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16. ABSTRACT
The results of two full scale vehicle impact tests into the California Type 15 Bridge Barrier Rail are reported. The Type 15 bridge rail is a semi-rigid system consisting of two 3 1/2 in. square structural steel tubular rails mounted at 14 and 27-in. above the pavement on 6 WF 25 steel posts bolted to the edge of the concrete bridge deck.

Tests were conducted at impact velocities of approximately 60 mph and approach angles of approximately 15 degrees. The test results indicated that this bridge rail design will retain and redirect a 4500 lb. passenger vehicle impacting at a 60 mph velocity and an angle of 15 degrees with tolerable deceleration rates, moderate vehicle damage; and if the barrier post spacing is 8.0 feet or less, minimal barrier damage.

An effective, economical and aesthetically pleasing bridge barrier rail has been developed for use on Federal Aid Secondary Highways and other secondary California State Highways as verified by the results of this study.

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HIGHWAY RESEARCH REPORT

DYNAMIC TESTS OF THE CALIFORNIA TYPE 15 BRIDGE BARRIER RAIL SERIES XXV

71-34

STATE OF CALIFOR AND RESEARCH DEPARTMENT

BUSINESS AND TRANSPORTATIC

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

RESEARCH REPORT

NO. M & R 656496

Prepared in Cooperation with the U.S. Department of Transportation, Federal Highway Administration June, 1971

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STATE OF CALIFORNIA—TRANSPORTATION AGENCY

RONALD REAGAN, *Governor*

DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT
5900 FOLSOM BLVD., SACRAMENTO 95819

June 1971
Final Report
M & R No. 656496

Mr. J. E. McMahon
Asst. State Highway Engineer--Bridges
Bridge Department

Dear Sir:

Submitted for your consideration is a research report titled:

DYNAMIC TESTS OF THE

CALIFORNIA TYPE 15 BRIDGE BARRIER RAIL

SERIES XXV

Principal Investigators

J. R. Stoker, R. P. Hackett, and R. N. Doty

Principal Assistant

R. A. Pelkey

Under the General Direction of

Eric F. Nordlin

Very truly yours,

JOHN L. BEATON
Materials and Research Engineer

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ABSTRACT

REFERENCE: Nordlin, E. F., Stoker, J. R., Hackett, R. P., and Doty, R. N., "Dynamic Tests of the California Type 15 Bridge Barrier Rail", State of California, Department of Public Works, Division of Highways, Materials and Research Department Research Report 656496.

ABSTRACT: The results of two full scale vehicle impact tests into the California Type 15 Bridge Barrier Rail are reported. The Type 15 bridge rail is a semi-rigid system consisting of two 3½-in. square structural steel tubular rails mounted at 14 and 27-in. above the pavement on 6 WF 25 steel posts bolted to the edge of the concrete bridge deck.

Tests were conducted at impact velocities of approximately 60 mph and approach angles of approximately 15 degrees. The test results indicated that this bridge rail design will retain and redirect a 4500 lb. passenger vehicle impacting at a 60 mph velocity and an angle of 15 degrees with tolerable deceleration rates, moderate vehicle damage; and if the barrier post spacing is 8.0 feet or less, minimal barrier damage.

An effective, economical and aesthetically pleasing bridge barrier rail has been developed for use on Federal Aid Secondary Highways and other secondary California State Highways as verified by the results of this study.

KEY WORDS: Dynamic tests, bridge rails, barriers, impact tests, bridge approaches.

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ACKNOWLEDGEMENTS

This work was accomplished in cooperation with the United States Department of Transportation, Federal Highway Administration, as Item D-4-90 of Work Program HPR-PR-1(7), Part 2, Research. The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Federal Highway Administration.

The dedication and competence of the following staff members of the Materials and Research Department made the successful testing described herein possible:

1. Roger Stoughton, Orvis Box, and Lee Staus assisted in the barrier construction and the test site preparation, instrumented the test vehicle, conducted the tests, and assisted with the data analysis and preparation of this report.

2. William Chow, Richard Johnson, Merle Wilson, and Delmar Gans calibrated the test vehicle and dummy instrumentation, assembled and operated the data acquisition and processing systems, and assisted in the interpretation of the data obtained with these electronic systems.

3. Robert Mortensen provided both data and documentary photographic coverage of the tests.

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1. INTRODUCTION

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 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 inches high on steel H-section posts bolted to the outside edge of the concrete bridge deck at 6-ft. 3-in. on centers (Exhibit 1, Appendix and Figures 1 and 2 below).

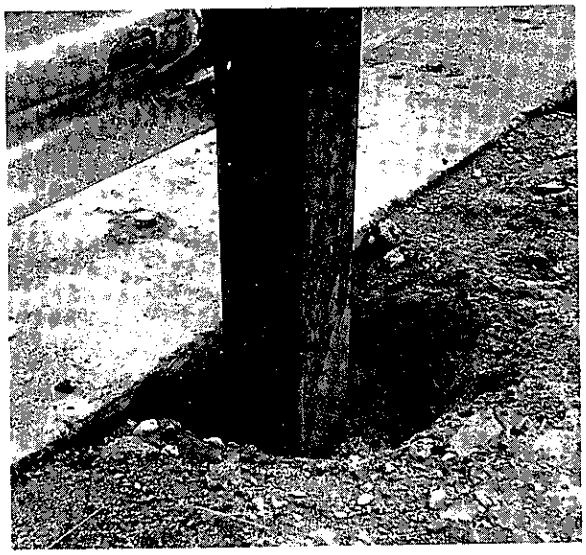

FIGURE 1

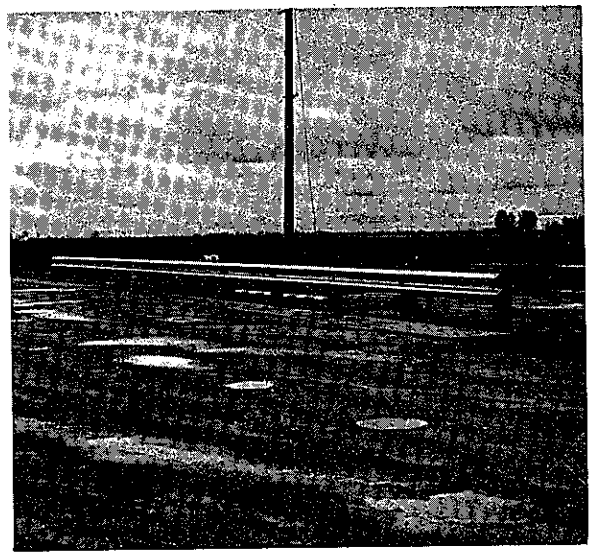

FIGURE 2

In the 1959 tests, a 55 mph/30 degree impact into this bridge railing with a 4,000 lb. passenger vehicle produced severe wheel/post entrapment and excessive rail deflections. (See Figures 3 and 4). Although this design was not judged adequate for freeway use, it was considered suitable 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 the secondary highways, failures began occurring even at low oblique impact angles. These failures were attributed

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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.

FIGURE 3

FIGURE 4

In an effort to correct this deficiency, the single W-section beam was replaced with two 3½-in. square structural steel tubular rails in 1967. This provided a post and rail system that conformed 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, their adequacy as applied to the Type 15 bridge rail, with the posts attached to the edge of the bridge deck, had not been evaluated as 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 in 1967[2]. Although somewhat similar overall appearance, the details of the New York barrier and the Type 15 barrier varied significantly, and it was felt that no analogy could be made between the two. Therefore, in January 1970 a research proposal for the "Development and Evaluation of Bridge Approach Guard Railing and Metal Tube Bridge Railing (Type 15)" was submitted to the Federal Highway Administration and subsequently approved for inclusion in the 1969-70 Fiscal Year work program.

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II. OBJECTIVES

The primary objectives of this research project were to:

1. Test the ability of the 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 degrees.

2. Determine the structural capabilities of the Type 15 bridge approach guardrail and its connection to the bridge abutment wing wall.

3. Develop and test subsequent design modifications to the systems as dictated by the results of the initial tests.

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III. CONCLUSIONS

The following conclusions are based on an analysis of the results of the full scale vehicle impact tests reported herein unless otherwise noted:

1. The initial Type 15 bridge barrier rail design impacted in Test No. 251 will retain and redirect a 4,500 lb. passenger car impacting at a velocity of 60 mph and an approach angle of 15 degrees. Barrier damage can be expected to be minor and vehicle damage moderate. However, due to the rigidity of this design (6-ft. 3-in. post spacing), 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 Type 15 bridge barrier rail design impacted in Test No. 252 will retain and redirect a 4,500 lb. passenger car impacting at a velocity of 60 mph and an approach angle of 15 degrees. Moderate vehicle damage and tolerable passenger compartment deceleration rates will be experienced. However, barrier damage, particularly at the post-to-deck connection, will be severe, 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 must be considered marginal.

3. A bridge rail 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 60 mph and 15 degrees.

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 up to 60 mph/15 degrees. This conclusion is based not only on the results of the tests reported herein but also on the results of a series of tests of the structurally similar California Type 8 BAGR (Test No. 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 herein, can be applied to the design of the approach rail-wing wall connection, thus eliminating the need to construct and test this appurtenance.

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IV. TEST CONDITIONS

A. Barrier Design

The Type 15 bridge barrier rail design was developed by the California Division of Highways Bridge Department and submitted to the Materials and Research Department for testing. The test installation, consisting of 67-ft. of Type 15 bridge barrier rail and 52-ft. of Type 15 bridge approach guard rail (BAGR), was constructed at the Materials and Research Department test facility at the Lincoln, California, airport (Figure 5, below).

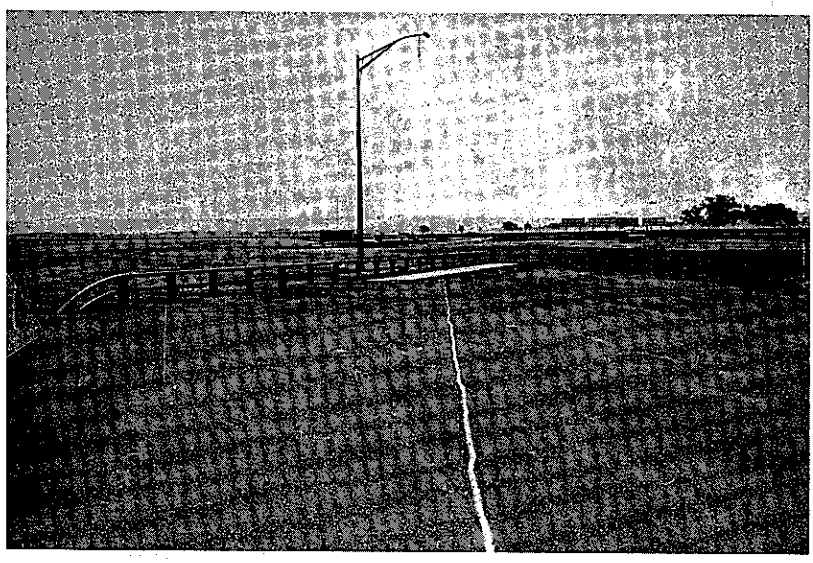

FIGURE 5

The Type 15 design consists of two structural steel tubular rails mounted at 14 and 27-in. above the pavement on steel WF posts spaced at 6-ft. 3-in. on centers (o.c.) On the bridge rail portion of the installation, the WF posts were bolted to the edge of a cantilevered reinforced concrete bridge deck (Figure 6). The BAGR WF posts were embedded in 3-ft. deep, 2-ft. diameter concrete footings. The posts for both the bridge rail and the BAGR were 6 WF 25 structural steel conforming to the requirements of ASTM Designation A-36.

Each bridge rail post was attached to the edge of the bridge deck with two 1-in. diameter by 2-ft. long high strength threaded rods and two 5/8-in. diameter by 1-ft. long high

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strength bolts cast into the 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, 10.50-lb. structural steel tubing conforming to the requirements of ASTM Designation A-500, grade B. The interior sleeve type rail splice (Figure 7, below) and the 3/4-in. welded stud rail-to-post connectors proven effective in a previous test series[4] were again employed.

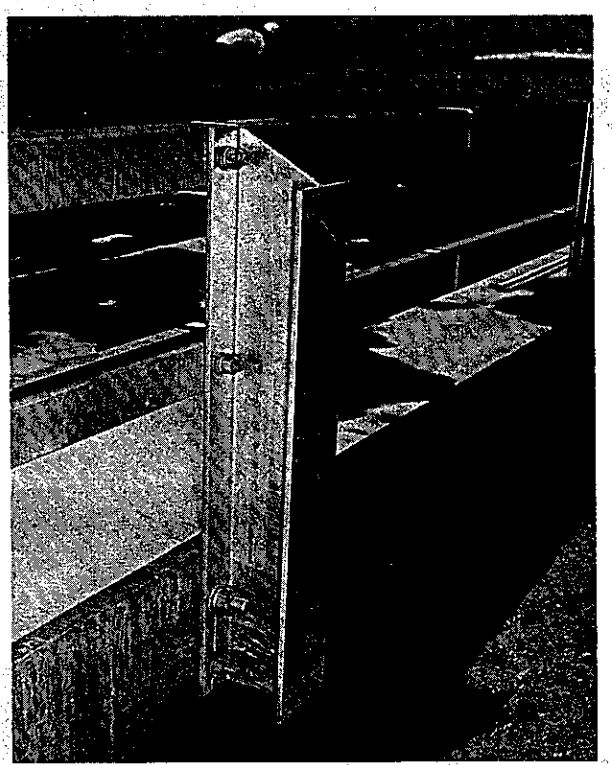

FIGURE 6

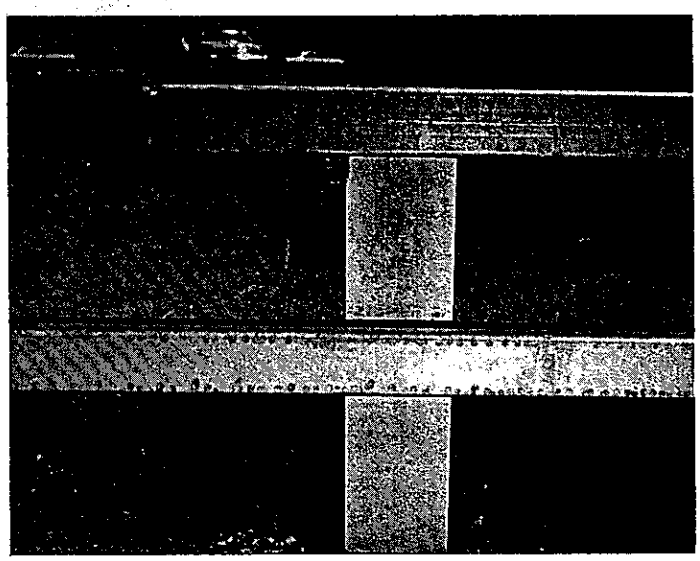

FIGURE 7

The bridge barrier rail was bolted to the outside edge of a 12-in. thick, 67-ft. long reinforced concrete bridge deck cantilevered 36-in. off a 24 by 30-in. by 68-ft. reinforced concrete anchor block. A six sack mix was used for the concrete. The 28-day compressive strength of the concrete was 4735 psi.

The posts for the bridge approach guard railing were set in seven 24-in. diameter, 36-in. deep concrete footings cast

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using a 5 sack mix. The leading end of the tubular rails were curved down and anchored to two 18-in. diameter, 36-in. deep reinforced concrete footings cast using a 6 sack mix. This concrete conformed to the minimum average compressive strength requirements of ASTM Designation C-94-68. The bridge deck reinforcing, as well as the other details of the Type 15 bridge barrier rail design, are shown on Exhibits 2 and 3 in the Appendix. Details of the Type 15 BAGR are shown on Exhibit 4 in the Appendix.

The Type 15 bridge barrier rail design, other than the post-to-deck connection, was designed in accordance with the requirements of the "Standard Specifications for Highway Bridges", adopted by the American Association of State Highway Officials in 1969.

B. Test Parameters

The test guidelines established by the Highway Research Board Committee on Guardrails and Guideposts[5] specify the use of a 4000-lb.± vehicle, an impact velocity of 60 mph, and an impact angle of 25 degrees. For the tests reported herein, the vehicle weighed 4550 lb. including an anthropometric dummy and on-board instrumentation. Although this weight exceeds the 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 was 60 mph at 15 degrees. 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 determined that under these conditions 60 mph/15° was the most probable maximum impact velocity and angle that would occur.

C. Vehicle Instrumentation

The test vehicles used in this study were 1968 Dodge sedans weighing approximately 4500 lbs., including an anthropometric dummy and on-board instrumentation. These vehicles were retired California Highway Patrol sedans and were modified for remote radio control as described in Reference 6.

D. Photographic Coverage

All the tests were photographed with high speed (250-400 frames per second) Photosonic cameras. These cameras were located on tripods to the front, rear, and side of the point of impact, and on a tower directly above the point of impact and were manually actuated from a central control console (see Exhibit 5, Appendix).

Another Photosonic camera was located in the rear of the test vehicle to film dummy kinematics. This camera was turned on

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when a switch, mounted on the rear bumper of the test vehicle, was actuated by the withdrawal of a pin attached to a 50-ft. length of nylon line anchored to the pavement behind the vehicle.

Most of the Photosonic data film had red-orange timing pips projected on it at a rate of 1000 per second. These pips were counted to determine the frame rates of the cameras. Targets were attached to the vehicle body and a target board bolted to the roof of the vehicle to facilitate data reduction of the film using a Vanguard Motion Analyzer.

Documentary coverage consisted of normal speed motion picture coverage, still photographs, and slides taken before, during, and after each test. A scaffold mounted 70mm Hulcher camera with a speed of 20 frames per second was also used for documentary coverage of the impact. Five tape switches, placed perpendicular to the vehicle path at ten-foot intervals leading into the point of impact, were actuated by the tires of the test vehicle and triggered a series of flashbulbs located in view of most of the data cameras. These flashbulbs were used for correlation between all the stationary cameras and for the determination of the impact velocity.

Flashbulbs mounted on top of the rear fenders of the test vehicle were used to establish the vehicle location and the time at which the brakes were applied. The bulbs also served to alert the control car driver that the test car's brakes had been applied. These flashbulbs were fired when the brake actuating relay was closed by either radio equipment failure or the remote operator.

E. Data Acquisition and Processing

Eight Statham accelerometers, of the unbonded strain gage type, were used to record impact decelerations. Four were installed in the chest and head cavity of the anthropometric dummy driver, and four were mounted on the test vehicle floorboard in the passenger compartment. In addition, one seat belt transducer was installed on the dummy's lap belt. Transducer information was transmitted through a Belden #877 umbilical cable from the crash vehicle to a 14 channel Hewlett Packard 3924 magnetic tape system See Exhibit 6, Appendix, for the location and description of the instrumentation. Two pressure activated tape switches were installed in the test vehicle's approach path. One was located parallel to the barrier such that the crash vehicle would activate it at the instant of vehicular impact with the barrier. The other was positioned adjacent to one of the flashbulb activating tape switches 20-ft. before impact. Vehicle tire contact activated the tape switches. This caused an "event marker" signal to be recorded along with the accelerometer data on the tape recorder. Concurrently, a 100 millisecond time cycle was recorded on the tape recorder. This data was played back through a Visicorder which produced an oscillographic trace (line) on paper. Each paper record contained an accelerometer

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data trace, the front and rear vehicle wheel event marker trace, and the 100 millisecond time cycle trace.

For Test 251, the tape recorder playback speed for the Visicorder was reduced from the recording speed by a ratio of 8 to 1. Theoretically, this produced an "unfiltered" trace with a frequency response of 2160 Hertz. A "filtered" trace was also obtained from the Visicorder using the tape recorder playback speed reduction of 8 to 1 and a Krohn-Hite filter for an effective filtering of the impact signal of 100 Hertz.

The filtered traces were easier to compare and to use for data reduction than the unfiltered traces. They also gave a better over-all record of the motion of the dummy and vehicle. The high frequency spikes on the unfiltered records were assumed to be relatively insignificant as related to the over-all motion of the vehicle.

After Test 252 there was a malfunction of the Krohn-Hite filter so the method of Visicorder playback was changed. The magnetic tape playback speed reduction was kept at an 8 to 1 ratio, but a Brush orange dot galvanometer with a frequency response of 100 Hertz was used to obtain an "unfiltered" record of the impact signal resulting in an effective response of 800 Hertz. A Brush brown dot galvanometer with a frequency response of 22 Hertz was used to obtain an effective response of 176 Hertz on the "filtered" records.

