Bridge Deck Designs for Railing Impacts

ALTHEA ARNOLD

ABSTRACT

Current specifications by the AASHTO Standard Specifications for Highway Bridges set forth certain structural design requirements for bridge railings and the corresponding bridge decks. Observations of deck failure patterns in recent crash tests and observed deck failures at vehicle collision sites have raised questions concerning current specifications. The Texas Transportation Institute, in conjunction with the Texas State Department of Highways and Public Transportation, has been studying the problem on three types of bridge rail used in Texas. Full-scale deck sections with a post, a parapet, or an 8-ft section of railing were built and tested. The three types tested were the Texas T101, T202, and T5 bridge railings. The findings of this testing are reported herein.

Current specifications by the AASHTO Standard Specifications for Highway Bridges (1) set forth certain structural design requirements for bridge railings and the corresponding bridge decks. AASHTO specifies design loads on bridge railings according to estimates of forces imposed on railings by traffic under normal operations and on railings during collisions by automobiles. It also specifies the manner in which these collision forces are to be transferred and distributed to the concrete deck. However, questions concerning these bridge deck specifications have arisen because of observed deck failure patterns in recent crash tests (2,3) and observed deck failures at actual vehicle collision sites.

It has been observed that when vehicles collide with a metal or concrete traffic railing, the traffic railing usually contains them but extensive damage may occur to the concrete bridge deck. Repair of a bridge deck is costly, time consuming, and dangerous. To repair a bridge deck, portions of the highway must be blocked off from traffic for several days while the damaged deck is removed, damaged steel is replaced, forms are built, and concrete is placed and allowed to cure. During this time traffic may become congested because of lane restrictions. This situation is hazardous to traffic as well as to construction workers.

The Texas Transportation Institute, in conjunction with the Texas State Department of Highways and Public Transportation, has been studying the problem on three types of bridges used in Texas. Full-scale deck sections with one post, parapet, or 8-ft rail section each were built and tested. Twenty-six tests, both static and dynamic, have been performed on three Texas standard bridge rails and on design variations of these bridge rails. The three types tested were the Texas T101, T202, and T5 bridge rails.

TEXAS BRIDGE RAILING TYPE T101

Figure 1 is a composite drawing of the cross section of the T101 post and deck test setup. The standard steel configuration is denoted by solid lines and the modifications by the broken lines. The T101 rail (not shown) is composed of two 4 x 3-in. structural steel tubes, faced with a corrugated sheet steel beam [AASHTO M180 (4)]. The strong structural steel posts (W6x20) are bolted to a 7.5-in. deck by means of four 3/4-in.-diameter A325 bolts. One-inch formed holes in the slab allow the post to be bolted through the slab to a bottom 8 x 0.25 x 9-in. plate with 7/8-in.-diameter bolt holes. The galvanized T101 post is constructed of a 26.125-in.-long W6x20 (A36 steel) wide flange welded to a 10 x 9 x 0.875-in. base plate with 1 x 1.5-in. slotted bolt holes. In the bottom of the deck, the dashed lines running the width of the deck represent the No. 4 transverse reinforcing steel at 10.5-in. centers. The longitudinal steel in the bottom consists of No. 5 bars. The deck reinforcing steel is grade 40.

In the top of the deck the transverse steel consists of No. 5 bars at 5.25-in. centers and the longitudinal steel consists of No. 4 bars. Also in the top is the bolt anchor plate represented by a solid line. The anchor plate is made of 2.5 x 0.25 x 39-in. A36 steel; it distributes the load in the bolts to a greater area of concrete. Figure 2 shows the three types of anchor plates used.

Photographs of typical crack patterns are shown in Figure 3. A summary of the tests performed on the T101 post and concrete bridge deck is given in Table 1. The data in Table 2 explain the abbreviations used in Tables 1, 3, and 4.

Tests on T101 Steel Post with a Rigid Support

Two static tests (T101-1PO and T101-2PO) were performed on the T101 steel post connected to a rigid foundation. These tests were performed to determine the strength of the post assembly. The T101 steel post is composed of a 26.125-in. section of a W6x20, A36 structural steel beam with a 10 x 0.875 x 9-in. steel plate welded to the bottom. The steel base plate has four 1 x 1.5-in. slotted holes to accommodate the 3/4-in.-diameter bolts. The bolts, nuts, and washers on the first test had sufficient strength to force the failure to occur in the post. The peak load was 43.7 kips at 3.6-in. deflection. The compression flange buckled and yielding occurred in the tension flange and web.

In the second test standard 3/4-in.-diameter A325 bolts, nuts, and washers were used to determine the strength of the system. The major failure mode was in the washers, which pulled through the 1 x 1.5-in. slotted holes in the post base plate. Buckling also occurred in the compression flange and some yielding occurred in the tension flange and in the web of the post. The peak load was 23.7 kips at 0.8-in. deflection. These tests demonstrate that the bolts and washers are the controlling factor in the strength of the post system.

Tests on Standard T101 Bridge Railing

Two static tests (T101-1S and T101-2S) were performed on the standard T101 post with a 7.5-in. deck to determine the strength of the existing system. The peak loads were 18.6 and 19.0 kips occurring at 1.6-in. lateral deflection. The post punched through

