# DYNAMIC TESTS OF THE CALIFORNIA TYPE 15 BRIDGE BARRIER RAIL

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The results of two full-scale vehicle impact tests of the California Type 15 bridge barrier rail are reported. The Type 15 bridge rail is a semirigid system consisting of two 3½-in. square structural steel tubular rails mounted 14 and 27 in. above the pavement on 6-WF-25-steel posts bolted to the edge of the reinforced concrete bridge deck. The post spacings tested were 6 ft 3 in. and 9 ft 4½ in. This bridge barrier rail was designed for use on secondary California highways with maximum bridge widths of 32 ft. The tests were conducted at impact velocities of approximately 60 mph and approach angles of approximately 15 deg. The test results indicate that the bridge rail designs tested will retain and redirect a 4,500-lb passenger vehicle impacting at a speed of 60 mph and an angle of 15 deg. Tolerable deceleration rates, moderate vehicle damage, and minor to moderate barrier damage will be sustained. However, it was concluded that a post spacing of 8 ft 0 in. would provide an effective, economical, and aesthetically pleasing compromise between the relatively rigid 6-ft 3-in. post spacing and the more flexible, but marginal, 9-ft 4½-in. post spacing. It was also concluded that, with a post spacing of 8 ft 0 in. or less, the California Type 15 bridge barrier rail is satisfactory for use on federal-aid secondary highways and other secondary California State highways.

•THE California Type 15 bridge barrier rail was designed by the California Division of Highways' Bridge Department to provide an effective and economical railing for use on bridges on secondary roads.

The metal beam bridge railing frequently used on California's secondary roads in the past was developed and tested in 1959 (1) as part of a test series to investigate existing and proposed bridge rail designs. This metal beam bridge railing consisted of a single steel W-section beam mounted 24 in. high on steel H-section posts bolted to the outside edge of the concrete bridge deck at 6 ft 3 in. on centers (Fig. 1).

In the 1959 tests, a 4,000-lb passenger vehicle was impacted into the bridge railing at a speed of 55 mph and an angle of 30 deg. The crash produced severe wheel-post entrapment and excessive rail deflections (Fig. 2). Although this design was not judged adequate for freeway use, it was considered suitable at that time for placement on federal-aid secondary highways and certain California state highways where only lower speed, flat, oblique-angle collisions were expected. It proved to be an economical and effective barrier under these conditions. However, as heavier, higher speed vehicles became more prevalent on these secondary highways, failures began occurring even at low, oblique impact angles. These failures were attributed to the inability of the single W-section beam to adequately distribute the larger impact loading outside the immediate impact area. Thus, only the posts very close to the impact area were being loaded, and failures were occurring at the post-to-deck connections in much the same manner as had been observed in the 1959 test series.

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In 1967, the single W-section beam was replaced with two 3½-in. square structural steel tubular rails in an effort to correct this deficiency. This provided a post and rail system that conforms to the requirements of the 1969 AASHO Specifications for Highway Bridges. However, these specifications stipulate loading requirements for bridge railings attached to "surface mount" posts. Thus, the adequacy of the AASHO Specifications as applied to the Type 15 bridge rail, with the posts attached to the edge of the bridge deck, had not been evaluated. This exact system had never been subjected to controlled full-scale vehicle impact tests.

A bridge rail system of this type was tested by the New York State Department of Public Works Bureau of Physical Research in 1963 and reported on in 1967 (2). Although somewhat similar in overall appearance, the details of the New York barrier and the Type 15 barrier varied significantly. It was felt that no analogy could be made between the two. Therefore, a series of dynamic tests was deemed necessary to accurately evaluate the effectiveness of the California Type 15 bridge barrier rail.

## OBJECTIVES

The primary objectives of this research project were to (a) test the ability of the California Type 15 bridge barrier rail to effectively retain and redirect a 4,500-lb vehicle impacting at a speed of 60 mph and an angle of 15 deg, (b) determine the structural capabilities of the California Type 15 bridge approach guardrail and its connection to the bridge abutment wing wall, and (c) develop and test subsequent systems design modifications as dictated by the results of the initial impact tests.