Copies of the filtered records of impact data for Tests 251 and 252 are contained in the Appendix, Plates A-1 through A-18. The vehicular deceleration data filtered at 176 Hertz in Test 252 proved to be somewhat unwieldy for numerical work so a "hand filtered" line was drawn (as shown in the Appendix) in order to eliminate high frequency spikes and to compute the maximum deceleration values reported herein.

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V. TEST RESULTS

A. Introduction

The following pages contain descriptions of the two full scale tests conducted in this study. These descriptions are based on analysis of high speed data film, interpretation of deceleration instrumentation recordings, physical evidence examined at the test site, and laboratory evaluation of damaged barrier and vehicle components.

B. Test No. 251

Test No. 251 was conducted to test the ability of the initial Type 15 bridge barrier rail design, with 6-ft. 3-in. post spacing, to redirect a passenger vehicle impacting at a moderate velocity and approach angle (Figures 8 and 9, below).

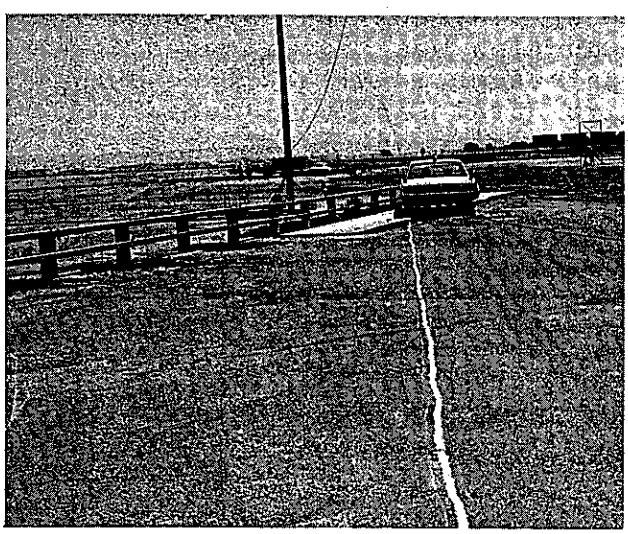

FIGURE 8

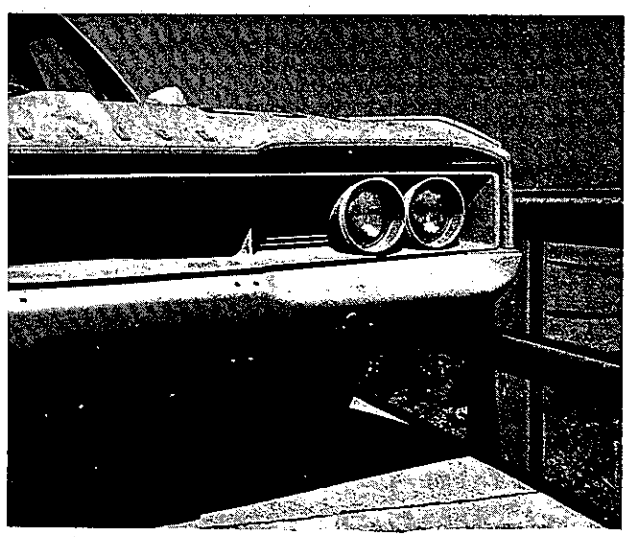

FIGURE 9

Initial barrier contact occurred at midspan between posts B-4 and B-5, approximately 20-ft. downstream of the BAGR to bridge rail connection. The impact velocity and approach angle were 64 mph and 12 degrees. 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 as the lower rail effectively deflected the left front wheel, thus averting any serious vehicle/barrier entrapment. There was little tendency for the vehicle to jump and only a 5 degree roll toward the

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barrier (Figure 10, below) as the vehicle was effectively redirected to an exit angle of 3 degrees with the barrier.

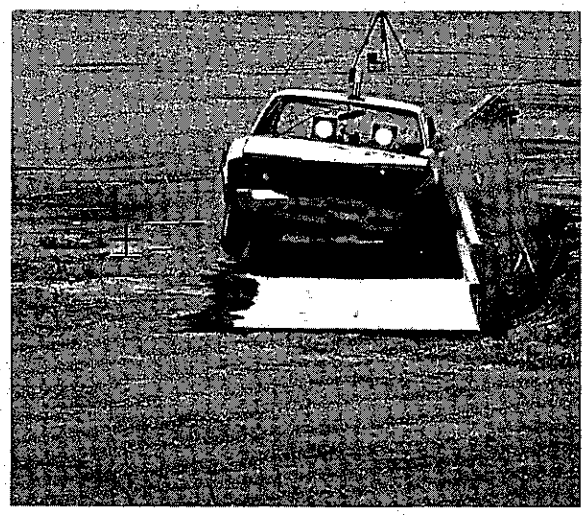

FIGURE 10

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

Barrier damage was relatively minor. Two rail sections and three posts were deformed and would have required replacement for aesthetics. However, all the barrier components were structurally intact 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 deformation of the upper and lower rails were 0.21-ft. and 0.14-ft., respectively (Figures 11 and 12, below).

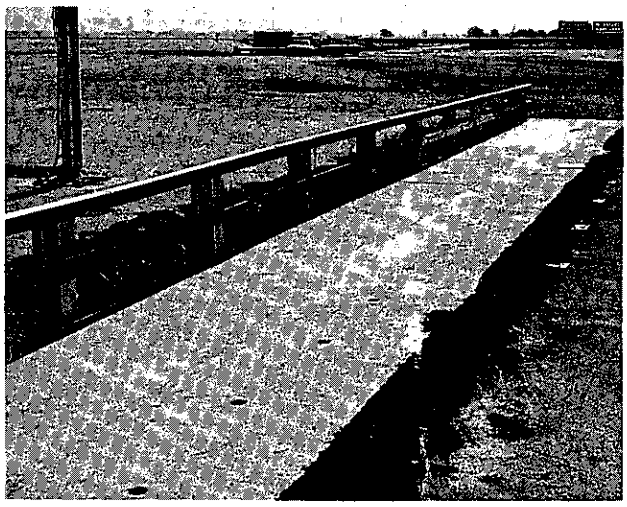

FIGURE 11

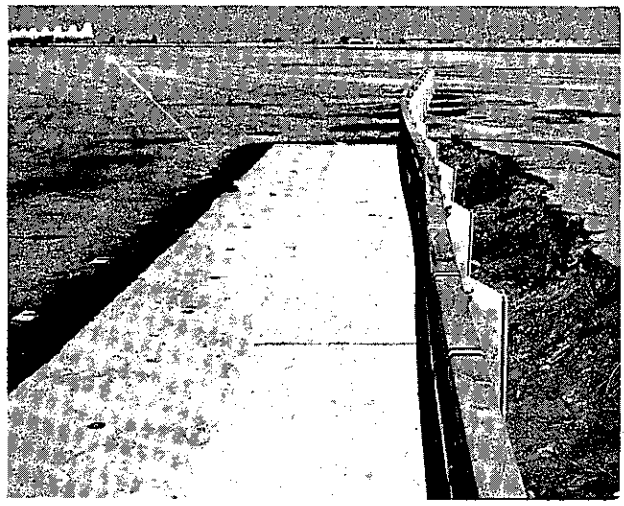

FIGURE 12

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All three damaged posts sustained flange deformation above the upper post-to-deck connections. Maximum residual lateral post deflections, measured from the upper edge of the simulated bridge deck, were (1) post B-4, 3.0-ft. upstream of impact, 1/4-in., (2) post B-5, 3.2-ft. downstream of impact, 1/2-in., and (3) post B-6, 9.4-ft. downstream of impact, 3/8-in. (Figures 13, 14, and 15, below).

FIGURE 13

FIGURE 14

FIGURE 15

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 left front tire, the vehicle appeared to be operable (Figures 16 and 17).

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

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high deceleration forces (Figure 18). However, deceleration recording instrumentation indicated that the deceleration forces, particularly in the lateral direction, were higher

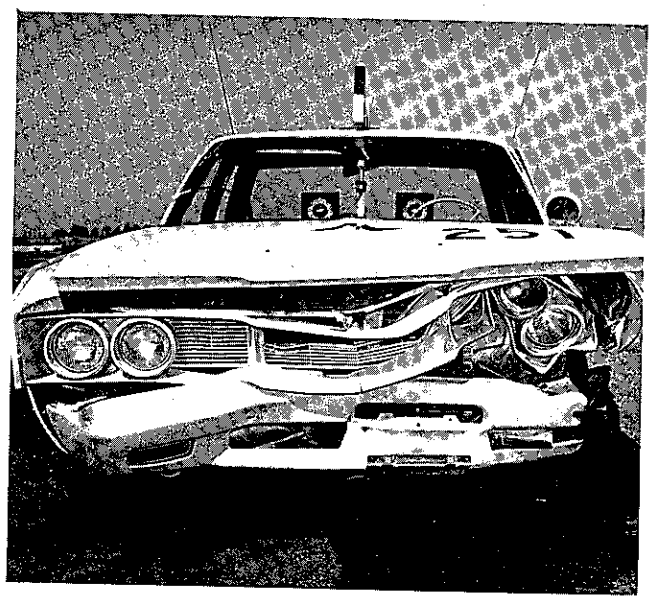

FIGURE 16

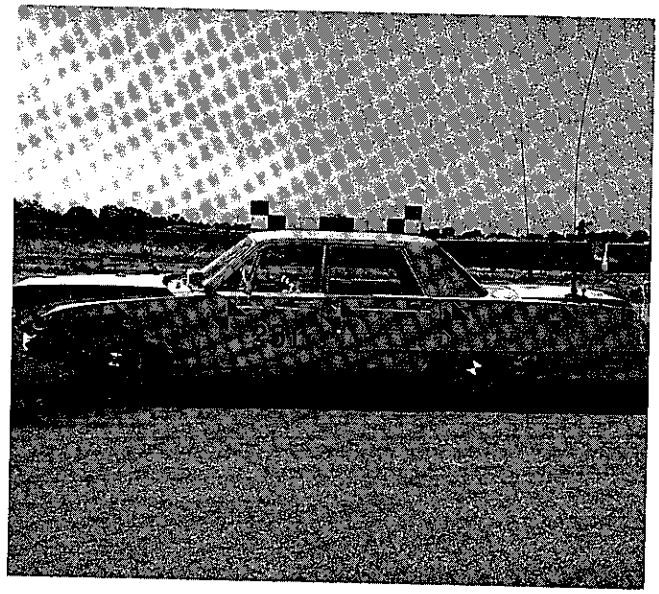

FIGURE 17

than anticipated or fully desired. Records of instrumentation data are contained in the Appendix, Plates A-1 through A-9. A summary of this data is presented as follows:

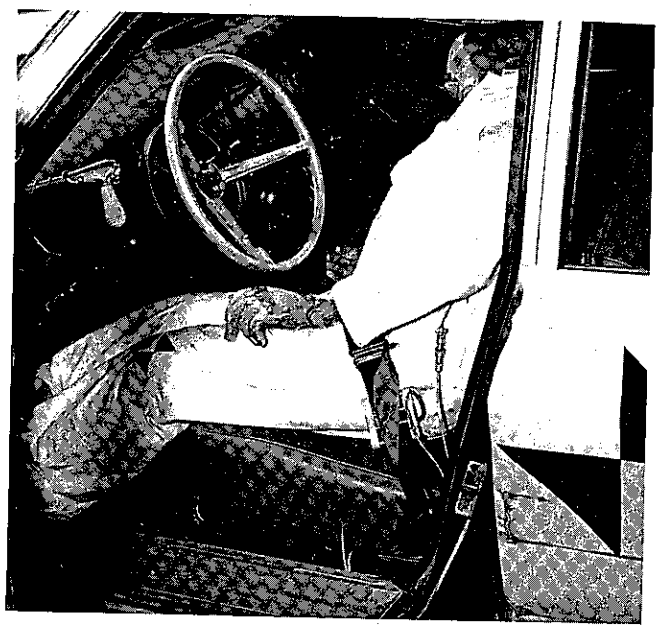

FIGURE 18

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Accelerometer Location and Orientation	Highest 50 Millisecond Average Value of Deceleration in G's
1. Vehicle - longitudinal motion (Avg. of two accelerometers)	4.7
2. Vehicle - lateral motion (Avg. of two accelerometers)	9.0
3. Dummy - head (resultant of long., lat., and vert. accelerometers)	25.0
4. Dummy - chest - longitudinal motion (one accelerometer)	4.6

The maximum seat belt load was 1350 lbs. The Gadd Severity Index was 278. See the Discussion, Section V, for an evaluation of these results and the following page for additional test data, information, and photographs.

C. Test No. 252

Analysis of the results of Test No. 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 with the intent of lessening the severity of impact. To achieve this modification, seven posts were removed and 2-ft. square sections of the cantilevered bridge deck were removed at three locations. Care was taken not to damage any of the existing deck reinforcing during the chipping operations. New post anchor bolts were installed at these locations, the deck edges within the removed sections were coated with epoxy, and new concrete cast using a 6 sack mix. The 28-day compressive strength of the concrete was 4540 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 (Figure 19). However, this discontinuity was far enough from the location of impact to have no effect on the test results. The height of the upper and lower rails was identical to that tested in Test No. 251.

This test (Test No. 252) was conducted to determine the ability of this modified Type 15 bridge barrier rail to redirect a passenger vehicle impacting at a moderate velocity and approach angle.

Initial barrier contact occurred 2.7-ft. upstream of post B-5 (approximately 35-ft. downstream of the BAGR to bridge rail transition) at 59 mph/14-degrees. Vehicle/barrier interaction was similar to that observed in the first test (Test No. 251). However, vehicle tire scrub marks on the bridge deck indicated that the left front wheel came close to projecting beyond the edge of the bridge deck. Had this occurred, serious

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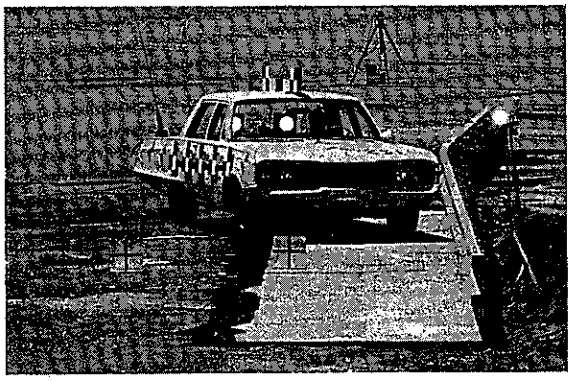
Plate 1



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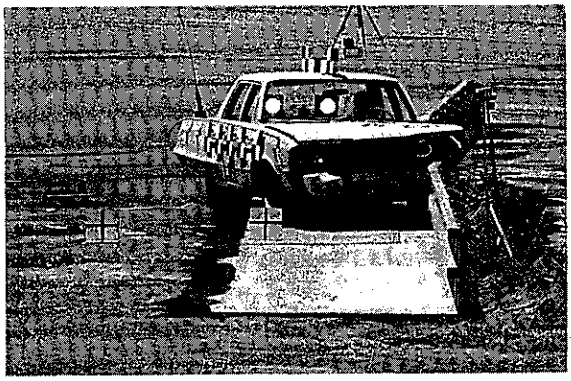

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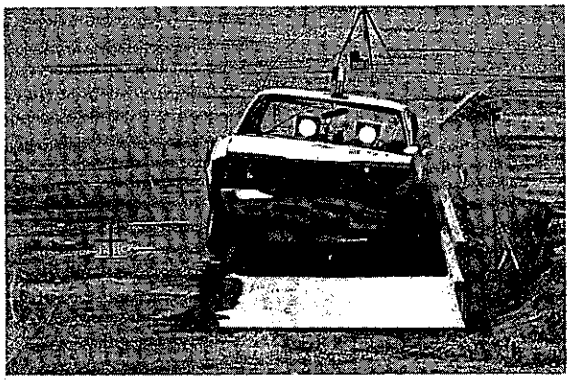

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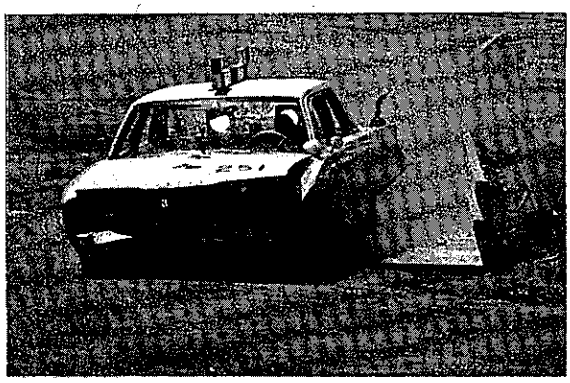

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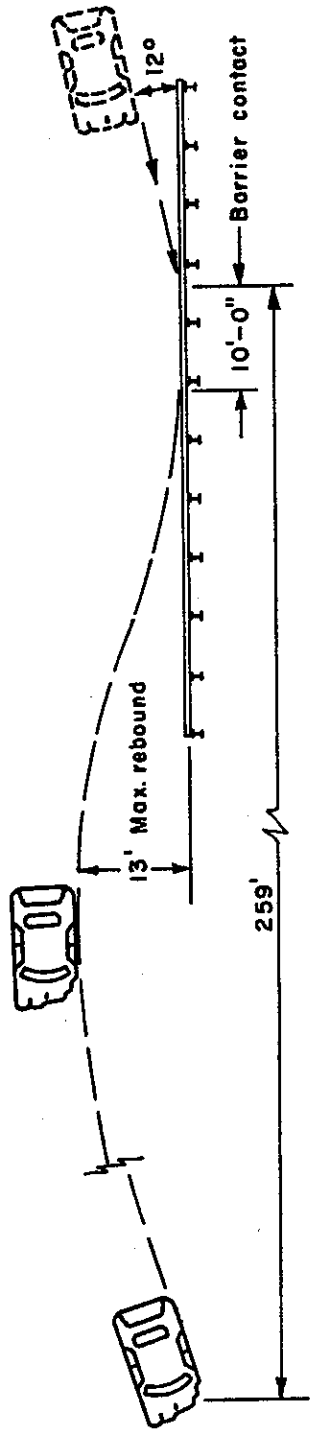

TEST NO.	251
DATE	7-7-70
VEHICLE	1968 Dodge Sedan
SPEED	64 mph
IMPACT ANGLE	12°
VEHICLE WEIGHT W/DUMMY & INSTRUMENTATION	4550 lbs

BARRIER TESTED	Type 15 Bridge Rail
POST SPACING	6'-3"
LENGTH OF INSTALLATION	56'-3"
MAX. PERM. RAIL DEFL. (LATERAL)	0.21'
VEHICLE/BARRIER CONTACT	10'
MAX REBOUND	13'
EXIT ANGLE	3°

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post/wheel entrapment could have resulted. Again, vehicle dynamics through impact were good with no tendency to jump.

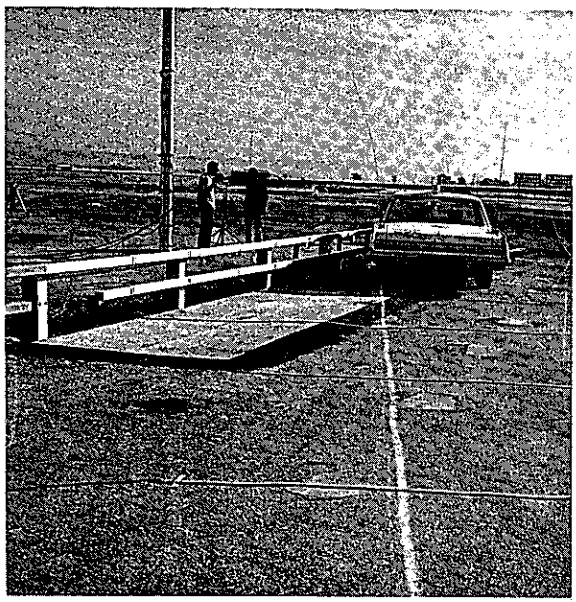

FIGURE 19

FIGURE 20

A 7 degree roll toward the barrier occurred (Figure 21, below) as the vehicle was effectively redirected to an exit angle of 2-degrees with the barrier. The total vehicle/barrier contact was approximately 14-ft.

FIGURE 21

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The maximum vehicle rebound into the traveled lanes was 22-ft. While this rebound is greater than desired, it is not excessive; and in view of the low 2-degree exit angle, over-all post impact vehicle trajectory was considered satisfactory.