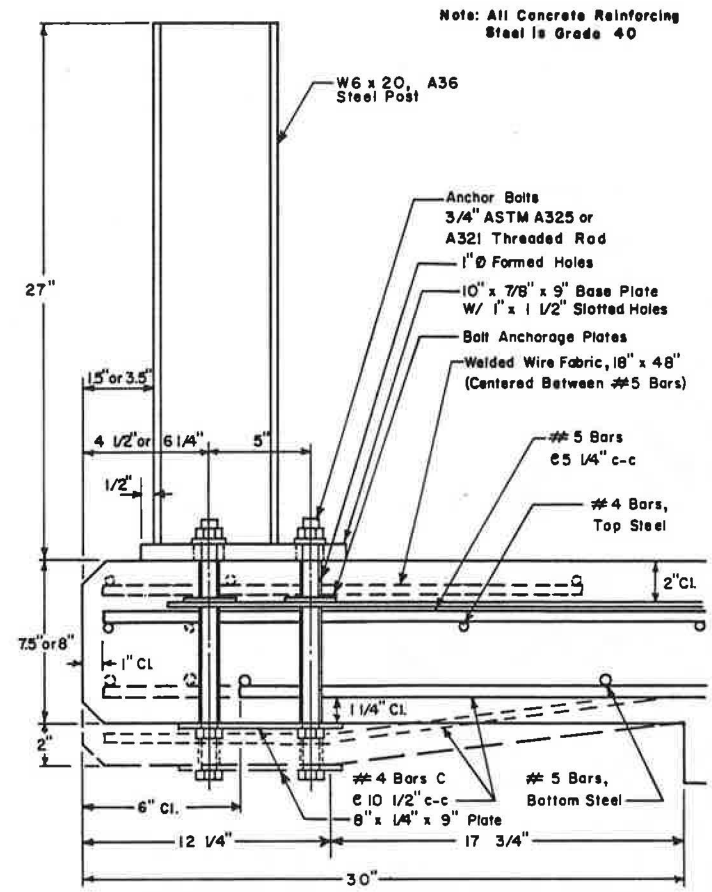


FIGURE 1 Composite drawing of modifications to Texas T101 bridge railing.

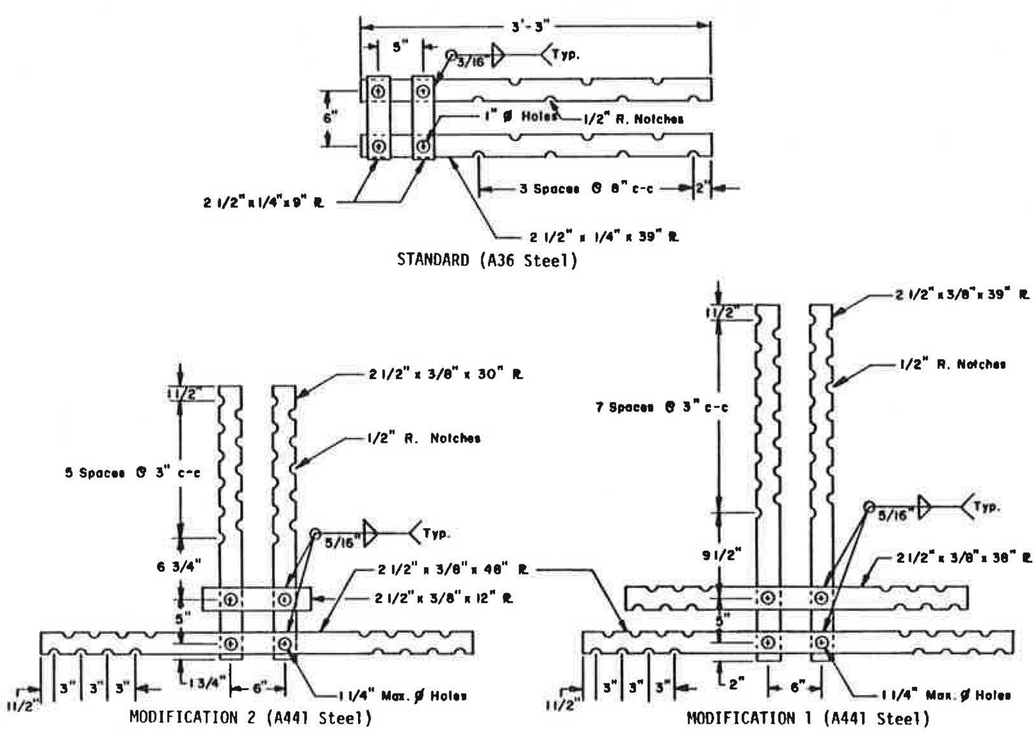


FIGURE 2 Types of anchor plates used in Texas T101 bridge deck.

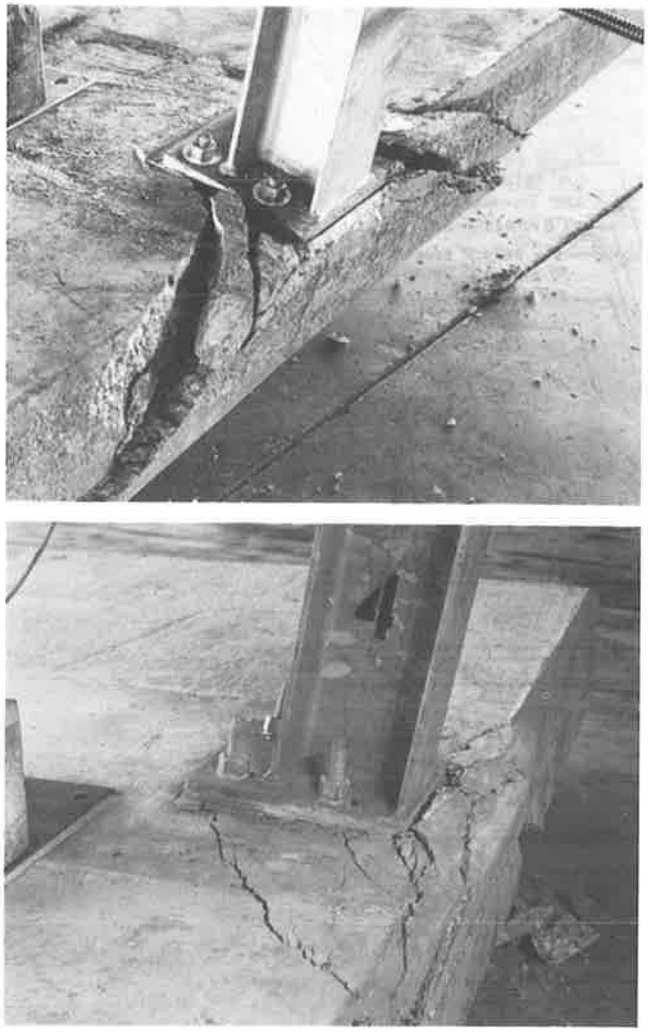


FIGURE 3 Typical crack patterns for Texas T101 post with concrete bridge deck.

the deck, with major cracks originating from the bolt holes and progressing to the edge of the deck. This behavior was observed from the general crack patterns. The concrete under the post had been pulverized because of high bearing loads. In all cases the bolt anchor plate was bent or broken because of compression forces under the post base plate. The No. 5 top bar that was located under the post between the anchor bolts broke at a 6-in. development length. The compression of the concrete covering this bar allowed for the short development length. This occurred in all tests on the standard design. No anchor bolt failure occurred in these tests.

