DEVELOPMENT OF A TL-1 TIMBER, CURB-TYPE, BRIDGE RAILING FOR USE ON TRANSVERSE, NAIL-LAMINATED, TIMBER BRIDGES

Submitted by

Scott K. Rosenbaugh, M.S.C.E., E.I.T.
Research Associate Engineer

Ronald K. Faller, Ph.D., P.E.
Research Assistant Professor

John D. Reid, Ph.D.
Professor, Mechanical Engineering

Christopher D. Benner
Undergraduate Research Assistant

Robert W. Bielenberg, M.S.M.E., E.I.T.
Research Associate Engineer

Dean L. Sicking, Ph.D., P.E.
MwRSF Director and Professor, Civil Engineering

MIDWEST ROADSIDE SAFETY FACILITY
Nebraska Transportation Center
University of Nebraska-Lincoln
527 Nebraska Hall
Lincoln, Nebraska 68588-0529
(402) 472-0965

Submitted to

Glenn F. Lough, P.E.
Bridge Design Engineer

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
Division of Highways
District 7 Bridge Department
255 Depot Street
Weston, West Virginia 26452-1288

MwRSF Research Report No. TRP-03-211-09-R1

December 7, 2023

December 2023 ADDENDUM / ERRATA

to MwRSF Report No. TRP-03-211-09
"Development of a TL-1 Timber, Curb-Type, Bridge Railing for Use on Transverse, Nail-Laminated, Timber Bridges"
Published May 6, 2009

Page	Revision
53	Part number for washers corrected from "a8" to "b7" on system drawing.

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No.	2.	3. Recipient's Accession No.
TRP-03-211-09		

4. Title and Subtitle	5. Report Date
Development of a TL-1 Timber, Curb-Type, Bridge Railing for Use on Transverse, Nail-Laminated, Timber Bridges	December 7, 2023
	6.
7. Author(s) Rosenbaugh, S.K., Benner, C.D., Faller R.K., Bielenberg, R.W., Reid, J.D., and Sicking, D.L.	8. Performing Organization Report No. TRP-03-211-09
9. Performing Organization Name and Address Midwest Roadside Safety Facility (MwRSF) University of Nebraska-Lincoln 527 Nebraska Hall Lincoln, Nebraska 68588-0529	10. Project/Task/Work Unit No. WV-09-2007-B1 11. Contract © or Grant (G) No.
12. Sponsoring Organization Name and Address West Virginia Department of Transportation Division of Highways - District 7 Bridge Department 255 Depot Street Weston, West Virginia 26452-1288	13. Type of Report and Period Covered Final Report: 2007 – 2009 14. Sponsoring Agency Code SPR-3(017) Supplement #53

15. Supplementary Notes

Prepared in cooperation with U.S. Department of Transportation, Federal Highway Administration.

16. Abstract (Limit: 200 words)

Historically, transverse, nail-laminated, timber bridge decks have been used in the State of West Virginia on rural low-speed, low-volume roads. Although many of these bridges have standard roadway widths of 32 ft (9.8 m) or more, some bridges are configured with widths of only 12 to 14 ft (3.7 to 4.3 m), as measured curb to curb. Since several narrow bridges exist, there is a need for a low-profile railing system to allow for the passage of large trucks and house trailers across these bridges. Several low-profile, curb-type, bridge railing systems have been developed for longitudinal, glue-laminated, timber deck bridges. However, no low-height railings have been developed for transverse, nail-laminated, timber decks.

The research objective was to develop a bridge railing for transverse, nail-laminated, timber decks in order to satisfy the Test Level (TL-1) safety performance criteria found in proposed Update to the National Cooperative Highway Research Program (NCHRP) Report No. 350, now referred to as the *Manual for Assessing Safety Hardware* 2008 (MASH-08). The new, curb-type railing system was adapted from an existing, low-profile, timber railing system that was developed for longitudinal timber decks and evaluated according to TL-1 of NCHRP Report No. 350. Design modifications were made in order to increase railing strength as well as railing height. Several static tests were performed to evaluate the structural capacity of the rail supports utilizing different shear transfer hardware. One full-scale crash test was successfully performed with a 2270P, ½-ton, Quad cab, Dodge pickup truck. The curb-type, timber bridge rail attached to a transverse, nail-laminated, timber bridge deck was deemed acceptable according to the TL-1 evaluation criteria specified in MASH-08.