The barrier damage was more severe than that observed in the first test (Test No. 251). Two rail sections and two posts were deformed and would have required replacement. Although all 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 the bottom rail 0.43-ft. (Figures 22 and 23, below). Maximum residual lateral post deflections, measured

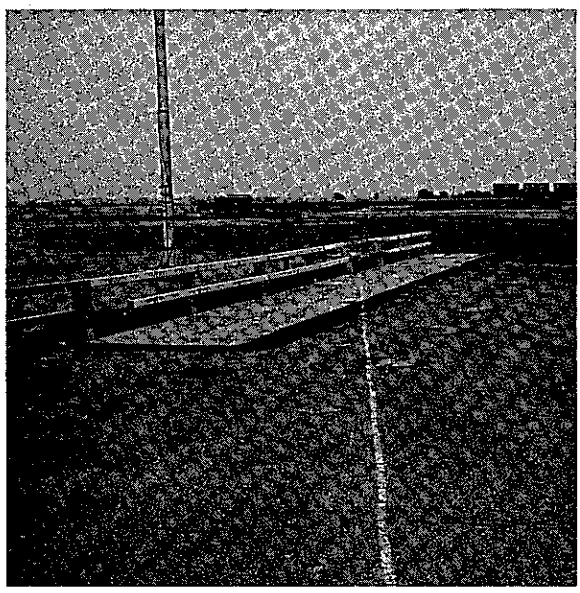

FIGURE 22

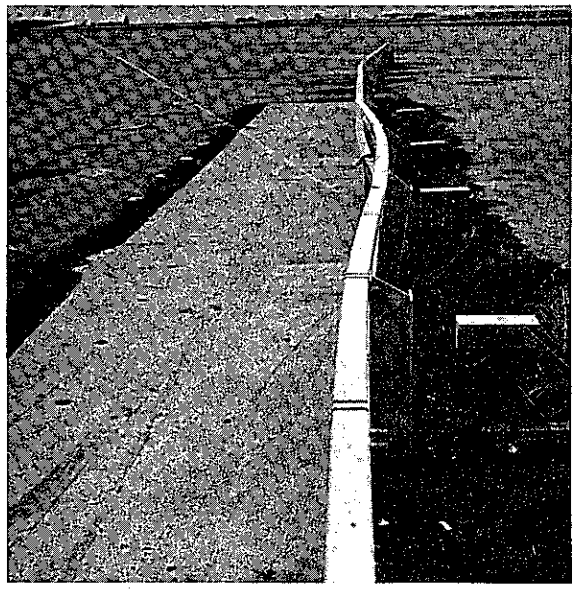

FIGURE 23

from the upper edge of the simulated bridge deck were: (1) post B-5, 2.7-ft., downstream of impact, 1-7/8 in. and (2) post B-6, 17.1-ft. downstream of impact, 1-7/8 in. Although the post deflections are numerically comparable, indicating similar loadings, at post B-6 the downstream upper post-to-deck connector (1-in. diameter high strength threaded rod) failed in tension and, consequently, post flange deformation was absent at that point. Minor post

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flange deformation did occur on that side of the post just above the lower connector (Figure 24, below). On the upstream side, post flange deformation occurred above the upper post-to-deck connector which remained intact (Figure 25, below).

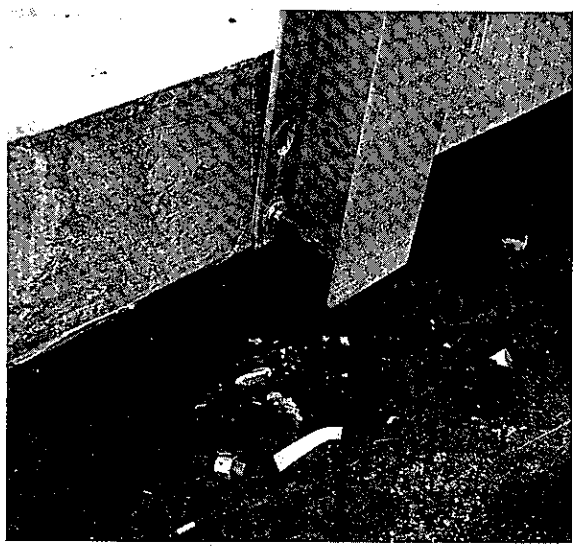

FIGURE 24

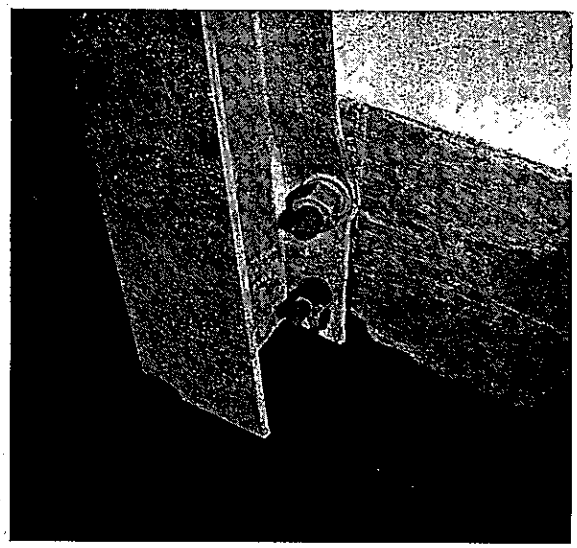

FIGURE 25

At post B-5, all post-to-deck connectors were intact and both post flanges deformed above the upper connectors (Figure 26, below). However, a flange/web fracture (0.1-in. by 3-in. separation) occurred adjacent to the downstream upper post-to-deck connector (Figure 27, below). There was no damage sustained by any of the rail stud bolts or splice sleeves.

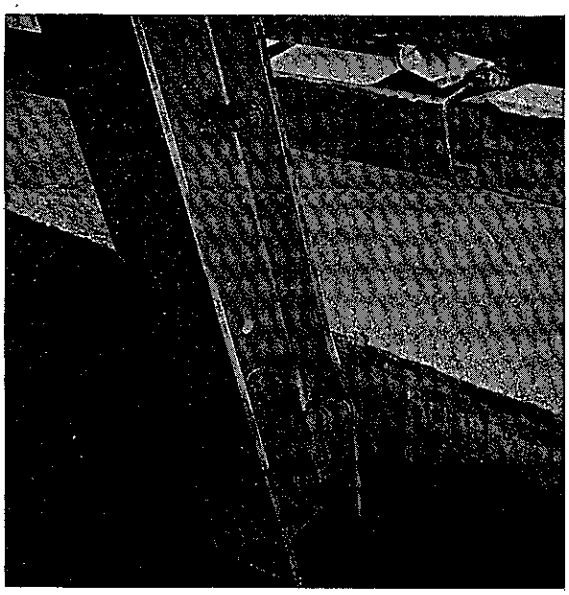

FIGURE 26

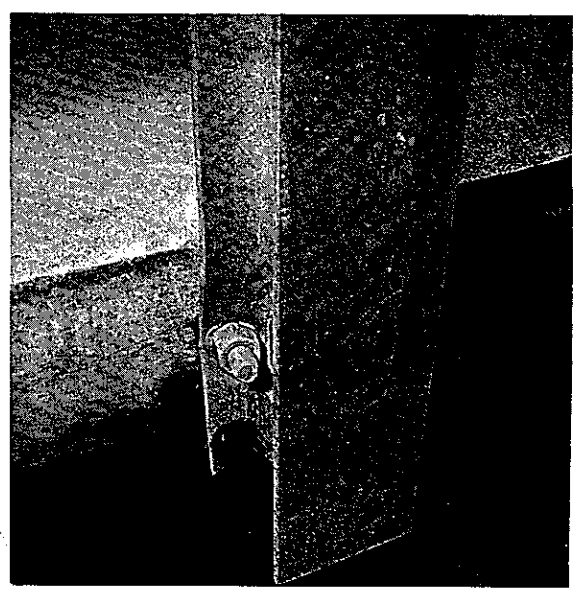

FIGURE 27

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Concrete damage was limited to minor spalling at the lower anchor bolts and on the underside of the bridge deck at post B-6 (Figures 24 and 25). 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 possible, due to the vertical bolt spacing requirement of the posts, to install the new lower post-to-deck connectors above the existing lower longitudinal deck reinforcing steel as specified on the plans. Had this 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 No. 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 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 also 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 test.

Vehicle damage was generally similar to that observed in the first test (Test No. 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 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 No. 251. This indicated that a harder rear end slap occurred as the vehicle was being redirected (Figures 28 and 29). Data film analysis revealed that this was due to the larger rail deflections of this test. These large deflections permitted the vehicle to pocket into the barrier somewhat and follow the deflecting rails rather than rebound,

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or "bounce", off the barrier as observed in the first test (Test No. 251) on the more rigid initial design tested. Although vehicle body damage was essentially superficial, the damaged left front wheel rendered the vehicle inoperable.

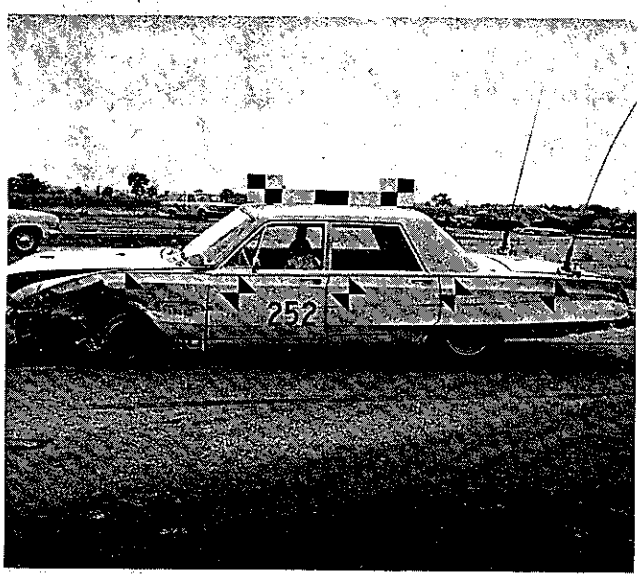

FIGURE 28

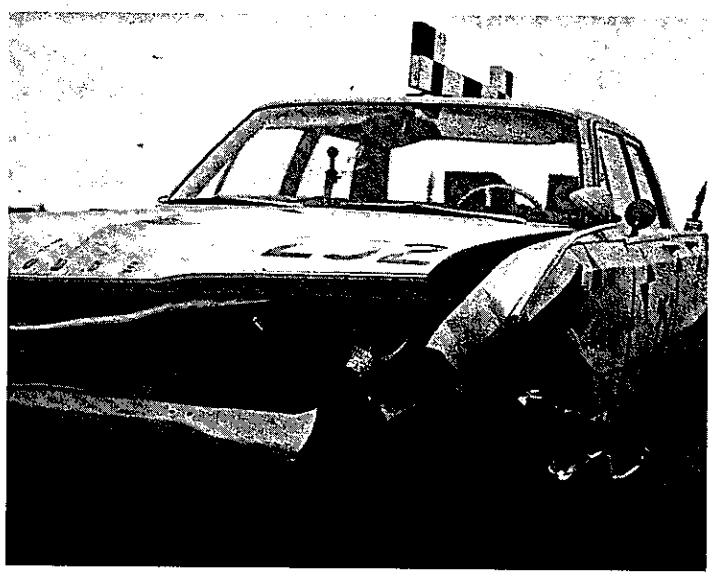

FIGURE 29

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 the first test (Test No. 251). See the Appendix, Plates A-10 through A-18 for the records of instrumentation data. A summary of this data is given below:

Accelerometer Location and Orientation	Highest 50 Millisecond Average Value of Deceleration in G's
1. Vehicle - longitudinal motion (Avg. of two accelerometers)	3.1
2. Vehicle - lateral motion (Avg. of two accelerometers)	3.9
3. Dummy - head (resultant of long., lat., and vert. accelerometers)	24.0
4. Dummy - chest - longitudinal motion (one accelerometer)	4.4

The maximum seat belt load was 120 lbs. and the Gadd Severity Index was 234. See Discussion, Section V, for an evaluation of these results.

See the following page for additional test data, information, and photographs.

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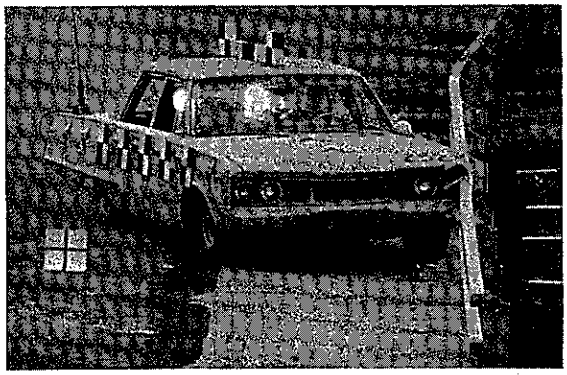
Plate 2



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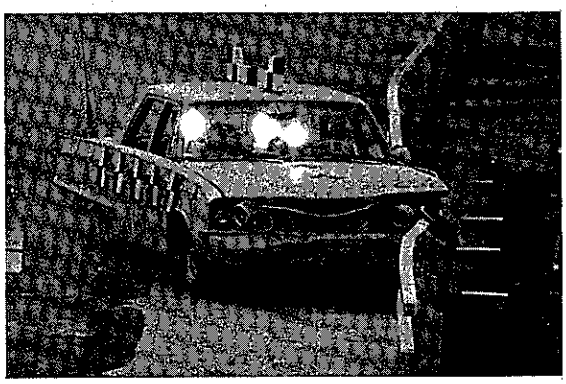

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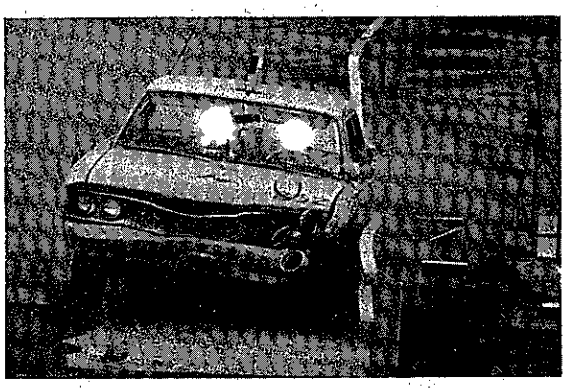

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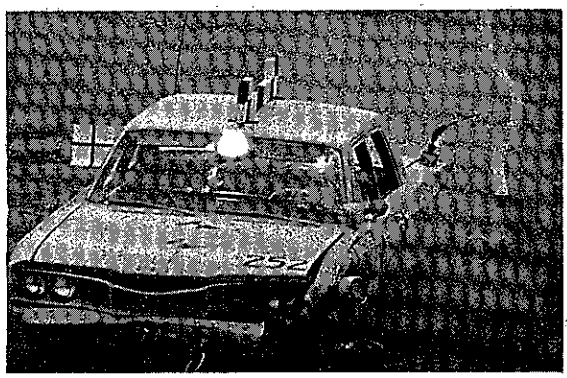

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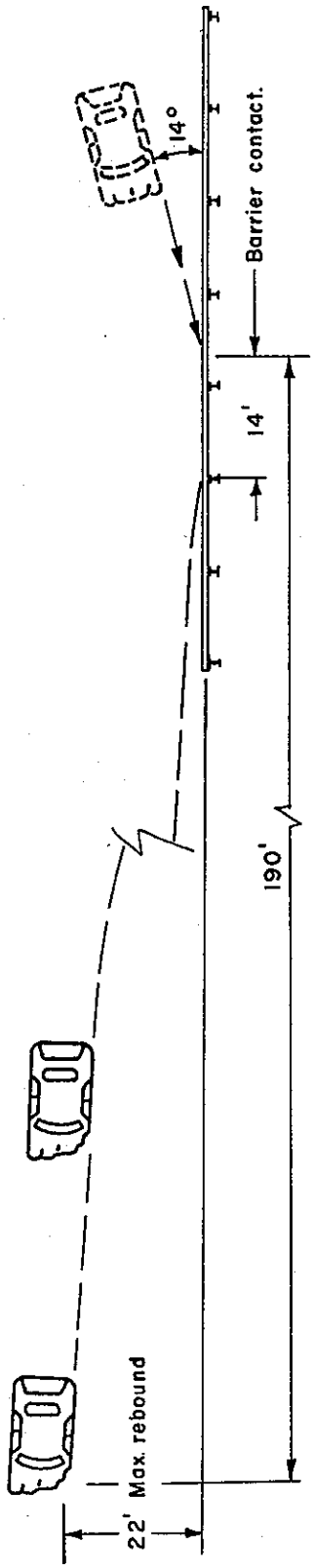

TEST NO.	252
DATE	10-30-70
VEHICLE	1968 Dodge Sedan
SPEED	59 mph
IMPACT ANGLE	14°
VEHICLE WEIGHT	4550 lbs
W/DUMMY & INSTRUMENTATION	

BARRIER TESTED	Type 15 Bridge Rail
POST SPACING	9'-4 1/2"
LENGTH OF INSTALLATION	65'-3"
MAX. PERM. RAIL DEFL. (LATERAL)	0.56'
VEHICLE/BARRIER CONTACT	14'
MAX REBOUND	22'
EXIT ANGLE	2°

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D. Discussion

General

The initial Type 15 bridge barrier rail design, impacted in Test No. 251, appeared to be effective in redirecting a passenger vehicle impacting at a moderate velocity and angle. Vehicular redirection was smooth, barrier damage minor, and vehicle damage moderate. However, it was also apparent, from the post impact vehicle trajectory and the low residual barrier deflections, that the rigidity of the system exceeded that required. It was decided that increasing the post spacing from 6-ft. 3-in. to 9-ft. 4½-in. would be desirable. This would result in lower decelerations sustained by the vehicle passenger compartment, due to the increased barrier flexibility, as well as an economic saving through the 33% decrease in the number of barrier posts. The 9-ft. 4½-in. post spacing was arbitrarily selected as an economic expedient as 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 No. 252, conducted on this modified system, substantiated the desirability of increased barrier flexibility. However, barrier damage, particularly at the post-to-deck connection, was such that the 9-ft. 4½-in. post spacing is considered marginal. Thus a post spacing of 8-ft. 0-in. has been determined to be the optimum to produce both 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 deceleration rates.

Neither the bridge approach flare or the approach flare wing wall were impact tested in this study. Although 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 herein 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 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 and the Type 8 rail element is a 6- by 2-in., 12.02-lb. structural steel tube conforming to the requirements of ASTM Designation A-500, grade B, whereas the Type 15 rail element is a 3½-in. square, 10.50-lb. structural steel tube conforming to the requirements of ASTM Designation A-500, grades A or B, or A-501. The section modulus of the Type 15 rail is approximately 70% that of the Type 8 rail. However, the 6-ft. 3-in. post spacing of the Type 15 system is approximately 63% that of the Type 8. Therefore, the forces required to exceed the ultimate strength of the Type 15 and Type 8 rail elements are

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reasonably comparable ($F_{15} = 0.86F_8$). Increasing the post spacing of the Type 15 BAGR from 6-ft. 3-in. to 8-ft. 0-in. will decrease this ratio to 0.67. However, the lateral kinetic energy imparted to the barrier during a 15 degree impact is only 37% of that imparted to the barrier at the 25 degree 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 on 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.

The only 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 tube rail was too great; thus tube alignment at the splices was not as close as 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 that the dimensional tolerance for the tube slot was such that when effecting repairs, the splice sleeves were not readily interchangeable, particularly when reusing a tube bent from the previous impact. This, however, could easily be remedied by increasing the tube slot from 7/16-in. to ½-in. This should provide the needed tolerance for interchangeability.

Also, some method of sliding the splice sleeve, other than hammering on the bolt head, should be devised. 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 should 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.

Other than the aforementioned items, barrier construction and collision repairs should be relatively easy and economical.

Interpretation of Instrumentation Data

The severity of the 50 ms vehicular decelerations reported herein was determined by comparing the deceleration magnitudes with the recommended 200 ms deceleration tolerance limits proposed by Cornell, Reference 8 (see Table 1). 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

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unless otherwise noted). Reference 6 contains a discussion of deceleration tolerances and the reasoning behind the choice of these values. These limits define what would be, in the opinion of the researchers, a survivable environment under almost all circumstances when applied to the 50 ms time period.

