sign #3 located approximately 130 feet from the final rest of the bus)

NTSB investigators documented the angle of approach in which the bus departed the travel lanes and initially collided with the strong post blocked-out W-beam guiderail. The angle of approach was calculated to be approximately 7.2 degrees (see Highway Factors Photograph (HWY Photo-10) illustrating the angle of approach in which the bus departed the travel lanes and initially collided with the guiderail). Table 9 summarizes the progression of the bus along the W-beam guiderail.

Table 9 – Progression of bus along the W-beam guiderail

Distance from Final Rest of the Bus	Comments
480 feet	Initial point of contact with the guiderail.
460 feet	Deformation of the guiderail measured to be approximately 18 inches. Posts moved perpendicular to the direction of traffic. Applied force created a moment about the strong axis of the posts.
453 feet	Location of Sign #1 (damage to lower left hand corner of sign) (see Highway Factors Photograph (HWY Photo-11) illustrating the damage to lower left hand corner of Sign #1).
439 feet	End of initial impact with guiderail.
430 feet	Beginning of continuous tire mark offset 0.5 feet from curb line.
403 – 407 feet	4 foot gouge mark in right shoulder offset 2 foot from curb line.
380 feet	Continuous tire mark offset 3 foot from curb line. Beginning of multiple tire marks offset 4 foot, 4.5 foot, and 5 foot from curb line.
331 feet	Second point of contact with the guiderail. Continuous tire mark offset 1 foot from curb line. End of multiple tire marks offset 1.5 foot, 2.5 foot, and 3 foot from curb line.
300 feet	Start of bus tilting on guiderail. Continuous tire mark offset 1 foot from curb line. Guiderail being pushed down to 20 inch height[37].
217 feet	Location of Sign #2 (damage to lower left hand corner of sign) (see Highway Factors Photograph (HWY Photo-12) illustrating the damage to lower left hand corner of Sign #2). Continuous tire mark offset 1.5 foot from curb line. Guiderail being pushed down to 18 inch height.
130 feet	Location of Sign #3 (damage and removal of left post) (see Highway Factors Photograph (HWY Photo-09) illustrating the damage and removal of left post supporting Sign #3). Continuous tire mark offset 1.5 foot from curb line. Guiderail being pushed down to 12 inch height. Guiderail element separates from posts. Posts deformed and turned down 45 degrees. All turned down posts were in the direction of travel, about the weak axis of the posts.
100 feet	Continuous tire mark offset 1.5 foot from curb line. Guiderail being pushed down to 10 inch height. Guiderail element separates from posts. Posts deformed and turned down 90 degrees. All turned down posts were in the direction of travel, about the weak axis of the posts (see Highway Factors Photograph (HWY Photo-13) illustrating the rail element completely separated from the posts and the posts deformed and turned down 45 degrees and 90 degrees).
0 feet	Final rest of the bus.

[37]Measurement was taken from the top of sloped curb to the top of W-beam rail element.

NTSB investigators determined the number of steel I-beam posts destroyed as a result of the accident was 40. Table 10 summarizes the description of damage to the steel I-beam posts.

Table 10 - Description of damage to the steel I-beam posts

Description of Damage to the Steel I-beam Posts	Number
Steel I-beam posts deformed and turned down 90 degrees flush with the ground surface	20
Steel I-beam posts deformed and turned down 45 degrees	11
Steel I-beam posts with minor deformation	9
Total	**40**

Two E-Z Pass transponder devices were mounted to the overhead sign structure support above the travel lanes (see Highway Factors Photograph (HWY Photo-14) illustrating the two E-Z Pass transponder devices mounted to the overhead sign structure support above the travel lanes). The conduit for the transponder devices was wired to a cabinet mounted to the cross bracing between the vertical poles (see Highway Factors Photographs (HWY Photo-15 and 16) illustrating the location of the cabinet mounted to the cross bracing before the accident). During the on-scene investigation, NTSB investigators observed the cabinet was dislodged from the cross bracing (see Highway Factors Photograph (HWY Photo-17) illustrating the final rest of the cabinet after the accident). In addition, two wire conduits were attached to the overhead sign structure support to feed a VMS sign located above the northbound lanes of I-95 (see Highway Factors Photograph (HWY Photo-18) illustrating the two wire conduits dislodged from the cross bracing to feed a VMS sign located above the northbound lanes of I-95).

NTSB investigators documented a punched hole in the west vertical pole. The punched hole was located at the base of the west vertical pole and measured 2 inches by 3 inches (see Highway Factors Photograph (HWY Photo-19) illustrating the punched hole located at the base of the west vertical pole). An indent was observed on the west vertical pole that measured 9 feet from the base of the pole. Multiple scrape marks were documented on the east vertical pole measured from the base of the pole to a height of 9 feet (see Highway Factors Photograph (HWY Photo-20) illustrating the multiple scrape marks on the east vertical pole measured from the base of the pole to a height of 9 feet).

4. REPAIRS MADE BY THE NYS THRUWAY AUTHORITY AFTER THE ACCIDENT

The NYS Thruway Authority repaired damaged sections of strong post blocked-out W-beam guiderail between March 15 and March 16, 2011. The repair included installing approximately 256 feet of new strong post W-beam guiderail with steel block-outs (see Highway Factors Photograph (HWY Photo-21) illustrating the repairs made by the NYS Thruway Authority between March 15 and March 16, 2011 to install new strong post W-beam guiderail with steel block-outs).

On April 29, 2011, the NYS Thruway Authority completed the replacement of the steel block-outs with plastic block-outs. The block-out replacement extended for approximately 627 feet from Sign #1 to south of the overhead sign structure support. The replacement (steel block-outs with plastic block-outs) upgraded the performance of the strong post W-beam guiderail from a TL-2 barrier to a TL-3 barrier (see Highway Factors Photograph (HWY Photo-22) illustrating the repairs made by the NYS Thruway Authority on April 29, 2011 to replace the steel block-outs with plastic block-outs). In addition, approximately 38 feet of strong post blocked-out W-beam guiderail was replaced for alignment adjustment as part of the plastic block-out replacement work.

The NYS Thruway Authority is determining whether the punched hole and indent to the 8 inch diameter steel tubular poles can be repaired or if the vertical poles need to be replaced in the near future. An examination conducted by the NYS Thruway Authority immediately after the accident determined the structural integrity of the 8 inch diameter steel tubular poles was not compromised as a result of the accident and the overhead sign structure support was fully functional.

5. TESTS AND RESEARCH

5.1 85th PERCENTILE SPEED STUDY

The NYS Thruway Authority conducted an 85th percentile speed study[38] on the southbound lanes of I-95 in the vicinity of the accident location on April 14, 2011. The speed study was performed using an UltraLyte[39] laser gun. Approximately 800 passenger cars, 200 trucks, and 29 buses were recorded as part of the speed study. The 85th percentile speed study revealed that passenger cars were traveling 67 miles per hour (mph), trucks were traveling 60 mph, and buses were traveling 59 mph. Table 11 summarizes the 85th percentile speed study on the southbound lanes of I-95 in the vicinity of the accident location.

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[38]The 85th percentile speed is the speed at which 85% of the vehicle traffic is traveling either at or below that speed or, 15% of the vehicle traffic is traveling above that speed.
[39]Manufactured by Laser Technology Inc., sample of specifications include weight 2.95 lbs, speed accuracy +/- 1 mph, maximum target range 2,000 feet, and speed range +/- 200 mph.

