

PERFORMANCE LEVEL 1 TESTS ON THE NEBRASKA OPEN CONCRETE BRIDGE RAIL



by

Ronald K. Faller
Research Associate Engineer

Brian G. Pfeifer
Research Associate Engineer

James C. Holloway
Research Associate Engineer

Barry T. Rosson, Ph.D.
Assistant Professor

submitted to

Gale Barnhill
Bridge Division
Nebraska Department of Roads
1500 Nebraska Highway 2
Lincoln, NE 68502
RES1 (0099) P450

TRANSPORTATION RESEARCH REPORT TRP-03-28-91

Midwest Roadside Safety Facility
Civil Engineering Department
W348 Nebraska Hall
University of Nebraska-Lincoln
Lincoln, Nebraska 68588-0531

February 1992

DISCLAIMER STATEMENT

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Nebraska Department of Roads nor the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation and thanks to the following individuals who made a contribution to the completion of this research project. A special thanks is given to the members of the Midwest States Regional Pooled Fund Program. This report is dedicated to Dr. Edward R. Post; since it was one of his last active projects before his death.

Nebraska Department of Roads

Gale Barnhill, Bridge Division
Dalyce Ronnau, Materials and Test Division

Missouri Department of Transportation

Dan Davidson, Design Division

Kansas Department of Transportation

Ron Seitz, Road Design Squad Leader

Federal Highway Administration

Bill Wendling, Safety Program Engineer, Region VII, Kansas City Office
Milo Cress, Bridge Engineer, Nebraska Division Office

Midwest Roadside Safety Facility

Syed Ataullah, Graduate Research Assistant
Jim Luedke, Graduate Research Assistant
Terry Atkins, C.E. Student
Virgil Oligmueller, C.E. Student

University of Nebraska - Lincoln

Tom Grady, Electronic Technician
Tom Hoffman, Mechanical Technician

Center for Infrastructure Research

Maher Tadros, Ph.D., Director
Sammy Elias, Ph.D., Associate Dean for Engineering Research Center

Midwest Transportation Center

Tom Maze, Ph.D., Director

Private

Jim Dunlap (Dunlap Photography)

ABSTRACT

A safety performance evaluation of the 29-in. Nebraska Open Concrete Bridge Rail was conducted for the Nebraska Department of Roads (NDOR). The evaluation included computer simulation modelling and two full-scale vehicle crash tests. The two crash tests were conducted to evaluate structural adequacy of concrete posts adjacent to a gap and the reduced reinforcement in the concrete rail, respectively.

The bridge rail design incorporates a 3-in. expansion gap. The NDOR Bridge Division was concerned that lateral deflections in the posts and attached rail could be of sufficient magnitude to cause vehicle snagging at the location of the gap. The bridge rail has been previously constructed in actual installations using six No. 6 longitudinal reinforcing bars. However, the NDOR Bridge Division reasoned that the structural adequacy of the concrete rail could be maintained while reducing the reinforcement from six No. 6's to six No. 5's for economic considerations.

BARRIER VII computer simulation modelling of the original bridge rail design was used to predict safety performance prior to crash testing and to modify the original design. The installation went through a number of redesign phases as a result of the computer simulation modelling. The final design (Design No. 4) consisted of 11-in. x 36-in. concrete posts adjacent to the gap, and 11-in. x 24-in. concrete posts at all other locations.

The safety performance evaluation was conducted and reported according to the criteria specified in the *Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*, National Cooperative Highway Research Program (NCHRP) Report 230, and the American Association of State Highway and Transportation Officials (AASHTO) *Guide Specifications for Bridge Railings*, 1989.

The first test (Test NEOCR-1), was conducted 5-ft 10½-in. upstream from the centerline of the 3-in. gap. The test was conducted with a 5,300-lb test vehicle at the impact conditions of 47.7 mph and 20 degrees. The safety performance of the 29-in. Nebraska Open Concrete Bridge Rail near the 3-in. expansion gap was determined to be satisfactory according to the AASHTO Performance Level 1 Guide Specifications.

The second test (Test NEOCR-2), was conducted to evaluate the reduction of longitudinal reinforcement in a continuous concrete rail section. The test was conducted with a 5,390-lb test vehicle at the impact conditions of 45.9 mph and 20 degrees. The location of impact was at a midspan location along the continuous rail section. The safety performance of the 29-in. Nebraska Open Concrete Bridge Rail with No. 5 longitudinal reinforcement was determined to be satisfactory according to the AASHTO Performance Level 1 Guide Specifications.

TABLE OF CONTENTS

	Page
DISCLAIMER STATEMENT	i
ACKNOWLEDGEMENTS	ii
ABSTRACT	iii
TABLE OF CONTENTS	v
1 INTRODUCTION	1
1.1 Background	1
1.2 Problem Statement	1
1.3 Objective	2
1.4 Scope	2
2 COMPUTER SIMULATION MODELLING	3
2.1 BARRIER VII	3
2.2 Computer Simulation Impact Conditions	3
2.3 Computer Simulation Test Vehicle	3
2.4 Design No. 1	3
2.5 Design No. 2	10
2.6 Design No. 3	17
2.7 Design No. 4	17
3 TEST CONDITIONS	29
3.1 Test Facility	29
3.1.1 <u>Test Site</u>	29
3.1.2 <u>Vehicle Tow System</u>	29
3.1.3 <u>Vehicle Guidance System</u>	29
3.2 Nebraska Open Concrete Bridge Rail Design Details	32
3.3 Test Vehicles	40
3.4 Data Acquisition Systems	45
3.4.1 <u>Accelerometers</u>	45
3.4.2 <u>High Speed Photography</u>	45
3.4.3 <u>Speed Trap Switches</u>	48
3.5 Test Parameters	48
4 PERFORMANCE EVALUATION CRITERIA	51
5 TEST RESULTS	54
5.1 Test NEOCR-1 (5,300 lb, 47.7 mph, 20.0 deg)	54
5.2 Test NEOCR-2 (5,390 lb, 45.9 mph, 20.0 deg)	63

6	CONCLUSIONS	70
7	REFERENCES	75
8	APPENDICES	76
	Appendix A. Relevant Correspondence	77
	Appendix B. Fortran Program	81
	Appendix C. Material Specifications	86
	Appendix D. Accelerometer Data Analysis, NEOCR-1	92
	Appendix E. Accelerometer Data Analysis, NEOCR-2	103

10. Computer Simulation Model Configuration, Design No. 1	27
11. Nebraska Open Concrete Bridge Rail, Design No. 4	27
12. Beam Design No. 2	28
13. Cable Tow and Guidance System	30
14. Tow Vehicles and Fifth Wheel	31
15. Bridge Rail Design Details	33
16. Nebraska Open Concrete Bridge Rail	34
17. Concrete Deck Attachment Detail	35
18. Reinforcement Layout for Bridge Deck	36
19. Steel Reinforcement in the 3-ft. Posts Adjacent to the 3-in. Gap	38
22. Test Vehicle Dimensions, NEOCR-1	42
23. Test Vehicle, NEOCR-2	43
24. Test Vehicle Dimensions, NEOCR-2	44
25. Flow Chart of Accelerometer Data Acquisition System	46
26. Layout of High Speed Cameras, NEOCR-1, 2	47
27. Impact Location, Test NEOCR-1	49
28. Impact Location, Test NEOCR-2	50
29. Summary and Sequential Photographs, NEOCR-1	55
30. Parallel Time Sequential Photographs, NEOCR-1	56
31. Vehicle Trajectory, NEOCR-1	57
32. Bridge Rail Damage, NEOCR-1	59
33. Bridge Rail Damage at the 3 in. Gap, NEOCR-1	60
34. Bridge Rail Damage of Second Impact, NEOCR-1	61
35. Vehicle Damage, NEOCR-1	62
36. Summary and Sequential Photographs, NEOCR-2	65
37. Parallel Sequential Photographs, NEOCR-2	66
38. Vehicle Trajectory, NEOCR-2	67
39. Bridge Rail Damage, NEOCR-2	68
40. Vehicle Damage, NEOCR-2	69

List of Tables

	Page
1. Structural Quantities for Computer Simulation - Design No. 1	9
2. Summary of Simulation Results - Design No. 1	11
3. Structural Quantities for Computer Simulation - Design No. 2	15
4. Summary of Simulation Results - Design No. 2	16
5. Structural Quantities for Computer Simulation - Design No. 3	19
6. Summary of Simulation Results - Design No. 3	20
7. Structural Quantities for Computer Simulation - Design No. 4	25
8. Summary of Simulation Results - Design No. 4	26
9. Crash Test Conditions and Evaluation Criteria.	52
10. AASHTO Evaluation Criteria	53
11. Summary of Safety Performance Results	72
12. Summary of Test Results	74

1 INTRODUCTION

1.1 Background

In 1986, a safety performance evaluation of the Nebraska Open Concrete Bridge Rail was conducted for the Nebraska Department of Roads (NDOR) by Ensco, Inc. of Springfield, Virginia (1). Two full-scale vehicle crash tests were performed.

The open concrete bridge rail was constructed with a 1-ft 2-in. wide x 1-ft 4-in. deep rail supported by concrete posts measuring 11-in. wide x 11-in. deep x 1-ft 1-in. high. The posts were spaced 7-ft 6-in. on centers. Although the open concrete bridge rail design incorporated a 3-in. expansion gap, the simulated bridge rail used in testing did not contain an expansion gap.

The two tests were conducted according to the "Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances," NCHRP 230 (2). Test 1769-F-1-86 was conducted with a 4,669-lb Ford Thunderbird at the impact conditions of 57.6 mph and 26 degrees. Test 1769-F-2-86 was conducted with a 1,971-lb VW Rabbit at the impact conditions of 59.8 mph and 21 degrees. The safety performance of the bridge rail was determined to be satisfactory according to NCHRP 230 criteria. The results of these tests have been summarized in Appendix A.

1.2 Problem Statement

Since the expansion-gap design feature of the Nebraska Open Concrete Bridge Rail does not provide structural continuity, the NDOR Bridge Division was concerned with the structural adequacy of the two concrete posts adjacent to the gap. To address this concern, and the concern for the reduction of steel in the continuous rail section, a second series of full-scale vehicle crash tests were performed by the Midwest Roadside Safety Facility (MwRSF). Engineers were specifically concerned with the potential for large lateral deflections in the posts and the attached

rail adjacent to the gap. It was thought that these deflections could be of sufficient magnitude to cause vehicle snagging on the end of the rail located on the downstream side of the gap.

1.3 Objective

The research project involved full-scale vehicle crash testing on the Nebraska Open Concrete Bridge Rail according to the PL-1 Performance Level, as presented in the AASHTO “Guide Specifications for Bridge Railings” (3). Two full-scale vehicle crash tests were to be conducted at different impact locations: one test to evaluate safety performance, more specifically, the structural adequacy of the rail and posts at the gap location; and a second test to evaluate the structural adequacy of a rail in which the longitudinal steel reinforcement had been reduced for economic considerations.

At the request of the NDOR Bridge Division, the project was expanded to include computer simulation modelling (see Appendix A). The modelling was used to verify the structural inadequacy of the bridge rail design prior to construction and testing, and also to provide information for analysis and redesign of the bridge rail.

1.4 Scope

Two full-scale vehicle crash tests were conducted with 5,400-lb pickups at target impact conditions of 45 mph and 20 degrees, according to the PL-1 Performance Level criteria described in AASHTO (3). The PL-1 Performance Level requires tests with both a 1,800-lb small automobile and a 5400-lb pickup, but the 1,800-lb test had previously been conducted on the open concrete bridge rail by Ensco, Inc. (1), using NCHRP 230 guidelines (2). Therefore the PL-1 test with a 1,800-lb vehicle was not conducted.

2 COMPUTER SIMULATION MODELLING

2.1 BARRIER VII

BARRIER VII (4) was used in the simulation modelling phase of the project. The following factors were used to monitor the structural adequacy of the installation: (1) deflections of the posts and rail, and relative position of vehicle; (2) flexural moments in the rail; (3) flexural moments in the posts; and (4) shear forces in the posts. Simulation runs were conducted both upstream and downstream of the gap in order to determine whether there was adequate strength along the longitudinal axis of the rail (i.e., no excessive deflections nor post "pullout" due to insufficient shear and moment capacity).

2.2 Computer Simulation Impact Conditions

Impact conditions for AASHTO PL-1 Performance Level tests require a 5,400-lb pickup at 45 mph and 20 degrees. Since the BARRIER VII program used by the MwRSF has been calibrated for 4,500-lb test vehicles, a 4,500-lb sedan was used instead of the 5,400-lb pickup. This reduced weight required an increase in either the impact angle or the impact speed in order to provide similar loading conditions; in this case, the impact condition of 20 degrees was increased to 25 degrees in order to provide a conservative estimate for the impact loading.

2.3 Computer Simulation Test Vehicle

A 1977 Plymouth Fury weighing approximately 4,500 lbs was selected as the simulation test vehicle; vehicle dimensions are shown in Figure 1.

2.4 Design No. 1

Two types of reinforced concrete posts were specified in the original design (Design No. 1) of the Nebraska Open Concrete Bridge Rail. Ten small posts measuring 11-in. wide x 11-in. long were used to support the continuous concrete rail, and two large posts measuring

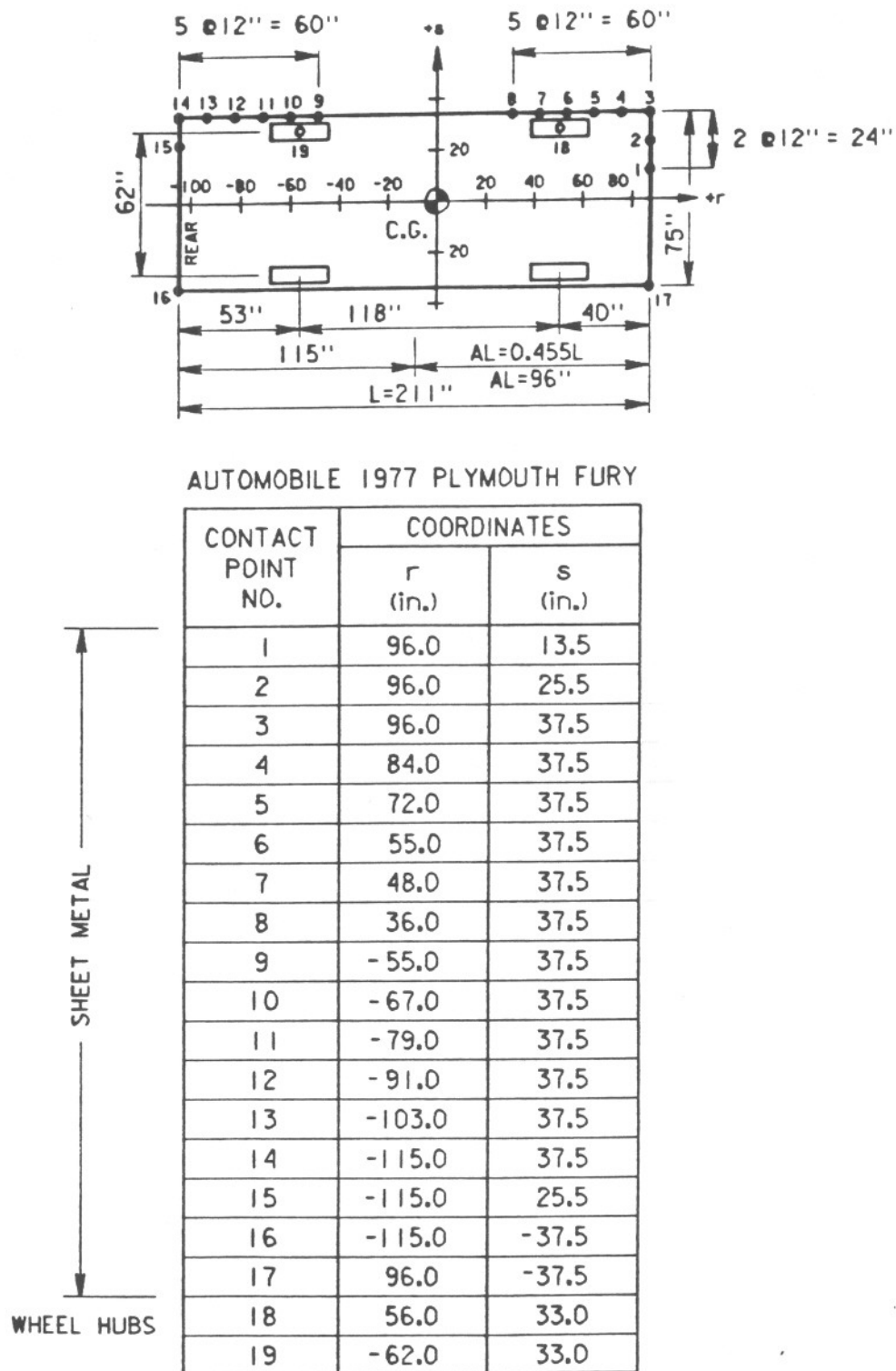


FIGURE 1. Idealization of Simulation Vehicle

11-in. wide x 1-ft 10-in. long were located adjacent to the 3-in. expansion gap. The layout of the original design is shown in Figure 2.

As shown in Figure 2, Design No.1 called for two small posts and one large post to be constructed upstream of the gap. This scenario would provide the worst case conditions and any subsequent analysis and redesign would provide conservative results. The steel reinforcement in the small posts consisted of three No. 6 bars in the traffic side face and two No. 6 bars in the back side face of the post (Figure 3). The steel reinforcement in the large posts consisted of four No. 6 bars in both the traffic side and back side faces of the post (Figure 3). The 1-ft 2-in. wide by 1-ft 4-in. deep bridge rail contained six No. 5 reinforcing bars, three bars in both the traffic and back side faces of the rail (Figure 3).

A total of eight computer simulation runs (Run Nos. 1 through 8) were conducted upstream and downstream of the 3-in. gap; the impact locations are shown in Figure 4.

Location from Centerline of Gap		
<u>Run Number</u>	<u>Node Number</u>	<u>Distance (ft)</u>
1	5	9-ft 7½-in.
2	6	7-ft 7½-in.
3	7	5-ft 7½-in.
4	8	3-ft 7½-in.
5	9	1-ft 7½-in.
6	10	0-ft 1½-in.
7	12	1-ft 1½-in.
8	13	3-ft 7½-in.

The structural quantities for the two post sizes and the rail are presented in Table 1. An in-house Fortran computer program (Appendix B) was used to calculate the yield and ultimate moment in a beam or post element for a given strain in the compressive face of the concrete surface.

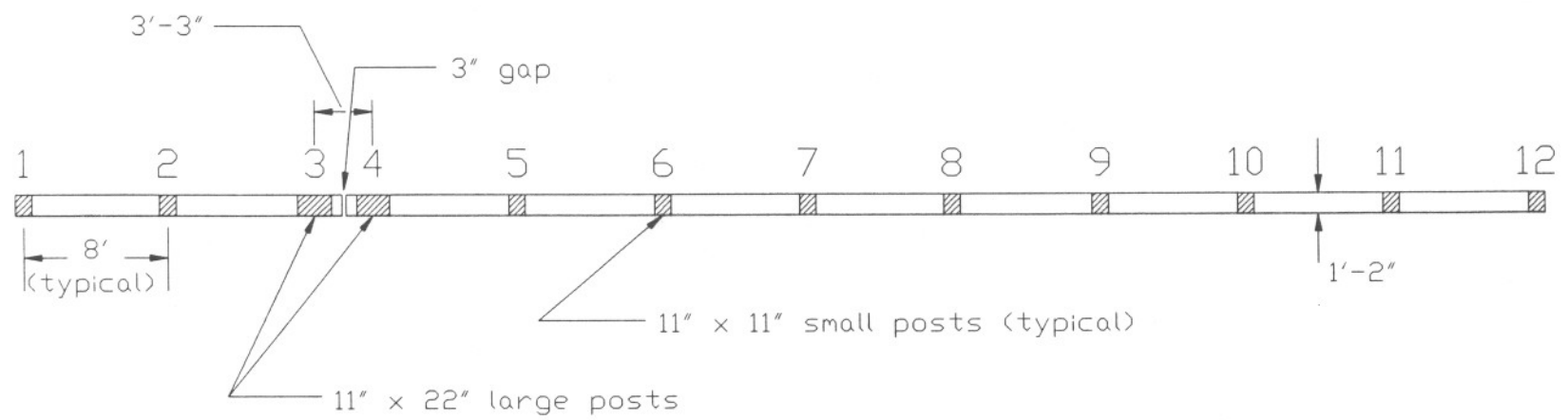
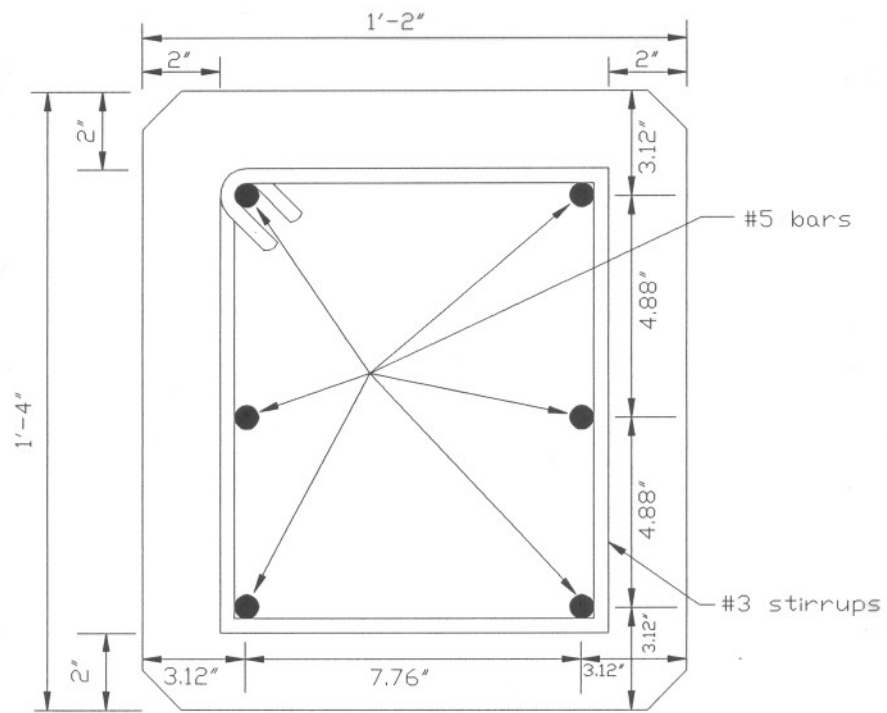
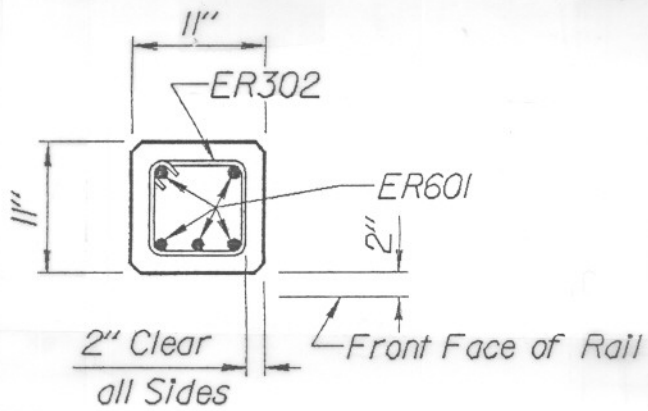


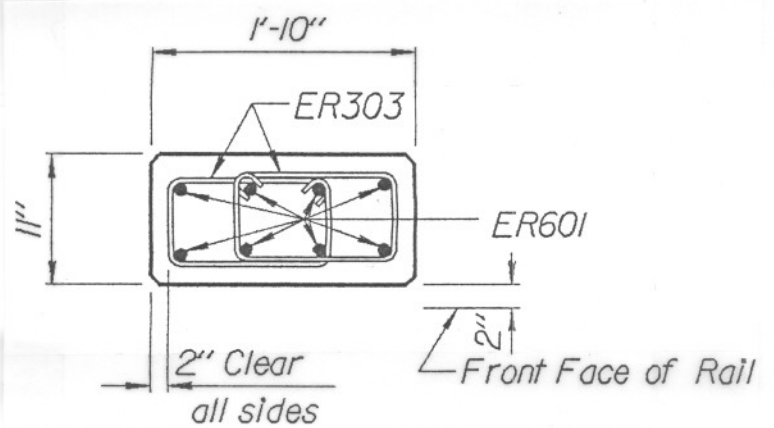
FIGURE 2. Nebraska Open Concrete Bridge Rail, Design No.1



Rail



Small Post



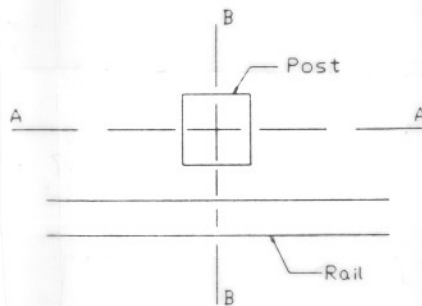
Large Post

FIGURE 3. Design No.1 - Posts and Rail

TABLE 1. Structural Quantities for Computer Simulation - Design No. 1

Member Type	Member Size	Steel Reinforcement	Stiffness Along (kips/in.)	Ultimate Base Moment About (kip-in.)	Ultimate Shear Along (kips) ¹	Tensile Force (kips)	Ultimate Bending Moment (kip-in.)	Effective Moment of Inertia (in. ⁴)	Modulus of Elasticity (ksi)
Post	11-in. x 11-in. (small)	5- #6's	A-axis 390	A-axis 612	A-axis 25 (30.8)	NA	NA	A-axis 361	NA
			B-axis 501	B-axis 539	B-axis 29.2 (34.6)			B-axis 281	
Post	11-in. x 22-in. (large)	8- #6's	A-axis 2,522	A-axis 908	A-axis 57.7	NA	NA	A-axis 498	NA
			B-axis 692	B-axis 1,940	B-axis 43.2 (47.5)			B-axis 1,816	
Beam	14-in. x 16-in.	6- #5's	NA	NA	NA	116	691	NA	4,031

¹ The computed values for ultimate shear were calculated as the minimum of the two methods: (1) from $V=M/h$ and (2) from Reinforced Concrete Design, 4th Ed., Wang and Salmon. If the governing value for shear was based upon (1), then the shear capacity as calculated by (2) is reported in parentheses.



The analysis of the eight computer simulation runs on Design No. 1 indicated that the design was structurally inadequate and did not provide sufficient strength along the longitudinal axis (A-axis) of the rail upstream of the gap. The results are summarized in Table 2. This conclusion was as expected, since the original design (Design No.1) specified only two small posts upstream of the gap; whereas, in fact, the Nebraska Open Concrete Bridge Rail is constructed with a concrete abutment or end-section to carry large impact loads. As stated earlier, Design No. 1, the worst-case scenario, was used as a baseline for testing because the NDOR Bridge Division had opted to be conservative.

Analysis of the computer simulation runs conducted at nodes (7-10) and (12-13) indicated that the large posts adjacent to the gap were structurally inadequate in both the longitudinal and transverse directions. In addition, the computer run at node 10 revealed that the small posts were inadequate in the downstream rail section (i.e., failure of Post No. 5 about the A-axis). Computer simulations revealed excessive beam moments at various locations which are identified in Table 2.

The original design (Design No. 1) was subsequently modified in an attempt to increase the structural adequacy of the system. The modifications were incorporated into Design No. 2.

2.5 Design No. 2

The modifications to Design No.1 addressed the following objectives: (1) increasing the shear strength of the large posts adjacent to the gap in both the longitudinal and transverse directions and (2) increasing the moment and shear capacities of the small posts upstream of the gap in order to prevent post “pullout” or failure along the longitudinal axis of the rail.

One possibility was to use a constant post size along the entire length of the installation. An 11-in. wide x 24-in. long post size was selected by MwRSF (Figure 5) and approved by the

TABLE 2. Summary of Simulation Results - Design No. 1

Impact Location	Design No. 1 Simulation Results
Node 5	1. Post No. 2 failed about A-axis. 2. Post No. 1 failed about B-axis. (Failure)
Node 6	1. Post No. 2 failed about A-axis and B-axis. 2. Post No. 1 failed about B-axis. 3. Excessive beam moment at node 7. (Failure)
Node 7	1. Post No. 4 failed about A-axis. 2. Potential snagging at gap. (Failure)
Node 8	1. Post No. 4 failed about A-axis. 2. Potential snagging at gap. 3. Excessive beam moment at nodes 8 and 16. (Failure)
Node 9	1. Large moment in Post No. 4 about A-axis. 2. Excessive beam moment at node 16. (Very Marginal)
Node 10	1. Post No. 5 failed about A-axis. 2. Large moment in Post No. 4 about A-axis. (Failure)
Node 12	1. Post No. 4 failed about B-axis. (Failure)
Node 13	1. Post No. 4 failed about B-axis. 2. Large moment in Post No. 5 about A-axis. 3. Large moment in Post No. 6 about A-axis. 4. Excessive beam moment at nodes 14 and 20. (Failure)

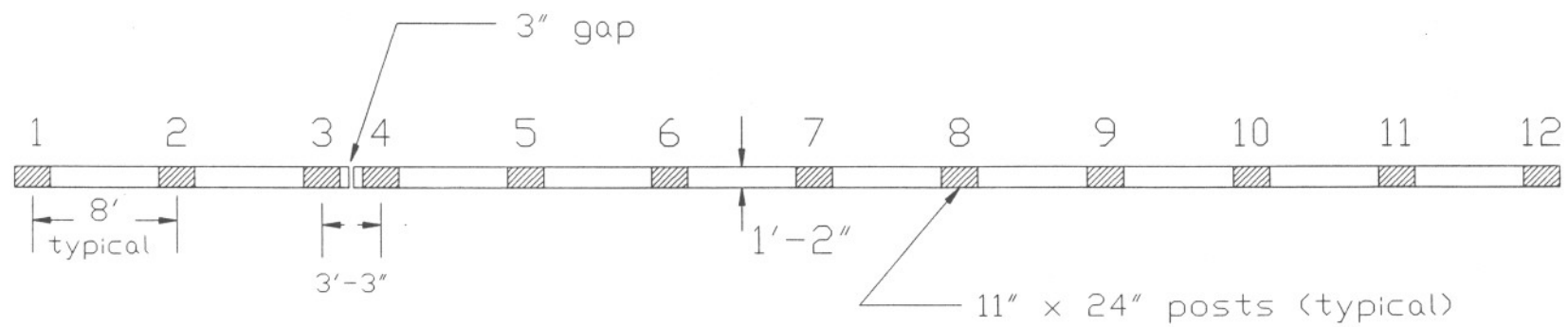


FIGURE 5. Nebraska Open Concrete Bridge Rail (Design No.2 and No. 3)

NDOR Bridge Division. The 11-in. x 11-in. small posts were not desirable because of the difficulty of placing the vertical reinforcement and stirrups in the posts and still providing sufficient concrete cover. Construction of the 11-in. x 11-in. posts on skewed bridges is even more difficult.

The steel reinforcement in the 11-in. x 24-in. large posts consisted of four No. 6 bars in the traffic side face and three No. 4 bars in the back side face of the post (Figure 6). Six No. 5 bars were used for the bridge rail reinforcement.

A total of eight computer simulation runs (Run Nos. 9 through 16) were conducted upstream and downstream of the 3-in. gap. Impact locations were the same as for Design No. 1. The structural quantities for the post and rail simulations are presented in Table 3.