## TEST CONDITIONS

### Barrier Design

The test installation consisted of 67 ft of Type 15 bridge barrier rail and 52 ft of Type 15 bridge approach guardrail (Fig. 3).

The initial Type 15 design consisted of two structural steel tubular rails mounted 14 and 27 in. above the pavement on steel posts spaced at 6 ft 3 in. on centers. On the bridge rail portion of the installation, the WF posts were bolted to the edge of a cantilevered reinforced concrete bridge deck (Fig. 4). The steel posts for the bridge approach guardrail (BAGR) were embedded in concrete footings. The posts for both the bridge rail and the BAGR were 6-WF-25-structural steel members conforming to the requirements of ASTM Designation A 36.

Each bridge rail post was attached to the edge of the deck with two high-strength threaded rods 1 in. in diameter and 2 ft long and two high-strength bolts (⅝ in. in diameter and 1 ft long) cast into the reinforced concrete. The high-strength steel rods conformed to the requirements of ASTM Designation A 108, grade 1144. The high-strength bolts conformed to the requirements of ASTM Designation A 325.

The rails were 3½-in. square, 10½-lb structural steel tubing that conformed to the requirements of ASTM Designation A 500, grade B. The interior sleeve-type rail splice (Fig. 5) and the ¾-in. welded stud rail-to-post connectors that proved effective in a previous test series (4) were again used.

The bridge barrier rail was bolted to the outside edge of a reinforced concrete bridge deck 12 in. thick and 67 ft long cantilevered 36 in. off a 24-in. by 30-in. by 68-ft reinforced concrete anchor block. A 6 sack mix was used for the concrete. The 28-day compressive strength of the concrete was 4,735 psi.

The posts for the bridge approcah guard railing were set in concrete footings (5 sack mix) 24 in. in diameter and 36 in. deep. The leading, or upstream, ends of the tubular rails were curved down and anchored to two reinforced concrete footings (6 sack mix) 18 in. in diameter and 36 in. deep.

The type 15 bridge barrier rail design[1], other than the post-to-deck connection, was

[1]The original manuscript of this paper included detailed drawings of the Type 15 bridge barrier rail design, the photographic instrumentation used in the tests, and the vehicle transducer instrumentation. These drawings are available in Xerox form at cost of reproduction and handling from the Highway Research Board. When ordering, refer to Xerox Supplement 39, Highway Research Record 386.

**Figure 1.**

**Figure 2.**

**Figure 3.**

**Figure 4.**

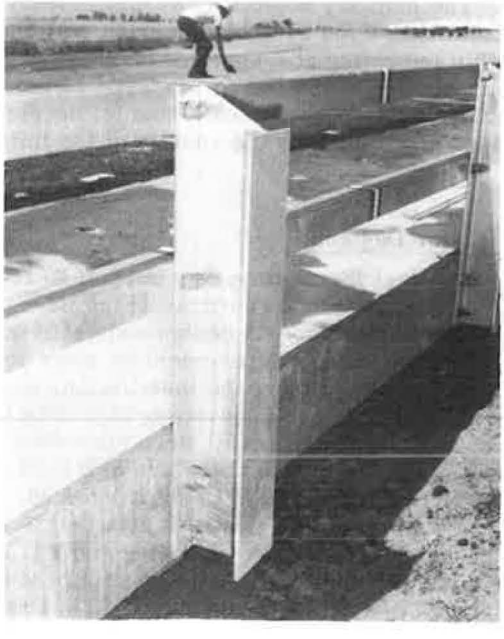
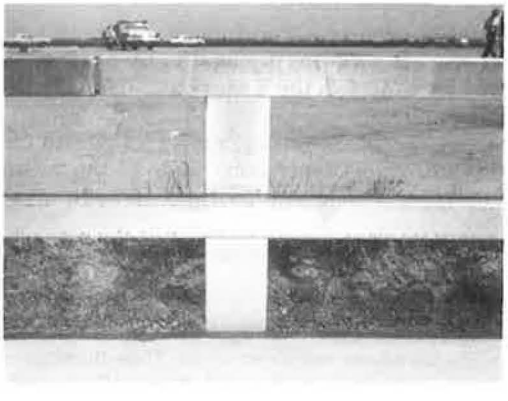