TABLE I

Vehicular Decelerations

Orientation of Accelerometer in Vehicle	Decelerations (G's) Highest 50 ms. avg. Test 251	Test 252	Cornell Study Avg. G's - 200 ms Unrestrained	Lap Belt	Lap Belt & Shoulder Harness
1. Lateral (Avg. of 2 accelerometers)	9.0	3.9	3	5	15
2. Longitudinal (Avg. of 2 accelerometers)	4.7	3.1	5	10	25

Filtered records of vehicular deceleration (reproduced in the Appendix, Plates A-1 to A-4 and A-10 to A-13) were used to compute the highest 50 millisecond (ms) average values (average of ten continuous 5 ms intervals).

The dummy used in Tests 251 and 252 was restrained with a conventional lap belt. Referring to the table above, only the vehicular lateral deceleration in Test 251 (9 G's) exceeded the recommended value for passengers restrained with lap belts. 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

$$\left(\int_{t_1}^{t_2} a^{2.5} \, dt \right)$$

was computed over the 50 millisecond period with the highest average resultant values of head deceleration, using 20 time intervals, i.e., dt = 0.0025 seconds (see Reference 9 for a discussion of the Gadd Severity Index and the tolerance of the human head to deceleration). 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

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of fatal head injuries is 1000 assuming no penetration of the skull occurs; 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 measured seat belt load was 1350 lbs. for Test 251 and 120 lbs. for Test 252 (see Plates A-9 and A-18). These are not excessive values, as expected, because of the proportion of the dummy's motion that was in the lateral direction and, as such, relatively independent of the lap belt. 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 longitudinal dummy chest decelerations, which were almost identical for both tests. It is possible that there was a malfunction in the instrumentation for one or both tests which caused the wide variation in recorded seat belt loads.

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

The above results also indicate that the bridge rail system used for Test 252 was slightly preferable to that of Test 251 from the injury potential standpoint due to 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 regards to barrier preference (dummy decelerations are tabulated in Section V, Test Results).

The vehicle accelerometer records show that for both tests vehicular backslap decelerations in the longitudinal direction were less than those recorded during the initial impact (Plates A-2, A-4, A-11, and A-13). However, in the lateral direction, the decelerations recorded during both the initial impact and the backslap were approximately equal (Plates A-1, A-3, A-10, and A-12).

Correlation with Math Model

The average lateral and longitudinal vehicular decelerations were calculated using the mathematical model described in Reference 11. In Test 251 an exit velocity, V_E, of 94 fps (same as impact velocity) and an estimated dynamic deflection, D, of 0.4 ft., when the vehicle was parallel to the bridge rail, were used in the calculations. In Test 252 an exit velocity, V_E, of 75.5 fps and a dynamic deflection, D, of 0.96 ft. were used. The above values were based on an analysis of film from the high speed cameras. Values of impact velocity, angle

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of impact, and other variables were as reported earlier in this paper. The following values of acceleration were obtained:

TABLE II

	Test 251	Test 252
G lat (average lateral vehicular deceleration in G's)	3.24	2.61
G long (average longitudinal vehicular deceleration in G's)	0	1.05
Δt (time interval over which the above average values occur in seconds)	0.187	0.249

For comparison, the average values of deceleration were computed using accelerometer records (Plates A-1 to A-4 and A-10 to A-13). The corresponding values of Δt were used for each test and each value of deceleration was based on the average of two filtered accelerometer records, one each at Location A and E in the vehicle. In the case of Test 252, which was filtered at 176 Hertz, the faired in curves of acceleration shown in the Appendix were used. It should be noted that these accelerations were all measured with fixed accelerometers referenced to the longitudinal axis of the vehicle whereas those calculated from the mathematical model were measured with respect to the barrier axis. The calculations from the accelerometer records were accomplished using time increments of 0.005 seconds. The following values were obtained:

TABLE III

	Test 251	Test 252
G lat	5.0	2.5
G long	2.2	1.6

The measured accelerations exceeded the predicted values by more than the 20% that was mentioned in NCHRP Report 86 with the exception of the average lateral deceleration in Test 252 which agrees quite well. Report 86 includes an estimate that the proportion of the unrestrained occupants who will receive injuries is one tenth the value, in G's, of the average lateral vehicular deceleration during angle impacts. Hence, based upon the decelerations presented in Tables II and III, above, 32 to 50% of the unrestrained occupants of a vehicle duplicating Test 251 and approximately 25% of those unrestrained occupants involved in a collision that duplicated Test 252 would receive injuries as per NCHRP Report 86.

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VI. IMPLEMENTATION

As a result of the tests reported herein, the post spacing will be changed from 6-ft. 3-in. to 8-ft. 0-in. on California Standard Plan B11-48 (Metal Tube Bridge Railing, Type 15). In addition, the tube splice modifications will be added on the Standard Plans for all bridge rails using this type tube splice. These design modifications will result in a more economical and effective bridge barrier rail for use on Federal Aid Secondary and certain California State Highways (Figure 30, below).

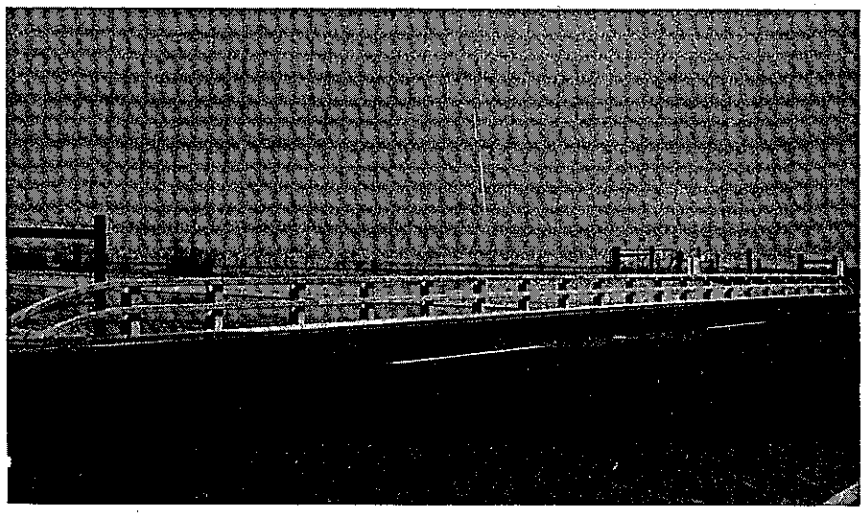

FIGURE 30

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2. Graham, M. D., et al, "New Highway Barriers, The Practical Application of Theoretical Design", State of New York, Department of Public Works, May 1967.

3. Nordlin, E. F., Ames, W. H., and Hackett, R. P., "Dynamic Tests of Type 9 Bridge Barrier Rail and Type 8 Bridge Approach Guardrail", California Division of Highways, June 1969.

4. Nordlin, E. F. and Field, R. N., "Dynamic Full Scale Impact Tests of Steel Bridge Barrier Rails, Series XI", California Division of Highways, June 1967.

5. Highway Research Board on Guardrails and Guide Posts, "Proposed Full-Scale Testing Procedures for Guardrails", Circular 482, September 1962.

6. Nordlin, E. F., Woodstrom, J. H., and Hackett, R. P., "Dynamic Tests of the California Type 20 Bridge Barrier Rail, Series XXIII", California Division of Highways, September 1970.

7. Field, R. N. and Prysock, R. H., "Dynamic Full Scale Impact Tests of Double Blocked-Out Metal Beam Barriers and Metal Beam Guardrail, Series X", California Division of Highways, February 1965.

8. "Highway Barrier Analysis and Test Program", Summary/Report for period July 1960 - July 1961, Cornell Aeronautical Laboratory Report No. VJ-1472-V-3, July 1969.

9. Nordlin, E. F., Stoker, J. R., and Doty, R. N., "Dynamic Tests of An Energy Absorbing Barrier Employing Sand-filled Frangible Plastic Barrels, Series XXIV", California Division of Highways, April 1971.

10. "10th Stapp Car Crash Conference - Proceedings", November 8-9, 1966; published by Society of Automotive Engineers; paper 660793 by Charles Gadd.

11. "Tentative Service Requirements for Bridge Rail Systems", National Cooperative Highway Research Program Report 86, Highway Research Board.

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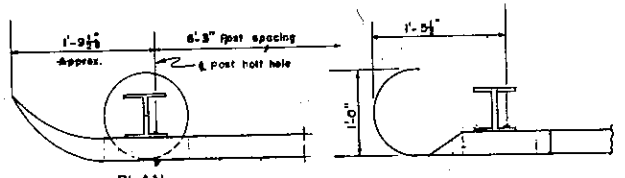
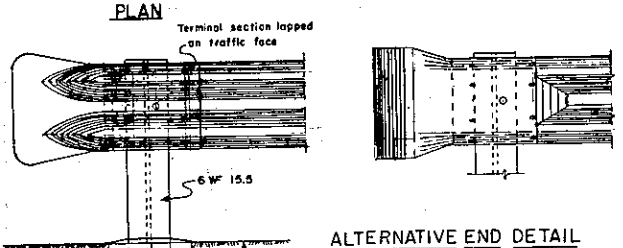


PLAN

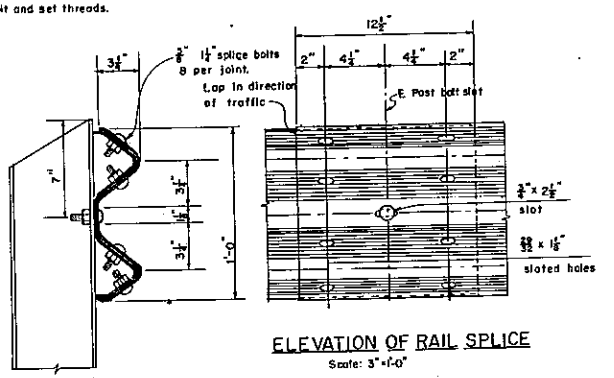


ALTERNATIVE END DETAIL

NOTE: Extend rail onto shoulder if structure does not accommodate 12'-6" unit length.

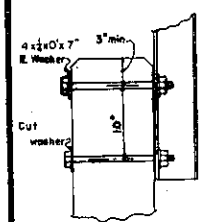
ELEVATION
END POST ON SHOULDER

NOTE: At rail splice nearest expansion joint in bridge deck, slot holes in movable section of rail to allow same amount of movement as is provided in bridge deck. Place ⅛" asbestos sheet packing between rail elements. Adjust bolts to provide sliding fit and set threads.

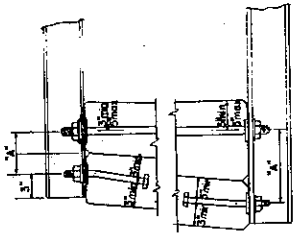


CROSS SECTION THROUGH RAIL SPLICE
Scale: 3"=1'-0"

ELEVATION OF RAIL SPLICE
Scale: 3"=1'-0"

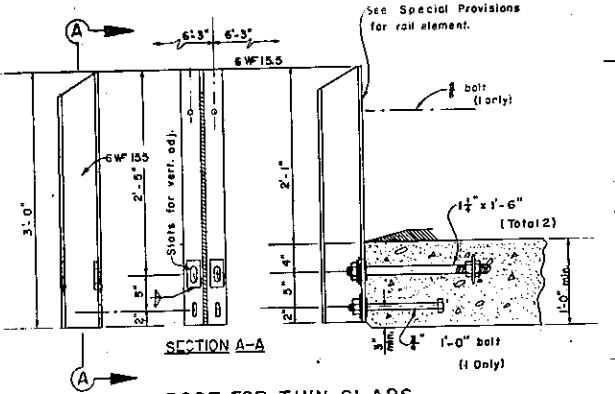


RAIL POST ON WALL SECTION



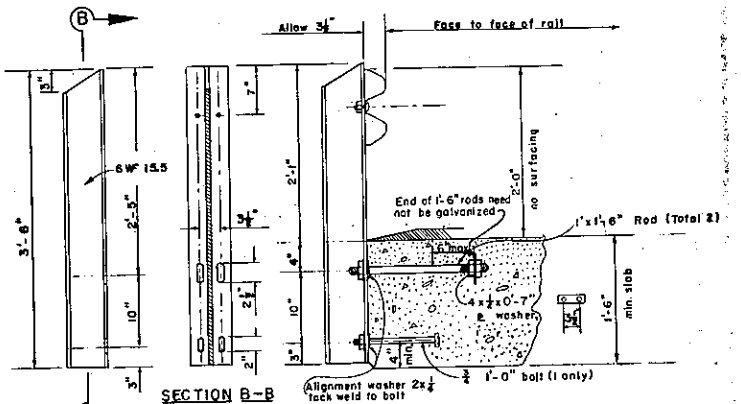
Dimension "A" will depend on cross slope of deck, height, width, and cross slope of curb. If possible use post shown for slabs with dimension "A" of 5" or 10". Otherwise a special post with a special dimension "A" will be required. Use 1'-3" long bolts in 1'-2" wide curbs.

RAIL POST ON CURBED SECTIONS



SECTION A-A

POST FOR THIN SLABS



SECTION B-B
POST FOR THICK SLABS

NOTES: Dimensions of all posts are similar except as noted. Use 3'-6" posts whenever possible. All anchor bolt to have hex nuts. Provide lock washers for top anchor bolts. Weld 4 x ¼ pads on WF posts when using ⅞ anchor bolts. Contractor to provide suitable metal shims to align posts. Space posts to clear expansion joints by 1'-0" min. to ℄ post.

All parts to be galvanized after fabrication. Railings on retaining walls to be placed after walls are backfilled. Bend bolts if necessary to maintain minimum clearance on superelevated slabs.

BRIDGE DEPARTMENT
*** DESIGN SECTION ***
Project Designer:
Chief Designer:
DESIGN — By — Checked
DETAILS — By R.J. Chrysler 12/57 — Checked
QUANTITIES — By — Checked
SPECIFICATIONS — Date of Spec. Revision
Approved for Publication — Date of Approval 5-58

5/58
STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
4A.1.10-1(c)

METAL BEAM BRIDGE RAILING

SCALE ½" = 1'-0" Except as shown | BRIDGE | FILE | DRAWN 12-9-62

EXHIBIT 1

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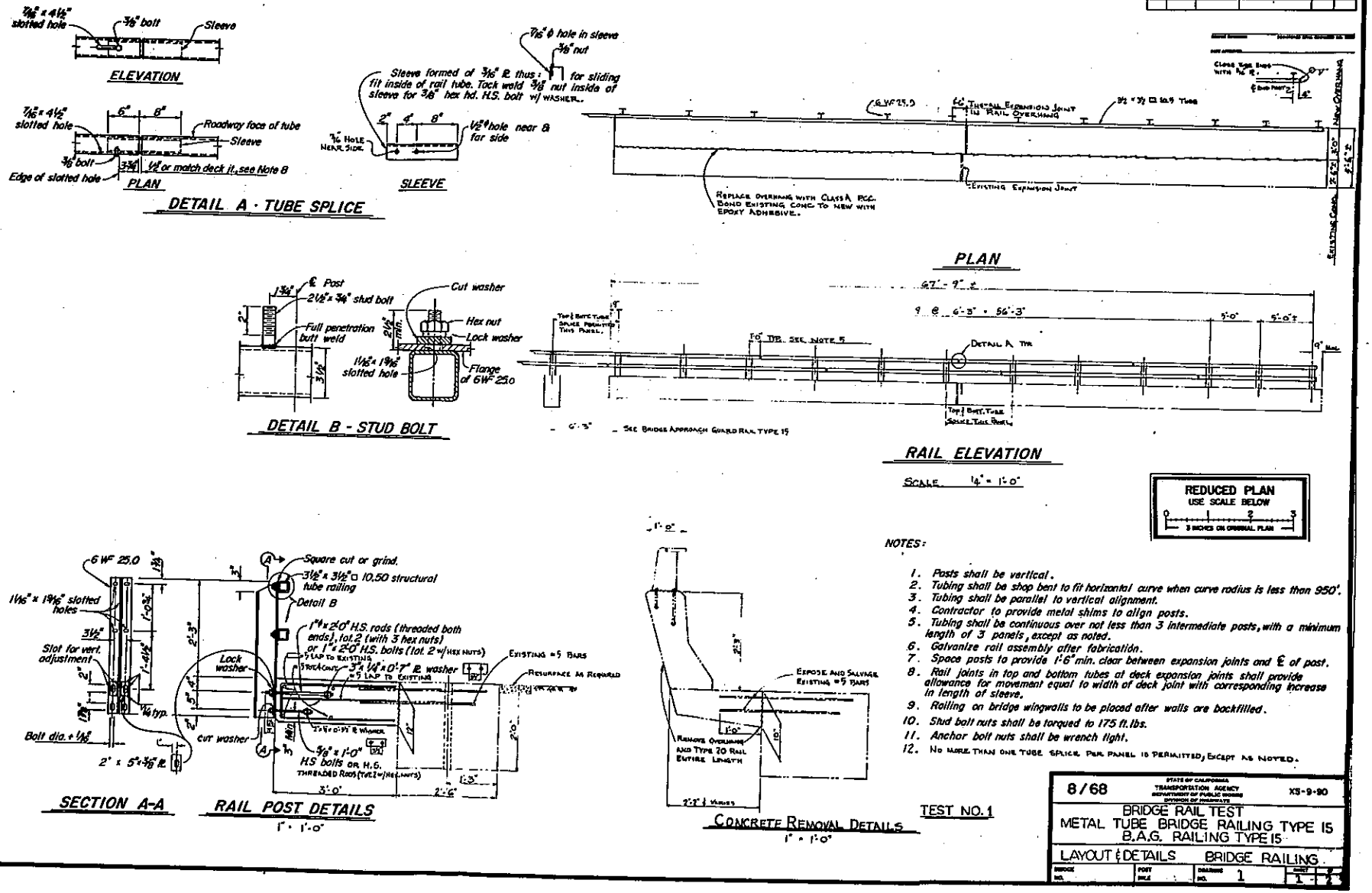
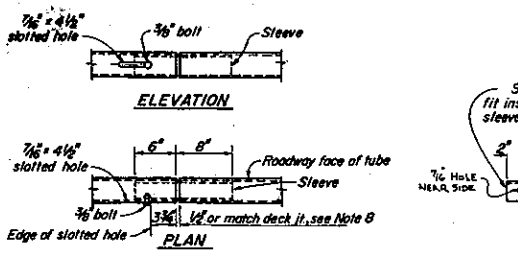
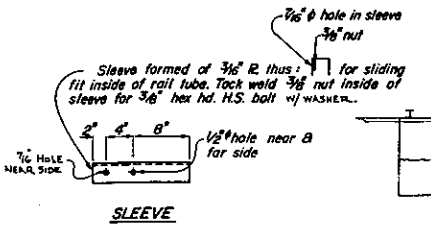
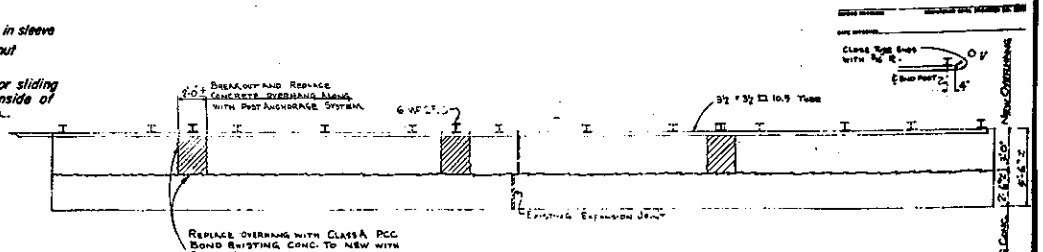


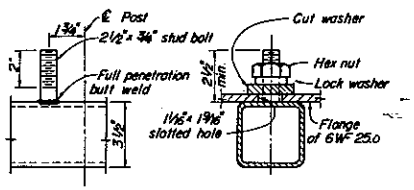
EXHIBIT 2

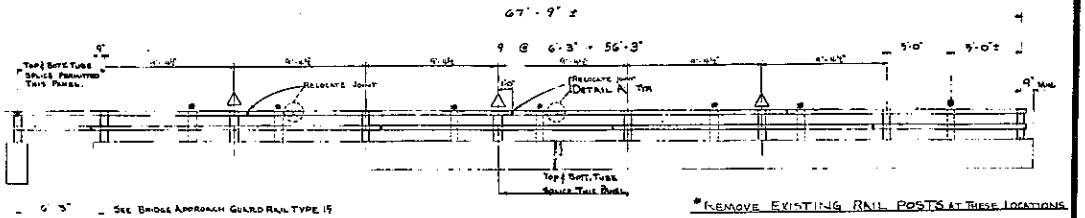
ClibPDF - www.fastio.com









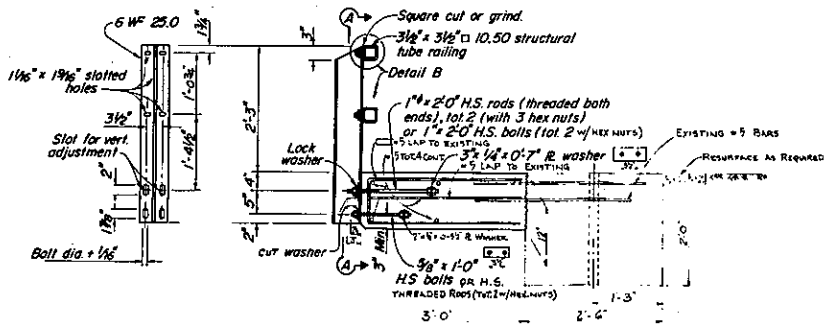


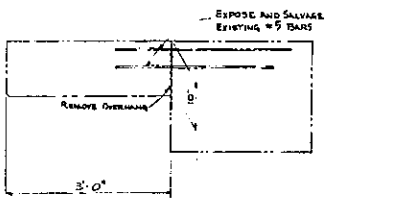
*Remove Existing Rail Posts at These Locations
△Install Rail Post at These Locations

RAIL ELEVATION

SCALE 1/4" = 1'-0"

REDUCED PLAN
USE SCALE BELOW
0 2 3
3 INCHES ON ORIGINAL PLAN





NOTES:

1. Posts shall be vertical.
2. Tubing shall be shop bent to fit horizontal curve when curve radius is less than 950'.
3. Tubing shall be parallel to vertical alignment.
4. Contractor to provide metal shims to align posts.
5. Tubing shall be continuous over not less than 3 intermediate posts, with a minimum length of 3 panels, except as noted.
6. Galvanize rail assembly after fabrication.
7. Space posts to provide 1'-6" min. clear between expansion joints and ℄ of post.
8. Rail joints in top and bottom tubes at deck expansion joints shall provide allowance for movement equal to width of deck joint with corresponding increase in length of sleeve.
9. Railing on bridge wingwalls to be placed after walls are backfilled.
10. Stud bolt nuts shall be torqued to 175 ft. lbs.
11. Anchor bolt nuts shall be wrench tight.
12. No more than one tube splice per panel is permitted, except as noted.