Analysis of the eight computer simulation runs on Design No. 2 indicated that the posts adjacent to the gap were structurally inadequate; however, test results for two runs at nodes 5 and 6 showed that the structural adequacy of the posts upstream of the gap had improved. The results are summarized in Table 4.

The posts adjacent to the gap failed in both the longitudinal and transverse directions. The computer run at node 13 revealed a failure of Post No. 5 about the A-axis. This failure was mainly due to the structural inadequacy of Post No. 4 about the B-axis. Computer simulations also revealed excessive beam moments at various locations which are identified in Table 4.

Design No. 2 was then modified in an attempt to increase the structural adequacy of the posts adjacent to the gap. The modifications were incorporated into Design No. 3.

2.6 Design No. 3

The intent of the Design No. 2 modification was to increase the moment capacity of the 11-in. x 24-in. posts adjacent to the gap. Therefore, the Design No.2 post was modified.

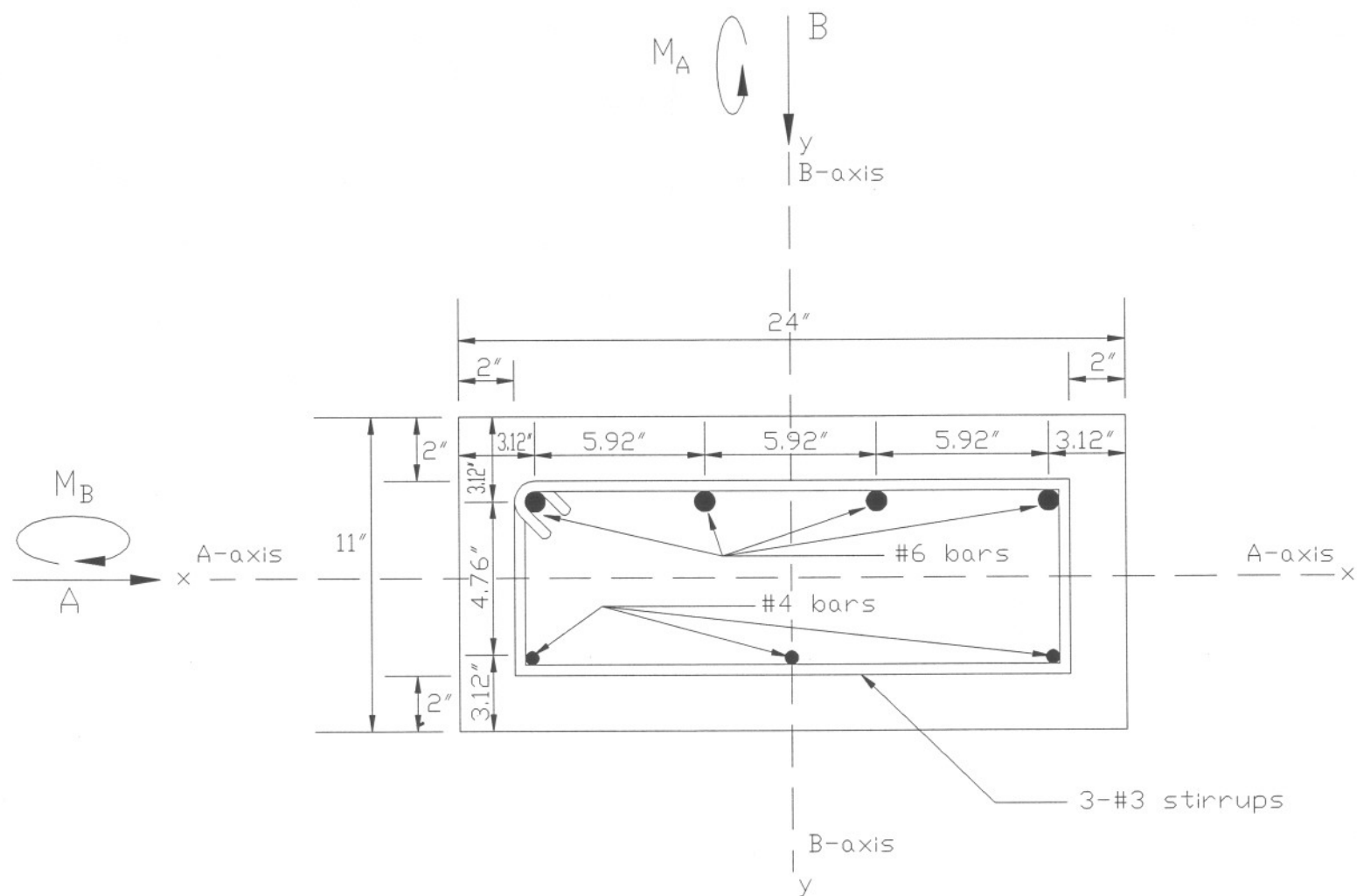


FIGURE 6. Design No. 2 Strong Post (11-in. x 24-in.)

TABLE 3. Structural Quantities for Computer Simulation - Design No. 2

Member Type	Member Size	Steel Reinforcement	Stiffness Along (kips/in.)	Ultimate Base Moment About (kip-in.)	Ultimate Shear Along (kips) ¹	Tensile Force (kips)	Ultimate Bending Moment (kip-in.)	Effective Moment of Inertia (in. ⁴)	Modulus of Elasticity (ksi)
Post	11-in. x 24-in. (large)	4- #6's 3- #4's	A-axis 3,507	A-axis 879	A-axis 64.2	NA	NA	A-axis 537	NA
			B-axis 745	B-axis 1,488	B-axis 41.8 (47.6)			B-axis 2,525	
Beam	14-in. x 16-in.	6- #5's	NA	NA	NA	116	691	NA	4,031

¹ The computed values for ultimate shear were calculated as the minimum of the two methods: (1) from $V=M/h$ and (2) from Reinforced Concrete Design, 4th Ed., Wang and Salmon. If the governing value for shear was based upon (1), then the shear capacity as calculated by (2) is reported in parentheses.

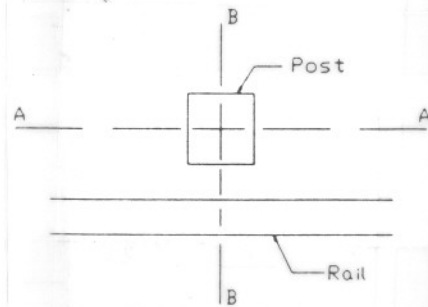


TABLE 4. Summary of Simulation Results - Design No. 2

Impact Location	Design No. 1 Simulation Results
Node 5	1. Post No. 3 failed about B-axis. 2. Large moment in Post No. 3 about A-axis. (Failure)
Node 6	1. Post No. 4 failed about B-axis. 2. Excessive beam moment at node 7. (Failure)
Node 7	1. Large moment in Post No. 3 about A-axis. 2. Potential snagging at gap. (Very Marginal)
Node 8	1. Post No. 3 failed about A-axis. 2. Post No. 4 failed about A-axis. 3. Potential snagging at gap. (Failure)
Node 9	1. Post No. 4 failed about B-axis. 2. Excessive beam moment at node 16. (Failure)
Node 10	1. Excessive beam moment at node 16. 2. Large deflection at Post No. 4. (Failure)
Node 12	1. Post No. 4 failed about B-axis. 2. Excessive beam moment at node 16. (Failure)
Node 13	1. Post No. 5 failed about A-axis. 2. Post No. 4 failed about B-axis. 3. Excessive beam moment at node 15. (Failure)

The steel reinforcement for the Design No.3 posts adjacent to the gap consisted of four No. 6 bars in both the traffic and back side faces of the post (Figure 7). The reinforcement in the posts which were not adjacent to the gap consisted of four No. 6 bars in the traffic side face and three No. 4 bars in the back side face of the post (Figure 6). Six No. 5 bars were used for the bridge rail reinforcement.

Computer simulation runs (Run Nos. 17 and 18) were conducted at two critical locations upstream of the 3-in. gap; impact locations were at nodes 7 and 8. The structural quantities for the two post types and the rail are presented in Table 5.

The results of the two computer simulation runs on Design No. 3 indicated that the posts adjacent to the gap were structurally inadequate. Results indicated that the discontinuity in the rail, due to the expansion gap, would require a further increase in the shear and moment capacity about the A-axis (in the transverse direction). Test results are summarized in Table 6.

Design No. 3 was subsequently modified to increase the strength of the posts at the gap location in the transverse direction, and to reduce the deflections of the post and rail upstream of the gap.

2.7 Design No. 4

Design No. 4 incorporated an increase in the size of the post adjacent to the gap from 11-in. wide x 24-in. long to 11-in. wide x 36-in. long. Vertical reinforcement was also increased (Figure 8).

The steel reinforcement in the 11-in. x 36-in. posts consisted of five No. 6 bars in both the traffic and back side faces of the post (Figure 9). The reinforcement in the 11-in. x 24-in. posts not adjacent to the gap consisted of four No. 6 bars in the traffic side face and three No.4 bars in the back side face of the post (Figure 6). Six No. 5 bars were used for bridge rail

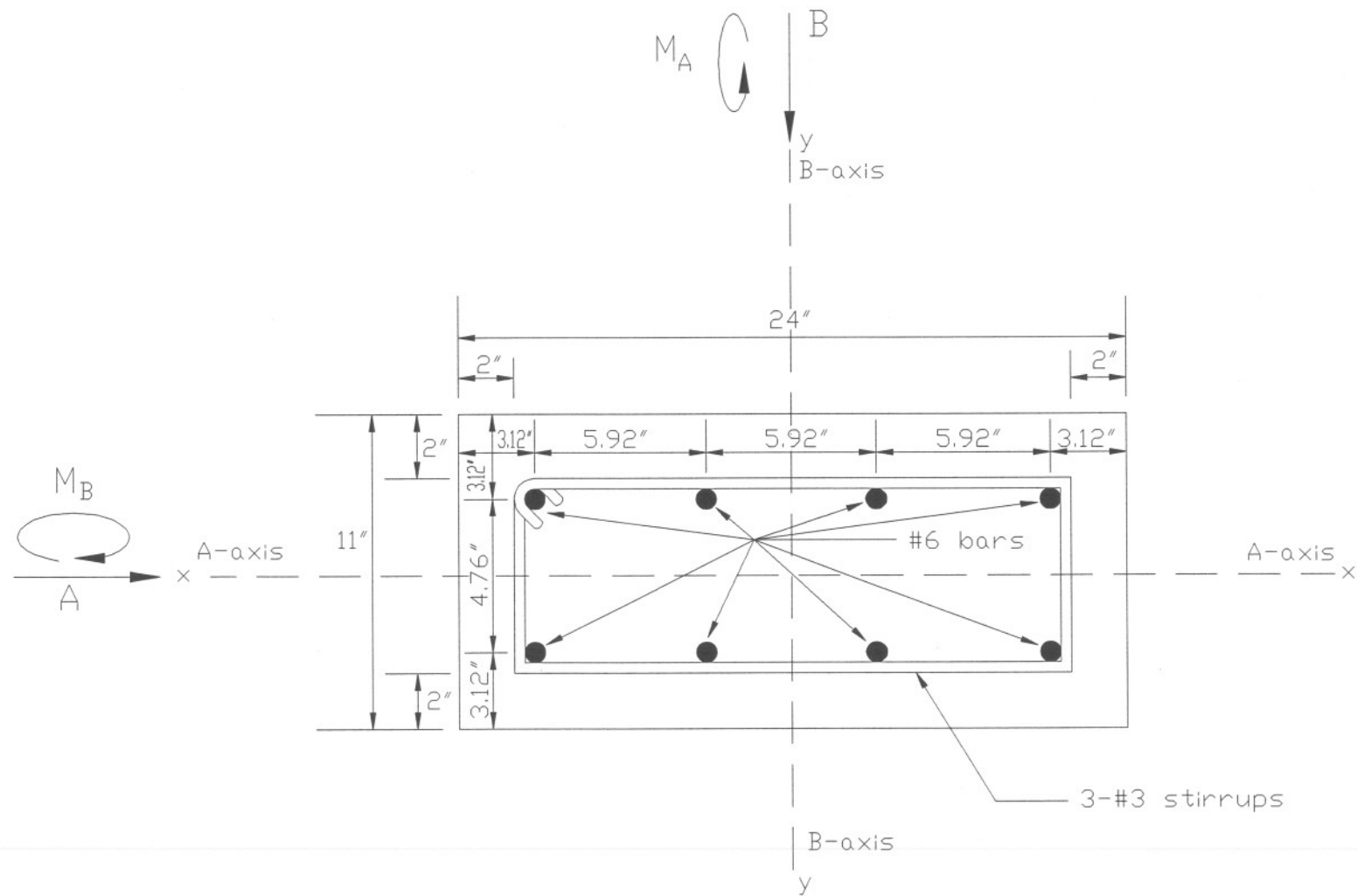


FIGURE 7. Design No. 3 Strong Post at Gap (11-in. x 24-in.)

TABLE 5. Structural Quantities for Computer Simulation - Design No. 3

Member Type	Member Size	Steel Reinforcement	Stiffness Along (kips/in.)	Ultimate Base Moment About (kip-in.)	Ultimate Shear Along (kips) ¹	Tensile Force (kips)	Ultimate Bending Moment (kip-in.)	Effective Moment of Inertia (in. ⁴)	Modulus of Elasticity (ksi)
Post	11-in. x 24-in. (large)	4- #6's 3- #4's	A-axis 3,507	A-axis 879	A-axis 64.2	NA	NA	A-axis 537	NA
			B-axis 745	B-axis 1,488	B-axis 41.8 (47.6)			B-axis 2,525	
Post	11-in. x 24-in. (modified)	8- #6's	A-axis 4,287	A-axis 937	A-axis 62.8	NA	NA	A-axis 530	NA
			B-axis 736	B-axis 2,128	B-axis 44.6 (47.5)			B-axis 3,087	
Beam	14-in. x 16-in.	6- #5's	NA	NA	NA	116	691	NA	4,031

¹ The computed values for ultimate shear were calculated as the minimum of the two methods: (1) from $V=M/h$ and (2) from Reinforced Concrete Design, 4th Ed., Wang and Salmon. If the governing value for shear was based upon (1), then the shear capacity as calculated by (2) is reported in parentheses.

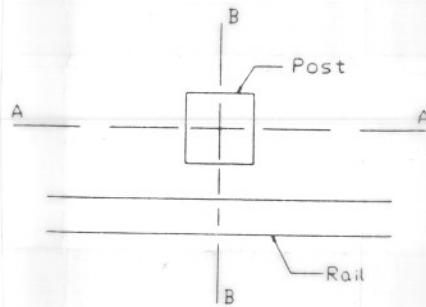


TABLE 6. Summary of Simulation Results - Design No. 3

Impact Location	Design No. 3 Simulation Results
Node 7	<ol style="list-style-type: none"> 1. Post No. 4 failed about A-axis. 2. Potential snagging at gap. (Failure)
Node 8	<ol style="list-style-type: none"> 1. Post No. 4 failed about A-axis. 2. Potential snagging at gap. 3. Excessive beam moment at node 8. (Failure)

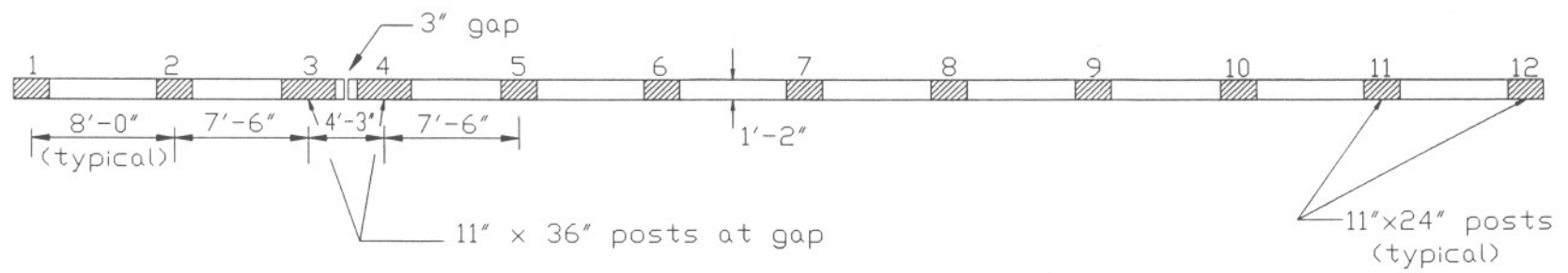


FIGURE 8. Modification of Design No. 3

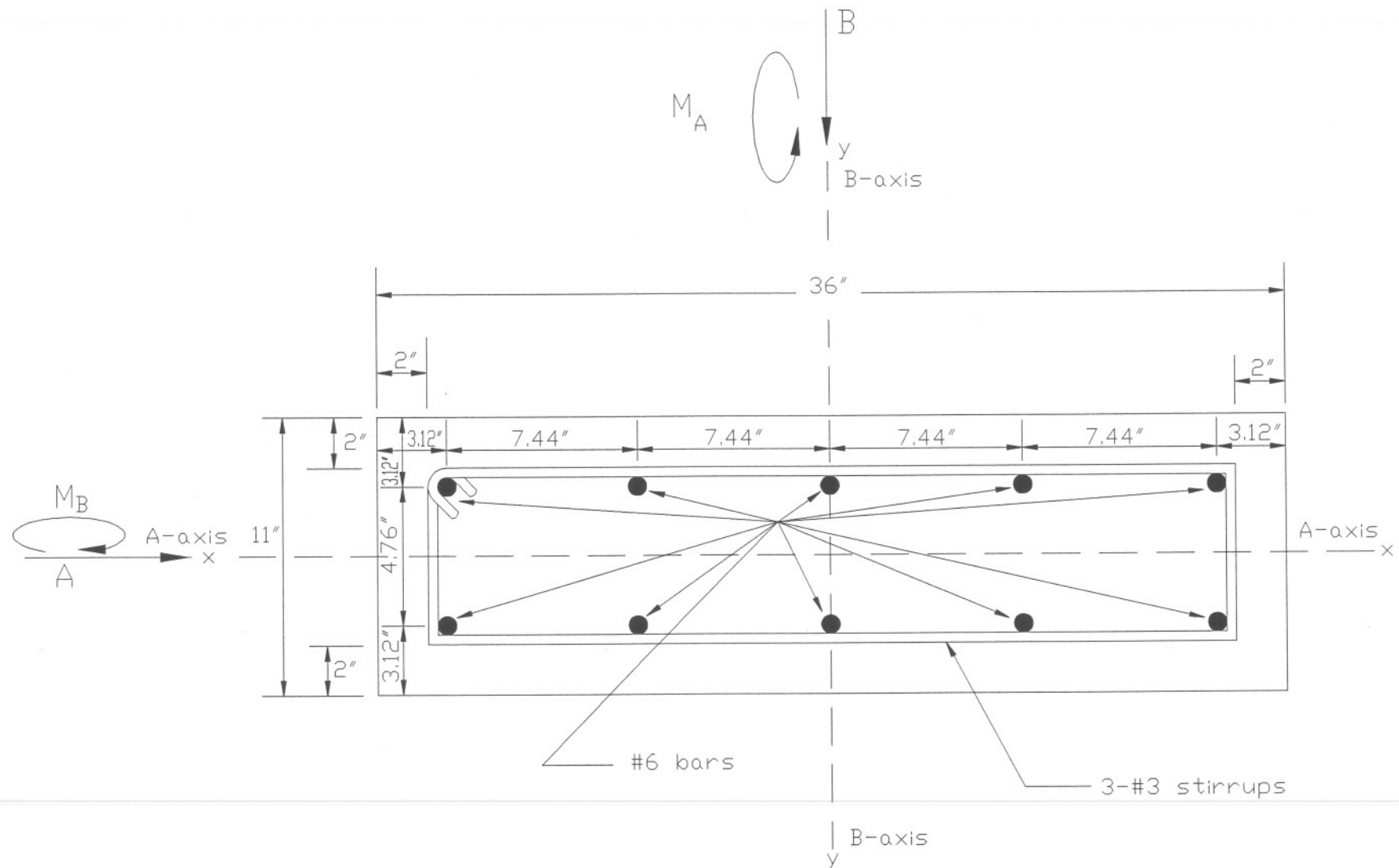


FIGURE 9. Design No. 4 Strong Post at Gap (11-in. x 36-in.)

reinforcement.

Two computer simulation runs (Run No. 19 and 20) were conducted upstream of the 3-in. gap. Impact locations were at nodes 7 and 8 (Figure 10). The structural quantities for the two post sizes and the rail are presented in Table 7.

Location from Centerline of Gap		
<u>Run No.</u>	<u>Node No.</u>	<u>Distance (ft)</u>
19	7	5-ft 10½-in.
20	8	4-ft 0-in.

Analysis of the two computer simulation runs on Design No. 4 indicated that the strength of the posts adjacent to the gap was adequate. The results are summarized in Table 8. Although there remained a potential for snagging due to excessive deflection of Post No. 3, no additional modifications were made to the post sizes or reinforcement. However, because Run No. 20 revealed excessive beam moment at node 9, the reinforcement in the rail near the gap location was increased to six No. 6's which was NDOR's existing standards for the concrete bridge rail (Figures 11 and 12).

In order to verify the adequacy of using six No. 5 bars in the rail when structural continuity exists, one additional computer simulation run (Run No. 21) was conducted at a downstream continuous rail location (node 30, refer to Figure 4). The results of the simulated impact at node 30 showed excessive beam moments at nodes 28 and 32. Although failure of the rail was predicted, the design was not modified.

The final design as constructed in the field and full-scale vehicle crash tested is discussed in Chapter 3. The plan view of this final design is shown in Figure 11.

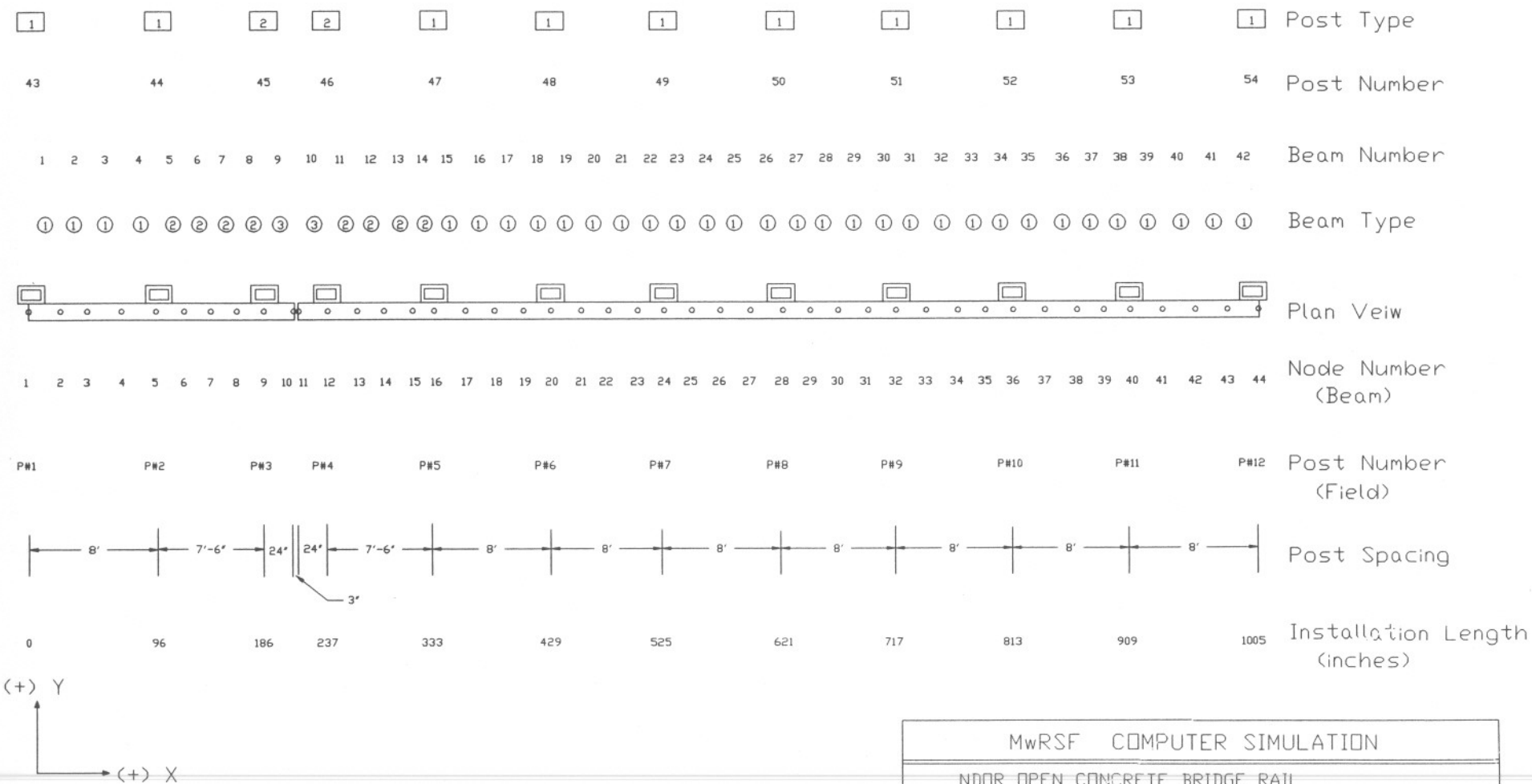


FIGURE 10. Computer Simulation Model Configuration, Design No. 4

TABLE 7. Structural Quantities for Computer Simulation - Design No. 4

Member Type	Member Size	Steel Reinforcement	Stiffness Along (kips/in.)	Ultimate Base Moment About (kip-in.)	Ultimate Shear Along (kips) ¹	Tensile Force (kips)	Ultimate Bending Moment (kip-in.)	Effective Moment of Inertia (in. ⁴)	Modulus of Elasticity (ksi)
Post	11-in. x 24-in. (large)	4- #6's 3- #4's	A-axis 3,507 B-axis 745	A-axis 879 B-axis 1,488	A-axis 64.2 B-axis 41.8 (47.6)	NA	NA	A-axis 537 B-axis 2,525	NA
Post	11-in. x 36-in. (gap)	10- #6's	A-axis 23,503 B-axis 1,018	A-axis 1,247 B-axis 4,176	A-axis 91.1 B-axis 59.4 (60.9)	NA	NA	A-axis 733 B-axis 16,925	NA
Beam	14-in. x 16-in.	6- #5's	NA	NA	NA	116	691	NA	4,031

¹ The computed values for ultimate shear were calculated as the minimum of the two methods: (1) from $V=M/h$ and (2) from Reinforced Concrete Design, 4th Ed., Wang and Salmon. If the governing value for shear was based upon (1), then the shear capacity as calculated by (2) is reported in parentheses.

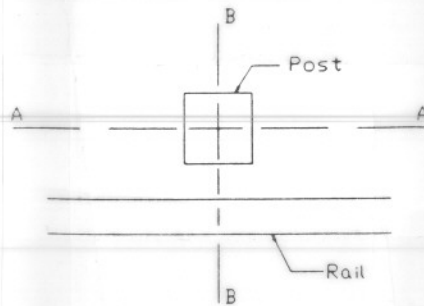
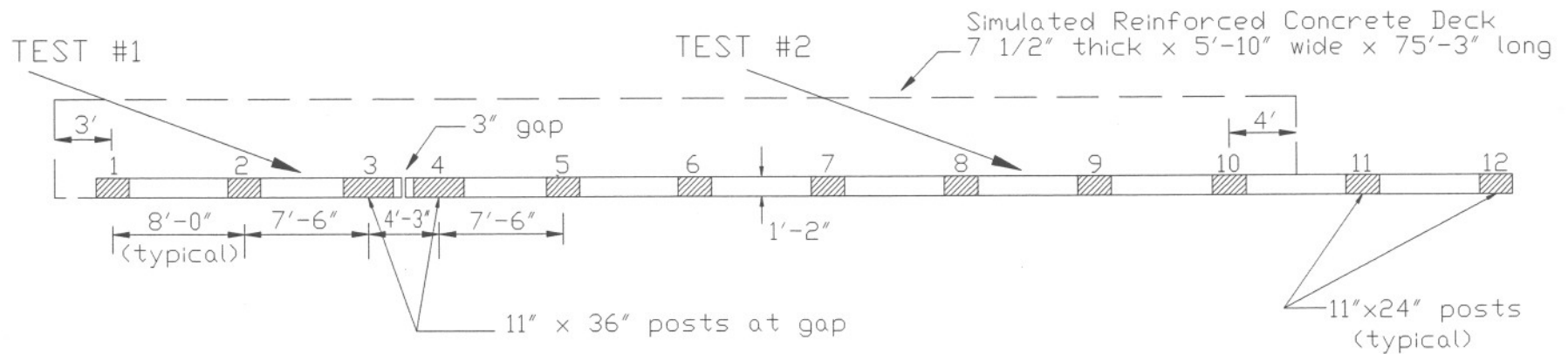


TABLE 8. Summary of Simulation Results - Design No. 4

Impact Location	Design No. 4 Simulation Results	
Node 7	1. Potential snagging at gap. (Marginal)	
Node 8	1. Potential snagging at gap. 2. Excessive beam moment at node 9. (Failure)	



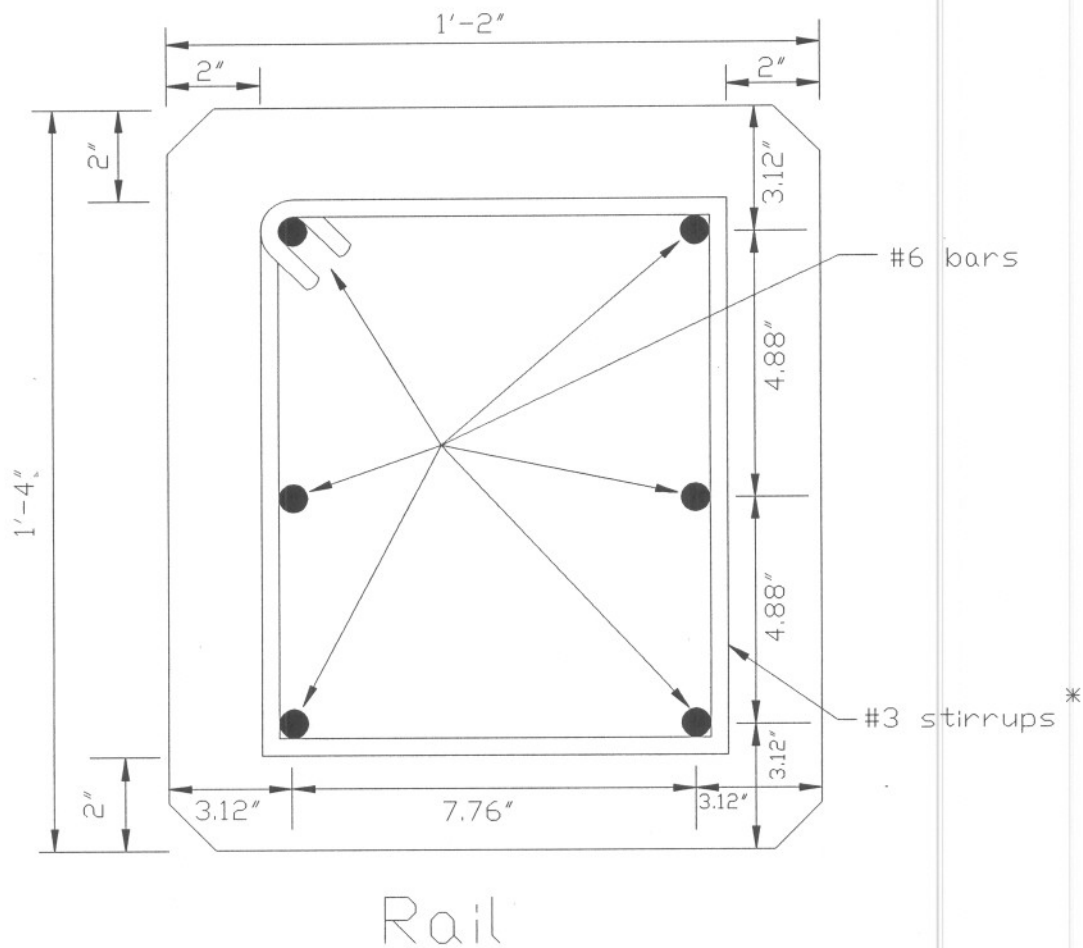
TEST #1 - 5400#, 45mph, 20°, 5'-10 1/2" upstream from centerline of gap

TEST #2 - 5400#, 45mph, 20°, midway between posts #8 and 9

Note: Following the computer simulation runs on Design No.4 (6-#5's near the gap), the longitudinal reinforcement in the rail was increased to 6-#6's which is also NDOR's existing standards.