**Figure 5.**



designed in accordance with the requirements of the Standard Specifications for Highway Bridges adopted by the American Association of State Highway Officials in 1969.

### Test Parameters

Test guidelines established by the Highway Research Board Committee on Guardrails and Guideposts (11) specify the use of a ±4,000-lb vehicle, an impact velocity of 60 mph, and an impact angle of 25 deg. For the tests reported here, the vehicle weighed 4,550 lb including an anthropometric dummy and on-board instrumentation. Although this weight exceeds HRB guidelines, it is more representative of the more severe conditions currently being encountered on California highways.

The planned impact velocity and impact angle for these tests were 60 mph and 15 deg. These values were selected because the bridge barrier rail design tested is intended for use on secondary California highways with maximum bridge widths of 32 ft. It was estimated that, under these conditions, 60-mph/15-deg collisions were representative of the more severe accidents that would actually occur.

### Test Procedures

A description of the procedures used to modify the test vehicles for remote radio control is given elsewhere (5). A description of the photographic and electronic data acquisition systems used during the tests reported here is given in the original report (6).

## TEST RESULTS

### Test 251

Test 251 was conducted to test the ability of the initial Type 15 bridge barrier rail design (6-ft 3-in. post spacing) to redirect a passenger vehicle impacting at a moderate velocity and approach angle (Fig. 6).

Initial barrier contact occurred at midspan between posts B-4 and B-5. The impact velocity and approach angle were 64 mph and 12 deg. The height of the barrier rail elements was such that upon impact the vehicle bumper and leading chassis members rode up and over the lower rail and the upper rail knifed into the body sheet metal just below the headlight. However, there was no further penetration because the lower rail effectively deflected the left front wheel, thus precluding any serious vehicle-barrier entrapment. There was only a 5-deg roll toward the barrier (Fig. 7), and the vehicle was effectively redirected to an exit angle of 3 deg with the barrier.

The total vehicle-barrier contact was approximately 10 ft. The post-impact vehicle trajectory was satisfactory with a maximum vehicle rebound into the traveled lanes of 13 ft.

Barrier damage was relatively minor. Two rail sections and three posts were deformed and would have required replacement for aesthetic reasons. However, all the barrier components were intact structurally and the barrier was still functional. The maximum residual lateral rail deflections occurred at post B-5, approximately 3 ft downstream of initial impact. The permanent deformations of the upper and lower rails were 0.21 ft and 0.14 ft respectively (Fig. 8).

The flanges of the three deformed posts were bent above their upper post-to-deck connections. Maximum residual lateral post deflections, measured from the upper edge of the deck, were (a) post B-4, 3.0 ft upstream of impact, ¼ in., (b) post B-5, 3.2 ft downstream of impact, ½ in., and (c) post B-6, 9.4 ft downstream of impact, ⅜ in. There was no damage to any of the post-to-deck connectors, rail stud bolts, or splice sleeves, and except for insignificant surface spalling, there was no concrete damage.

Vehicle damage was moderate, consisting of paint scratches and sheet metal deformation at the left front corner, along the left side, and at the left rear fender. The grill, headlights, and fender at the left front end were extensively deformed, a portion of the front bumper was torn away, and the bumper mounting brackets and leading frame members were distorted back toward the front wheel. However, the deformation was essentially superficial, and, except for the possible rubbing of distorted sheet metal against the front tire, the vehicle appeared to be operable (Fig. 9).

Figure 6.

Figure 7.