Table 11 – 85th percentile speed study on the southbound lanes of I-95 in the vicinity of the accident location

Passenger Cars			Trucks			Buses		
Miles Per Hour	Number of Passenger Cars	Total	Miles Per Hour	Number of Trucks	Total	Miles Per Hour	Number of Buses	Total
81 mph	1	800	65 mph	1	200	68 mph	1	29
80 mph	1	799	64 mph	1	199	67 mph	0	28
79 mph	0	798	63 mph	5	198	66 mph	0	28
78 mph	1	798	62 mph	3	193	65 mph	0	28
77 mph	3	797	61 mph	7	190	64 mph	0	28
76 mph	0	794	60 mph	16	183	63 mph	0	28
75 mph	2	794	59 mph	17	167	62 mph	0	28
74 mph	2	792	58 mph	9	150	61 mph	0	28
73 mph	3	790	57 mph	19	141	60 mph	2	28
72 mph	11	787	56 mph	16	122	59 mph	5	26
71 mph	11	776	55 mph	25	106	58 mph	3	21
70 mph	13	765	54 mph	21	81	57 mph	0	18
69 mph	26	752	53 mph	21	60	56 mph	0	18
68 mph	35	726	52 mph	7	39	55 mph	2	18
67 mph	36	691	51 mph	9	32	54 mph	3	16
66 mph	31	655	50 mph	7	23	53 mph	3	13
65 mph	44	624	49 mph	2	16	52 mph	3	10
64 mph	37	580	48 mph	4	14	51 mph	2	7
63 mph	59	543	47 mph	5	10	50 mph	0	5
62 mph	73	484	46 mph	0	5	49 mph	1	5
61 mph	59	411	45 mph	0	5	48 mph	1	4
60 mph	60	352	44 mph	1	5	47 mph	1	3
59 mph	57	292	43 mph	1	4	46 mph	0	2
58 mph	45	235	42 mph	0	3	45 mph	2	2
57 mph	35	190	41 mph	2	3			
56 mph	40	155	40 mph	1	1			
55 mph	34	115						
54 mph	23	81						
53 mph	13	58						
52 mph	13	45						
51 mph	17	32						
50 mph	5	15						
49 mph	3	10						
48 mph	2	7						

47 mph	2	5		
46 mph	1	3		
45 mph	1	2		
44 mph	1	1		
Top Speed = 81 mph Total Passenger Cars = 800 85th Percentile Speed = 67 mph			**Top Speed = 65 mph Total Trucks = 200 85th Percentile Speed = 60 mph**	**Top Speed = 68 mph Total Buses = 29 85th Percentile Speed = 59 mph**

5.2 AASHTO ROADSIDE DESIGN GUIDE – ROADSIDE BARRIERS

The guidance for the selection of roadside barriers was contained in the American Association of State Highway and Transportation Officials (AASHTO) Roadside Design Guide (RDG), 2006, Third Edition. The Roadside Design Guide[40] indicated the following:

*"**5.0 OVERVIEW***

A roadside barrier is a longitudinal barrier used to shield motorists from natural or man-made obstacles located along either side of a traveled way…

5.1 PERFORMANCE REQUIREMENTS

The primary purpose of all roadside barriers is to prevent a vehicle from leaving the traveled way and striking a fixed object or terrain feature that is less forgiving than striking the barrier itself…

5.1.1 Current Crash Test Criteria

…TL-1, TL-2, and TL-3 require successful tests of an 820 kg [1,800 lb] car impacting a barrier at an angle of 20 degrees and a 2000 kg [4,400 lb] pickup truck impacting a barrier at an angle of 25 degrees, at speeds of 50 km/h, 70 km/h, and 100 km/h [30 mph, 40 mph, and 60 mph], respectively. TL-4 adds an 8000 kg [17,600 lb] single-unit truck at an impact angle of 15 degrees and 80 km/h [50 mph] to the TL-3 matrix. TL-5 substitutes a 36000 kg [80,000 lb] tractor-trailer (van) for the single-unit truck and TL-6 substitutes a 36000 kg [80,000 lb] tractor-trailer (tanker)…

[40]Roadside Design Guide, American Association of State Highway and Transportation Officials (AASHTO), 2006, Third Edition, pages 5-1 through 5-24.

5.2 WARRANTS

Barrier warrants are based on the premise that a traffic barrier should be installed only if it reduces the severity of potential crashes…

…Typically, guardrail warrants have been based on a subjective analysis of certain roadside elements or conditions. If the consequences of a vehicle striking a fixed object or running off the road are believed to be more serious than hitting a traffic barrier, then the barrier is considered warranted…

5.2.2 Roadside Obstacles

…However, a barrier should be installed only if it is clear that the result of a vehicle striking the barrier will be less severe than the crash resulting from hitting the unshielded object.

Non-traversable terrain and roadside obstacles that normally warrant shielding are listed in Table 5.1…

Table 5.1 Barrier warrants for non-traversable terrain and roadside obstacles (See Notes 1&2)

Obstacle	***Warrants***
Bridge piers, abutments, and railing ends	*Shielding generally required*
Boulders	*Judgment decision based on nature of fixed object and likelihood of impact*
Culverts, pipes, headwalls	*Judgment decision based on size, shape, and location of obstacle*
Cut & fill slopes (smooth)	*Shielding not generally required*
Cut & fill slopes (rough)	*Judgment decision based on likelihood of impact*
Ditches (parallel)	*Refer to Figures 3.6 and 3.7*
Ditches (transverse)	*Shielding generally required if likelihood of head-on impact is high*
Embankment	*Judgment decision based on fill height and slope (see Figure 5.1)*
Retaining walls	*Judgment decision based on relative smoothness of wall*

	and anticipated maximum angle of impact
Sign / luminaire supports (See Note 3)	*Shielding generally required for non-breakaway supports*
Traffic signal supports (See Note 4)	*Isolated traffic signals within clear zone on high-speed rural facilities may warrant shielding*
Trees	*Judgment decision based on site-specific circumstances*
Utility poles	*Shielding may be warranted on a case-by-case basis*
Permanent bodies of water	*Judgment decision based on location and depth of water and likelihood of encroachment*

Notes:

1. ***Shielding non-traversable terrain or a roadside obstacle is usually warranted only when it is within the clear zone and cannot practically or economically be removed, relocated, or made breakaway, and it is determined that the barrier provides a safety improvement over the unshielded condition.***
2. ***Marginal situations, with respect to placement or omission of a barrier, will usually be decided by crash experience, either at the site or at a comparable site.***
3. ***Where feasible, all sign and luminaire supports should be a breakaway design regardless of their distance from the roadway if there is reasonable likelihood of their being hit by an errant motorist. The placement and locations for breakaway supports should also consider the safety of pedestrians from potential debris resulting from impacted systems.***
4. ***In practice, relatively few traffic signal supports, including flashing light signals and gates used at railroad crossings, are shielded. If shielding is deemed necessary, however, crash cushions are sometimes used in lieu of a longitudinal barrier installation.***

5.3 PERFORMANCE LEVEL SELECTION FACTORS

Most roadside barriers were developed, tested, and installed with the intention of containing and redirecting passenger vehicles with masses up to 2000 kg [4,400 lb]. Properly designed and installed barrier systems have proven to be very effective in reducing the amount of damage and lessening the severity of personal injuries when struck by automobiles and similar-sized vehicles at relatively shallow angles (less than 25 degrees) and at reasonable impact speeds (less than 110 km/h [65 mph]). However, it has long been understood that barriers designed for cars should not be expected to perform equally well for larger vehicles, such as buses and trucks. Recognizing this fact, several highway agencies have developed and used barrier systems capable of redirecting vehicles as heavy as 36,000 kg [80,000 lb] tractor-trailer combination trucks. Although objective warrants for the use of higher performance traffic barriers do not presently exist, subjective factors most often considered for new construction or safety upgrading include:

- *high percentage of heavy vehicles in traffic stream,*

- *adverse geometrics, such as sharp curvature, which are often combined with poor sight distance, and*

- *severe consequences associated with penetration of a barrier by a large vehicle.*

These same factors apply on reconstruction or rehabilitation projects but, in these cases, the designer will usually have the added benefit of past crash history, the past performance of the system, and maintenance costs associated with the existing barrier. In addition, a higher performance barrier is likely to lessen the severity of future crashes or reduce maintenance costs significantly…

5.4.1 Standard Sections of Roadside Barriers

Roadside barriers are usually categorized as flexible, semi-rigid, or rigid, depending on their deflection characteristics on impact. Flexible systems are generally more forgiving than the other categories since much of the impact energy is dissipated by the deflection of the barrier and lower impact forces are imposed upon the vehicle.

This section is not intended to be all-inclusive, but to cover the most widely used roadside barriers. The barriers and approved test levels included in the following sub-sections are listed in Table 5.2.