Note: Longitudinal Beam Steel
 Post#1-6: 6-#6's (see footnote)
 Between Post#6 and 7:
 Lap splice #5's and #6's
 Post#7-12: 6-#5's
 Total Rail Length: 85'-3"

FIGURE 11. Nebraska Open Concrete Bridge Rail, Design No. 4



*Note: Stirrup spacing varies. See bridge rail design details.

FIGURE 12. Reinforcement Modification to Design No. 4

3 TEST CONDITIONS

3.1 Test Facility

3.1.1 Test Site

The test site facility is located at the Lincoln Air-Park on the NW end of the Lincoln Municipal Airport. The test facility is approximately 5 mi. NW of the University of Nebraska-Lincoln. The site is surrounded and protected by an 8-ft high chain-link security fence.

3.1.2 Vehicle Tow System

A reverse cable tow with a 1:2 mechanical advantage was used to propel the test vehicle. The distance traveled and the speed of the tow vehicle are one-half that of the test vehicle. A sketch of the cable tow system is shown in Figure 13. The test vehicle is released from the tow cable before impact with the bridge rail. The tow vehicle and the attached fifth-wheel are shown in Figure 14. The fifth wheel, built by the Nucleus Corporation, was used in conjunction with a digital speedometer to increase the accuracy of the test vehicle impact speed.

3.1.3 Vehicle Guidance System

A vehicle guidance system developed by Hinch (5) was used to steer the test vehicle. The guidance system is shown in Figure 13. A guide flag attached to the front left wheel and the guide cable was sheared off before impact. The 3/8-in. diameter guide cable was tensioned to 3,000 lbs, and supported laterally and vertically every 100 ft by hinged stanchions. The hinged stanchions stood upright while holding up the guide cable, but as the vehicle was towed down the line, the guide-flag struck and knocked each stanchion to the ground. The vehicle guidance system was 1,500-ft long for both tests.

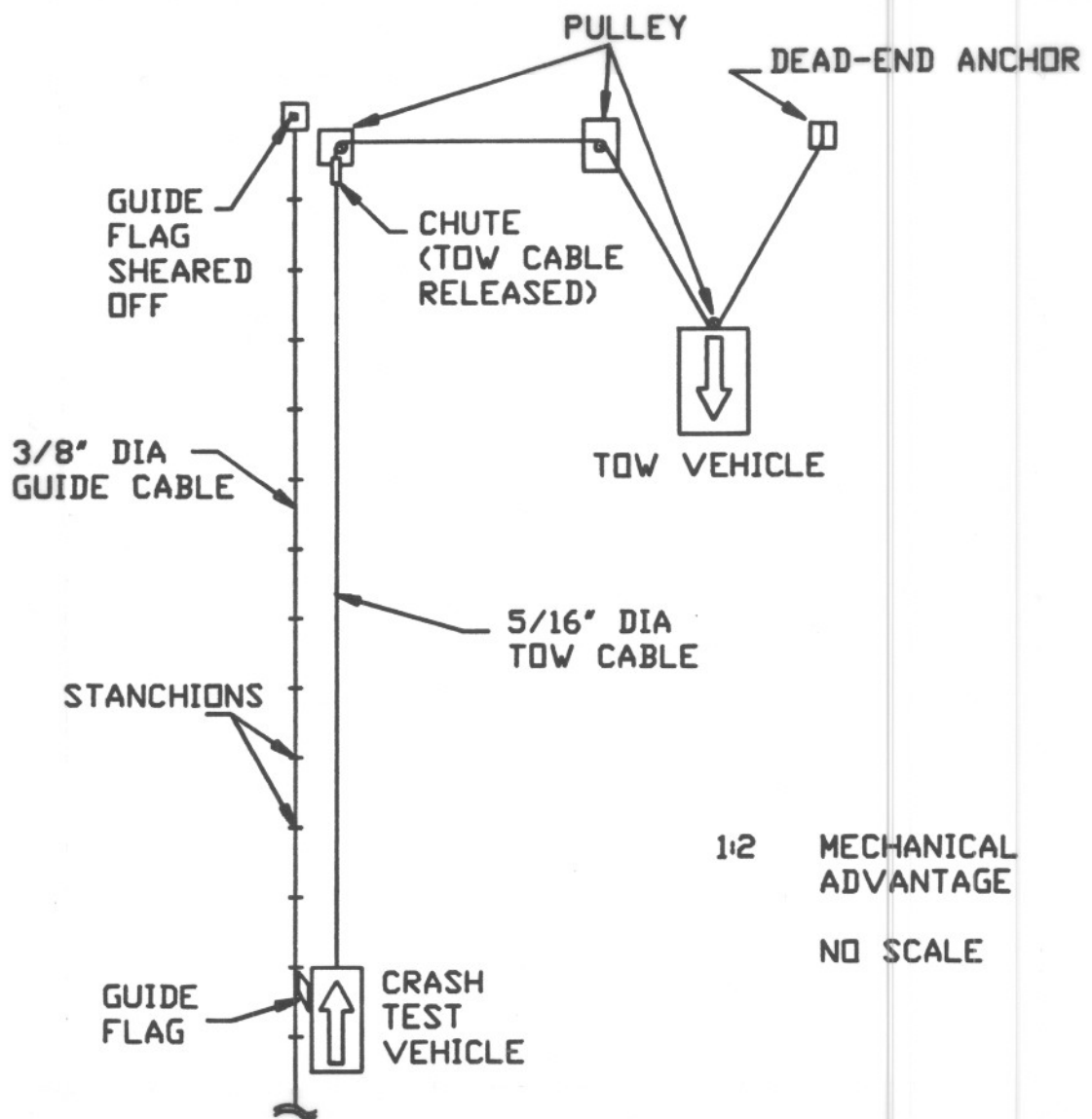


FIGURE 13. Cable Tow and Guidance System.



FIGURE 14. Tow Vehicles and Fifth Wheel

3.2 Nebraska Open Concrete Bridge Rail Design Details

A detailed drawing of the Nebraska Open Concrete Bridge Rail is shown in Figure 15. Photographs of the actual installation are shown in Figure 16. The total length of the installation was 85-ft 3-in. The installation consisted of three major structural components: (1) simulated concrete bridge deck, (2) concrete posts, and (3) concrete bridge rail. The design details for each of these components are shown in Figure 15.

The installation was constructed with a simulated bridge deck in order to test the post-to-deck connection as well as the rail itself. The length of the bridge deck was 75-ft 3-in. The 7½-in. thick deck had a total width of 5-ft 10-in., producing a 3-ft 1-in. cantilever. The deck was reinforced with two No. 5 transverse bars spaced at 4½-in. and 7-in. on the top and bottom rows, respectively. Two and one-half inches of clear cover was available on the top bar, and 1 in. on the bottom bar. Two longitudinal bars were placed between the transverse bars and spaced at 12-in. centers. The transverse bars were attached to the existing concrete apron. This connection detail is also shown in Figure 17. Grade 60 epoxy-coated reinforcement was used in the deck. The reinforcement layout for the bridge deck is shown in Figure 18, details are shown in Figure 15.

The second major component of the installation was the concrete bridge posts. Twelve reinforced concrete posts were constructed to support the reinforced concrete rail, as shown in Figure 15. Two post sizes were incorporated in the installation: ten 11-in. wide x 2-ft long x 1-ft 1-in. high posts were used to support the continuous rail; two 11-in. wide x 3-ft long x 1-ft 1-in. high posts were placed at the gap location in the rail. The post spacing between the first and second posts, and between the fourth through the twelfth posts, was 8-ft 0-in. on centers. The post spacing between the second and third posts, and the fourth and fifth posts, was 7-ft 6-in.

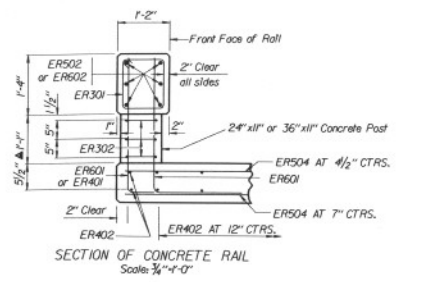
May 16, 1991 13:01:39

BRGT-USR/CAB/RAILTEST.DGN

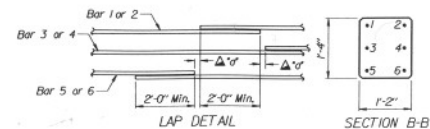
AUG 90

Sheet B I-4061 RAILTEST.DGN

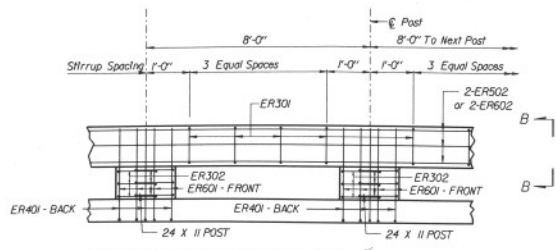
1



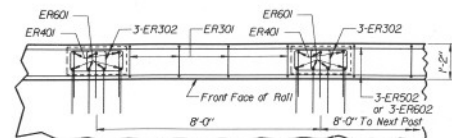
NOTE: SLAB SHALL BE SIMULATED DECK WITH 3'-5" CANTILEVER. DECK SHALL BE 7 1/2" UNIFORM THICKNESS.



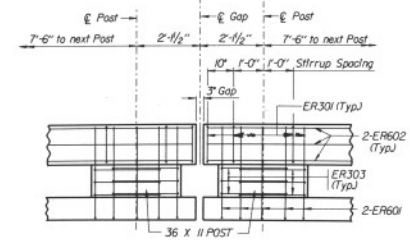
▲ Laps for Bars 1 & 5 shall be staggered
Laps for Bars 2 & 6 shall be staggered
Bar 3 to be continuous through laps for Bars 1 & 5
Bar 4 to be continuous through laps for Bars 2 & 6
"d" ≥ Zero



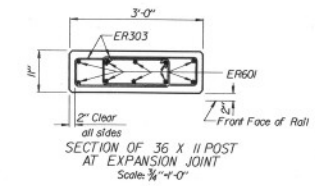
PART ELEVATION OF CONCRETE RAIL
Scale 1/2"-1'-0"



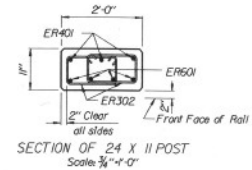
PART PLAN OF CONCRETE RAIL
Scale 1/2"-1'-0"



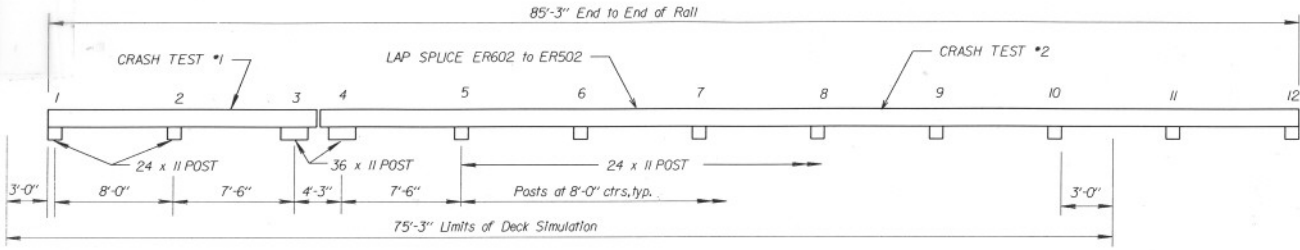
PART ELEVATION AT EXPANSION JOINT
Scale 1/2"-1'-0"



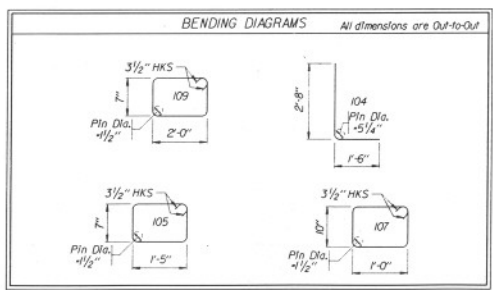
SECTION OF 36 X 11 POST
AT EXPANSION JOINT
Scale 3/4"-1'-0"



SECTION OF 24 X 11 POST
Scale 3/4"-1'-0"



▲ All dimensions marked thus are measured at Front Face of Rail.



SLAB BARS
ER504 BARS AT 4 1/2" TRANSVERSE TOP (2 1/2" CLEAR)
ER504 BARS AT 7" TRANSVERSE BOTTOM (1" CLEAR)
ER402 BARS AT 12" LONGITUDINAL TOP AND BOTTOM

CONCRETE RAIL BILL OF BARS			
MARK	NO.	LENGTH	TYPE
ER601		4'-2"	104
ER602			STR.
ER502			STR.
ER504			STR.
ER401		4'-2"	104
ER402			STR.
ER301		4'-3"	107
ER302		4'-7"	105
ER303		5'-9"	109
INCLUDES 2'-0" LAPS			

NEBRASKA DEPARTMENT OF ROADS BRIDGE DIVISION	
STATE ROAD HWY. NO. REF. POST COUNTY	SKED DESIGN ROADWAY STA.
CONCRETE RAIL FOR CRASH TEST	
DESIGNED BY DEPT. BRIDGE ENGINEER C. J. Johnson	SPECIAL PLAN NO. 1
DETAILED BY DEPT.	
CHECKED BY DEPT.	
DATE 16 MAY 91	PROJECT NUMBER

FIGURE 15. Bridge Rail Design Details



FIGURE 16. Nebraska Open Concrete Bridge Rail

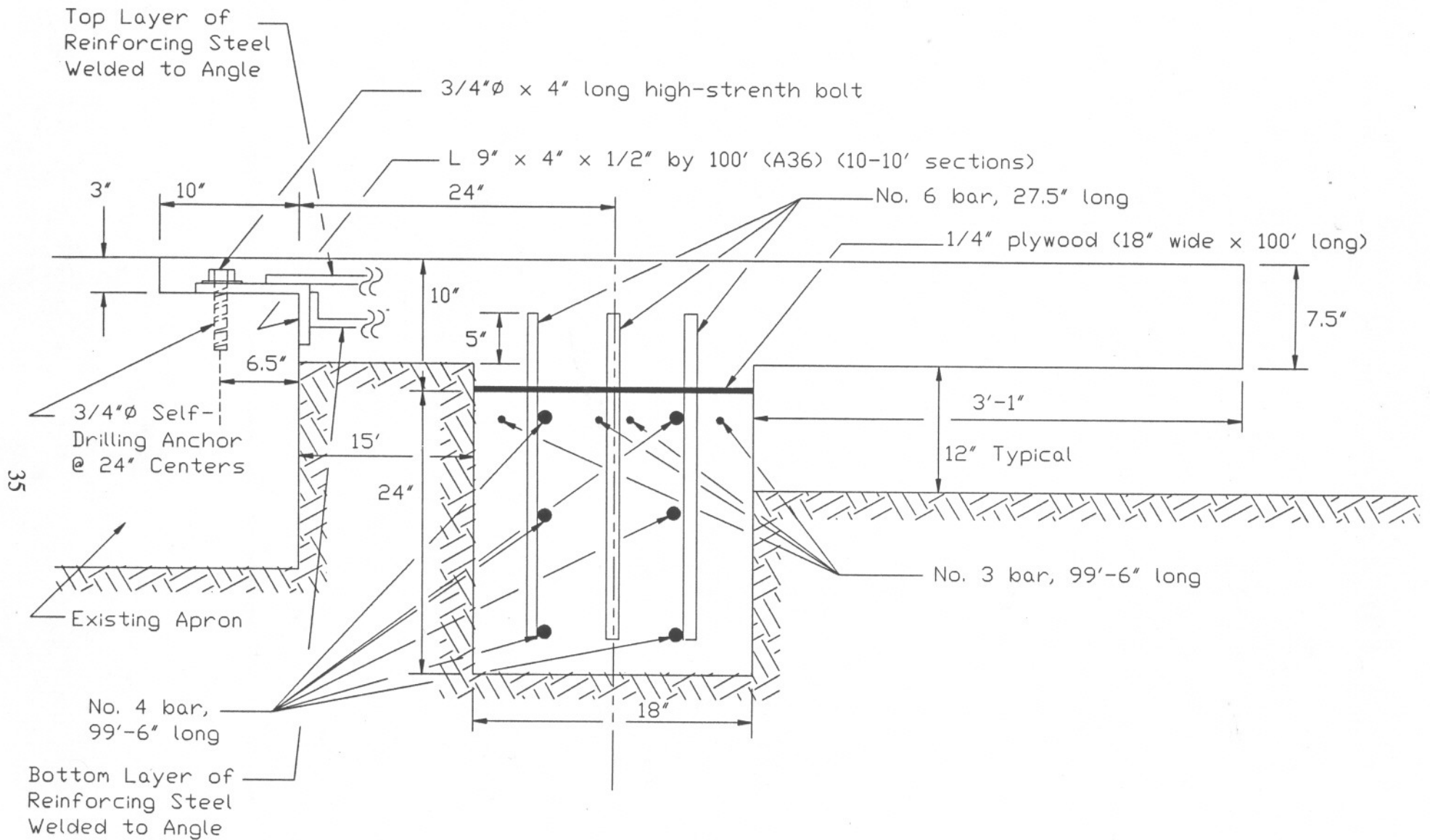


FIGURE 17. Concrete Deck Attachment Detail



FIGURE 18. Reinforcement Layout for Bridge Deck

on centers. The spacing between the third and fourth posts was 4-ft 3-in. The 11-in. x 24-in. posts were reinforced with three No. 4 bars and four No. 6 bars in the back side and traffic side locations of the post, respectively. The 11-in. x 36-in. posts were reinforced with five No. 6 bars in both the back side and traffic side locations of the post, as shown in Figure 19. Grade 60 epoxy-coated reinforcement was used in the posts.

The third major component of the installation was the concrete bridge rail. The construction of the rail is shown in Figure 20. The bridge rail was 1-ft 2-in. wide x 1-ft 4-in. deep x 85-ft 3-in. long, including a 3-in. expansion gap between the third and fourth posts. The reinforcement in the rail consisted of six longitudinal Grade 60 epoxy-coated bars. Six No. 6 bars were placed from Post Nos. 1 through 6, while six No. 5 bars were placed from Post Nos. 7 through 12. The No. 5 and No. 6 bars were lapped between the sixth and seventh posts, as shown in Figure 15.

The concrete used for all of the above components was a Nebraska 47-BD Mix, with a minimum 3500 psi compressive strength. The 28-day concrete compressive strengths for the simulated bridge deck and the monolithic concrete posts and attached rail were approximately 6,240 psi and 5,200 psi, respectively, as shown in Appendix C.

As previously stated, all of the reinforcement in the simulated bridge deck, posts, and rail was Grade 60 epoxy-coated rebar. The Certificate of Compliance and Certified Test Report are shown in Appendix C.

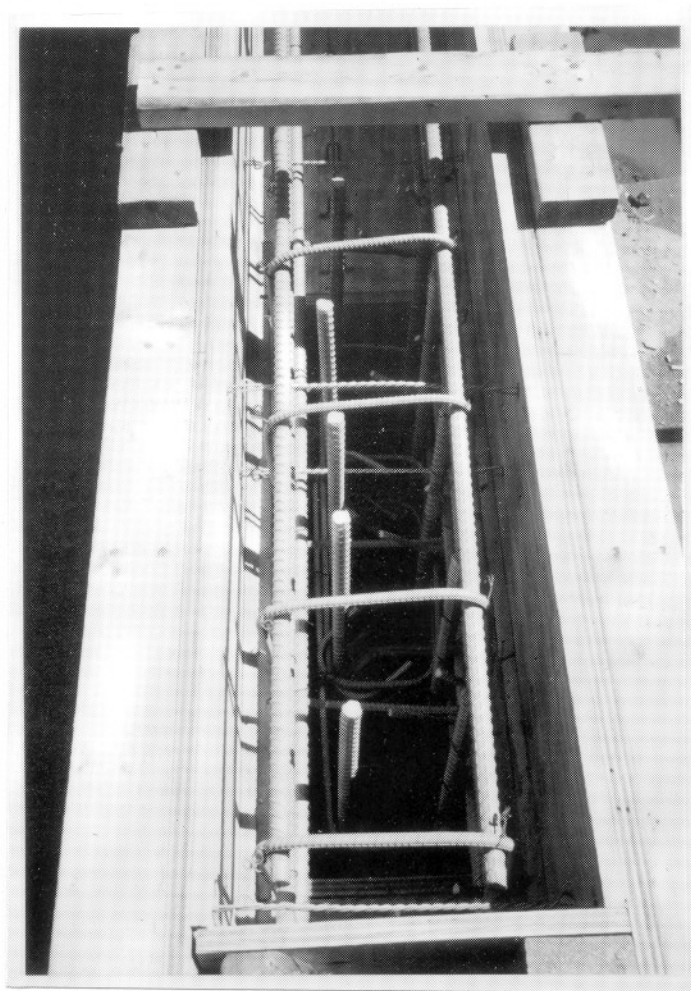
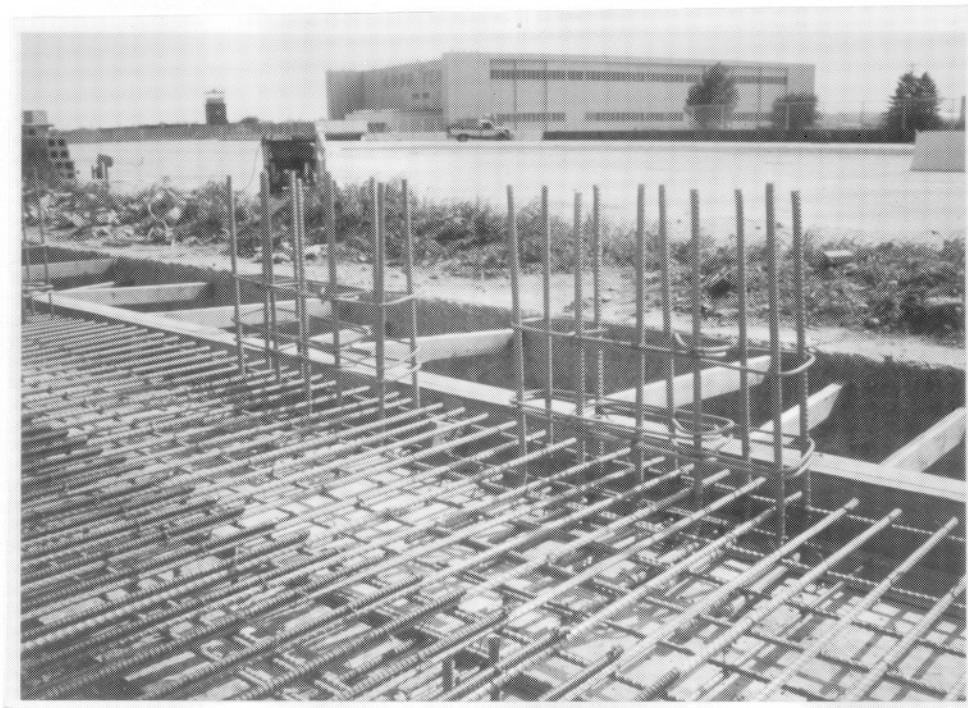


FIGURE 19. Steel Reinforcement in the 3-ft Posts Adjacent to the 3-in. Gap

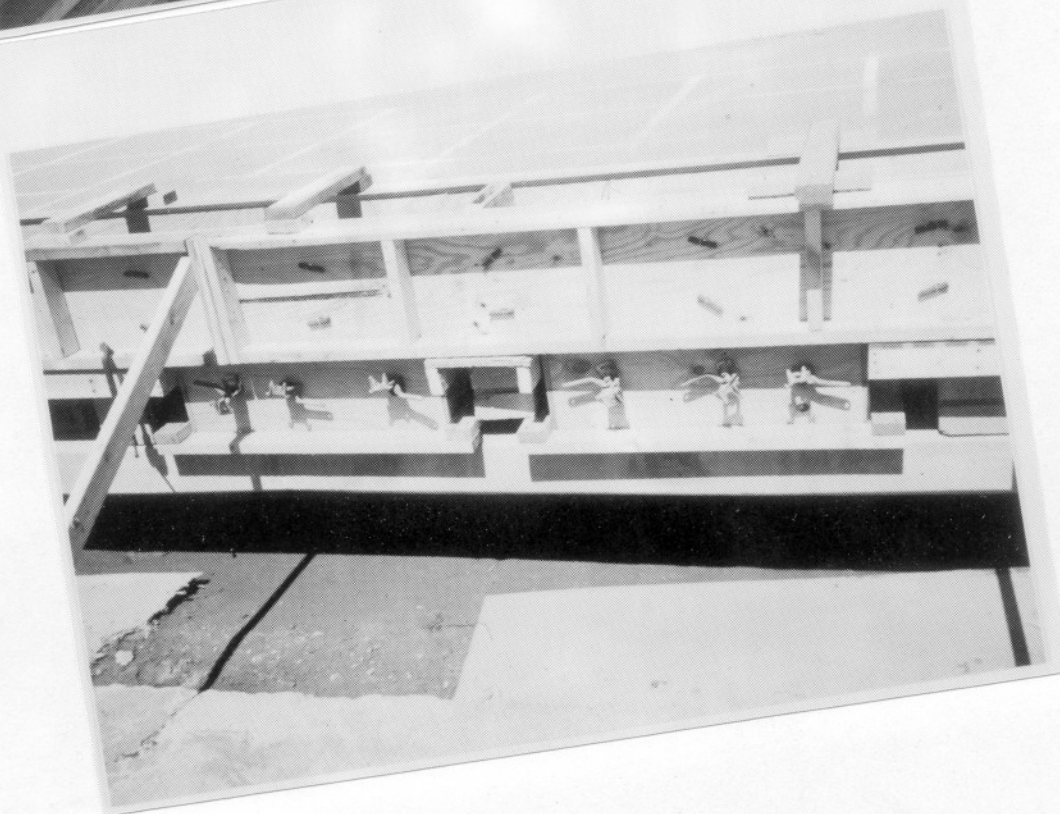


FIGURE 20. Bridge Rail Construction

3.3 Test Vehicles

The test vehicle used for Test NEOCR-1 was a 1985 Chevrolet 3/4-ton Custom Deluxe-20 pickup. The test vehicle had a test inertial and a gross static weight of 5,300-lb. The test vehicle is shown in Figure 21 and the vehicle dimensions are shown in Figure 22.

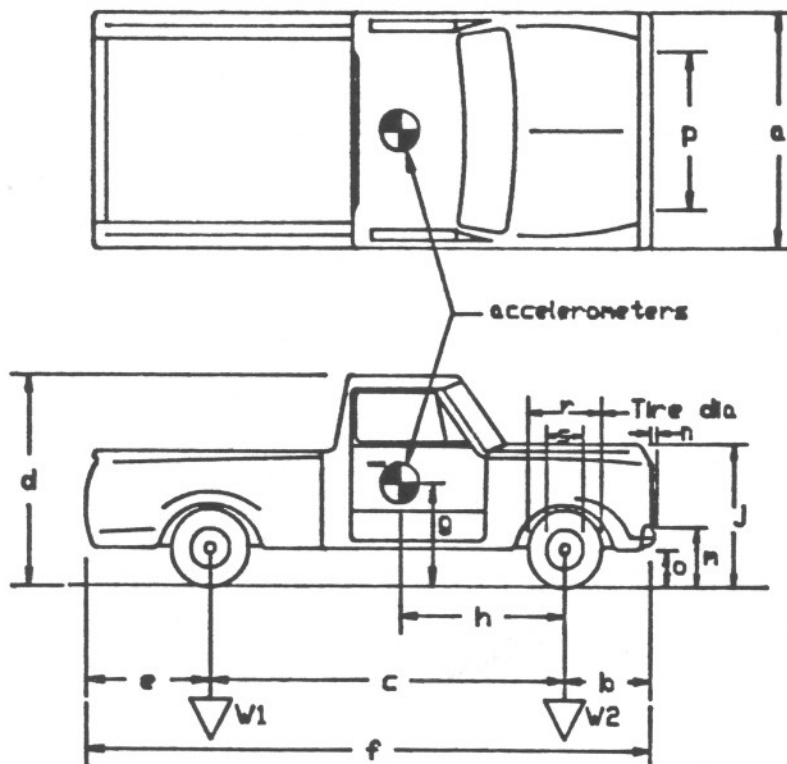
The test vehicle used for Test NEOCR-2 was a 1986 Chevrolet 3/4-ton Custom Deluxe-20 pickup. This test vehicle had a test inertial and a gross static weight of 5,390-lb. The test vehicle is shown in Figure 23, and the vehicle dimensions are shown in Figure 24.

The suspension method (6) was used to calculate the vertical component of the center of gravity for the test vehicles. This method is based on the principle that the center of gravity of any freely suspended body is in the vertical plane through the point of suspension. The vehicle was suspended successively in three positions, and the respective planes containing the center of gravity were established. The intersection of these planes pinpointed the location of the center of gravity. The longitudinal component of the center of gravity was determined by using the axle weights of the vehicles.

Eight 12-in., square, black and white-checkered targets were placed on the vehicle. These targets were used in the high-speed film analysis. Two targets were located on the center of gravity, one on the top and one on the driver's side of the test vehicle. The remaining targets were located such that they could be viewed from all three cameras.