Two dynamic tests (T101-1D and T101-2D) were performed on the standard T101 post with a 7.5-in. deck to determine the dynamic strength and the energy-absorbing capabilities. The peak loads were 57.9 kips at 2.3-in. and 36.2 kips at 2.1-in. deflection. The difference in these two values can be attributed to the type of pendulum nose used in each test. The energy absorbed was 17,475 and 28,605 kip-ft. The cracking patterns were identical with the static tests.

From the pendulum test it was evident that the outermost longitudinal bar supported the post. Close inspection of the post-deck connection showed the punching effect of the post, the fracture of the reinforcing steel, and the fracture of the bolt anchor plate inside the deck.

These tests indicated that methods for strengthening the slab needed to be investigated. These methods are discussed in ensuing paragraphs.

Tests on Modified Designs for T101 Bridge Railing

It was hypothesized that the punching effect of the post through the slab was caused by high stress concentrations under the post base plate. To spread out these forces in the slab, it was suggested that more tension and longitudinal steel be used in the top and bottom of the slab. To do this a 48 x 18-in. welded wire fabric mat made of D20 bars was placed on top of the existing steel. The existing top and bottom steel was extended to within 1 in. of the edge of the deck. The longitudinal steel was increased to include two No. 4 bars on the top and two

TABLE 1 Summary of Test Results on T101 Bridge Railing

TEST NO. AND TYPE	DECK DEPTH (in.)	EDGE DIST. (in.)	TDLR	BDLR	B (in.)	BOLT AP	SPECIAL REINF.	PEAK LOAD (kip)	DISPL (in.)	Remarks
T101-1D	7.5	1.75	1, No. 4	1, No. 5	6	Std.	–	57.9	2.3	2,115-lb pendulum at 20 mph, plywood nose; severe deck cracking and spalling
T101-2D	7.5	1.75	1, No. 4	1, No. 5	6	Std.	–	36.2	2.1	2,293-lb pendulum at 20 mph, rubber nose; severe deck cracking and spalling
T101-1S	7.5	1.75	1, No. 4	1, No. 5	6	Std.	–	18.6	1.6	Severe deck cracking and spalling of concrete; load falls off to 9.5 kips at 7.5-in. displacement
T101-2S	7.5	1.75	1, No. 4	1, No.5	6	Std.	–	19.0	1.6	Severe deck cracking and spalling of concrete; load falls off to 5 kips at 8-in. displacement
T101-3S	7.5	1.75	2, No. 4	2, No. 5	1	Std.	Welded wire fabric, 48 x 18 in.	24.0	2.0	7/8-in. bolts used; severe deck cracking but taut; final load 25 kips at 8 in.
T101-4S	10	1.75	2, No. 4	2, No. 5	1	Std.	Welded wire fabric, 48 x 18 in.	27.0	3.3	7/8-in. bolts used; severe deck cracking but taut; final load 23.5 kips at 8 in.
T101-5S	8	1.75	1, No. 4	2, No. 5	1	Mod. 1	–	21.4	2.0	Anchor bolts broke at 20.5 kips and 7 in. displacement; severe deck cracking
T101-6S	10	1.75	1, No. 4	2, No. 5	1	Mod. 1	–	21.2	4.7	Anchor bolts broke at 21.2 kips and 4.9-in. displacement; moderate cracking
T101-7S	8	3.5	2, No. 4	2, No. 5	1	Mod. 2	–	22.0	2.3	Anchor bolts broke at 22.0 kips and 2.3-in. displacement; moderate cracking
T101-8S	10	3.5	2, No. 4	2, No. 5	1	Mod. 2	–	25.4	2.1	Test terminated at 21.0 kips and 6-in. displacement; moderate cracking
T101-1PO[a]								43.7	3.6	Post failure; flange buckled
T101-2PO[b]								23.7	0.8	Nut and washer pulled through hole in base plate at 2.8 in.

Note: See Table 2 for key to abbreviations.
[a]Oversized bolts and washers used.
[b]Standard 3/4-in. A325 anchor bolts used.

TABLE 2 Key to Abbreviations in Tables 1, 3, and 4

Abbreviation	Explanation
TEST NO. AND TYPE	The test number is given; the type of loading is indicated by letters, where D = dynamic, S = static, and PO = post only
DECK DEPTH	Depth of the deck at the edge
EDGE DIST.	Distance between the deck edge and the post or wall
TDLR	Top deck longitudinal reinforcing located at the edge of the deck
BDLR	Bottom deck longitudinal reinforcing located at the deck edge
B	Distance from end of bottom transverse reinforcing steel to deck edge
BOLT AP	Type of bolt anchor plate
REINF. PATTERN	Four types of reinforcing were used on the T202: Std. = standard reinforcing pattern currently used; Std. w/8-in. Post Lap Spl. = standard with an 8-in. wall lap splice; Mod. 1 = in the first modification the top steel bends into the traffic side with a 12-in. lap splice in the post; and Mod. 2 = in the second modification the top steel bends into the traffic side with a 17-in. lap splice in the post
REINF. PATTERN	Two types of reinforcing were used on the T5: Std. = the standard reinforcing pattern currently used, and Mod. 1 = the tension leg of the hair-pin bar was moved to 10.75 in. from the deck edge and the compression bar was deleted
SPECIAL REINF.	Two sizes of welded wire fabric and No. 4 bar stirrups were used as special reinforcing
PEAK LOAD	The ultimate (failure) load in kips that the system withstood; one post or an 8-ft section was tested
DISPL	The lateral displacment of the post at the loading height at the peak load
Std.	Standard
Mod.	Modification

No. 5 bars on the bottom between the outside bolts and the edge on the deck. This steel configuration was tested in a 7.5-in. deck (T101-3S) and a 10-in. deck (T101-4S). An immediate increase in the deck strength was observed. The 7.5-in. deck reached a peak load of 24.0 kips at 2.0-in. lateral deflection. The 10-in. deck reached a peak load of 27.0 kips at 3.3-in. lateral deflection. The bolts used in this test were 7/8 in. in diameter in order to develop the strength of the slab before developing that of the bolts to determine the net increase in slab strength. Crack patterns in the deck were similar to the previous test. However, no broken steel was found.