Table 5.2 Roadside barriers and approved test levels

Barrier System (with AASHTO-AGC-ARTBA designation)	***Test Level***
Flexible Systems	
3-Strand Cable (Weak Post)	*TL-3*
W-Beam (Weak Post)	*TL-2*
Modified W-Beam (Weak Post)	*TL-3*
Ironwood Aesthetic Barrier	*TL-3*
Semi-Rigid Systems	
Box Beam (Weak Post)	*TL-3*
Blocked-out W-Beam (Strong Post) - *Steel or Wood Post with Wood or Plastic Block* - *Steel Post with Steel Block*	 *TL-3* *TL-2*
Blocked-out Thrie-Beam (Strong Post) - *Wood or Steel Post with Wood or Plastic Block*	 *TL-3*
Modified Thrie-Beam (Strong Post)	*TL-4*
Merritt Parkway Aesthetic Guardrail	*TL-3*
Steel-Backed Timber Guardrail	*TL-3*
Rigid Systems (Concrete & Masonry)	

New Jersey Concrete Safety Shape - *810 mm [32 in.] tall* - *1070 mm [42 in.] tall*	 *TL-4* *TL-5*
F-Shape Barrier - *810 mm [32 in.] tall* - *1070 mm [42 in.] tall*	 *TL-4* *TL-5*
Vertical Concrete Barrier - *810 mm [32 in.] tall* - *1070 mm [42 in.] tall*	 *TL-4* *TL-5*
Single Slope Barrier - *810 mm [32 in.] tall* - *1070 mm [42 in.] tall*	 *TL-4* *TL-5*
Ontario Tall Wall Median Barrier	*TL-5*
Stone Masonry Wall/Precast Masonry Wall	*TL-3*

5.5 SELECTION GUIDELINES

Once it has been decided that a roadside barrier is warranted, a specific barrier type must be selected. Although a number of variables and the lack of objective criteria complicate this selection process, there are some general guidelines that may be followed. The most desirable system is usually one that offers the required degree of shielding at the lowest cost for the specific application. Table 5.3 summarizes the factors that should be considered before making a final selection. Each of these factors is described in more detail in the following subsections.

5.5.1 Barrier Performance Capability

The first decision to be made when selecting an appropriate traffic barrier concerns the level of performance required. Barriers passing NCHRP Report 350, TL-2 have been developed primarily for passenger cars and light trucks in low-severity impacts. TL-2 barriers offer marginal and/or limited protection when struck by heavier vehicles such as trucks and buses at high speeds and large angles of impact. If passenger vehicles are the main concern, a standard railing that satisfies other criteria (as listed in subsequent sections) will normally be selected. Locations with poor geometrics, high traffic volumes or speeds, or both, and a significant volume of heavy truck traffic, may warrant a higher performance level or stronger railing system (i.e., NCHRP Report 350, TL-4 or greater). This is especially true if barrier penetration by a vehicle is likely to have serious consequences to other than the motorist.

Table 5.3 Selection criteria for roadside barriers

Criteria	***Comments***
Performance capability	*Barrier must be structurally able to contain and redirect design vehicle*
Deflection	*Expected deflection of barrier should not exceed available deflection distance*
Site conditions	*Slope approaching the barrier and distance from traveled way may preclude use of some barrier types*
Compatibility	*Barrier must be compatible with planned end anchor and capable of transitioning to other barrier systems (such as bridge railing)*
Cost	*Standard barrier systems are relatively consistent in cost, but high-performance railings can cost significantly more*
Maintenance	
- ***Routine***	*Few systems require a significant amount of routine maintenance*
- ***Collision***	*Generally, flexible or semi-rigid systems require significantly more maintenance after a collision than rigid or high-performance railings*
- ***Material storage***	*The fewer different systems used, the fewer inventory items/storage space required*
- ***Simplicity***	*Simpler designs, besides costing less, are more likely to be reconstructed properly by field personnel*
Aesthetics	*Occasionally, barrier aesthetics are an important consideration in selection*
Field experience	*The performance and maintenance requirements of existing systems should be monitored to identify problems that could be lessened or eliminated by using a different barrier type*

5.3 AASHTO ROADSIDE DESIGN GUIDE – CURB/GUARDRAIL COMBINATIONS

The guidance for the selection of curb/guardrail combinations was contained in the American Association of State Highway and Transportation Officials (AASHTO) Roadside Design Guide (RDG), 2011 Ballot Version. The Roadside Design Guide[41] indicated the following:

*"**5.6.2.1 CURBS***

Section 3.4.1 addresses the use of curbs primarily as drainage control features and presents only very general guidelines concerning their use in conjunction with traffic barriers. When a vehicle strikes a curb, the trajectory of the vehicle depends upon several variables: the size and suspension characteristics of the vehicle, its impact speed and angle, and the height and shape of the curb itself. Crash tests have shown that use of most guardrail/curb combinations where high-speed, high-angle impacts are likely should be discouraged. However, the MGS and Trinity $T\text{-}31^{TM}$ *barrier have been developed and approved to be used in conjunction with curbs. Where there are no feasible alternatives, the designer should consider using a sloping curb no higher than 100 mm [4 in.] and consider stiffening the guardrail to reduce potential deflection. Other measures that may improve performance are bolting a W-beam to the back of the posts, reducing post spacing, nesting the rail, or adding a rubrail. On lower-speed facilities, a vaulting potential still exists, but since the risk of such an occurrence is lessened, a design change may not be cost-effective. A case-by-case analysis of each situation considering the anticipated speeds and consequences of vehicular penetration should be used. The layout of the barrier and curb should be considered by the designer.*

Preferably, strong-post W-beam guardrail should not be located at an offset from a curb on roads with design speeds of greater than 60 km/h [40 mph], unless a crash tested system has been developed. However, sometimes it is required to offset the barrier from the curb. In these locations where the curb is offset or the barrier flares away from the edge of the roadway, the curb should be transitioned to a laydown curb similar to Figure 5.35. The performance of guardrail terminals behind curbs has not been tested. One transportation agency addresses this issue by transitioning the curb to a laydown curb and carrying this laydown style curb past the terminal to accommodate grading near the terminal. This is typically 30 m [100 ft] in advance of the terminal. A curb similar to this detail could be used for all speeds when the barrier is required to be offset from the face of the rail or when a curb is required adjacent to a terminal.

[41]Roadside Design Guide, American Association of State Highway and Transportation Officials (AASHTO), 2011 Ballot Version, Chapter 5.

5.6.2.1.1 Curb/Guardrail Combinations for Strong-Post W-Beam Guardrail

A strong-post W-beam guardrail can be used with any combination of a sloping-faced curb that is 150 mm [6 in.] or shorter if installed flush with the face of the guardrail on roads with design speeds up to 80 km/h [50 mph]. For design speeds above 80 km/h [50 mph], a 100 mm [4 in.] or shorter sloping curb is recommended for installations where the face of the curb is flush with the face of the guardrail.

For strong-post W-beam guardrails not installed flush with the curb, or if the curb is not of the laydown design, specific curb/guardrail offset guidelines for various design speeds are presented in the following subsections. (17) Note that there are exceptions to these guidelines, such as if the guardrail system has been successfully been crash tested based on NCHRP Report 350 or MASH evaluation criteria with a curb.

Less than 70 km/h [45 mph]

For design speeds of 70 km/h [45 mph] or less, traditional strong-post W-beam guardrail should be installed either flush with the curb face or no closer than 2.5 m [8 ft] to the curb. The vehicle bumper may rise above the critical height of the guardrail in this region for many road departure angles and speeds, which increases the chance for vaulting. A lateral distance of 2.5 m [8 ft] is needed to allow the vehicle suspension to return to its normal pre-departure state. Once the suspension and bumper have returned to their normal position, then impacts with the barrier would not be adversely affected. Guardrails may be used with 150 mm [6 in.] high or shorter sloping-faced curbs as long as the face of the guardrail is located flush with or at least 2.5 m [8 ft] behind the face of the curb. Refer to Figure 5.35 for additional details regarding the use of laydown style curb when these guidelines are not practical.