FIGURE 21. Test Vehicle, NEOCR-1

Date: 7/16/91Test No.: NEOCR-1Vehicle I.D. #: 1GBGC24MYFJ147626Make: ChevroletModel: Custom Deluxe 20Year: 1985Odometer: 140458Tire Size: LT235/85R16

Vehicle Geometry - inches

a	77"	b	33"
c	131.5"	d	71.5"
e	46.5"	f	211"
g	26"	h	69"
i	NA	j	44.5"
k	NA	l	NA
m	26.5"	n	3.5"
o	17"	p	66"
r	31"	s	17.5"

Engine Type: GasolineEngine Size: 5.7 Liter

4 - wheel weight: lf____ rf____ lr____ rr____

Transmission Type:

Automatic or ☒ ManualFWD or ☒ RWD or 4WD

Weight - pounds	Curb	Test Inertial	Gross Static
W1	1920	2781	2781
W2	2650	2519	2519
Wtotal	4570	5300	5300

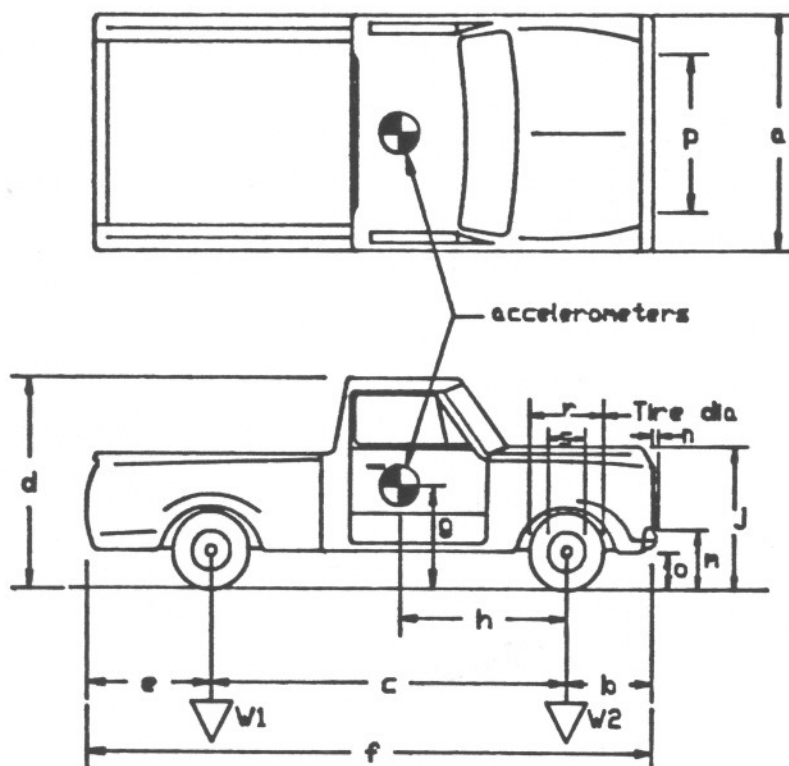
Note any damage prior to test: Minor dents on hood and passenger door.

FIGURE 22. Test Vehicle Dimensions, NEOCR-1



FIGURE 23. Test Vehicle, NEOCR-2

Date: 7/19/91 Test No: NEOCR- 2 Vehicle I.D. #: 1GCGC24M3GJ131684
 Make: Chevrolet Model: Custom Deluxe 20 Year: 1986 Odometer: 112959
 Tire Size: LT235/85R16



Vehicle Geometry - inches

a 77" b 35"
 c 130.5" d 71"
 e 47.5" f 213"
 g 26" h 60"
 i NA j 48"
 k NA l NA
 m 26" n 3.5"
 o 17" p 66"
 r 32.5" s 17.5"

Engine Type: Gasoline
 Engine Size: 5.7 Liter

4 - wheel weight: lf rf lr rr

Transmission Type:

☒ Automatic or Manual

FWD or ☒ RWD or 4WD

Weight - pounds	Curb	Test Inertial	Gross Static
W1	<u>2590</u>	<u>232370</u>	<u>2370</u>
W2	<u>2110</u>	<u>3020</u>	<u>3020</u>
Wtotal	<u>5700</u>	<u>5390</u>	<u>5390</u>

Note any damage prior to test: Minor dents on passenger side of vehicle.

FIGURE 24. Test Vehicle Dimensions, NEOCR-2

The front wheels of the test vehicle were aligned for camber, caster, and toe-in values of zero so that the vehicle would track properly along the guide cable. Two 5B flash bulbs were mounted on the roof of the vehicle to pinpoint the time of impact with the bridge rail on the high-speed film. The flash bulbs were fired by a pressure tape switch mounted on the front face of the bumper.

3.4 Data Acquisition Systems

3.4.1 Accelerometers

Four Endevco triaxial piezoresistive accelerometers (Model 7264) with a range of ± 200 g's were used to measure the acceleration in the longitudinal and the lateral directions of the test vehicle. Two accelerometers were mounted in each of the two directions so that there would be two accelerometer traces for validation of results. The accelerometers were rigidly attached to a metal block mounted at the center of gravity.

The signals from the accelerometers were received and conditioned by an onboard vehicle Metraplex Unit. The multiplexed signal was then transmitted to the Honeywell 101 Analog Tape Recorder in the central control van. A flow chart of the accelerometer data acquisition system is shown in Figure 25. State-of-the-art computer software, "Computerscope and DSP," was used to analyze and plot the accelerometer data on a Cyclone 386/AT, which uses a high-speed data acquisition board.

3.4.2 High Speed Photography

Three high-speed 16-mm cameras were used to film the crash tests. The cameras' normal operating speed is approximately 500 frames/sec. The overhead camera was a Red Lake Locam with a wide-angle 12.5-mm lens. The parallel camera was a Photec IV with a 80-mm lens. The perpendicular camera was a Photec IV with a 55-mm lens. A schematic of all three camera locations for each test is shown in Figure 26.

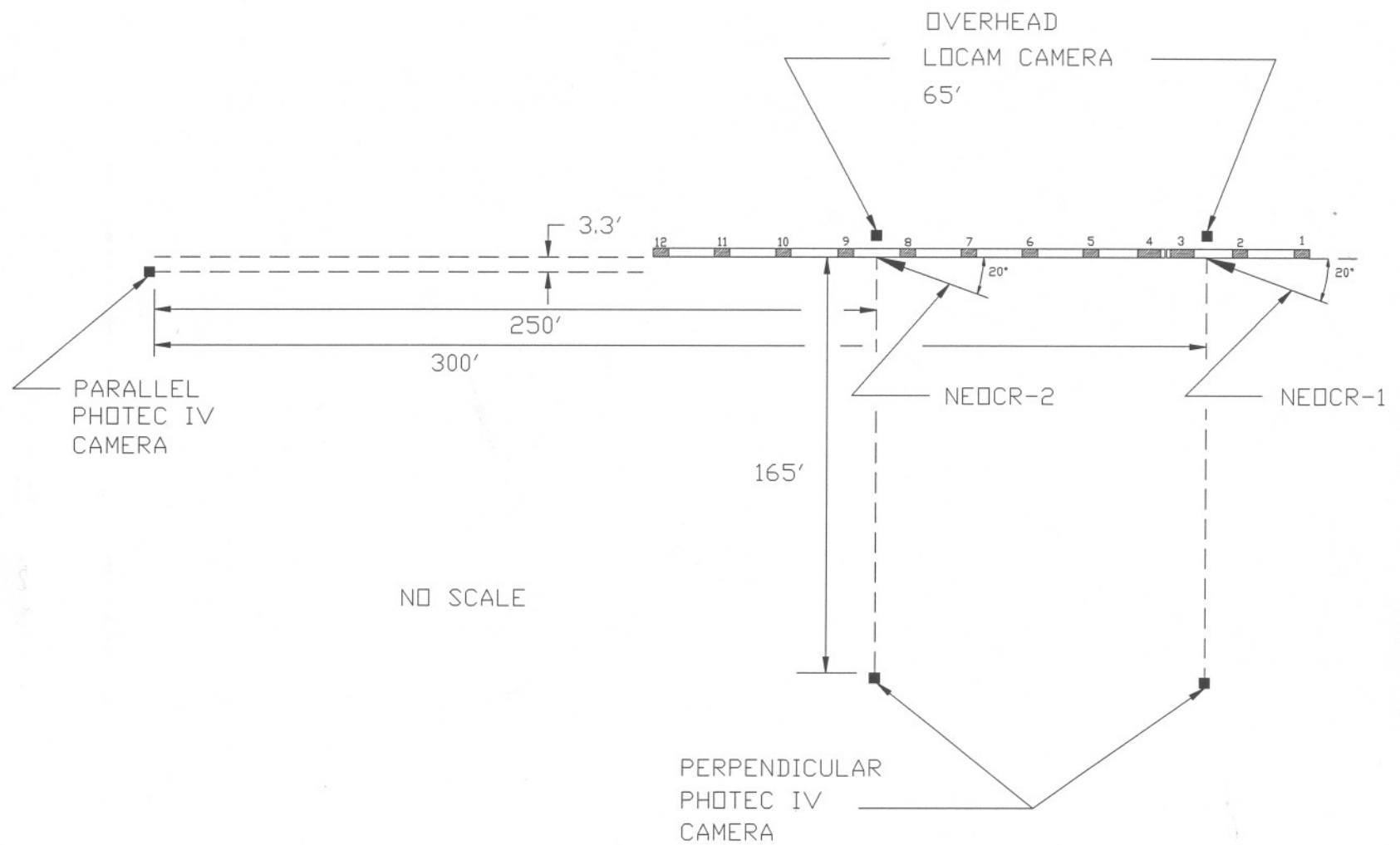


FIGURE 26. Layout of High Speed Cameras, NEOCR-1, 2

A 20-ft wide by 100-ft long grid was painted on the concrete surface parallel and perpendicular to the barrier. The white-colored grid was incremented with 5-ft divisions in both directions to give a visible reference system which could be used in the analysis of the overhead high-speed film.

The film was analyzed using the Vanguard Motion Analyzer. Actual camera speed and camera divergence factors were considered in the analysis of the high-speed film.

3.4.3 Speed Trap Switches

Eight tape pressure switches spaced at 5-ft intervals were used to determine the speed of the vehicle before and after impact. Each tape switch fired a strobe light as the left front tire of the test vehicle passed over it. The average speed of the test vehicle between the tape switches was determined by knowing the distance between pressure switches, the calibrated film speed, and the number of frames from the high speed film between flashes. In addition, the average speed was determined from electronic timing mark data, recorded on the oscilloscope software used with the 386/AT computer, as the test vehicle passed over each tape switch.

3.5 Test Parameters

Two full-scale vehicle crash tests were conducted on the Nebraska Open Concrete Bridge Rail in order to satisfy the AASHTO (3) PL-1 performance level. Test NEOCR-1 was conducted with a 1985 3/4 ton Chevy pickup weighing approximately 5,300-lb. The impact speed was 47.7 mph, with an impact angle of 20 deg. The location of impact was 5-ft 10½-in. upstream of the centerline of the 3-in. gap. The impact point is shown in Figure 27.

Test NEOCR-2 was conducted with a 1986 3/4-ton Chevy pickup weighing approximately 5,390-lb. The impact speed was 45.9 mph, with an impact angle of 20 deg. The impact location was midspan of the section between Post Nos. 8 and 9. The impact point is shown in Figure 28.



FIGURE 27. Impact Location, Test NEOCR-1



FIGURE 28. Impact Location, Test NEOCR-2

4 PERFORMANCE EVALUATION CRITERIA

The safety performance objective of a bridge rail is to reduce death and injury to the occupants of errant vehicles and to protect lives and property on, adjacent to, or below a bridge (3). In order to prevent or reduce the severity of such accidents, special attention should be given to four major design factors. These factors are: (1) strength of the railing, to resist impact forces; (2) effective railing height; (3) shape of the face of the railing; and (4) deflection characteristics of the railing (7).

The performance evaluation criteria used to evaluate the two crash tests were taken from the AASHTO *Guide Specifications for Bridge Railings* (3). The test conditions for the required test matrix are shown in Table 9. The specific evaluation criteria are shown in Table 10. As previously mentioned, the PL-1 test with the 1,800-lb test vehicle was not necessary since the bridge rail geometry (effective railing height and shape of the face of the railing) had not changed since a crash test using a 1,800-lb vehicle had been previously conducted on a similar installation at Ensco, Inc. (1). These test results are shown in Appendix A.

The safety performance of the bridge rail was evaluated according to three major factors: (1) structural adequacy, (2) occupant risk, and (3) vehicle trajectory after collision. These three evaluation criteria are defined and explained in NCHRP 230 (2). After each test, vehicle damage was assessed by the traffic accident scale (TAD) (8) and the vehicle damage index (VDI) (9).

TABLE 9. Crash Test Conditions and Evaluation Criteria

Guidelines	Performance Level	Appurtenance	Test Vehicle	Impact Conditions		Evaluation Criteria ¹	
				Speed (mph)	Angle (deg)	Required	Desirable
AASHTO	PL-1	Bridge Rail	Pickup Truck	45	20	3. a,b,c,d	3. e,f,g,h

¹ Evaluation criteria explained in Table 10.

TABLE 10. AASHTO Evaluation Criteria

3.a. The test article shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the test article is acceptable.												
3.b. Detached elements, fragments, or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.												
3.c. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.												
3.d. The vehicle shall remain upright during and after collision.												
3.e. The test article shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle does not yaw more than 5 degrees away from the railing from time of impact until the vehicle separates from the railing.												
3.f. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction μ , where $\mu = (\cos\theta - V_p/V)/\sin\theta$. <table><tr><td>μ</td><td>Assessment</td></tr><tr><td>0.0 - 0.25</td><td>Good</td></tr><tr><td>0.26 - 0.35</td><td>Fair</td></tr><tr><td>> 0.35</td><td>Marginal</td></tr></table>	μ	Assessment	0.0 - 0.25	Good	0.26 - 0.35	Fair	> 0.35	Marginal				
μ	Assessment											
0.0 - 0.25	Good											
0.26 - 0.35	Fair											
> 0.35	Marginal											
3.g. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 2.0-ft longitudinal and 1.0-ft lateral displacements, shall be less than: <table><tr><td colspan="2">Occupant Impact Velocity - fps</td></tr><tr><td>Longitudinal</td><td>Lateral</td></tr><tr><td>30</td><td>25</td></tr></table> and for the vehicle highest 10-ms average accelerations subsequent to the instant of hypothetical passenger impact should be less than: <table><tr><td colspan="2">Occupant ridedown Accelerations - g's</td></tr><tr><td>Longitudinal</td><td>Lateral</td></tr><tr><td>15</td><td>15</td></tr></table>	Occupant Impact Velocity - fps		Longitudinal	Lateral	30	25	Occupant ridedown Accelerations - g's		Longitudinal	Lateral	15	15
Occupant Impact Velocity - fps												
Longitudinal	Lateral											
30	25											
Occupant ridedown Accelerations - g's												
Longitudinal	Lateral											
15	15											
3.h. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 100 ft plus the length of the test vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 20 ft from the line of the traffic face of the railing.												

5 TEST RESULTS

5.1 Test NEOCR-1 (5,300-lb, 47.7 mph, 20.0 deg)

The purpose of Test NEOCR-1 was to evaluate the safety performance of the discontinuous rail section of the Open Concrete Bridge Rail design; more specifically, the structural adequacy of the rail and posts at the gap location. A summary of the test results and the overhead sequential photographs are shown in Figure 29. Additional sequential photographs are shown in Figure 30.

After the initial impact with the bridge rail (5-ft 10½-in. upstream from the centerline of the 3-in. gap), the right front corner of the vehicle crushed inward. At 0.10 sec, the front corner contacted the 3-in. gap, causing the right front tire to blow out. At approximately 0.23 sec, the right rear side of the vehicle contacted the bridge rail causing the right rear tire to blow out. The vehicle became parallel to the rail at this time.

The vehicle began to be redirected at 0.43 sec. The vehicle's exit angle (2.8 deg) was measured at the loss of contact between the vehicle and the rail (0.50 sec). This angle was much lower than the 12 deg limit shown in Table 10. The right side tire blowouts caused the vehicle to veer back towards the rail resulting in a second impact with the bridge rail. This second impact occurred 56 ft downstream of impact at approximately 1.23 sec. The vehicle's trajectory is shown in Figure 31. The maximum rebound distance on the traffic side face of the railing was approximately 3 ft, and the maximum rebound distance on the back side of the railing was approximately 18 ft. Both of these rebound distances were measured from the right side of the vehicle to a line extended parallel to the traffic side face of the bridge railing. The effective coefficient of friction was determined to be fair ($\mu = 0.45$).

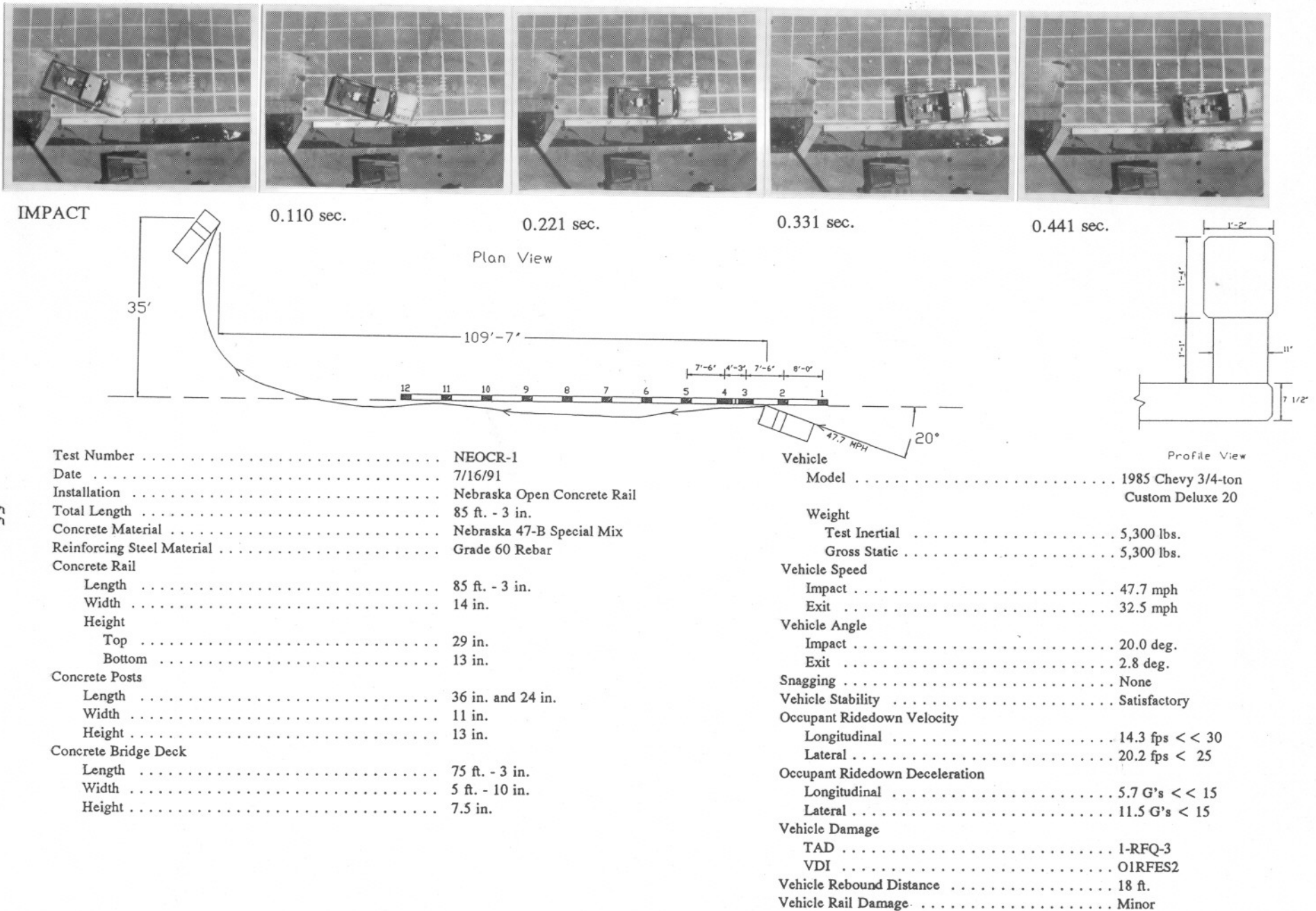
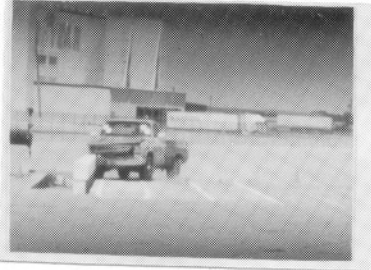


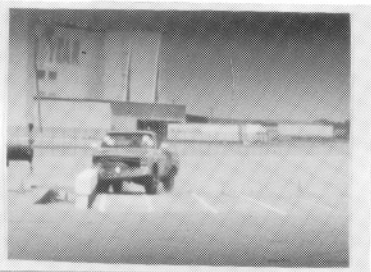
FIGURE 29. Summary and Sequential Photographs, NEOCR-1



Impact



0.100 sec



0.148 sec



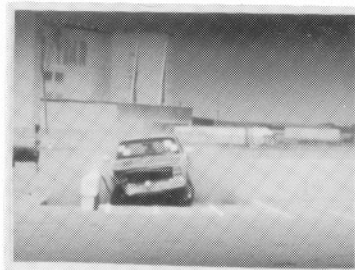
0.230 sec



0.379 sec



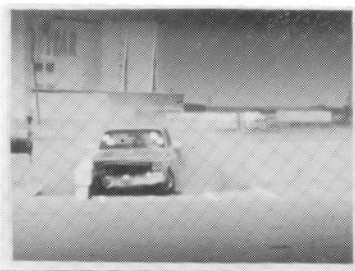
0.549 sec



1.000 sec



1.208 sec



1.598 sec



1.997 sec

FIGURE 30. Parallel Time Sequential Photographs, NEOCR-1

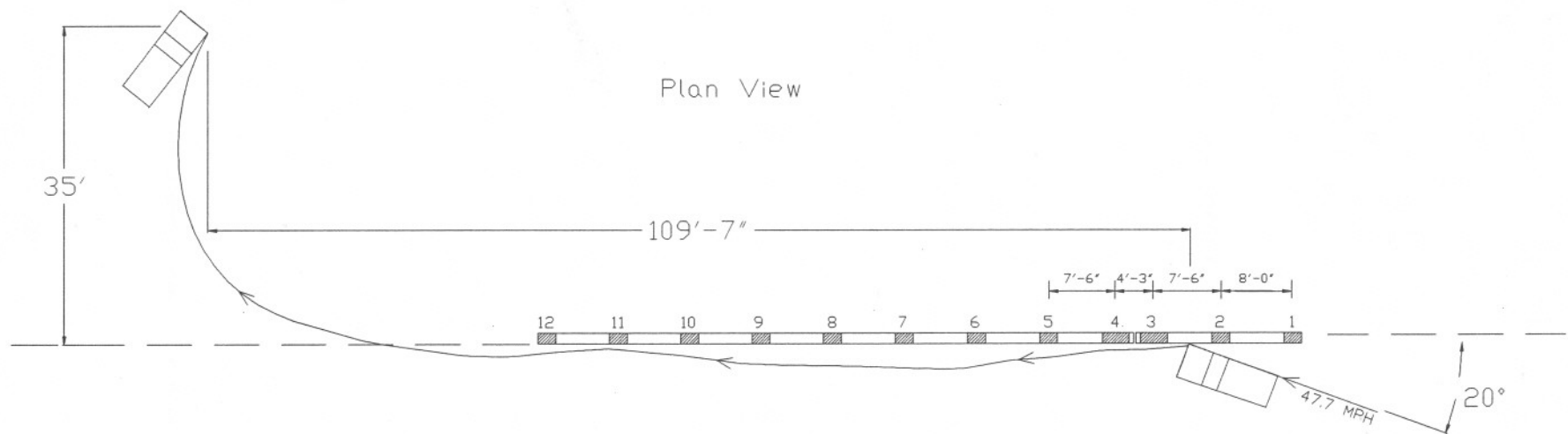


FIGURE 31. Vehicle Trajectory, NEOCR-1

Bridge rail damage is shown in Figure 32. Tire marks and scrapes accounted for the majority of the damage. Tire marks on the face of the rail were approximately 10-ft long, as shown in Figure 32. Tire marks were also visible on Post No. 3, and may have occurred when the tire impacted the post and then blew out. The length of the tire mark on Post No. 3 was approximately 17-in. The minimal bridge rail damage at the 3-in. gap is shown in Figure 33. Bridge rail damage which resulted from the second impact is shown in Figure 34.

Vehicle damage is shown in Figure 35. Most of the vehicle damage occurred near the right-front corner of the vehicle, consisting primarily of fender, hood, bumper, and undercarriage damage. Other damage included deformation to the door and pickup box and both right side tires. There was no intrusion nor deformation of the occupant compartment. The vehicle remained upright both during and after the collision. The vehicle damage was assessed by the traffic accident scale (TAD) (8) and the vehicle damage index (VDI) (9), as shown in Figure 29 and Table 12.

The longitudinal occupant impact velocity was determined to be 14.3 fps and the lateral occupant impact velocity was 20.2 fps. The highest 0.010-sec average occupant ridedown decelerations were 5.7 g's (longitudinal), and 11.5 g's (lateral). The results of the occupant risk assessment, as determined from the accelerometer data, are summarized in Figure 29 and Table 12. The results are shown graphically in Appendix D.

The performance of the bridge railing system tested was determined to be satisfactory according to the Performance Level 1 criteria given in Tables 9 and 10.

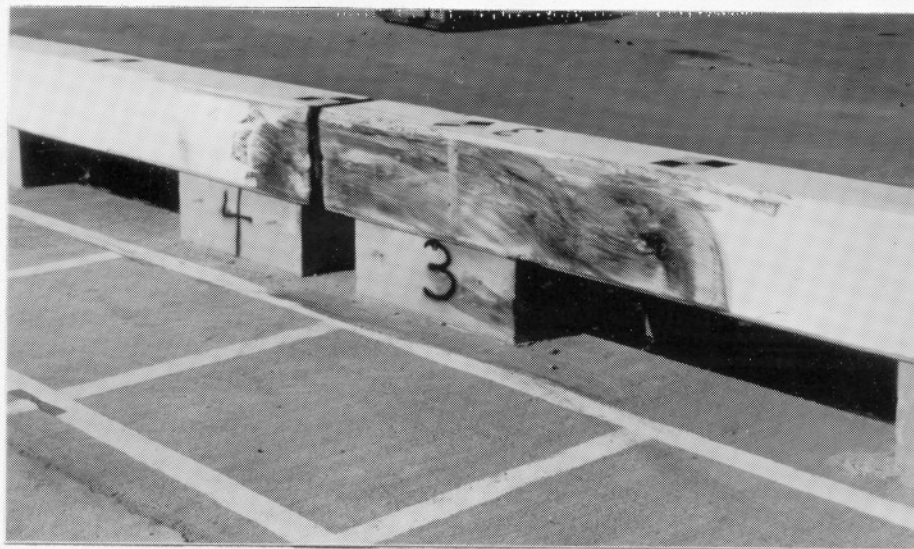


FIGURE 32. Bridge Rail Damage, NEOCR-1

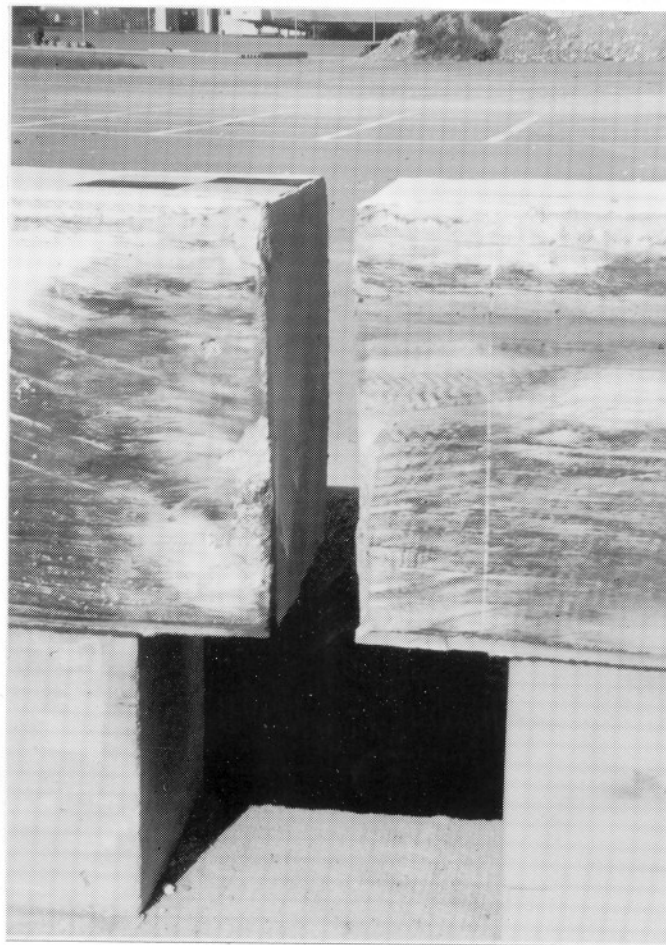


FIGURE 33. Bridge Rail Damage at the 3 in. Gap, NEOCR-1

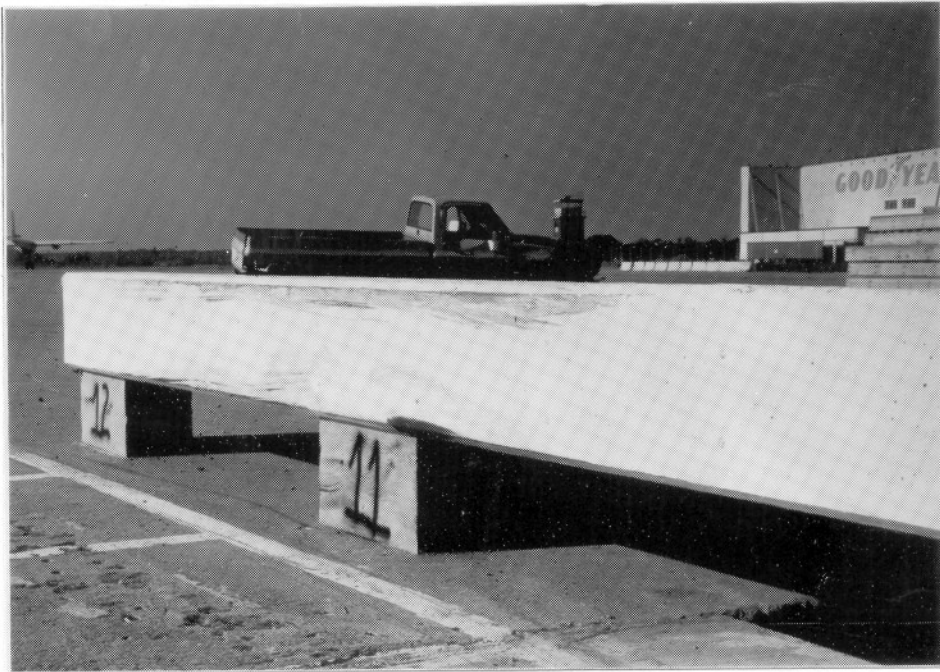


FIGURE 34. Bridge Rail Damage of Second Impact, NEOCR-1



FIGURE 35. Vehicle Damage, NEOCR-1

5.2 Test NEOCR-2 (5,390-lb, 45.9 mph, 20.0 deg)

The purpose of Test NEOCR-2 was to evaluate the structural adequacy of a rail in which the longitudinal steel reinforcement had been reduced. A summary of the test results and sequential photographs are shown in Figure 36. Additional sequential photographs are shown in Figure 37.

After the initial impact with the bridge rail (midspan of the section between Post Nos. 8 and 9), the right-front corner of the vehicle crushed inward. All four wheels remained on the ground following this event, and there were no tire blowouts. The vehicle became parallel to the rail at approximately 0.20 sec. The vehicle began to exit at approximately 0.26 sec with an exit angle of 8 deg and a speed of 37.6 mph. The vehicle contact length with the bridge rail was 10 ft. The vehicle's trajectory is shown in Figure 38. The vehicle's maximum rebound distance on the traffic side face of the railing was approximately 20 ft, which was measured from a line extended parallel to the traffic side face of the bridge rail to the right side of the vehicle. The effective coefficient of friction was determined to be fair ($\mu=0.30$).

Bridge rail damage is shown in Figure 39. The damage was only cosmetic, consisting of tire marks and minor scrapes. Tire marks on Post No. 9 indicated that it was struck by the right side front tire shortly after impact. There was no visible lateral deflection of the bridge rail.