Figure 8.

Figure 9.

Figure 10.

Inside the passenger compartment, there was no appreciable deformation of the steering wheel rim or of the left front door frame to indicate that the dummy had been subjected to high deceleration forces (Fig. 10). However, deceleration recording instrumentation indicated that the deceleration forces, particularly in the lateral direction, were higher than anticipated or desired. Records of the instrumentation data are contained elsewhere (6). A summary of these data is as follows:

1. The highest 50-msec average vehicle deceleration (longitudinal) was 4.7 g (using two accelerometers);
2. The highest 50-msec average vehicle deceleration (lateral) was 9.0 g (using two accelerometers);
3. The highest 50-msec average dummy (head) deceleration was 25.0 g (using three accelerometers); and
4. The highest 50-msec average dummy (chest) deceleration (longitudinal) was 4.6 g (using one accelerometer).

The maximum seat belt load was 1,350 lb. The Gadd Severity Index was 278.

Test 252

Analysis of the results of Test 251 led to the modification of the test barrier installation to provide a post spacing of 9 ft 4½ in. The post spacing was increased to introduce more flexibility into the barrier rail system, thereby lessening the severity of a collision with the barrier. To achieve this modification, seven posts were removed and 2-ft square sections of the cantilevered bridge deck were removed at three locations. New post anchor bolts were installed at these locations, the deck edges within the removed sections were coated with epoxy, and new concrete was cast using a 6 sack mix. The 28-day compressive strength of the concrete was 4,540 psi. The steel rail sections from the original barrier were modified to provide stud bolts and rail splices at the new locations as required. This resulted in a discontinuity in the lower rail. However, this discontinuity was far enough from the location of impact such that it did not affect the test results. The height of the upper and lower rails was identical to that tested in Test 251 (Fig. 11).

Initial barrier contact occurred 2.7 ft upstream of post B-5 at a speed of 59 mph and an angle of 14 deg. Vehicle-barrier interaction was similar to that observed in Test 251.

Vehicle tire scrub marks on the bridge deck indicated that the left front wheel had come dangerously close to the edge of the bridge deck. If the wheel had dropped off the deck, serious wheel-post entrapment could have resulted.

Again, vehicle dynamics through impact were good. A 7-deg roll toward the barrier occurred (Fig. 12) and the vehicle was effectively redirected to an exit angle of 2 deg with the barrier. The total vehicle-barrier contact was approximately 14 ft. The maximum vehicle rebound into the traveled lanes was 22 ft. In view of the low 2-deg exit angle, the overall post-impact vehicle trajectory was considered satisfactory.

The barrier damage was more severe than that which was observed after Test 251. Two rail sections and two posts were deformed and would have required replacement. Although all of the principal barrier components remained physically intact, it is doubtful that the barrier could have sustained a subsequent impact into the damaged section without failure. The maximum residual lateral rail deflections occurred at midspan between posts B-5 and B-6, approximately 7.4 ft downstream of initial impact. Deflection of the top rail was 0.56 ft and of the bottom rail 0.43 ft (Fig. 13). Maximum residual lateral post deflections, measured from the upper edge of the deck, were (a) post B-5, 2.7 ft downstream of impact, 1⅞ in. and (b) post B-6, 17.1 ft downstream of impact, 1⅞ in. Although the post deflections are numerically equal, thus indicating similar loadings, at post B-6 the downstream upper post-to-deck connector (high-strength threaded rod 1 in. in diameter) failed in tension and, consequently post flange deformation was absent at that point. Minor post flange deformation did occur on that side of the post just above the lower connector (Fig. 14). On the upstream side, post flange deformation occurred above the upper post-to-deck connector, which remained intact (Fig. 15).

At post B-5, all post-to-deck connectors were intact and both post flanges deformed above the upper connectors (Fig. 16). However, a flange-web fracture (0.1-in. by 3-in.

Figure 11.

Figure 12.

Figure 13.

Figure 14.

Figure 15.