TEST NO. 2

8/68	STATE OF CALIFORNIA TRANSPORTATION AGENCY DEPARTMENT OF PUBLIC WORKS DIVISION OF HIGHWAYS	XS-9-'80
BRIDGE RAIL TEST		
METAL TUBE BRIDGE RAILING TYPE 15		
LAYOUT & DETAILS	BRIDGE RAILING	
BRIDGE NO.	POST MILE	DRAWING NO. 1

EXHIBIT 3

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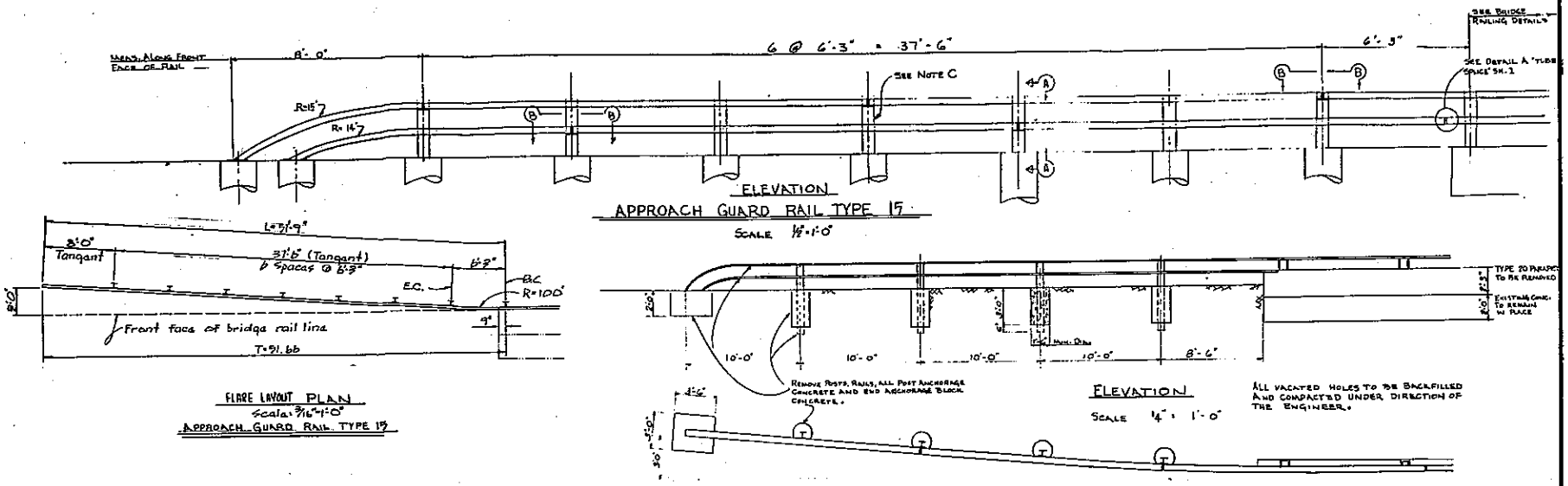
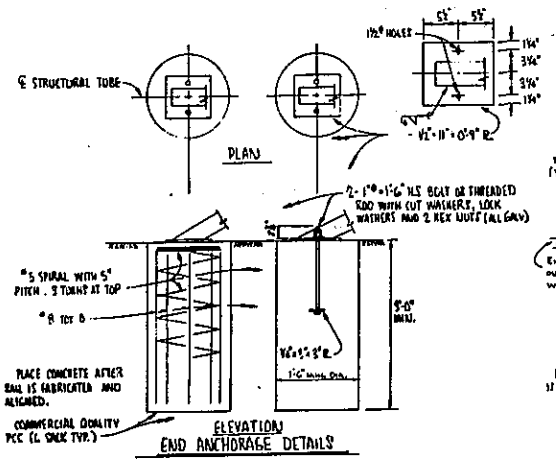
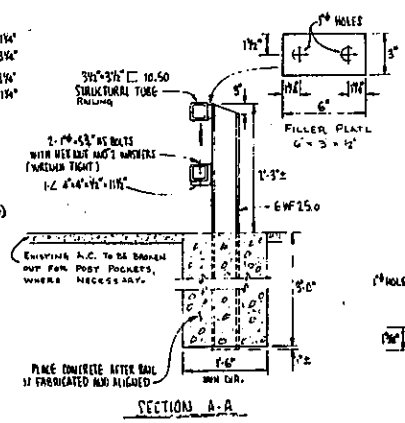
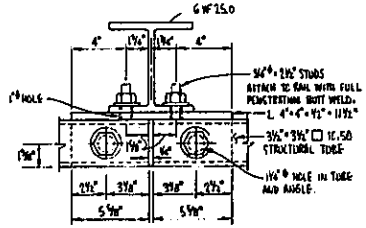


FLARE LAYOUT PLAN
Scale: 3/16"=1'-0"
APPROACH GUARD RAIL TYPE 15







PLAN
REMOVE EXISTING B.A.G. RAIL TYPE 8

NOTES:

A. TUBING SHALL BE SHOP BENT OR FABRICATED TO FIT HORIZONTAL CURVES OF LESS THAN 950' RADIUS.
B. TOP OF RAIL ELEVATIONS TO BE DETERMINED BY THE ENGINEER UNLESS SHOWN ON PLANS.
C. NO MORE THAN ONE TUBE SPLICE PER POST IS PERMITTED.
D. GALVANIZE RAIL ASSEMBLY AFTER FABRICATION.
E. FOR DETAILS NOT SHOWN SEE "BRIDGE RAILING" SHEET.
F. RAILING TO BE ADJUSTED TO FIT CURVED ROADWAY.
G. TUBING SHALL BE 3 PANELS LONG, EXCEPT AS NOTED.
H. TORQUE RAIL TO POST NUTS TO 175 FT. LBS.

REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

TEST NO. 1

TRANSPORTATION AGENCY
BRIDGE RAIL TEST
METAL TUBE BRIDGE RAILING TYPE 15
B.A.G. RAILING TYPE 15
LAYOUT & DETAILS B.A.G. RAILING
BRIDGE NO. | POST MILE | DRAWING NO. | 2

EXHIBIT 4

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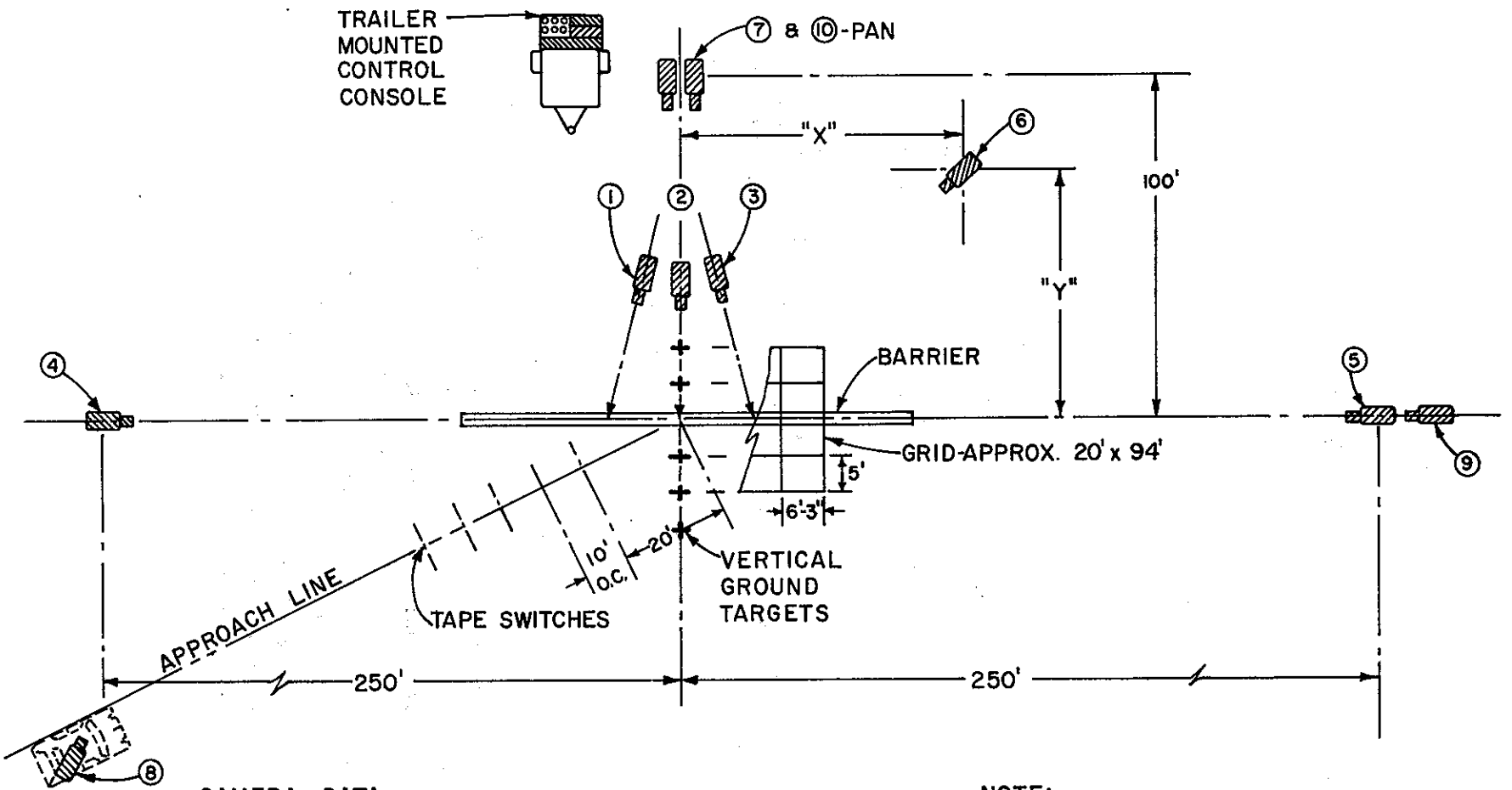


CAMERA DATA

①②③ PHOTO-SONICS, 13.0 MM LENS, 380 FPS,* MOUNTED ON 35' TOWER AND ORIENTED TO COVER THE AREAS INDICATED ABOVE.

④⑤ PHOTO-SONICS, 4" LENS, 380 FPS.

⑥ PHOTO-SONIC, 2" LENS, 380 FPS.

⑦ PHOTO-SONIC, 2" LENS, 380 FPS.

⑧ PHOTO-SONIC, 5.3 MM WIDE ANGLE LENS, 200 FPS, INSIDE TEST CAR.

⑨ HULCHER, 70 MM SEQUENCE CAMERA, 12" LENS, MOUNTED ABOUT 12' HIGH ON SCAFFOLD.

⑩ BOLEX, 1" LENS, 24 FPS.

* FRAMES PER SECOND.

NOTE:

CAMERAS 4 AND 5 HAVE THEIR HORIZONTAL AXES ALIGNED ON A PLANE 5' ABOVE GRADE AS MEASURED AT THE DESIRED POINT OF IMPACT.

DIMENSIONS "X" & "Y" DEPEND UPON BARRIER LENGTH, AND IMPACT ANGLE.

EXHIBIT 5

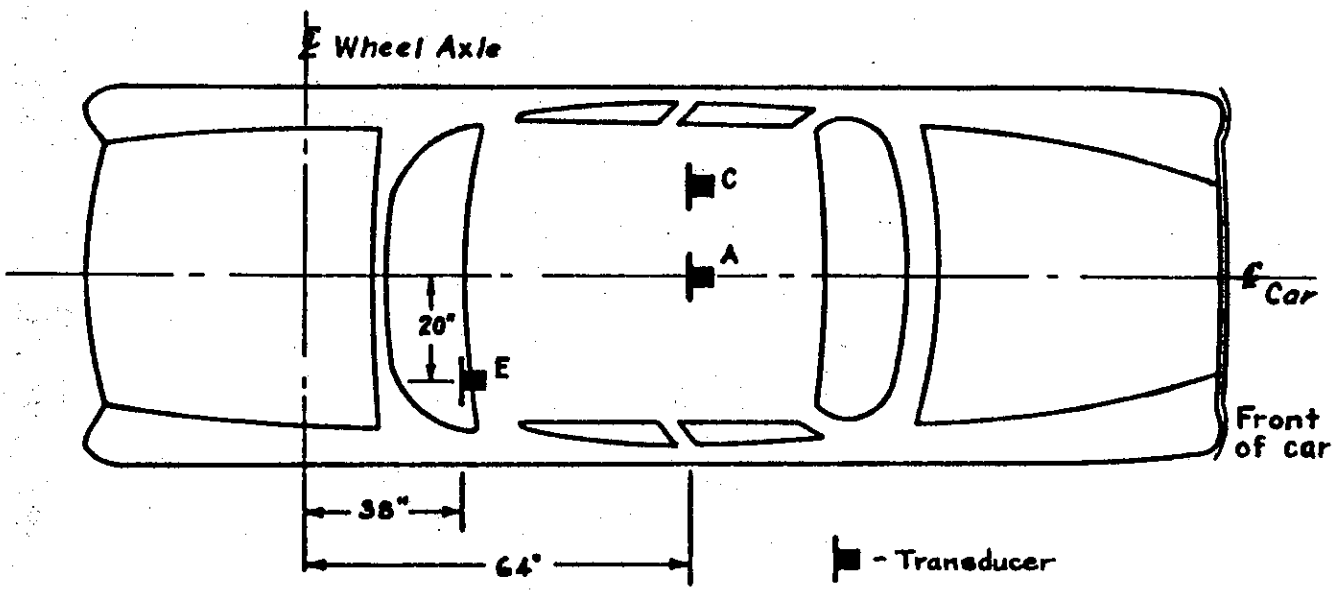
DYNAMIC FULL SCALE BARRIER TESTS
PHOTOGRAPHIC INSTRUMENTATION – TYPICAL PLAN

EXHIBIT 6

CALIFORNIA DIVISION OF HIGHWAYS

VEHICLE TRANSDUCER INSTRUMENTATION

Type 15 Barrier Rail Impact Test
W.O. 656496



DATA CHANNEL NO.	LOCATIONS	
1	C	Longitudinal accelerometer mounted in Stan's head.
2	C	Lateral accelerometer mounted in Stan's head.
3	C	Vertical accelerometer mounted in Stan's head.
4	C	Longitudinal accelerometer mounted in Stan's chest.
5	A	Longitudinal accelerometer mounted on car floor.
6	E	Longitudinal accelerometer mounted on car floor.
7	A	Lateral accelerometer mounted on car floor.
8	E	Lateral accelerometer mounted on car floor.
9	C	Seat belt transducer across Stan's lap.