Vehicle damage is shown in Figure 40. The right-front corner had a maximum crush depth of approximately 24 in. All of the vehicle damage occurred on the right-side. This included damage to the front fender, door, and the pickup box. No tire blowouts or any undercarriage damage occurred as a result of the impact. There was no intrusion nor deformation of the occupant compartment. The vehicle remained upright both during and after the impact. The vehicle damage was assessed by the traffic accident scale (TAD) (7) and the vehicle damage

index (VDI) (8), as shown in Figure 28 and Table 12.

The longitudinal occupant impact velocity was determined to be 13.2 fps and the lateral occupant impact velocity was 19.0 fps. The highest 0.010-sec average occupant ridedown decelerations were 10.1 g's (longitudinal) and 9.5 g's (lateral). The results of the occupant risk, as determined from the accelerometer data are summarized in Figure 28 and Table 12. The results are shown graphically in Appendix E.

The performance of the bridge railing system tested was determined to be satisfactory according to the Performance Level 1 criteria given in Tables 9 and 10.



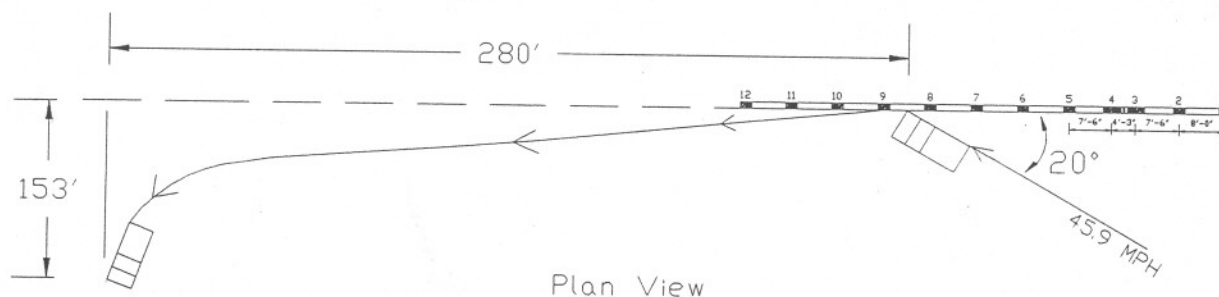
IMPACT

0.089 sec.

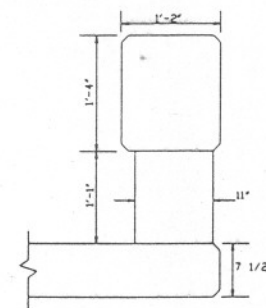
0.207 sec.

0.395 sec.

0.988 sec.



Plan View



Profile View

Test Number	NEOCR-2
Date	7/19/91
Installation	Nebraska Open Concrete Rail
Total Length	85 ft. - 3 in.
Concrete Material	Nebraska 47-B Special Mix
Reinforcing Steel Material	Grade 60 Rebar
Concrete Rail	
Length	85 ft. - 3 in.
Width	14 in.
Height	
Top	29 in.
Bottom	13 in.
Concrete Posts	
Length	24 in.
Width	11 in.
Height	13 in.
Concrete Bridge Deck	
Length	75 ft. - 3 in.
Width	5 ft. - 10 in.
Height	7.5 in.

Vehicle	
Model	1986 Chevy 3/4-ton Custom Deluxe 20
Weight	
Test Inertial	5,390 lbs.
Gross Static	5,390 lbs.
Vehicle Speed	
Impact	45.87 mph
Exit	37.6 mph
Vehicle Angle	
Impact	20.0 deg.
Exit	8.0 deg.
Snagging	None
Vehicle Stability	Satisfactory
Occupant Ridedown Velocity	
Longitudinal	13.2 fps < 30
Lateral	19.0 fps < 25
Occupant Ridedown Deceleration	
Longitudinal	10.1 G's < 15
Lateral	9.5 G's < 15
Vehicle Damage	
TAD	1-RFQ-4
VID	01RFES2
Vehicle Rebound Distance	20 ft.
Bridge Rail Damage	Minor

FIGURE 36. Summary and Sequential Photographs, NEOCR-2



Impact



0.256 sec



0.089 sec



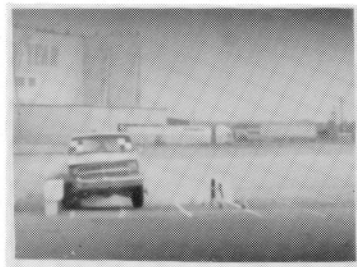
0.395 sec



0.158 sec



0.593 sec



0.207 sec



0.988 sec

FIGURE 37. Paralell Sequential Photographs, NEOCR-2

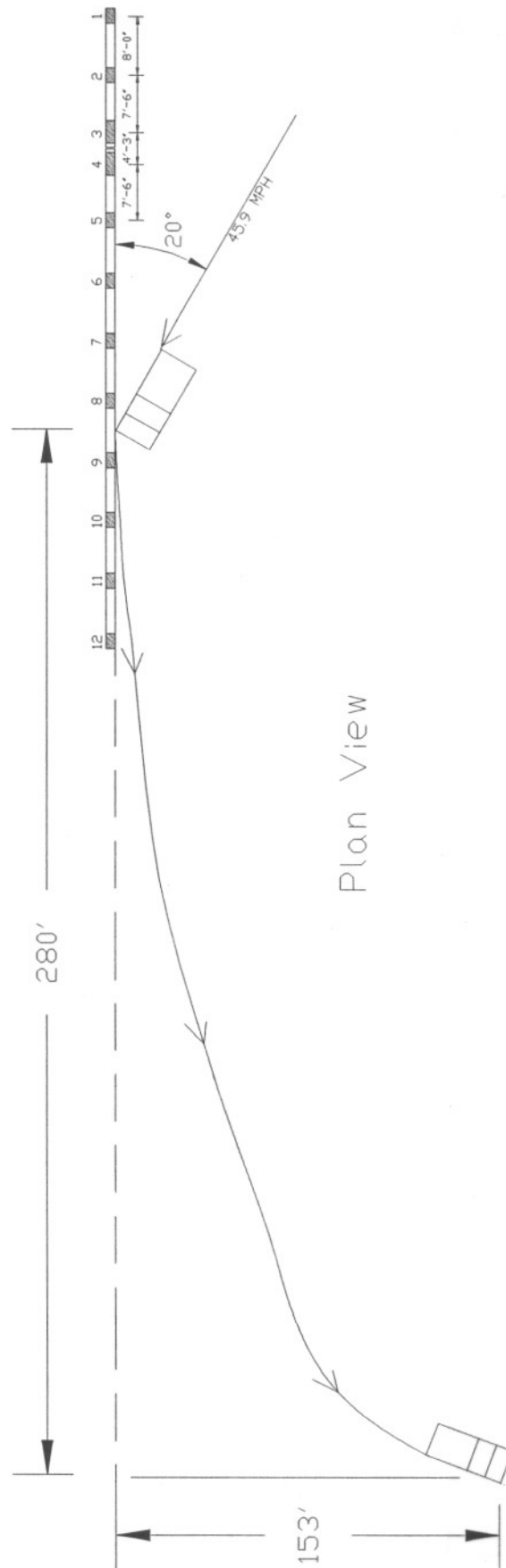


FIGURE 38. Vehicle Trajectory, NEOCR-2



FIGURE 39. Bridge Rail Damage, NEOCR-2



FIGURE 40. Vehicle Damage, NEOCR-2

6 CONCLUSIONS

The PL-1 performance level tests on the Nebraska Open Concrete bridge rail proved to be satisfactory according to the safety performance criteria given in AASHTO (3). The safety performance evaluation summary is presented in Table 11. The results of both tests are summarized and presented in Table 12. The analysis of the tests revealed the following:

1. The bridge rail did contain the vehicles without any lateral deflection.
2. No detached elements or fragments penetrated the occupant compartments.
3. The integrity of the occupant compartments was maintained.
4. The vehicles remained upright both during and after impact.
5. The bridge rail's redirection capability was determined to be satisfactory.
6. The effective coefficient of friction for NEOCR-1 and NEOCR-2 were determined to be marginal ($\mu=0.45$) and fair ($\mu=0.30$) respectively.
7. The occupant ridedown decelerations were determined to be satisfactory.
8. The occupant impact velocities were determined to be satisfactory.
9. The vehicle's exit angle and rebound distance were determined to be satisfactory.

The safety performance of the Nebraska Open Concrete Bridge Rail was determined to be satisfactory according to the safety performance evaluation criteria presented in Tables 9 and 10. The summary of the results for the safety performance evaluation is presented in Table 11 and a summary of the test is shown in Table 12.

TABLE 11. Summary of Safety Performance Results

Evaluation Criteria	Results	
	NEOCR-1	NEOCR-2
3.a. The test article shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the test article is acceptable.	S	S
3.b. Detached elements, fragments, or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.	S	S
3.c. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.	S	S
3.d. The vehicle shall remain upright during and after collision.	S	S
3.e. The test article shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle does no yaw more than 5 degrees away from the railing from time of impact until the vehicle separates from the railing.	S	S
3.f. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction μ , where $\mu = (\cos\theta - V_p/V)/\sin\theta$.	M($\mu=0.45$)	F($\mu = 0.30$)
<div><div>μ</div><div>0.0 - 0.25</div><div>0.26 - 0.35</div><div>> 0.35</div></div> <div><div>Assessment</div><div>Good</div><div>Fair</div><div>Marginal</div></div>		

Table 11. Summary of Safety Performance Results (continued)

Evaluation Criteria	Results			
	NEOCR-1		NEOCR-2	
3.g. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 2.0-ft longitudinal and 1.0-ft lateral displacements, shall be less than: <div>Occupant Impact Velocity - fps</div> <div><div>Longitudinal</div><div>Lateral</div></div> <div>3025</div> and for the vehicle highest 10-ms average accelerations subsequent to the instant of hypothetical passenger impact should be less than: <div>Occupant ridedown Accelerations - g's</div> <div><div>Longitudinal</div><div>Lateral</div></div> <div>1515</div>	Occupant Impact Velocity (fps)			
	Longitudinal	Lateral	Longitudinal	Lateral
	S (14.3)	S (20.2)	S (13.2)	S (19.0)
	Occupant Ridedown Accelerations (g's)			
	Longitudinal	Lateral	Longitudinal	Lateral
	S (5.7)	S (11.5)	S (10.1)	S (9.5)
3.h. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 100 ft plus the length of the test vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 20 ft from the line of the traffic face of the railing.	S (2.8 deg)		S (8.0 deg)	
	S (3.0 ft)		S (20.0 ft)	

TABLE 12. Summary of Test Results

Test Item	Test NEOCR-1	Test NEOCR-2
Vehicle Weight (lb)	5,300	5,390
Vehicle Impact Speed (mph)	47.7	45.9
Vehicle Exit Speed (mph)	32.5	37.6
Vehicle Impact Angle (deg)	20.0	20.0
Vehicle Exit Angle (deg)	2.8	8.0
Effective Coefficient of Friction	0.45	0.30
Vehicle Rebound Distance (ft)	3.0	20.0
Vehicle Damage (TAD)	1-RFQ-3	1-RFQ-4
Vehicle Damage (VDI)	01RFES2	01RFES2
Occupant Impact Velocity (fps)		
Longitudinal	14.3	13.2
Lateral	20.2	19.0
Occupant Ridedown Decelerations (g's)		
Longitudinal	5.7	10.1
Lateral	11.5	9.5
Did Snagging Occur?	No	No
Did Vehicle Rollover Occur?	No	No

7 REFERENCES

1. Stout, D., Hinch, J., *Test and Evaluation of Traffic Barriers: Final Report - Technical*, Office of Safety and Traffic Operations R & D, Federal Highway Administration, FHWA-RD-89-119, April 1989.
2. *Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*, National Cooperative Highway Research Program Report 230, Transportation Research Board, Washington, D.C., March 1981.
3. *Guide Specifications for Bridge Railings*, American Association of State Highway and Transportation Officials, Washington, D.C., 1989.
4. Powell, G.H., *BARRIER VII : A Computer Program for Evaluation of Automobile Barrier Systems*, Prepared for: Federal Highway Administration, Report No. FHWA-RD-73-51, April 1973.
5. Hinch, J., Yang, T-L, and Owings, R., *Guidance Systems for Vehicle Testings*, ENSCO, Inc., Springfield, VA, 1986.
6. *Center of Gravity Test Code - SAE J874 March 1981*, SAE Handbook Vol.4, Society of Automotive Engineers, Inc., Warrendale, Penn., 1986.
7. *Roadside Design Guide*, American Association of State Highway and Transportation Officials, Washington, D.C., October, 1988.
8. *Vehicle Damage Scale for Traffic Investigators*, Traffic Accident Data Project Technical Bulletin No. 1, National Safety Council, Chicago, IL, 1971.
9. *Collision Deformation Classification, Recommended Practice J224 March 1980*, SAE Handbook Vol. 4, Society of Automotive Engineers, Warrendale, Penn., 1985.

8 APPENDICES

APPENDIX A.
RELEVANT CORRESPONDENCE

STATE OF NEBRASKA

DEPARTMENT OF ROADS

KAY A. ORR
GOVERNOR

G. C. STROBEL
DIRECTOR-STATE ENGINEER



Dr Edward R Post PE
Civil Engineering Department
University of Nebraska - Lincoln
W348 Nebraska Hall
Lincoln NE 68588-0531

Back
Back
p

Dear Dr. Post:

Preliminary to an actual crash test, the Bridge Design Division of the Nebraska Department of Roads would like input regarding the possibility of running a computer simulation of the proposed design to see if it is adequate to warrant running the full PL-1 crash test. Since this activity is not included within the proposed crash test program, it is requested that you develop a proposal which would detail the cost and the time needed for the computer simulation testing.

A set of plans detailing the proposed design accompanies this letter. If you feel the computer program is not adequate to model this design, please advise.

Sincerely,

A handwritten signature in cursive script that reads "Dalyce Ronnau".

Dalyce Ronnau
Research & Tests Engineer
Nebraska Department of Roads

DR/bb



STATE OF NEBRASKA

E. BENJAMIN NELSON
~~XXXXXXXXXX~~
GOVERNOR

DEPARTMENT OF ROADS

G. C. STROBEL
DIRECTOR-STATE ENGINEER

February 19, 1991

Mr. Ron Faller
University of Nebraska, Lincoln
Department of Civil Engineering
W348 Nebraska Hall
Lincoln, NE 68588-0531

Re: Concrete Bridge Rail Crash Test - Simulation

This is to confirm our phone conversation regarding the post configuration for the crash test.

Please modify the simulation model to show an 11 by 24 inch post at all locations for the rail to be tested. The post should include four No. 6 rebars in the traffic side and three No. 4 rebars in the back side. Use a No. 3 rebar for stirrups with 2 inches clear between the stirrup and the surface of the concrete.

The number and location of the posts should remain as shown in the original details of the crash test.

If you have any additional questions, please call me at 402-479-3921.

Sincerely,

Gale A. Barnhill
Structural Engineer

cc: Dalyce Ronnau, NDOR Materials & Test

Minute-Memo

May 16, 1991

SUBJECT: Nebraska Open Concrete Bridge Rail - Design No. 4

TO: Ron Faller

Charles F. McDevitt

FROM: Charles F. McDevitt, HSR-20

Per your request, I have reviewed the subject sketch. My comments are as follows.

- For Test No. 1, the pickup truck should just miss post no. 2.
- The impact point for Test No. 2 seems appropriate.
- In addition to Tests Nos. 1 and 2, a test should be conducted with an 1,800-lb car at 50 mph and 20 degrees in order to satisfy the requirements for Performance Level 1 (PL-1).
- The reinforcement in the deck in the vicinity of the posts should be checked, particularly for posts Nos. 3 and 4. The simulated concrete deck is only 7.5-in thick. You should check with the FHWA Division office on the amount of concrete cover that will be required on the top and bottom of the slab. (In general, FHWA prefers a minimum of 2-in of concrete cover over main rebars.)
- Since the test installation is only 68-ft, 3-in long it will be marginal, but adequate for these PL-1 tests.

I hope that this information will be helpful to you. If you have any questions, please call me at 703/285-2418.

August 15, 1990

TO: State Highway Departments of Nebraska, Kansas, and Missouri

FROM: Midwest Roadside Safety Facility, Civil Engineering
Department, University of Nebraska-Lincoln

SUBJECT: Research Proposal For The Midwest Regional Pooled Fund
Program (Year 1)

The Midwest Roadside Safety Facility (MwRSF) proposes to conduct six full-scale vehicle crash tests using three different concrete bridge rail systems for a total of \$, as shown in Table 1. This includes two 18,000 lb., one 1,800 lb., and three 5,400lb. vehicle tests.

The three systems which will be constructed, removed, and disposed are as follows:

- (1) the 30" high barrier rail (Missouri)
- (2) the open concrete rail (Nebraska)
- (3) the 32" high corral rail (Kansas)

The estimated construction, removal, etc., costs were determined from the preliminary provided plans. The preliminary work schedule is shown in Table 2.

MISSOURI

Three full-scale vehicle crash tests are required to satisfy the PL-2 Performance Level on the 30" high barrier rail.

NEBRASKA

The open concrete rail was previously tested under NCHRP 230 (FHWA/RD-89-119), but, a modification using less reinforcement would require the 5,400 lb. test at the expansion joint to satisfy the PL-1 Performance Level. An 1,800 lb. vehicle test would not be required. If a failed performance evaluation would occur, a redesign would follow, along with a 5,400 lb. test.

The previous testing was conducted at ENSCO consisting of a 29" high, open concrete rail. The results of the tests are as follows:

Test 1769-F-1-86: 4,669 lb. test vehicle
57.6 mph and 26 degrees
barrier contact - 11 ft.
impact velocity (fps) - (accelerometer)
 longitudinal ...17.2 <30 ok
 lateral31.2 >20 ?
ridedown acceleration (g's) - (accel.)
 longitudinal ...-2.8 <15 ok
 lateral-14.3 <15 ok

Test 1769-F-2-86: 1,971 lb. test vehicle
59.8 mph and 21 degrees
barrier contact - 12 ft.
impact velocity (fps) - (accelerometer)
 longitudinal ...21.8 <30 ok
 lateral24.1 >20 ?
ridedown acceleration (g's) - (accel.)
 longitudinal ...-4.9 <15 ok
 lateral-10.5 <15 ok

KANSAS

A 27" high corral rail was previously tested under NCHRP 230 (FHWA/RD-87-049) which was cited as a basis for not requiring the 1,800 lb. and 5,400 lb. vehicle tests. Thus, only an 18,000 lb. vehicle test is required to satisfy the PL-2 Performance Level.

The previous testing was conducted at Southwest Research Institute consisting of two designs, (1) the KBR Series and (2) the MKS Series. The KBR Series consisted of the 27" high, Kansas corral rail without curb. The MKS Series comes from a modification to the Kansas corral rail due to an addition of longitudinal beam steel and stirrups in both the beam and posts. The results of the MKS Series are as follows:

Test MKS-1: 1,850 lb. test vehicle
59.0 mph and 18.9 degrees
barrier contact - 7.8 ft.
impact velocity (fps) - (film/accelerometer)
 longitudinal ... 9.2/14.0 <30 ok
 lateral19.5/18.2 <20 ok
ridedown acceleration (g's) - (accelerometer)
 longitudinal ... 1.4 <15 ok
 lateral-14.8 <15 ok

Test MKS-2: 4,690 lb. test vehicle
59.2 mph and 24.9 degrees
barrier contact - 12.2 ft.
impact velocity (fps) - (film/accelerometer)
 longitudinal ... 6.7/13.9 <30 ok
 lateral19.3/24.9 <20 ok
ridedown acceleration (g's) - (accelerometer)
 longitudinal ...-1.7 <15 ok
 lateral-13.9 <15 ok

APPENDIX B.
FORTTRAN PROGRAM

1 Page


```

C      CE 943 REINFORCED CONCRETE
C      PROGRAM THAT CALCULATES THE MOMENT IN A BEAM DUE TO FLEXURE WHEN
C      GIVEN VALUES OF CONCRETE STRAIN(TOP SURFACE) ARE INPUT INTO
C      THE PROGRAM.  ALSO, VALUES FOR ANGLE 1 ARE INPUTTED.
C
C
C      $LARGE
C      $DEBUG
C      DIMENSION EPC(1000),K(1000),PT1(1000),1
C      *EPS(1000),EPPS(1000),FS(1000),FPS(1000)
C      *T(1000),YO(1000),MO(1000),PHI(1000)
C      REAL K,L,M,MO,N
C
C      OPEN(UNIT=3,FILE='BEAM.DAT')
C      READ(3,*) AS,APS,D,DP,B,H
C      PRINT*, AS,APS,D,DP,B,H
C      READ(3,*) FPC,FY,ES,EPO
C      PRINT*, FPC,FY,ES,EPO
C
C      AS - AREA OF TENSION STEEL (IN**2)
C      APS - AREA OF COMPRESSION STEEL (IN**2)
C      D - EFFECTIVE DEPTH TO TENSION STEEL (IN)
C      DP - EFFECTIVE DEPTH TO COMPRESSION STEEL (IN)
C      B - WIDTH OF SECTION (IN)
C      H - OVERALL DEPTH OF SECTION (IN)
C      FPC - COMPRESSION STRENGTH OF CONCRETE (KSI)
C      FY - YIELD STRESS OF STEEL (KSI)
C      ES - MODULUS OF ELASTICITY OF STEEL (KSI)
C      EPO - CONCRETE STRAIN AT F'c (IN/IN)
C
C      READ(3,*) EPC(1),DELTA
C      PRINT*, EPC(1),DELTA
C      EPC - CONCRETE STRAIN AT SURFACE (IN/IN)
C      DELTA - INCREMENT OF CONCRETE STRAIN (IN/IN)
C
C      OPEN(UNIT=7,FILE='LPT1')
C      WRITE(7,10) AS,APS,D,DP,B,H,FPC,FY
10  FORMAT(1X,'BEAM DIMENSIONS AND PROPERTIES',///,' AREA OF TENSION
* STEEL =',F5.2,' IN**2',///,' AREA OF COMPRESSION STEEL =',F5.2,
* ' IN**2',///,' EFFECTIVE DEPTH TO TENSION STEEL =',F5.2,' IN',///,
* ' EFFECTIVE DEPTH TO COMPRESSION STEEL =',F5.2,' IN',///,' WIDTH OF
* SECTION =',F5.2,' IN',3X,'OVERALL DEPTH OF SECTION =',F5.2,
* ' IN',///,' COMPRESSION STRENGTH OF CONCRETE =',F5.2,' KSI',///,
* ' YIELD STRESS OF STEEL =',F7.2,' KSI',///)
C      CLOSE(UNIT=7)
C
C      EPCU=0.004
C      E=2.71828
C      EPY=FY/ES
C
C      DO 700 I=1,1000
C      EPC(I)=EPC(I)

```

*Back to
Back*

```

PRINT*, EPCU,E,EPY,EPC(I)
K(I)=(E*FPC*B/EPC(I))*(-EPC(I)*E**(-EPC(I)/EPO)-EPO*E**(-EPC(I)/
*EPO)+EPO)
C
PT1(I)=(EPC(I))*((E**(-EPC(I)/EPO))*(-1.0-2.0*(EPO/EPC(I))-2.0*
*(EPO**2.0/EPC(I)**2.0))+(2.0*(EPO**2.0/EPC(I)**2.0)))
C
PT2(I)=(-EPC(I)*E**(-EPC(I)/EPO)-EPO*E**(-EPC(I)/EPO)+EPO)
C
L=APS*ES*EPC(I)
M=AS*ES*EPC(I)
N=APS*FY
P=AS*FY
C
25 AA=1.0
BB=(1.0/K(I))*(L+M)
CC=(-1.0/K(I))*(L*DP+M*D)
X(I)=(-BB+SQRT(BB**2.0-4.0*AA*CC))/(2.0*AA)
C
CALL SSSN(I,D,DP,ES,X,EPC,EPS,EPPS,FS,FPS)
C
IF(FS(I).GE.FY) GO TO 50
IF(FPS(I).GE.FY) GO TO 200
GO TO 100
50 IF(FPS(I).GE.FY) GO TO 400
GO TO 300
C
CASE 1
100 JJ=1
PRINT*,JJ
PRINT*, PT1(I),PT2(I),K(I)
C(I)=K(I)*X(I)
CS(I)=L-L*(DP/X(I))
T(I)=M*(D/X(I))-M
YO(I)=PT1(I)*X(I)/PT2(I)
MO(I)=C(I)*(D-X(I)+YO(I))+CS(I)*(D-DP)
PHI(I)=EPC(I)/X(I)
GO TO 600
C
200 AA=1.0
BB=(1.0/K(I))*(N+M)
CC=-M*D/K(I)
X(I)=(-BB+SQRT(BB**2.0-4.0*AA*CC))/(2.0*AA)
C
CALL SSSN(I,D,DP,ES,X,EPC,EPS,EPPS,FS,FPS)
C
IF(FS(I).GE.FY) GO TO 50
IF(FPS(I).NE.FY) GO TO 50
C
CASE 2
JJ=2
PRINT*, JJ
C(I)=K(I)*X(I)
CS(I)=N

```

```

T(I)=M*(D/X(I))-M
YO(I)=PT1(I)*X(I)/PT2(I)
MO(I)=C(I)*(D-X(I)+YO(I))+CS(I)*(D-DP)
PHI(I)=EPC(I)/X(I)
GO TO 600

```

```

300 AA=1.0
BB=(1.0/K(I))*(L-P)
CC=-L*DP/K(I)
X(I)=(-BB+SQRT(BB**2.0-4.0*AA*CC))/(2.0*AA)

```

```

CALL SSSN(I,D,DP,ES,X,EPC,EPS,EPPS,FS,FPS)

```

```

IF(FPS(I).GE.FY) GO TO 400

```

```

CASE 3
JJ=3
PRINT*, JJ
C(I)=K(I)*X(I)
CS(I)=L-L*(DP/X(I))
T(I)=P
YO(I)=PT1(I)*X(I)/PT2(I)
MO(I)=C(I)*(D-X(I)+YO(I))+CS(I)*(D-DP)
PHI(I)=EPC(I)/X(I)
GO TO 600

```

```

400 X(I)=(1.0/K(I))*(P-N)

```

```

CALL SSSN(I,D,DP,ES,X,EPC,EPS,EPPS,FS,FPS)

```

```

CASE 4
JJ=4
PRINT*, JJ
C(I)=K(I)*X(I)
CS(I)=N
T(I)=P
YO(I)=PT1(I)*X(I)/PT2(I)
MO(I)=C(I)*(D-X(I)+YO(I))+CS(I)*(D-DP)
PHI(I)=EPC(I)/X(I)
GO TO 600

```

```

600 OPEN(UNIT=8,FILE='LPT1')
WRITE(8,601) JJ,EPC(I),EPS(I),EPPS(I)
601 FORMAT(1X,'CASE ',I1,4X,'CONCRETE STRAIN =' ,F8.6,4X,'STEEL STRAIN
* (T) =' ,F8.6,/, ' STEEL STRAIN (C) =' ,F8.6)
WRITE(8,602) C(I),CS(I),T(I),X(I),YO(I),MO(I),PHI(I)
602 FORMAT(1X,' C =' ,F5.1, ' KIPS' ,4X,'CS =' ,F5.1, ' KIPS' ,4X,'T =' ,
*F5.1, ' KIPS' ,4X,'X =' ,F7.2, ' IN' ,/, ' YO =' ,F7.2, ' INCHES' ,4X,
*'MOMENT =' ,F7.1, ' INCH-KIPS' ,4X,'PHI =' ,E14.7,///)

```

C

```
      IF(EPC(I).LE.EPCU) GO TO 699
      GO TO 750
699   EPC(I+1)=EPC(I)+DELTA
700   CONTINUE
750   STOP
      END
```

C

C

C

```
      SUBROUTINE SSSN(I,D,DP,ES,X,EPC,EPS,EPPS,FS,FPS)
      DIMENSION X(1000),EPC(1000),EPS(1000),EPPS(1000),
*FS(1000),FPS(1000)
```

C

```
      EPS(I)=(D-X(I))*EPC(I)/X(I)
      EPPS(I)=(X(I)-DP)*EPC(I)/X(I)
      FS(I)=ES*EPS(I)
      FPS(I)=ES*EPPS(I)
      RETURN
      END
```

APPENDIX C.
MATERIAL SPECIFICATIONS

REPORT OF CONCRETE CORES
UNIVERSITY OF NEBRASKA BARRIER TESTING

Project: Nebraska Bridge Rail Crash Test

Examined For: Compressive Strength

Date		Location	Strength - PSI
Placed	Tested		
6-20-91	7-11-91	Bridge Deck	6080
6-20-91	7-18-91	Bridge Deck	6240
7-01-91	7-11-91	Bridge Rail	4430
7-01-91	7-29-91	Bridge Rail	5200

Remarks:


For NDOR Materials & Tests Division



SIMCOTE, INC.

CERTIFICATE OF COMPLIANCE

6-13-91

TO: CONCRETE INDUSTRIES
6300 CORNHUSKER HIGHWAY
P.O. BOX 29298
LINCOLN, NE 68529

ATTENTION: Bridge Engineer in Charge

Re: Project No: STOCK MATERIAL-TEST RAIL
County: Lincoln Co. NE
Contractor: UNL

Gentlemen:

The representative samples of the coated bars have been coated and tested and they conform to the requirements of the State of Nebraska Department of Roads Specification.

Very Truly Yours,

SIMCOTE, INC.

Robert P. Simmet
Vice President
General Manager



1645 Red Rock Rd. • P.O. Box 9 • Newport, MN 55055
Phone (612) 735-9660 • Fax # (612) 735-9664



250 N. Greenwood • Marion, OH 43302
Phone (614) 382-5000 • Fax # (614) 383-1167



REPORT OF CONCRETE CORES

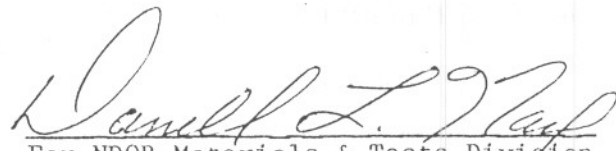
UNIVERSITY OF NEBRASKA BARRIER TESTING

Project: Nebraska Bridge Rail Crash Test

Examined For: Compressive Strength

Date		Location	Strength - PSI
Placed	Tested		
6-20-91	7-11-91	Bridge Deck	6080
6-20-91	7-18-91	Bridge Deck	6240
7-01-91	7-11-91	Bridge Rail	4430
7-01-91	7-29-91	Bridge Rail	5200

Remarks:


For NDOR Materials & Tests Division

B38962



THE MARION STEEL CO.

THE MARION STEEL COMPANY

912 CHENEY AVE.
MARION, OHIO 43302

CERTIFIED TEST REPORT

HEAT # B38962

SIZE #5 REBAR

C	Mn	P	S	Si	Cu	Cr	Mo	Ni	Sn	Cb	V	Other
.41	.94	.038	.036	.22	.44	.09	.02	.08	.021			

CHEMICAL
ANALYSIS

SPECIMEN AREA	YIELD KIPS	YIELD K.S.I.	TENSILE KIPS	TENSILE K.S.I.	GA. LENGTH	ELON %	BEND	R/R %		GRADE	DATE ROLLED
		62600		92200	8 IN.	15.0	OK			G60	5-31-91
										ASTM-A615-89	

THIS MATERIAL HAS BEEN PRODUCED AND TESTED IN ACCORDANCE WITH THE REQUIREMENTS OF APPLICABLE A.S.T.M. SPECIFICATIONS UNLESS OTHERWISE LISTED BELOW. WE HEREBY CERTIFY THAT THE ABOVE TEST RESULTS ARE REPRESENTATIVE OF THOSE CONTAINED IN THE RECORDS OF THE COMPANY. BEND TEST CONFORMS TO ACI 318-77 (1975 SUPPLEMENT).