Because it was determined that the slab strength could be increased to more than the strength of the bolts, more economical designs were sought. The anchor plate was enlarged (modification 1) to replace the welded wire fabric. The longitudinal top steel was reduced to one bar between the outside bolt and the deck edge. All other steel modifications remained the same as in the previous two tests. The deck depths tested were 8 in. (T101-5S) and 10 in. (T101-6S). The standard 3/4-in.-diameter A325 bolts, nuts, and washers were used. For the 8-in. deck the peak load was 21.4 kips at 2.0-in. deflection and for the 10-in. deck the peak load was 21.2 kips at 4.7-in. deflection. The toughness of the slabs was increased as seen in the load-deflection curves. The load on the 8-in. deck did not drop off until the bolt broke at 20.5 kips and 7-in. deflection. The load on the 10-in. deck was steadily increasing until the bolt broke. In both tests the post base plate was bent and some yielding of the post was evident. Cracking of the concrete in the 8-in. deck was similar to that in previous tests. However, the longitudinal steel was not bent as much as in the previous tests. In the 10-in. deck major cracks occurred only through the field-side bolt holes.

Post edge distance was considered a problem. The last two tests in this series were performed on an 8-in. deck (T101-7S) and a 10-in. deck (T101-8S) with the post edge distance increased from 1.75 to 3.5 in. The modified anchor plate was reduced (modification 2) to provide a more economical design, and the top longitudinal steel was two No. 4 bars between the post and the deck edge. The load on the 8-in. deck steadily increased to a peak load of 22.0 kips at 2.3-in. deflection, at which time an anchor bolt broke. This test shows an improvement in crack control at the same failure loads.

The 10-in. deck reached a peak load of 25.4 kips at 2.1-in. deflection. At a load of 21.0 kips and 6-in. deflection, one bolt and washer had pulled through the base plate hole and the test was terminated. The phenomenon that occurred here was that the bolts pulled through the base plate, moving the neutral axis toward the back of the post, causing all bolts to go into tension, thus a greater load. This bolt pull-out could have been prevented by using stronger washers. Cracking of concrete was confined to the field-side bolt holes and to the edge of the deck.

Conclusions

The strength of the W6x20 post was 43.7 kips; however, the strength of the post, bolts, and washers was only about 23.7 kips. For easy repair, the deck must be able to withstand the bolt failure load with minimal cracking. The standard Texas bridge deck was unable to withstand these loads. However, all modifications to the Texas T101 post and concrete decks tested were capable of developing full strength of the bolts before severe damage occurred to the deck. These modifications were

1. Extending the bottom reinforcing steel in the deck to within 1 in. of the deck edge,
2. Increasing the number of longitudinal bars between the anchor bolts and the deck edge,
3. Adding welded wire fabric in the top of the deck under the posts,
4. Enlarging the bolt anchor plate as shown in Figure 2,
5. Increasing the post edge distance, and
6. Increasing the deck thickness under the post.

Several observations were made from this test series. The crushing of concrete is not as great for tests with the modified anchor plate as for the welded wire fabric. The edge distance affects the amount of spalling of the concrete behind the post. The thickness of the slab affects the strength of the slab, but not the crack pattern.

TEXAS BRIDGE RAILING TYPE T202

Figure 4 is a composite drawing of the cross section of the T202 bridge rail constructed on the 7.5-in. bridge deck. The solid lines represent the standard steel configuration and the broken lines represent the modifications. The T202 bridge rail is constructed of 5-ft x 7.5-in. reinforced concrete posts with 5-ft openings and a heavily reinforced concrete rail on top. The deck reinforcing steel is grade 40 and the post and rail reinforcing steel is grade 60. The 13 No. 4 bars in the traffic side of the post bend into the deck bottom steel and the 5 No. 4 bars in the field side of the post extend straight into the deck. The distance from the back of the post to the deck edge is 1.5 in. The bottom bars stop 6 in. from the edge of the deck as in the standard deck.