17. Document Analysis/Descriptors		18. Availability Statement	
Bridge Rail, Timber Rail, Curb, Low-Height, TL-1, Transverse Timber Deck, Nail-Laminated Deck, MASH-08, Roadside Safety, and Crash Test		No restrictions. Document available from: National Technical Information Services, Springfield, Virginia 22161	
19. Security Class (this report) Unclassified	20. Security Class (this page) Unclassified	21. No. of Pages 151	22. Price

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 West Virginia Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

UNCERTAINTY OF MEASUREMENT STATEMENT

The Midwest Roadside Safety Facility (MwRSF) has determined the uncertainty of measurements for several parameters involved in non-standard testing of roadside safety hardware as well as in standard full-scale crash testing of roadside safety features. Information regarding the uncertainty of measurements for critical parameters is available upon request by the sponsor and the Federal Highway Administration.

ACKNOWLEDGEMENTS

The authors wish to acknowledge several sources that made a contribution to this project: (1) the West Virginia Department of Transportation for sponsoring the research project and (2) MwRSF personnel for constructing the barriers and conducting the static tests and full-scale crash test.

Acknowledgement is also given to the following individuals who made a contribution to the completion of this research project.

Midwest Roadside Safety Facility

J.R. Rohde, Ph.D., P.E., Associate Professor
J.C. Holloway, M.S.C.E., E.I.T., Research Manager
K.A. Lechtenberg, M.S.M.E., E.I.T., Research Associate Engineer
C.L. Meyer, B.S.M.E., E.I.T., Research Engineer II
A.T. Russell, B.S.B.A., Laboratory Mechanic II
K.L. Krenk, B.S.M.A, Field Operations Manager
Tom McMaster, B.A., Laboratory Mechanic I
Undergraduate and Graduate Assistants

Dunlap Photography

James Dunlap, President and Owner

TABLE OF CONTENTS

TECHNICAL REPORT DOCUMENTATION PAGE i

DISCLAIMER STATEMENT ii

UNCERTAINTY OF MEASUREMENT STATEMENT ii

ACKNOWLEDGEMENTS iii

TABLE OF CONTENTS iv

LIST OF FIGURES vi

LIST OF TABLES ix

1 INTRODUCTION 1
- 1.1 Background 1
- 1.2 Research Objective 2
- 1.3 Research Approach 2

2 LITERATURE REVIEW 3

3 TIMBER RAIL DESIGN AND MODIFICATIONS 5
- 3.1 Introduction 5
- 3.2 Glulam Timber Rail 5
- 3.3 Railing Height 6
- 3.4 Rail Splice 7
- 3.5 Timber Shear Connectors 7
- 3.6 Transverse Timber Deck 8
- 3.7 End Treatment 9

4 STATIC POST TESTING 11
- 4.1 Static Testing Setup 11
- 4.2 Static Testing Results 19
  - 4.2.1 Test No. WVS-1 – No Shear Connectors 19
  - 4.2.2 Test No. WVS-2 – Split Rings in Each Interface 23
  - 4.2.3 Test No. WVS-3 – Shear Plates in Each Interface 26
  - 4.2.4 Test No. WVS-4 – No Shear Connectors 29
  - 4.2.5 Test No. WVS-5 – Split Rings in Lower Interface Only 32
- 4.3 Static Testing Summary and Conclusions 35

5 DESIGN DETAILS 38
- 5.1 System Overview 38
- 5.2 Timber, Curb-Type Bridge Railing 38
- 5.3 End Treatment 40

5.4 Transverse, Nail-Laminated Timber Deck 41
5.5 Bridge Substructure 43

6 TEST REQUIREMENTS AND EVALUATION CRITERIA 80
6.1 Test Requirements 80
6.2 Evaluation Criteria 81

7 TEST CONDITIONS 83
7.1 Test Facility 83
7.2 Vehicle Tow and Guidance System 83
7.3 Test Vehicles 83
7.4 Data Acquisition Systems 88
7.4.1 Accelerometers 88
7.4.2 Rate Transducers 89
7.4.3 Pressure Tape Switches 89
7.4.4 High-Speed Photography 89

8 FULL-SCALE CRASH TEST NO. WVBR-1 91
8.1 Test No. WVBR-1 91
8.2 Weather Conditions 91
8.3 Test Description 91
8.4 Barrier Damage 93
8.5 Vehicle Damage 94
8.6 Occupant Risk 95
8.7 Discussion 95