SWORN AND SUBSCRIBED TO BEFORE ME

THIS _____ DAY _____

(Notary Public)

MY COMMISSION EXPIRES _____

SIGNED

Lab Tech.

DATE _____

APPROVED BY

THIS CERTIFICATE IS NOTARIZED ONLY WHEN REQUESTED.



NORTH STAR STEEL IOWA

P.O. Box 749
Corner of Greens Road & Highway 38
Wilton, Iowa 52778

CERTIFIED TEST REPORT

Heat # 1-48745

Size #3 REBAR

Grade A615-87/60

Roll Date 7/5/90

CHEMICAL ANALYSIS

C	MN	P	S	SI	CU	NI	CR	MO	SN	CB	V
.50	0.947	0.032	.060	0.24	0.282	0.11	0.10	0.02	0.014	0.004	

AL

--	--	--	--	--

JOMINY END-QUENCH HARDENABILITY RESULTS (HR_c)

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12

J13	J14	J15	J16	J18	J20	J22	J24	J26	J28	J30	J32

PHYSICAL TEST REPORT

SPECIMEN AREA (in ²)	YIELD KIPS	YIELD K.S.I.	TENSILE KIPS	TENSIL K.S.I.	GAUGE LENGTH (in)	% ELONG	BEND	% R.A.	
.11		76.3		117.3		12.5	3.5d		
.11		77.3		117.7		12.5	3.5d		

Grain Size: _____ Cleanliness: _____

Macro Etch: Specification: _____ Results: _____

Impact Test	Temp (F)	ft./lb.	% Shear*	% L.E.*

*By specification or request only

Reinforcing steel covered by all shipments and mill analysis has been produced in the United States of America including manufacturing of ingots.

This material has been produced and tested in accordance with the requirements of applicable specifications unless otherwise listed below. We hereby certify that the above test results are representative of those contained in the records of the company.

Any modification to this certification as provided by North Star Steel Iowa without the expressed written consent of North Star Steel Iowa negates the validity of this test report.

North Star Steel Iowa is not responsible for the inability of this material to meet specific applications.

P.O. # _____

ASTM A36-84a

ASME SA36-81a

1982 SUMMER ADDENDUM THIS _____ DAY _____

SWORN AND SUBSCRIBED TO BEFORE ME

(Notary Public)

MY COMMISSION EXPIRES _____

THIS CERTIFICATE IS NOTARIZED ONLY WHEN REQUESTED.

July 9, 1990

DATE

148745



NORTH STAR STEEL MINNESOTA

P.O. Box 64189
1678 Red Rock Road
Saint Paul, Minnesota 55164

CERTIFIED TEST REPORT

Heat # **A46681**

Size **#8 Rb**

Grade **60**

Date Rolled **11/14/90**

P.O. _____

M.O. # _____

CHEMICAL ANALYSIS

C	MN	P	S	SI	SN	CU	NI	CR	MO	CB	V
.40	.82	.026	.041	.24	.008	.29	.11	.13	.043	.020	

AL "ALL MANUFACTURING PROCESSES FOR THIS STEEL, INCLUDING MELTING FROM SCRAP, HAVE BEEN PERFORMED IN THE U. S. A. AT NORTH STAR STEEL MINNESOTA, 1678 RED ROCK ROAD, ST. PAUL, MINNESOTA, USA." NO WELD REPAIRMENT PERFORMED

JOMINY END-QUENCH HARDENABILITY RESULTS (HR_c)

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12
J13	J14	J15	J16	J18	J20	J22	J24	J26	J28	J30	J32

PHYSICAL TEST REPORT

SPECIMEN AREA (in ²)	YIELD KIPS	YIELD K.S.I.	TENSILE KIPS	TENSILE K.S.I.	GAUGE LENGTH (in)	% ELONG	BEND	% R.A.	Specification/Grade
.79	52.5	66.4	81.0	102.5	8.0	14.4	OK		A615-89
									Gr 60

Grain Size: _____ Cleanliness: _____

Macro Etch: Specification: _____ Results: _____

Impact Test	Temp (F)	ft./lb.	% Shear*	% L.E.*

* By specification or request only

This material has been produced and tested in accordance with the requirements of applicable specifications unless otherwise listed below. We hereby certify that the above test results are representative of those contained in the records of the company.

Any modification to this certification as provided by North Star Steel Minnesota without the expressed written consent of North Star Steel Minnesota negates the validity of this test report.

North Star Steel Minnesota is not responsible for the inability of this material to meet specific applications.

SIGNED _____

DATE **11/14/90**

APPROVAL _____

SWORN AND SUBSCRIBED TO BEFORE ME

THIS _____ DAY _____

(Notary Public)

MY COMMISSION EXPIRES _____

THIS CERTIFICATE IS NOTORIZED ONLY WHEN REQUESTED.

QC-7 Rev. 10-86

A46681

APPENDIX D.

ACCELEROMETER DATA ANALYSIS, NEOCR-1

Figure D-1 Sketch of Accelerometer Locations, Test NEOCR-1

Figure D-2 Graph of Longitudinal Deceleration, Acc. #1

Figure D-3 Graph of Vehicle Change in Speed, Acc. #1

Figure D-4 Graph of Longitudinal Occupant Displacement, Acc. #1

Figure D-5 Graph of Longitudinal Deceleration, Acc. #2

Figure D-6 Graph of Vehicle Change in Speed, Acc. #2

Figure D-7 Graph of Longitudinal Occupant Displacement, Acc. #2

Figure D-8 Graph of Lateral Deceleration, Acc. #3

Figure D-9 Graph of Lateral Occupant Impact Velocity, Acc. #3

Figure D-10 Graph of Lateral Occupant Displacement, Acc. #3

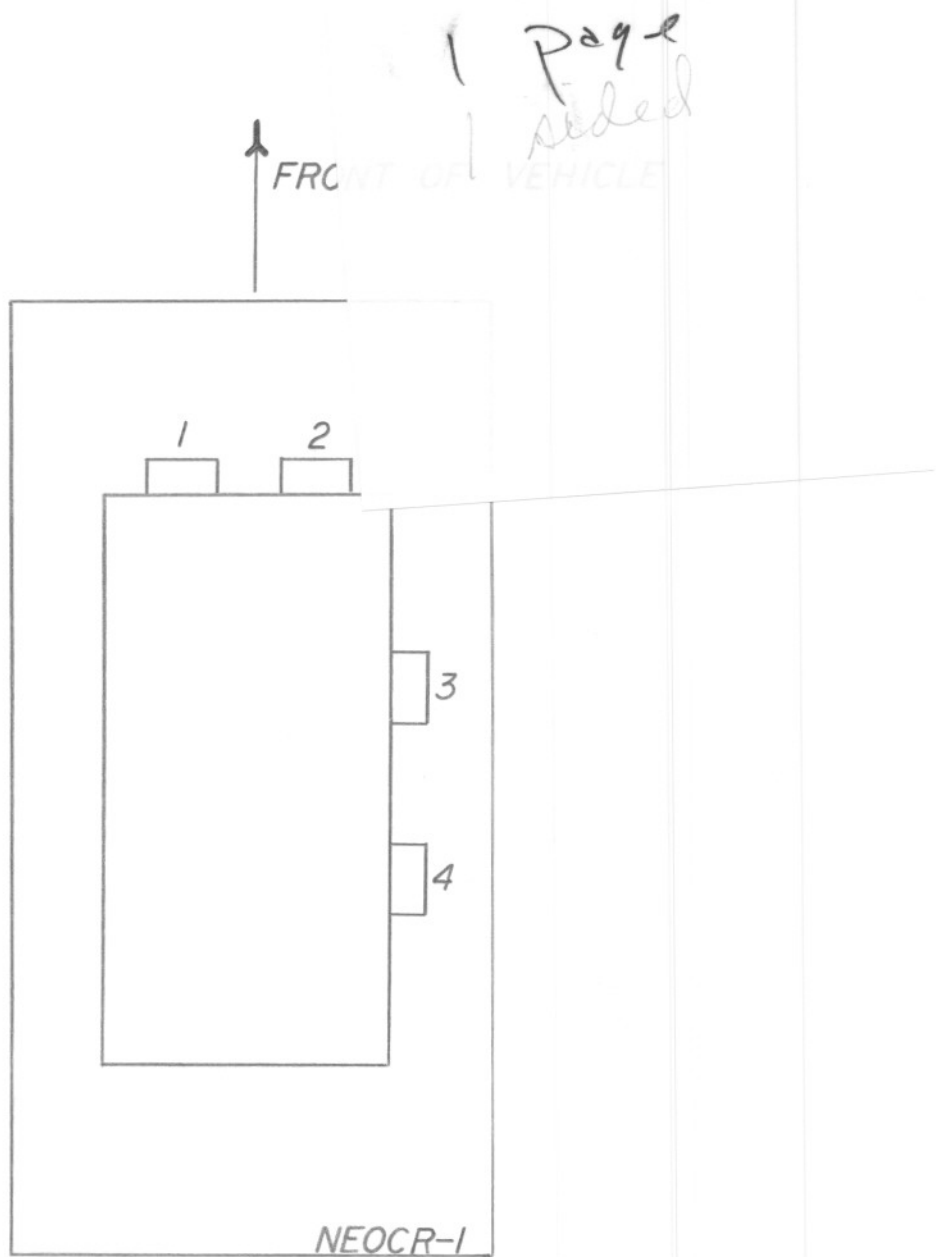


FIGURE D-1. SKETCH OF ACCELEROMETER LOCATIONS, TEST NEOCR-1

NER1601

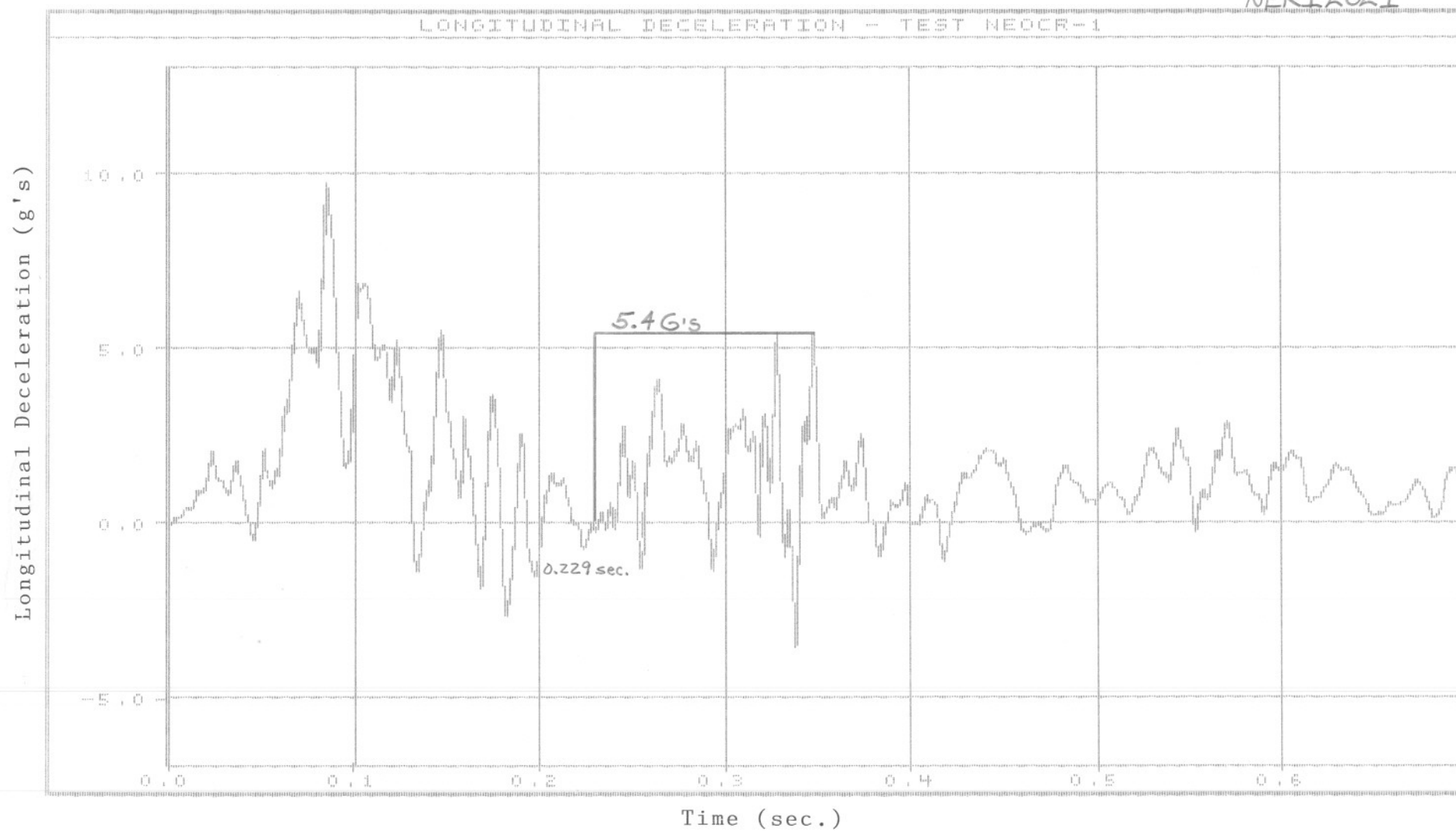


FIGURE D-2. GRAPH OF LONGITUDINAL DECELERATION, ACC. #1

NER17011

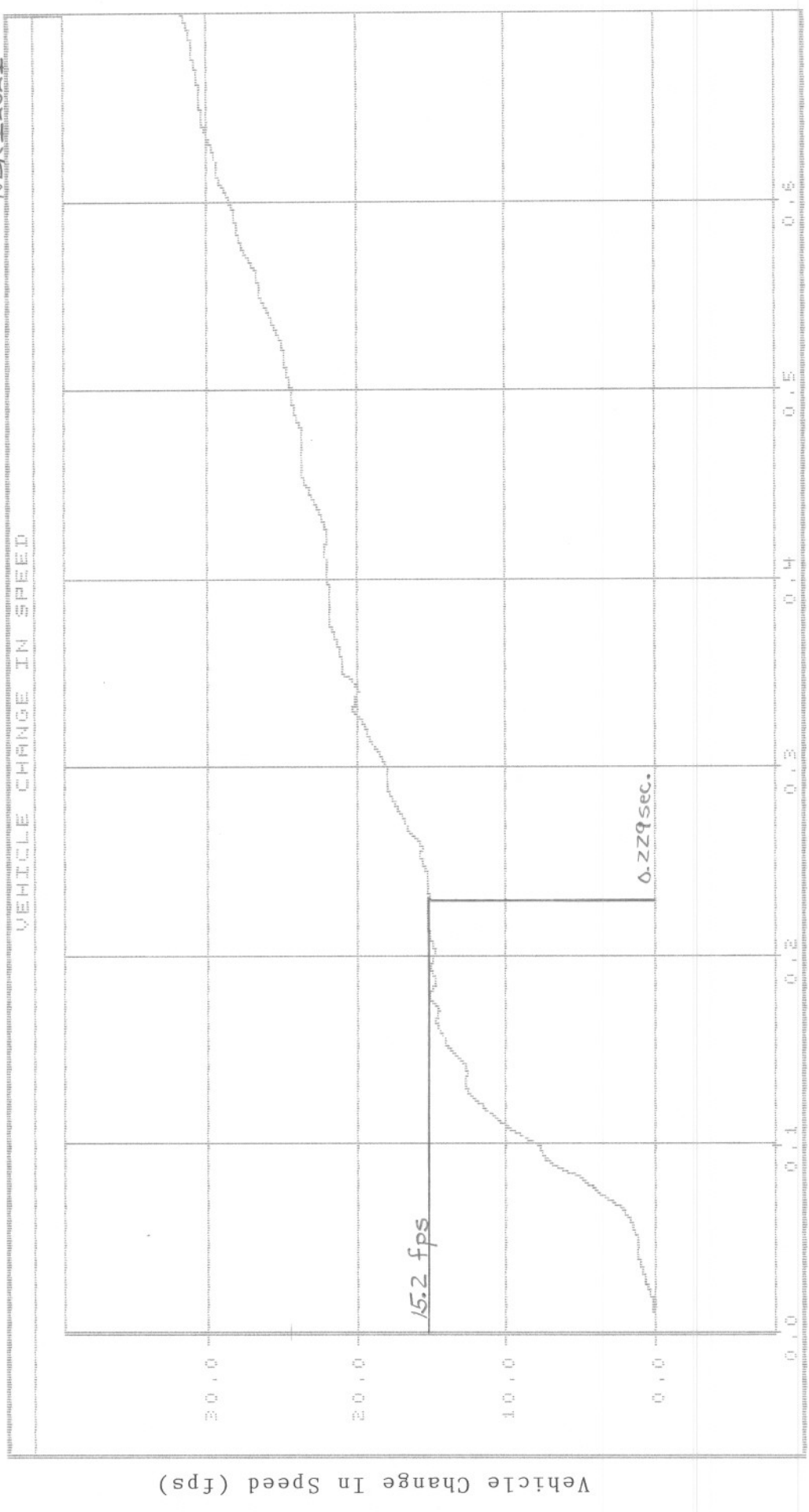


FIGURE D-3. GRAPH OF VEHICLE CHANGE IN SPEED, ACC. #1

NER11011

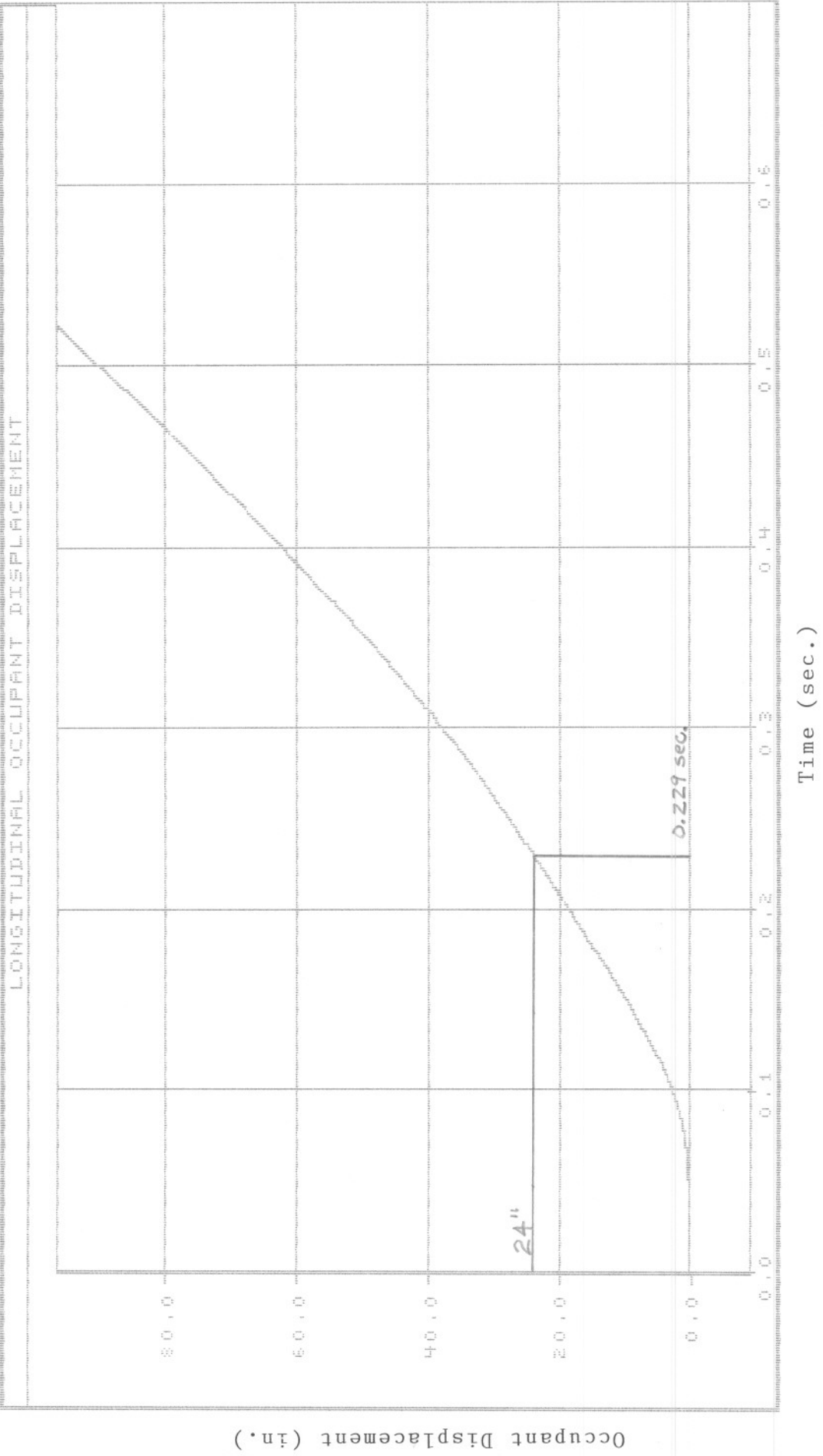
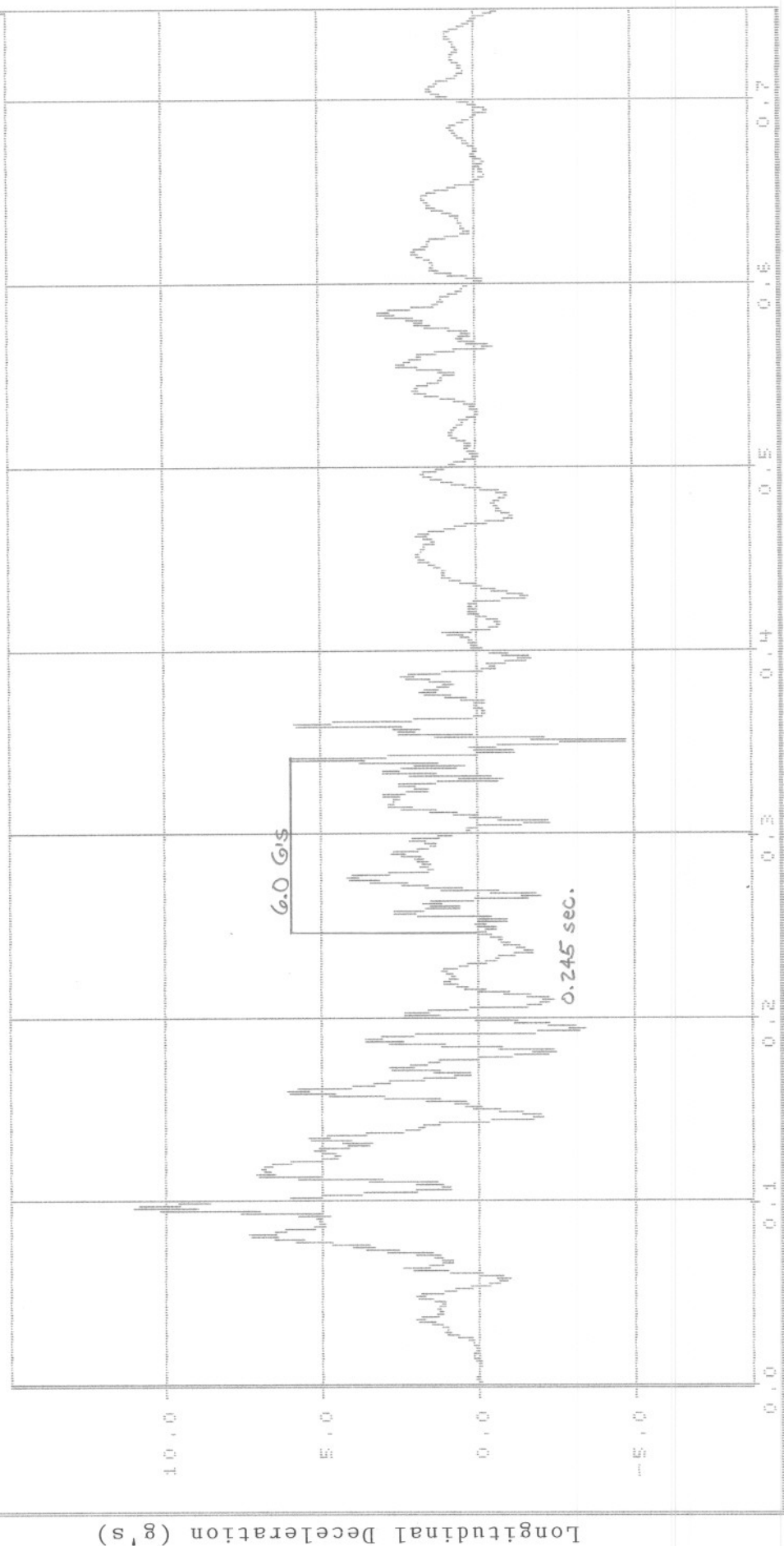


FIGURE D-4. GRAPH OF LONGITUDINAL OCCUPANT DISPLACEMENT, ACC. #1

NER 160RZ

LONGITUDINAL DECELERATION - TEST NEOCR-1



Time (sec.)

FIGURE D-5. GRAPH OF LONGITUDINAL DECELERATION, ACC.#2

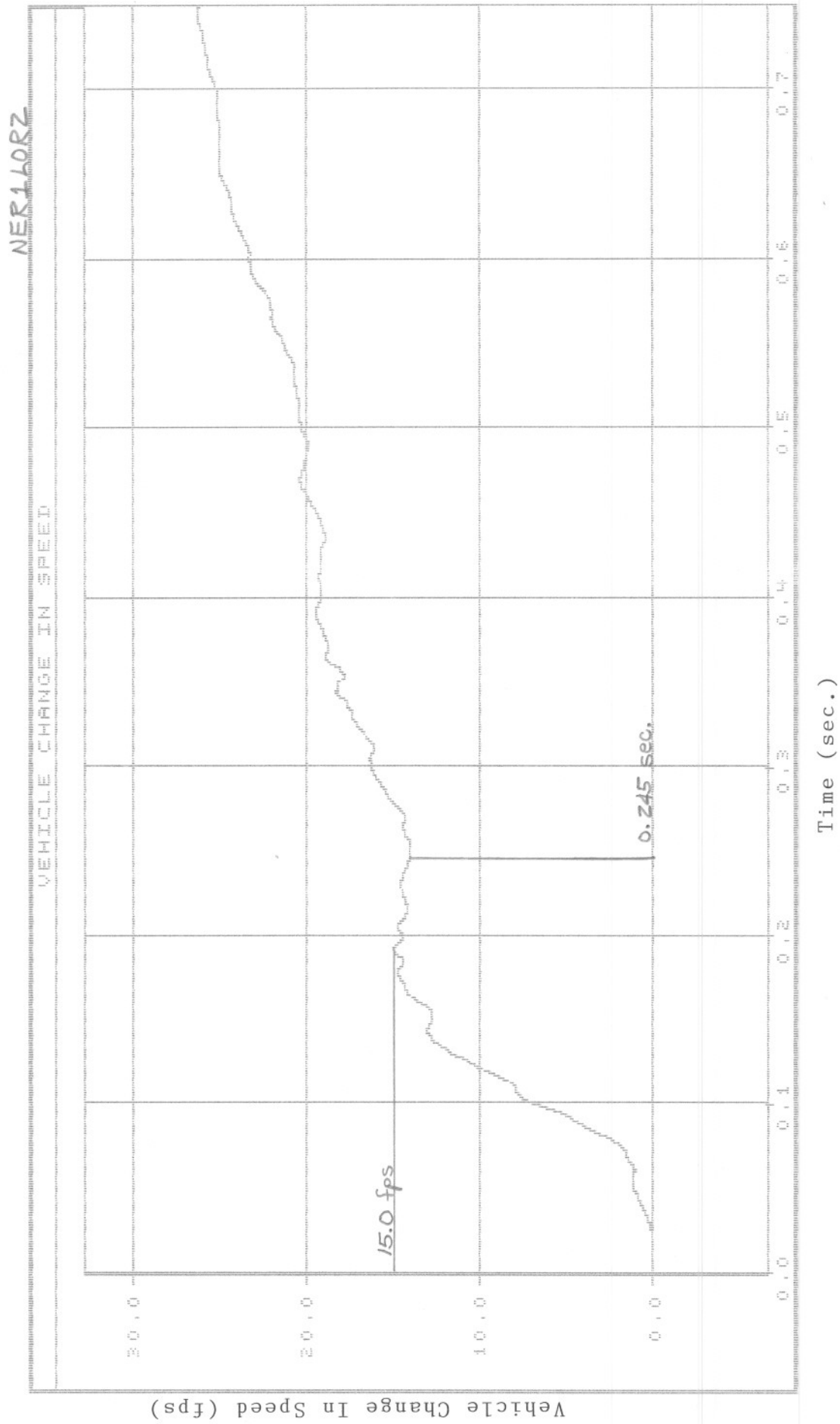
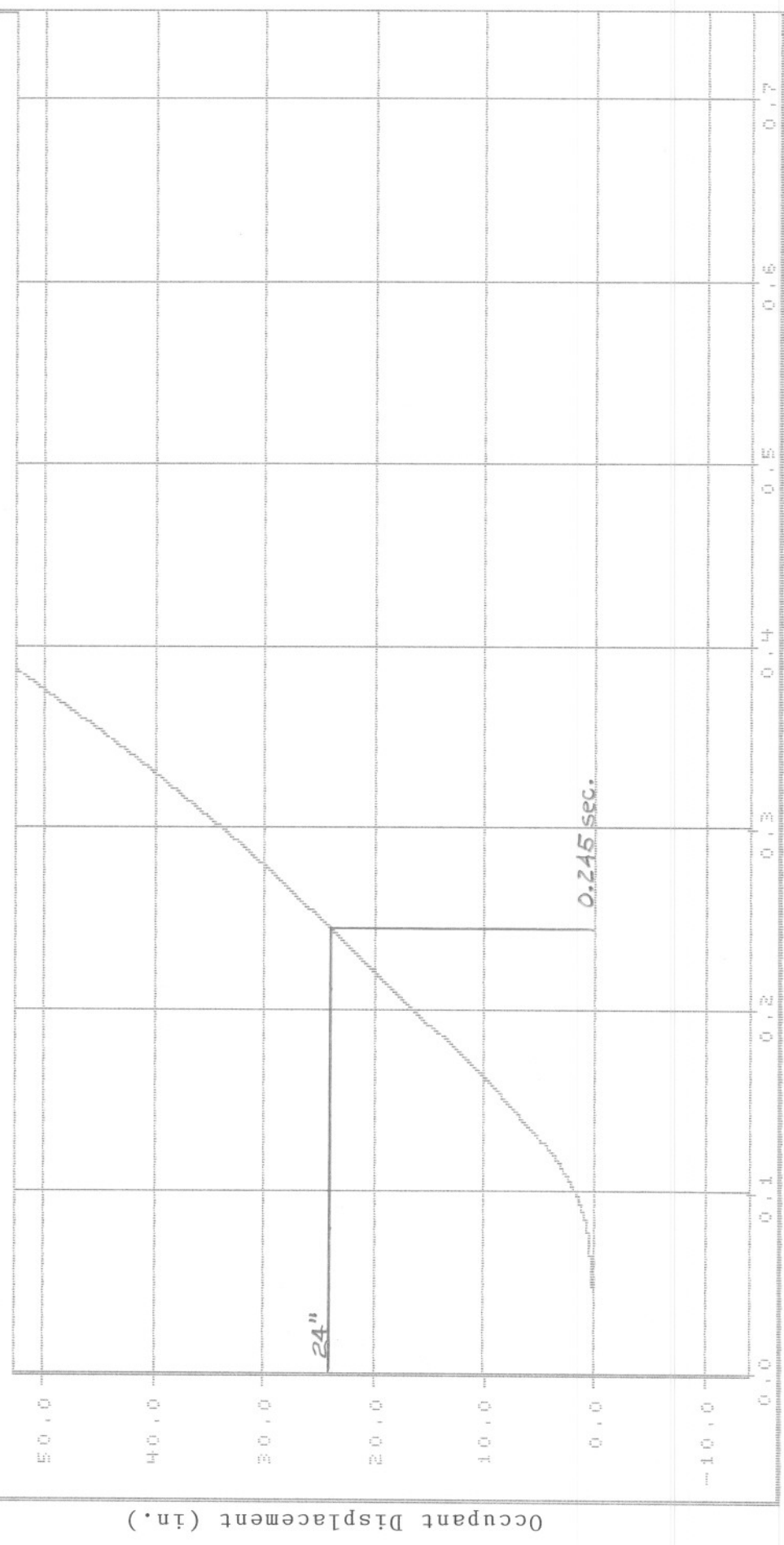


FIGURE D-6. GRAPH OF VEHICLE CHANGE IN SPEED, ACC. #2

NER140RZ

LONGITUDINAL OCCUPANT DISPLACEMENT



Time (sec.)