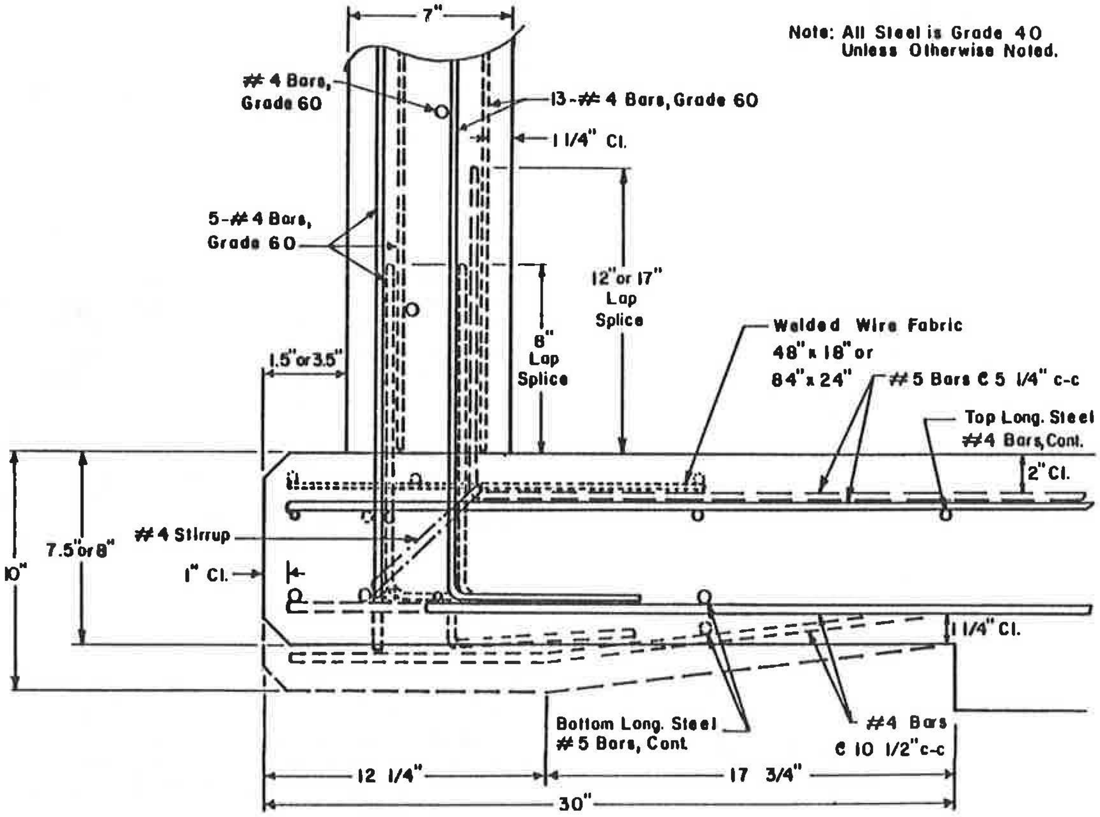


FIGURE 4 Composite drawing of modifications to Texas T202 bridge railing.

Photographs of typical crack patterns are shown in Figure 5. A summary of the tests performed on the T202 concrete bridge rail and deck is given in Table 3.

Tests on Standard T202 Bridge Railing

One static test (T202-1S) and one dynamic test (T202-1D) were performed on the standard T202 concrete bridge rail. The peak load on the static test was 26.3 kips at 0.3-in. deflection. Concrete along the deck edge spalled off, thus exposing the outermost reinforcing bar. Cracks also appeared in the deck at each end of the 5-ft concrete post. No cracks appeared in the post.

For the dynamic test, the peak load was 109 kips at 1.3-in. deflection. The energy absorbed was 79,677 kip-ft. The deck under the post was broken along the field-side steel. The post proper was not damaged. The dynamic test was conducted with a 5,143-lb cart impacting the post at 23 mph.

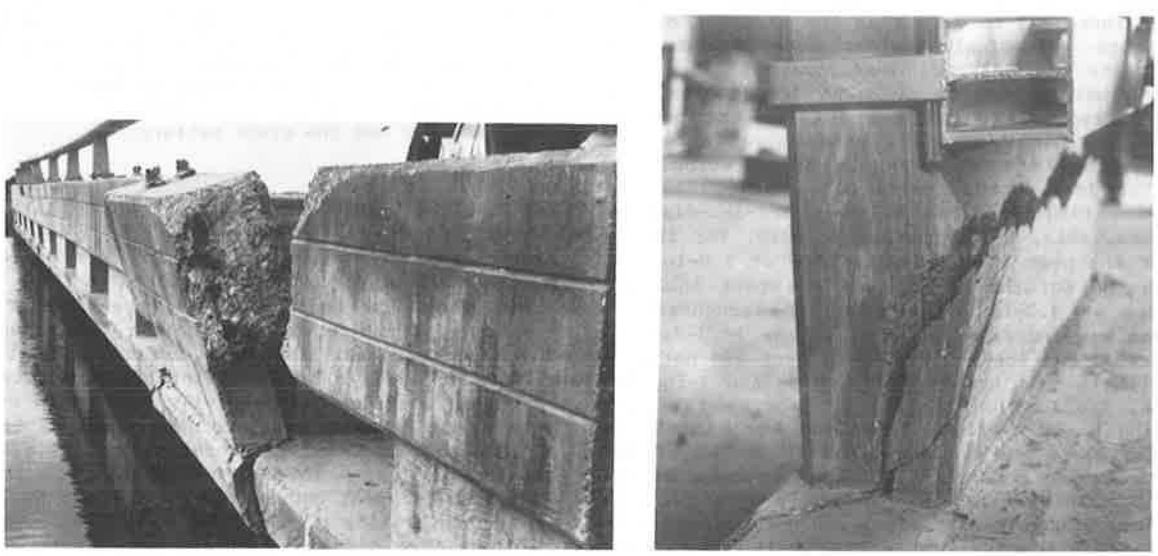


FIGURE 5 Typical crack patterns for Texas T202 concrete post and bridge deck.

TABLE 3 Summary of Test Results on T202 Bridge Railing

TEST NO. AND TYPE	DECK DEPTH (in.)	EDGE DIST. (in.)	REINF. PATTERN	TDLR	BDLR	B (in.)	SPECIAL REINF.	PEAK LOAD (kip)	DISPL (in.)	Remarks
T202-1D	7.5	1.5	Std.	1, No. 4	1, No. 5	6	–	109	1.6	Severe cracking of concrete deck; load falls to 9 kips at 7 in.
T202-1S	7.5	1.5	Std.	1, No. 4	1, No. 5	6	–	26.3	0.3	Severe cracking and spalling of concrete deck; load falls to 7 kips at 8 in.
T202-2S	7.5	1.5	Std.	1, No. 4	1, No. 5	1	Welded wire fabric, 48 x 18 in.	25.1	0.5	Severe cracking of concrete deck; load falls to 7.5 kips at 8 in.
T202-3S	7.5	1.5	Std. w/8-in. Post Lap Spl.	1, No. 4	1, No. 5	1	Welded wire fabric, 48 x 18 in.	21.4	0.4	Severe cracking of concrete deck; load falls to 7 kips at 8 in.
T202-4S	7.5	1.5	Std. w/8-in. Post Lap Spl.	1, No. 4	1, No. 5	1	Welded wire fabric, 85 x 24 in.	21.7	0.9	Severe cracking of concrete deck; load falls to 9 kips at 8 in.
T202-5S	8	1.5	Mod. 1, 12-in. Post Lap Spl.	2, No. 4	2, No. 5	1	No. 4 sitrrup at 2.625 in.	24.9	0.8	Severe cracking of concrete deck; load falls to 9 kips at 8 in.
T202-6S	10	1.5	Mod. 1, 12-in Post Lap Spl.	2, No. 4	2, No. 5	1	No. 4 stirrup at 2.625 in.	31	0.7	Minor deck cracking; posts crack at end 12-in. lap splice; load falls to 7.5 kips at 4 in.
T202-7S	8	3.5	Mod. 2, 17-in. Post Lap Spl.	2, No. 4	2, No. 5	1	No. 4 stirrup at 2.625 in.	23.4	1.1	Minor deck cracking; post cracks at end 17-in. lap splice; load falls to 3.5 kips at 3 in.
T202-8S	10	3.5	Mod. 2, 17-in. Post Lap Spl.	2, No. 4	2, No. 5	1	No. 4 stirrup at 2.625 in.	29.2	0.9	Minor deck cracking; post cracks at end 17-in. lap splice; load falls to 8 kips at 5.8 in.
T202-9S	8	3.5	Std.	2, No. 4	2, No. 5	1	–	35	0.9	Deck cracking; load falls to 6 kips at 8 in.
T202-10S	10	3.5	Std.	2, No. 4	2, No. 5	1	–	40	0.9	Deck cracking; load falls to 8 kips at 8 in.