Figure 16.

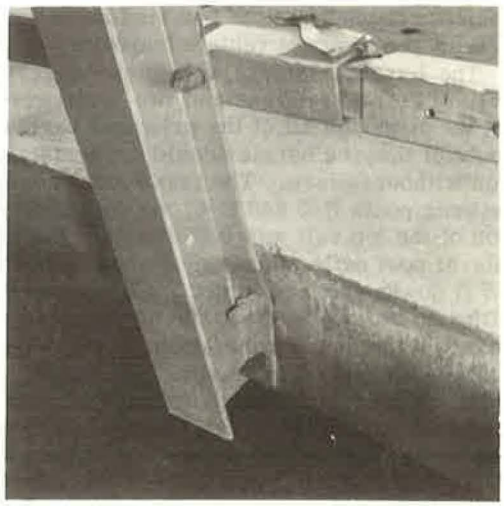


**Figure 17.**

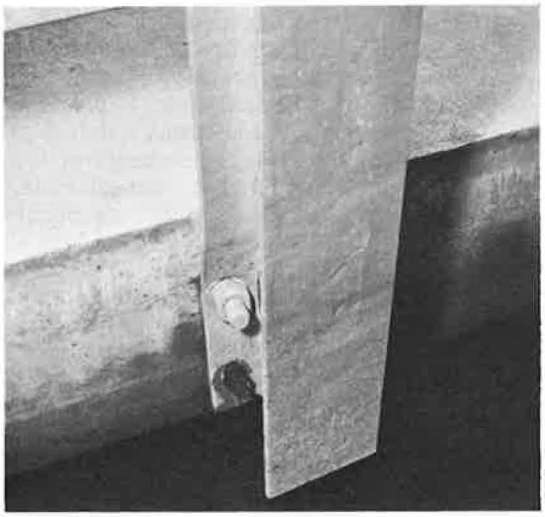


**Figure 18.**

**Table 1. Vehicular decelerations.**

| Accelerometer Orientation in Vehicle | No. Accelerometers | Highest 50-msec Average Deceleration (g) | | Highest 200-msec Average Deceleration[a] (g) | | |
|---|---|---|---|---|---|---|
| | | Test 251 | Test 252 | Unrestrained | Lap Belt | Lap Belt and Shoulder Harness |
| Lateral | 2 | 9.0 | 3.9 | 3 | 5 | 15 |
| Longitudinal | 2 | 4.7 | 3.1 | 5 | 10 | 25 |

[a] Ref. 8.

separation) occurred adjacent to the downstream upper post-to-deck connector (Fig. 17). There was no damage sustained by any of the rail stud bolts or splice sleeves.

Concrete damage was limited to minor spalling at the lower anchor bolts and on the underside of the bridge deck at post B-6 (Figs. 14 and 15). The failure of the upper post-to-deck connector at this post transferred impact loading to the lower connector and contributed to the concrete damage. However, it should be noted that post B-6 was at one of the rebuilt sections of the bridge deck. During reconstruction it was not always possible to install the new lower post-to-deck connectors above the existing lower longitudinal deck reinforcing steel as specified on the plans. If this had not been the case, the load transfer capability of the deck reinforcing may have precluded some of the concrete spalling.

Laboratory tests of the failed barrier components from Test 252 were conducted to check on the possibility of defective material. A hardness test was performed on the sheared-off end of the failed post-to-deck connector from post B-6. This produced an average Brinell reading, with a $\frac{1}{16}$-in. ball, of 94 on the B scale. This value approximates a tensile value of 100,000 psi, which is comparable with the minimum specified tensile strength requirement for the anchor bolts of 105,000 psi. However, because it was both an approximate value and slightly below specification, the remainder of this connector was jackhammered from the bridge deck for further testing. A standard tensile test resulted in values of 108,700 psi ultimate and 91,300 psi yield. Both values are well above the specified minimum strength for this material. A tensile specimen was also cut from the failed post (B-5). The test results were 67,400 psi ultimate and 41,400 psi yield. Both of these values were well above the minimum specified values for the post material. The failures were therefore attributed to the inability of the rails to transmit the impact loading to a sufficient number of posts due to the greater post spacing in this design.