9 SUMMARY, CONCLUSIONS, AND RECCOMENDATIONS 110

10 REFERENCES 114

11 APPENDICES 116
APPENDIX A. Vehicle Center of Gravity Determination 117
APPENDIX B. Test Summaries and Sequential Photographs (SI) 119
APPENDIX C. Occupant Compartment Deformation Data 121
APPENDIX D. Accelerometer and Rate Transducer Data Plots 125
APPENDIX E. Material Specifications and Documentation 139

LIST OF FIGURES

Figure 1. TL-1 Bridge Railing for Longitudinal Glue-Laminated Timber Decks [3-4] 4
Figure 2. Static Testing Matrix and Setup 13
Figure 3. Static Test Component Details, Test Nos. WVS-1 and 4 14
Figure 4. Static Test Component Details, Test No. WVS-2 15
Figure 5. Static Test Component Details, Test No. WVS-3 16
Figure 6. Static Test Component Details, Test No. WVS-5 17
Figure 7. Static Test Setup - Test No. WVS-1 18
Figure 8. Modified Static Test Setup - Test Nos. WVS-1 through 5 18
Figure 9. Force vs. Deflection Curve, Test No. WVS-1 20
Figure 10. Component Damage and Permanent Set Deflection, Test No. WVS-1 21
Figure 11. Bridge Deck Damage, Test No. WVS-1 22
Figure 12. Force vs. Deflection Plot, Test No. WVS-2 23
Figure 13. Component Damage and Permanent Set Deflection, Test No. WVS-2 24
Figure 14. Bridge Deck Damage, Test No. WVS-2 25
Figure 15. Force vs. Deflection Plot, Test No. WVS-3 26
Figure 16. Component Damage and Permanent Set Deflection, Test No. WVS-3 27
Figure 17. Bridge Deck Damage, Test No. WVS-3 28
Figure 18. Force vs. Deflection Plot, Test No. WVS-4 29
Figure 19. Component Damage and Permanent Set Deflection, Test No. WVS-4 30
Figure 20. Bridge Deck Damage, Test No. WVS-4 31
Figure 21. Force vs. Deflection Plot, Test No. WVS-5 32
Figure 22. Component Damage and Permanent Set Deflection, Test No. WVS-5 33
Figure 23. Bridge Deck Damage, Test No. WVS-5 34
Figure 24. Force vs. Deflection Comparison for All Five Static Tests 37
Figure 25. Test Installation Layout, Test No. WVBR-1 44
Figure 26. Test Installation and Substructure, Test No. WVBR-1 45
Figure 27. Curb-Type, Timber Bridge Rail Cross Section, Test No. WVBR-1 46
Figure 28. Timber Bridge Rail, Top and Front Views, Test No. WVBR-1 47
Figure 29. Exterior Nail Pattern for Transverse, Nail-Laminated, Timber Bridge Deck 48
Figure 30. Nail Pattern for Transverse, Nail-Laminated, Timber Bridge Deck 49
Figure 31. Rail Connection Details, Test No. WVBR-1 50
Figure 32. Rail End Treatment, Test No. WVBR-1 51
Figure 33. Rail End Treatment Connection Details, Test No. WVBR-1 52
Figure 34. Rail End Section Steel Posts, Test No. WVBR-1 53
Figure 35. Scupper Block Details, Test No. WVBR-1 54
Figure 36. Glulam Rail Segment Details, Test No. WVBR-1 55
Figure 37. Glulam Rail End Segment Details, Test No. WVBR-1 56
Figure 38. Splice Plate Assembly Details, Test No. WVBR-1 57
Figure 39. Splice Plate Component Details, Test No. WVBR-1 58
Figure 40. Dimensional Lumber Details for Timber Deck, Test No. WVBR-1 59
Figure 41. Steel Post Assembly Details, Test No. WVBR-1 60
Figure 42. Steel Post Details, Test No. WVBR-1 61
Figure 43. Deck Anchor Bracket Details, Test No. WVBR-1 62