70 to 80 km/h [45 to 50 mph]

A lateral offset distance of 4 m [13 ft] is needed to allow the vehicle suspension to return to its normal pre-departure state at these operating speeds. Once the suspension and bumper have returned to their normal position, then impacts with the barrier would not be adversely affected. For design speeds of 70 to 80 km/h [45 to 50 mph], guardrails may be used with 100 mm [4 in.] high or shorter sloping curbs as long as the face of the guardrail is flush with the face of the curb or located at least 4 m [13 ft] behind the curb. Refer to Figure 5.35 for additional details regarding the use of laydown style curb when these guidelines are not practical.

Greater than 80 km/h [50 mph]

For design speeds above 80 km/h [50 mph], guardrails should be used with 100 mm [4 in.] or shorter sloping-face curbs, and the face of the curbs should be flush with the face of the guardrail. Above operating speeds of 100 km/h [60 mph], the *sloping face should be 1V:3H or flatter and no taller than 100 mm [4 in.] high. Refer to Figure 5.35 for additional details regarding the use of laydown style curb when these guidelines are not practical.*

5.6.2.1.2 Crash Tested Curb/Guardrail Combinations

Curb/barrier combinations can be crash tested to quantify expected railing performance under typical impact conditions if extensive use of a specific combination is planned.

The Midwest Guardrail System (MGS), as described in Section 5.4.1.7, has been successfully crash tested to TL-3 when used in combination with a 150 mm [6 in.] AASHTO Type B curb. The face of the MGS barrier is located 150 mm [6 in] behind the face of the curb, as shown in Figure 5.36. The designer can also use the laydown curb as shown in Figure 5.35 in lieu of the 150 mm [6 in.] AASHTO Type B curb. In addition, a TL-2 MGS guardrail has been developed that is installed 1.8 m [6 ft] behind a 150 mm [6 in.] curb. Additional research and testing for the MGS at various test levels and offsets from the curb is being conducted.

Trinity Industries has also developed the $T\text{-}31^{TM}$ Guardrail system for use in conjunction with a curb. Refer to Section 5.4.1.8 as well as the manufacturer for additional information.

As described in Section 5.4.1.10, the Merritt Parkway Aesthetic Guardrail has been crash tested in combination with a 100 mm [4 in.] slope-faced curb. The crash tested offset of the railing was 300 mm [12 in.] behind the face of the curb.

Note that the area between the curb and barrier should be backfilled to the height of the curb."

5.4 AASHTO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS

The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals[42] indicated the following:

*"**SECTION 2: GENERAL FEATURES OF DESIGN***

2.5.1 – Clear Zone Distance

Structural supports should be located in conformance with the clear zone concept as contained in Chapter 3, "Roadside Topography and Drainage Features," of

[42]Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, American Association of State Highway and Transportation Officials (AASHTO), 2009, Fifth Edition, pages 2-6, 2-7, and 3-1.

the Roadside Design Guide, or other clear zone policy accepted by the FHWA. Where the practical limits of structure costs, type of structures, volume and design speed of through-traffic, and structure arrangement make conformance with the Roadside Design Guide impractical, the structural support should be provided with a breakaway device or protected by the use of a guardrail or other barrier.

C2.5.1

The clear zone, illustrated in Figure 2-1, is the roadside border area beyond the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a nonrecoverable slope, and/or a clear run-out area. The desired width is dependent on the traffic volumes and speeds and on the roadside geometry.

Suggested minimum clear zone distances are provided in the Roadside Design Guide and are dependent on average daily traffic, slope of roadside, and design vehicle speed. Additional discussions of clear zone distances and lateral placement of structural support may be found in the Manual on Uniform Traffic Control Devices and A Policy on Geometric Design of Highways and Streets.

2.5.3 – Guardrails and Other Barriers

The location of roadside sign and luminaire supports behind a guardrail should provide clearance between the back of the rail and the face of the support to ensure that the rail will deflect properly when struck by a vehicle. Continuity of the railing on rigid highway structures should not be interrupted by sign or luminaire supports.

The clearance between the edge of a sign panel, which could present a hazard if struck, and the back of a barrier should also take into consideration the deflection of the rail. The edge of a sign shall not extend inside the face of the railing.

C2.5.3

Guardrails, as illustrated in Figure 2-1, are provided to shield motorists from fixed objects and to protect fixed objects, such as overhead sign supports. The Roadside Design Guide provides guidelines for the provision of roadside barriers for fixed objects.

The clearance between the back of the barrier and the face of the support may vary, depending on type of barrier system used. The Roadside Design Guide may be used to determine the proper clearance.

SECTION 3: LOADS

3.1 – Scope

This Section specifies minimum requirements for loads and forces, the limits of their application, and load combinations that are used for the design or structural evaluation of supports for highway signs, luminaires, and traffic signals.

Where different mean recurrence intervals may be used in specifying the loads, the selection of the proper mean recurrence interval is the responsibility of the Owner.

C3.1

This Section includes specifications for the dead load, live load, ice load, and wind load."

5.5 NYS DOT HIGHWAY DESIGN MANUAL

The NYS Thruway Authority indicated they generally rely on Chapter 10 of the NYS DOT Highway Design Manual[43] as modified by Thruway Authority policies and practices for roadside design. The NYS DOT Highway Design Manual[44] indicated the following:

"10.2 NEW, RECONSTRUCTION, AND FREEWAY 2R/3R PROJECTS

The general roadside design policy for new, reconstruction, and freeway 2R/3R projects is to provide satisfactory clear zones, whenever it is practical to do so, and appropriately designed barriers, when it is not.

10.2.1 <u>Clear Zones</u>

NYSDOT defines the Clear Zone as that portion of the roadside border width, starting at the edge of the through traveled way, that the Department commits to maintaining in a cleared condition for safe use by errant vehicles. The width of the Clear Zone will be as last documented in the Design Approval Document, the Project Files, or in the contract documents. If warranted by special conditions, the Clear Zone may include occasional unshielded fixed objects, provided a reasonable rationale is documented.

[43]The NYS DOT Highway Design Manual (Chapter 10) refers to capital projects only and not maintenance repairs.
[44]New York State Department of Transportation Highway Design Manual, Chapter 10 – Roadside Design, Guide Rail, and Appurtenances, Revision 57, June 28, 2010.

10.2.2.4 Vaulting Considerations and Policy on Curbs and Curb/Barrier Combinations

Curbing has been shown to be a major contributor to vaulting and destabilization problems, particularly at high speeds and with higher curbs. When the tires of an errant vehicle strike a curb, the impact tends to bounce the vehicle upwards which can contribute to vaulting or penetration of the rail. The problem is generally worst for curbs located more than 1 ft and less than 10 ft in front of the guide rail. In addition to the vertical bounce, striking the curb tends to slow one side of the vehicle. Both of these effects contribute to destabilization. When the destabilizing or vertical bounce effects act in combination with either the destabilizing effects of striking a concrete barrier or the large deflection of cable guide rail, unsatisfactory results may occur. ***Therefore, do not place curbs of any height in front of concrete barrier (other than bridge barriers) or use (except in low-speed situations) in conjunction with cable barriers.***

A. Curbs and Curb/Barrier Combinations on High-Speed (50 mph or greater) Highways

- *Curbing of any height is not to be used in conjunction with concrete barriers, attenuating devices, or cable guide rail.*
- *Due to its destabilizing effects, vertical faced curbing (formerly referred to as nonmountable) is not to be installed on new construction projects on high-speed highways (operating speeds 50 mph or greater) and is to be removed when practical on reconstruction projects. Vertical faced curb is not to be placed or permitted to remain along the mainline or in gore areas of interstates, freeways, or high-speed parkways. Refer to the Bridge Detail sheets for exceptions at abutments. Any other necessary exceptions are to be explained in the design approval documents.*
- *Mountable curbing of any height is not to be installed on new or reconstruction projects, except that, when curbing is necessary for drainage control on high-speed roads, mountable curbs with a maximum height of 4 inches may be used at the outside edge of shoulder where the shoulder is of the minimum width specified in Chapter 2 of this manual. Preference should be given to using the T100 traversable curb profile rather than mountable curb.*
- *Curbing is not to be placed along high-speed highways for the purpose of shielding pedestrians. Curbing is ineffective as a barrier, and, at high speeds, vehicles that come into contact with curbing are at increased risk of being pulled out of the traveled way and into areas frequented by pedestrians.*
- *Because of the vaulting concerns mentioned above, when it is necessary to use guide rail adjacent to mountable curbs on high-speed highways, the preferred location is within one foot of the face of the curb. The second place choice would be ten or more feet behind the face of the curb.*

Placement in the zone between one and ten feet behind the face of curb shall be avoided unless the preferred locations are not reasonable options. Documentation of the latter choice should be provided if the unreasonableness of the other choices is not readily apparent.