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ACCELERATION VS TIME

TEST NO. 251 _____64 MPH, 12°
LOCATION _____VEHICLE - A
ORIENTATION _____LATERAL
FILTER _____100 HERTZ

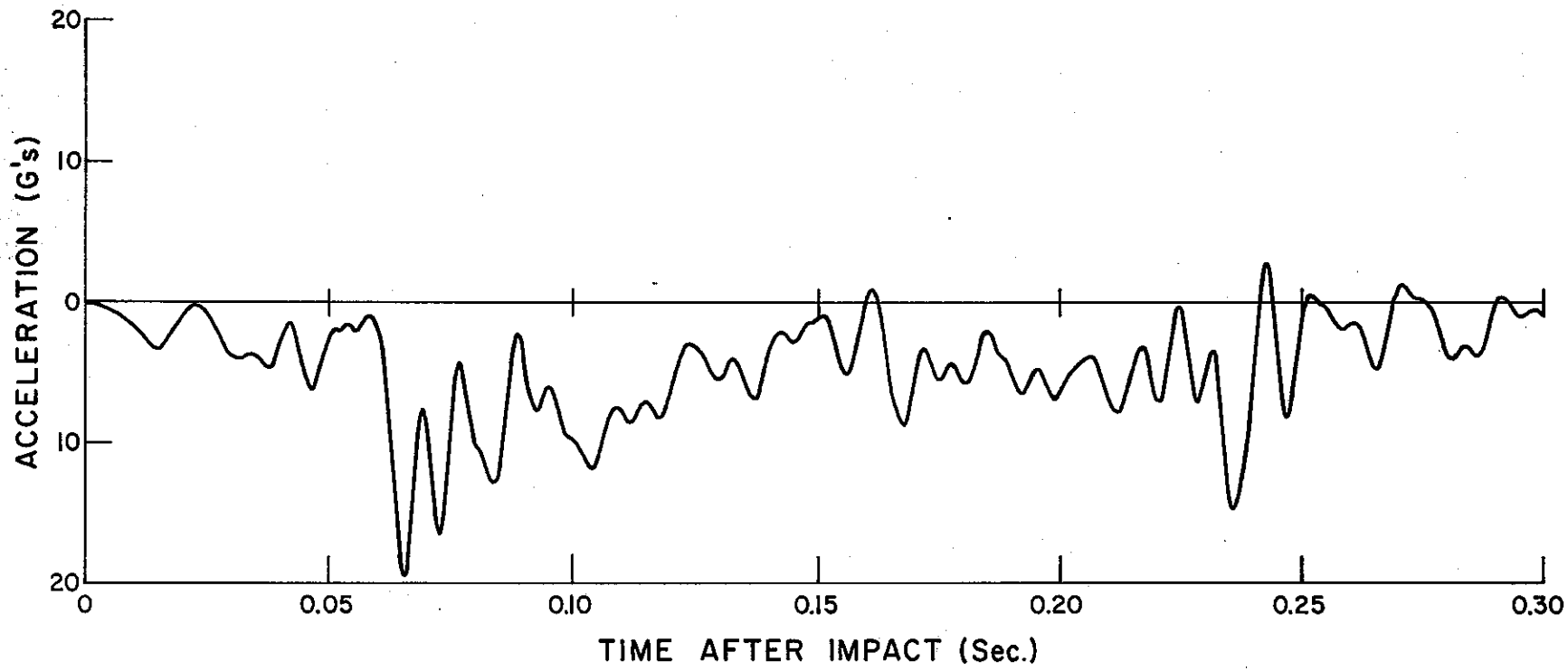

PLATE A-1

PLATE A-2

ACCELERATION VS TIME

TEST NO. 251 ——————64 MPH, 12°

LOCATION ——————VEHICLE - A

ORIENTATION ——————LONGITUDINAL

FILTER ——————100 HERTZ

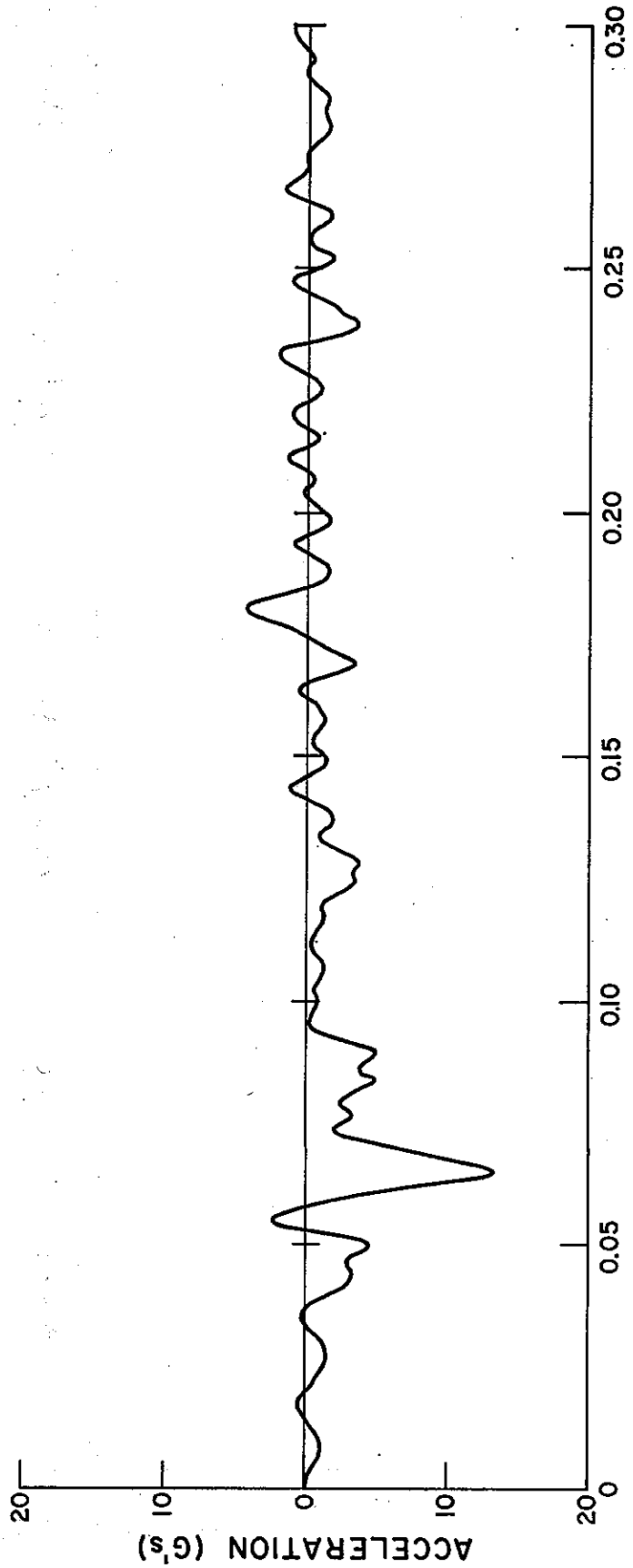

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PLATE A-3

ACCELERATION VS TIME

TEST NO. 251 _____64 MPH, 12°

LOCATION _____VEHICLE - E

ORIENTATION _____LATERAL

FILTER _____100 HERTZ

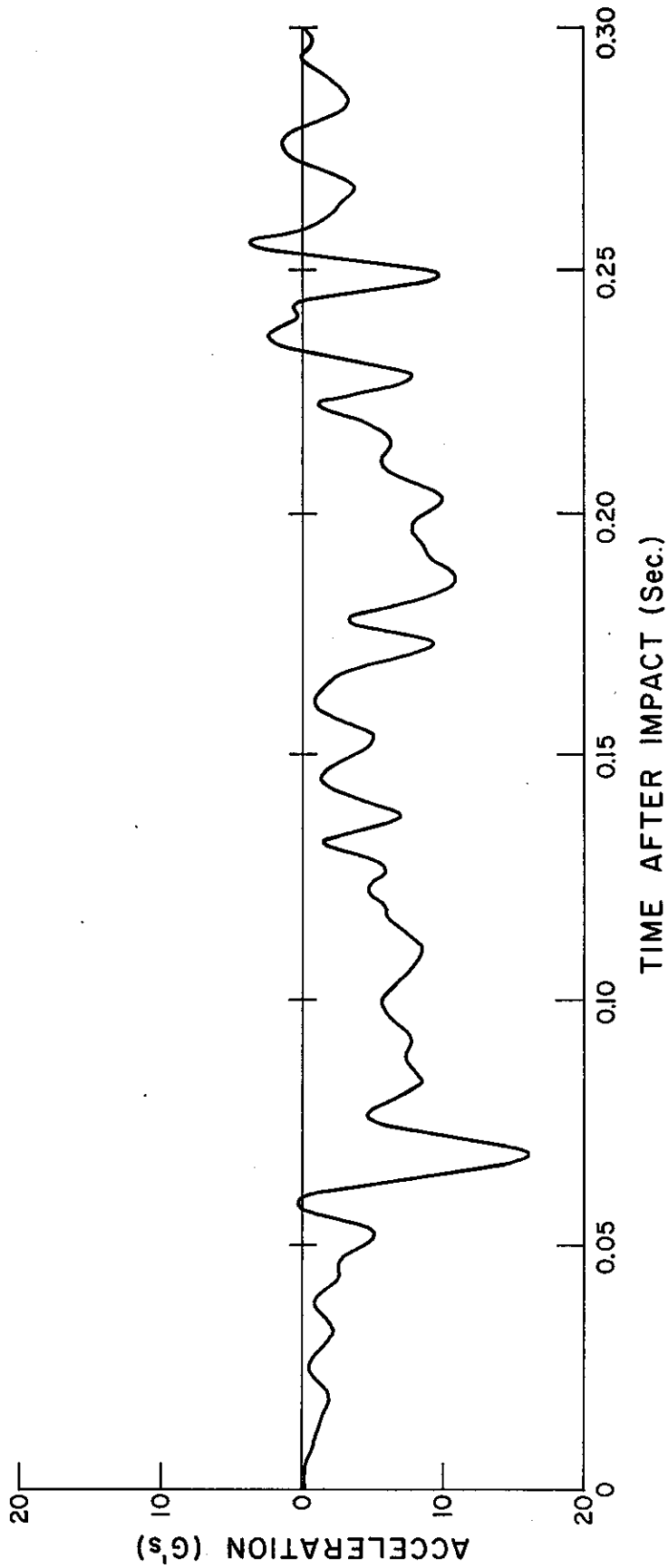

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PLATE A-4

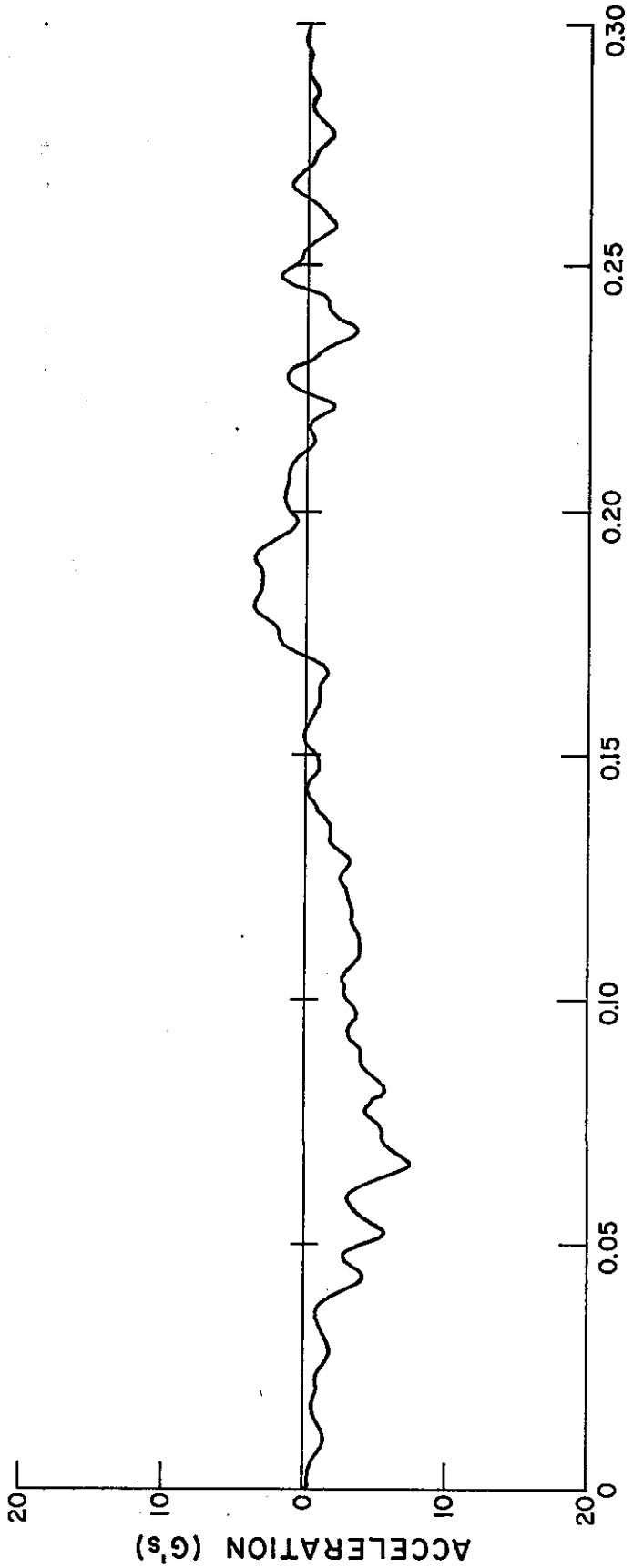
ACCELERATION VS TIME

TEST NO. 251 ——————64 MPH, 12°

LOCATION ——————VEHICLE-E

ORIENTATION ——————LONGITUDINAL

FILTER ——————100 HERTZ



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ACCELERATION VS TIME

TEST NO. 251 ____ 64 MPH, 12°
LOCATION ____ DUMMY - HEAD
ORIENTATION ____ LATERAL
FILTER ____ 100 HERTZ

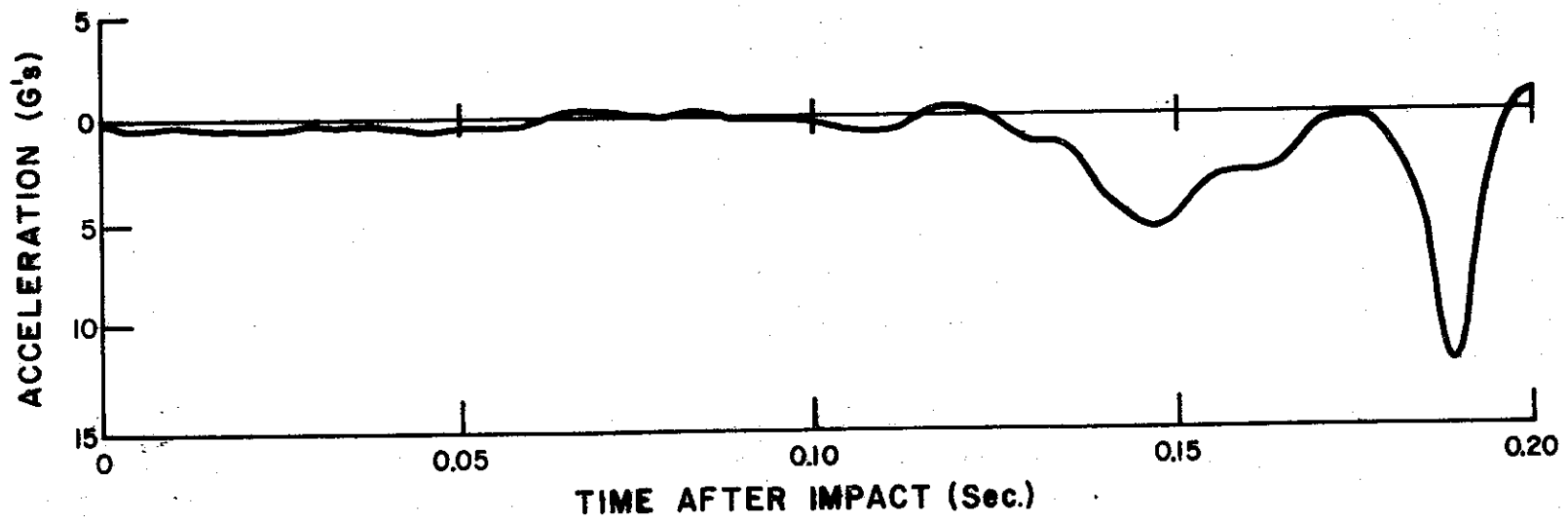

PLATE A-5

PLATE A-6

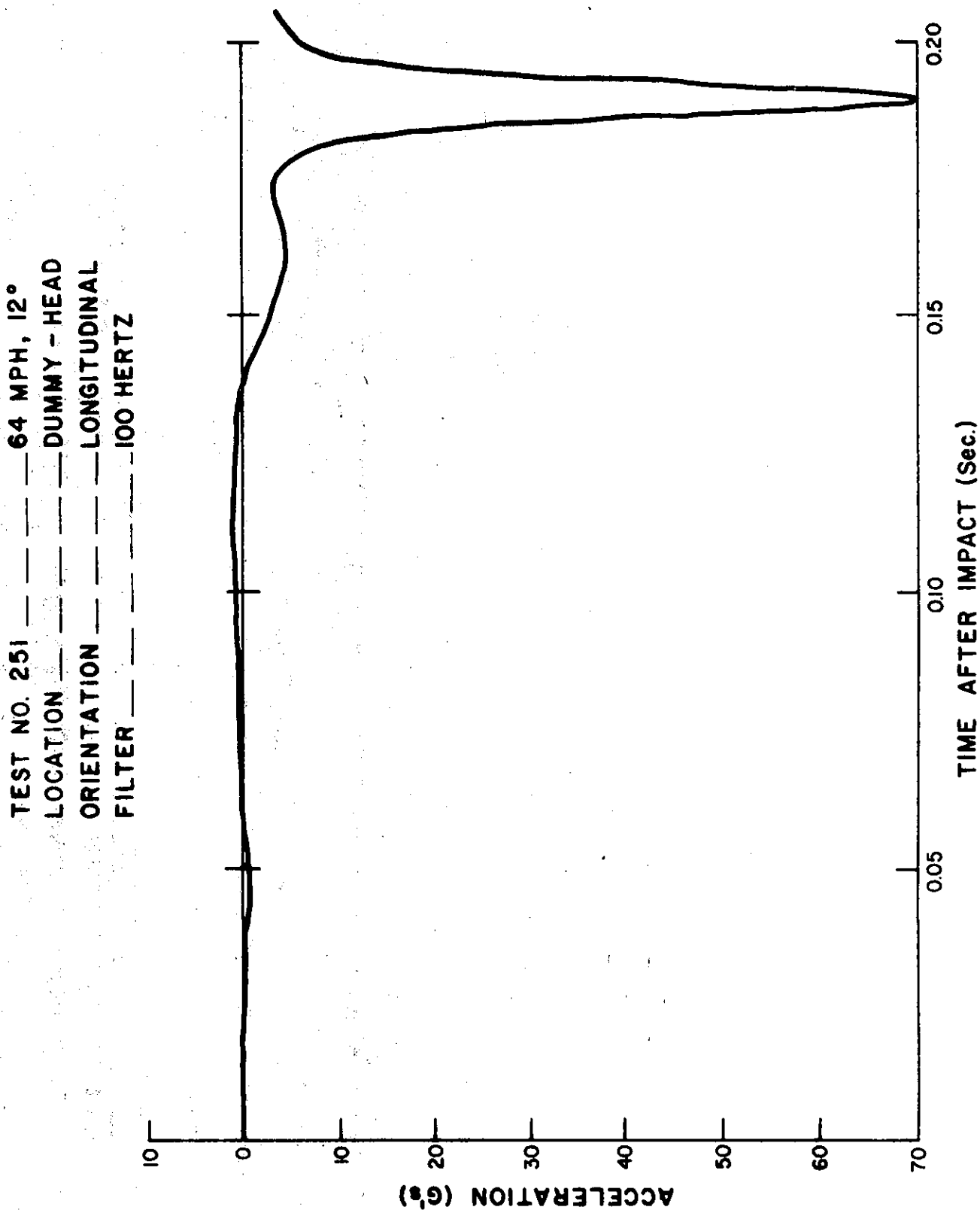

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PLATE A-7

ACCELERATION VS TIME

TEST NO. 251 ——————64 MPH, 12°