Vehicle damage was generally similar to that observed in Test 251 and consisted of paint scratches and sheet metal deformation at the left front corner, along the left side, and at the left rear fender. At the left front corner, sheet metal deformation was slightly less than that observed in the first test. However, the bumper mounting brackets and leading frame members were more extensively distorted, the left front wheel rim was deformed, and the tire was ruptured. Damage along the left side was also similar to that observed in the first test. However, the left rear fender damage was more severe than that observed after Test 251. This indicated that a harder rear end slap occurred as the vehicle was being redirected (Fig. 18). Data film alalysis revealed that this was due to the larger rail deflections in this test. These large deflections permitted the vehicle to pocket into the barrier and follow the deflecting rails rather than rebound, or "bounce," off as observed in Test 251 on the more rigid initial design. Although vehicle body damage was essentially superficial, the damaged left front wheel rendered the vehicle inoperable. There was no evidence inside the vehicle passenger compartment to indicate that the dummy driver was subjected to excessive deceleration forces. This was verified by the accelerations recorded, which were generally less than those recorded in Test 251 (6). A summary of the data is as follows:

1. The highest 50-msec average vehicle deceleration (longitudinal) was 3.1 g (using two accelerometers);
2. The highest 50-msec average vehicle deceleration (lateral) was 3.9 g (using two accelerometers);
3. The highest 50-msec average dummy (head) deceleration was 24.0 g (using three acelerometers); and
4. The highest 50-msec average dummy (chest) deceleration was 4.4 g (using one accelerometer).

The maximum seat belt load was 120 lb, and the Gadd Severity Index was 234.

## DISCUSSION OF FINDINGS

### General

The initial Type 15 bridge barrier rail design, impacted in Test 251, appeared to be effective in redirecting a passenger vehicle impacting at a moderate velocity and angle.

Vehicular redirection was smooth, barrier damage was minor, and vehicle damage was moderate. However, it was also apparent, from the post-impact vehicle trajectory and the low residual barrier deflections, that the system was more rigid than necessary. Spacing was therefore increased from 6 ft 3 in. to 9 ft 4½ in., which would result in lower decelerations in the vehicle passenger compartment because of the increased barrier flexibility and an economic saving through a 33 percent decrease in the number of barrier posts used. The 9-ft 4½-in. post spacing was arbitrarily selected as an economic expedient because this modification could easily be effected on the existing test installation by removing every second and third post and replacing them with a single post.

Test 252, conducted on this modified system, substantiated the desirability of increasing the barrier's flexibility. However, the barrier damage, particularly at the post-to-deck connection, and the proximity of the left front wheel of the vehicle with the edge of the deck during vehicle redirection was such that the 9-ft 4½-in. post spacing was considered marginal. Thus a post spacing of 8 ft 0 in. was chosen to obtain the desired flexibility and yet retain sufficient rigidity within the barrier system to effectively contain and redirect an impacting vehicle with moderate vehicle damage, minor barrier damage, and tolerable passenger decelerations.