- *AASHTO's A Policy on Design Standards – Interstate System, 1991, further stipulates that, where it is necessary to use mountable curb and guide rail together, the face of the curb should be flush with the face of the guide rail or behind it. Where the 4 inch high, 12 inch wide gutter/berm is used as a curb at the outside of the shoulder width, the guide rail post should be placed as close to its back face as possible. This requirement applies to freeways as well.*
- *Theoretical studies have indicated the potential for curbs located under flexible or semirigid guide rail to increase the chances of vaulting or rollover. Therefore, the allowable deflection of barriers used in conjunction with mountable curbs should not exceed 4 feet.*
- *Whenever a parkway project calls for any curb to be located closer to the travel lane than a standard-width shoulder (see Chapter 2 of this manual for design criteria), the 4 inch (100 mm) high, 12 inch wide T100 traversable curb or other approved traversable design is to be used. Examples of this would be curbed, raised, grass shoulders on parkways or curbed reduced shoulder sections approaching a bridge.*

B. Use of Curb and Curb/Barrier Combinations on Medium-Speed Highways (with Design Speed of less than 50 mph and greater than 40 mph)

- *Curbing of any height is not to be used in conjunction with either concrete barriers or cable guide rail.*
- *Curbing is not to be used in conjunction with attenuating devices.*
- *The designer should judge whether the project area conditions are typically rural, in which case the high-speed guidance presented as Section A, above, should be followed, or whether the conditions are predominantly urban or developing urban, in which case the guidance presented in this Section B should be followed.*
- *Mountable curbing may be used in conjunction with rail systems other than cable, but because of the vaulting concerns mentioned above, when it is necessary to use guide rail adjacent to mountable curbs, the placement preferences should be as noted in Section A above.*
- *The T100 traversable curb profile is acceptable for use with any guide rail at any offset.*
- *As general guidance, vertical-faced curbs (formerly referred to as nonmountable) may be used, but should only be used where justified by present or anticipated pedestrian traffic. Note that vertical-faced curb has little redirective or shielding capacity and is meant primarily to discourage the mingling of vehicular and pedestrian traffic. Because of destabilization problems, guide rails should preferably be no farther than 1 ft from the face of vertical faced curb. (Even though the effect is most*

pronounced between 1 ft and 10 ft, there is still a potential for vehicles to destabilize when striking a vertical faced curb and to vault a barrier even if it is located 10 ft or more from the curb.)

- *As mentioned above, theoretical studies have indicated the potential for curbs located under flexible or semirigid guide rail to increase the chances of vaulting or rollover. Therefore, on highways with design speeds less than 50 mph and greater than 40 mph, the allowable deflection of guide rails used in conjunction with mountable or vertical-faced curbs should not exceed 5 ft.*
- *Since vertical faced curb has little redirective capacity (for the low-speed range it may redirect low angle impacts), efforts should be made to address clear zone concerns behind curbs exposed to traffic rather than being satisfied with the 18 inch lateral clearance requirement discussed in Chapter 2 of this manual. The designer should try to maintain the quality of the clear zone by limiting the number of obstructions behind the curb and should try to maintain the quantity, or width, of the zone by locating any required obstructions as far from the curb as possible.*

Note: The AASHTO guidance on curbs originally recognized high-speed, medium-speed, and low-speed highways. The 2001 AASHTO A Policy on Geometric Design of Highways and Streets consolidated the medium-speed into the low-speed design category.

C. Use of Curb and Curb/Barrier Combinations on Low-Speed Highways with Design Speeds of 40 mph or less

- *Curbing of any height is not to be used in conjunction with concrete barriers.*
- *Curbing is not to be used in conjunction with attenuating devices.*
- *As general guidance, vertical-faced curbs may be used in low-speed situations (35 mph or less). Note that vertical-faced curb has little redirective or shielding capacity. When used in conjunction with guide rail, the rail should generally be placed within 1 ft of the face of the vertical faced curb. However, offset is not critical, as there is little risk of vaulting at these lower operating speeds. Where the rail is being placed for the protection of pedestrians, a system with an appropriately low deflection distance should be selected. See the Bridge Detail sheets for exceptions at abutments. See Chapter 18 of this manual for details of treatment for the back side of guide rails to reduce the potential hazard that posts represent when in close proximity to sidewalks or bicycle paths.*
- *Mountable curbing may generally be used in low-speed setting in conjunction with any type of guide rail.*

10.2.3 __Barrier Types__

There are four types of barrier in common use in New York: cable guide rail, corrugated metal or W-beam guide rail, box beam guide rail, and concrete barriers. They are discussed in the following subsections in order of increasing rigidity. W-beam may be mounted on either weak posts or heavy posts (see Section 10.2.3.5) and, in the latter case, is much more rigid.

The selection of an appropriate barrier is primarily governed by safety considerations and secondarily by cost. In general, the most flexible barriers will have the lowest lateral deceleration rates and will perform better at gradually redirecting an errant vehicle. Unfortunately, barriers with large deflections may not perform well adjacent to steep slopes. Additionally, when a flexible system is struck, it will usually require extensive repair work before it will function properly again. In areas with frequent accidents, this may result in a significant accumulation of time during which the barrier is not operational. Also, the regular presence of repair crews must be considered as a potential hazard, both for the motorist and for the workers themselves. In such circumstances, use of a heavy-post blocked-out corrugated barrier or a rigid concrete barrier may be warranted, as they seldom require repair work. Refer to Section 10.2.4.1 for further discussion.

The safety of a given barrier system will also vary depending on the type of vehicle involved. Most barrier systems presently in service have been crash tested with either a standard passenger car or a standard and a lightweight car. Recently installed systems were crash tested with a 4450 lb pickup truck and a small car. As a result, the barrier systems are well adapted to the protection of the most common vehicles, but may not be well adapted to larger vehicles such as vans and tractor trailers. The point should be stressed that the barrier systems that have been developed are a __**compromise**__ *intended to provide protection for occupants of the average, more common vehicles in a fleet with broad diversity. Preference should be given to improving clear zones where practical rather than simply installing barriers. However, it should also be pointed out that, with modern testing and improvements, barriers, and particularly terminals, are much less likely to contribute to unfavorable outcomes than they once were. While lateral decelerations on stout barrier systems can still be very harsh, the results of collisions with other fixed objects will almost always be more severe, especially if the effective clear area is at or less than the recommended clear zone width.*

Because of their size, buses and large trucks are not well protected by W-beam guide rails. Box beam is unlikely to rupture, but may get pushed down under large-tired vehicles. Cable stands the best chance of capturing a vehicle, but the extra vehicle weight may cause larger than normal deflections. If the cable is adjacent to an embankment, large vehicles may still reach the slope. With their higher centers of gravity, they will be more likely to roll over, even on relatively *mild slopes. Concrete barriers function best for large vehicles and higher barriers reduce the chance that the large vehicles will trip and flip over the barrier. The designer should review the distribution of vehicle types expected on a finished project as a factor in selecting appropriate barrier types. The Design Quality Assurance Bureau should be consulted for barrier selection and design guidance for areas where truck penetration is deemed unacceptable.*

In some situations, it may be desirable to evaluate the cost of providing a barrier system for comparison to other options such as buying right of way so slopes may be flattened. When evaluating the cost of a barrier system, the designer should consider (1) the initial cost of the system, (2) the cost of the types of repairs that may be required, (3) the frequency at which the various repairs will be required, and (4) the anticipated relative safety benefit. The first factor may be estimated from previous bid prices which are published in the Department's "Weighted Average Item Prices". The second factor should be available from maintenance records for that Region or, for new records, predictions may be obtained by the use of the computer program Roadside. The third factor may be estimated based on a combination of traffic projections, accident history data, and maintenance records. The fourth factor will generally be based on professional judgment and consideration of such concerns as frequency, type, and severity of accidents. In some situations, the potential for damage to adjoining property and road closures due to truck overturning should also be considered.