Figure 44. Connection Hardware, Test No. WVBR-1 63
Figure 45. Bill of Materials, Test No. WVBR-1 64
Figure 46. Bridge Substructure and Deck Anchor Bracket Layout, Test No. WVBR-1 65
Figure 47. Bridge Pit Substructure, Test No. WVBR-1 66
Figure 48. Bridge Girder Diaphragms, Test No. WVBR-1 67
Figure 49. Bridge Abutment Details, Test No. WVBR-1 68
Figure 50. Bridge Bent Details, Test No. WVBR-1 69
Figure 51. Diaphragm Details, Test No. WVBR-1 70
Figure 52. Diaphragm Attachment Brackets, Test No. WVBR-1 71
Figure 53. Bridge Substructure Hardware, Test No. WVBR-1 72
Figure 54. Bridge Substructure Hardware, Test No. WVBR-1 73
Figure 55. Substructure Bill of Materials, Test No. WVBR-1 74
Figure 56. Isometric View of Timber Bridge Rail and End Treatment 75
Figure 57. Curb-Type Bridge Railing System, Test No. WVBR-1 76
Figure 58. Rail, Post, and Bridge Deck Connection Details, Test No. WVBR-1 77
Figure 59. Splice Plate Connections for Glulam Rail, Test No. WVBR-1 78
Figure 60. Sloped End Treatment, Test No. WVBR-1 79
Figure 61. Test Vehicle, Test No. WVBR-1 84
Figure 62. Vehicle Dimensions, Test No. WVBR-1 85
Figure 63. Target Geometry, Test No. WVBR-1 87
Figure 64. Camera Locations, Test No. WVBR-1 90
Figure 65. Summary of Test Results and Sequential Photographs, Test No. WVBR-1 97
Figure 66. Additional Sequential Photographs, Test No. WVBR-1 98
Figure 67. Additional Sequential Photographs, Test No. WVBR-1 99
Figure 68. Additional Sequential Photographs, Test No. WVBR-1 100
Figure 69. Documentary Photographs, Test No. WVBR-1 101
Figure 70. Target Impact Location, Test No. WVBR-1 102
Figure 71. Vehicle Final Position and Trajectory Marks, Test No. WVBR-1 103
Figure 72. System Damage, Test No. WVBR-1 104
Figure 73. Rail Damage and Displacement, Test No. WVBR-1 105
Figure 74. System Damage at Joint Between Post Nos. 4 and 5, Test No. WVBR-1 106
Figure 75. Concrete Gaps at Post Nos. 4 and 5, Test No. WVBR-1 107
Figure 76. Vehicle Damage, Test No. WVBR-1 108
Figure 77. Vehicle Damage at Right-Front Corner, Test No. WVBR-1 109
Figure A-1. Vehicle Mass Distribution, Test No. WVBR-1 118
Figure B-1. Summary of Test Results and Sequential Photographs (SI), Test No. WVBR-1 120
Figure C-1. Occupant Compartment Deformation Data – Set 1, Test No. WVBR-1 122
Figure C-2. Occupant Compartment Deformation Data – Set 2, Test No. WVBR-1 123
Figure C-3. Occupant Compartment Deformation Index (OCDI), Test No. WVBR-1 124
Figure D-1. 10-ms Average Longitudinal Acceleration, Test No. WVBR-1 (EDR-3) 126
Figure D-2. Longitudinal Occupant Impact Velocity, Test No. WVBR-1 (EDR-3) 127
Figure D-3. Longitudinal Occupant Displacement, Test No. WVBR-1 (EDR-3) 128
Figure D-4. 10-ms Average Lateral Acceleration, Test No. WVBR-1 (EDR-3) 129
Figure D-5. Lateral Occupant Impact Velocity, Test No. WVBR-1 (EDR-3) 130
Figure D-6. Lateral Occupant Displacement, Test No. WVBR-1 (EDR-3) 131

Figure D-7. 10-ms Average Longitudinal Acceleration, Test No. WVBR-1 (DTS) 132
Figure D-8. Longitudinal Occupant Impact Velocity, Test No. WVBR-1 (DTS) 133
Figure D-9. Longitudinal Occupant Displacement, Test No. WVBR-1 (DTS) 134
Figure D-10. 10-ms Average Lateral Acceleration, Test No. WVBR-1 (DTS) 135
Figure D-11. Lateral Occupant Impact Velocity, Test No. WVBR-1 (DTS) 136
Figure D-12. Lateral Occupant Displacement, Test No. WVBR-1 (DTS) 137
Figure D-13. Vehicle Roll, Pitch, and Yaw Angular Displacements, Test No. WVBR-1 138
Figure E-1. Glulam Timber Beam Invoice 140
Figure E-2. Timber Rail and Scupper Products Bill of Materials 141
Figure E-3. Timber Products Certificate of Performance 142
Figure E-4. Deck Lumber Invoice 143
Figure E-5. Deck Anchor Bracket Invoice 144
Figure E-6. Deck Anchor Bracket Certification 145
Figure E-7. Steel Connection Hardware Invoice 146
Figure E-8. Steel Connection Hardware Certification 147
Figure E-9. Steel Connection Hardware Certification Continued 148
Figure E-10. Splice Plate Steel Certification 149
Figure E-11. Splice Plate Steel Certification Continued 150