FIGURE D-7. GRAPH OF LONGITUDINAL OCCUPANT DISPLACEMENT, ACC. #2

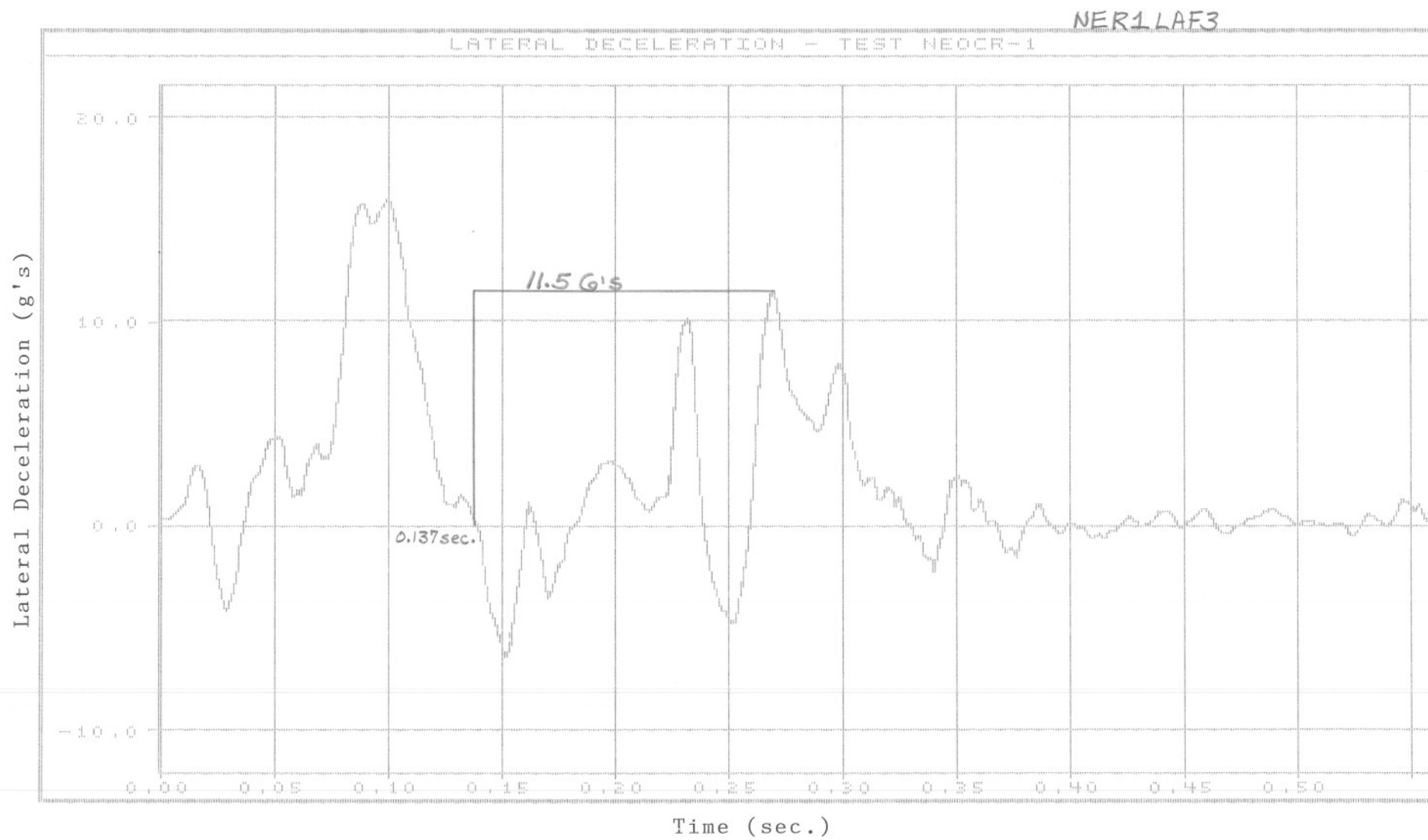


FIGURE D-8. GRAPH OF LATERAL DECELERATION, ACC. #3

NER12AF3

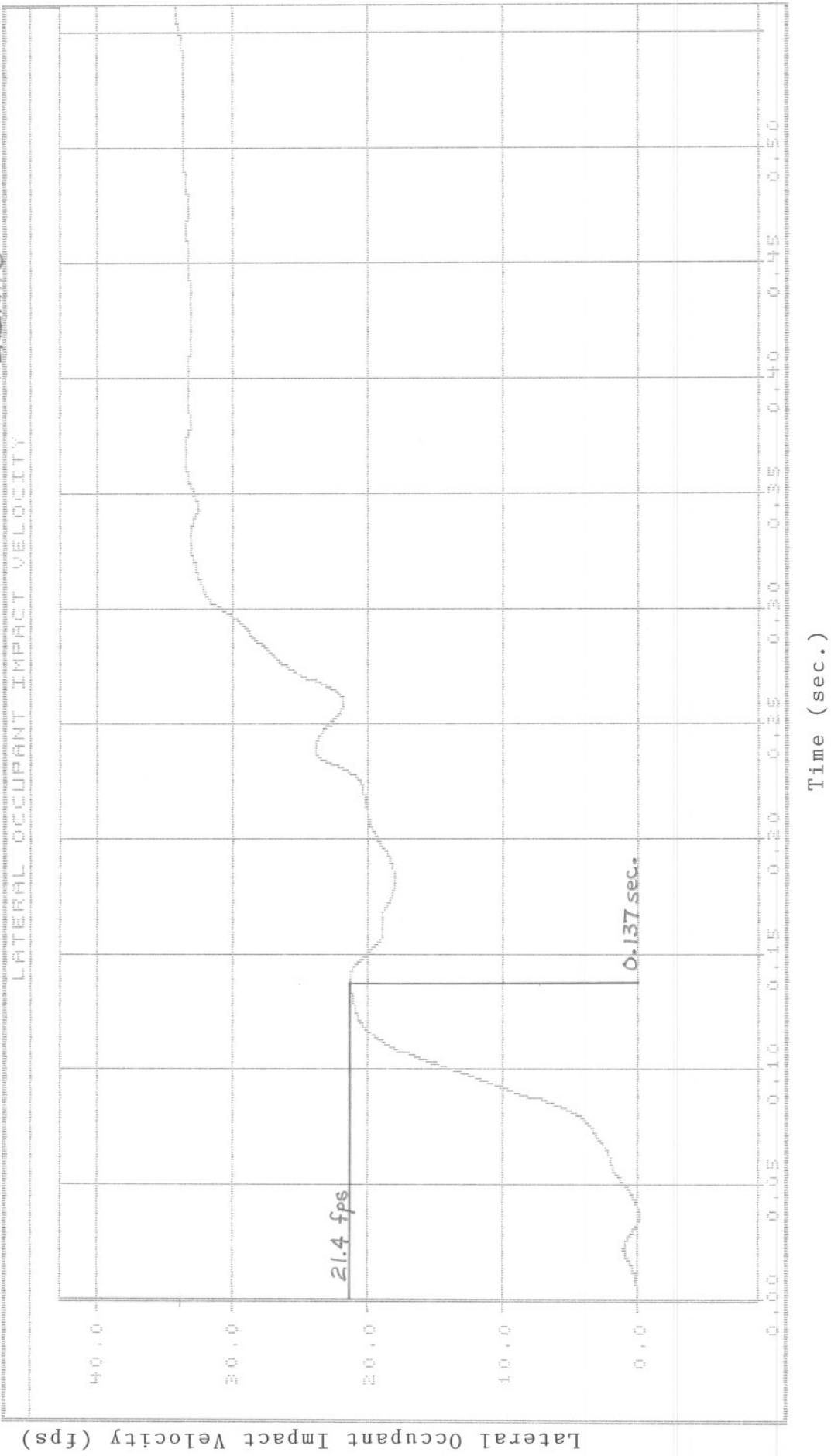


FIGURE D-9. GRAPH OF LATERAL OCCUPANT IMPACT VELOCITY, ACC. #3

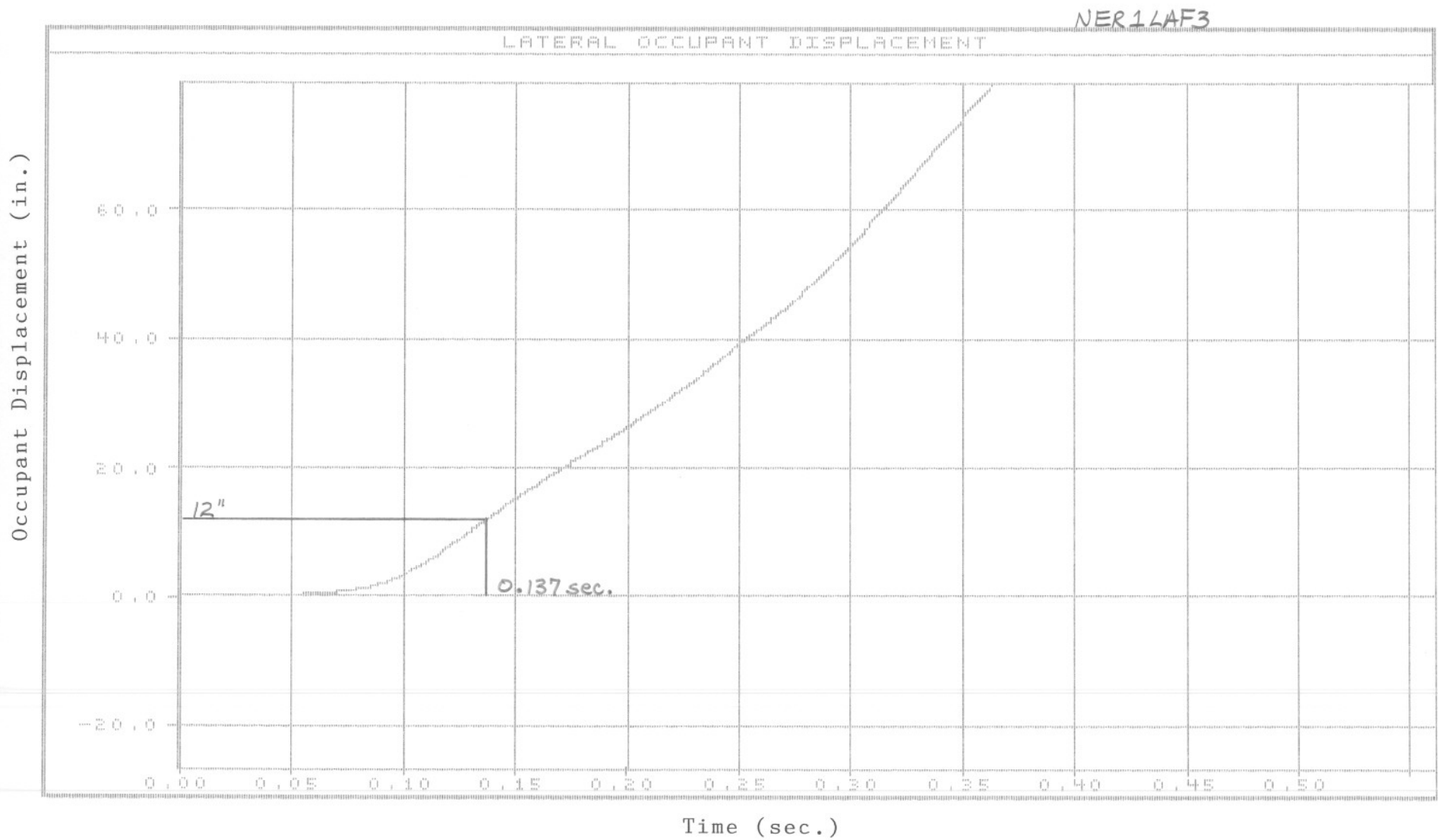


FIGURE D-10. GRAPH OF LATERAL OCCUPANT DISPLACEMENT, ACC. #3

APPENDIX E.

ACCELEROMETER DATA ANALYSIS, NEOCR-2

- Figure E-1 Sketch of accelerometer locations, Test NEOCR-2
- Figure E-2 Graph of Lateral Deceleration, Acc. #1
- Figure E-3 Graph of Lateral Occupant Impact Velocity, Acc. #1
- Figure E-4 Graph of Lateral Occupant Displacement, Acc. #1
- Figure E-5 Graph of Lateral Deceleration, Acc. #2
- Figure E-6 Graph of Lateral Occupant Impact Velocity, Acc. #2
- Figure E-7 Graph of Lateral Occupant Displacement, Acc. #2
- Figure E-8 Graph of Longitudinal Deceleration, Acc. #3
- Figure E-9 Graph of Vehicle Change in Speed , Acc. #3
- Figure E-10 Graph of Longitudinal Occupant Displacement, Acc. #3
- Figure E-11 Graph of Longitudinal Deceleration, Acc. #4
- Figure E-12 Graph of Vehicle Change in Speed, Acc. #4
- Figure E-13 Graph of Longitudinal Occupant Displacement, Acc. #4