Note: See Table 2 for key to abbreviations.

Tests on Modified Designs for T202 Bridge Railing

For less repair work, the failure should occur in the post, not the deck. To do this, the deck was strengthened by adding 48 x 18-in. welded wire fabric to the top steel and lengthening the top and bottom steel to within 1 in. of the deck edge. The peak load in test T202-2S was 25.1 kips at 0.5-in. deflection. The shape of this load deflection curve and the peak load in this test was similar to the test on the standard design. Spalling of concrete occurred, and the deck cracking mode was also similar to that of previous tests.

The design for test T202-3S was the same as test T202-2S, except the tension and compression steel in the post have an 8-in. lap splice beginning on top of the deck. The 8-in. lap splice as used to help force the failure in the deck. The compression steel in the post is bent in the deck to join the bottom steel, similar to the tension steel configuration. The peak load was 21.4 kips at 0.4-in. deflection. Severe cracking occurred in the deck at each end of the post, but the concrete did not spall off the deck edge along the post. This could be due to the reduced failure load.

The previous design was modified to contain an 85 x 24-in. welded wire fabric mat. In this test (T202-4S) the peak load was 21.7 kips at 0.9-in. deflection. The crack pattern was the same as the previous test, with no spalling of concrete. It was determined that the welded wire fabric gave no advantage to the system and was eliminated.

The next step was to drastically modify the steel in the deck. The No. 5 bars at 5.25-in. centers in the top of the deck were bent up to form the tension steel in the post with a 12-in. lap splice. The compression steel in the post had no splices and was straight in the deck. A longitudinal No. 4 bar was placed in the top of the deck and a longitudinal No. 5 bar was placed in the bottom of the deck on either side of the compression steel. A No. 4 bar stirrup was placed at 45 degrees in the deck, thus connecting the top and bottom steel to strengthen the deck edge under the post. In an 8-in. deck (T202-5S) the peak load was 24.9 kips at 0.8-in. deflection. In a 10-in. deck (T202-6S) the peak load was 31.0 kips at 0.7-in. deflection. Severe spalling and cracking of the concrete occurred in the 8-in. deck and in the post. In the deck on the traffic side, the concrete above the top steel lifted up. However, in the 10-in. deck severe cracks appeared in the wall along the 12-in. lap splice before cracking appeared in the deck. Even after the load fell to 12 kips at 1.5-in. deflection, the cracks in the deck were repairable.

To improve on this design the lap splice in the tension steel was increased to 17 in. The post edge spacing was increased from 1.5 to 3.5 in. to try to curtail the spalling concrete behind the post. Two top longitudinal No. 4 bars in the deck were placed between the post compression steel and the edge of the deck. In the 8-in. deck (T202-7S) the peak load was 23.4 kips at 1.1-in. deflection, with no spalling concrete behind the post. The post had cracks along the 17-in. lap splice and the deck had cracks at each end of the post. The 10-in. deck (T202-8S) had a peak load of 29.2 kips at 0.9-in. deflection. The major cracking is confined to the wall, with minor cracking in the deck at either side of the post.

The final tests in this series were on a simplified design. The top and bottom reinforcing steel in the deck were both straight and continued to within 1 in. of the edge of the deck. The compression steel in the post consisted of five No. 4 bars that were straight and continued into the deck to bottom steel with no lap splices. The 13 No. 4 tension bars were continuous from the post to bend into the bottom reinforcing steel of the deck with an 8-in. splice in the deck. The post edge distance was 3.5 in. and there were two longitudinal No. 4 bars continuous in the top steel of the post and two No. 5 bars bottom steel. For the 8-in. deck (T202-9S), the peak load was 35 kips at 0.9-in. deflection. For the 10-in. deck (T202-10S), the peak load was 40 kips at 0.9-in. deflection. In both of these tests the concrete was cracked along the edge of the deck but did not spall off. The cracking may be due to the lack of stirrups; however, the increased edge distance and additional longitudinal steel prevented major spalling of the concrete in the deck along the post.

Conclusions

An observation made from this test series is that the wall strength is much greater than the deck strength. To force the failure in the wall means to weaken the wall by using lap splices in the deck-to-wall steel and to strengthen the deck by using increased wall setback distances and increased deck thicknesses.

Four of the modifications to the T202 railing and deck gave the deck added strength. These modifications are to

1. Extend the slab reinforcing steel to within 1 in. of the deck edge,
2. Increase the edge distance from 1.5 to 3.5 in.,
3. Increase the slab thickness, and
4. Place additional longitudinal reinforcing bars in the slab.

The modifications that did not increase the strength were the addition of welded wire fabric and the addition of the No. 4 bar stirrup. However, the No. 4 bar stirrup appeared to reduce cracking in the deck. It should also be noted that the length of the wall lap splice is a major factor in the load capacity of the wall.