In general, the initial cost of weak-post W-beam will be about twice the cost of cable guide rail. Heavy-post blocked-out W-beam will be about three times the cost of cable guide rail. The cost of box beam will be about four times the cost of cable, and the cost of concrete may be as much as ten times the cost of cable. The maintenance costs may be significant for weaker systems and will be strongly controlled by traffic conditions.

10.2.3.2 W-Beam

B. Heavy-post Blocked-Out W-beam

To remedy the high repair incidence while still providing a yielding system, the heavy-post blocked-out W-beam guide rail was developed. The blockout piece holds the rail away from the post to reduce the chance that part of an impacting vehicle will extend under the rail and snag on the posts. The heavy posts are much stouter than the weak posts and snagging on them could cause a vehicle to turn and roll over. The typical details are shown on the Standard Sheets for 606 items. To limit deflections and the potential for pocketing and wheel snagging, the typical post spacing is only 6'-3". The main advantages of heavy-post blocked-out corrugated beam guide rail are that it has a low deflection distance and it can survive mild hits with minimal need for repairs. The main disadvantage of the system is that it produces more severe lateral deceleration of impacting cars than do the weak-post systems. A secondary disadvantage of the

HPBO system is its total width, which can be difficult to fit between the paved shoulder and a steep shoulder break. The heavy-post system may be warranted where barrier is needed and the traffic volume exceeds 50,000 vehicles/day. The decreased safety due to the high rigidity is offset by the increased safety obtained by limiting repair interruptions. In instances where a guide rail is needed but there is not enough clear area to accommodate cable, either heavy-post blocked-out W-beam or box beam are the logical alternatives to weak-post W-beam guide rail.

10.2.3.5 Post Systems

The "heavy post" is a W 6 x 9 (or W 6 x 8.5), which is approximately four times as rigid as the weak-post, and must, therefore, be considered as more of a potential hazard. To minimize the danger of vehicles snagging on the posts below the rail, the rail is blocked-out in front of the posts. The traditional metal block-out has been replaced with a solid block-out that provides 7.5 inches of separation between the rail and the post (versus the traditional 6 inches). The solid block-outs are to be made of either wood (Standard Specifications 710-20 and 710-13, issued by El 97-016) or plastic and synthetic (Standard Specification 710-26, issued by El 99-035). Steel block-outs should not be reset or used for repair of damaged HPBO guide rail. To maintain the usable shoulder widths, heavy steel posts should now typically be positioned 10 inches from the edge of usable shoulder.

10.3.1 Evaluation of Existing Facilities

The proper evaluation of an existing facility includes two primary activities. First, the relevant accident data should be reviewed for indications of features that are not performing well or locations where extra attention to roadside design may be appropriate. Second, a detailed site inspection should be performed to determine possible explanations for recorded accidents and to identify nonconforming features and roadside safety concerns and opportunities…

10.3.1.2 Site Inspection

Section 10.2.1 describes a key issue that should be understood before discussing roadside site inspections. The issue relates to the distinctions between concerns for safety and for liability. The primary concern is to address safety. With respect to roadside design, safety is addressed by the term "clear area", while the term "clear zone" relates to liability. Clear area is the portion of the roadside environment, starting at the edge of traveled way, from which hazards are essentially absent. The clear zone is the portion of the roadside environment, starting at the edge of traveled way, which the Department commits to maintaining in a cleared condition for safe use by errant vehicles. "Clear area" refers to a physical reality while "clear zone" refers to an obligation. The clear zone commitment may be conveniently defined as one or several uniform widths.

The width of the clear area cannot be conveniently defined as it varies continuously along the highway and will change over time. However, it is the actual clear area, not the invisible clear zone, which affects the safety of occupants of errant vehicles. Therefore, when evaluating the safety of a facility, the scope/designer should first examine the clear area. Only after the safety provided by the clear area has been addressed should attention be given to establishing limits of liability by documenting the clear zone width(s) for a project…

It is desirable for the project developer/designer to make a written record of the site inspection for inclusion in the project documents. The purpose of this effort is to further document that the Department was diligent in its efforts to provide a reasonably safe highway…

B. Identification of Roadside Safety Concerns and Nonconforming Features

Barrier-Related Nonconforming Features and Safety Concerns The designer should be cautious about specifying "Reset" or "Replace in kind" when dealing with barriers and attenuators. The adequacy of a barrier's type, placement, anchorage, etc., must be carefully reviewed. Attenuators of any kind should be reviewed to confirm that their design and placement are appropriate for the anticipated speeds. Any new or replacement barriers or attenuators shall be installed in conformance with current standards, including point of need, or an explanation provided in the design approval document. In general, the following and similar instances of outmoded guide rails shall be upgraded to current standards or the conditions warranting their use shall be eliminated, unless it is prudent and permitted to do otherwise for the type of project being progressed.

Fixed Objects and Roadside Obstacles. Fixed objects are defined as permanent installations, limited in length, which can be struck by vehicles running off of the road. Because of their limited extent, fixed objects should usually be removed from the clear zones, rather than being shielded with a barrier. During the site inspection, attention may be limited to those objects that are within the existing designed clear zone width, except in areas where it is reasonable to consider expanding the clear zone, in which case objects within the potential clear zone width should be noted. The following items are examples of fixed objects and roadside obstacles…

5. Nonbreakaway signs.
13. Presence of curbs over 4 inches high on roads with operating speeds of 50 mph or greater. (Also check Section 10.2.2.4 to see whether use of any curbs is appropriate.)

10.3.2 Detailed Scope of Work Determinations

If the project is to be a reconstruction project or an interstate or freeway 3R or 2R project, the clear zone width should be selected in conformance with the guidance in Section 10.2.1. Section 10.3.2.1 contains guidance for establishing the detailed roadside design scope, while the remediation of nonconformities and creation of roadside design safety features should be performed in accordance with the guidance for new and reconstructed facilities contained in Section 10.2…

10.3.2.1 Reconstruction Projects

It is intended that a reconstruction project bring an existing facility up to current standards. It is occasionally (frequently, in developed areas) not reasonable to do so in all areas of the project because of environmental, economic, or other considerations.

10.3.2.2 3R Projects (Resurfacing, Restoration, and Rehabilitation)

(Note: In regard to roadside design, Interstate and other freeway 2R and 3R projects should be treated the same way as reconstruction projects. For further details, refer to Sections 10.2 and 7.2 of this manual.)

The major difference between roadside design for a reconstruction project and for a 2R or 3R project is the amount of effort that should be applied towards obtaining the desired clear zone width. On a reconstruction project, the effort to achieve the desired width should extend to what can be reasonably attained when considering factors such as cost, environmental impacts, timeliness, project scope, etc. On a 3R project, unless there is an accident history related to the roadside or the clear zone width is otherwise judged inadequate, any increase in the existing width may be limited to that which may be conveniently attained. While the widths that are developed for either project may vary, in both cases the quality of the zone (traversability, absence of fixed objects) should be similar…

A. Clear Zone Width Determinations for Nonfreeway 3R and 2R Projects

Wherever reconstruction or realignment work is included within a 3R project, that portion of the project should follow clear zone requirements for a reconstruction project.