LIST OF TABLES

Table 1. MASH-08 Test Level 1 Crash Test Conditions 80
Table 2. MASH-08 Test Level 1 Evaluation Criteria for Longitudinal Barriers 82
Table 3. Camera and Lens Information, Test No. WVBR-1 90
Table 4. Weather Conditions, Test No. WVBR-1 91
Table 5. Summary of OIV, ORA, THIV, and PHD Values, Test No. WVBR-1 95
Table 6. Summary of MASH-08 Safety Performance Evaluation Results (Test No. 1-11) 113

1 INTRODUCTION

1.1 Background

Historically, the District Offices of the West Virginia Department of Transportation, Division of Highways, have been responsible for the construction, maintenance, and repair of many bridges that utilize transverse, timber, nail-laminated deck systems placed on steel wide-flange girders. Although many of these bridges have standard roadway widths of 32 ft (9.8 m) or more, some bridges are configured with widths of only 12 to 14 ft (3.7 to 4.3 m), as measured curb to curb. Since several narrow bridges exist, there has been a need to use a low-profile railing system in order to allow for the passage of large trucks and house trailers across these bridges.

According to Section 3.2.2 of the West Virginia Bridge Design Manual, all new or replacement bridge barriers shall meet or exceed current crash testing criteria. Unfortunately, no crashworthy, curb-type bridge railing systems have been developed for use on transverse, timber, nail-laminated bridge decks. However, several low-height, curb-type bridge railings have been developed for longitudinal glue-laminated, timber bridge decks [1-4]. One of these railings developed at the Midwest Roadside Safety Facility (MwRSF) was successfully crash tested to meet the Test Level 1 (TL-1) safety performance criteria found in the National Cooperative Highway Research Program (NCHRP) Report No. 350, *Recommended Procedures for the Safety Performance Evaluation of Highway Features* [5]. It was believed that this curb-type bridge railing could be modified to satisfy the TL-1 safety performance criteria established in the proposed Update to NCHRP Report No. 350, now referred to as the *Manual for Assessing Safety Hardware* 2008 (MASH-08) [6].

1.2 Research Objective

For this project, the research objective was to adapt an existing, crashworthy, TL-1 curb-type bridge railing for use on transverse, timber, nail-laminated bridge decks supported by steel wide-flange beams. The railing system was redesigned to meet the TL-1 impact safety standards set forth by MASH-08.

1.3 Research Approach

This project began with an analysis of the prior TL-1, curb-type timber bridge rail. Since the new railing was designed to satisfy the TL-1 safety criteria of MASH-08, design modifications were incorporated in order to improve railing geometry, increase vehicle containment, as well as to increase the structural adequacy of the bridge rail. These changes were made in order to accommodate the larger and heavier vehicles. Then, a static testing program was conducted on five separate scupper block post assemblies. These static tests were used to ensure that the bridge rail posts would provide adequate strength as well as to determine the appropriate use for timber shear connectors in the post-to-deck and post-to-rail connections. Upon completion of the static testing program, the bridge rail design was finalized, and an appropriate safety end treatment was configured. A 120-ft (36.6-m) long section of bridge rail was constructed on top of a transverse, nail-laminated, timber bridge deck equipped with a 35-ft (10.7-m) long, end treatment on the upstream end. Next, a full-scale vehicle crash test was conducted adhering to the impact conditions of test designation no. 1-11 of MASH-08. Finally, the test results were analyzed, and conclusions were made pertaining to the safety performance of the timber bridge rail attached to a transverse, nail-laminated, timber bridge deck.