Neither the bridge approach flare nor the approach flare wing wall were impact tested in this study even though both were included in the initial project proposal. It was decided that the design assumptions that were verified by the results of the tests reported here could be utilized in the design of these appurtenances. Also, because the Type 15 BAGR was structurally similar to the successfully tested Type 8 BAGR (3), it was felt that the results of the Type 8 BAGR tests would be applicable. The Type 8 BAGR utilizes the same 6-WF-25-post and concrete post footing as does the Type 15 BAGR. However, the Type 8 post spacing is 10 ft on centers as compared to the 6-ft 3-in. spacing utilized for the Type 15. The Type 8 rail element is a 6-in. by 2-in., 12.02-lb structural steel tube that conforms to the requirements of ASTM Designation A500, grade B, whereas the Type 15 rail element is a 3½-in. square, 10.50-lb structural steel tube that conforms to the requirements of ASTM Designation A500, grade A or B, or A501. The section modulus of the Type 15 rail is approximately 70 percent that of the Type 8 rail. However, the 6-ft 3-in. post spacing of the Type 15 system is approximately 63 percent that of the Type 8. Therefore, the forces required to exceed the ultimate strength of the Type 15 and Type 8 rail elements are reasonably comparable ($F_{15} = 0.86F_8$). By increasing the post spacing of the Type 15 BAGR from 6 ft 3 in. to 8 ft 0 in. we can decrease this ratio to 0.67. However, the lateral kinetic energy imparted to the barrier during a 15-deg impact is only 37 percent of that imparted to the barrier at the 25-deg impact angle used for the tests of the Type 8 BAGR. Thus, an 8-ft 0-in. post spacing should be adequate for the Type 15 BAGR as well as for the Type 15 bridge rail.

Observation of the effect of the impact load distribution into the reinforced concrete bridge deck led to the decision that the structural design criteria utilized for the deck could be applied to the design of the approach flare reinforced concrete wing wall. It was felt that this would be an appropriate application, thus obviating the necessity of constructing a test installation and performing a full-scale impact test.

One problem encountered during construction or reconstruction of the bridge barrier installation was with the interior sleeve rail splice. It was reported by construction personnel that the lateral sliding tolerance between the sleeve and the interior of the tubular rail was too great; thus rail alignment at the splices was not as close as was desired. However, it should be noted that this clearance must be adequate to permit the splice sleeve to slide readily inside the tube for ease of barrier construction and rail replacement.

Another point of concern was the dimensional tolerances for the slotted hole in the tubular rail. When repairs were made the splice sleeves were not readily interchangeable, particularly when a tube that had been bent from the previous impact was used. This, however, could easily be remedied by increasing the slot width from 7/16 in. to ½ in. This should provide the needed tolerance for interchangeability.

Also, some method of sliding the sleeve other than hammering on the bolt head should be devised. The use of either a slot in the adjoining tube, with a corresponding hole in the splice sleeve, or a slot and corresponding hole on the opposite side of the

slotted tube would suffice. This would provide for the use of a drift pin to slide the splice sleeve and would facilitate assembly and disassembly of the barrier. Except for the aforementioned items, barrier construction and collision repairs were relatively easy and economical.

Interpretation of Instrumentation Data

The severity of the 50-msec vehicular decelerations reported here was determined by comparing the deceleration magnitudes with the recommended 200-msec deceleration tolerance limits proposed by Cornell. Injury severity predictions are related only to the direction of deceleration that appears to be most critical (i.e., no vectorial addition of deceleration was accomplished unless otherwise noted). A discussion of deceleration tolerances and the reasoning behind the choice of these values are given elsewhere (5). These limits define what would be, in the opinion of the researchers, a survivable environment under almost all circumstances when applied to the 50-msec time period (Table 1).

Filtered records of vehicular deceleration (100 Hz for Test 251 and 176 Hz for Test 252) were used to compute the highest 50-msec average values (average of ten continuous 5-msec intervals).

The dummy used in Tests 251 and 252 was restrained with a conventional lap belt. Only the vehicular lateral deceleration in Test 251 (9g) exceeded the recommended value for passengers restrained with lap belts (Table 1). This higher value was probably due to the closer post spacing and, hence, more rigid bridge rail system impacted in Test 251.