TEXAS BRIDGE RAILING TYPE T5

The final tests series was performed on 8-ft sections of the T5 bridge rail. The purpose of testing this widely used bridge rail was to investigate its strength characteristics. The T5 bridge rail is a 32-in.-high continuous concrete parapet rail with face geometry similar to the New Jersey concrete safety shape. Figure 6 is composite drawing of the T5 bridge rail. The railing contains seven No. 4 bars continuous the length of the rail, and No. 5 bar stirrups at 8-in. centers that are formed in the shape of the rail with a 1.25-in. clearance all

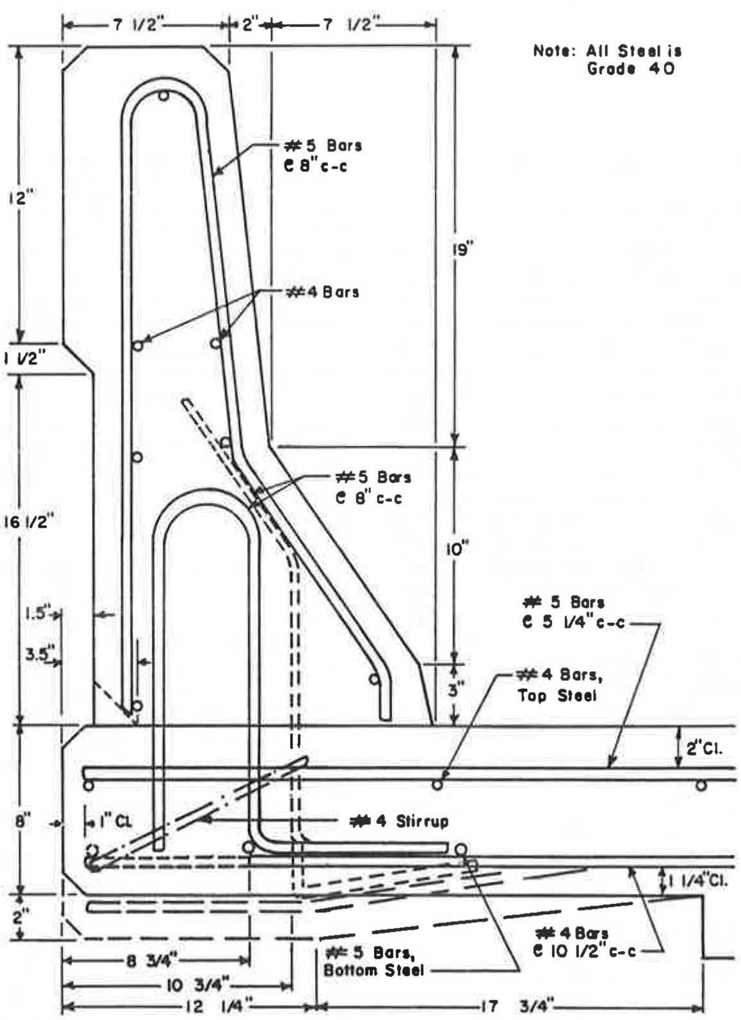
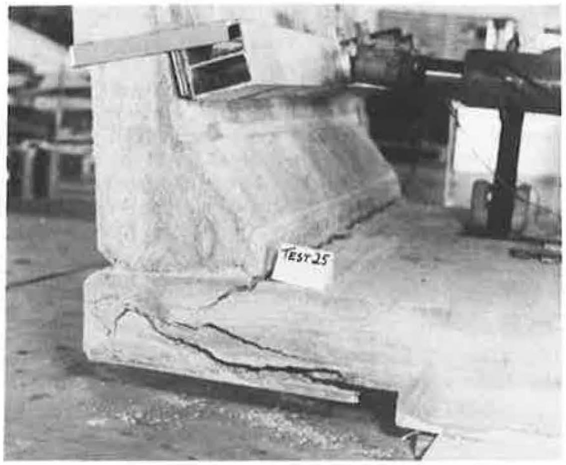


FIGURE 6 Composite drawing of modifications to Texas T5 bridge railing.



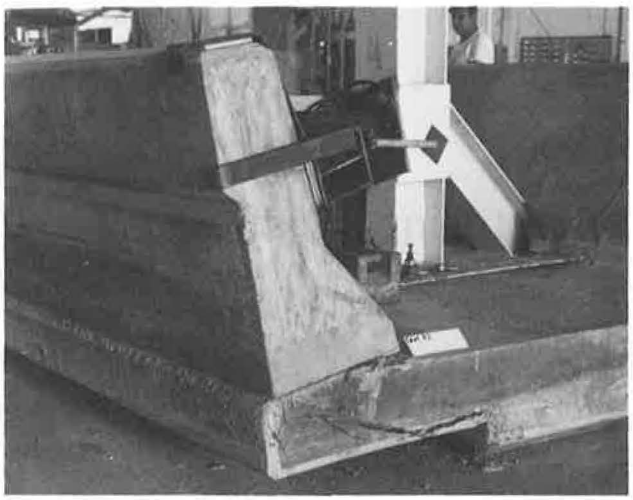


FIGURE 7 Typical crack patterns for Texas T5 concrete post and bridge deck.

around. The concrete rail element is connected to the deck by two legs of a No. 5 hair-pin bar at 8 in. on center. The tension leg is 8.75 in. from the deck edge. One static test was performed on the standard design, and three static tests were performed on modified designs. Photographs of typical crack patterns are shown in Figure 7. A summary of the tests performed on the T5 railing is given in Table 4.

Test on T5 Bridge Railing

The test performed on the standard T5 bridge rail with an 8-in. deck revealed the peak load to be 45 kips at 1.2-in. deflection. The rail sustained no cracking, although the deck sustained severe cracking along the tension side of the hair-pin bars. Hairline cracks along the deck edge showed a potential problem of spalling concrete. Prying action of the rail on the deck was considered the major problem.

Tests on Modified Designs of T5 Bridge Railing

For the first modification, the top and bottom steel of the deck was extended to within 1 in. of the edge of the deck. The hair-pin bar was modified so that only one leg, in tension, connected the rail to the deck. This tension leg was 10.75 in. from the edge of the deck. The backside of the rail was chamfered 2 in. to help reduce the prying action. The chamfer effectively increases the edge distance from 1.5 to 3.5 in. without moving the rail. The peak load was 36.2 kips at 0.4-in. deflection. The crack pattern was similar to the crack pattern of the standard design without the hairline cracks along the deck edge.