2 LITERATURE REVIEW

In 1993, researchers at the Midwest Roadside Safety Facility (MwRSF) developed three low-height, timber curb-type railings for longitudinal, glue-laminated (glulam), timber deck bridges according to test conditions below published impact safety standards [1-2]. For this effort, the curb railings were developed for low-volume roads and were crash tested with a 4,400-lb (2,000-kg) pickup truck impacting at approximately 15 mph (24 km/h) and 15 degrees. Square, rectangular, and trapezoidal rail shapes constructed out of sawn lumber were used and had mounting heights of 12, 12, and 14 in. (305, 305, and 356 mm), respectively.

In 1995, MwRSF researchers developed a fourth low-height, timber, curb-type railing system [3-4]. This bridge rail system was also designed for longitudinal, glulam, timber deck bridges. For this design, a 6 ¾-in. by 10 ½-in. (171 x 267 mm) rectangular, glulam beam was supported by scupper blocks using a top rail mounting height of 17 ¾ in. (451 mm). Steel splice plates were used to connect adjacent rail elements end-to-end, while steel split rings and vertical bolts were utilized to transfer the impact loads from the rail, through the wooden support blocks, and into the deck. During the testing program, the bridge railing system safely redirected a 3/4-ton pickup truck impacting at 31.6 mph (50.9 km/h) and 24.3 degrees. This crash testing and evaluation program was conducted according to the TL-1 impact conditions found in NCHRP Report No. 350 [5]. This TL-1 railing system formed the basis for design work completed in this project. Design drawings for this TL-1 curb-type railing system are shown in Figure 1.