LOCATION ——————DUMMY-HEAD

ORIENTATION ——————VERTICAL

FILTER ——————100 HERTZ

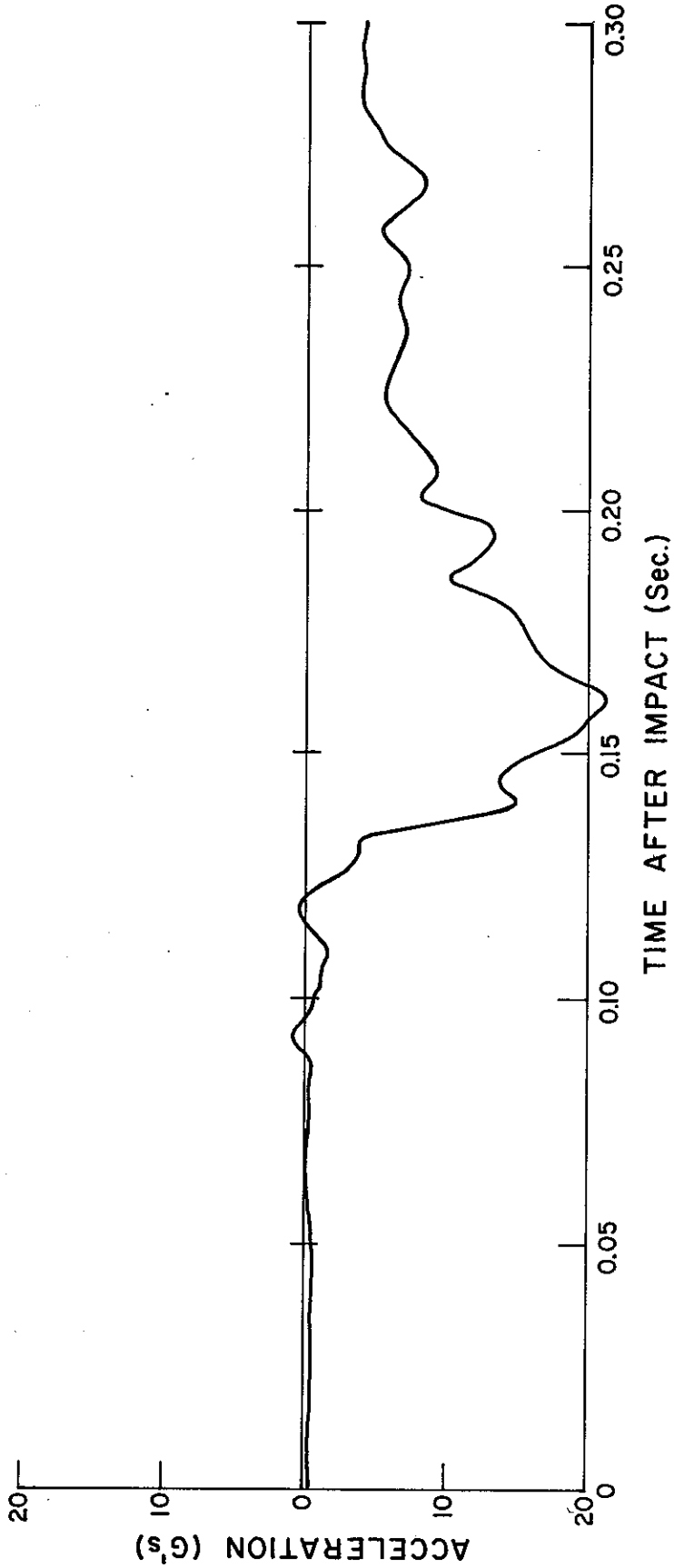

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PLATE A-8

ACCELERATION VS TIME

TEST NO. 251 ——————64 MPH, 12°

LOCATION ——————DUMMY - CHEST

ORIENTATION ——————LONGITUDINAL

FILTER ——————100 HERTZ

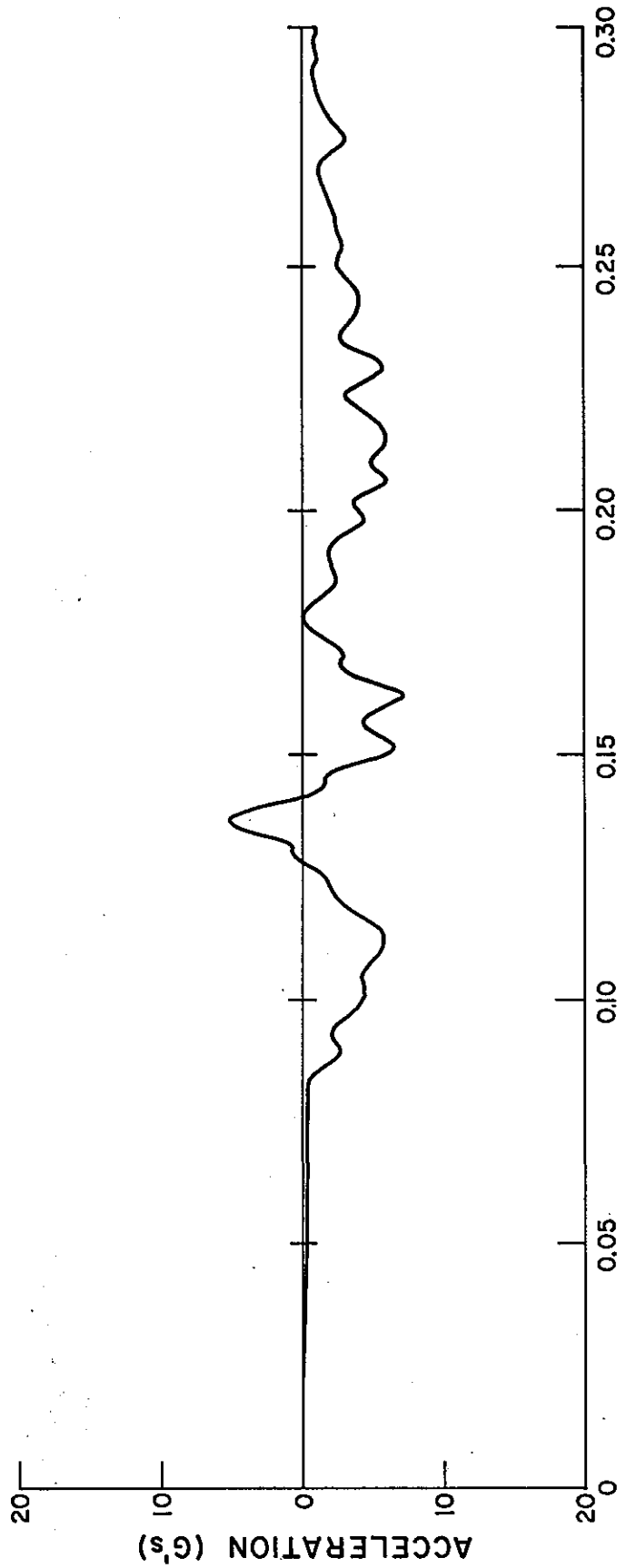

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PLATE A-9

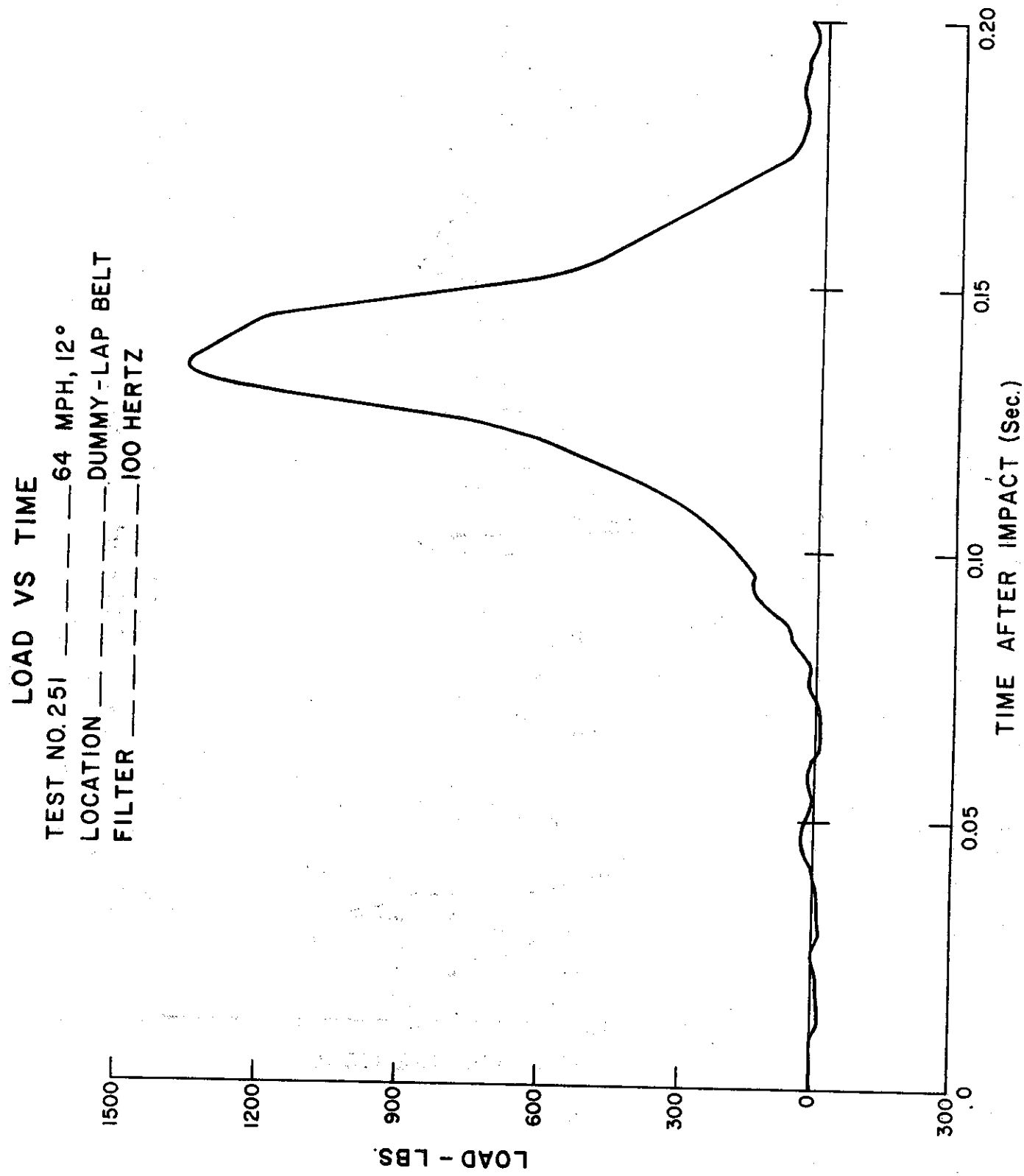

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PLATE A-10

ACCELERATION VS TIME

TEST NO. 252 ——————— 59 MPH, 14°
LOCATION ——————— VEHICLE-A
ORIENTATION ——————— LATERAL
FILTER ——————— 176 HERTZ

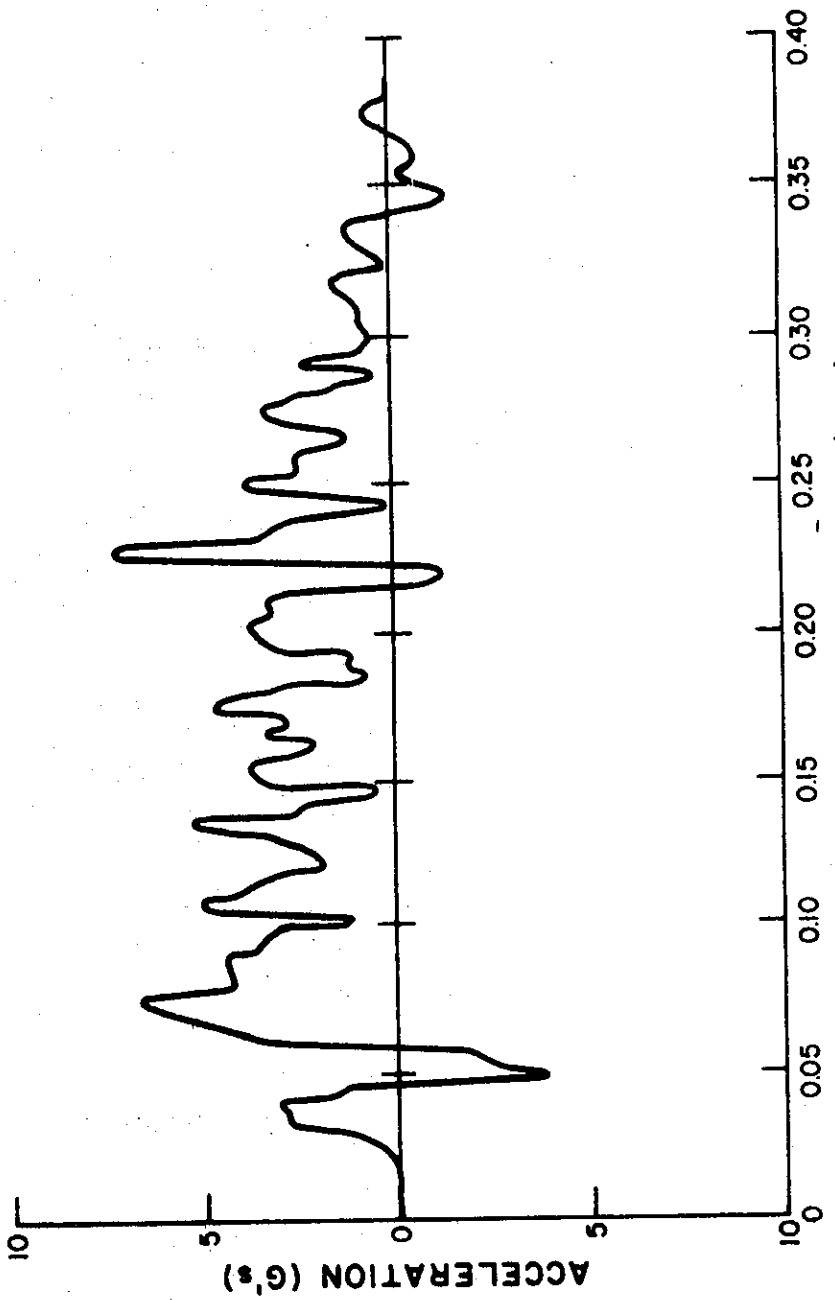

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PLATE A-11

ACCELERATION VS TIME

TEST NO. 252 ---------- 59 MPH, 14°
LOCATION ---------- VEHICLE-E
ORIENTATION ---------- LONGITUDINAL
FILTER ---------- 176 HERTZ

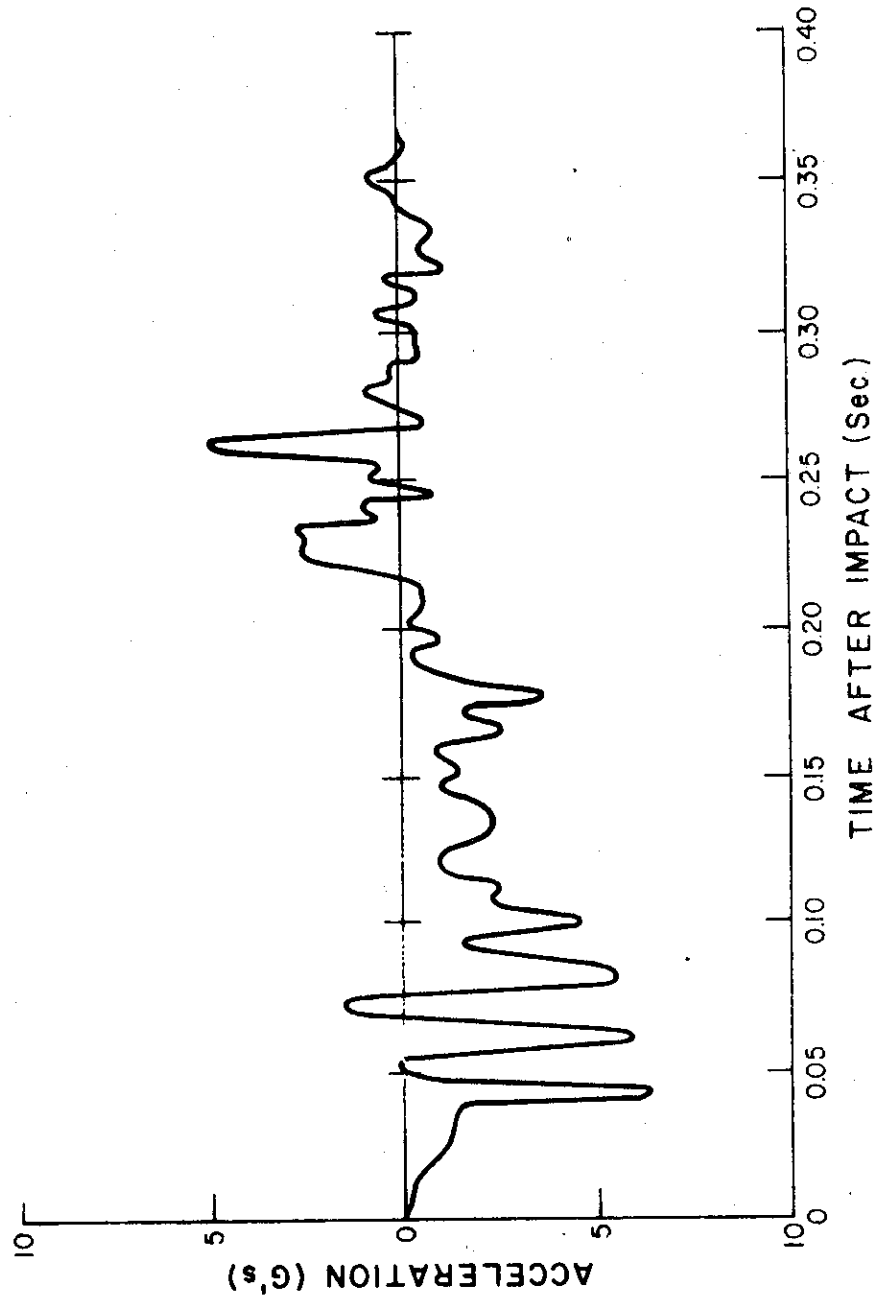

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PLATE A-12

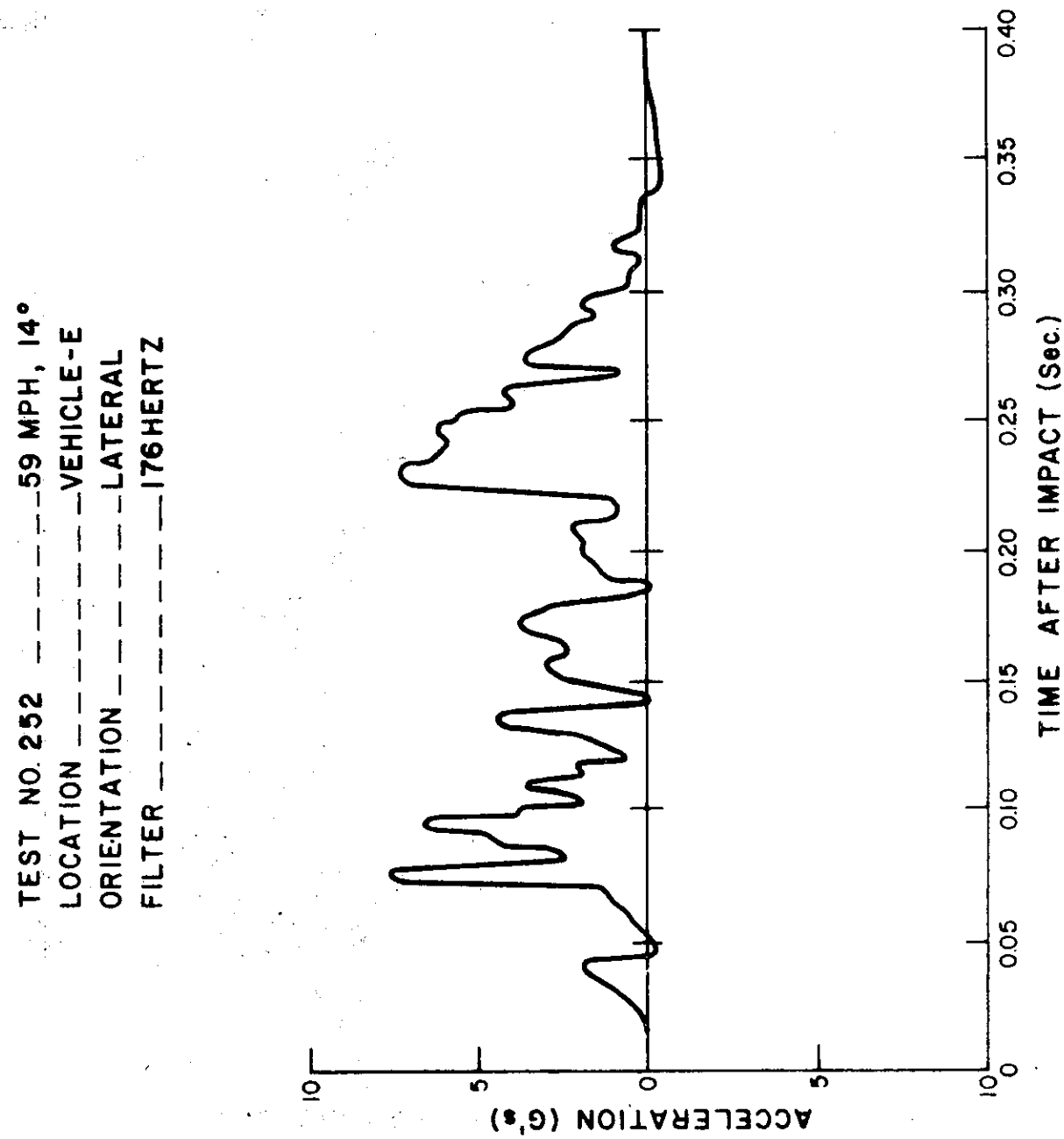

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PLATE A-13

ACCELERATION VS TIME

TEST NO. 252 ---------59 MPH, 14°
LOCATION ---------VEHICLE-A
ORIENTATION ---------LONGITUDINAL
FILTER ---------176 HERTZ