B1. "Basic Safety Package" for Roadside Work on Nonfreeway 2R and 3R Segments with Design Speeds over 40 mph (60 km/h)

1. *Clear Zone –*
   a. *Fixed objects should be removed from the clear zone widths or suitably modified.*

b. *Where a reasonable clear zone width cannot be obtained, appropriate guide rail or other barriers are to be installed or an explanation provided.*

5. *Guide Rail -*
   a. *All guide rail is to be of a type currently approved to be in service, meet the required standard details prevailing at the time of its installation, set to currently approved heights, and acceptably anchored and flared as necessary."*

5.6 NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM (NCHRP) REPORT 638 – GUIDELINES FOR GUARDRAIL IMPLEMENTATION

The National Cooperative Highway Research Program (NCHRP) Report 638 – Guidelines for Guardrail Implementation[45] indicated the following:

*"**Chapter 1 – Introduction***

The goal of roadside safety devices is to protect motorists from potentially serious hazards located near the travelway. Bridge piers, utility poles, and severe embankments are hazards that, if encountered, may be deadly. In order to protect motorists, barriers must be placed in front of a roadside obstacle and must be much longer than the hazard in order to limit the risk of a serious crash when vehicles leave the road in advance of the barrier…

…Guardrail warrant recommendations contained in the AASHTO Roadside Design Guide (RDG) are based on relative severity indices, which were determined by making a subjective evaluation of the relative severities of striking a roadside obstacle or a barrier. If the consequences of a vehicle striking a fixed object are estimated to be more serious than hitting a traffic barrier, then the barrier is recommended. The current guidelines are presented in the form of a table that offers guidance to designers. Unfortunately, the table incorporates a number of imprecise terms (i.e., "judgment decision," "generally required," and "may be warranted").

Existing guidelines for guardrail application allow for a great deal of inconsistency. Two virtually identical sites can be treated much differently, depending upon the discretion of the individual designers. Objective criteria are needed to help reduce or eliminate inconsistencies and provide optimal safety for all motorists and minimize the number of serious crashes along the roadways.

Further, the RDG does not provide objective guidance that designers could use to determine what barrier performance level should be implemented. Instead, the RDG merely suggests using higher performance-level barriers when an above

[45]National Cooperative Highway Research Program (NCHRP) Report 638 – Guidelines for Guardrail Implementation, Transportation Research Board, Dean L. Sicking, Karla A. Lechtenberg, Scott Peterson, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, 2009, pages 1 through 28.

average percentage of heavy truck traffic or adverse geometrics with poor sight distance are present. This very general guidance does not provide and specifics regarding at what truck volumes higher performance-level barriers become warranted, nor does it specifically address what the term adverse geometrics should include. Clearly, this type of general guidance directly considers neither the crash frequencies nor the costs associated with the use of higher performance guardrails.

In recognition of the need for better guidance for selecting the appropriate guardrail performance level, NCHRP funded the study described herein.

Chapter 4 – Roadside Safety Analysis Program (RSAP)

…In order to develop comprehensive guidelines, it was necessary to study a full range of hazard sizes and severity. Two hazard size classifications and three different hazard severities were selected for inclusion in the study. Point hazards were chosen to represent situations of least cost beneficial guardrail applications while long roadside slopes were selected to represent situations where guardrail is most likely to be cost beneficial. It should be noted that long hazards are 4,000 ft (1,219.2 m) long. Specific hazards selected for each of the six categories are shown in Table 4.

Table 4. Hazard categories

Category	***Severe***	***Moderately Severe***	***Moderate***
Point Hazard	*3-ft diameter bridge pier*	*10-in. diameter utility pole*	*6-in. diameter tree*
Slope Hazard	*1.5:1 slope, 26 ft deep*	*2:1 slope, 20 ft deep*	*2.5:1 slope, 13 ft deep*

Chapter 5 – Benefit/Cost Analysis[46]

…This analysis produced some troubling findings. Guardrail treatment of even the most severe point hazard was found never to be cost beneficial…

…TL-5 barriers were found to be the most cost beneficial option for long, severe, and moderately severe hazards adjacent to high-volume freeways…

[46]The purpose of a benefit/cost analysis is to provide an economic assessment of the extent to which a project or program may achieve its ultimate goal of reducing the number and/or severity of crashes. The benefit/cost analysis ultimately provides a means of selecting the most cost-effective countermeasure for any given project. The benefit/cost ratio is computed by dividing the annual benefit by the annual cost. The countermeasure with the highest benefit/cost ratio is normally the recommended alternative.

Chapter 6 – Route-Specific Selection Guidelines

Hazard Size

Guardrail shielding of long hazards was found to be much more cost beneficial than treatment of point hazards. When viewed in terms of the benefits associated with a higher barrier test level, this finding is not surprising. As noted above, the benefit of increasing test level is primarily related to the risk of a vehicle striking a roadside hazard after penetrating through or over the barrier. When a vehicle penetrates through or over the portion of any guardrail placed upstream of an object, the risk of the vehicle continuing on to strike the hazard is still relatively modest. When a vehicle penetrates through a barrier immediately adjacent to an obstacle, however, it will almost certainly encounter the hazard. Because of the significantly different risks of a vehicle penetrating through or over the barrier and then striking the hazard, higher test level barriers are shown to be much more cost beneficial when placed adjacent to long hazards.

Curvature

…When the effects of curvature on guardrail protection of long hazards are studied, a barrier is found to be only modestly more cost beneficial when the hazard is placed on the outside of a left curve. When the effects of curvature on guardrail benefits are examined for point hazards, just the opposite is found. A barrier is found to be less cost beneficial when protecting motorists from point hazards placed outside of a curve. This effect is related to the risk of impacting a point hazard when a vehicle encroaches from a curved highway."

NCHRP Report 638 would recommend a TL-3 barrier for benefit/cost ratios equal to 1, 2, 3, and 4 under conditions that represent the following:

- A severe point hazard offset 18 feet from the edge of traveled way on freeways,
- On a curvature that is 2 degrees to the left,
- On a grade varying from 0 to -2 percent, and
- Traffic volumes of 100,000 vehicles per day.

5.7 TESTS AND EVALUATION OF W-BEAM AND THRIE-BEAM GUARDRAILS

The Texas Transportation Institute[47] conducted tests and evaluation of W-beam[48] and Thrie-beam[49] guardrails for the Office of Research, Federal Highway Administration in March 1982. The report entitled Test and Evaluation W-Beam and Thrie-Beam Guardrails[50] indicated the following:

> *"This report describes work which was aimed at investigating the feasibility of enlarging the spectrum of vehicles considered in the guardrail design process. Up until now, guardrails have been designed to accommodate a 2041 kg (4500 lb) automobile at 96.5 km/h (60 mph) and 25 deg as the most severe strength test. The goal of this study was to determine if a relatively conventional guardrail design is suitable to safely redirect a 9072 kg (20,000 lb) school bus moving at 96.5 km/h (60 mph) with an impact angle of 15 deg and, if this is not the case, to see if reasonably economical guardrails could be designed to accomplish this task."*

Three tests were performed on the W-beam and Thrie-beam guardrails. The configuration of the W-beam and Thrie-beam guardrails consisted of the following:

Test No. 1 – Conventional Thrie-beam guardrail with steel block-out. The height of the Thrie-beam rail element was approximately 20 inches. The block-outs consisted of steel W6 x 8.5 beams. The posts consisted of steel W6 x 8.5 beams. The total height from the ground surface to the top of the posts was approximately 33.25 inches.

Test No. 2 – Conventional W-beam guardrail with steel block-out. The height of the W-beam rail element was approximately 12 inches. The block-outs consisted of steel W6 x 8.5 beams. The posts consisted of steel W6 x 8.5 beams. The total height from the ground surface to the top of the posts was approximately 28 inches.

Test No. 3 – Modified Thrie-beam guardrail with steel block-out and a triangular segment cut-out. The height of the Thrie-beam rail element was approximately 20 inches. The block-outs consisted of steel M14 x 17.2 beams with a

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[47]Texas A&M Research Foundation, Texas Transportation Institute, The Texas A&M University System.

[48]A W-beam guardrail is a steel beam rail element that is shaped in the form of a "W". The height of the W-beam rail element used in the Texas Transportation Institute tests was approximately 12 inches.

[49]A Thrie-beam guardrail is a steel beam rail element that is shaped in the form of a "W" but includes an additional undulation in the rail element. The height of the Thrie-beam rail element used in the Texas Transportation Institute tests was approximately 20 inches.