To reduce the cracking in the slab, a No. 4 bar stirrup was placed in the deck of the previous design. For an 8-in. deck, the peak load was 42.2 kips at 0.6-in. deflection.

A test of this modified design without the No. 4 bar stirrup was performed on a 10-in. deck. The peak load was 49.1 kips at 0.5-in. deflection.

Major cracks occurred in the deck along the tension leg of the hair-pin bar, except in the design with the No. 4 bar stirrup. In this test the related crack formed at the traffic edge of the parapet and angled 45 degrees into the deck. The 10-in. deck also had a hairline crack where the deck thickness was reduced to join the 8-in. standard deck thickness.

Conclusions

Four modifications were made on the standard design:

1. Change the No. 4 hair-pin bar from two legs with the tension member at 8.75 in. to a modified hair-pin bar with one leg (tension member) at 10.75 in.,
2. Increase the edge distance by adding a 2-in. chamfer at the back of the rail,
3. Add a No. 4 bar stirrup, and
4. Increase the deck thickness.

Of these, only the increased deck thickness gave added strength to the deck and rail design over the standard design. The No. 4 bar stirrup added strength to the modified deck design. The 2-in.

TABLE 4 Summary of Test Results on T5 Bridge Railing

TEST NO. AND TYPE	DECK DEPTH (in.)	EDGE DIST. (in.)	REINF. PATTERN	TDLR	BDLR	B (in.)	SPECIAL REINF.	PEAK LOAD (kip)	DISPL (in.)	Remarks
T5-1S	8	1.5	Std.	1, No. 4	1, No. 5	6	—	45	1.2	Moderate deck cracking; load falls to 15 kips at 5 in.
T5-2S	8	3.5	Mod. 1	1, No. 4	1, No. 5	1	—	36.2	0.4	Moderate deck cracking; load falls to 10 kips at 3 in.
T5-3S	8	3.5	Mod. 1	1, No. 4	1, No. 5	1	No. 4 stirrups at 2.625 in.	42.2	0.6	Moderate deck cracking; load falls to 18 kips at 3.5 in.
T5-4S	10	3.5	Mod. 1	2, No. 4	1, No. 5	1	—	49.1	0.5	Moderate deck cracking; load falls to 10 kips at 5 in.

Note: See Table 2 for key to abbreviations.

chamfer decreased the strength of the system by decreasing the strength of the rail.

ACKNOWLEDGMENT

The data in this paper originated from a study conducted by the Texas Transportation Institute for the Texas State Department of Highways and Public Transportation in cooperation with the FHWA, U.S. Department of Transportation.

REFERENCES

1. Standard Specifications for Highway Bridges, 12th ed. AASHTO, Washington, D.C., 1977.
2. C.E. Buth, J.S. Noel, A. Arnold, and T.J. Hirsch. Safer Bridge Railings. Final Report. Texas Transportation Institute, Texas A&M University, College Station, May 1983.
3. T.J. Hirsch. Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks. Res. Report 230-2. Texas Transportation Institute, Texas A&M University, College Station, 1978.
4. Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 11th ed. AASHO, Washington, D.C., Sept. 1974, Parts I and II.

Publication of this paper sponsored by Committee on Safety Appurtenances.

Design and Development of Self-Restoring Traffic Barriers

MAURICE E. BRONSTAD and CHARLES F. McDEVITT

ABSTRACT

The development of the self-restoring barrier (SERB) guardrail system for the FHWA demonstrated that a high-performance flexible barrier that was damage resistant was technically and economically feasible. To extend the SERB concept into other applications, FHWA contracted with Southwest Research Institute to design and develop SERB retrofit bridge railing, SERB deck-mounted bridge railing, and SERB median barrier systems. In this paper the SERB retrofit and median barrier designs that have been fully evaluated at this time are described. The SERB bridge rail retrofit, consisting of an articulated tubular Thrie-beam mounted on a narrow safety walk and parapet installation, was subjected to a full range of vehicle impacts from a 40,000-lb (18 000-kg) intercity bus to an 1,800-lb (800-kg) Honda Civic. Results of these 60 mph (95 km/h) tests indicate satisfactory performance. The SERB median barrier concept constructed of single Thrie-beams with internal truss shear webs was successfully evaluated in a test series that included a 40,000-lb intercity bus and an 1,800-lb Honda. Development of the SERB deck-mounted bridge railing is currently in progress.

The popularity of the concrete safety shape barrier is attributed to generally satisfactory performance with a wide range of vehicles and the resulting low damage repair due to these impacts. Design and development of the self-restoring barrier (SERB) guardrail for the FHWA was reported to TRB at the 1981 Annual Meeting (1). The SERB guardrail performance range exceeds that of the concrete safety shape and is damage resistant for the majority of expected impacts. As an extension of the SERB concept, FHWA contracted with Southwest Research Institute (SwRI) to design and develop SERB concepts for bridge railing retrofit, deck-mounted bridge railing, and median barriers.

In this paper the design and development of the SERB bridge rail retrofit and median barrier are described. The SERB deck-mounted bridge railing is currently in the development stage.

OBJECTIVES AND SCOPE

The objectives of this work were to design and develop self-restoring systems to upgrade existing bridge railings, and to design and develop a new self-restoring median barrier system.

Work on the barriers discussed in this paper included designs that used computer simulations, and development that used component and full-scale crash tests; the emphasis of this paper is on the full-scale crash tests. Test vehicles used in the evaluations included

1. A 40,000-lb (18 000-kg) intercity bus,
2. A 20,000-lb (9000-kg) school bus,
3. A 4,500-lb (2000-kg) car, and
4. A 1,800-lb (800-kg) car.

Impact speed was 60 mph (95 km/h) and impact angles were 15 degrees, except for the 25-degree angle used in the 4,500-lb car tests.

With the exception of test SMB-3, each of the test vehicles contained two fully instrumented part 572 anthropometric dummies (50th percentile males).