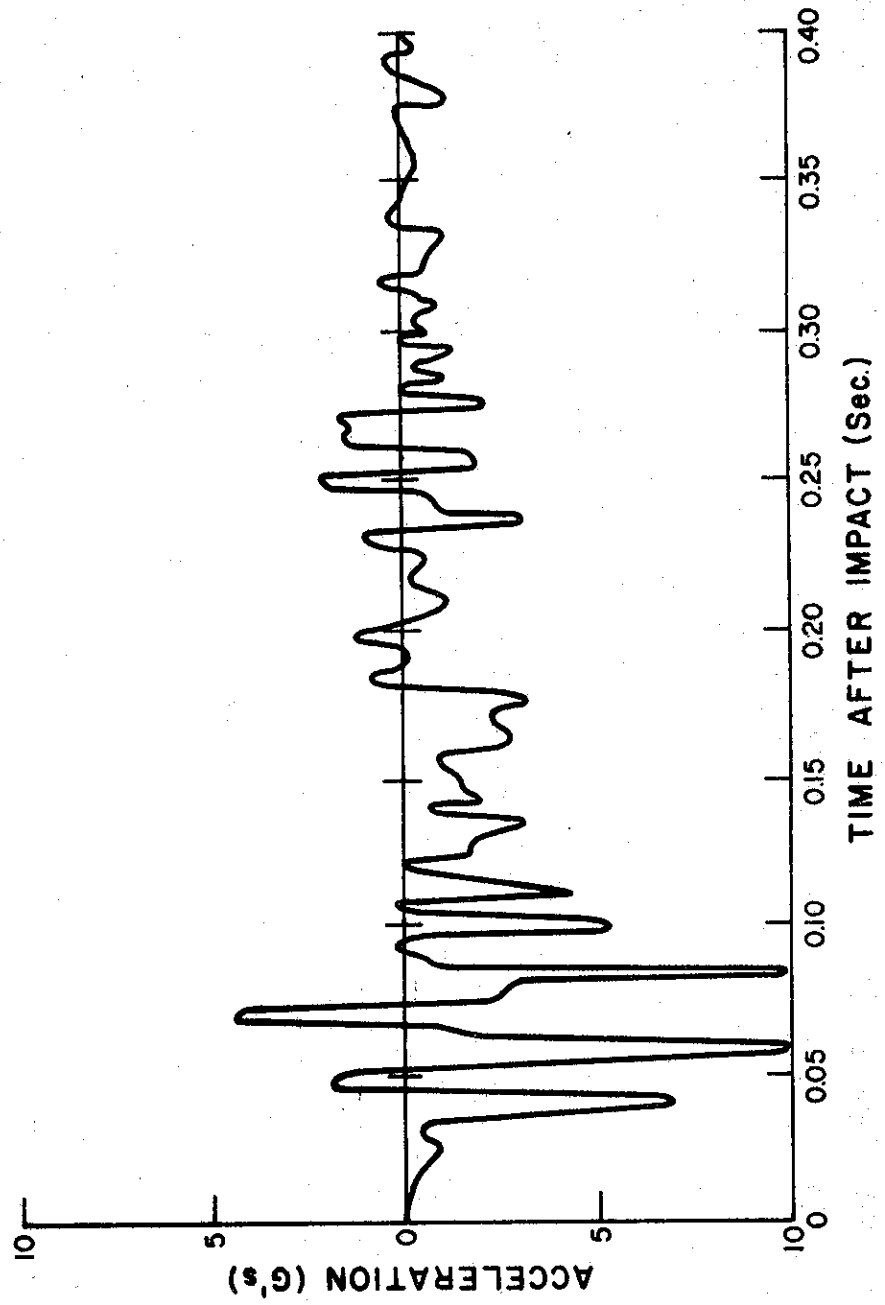

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PLATE A-14

ACCELERATION VS TIME

TEST NO. 252 ——————59 MPH, 14°
LOCATION ——————DUMMY HEAD
ORIENTATION——————LATERAL
FILTER ——————176 HERTZ

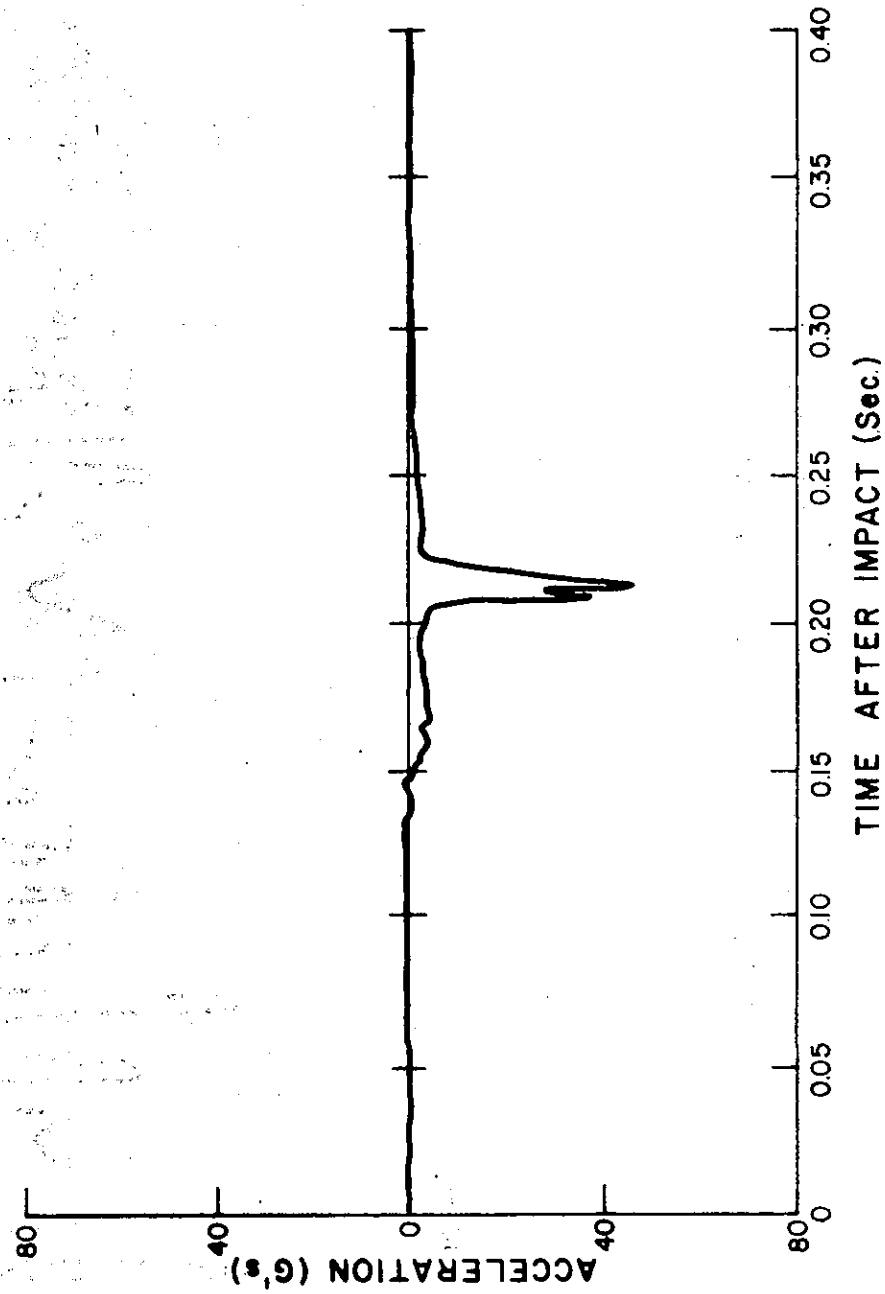

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PLATE A-15

ACCELERATION VS TIME

TEST NO. 252 ---------59 MPH, 14°
LOCATION ---------DUMMY HEAD
ORIENTATION ---------LONGITUDINAL
FILTER ---------176 HERTZ

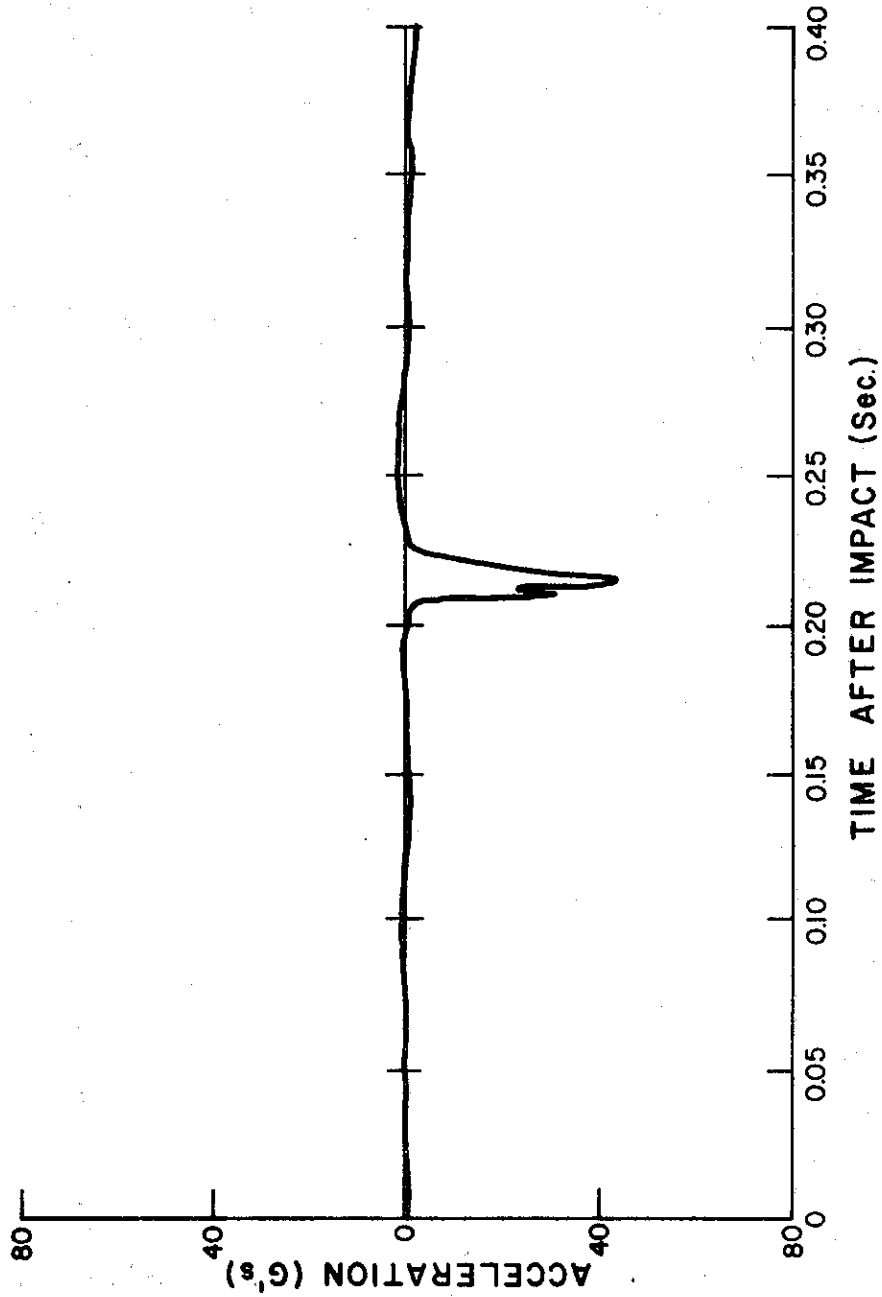

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PLATE A-16

ACCELERATION VS TIME

TEST NO. 252 ---------59 MPH, 14°
LOCATION ---------DUMMY HEAD
ORIENTATION ---------VERTICAL
FILTER ---------176 HERTZ

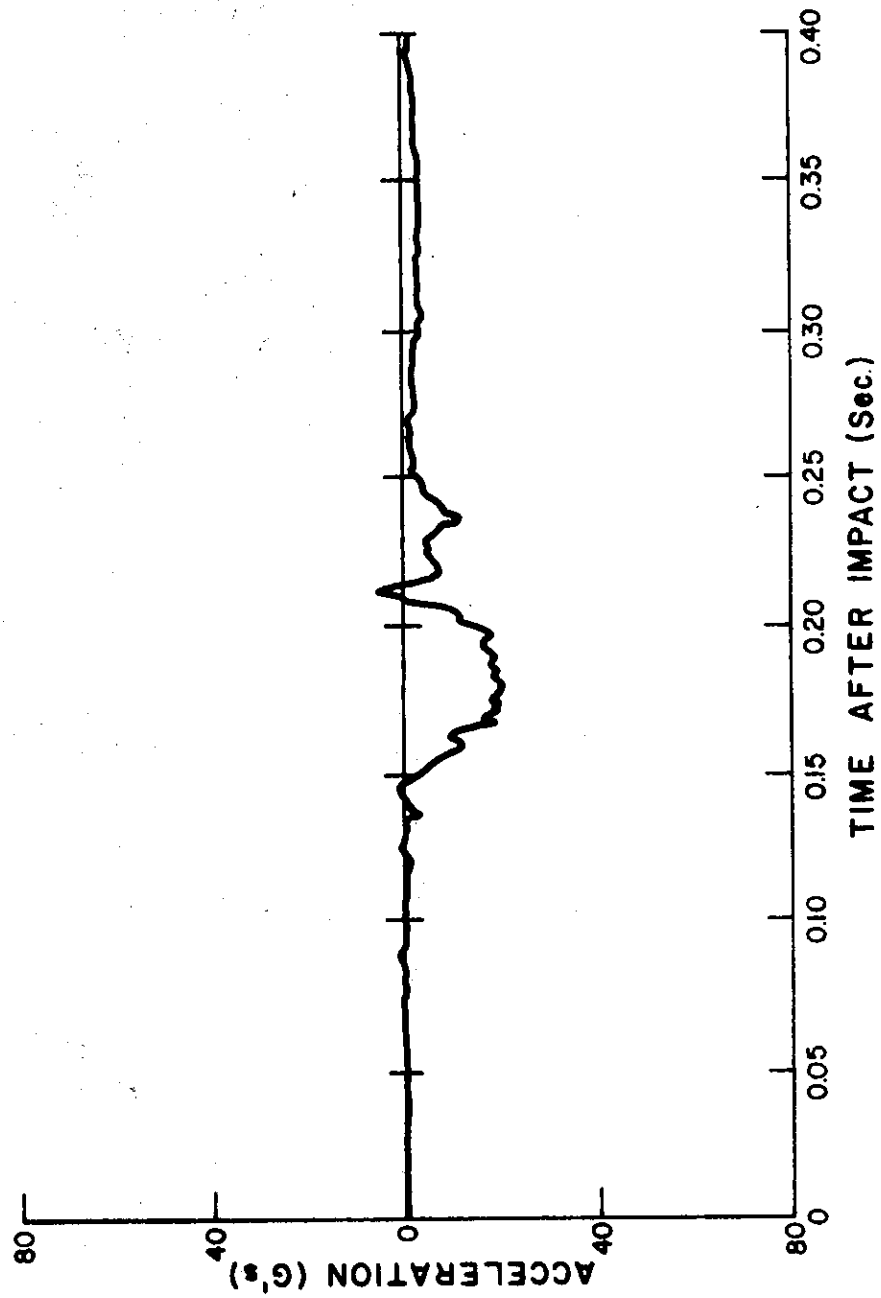

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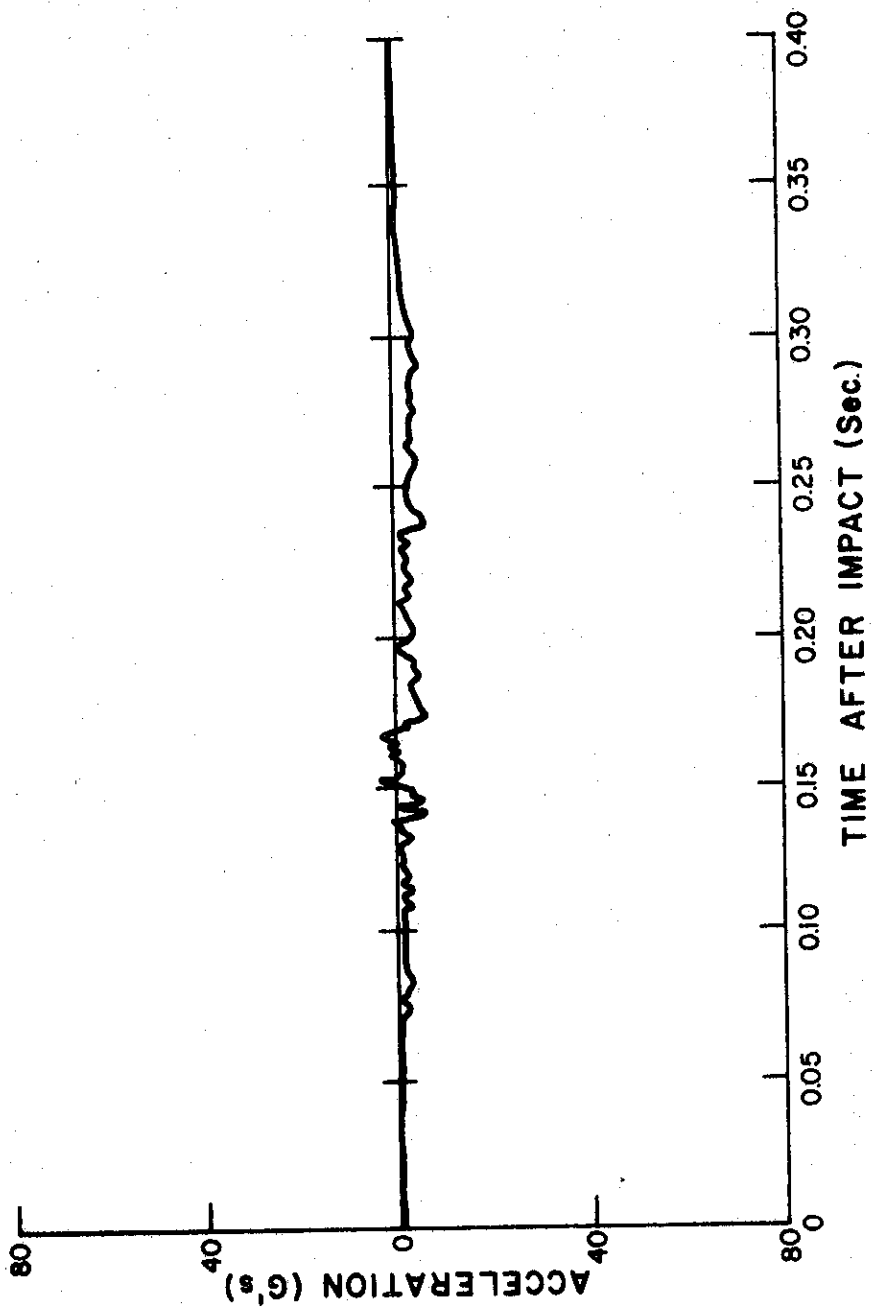
PLATE A-17

ACCELERATION VS TIME

TEST NO. 252 ---------59 MPH, 14°
LOCATION ---------DUMMY CHEST
ORIENTATION ---------LONGITUDINAL
FILTER ---------176 HERTZ



ClibPDF - www.fastio.com

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LOAD VS TIME

TEST NO. 252 ___ ___ ___ ___ 59 MPH, 14°
LOCATION ___ ___ ___ ___ ___ DUMMY - LAP BELT
FILTER ___ ___ ___ ___ ___ 176 HERTZ

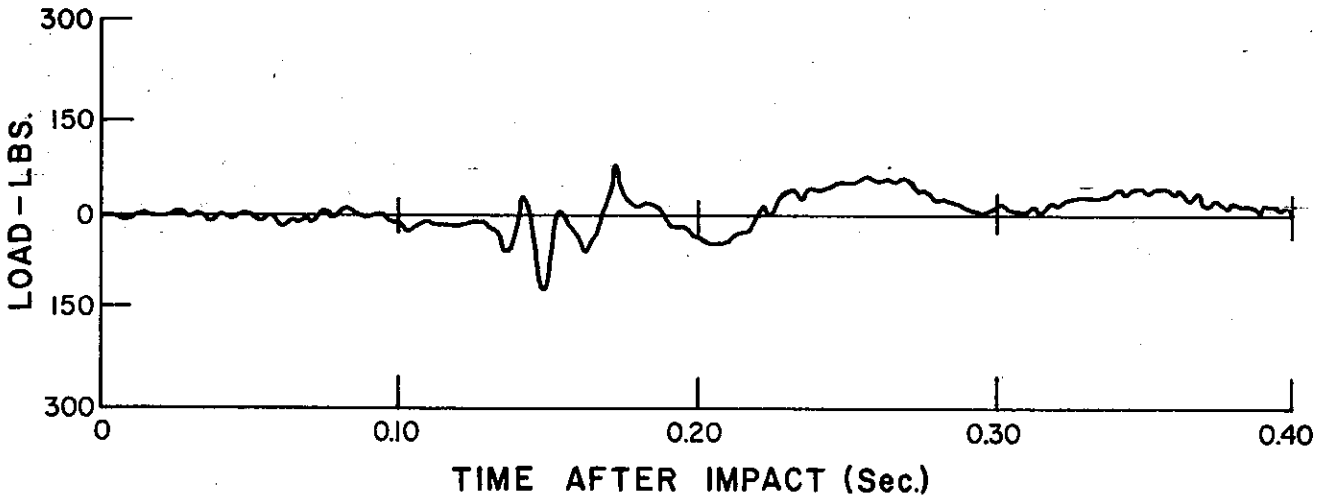

PLATE A-18