Longitudinal, lateral, and vertical components of deceleration from the dummy's head were vectorially combined to obtain a resultant value of deceleration. The Gadd Severity Index was

$$\int_{t_1}^{t_2} a^{2.5}\, dt$$

computed for the 50-msec period with the highest average resultant values of head deceleration using 20 time intervals, i.e., dt = 0.0025 sec. A discussion of the Gadd Severity Index and the tolerance of the human head to deceleration is contained elsewhere (9). The Gadd Severity Index (10), based on the resultant deceleration of the dummy's head, was 278 for Test 251 and 234 for Test 252. The lower threshold of fatal head injuries is 1,000 if we assume that penetration of the skull does not occur; therefore, the dummy would only have suffered moderate injuries in both tests, provided the impact occurred on the dummy's forehead or an equally strong portion of the skull and was distributed such that no penetration of the skull occurred.

The maximum seat belt loads measured were 1,350 lb for Test 251 and 120 lb for Test 252, which are not excessive values. The reason for the wide variation in seat belt loads is not readily apparent. It appears that the magnitude of these loads is independent of the 6-g maximum longitudinal dummy chest decelerations, which are almost identical for both tests. It is possible that there was a malfunction in the instrument for one or both tests that caused the wide variation in recorded seat belt loads.

An estimate of injury severity for both collisions can be inferred from the preceding results. Passengers restrained with lap belts and shoulder harnesses would probably have incurred minor or no injuries, passengers with lap belts would have sustained moderate injuries, and passengers who were unrestrained could have suffered serious injuries, particularly in Test 251.

The preceding results indicate that the bridge rail system used for Test 252 was slightly preferable to that used for Test 251 with regard to injury potential because of the lower vehicle decelerations recorded in Test 252 (particularly in the lateral direction). However, the dummy decelerations were approximately the same for both tests and are therefore inconclusive with regard to barrier preference.

The vehicle accelerometer records show that the vehicular backslap decelerations in the longitudinal direction for both tests were less than those recorded during the in-

itial impact. However, the lateral decelerations recorded during both the initial impact and the backslap were approximately equal in both tests.

## CONCLUSIONS

The following conclusions are based on an analysis of the results of the full-scale vehicle impact tests reported here:

1. The initial California Type 15 bridge barrier rail design impacted in Test 251 will retain and redirect a 4,500-lb passenger car impacting at a velocity of 60 mph and an approach angle of 15 deg. Barrier damage can be expected to be minor and vehicle damage moderate. Because of the rigidity of this design (6-ft 3-in. post spacing), however, very little impact energy will be absorbed by the barrier. Thus, vehicle deceleration rates, particularly in the lateral direction, will be somewhat higher than desirable.
2. The modified California Type 15 bridge barrier rail design impacted in Test 252 will retain and redirect a 4,500-lb passenger car impacting at a velocity of 60 mph and an approach angle of 15 deg. Moderate vehicle damage and tolerable passenger compartment deceleration rates will be experienced. Barrier damage, particularly at the post-to-deck connection, will be significant and the barrier deflection will be such that the wheel (s) of the vehicle on the impact side will be very close to the edge of the bridge deck at the time of maximum barrier deflection. Thus, the 9-ft 4½-in. post spacing used in this design is considered marginal.
3. The California Type 15 bridge barrier rail design with post spacing of 8 ft on centers should produce both the desired flexibility within the barrier system and yet retain sufficient rigidity to effectively contain and redirect a 4,500-lb vehicle impacting at a speed of 60 mph and an angle of 15 deg. This 8-ft post spacing will also provide an economical and aesthetically pleasing compromise between the 6-ft 3-in. and the 9-ft 4½-in. post spacings tested.
4. The California Type 15 bridge approach guardrail (BAGR) with a post spacing of 8 ft 0 in. will effectively contain and redirect a passenger vehicle impacting at speeds of up to 60 mph and angles of 15 deg. This conclusion is based not only on the results of the bridge barrier rail tests reported here but also on the results of a previous series of tests of the structurally similar California Type 8 BAGR (Test 174) (3).
5. The assumptions used for the design of the barrier rail-bridge deck connection, which were verified by the results of the tests reported here, can be applied to the design of the approach rail-wing wall connection, thus eliminating the need to construct and test this appurtenance.

## ACKNOWLEDGMENTS

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