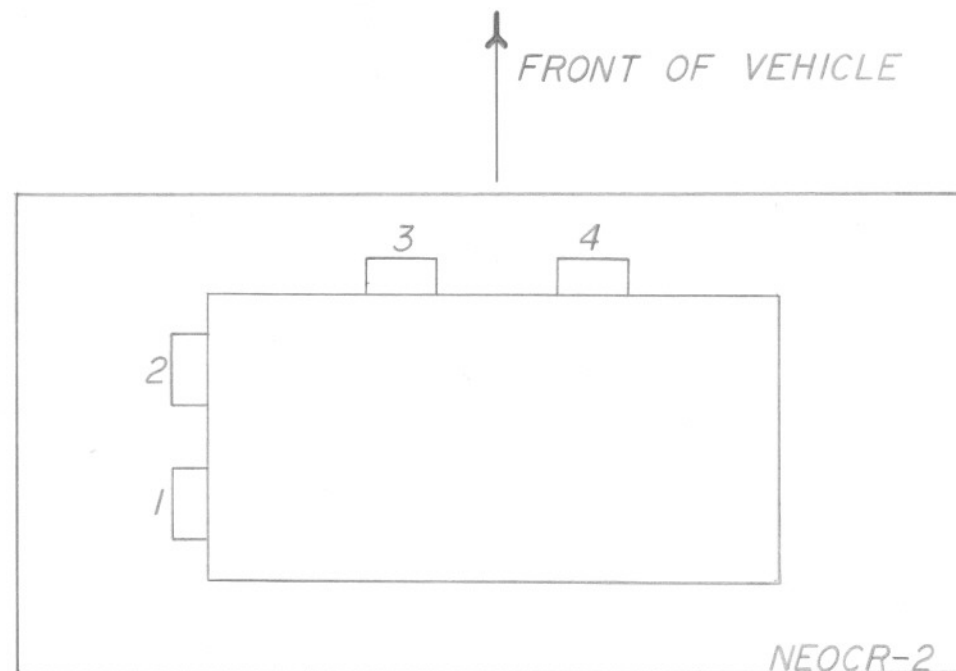


FIGURE E-1. SKETCH OF ACCELEROMETER LOCATIONS, TEST NEOCR-2

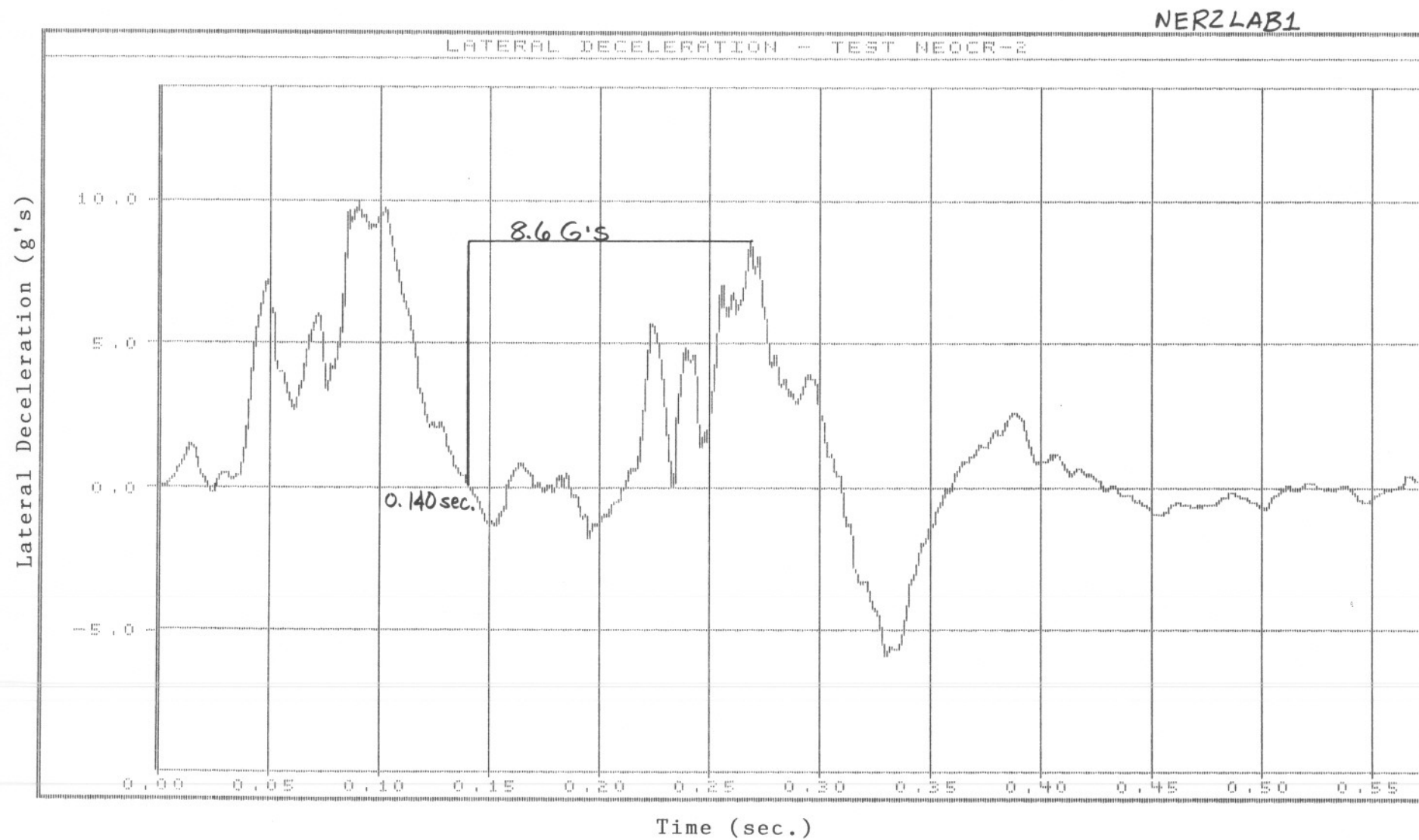


FIGURE E-2. GRAPH OF LATERAL DECELERATION, ACC. #1

NER22LAB1

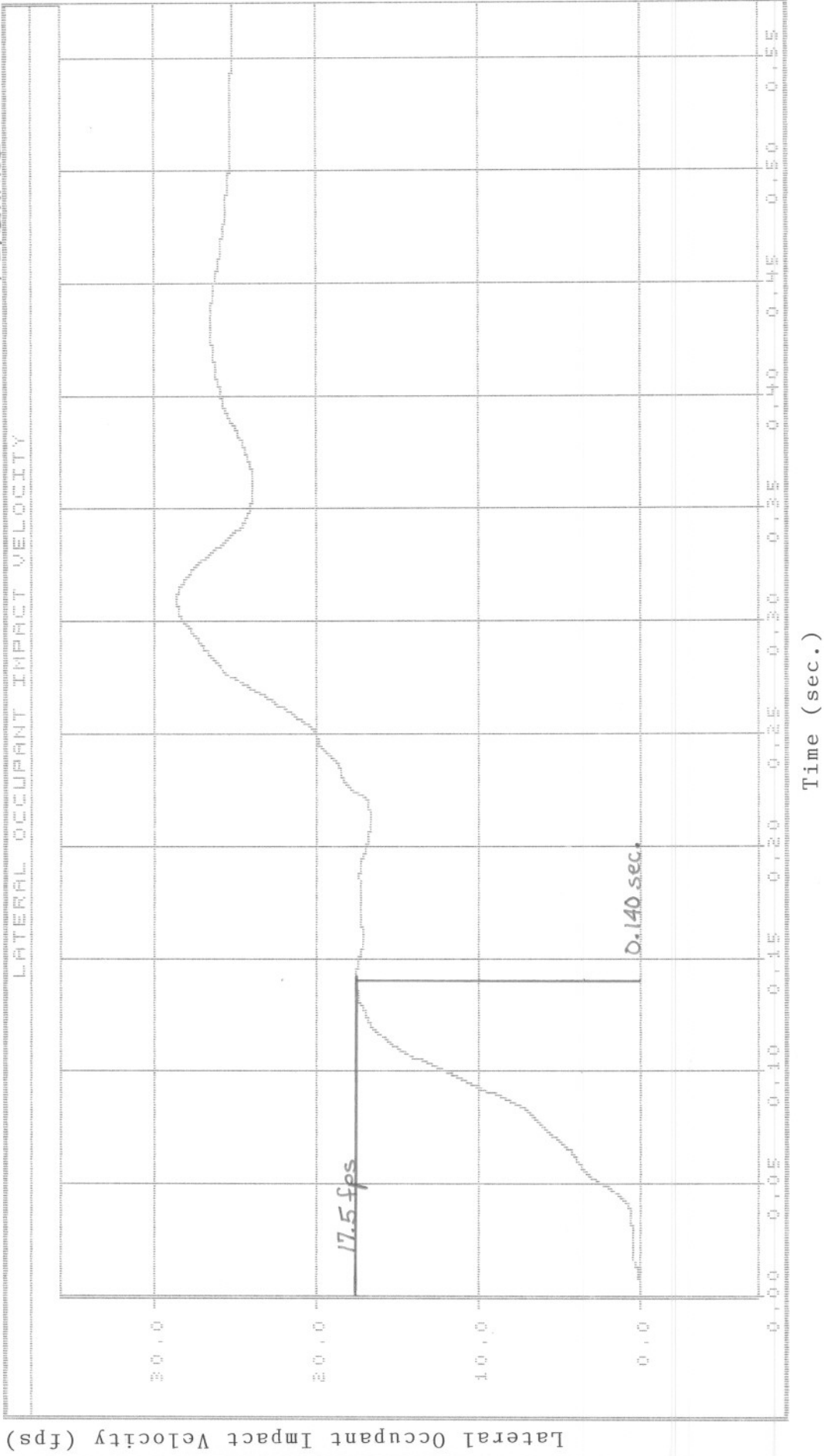


FIGURE E-3. GRAPH OF LATERAL OCCUPANT IMPACT VELOCITY, ACC. #1

NERZLAB4

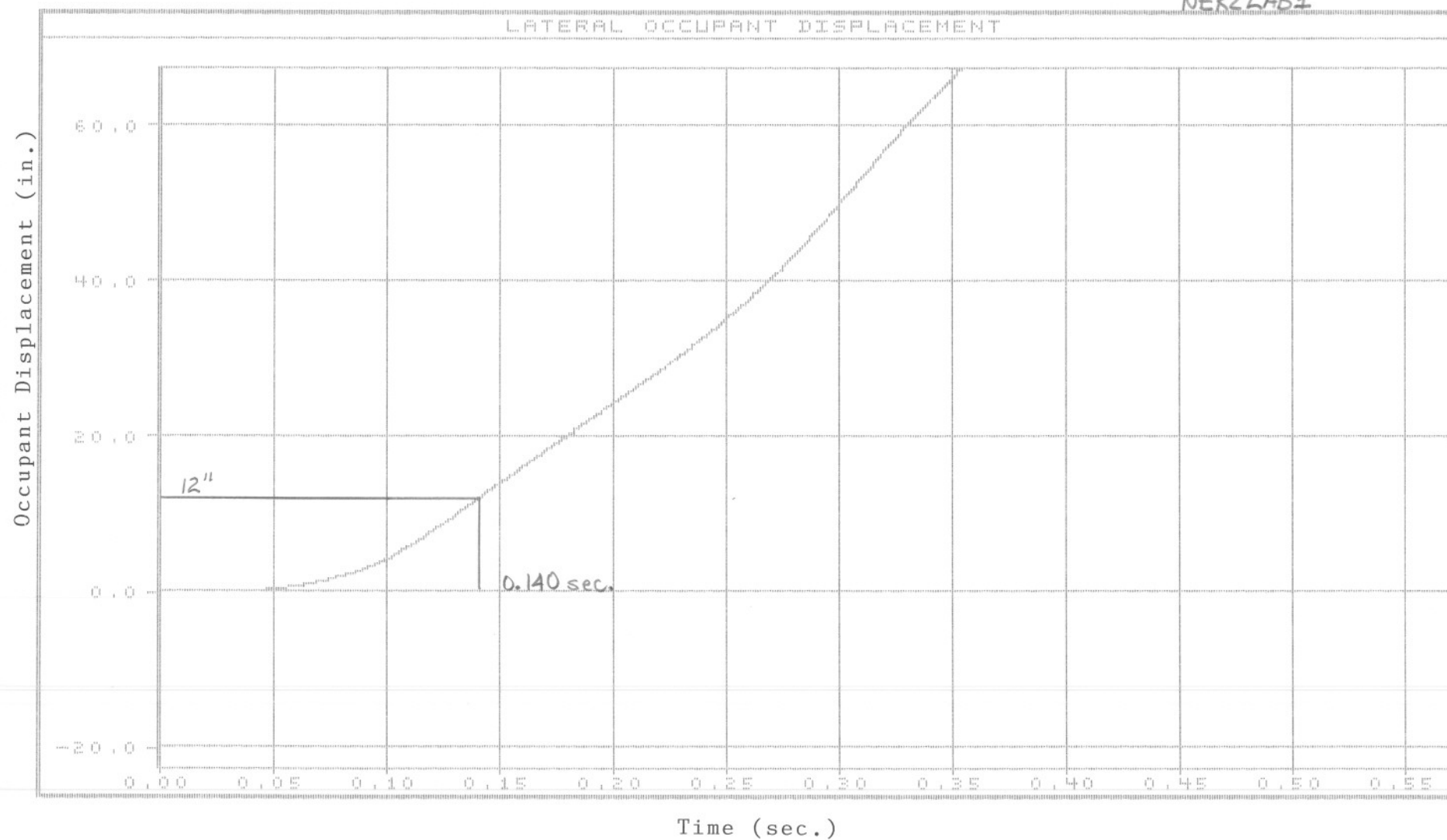


FIGURE E-4. GRAPH OF LATERAL OCCUPANT DISPLACEMENT, ACC. #1

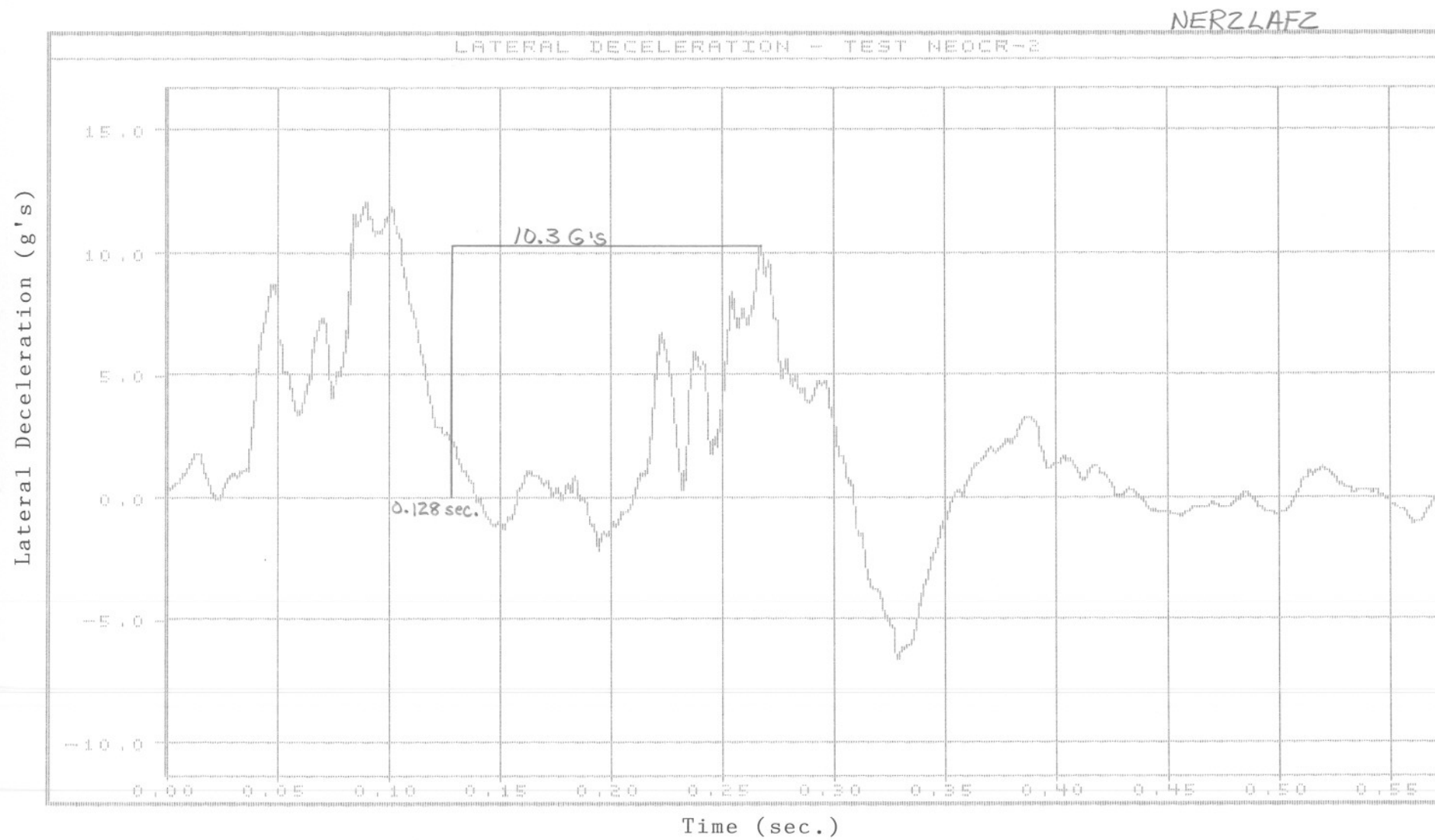


FIGURE E-5. GRAPH OF LATERAL DECELERATION, ACC. #2

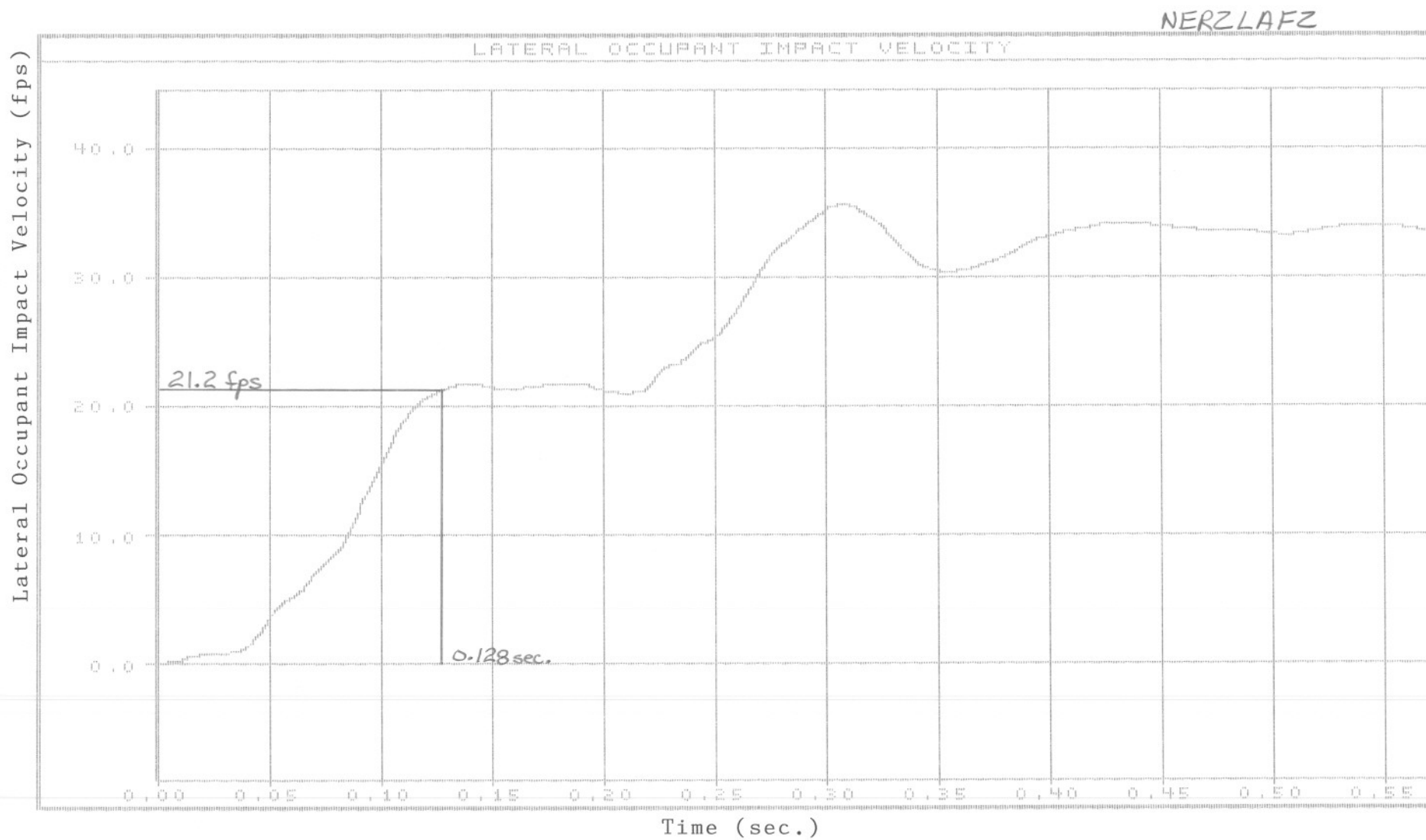


FIGURE E-6. GRAPH OF LATERAL OCCUPANT IMPACT VELOCITY, ACC. #2

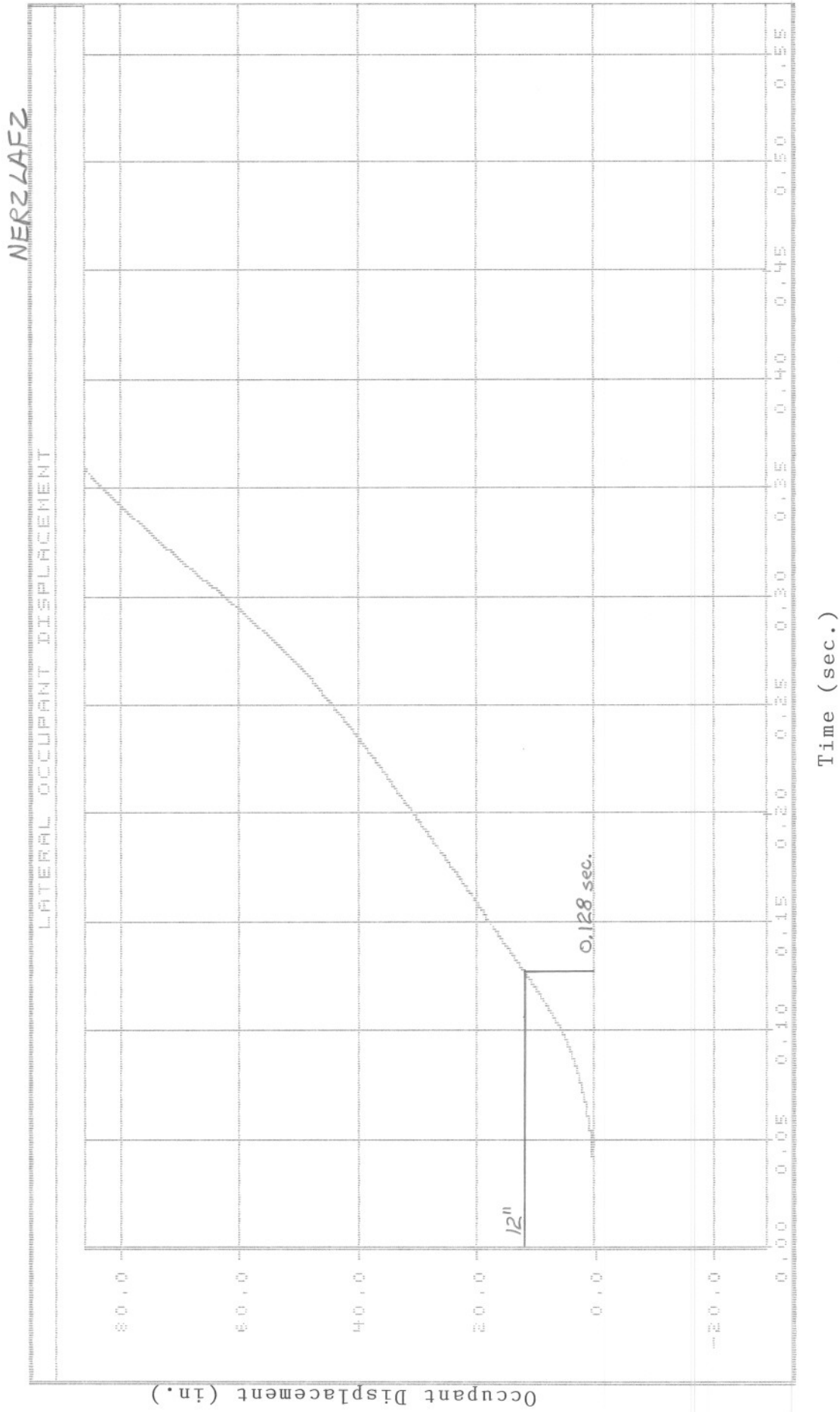


FIGURE E-7. GRAPH OF LATERAL OCCUPANT DISPLACEMENT, ACC. #2

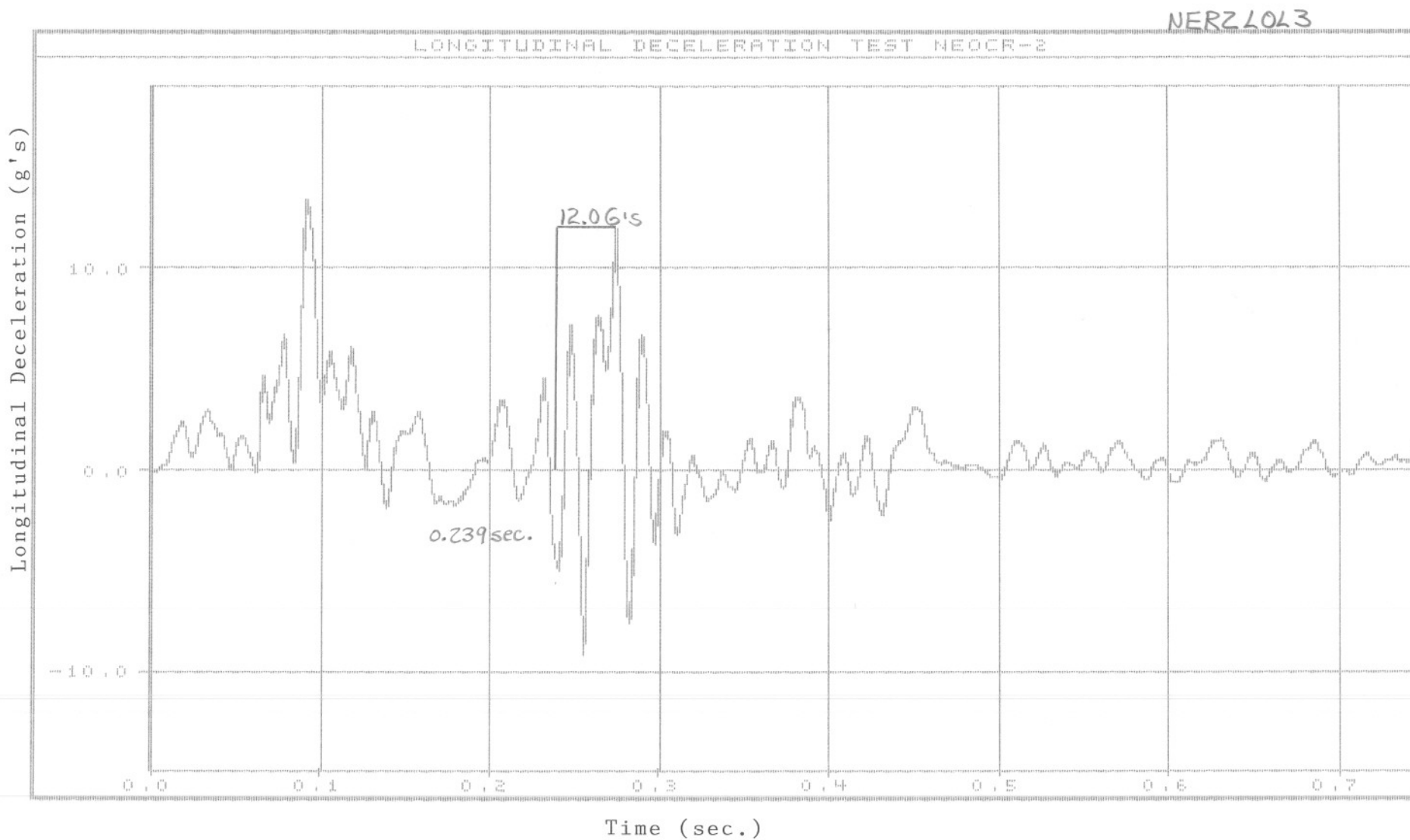


FIGURE E-8. GRAPH OF LONGITUDINAL DECELERATION, ACC. #3

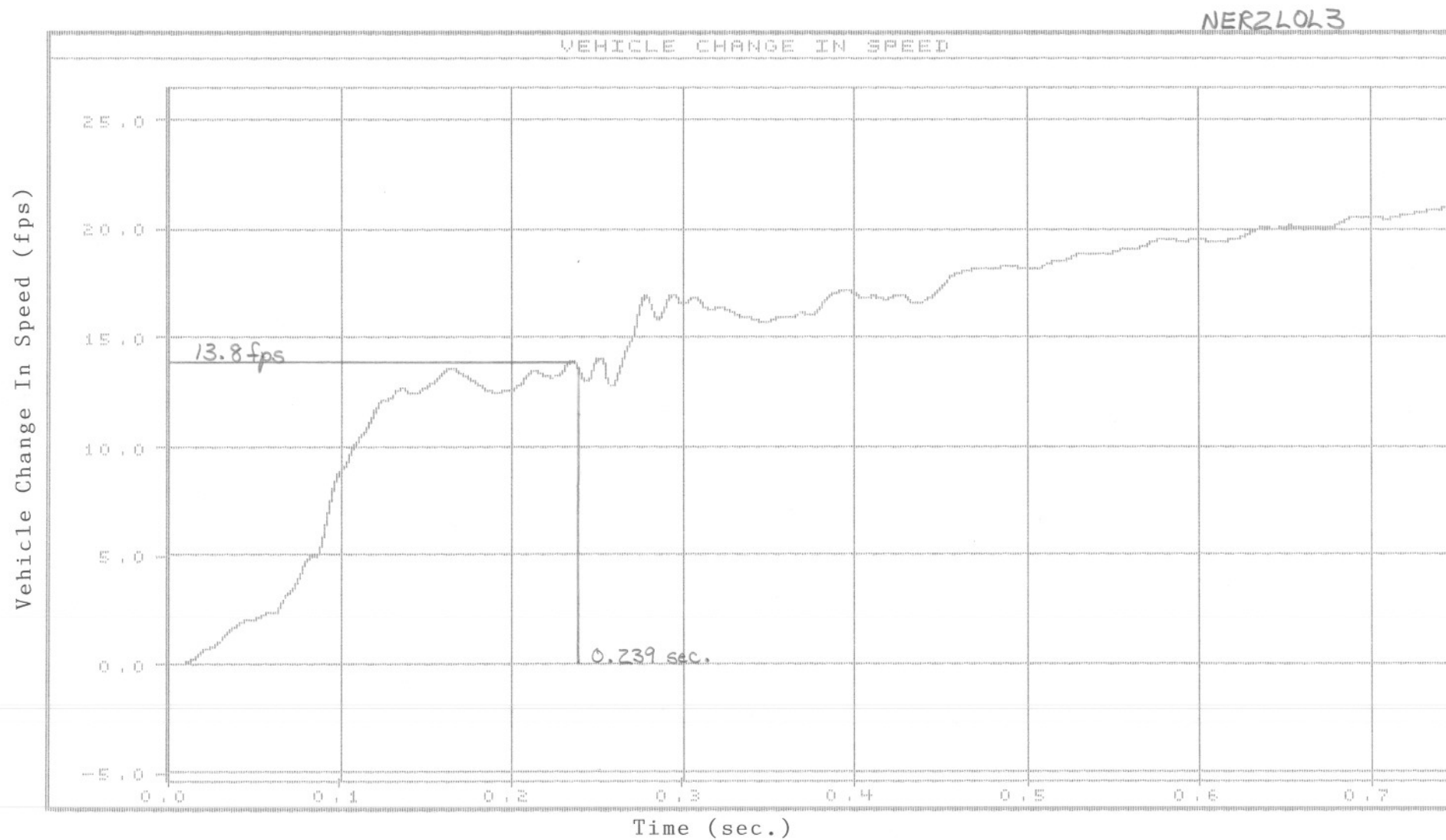


FIGURE E-9. GRAPH OF VEHICLE CHANGE IN SPEED, ACC. #3

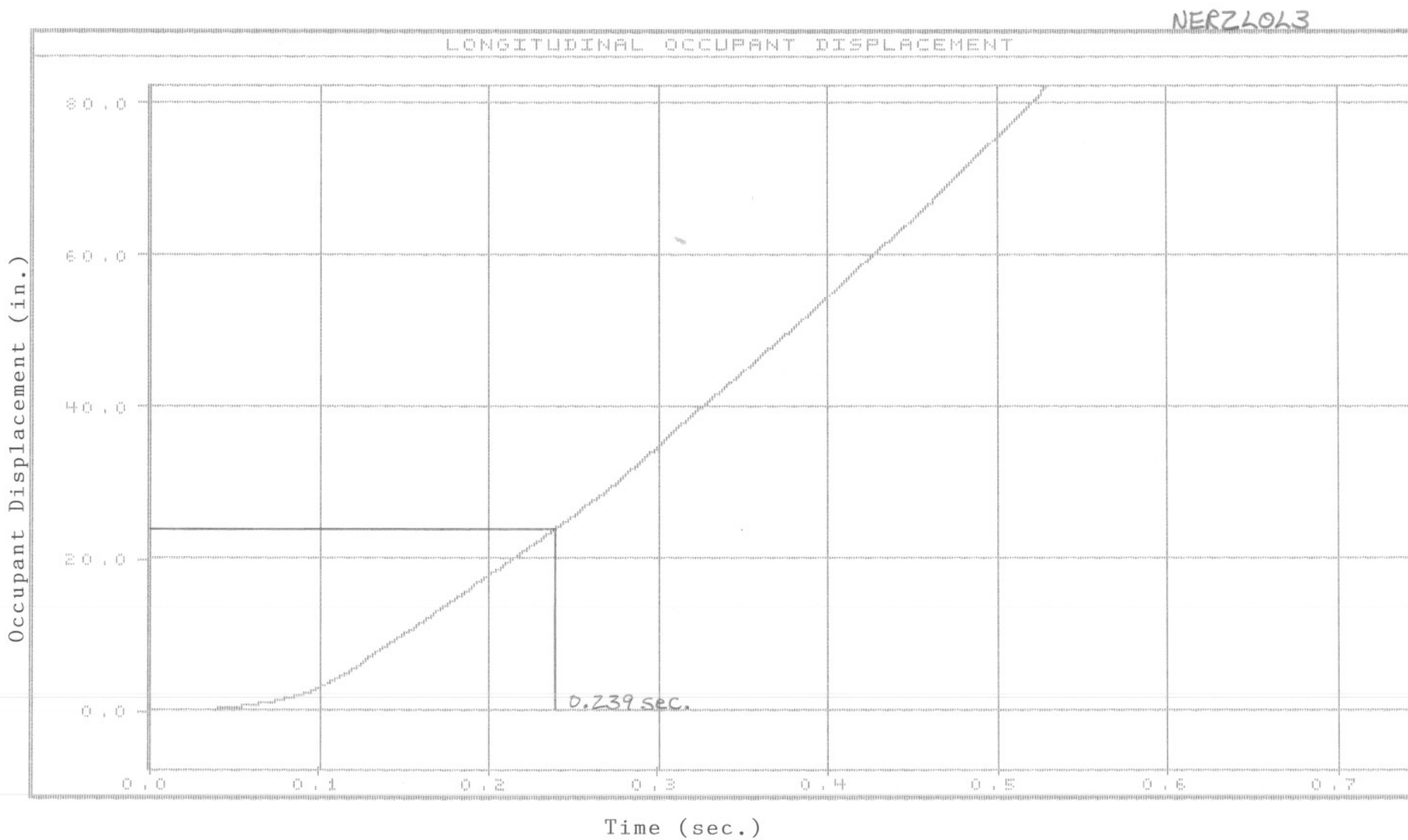
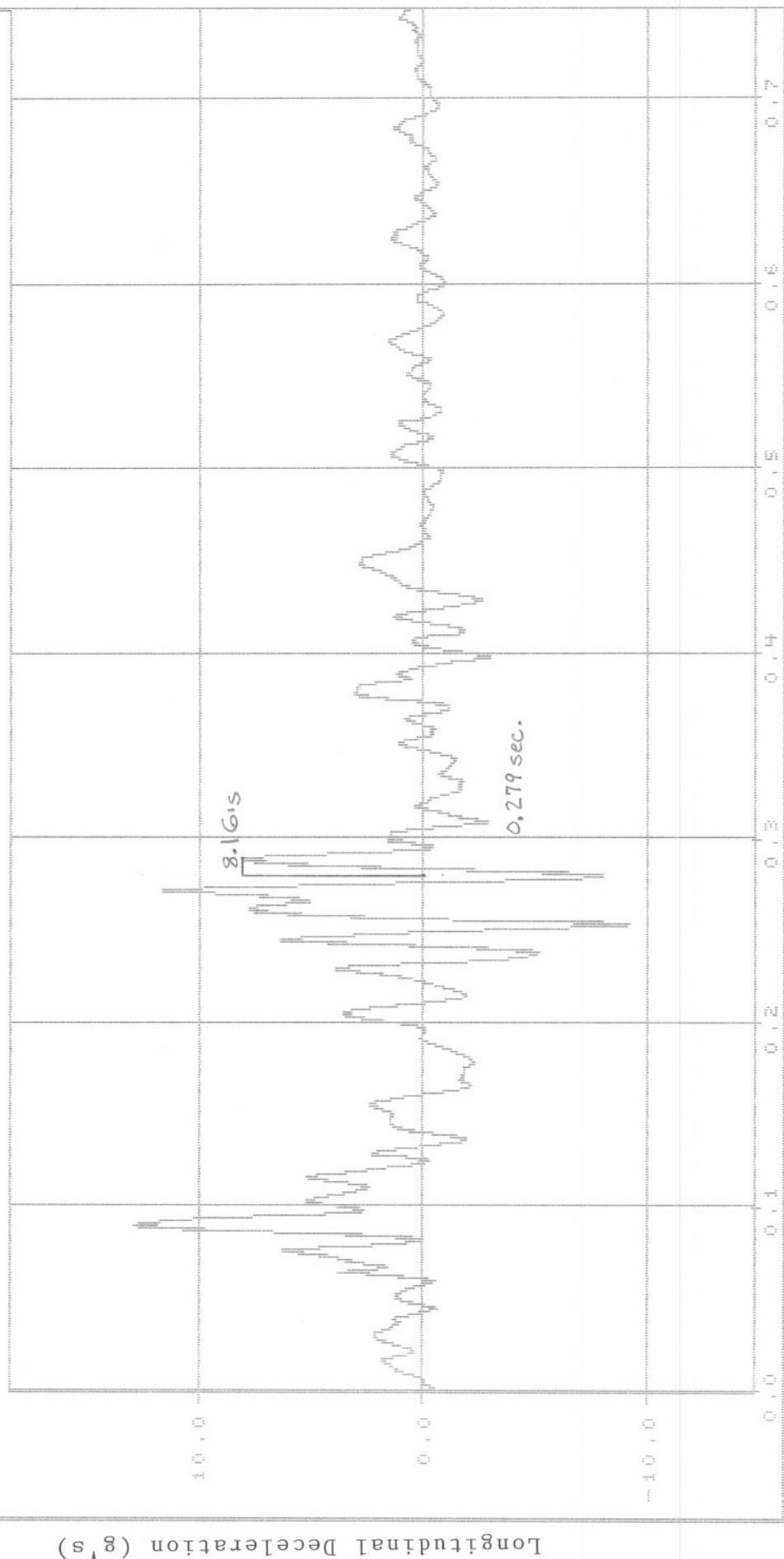


FIGURE E-10. GRAPH OF LONGITUDINAL OCCUPANT DISPLACEMENT, ACC. #3

NER260R4

LONGITUDINAL DECELERATION - TEST NEOCR-2



Time (sec.)

FIGURE E-11. GRAPH OF LONGITUDINAL DECELERATION, ACC. #4

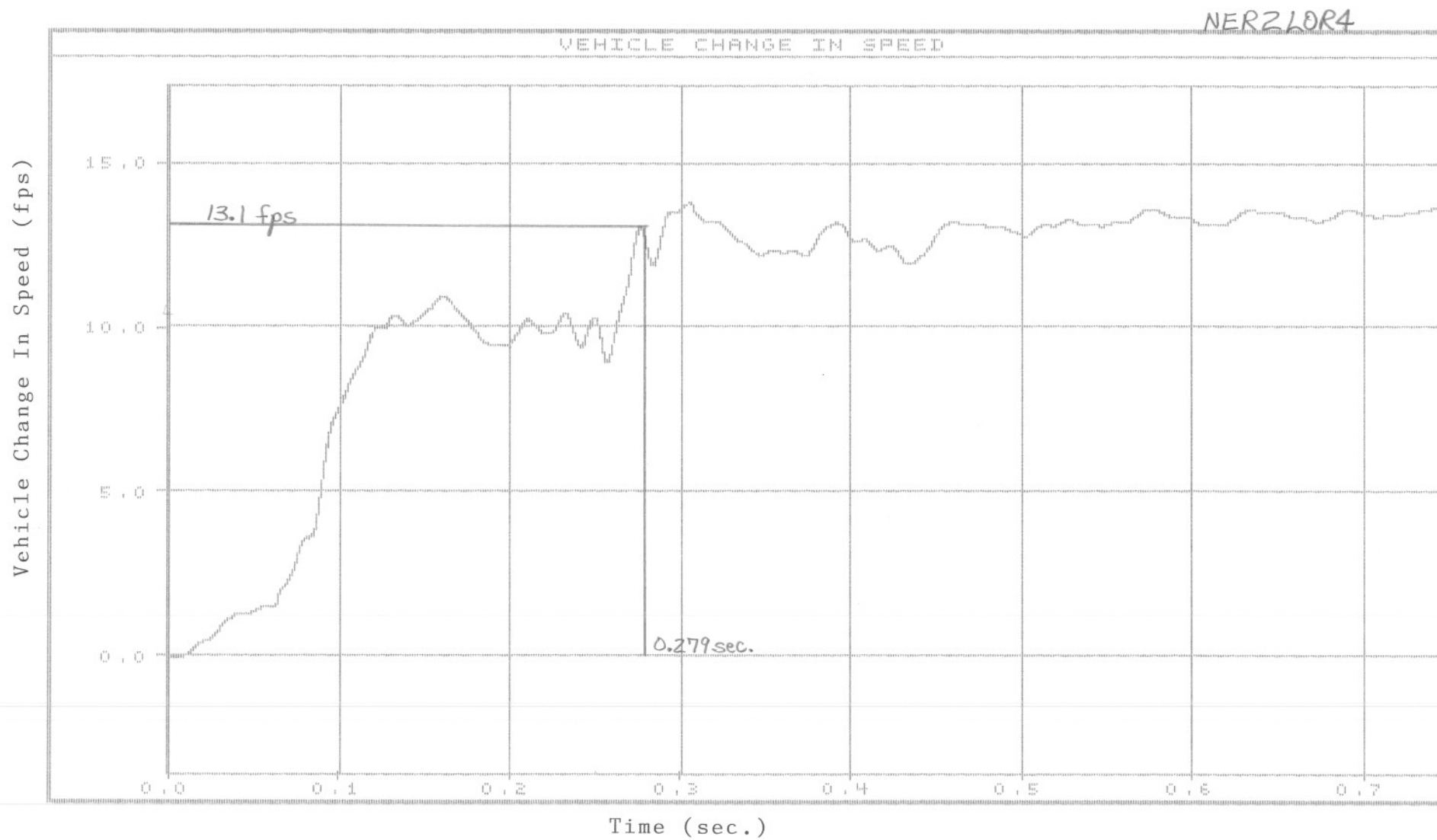


FIGURE E-12. GRAPH OF VEHICLE CHANGE IN SPEED, ACC. #4

NER2LOR4

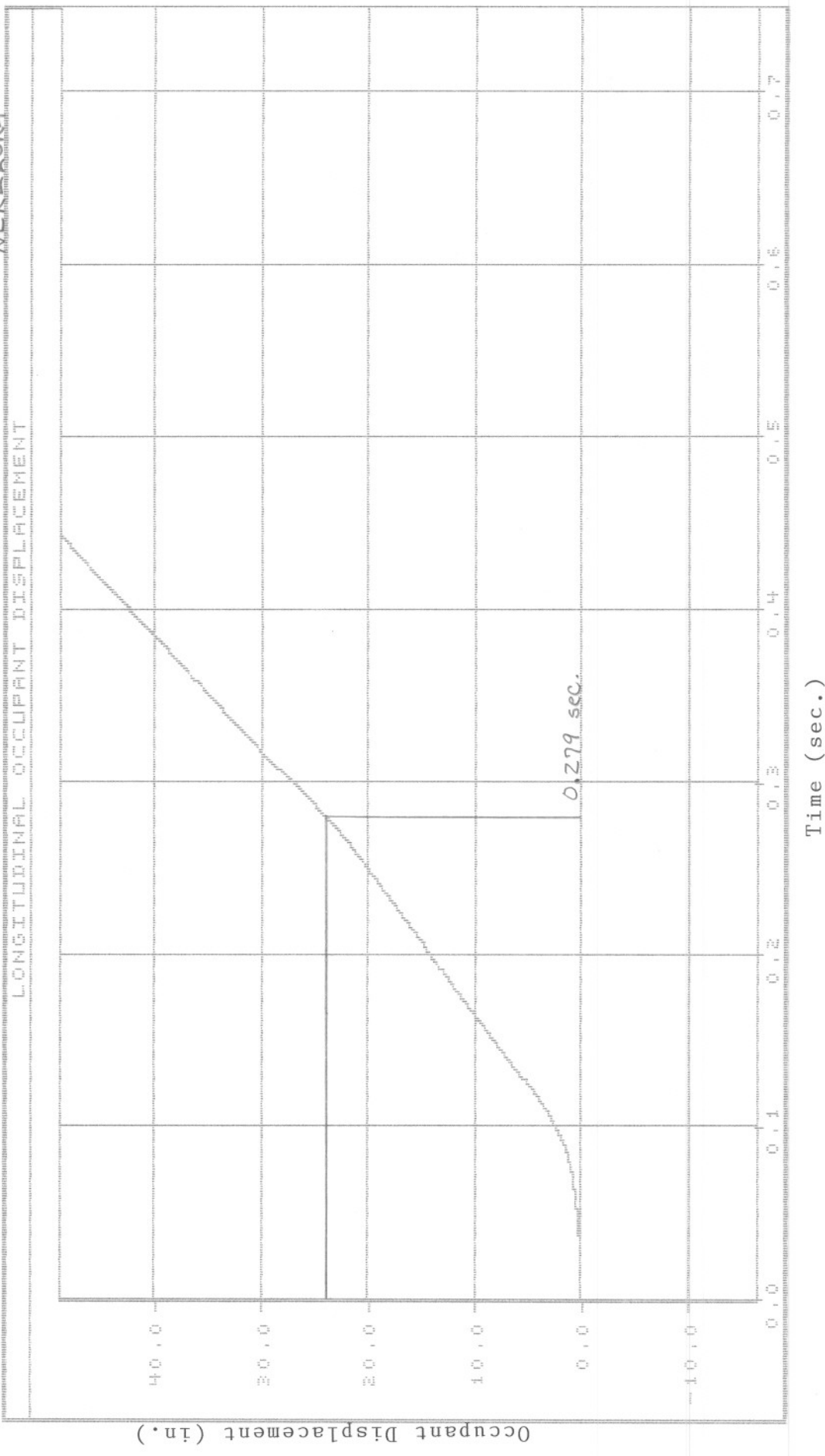


FIGURE E-13. GRAPH OF LONGITUDINAL OCCUPANT DISPLACEMENT, ACC. #4