[50]Test and Evaluation W-Beam and Thrie-Beam Guardrails, Don L. Ivey, Richard Robertson, and C.E. Buth with contributions by Charles F. McDevitt, Contract Manager, Prepared for the Office of Research, Federal Highway Administration, U.S. Department of Transportation, Texas A&M Research Foundation, Texas Transportation Institute, The Texas A&M University System, March 1982, pages 1 through 27.

triangular segment cut-out. The posts consisted of steel W6 x 8.5 beams. The total height from the ground surface to the top of the posts was approximately 35.25 inches.

"In the first test, conducted on the thrie-beam guardrail shown in Figure 1, the 9081 kg (20,020 lb) bus at 89.5 km/h (55.6 mph) and 15 deg was contained and redirected with the bus going through a slow, smooth 90 deg counterclockwise roll before falling onto its left side and sliding to a stop. Although the 90 deg roll was not an ideal reaction, it was a slow, fairly smooth roll which would not be extremely hazardous to passengers if the integrity of the left side windows were maintained. The test results were therefore considered a marginal success (Figure 4, 5 and 6). The guardrail exhibited enough strength and maintained continuity so that the bus was contained and redirected. Accelerations on the bus during the event were low, while permanent deflection of the rail was about .53 m (21 in.)…

…In the second test, conducted on the W-beam guardrail shown in Figure 2, the bus was not contained. At a speed slightly higher than in the first test, 96.0 km/h (59.6 mph) compared to 89.5 km/h (55.6 mph), the bus started to redirect as the left front corner made contact. However, as it rolled left and yawed to the right, the rear of the bus went over the barrier, penetrating into the zone behind the rail. At one point the bus was sliding upside down across the guardrail, resulting in a shredding of the bus top (See Figures 7, 8 and 9). This reaction was obviously unacceptable because it would have resulted in many severe passenger injuries and fatalities.

With the experience gained from the first two tests it was apparent that significant design changes would have to be made if a guardrail was to safely contain and redirect a bus in a 96.0 km/h (60.0 mph) 15 deg collision. The thrie-beam guardrail used in test 1 had proven strong enough, but had exerted its resisting force at a point too low to prevent the bus from rolling. It was considered the prime candidate for redesign. The emphasis would be to make design changes that would elevate the point of resistance during a collision…

…The removal of the triangular segment of the web is the critical factor in keeping the thrie-beam face vertical and in elevating the point of resistance of the rail.

The guardrail shown in Figure 3 is the result of those efforts. The following design changes were made in arriving at this design:

3. *…A triangular shaped segment was cut from the web of the M14 x 17.2 spacer as shown in Figure 3. This notch allows the lower portion of the thrie-beam and the adjacent spacer block flange to bend in during a collision. This tends to keep the rail face vertical in the impact zone. It also reduces the contact forces between an impacting vehicle and the*

lower part of the thrie-beam, requiring the resisting loads to move up onto the fully supported part of the rail. The net effect is that the resultant resisting force of the rail is raised to a higher position which produces a smaller roll moment on the vehicle…

…The third test with a school bus at 89.8 km/h (55.8 mph) and 15 deg produced a bus reaction that was quite reasonable. The bus was contained and smoothly redirected, remaining upright throughout the event. There was approximately 25 deg of bus roll to the left, or counterclockwise, when viewed from the rear, during contact with the modified thrie-beam guardrail. Overall it was interpreted as a stable collision (Figures 10 and 11)…

…The next question to be addressed was whether the modified thrie-beam could redirect a 14,515 kg (32,000 lb) intercity bus at 60 mph and 15 deg. This question was addressed with several analytical approached and finally with a full-scale crash test…

…The intercity bus test, was conducted on Nov. 16, 1981. The results were excellent as evident in Figures 19, 20 and 21. The impact angle was 14.0 deg. The electronically determined speed just prior to impact was 95.9 km/h (59.6 mph). Vehicle stability was good, with a maximum counterclockwise roll angle of approximately 15 deg (i.e., roll into the barrier). The dynamic deflection was approximately 1.4 m (4.5 ft). Eight posts were deformed by the left front wheel but the rail remained intact and at a level suitable for redirection. The peak 0.050 sec average lateral acceleration was 2.5 g's. The corresponding longitudinal acceleration was only 0.8 g's, showing the very slight forces exerted by the deforming posts on the left front wheel. Damage to the bus was modest, with light sheet metal damage occurring at the left front and left rear corners. The bus was driveable after the test. Overall reaction of the bus to the forces imposed during the collision could be termed ideal."

D. ATTACHMENTS

Attachment 1 – New York State Department of Motor Vehicles Police Accident Report dated November 23, 2006

Attachment 2 – New York State Department of Motor Vehicles Police Accident Report dated December 14, 2010

Attachment 3 - NYS Thruway Authority Maintenance Directive (MD) 2002-4, Guidelines for the Repair and Upgrade of Guide Rail and Bridge Rail

Attachment 4 - NYS Thruway Authority Regulatory Speed Limits

Attachment 5 - NYS Thruway Authority Horizontal and Vertical Plans for I-95, Construction Contract TANE 84-25

Attachment 6 - NYS DOT Mountable Concrete Curb (Type AB)

Attachment 7 – Design Report for Construction Contract TANE 84-25

Attachment 8 - NYS Thruway Authority Plans for strong post blocked-out W-beam guiderail as part of Construction Contract TANE 84-25

Attachment 9 - NYS Thruway Authority Plans for strong post blocked-out W-beam guiderail as part of Construction Contract TANE 98-70

Attachment 10 - FHWA Memorandum to Division Administrators and Federal Lands Highway Division Engineers dated February 14, 2000

Attachment 11 - NYS Thruway Authority Signage and Structure Plans for I-95, Construction Contract TANE 84-25

Attachment 12 - NYS Thruway Authority Plans and Specifications for Shoulder Treatment for Accident Reduction (STAR) for I-95, Construction Contract TANE 06-21

Attachment 13 - NYS Thruway Authority Highway Lighting Plans for I-95, Construction Contract TANE 84-25

E. PHOTOGRAPHS

HWY Photo-01 - Illustrating the three southbound travel lanes of I-95

HWY Photo-02 - Illustrating the 1,600 foot radius and 4,500 foot radius horizontal curves

HWY Photo-03 - Illustrating the overhead sign structure support above the southbound travel lanes of I-95

HWY Photo-04 - Illustrating the height of the east vertical pole was approximately 28 feet and the height of the west vertical pole was approximately 27 feet

HWY Photo-05 - Illustrating the grooved rumble strips on the paved right shoulder of the southbound lanes of I-95 looking to the north

HWY Photo-06 - Illustrating the initial point of contact with the guiderail

HWY Photo-07 - Illustrating Sign #1 located approximately 453 feet from the final rest of the bus

HWY Photo-08 - Illustrating Sign #2 located approximately 217 feet from the final rest of the bus

HWY Photo-09 - Illustrating Sign #3 located approximately 130 feet from the final rest of the bus and the damage and removal of the left post

HWY Photo-10 - Illustrating the angle of approach in which the bus departed the travel lanes and initially collided with the guiderail

HWY Photo-11 - Illustrating the damage to lower left hand corner of Sign #1

HWY Photo-12 - Illustrating the damage to lower left hand corner of Sign #2

HWY Photo-13 - Illustrating the rail element completely separated from the posts and the posts deformed and turned down 45 degrees and 90 degrees

HWY Photo-14 - Illustrating the two E-Z Pass transponder devices mounted to the overhead sign structure support above the travel lanes

HWY Photo-15 and 16 - Illustrating the location of the cabinet mounted to the cross bracing before the accident

HWY Photo-17 - Illustrating the final rest of the cabinet after the accident

HWY Photo-18 - Illustrating the two wire conduits dislodged from the cross bracing to feed a VMS sign located above the northbound lanes of I-95

HWY Photo-19 - Illustrating the punched hole located at the base of the west vertical pole

HWY Photo-20 - Illustrating the multiple scrape marks on the east vertical pole measured from the base of the pole to a height of 9 feet

HWY Photo-21 - Illustrating the repairs made by the NYS Thruway Authority between March 15 and March 16, 2011 to install new strong post W-beam guiderail with steel block-outs

HWY Photo-22 - Illustrating the repairs made by the NYS Thruway Authority on April 29, 2011 to replace the steel block-outs with plastic block-outs

Dan Walsh /S/

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