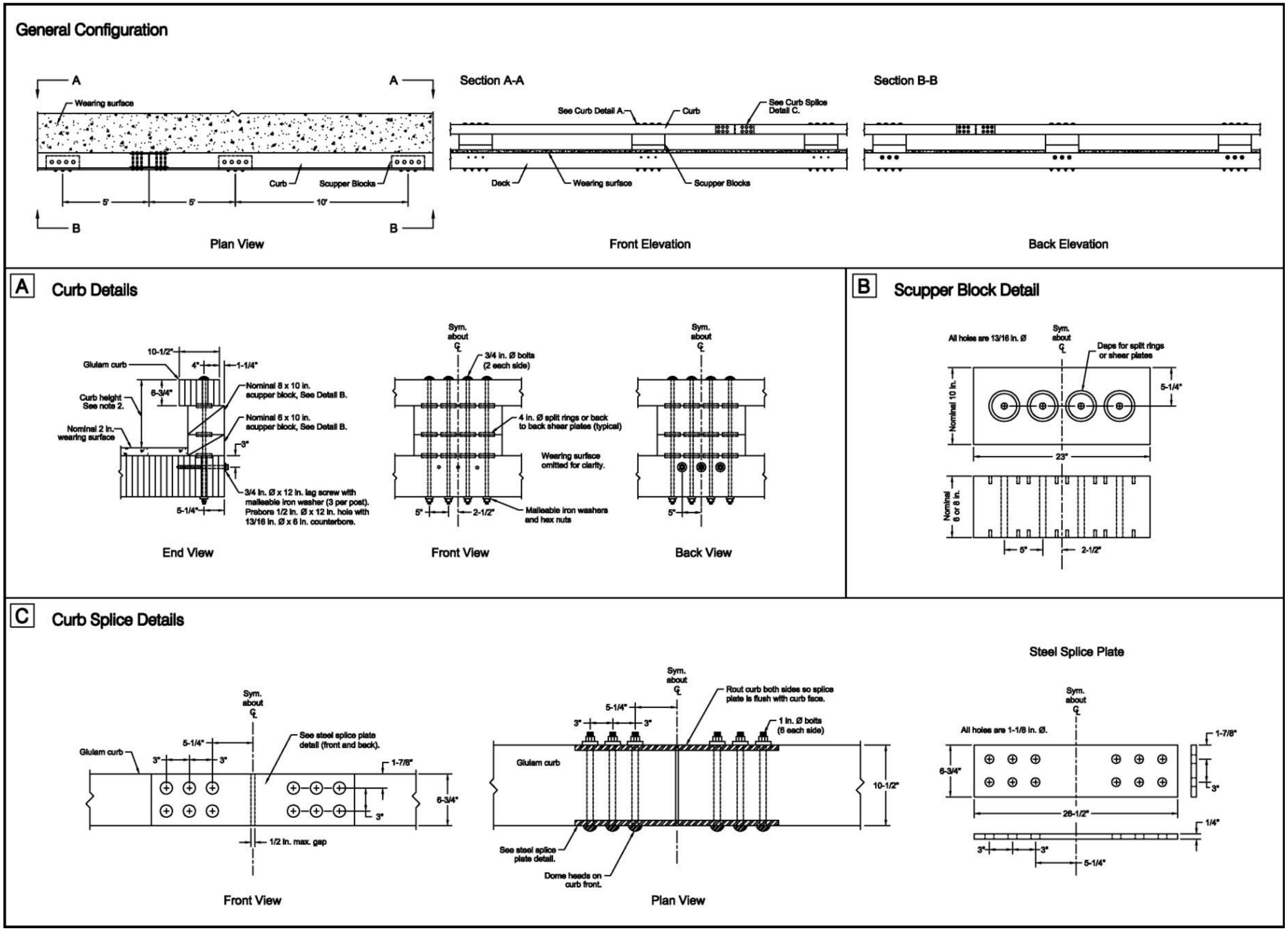


Figure 1. TL-1 Bridge Railing for Longitudinal Glue-Laminated Timber Decks [3-4]

3 TIMBER RAIL DESIGN AND MODIFICATIONS

3.1 Introduction

MwRSF's curb-type, timber bridge rail system, as shown previously in Figure 1, was successfully crash tested to the TL-1 safety performance criteria found in NCHRP Report No. 350. This timber bridge rail system served as the basis for the new timber bridge rail design. However, the railing for this project was required to meet the TL-1 safety performance criteria provided in MASH-08. Therefore, modifications were made to the previous system in order to accommodate the increased impact severity and increased vehicle height, resulting from the 2270P vehicle provided in the MASH-08 guidelines. These changes are described throughout the remainder of this chapter.

3.2 Glulam Timber Rail

For MASH-08, the strength test utilizes a 5,000-lb (2268-kg) pickup truck vehicle, while a 4,409-lb (2,000-kg) pickup truck is used for the strength test in NCHRP Report No. 350. As a result of the increased vehicle mass, the target impact severity was increased by 13.4 percent. Due to the increased impact severity and expected increase in impact forces, it was necessary to increase the size and strength of the 6 ¾-in. by 10 ½-in. (171-mm x 267-mm) Combination No. 2 Douglas Fir rectangular beam. Also, it was desired to select beam sizes that would provide adequate strength for both Southern Yellow Pine and Douglas Fir timber species. Thus, the project engineer could select a timber species based on material cost and availability for a given location. After analyzing multiple beams for both strength and cost, the two selected options were: (1) a 6 ¾-in. by 12-in. (171-mm x 305-mm) Combination No. 2 Douglas Fir (DF) glulam beam and (2) a 6 ¾-in. by 12 ⅜-in. (171-mm x 314-mm) Combination No. 48 Southern Yellow Pine (SYP) glulam beam. Both rail options would utilize 20-ft (6.1-m) segment lengths. The Southern Yellow Pine beam was selected for full-scale crash testing since it was the weaker of the two beams. Thus, either the SYP or DF glulam beams could be used in the bridge railing system if a successful outcome was observed during the crash test into the SYP curb-type railing system. Final design details for the glulam rail segments are provided in Chapter 5.

3.3 Railing Height

The prior curb-type railing system had a top rail height of 17 ¾ in. (451 mm) and allowed the right-front tire to override the rail during the full-scale crash test. The 2000P test vehicle eventually came to a stop with its front axle on top of the rail. Recall, both the height to the center of mass and the overall vehicle mass for the 2270P vehicle were increased over that provided by the 2000P vehicle. Accordingly, it was determined that the railing height needed to be increased in order to prevent the vehicle from overriding the barrier. Therefore, the railing height was increased by 2 in. (51 mm) to obtain a new height of 19 ¾ in. (502 mm), as measured from the top of the wearing surface to the top of the curb rail. This change in height was achieved by increasing the height of the lower scupper block from 5 ½ in. (140 mm) to 7 ½ in. (191 mm). This modification not only provided the desired rail height, but it also allowed for the lower and upper scupper blocks to utilize the same dimensions. Thus, it was only necessary to utilize one size for all of the timber scupper blocks within the bridge railing system.

Similar to the rail, it was desired to allow the use for either Southern Yellow Pine or Douglas Fir timber materials for fabricating the sawn scupper blocks. Grade No. 1 Southern Yellow Pine was selected for use in the static and dynamic testing programs for this project since SYP has a lower strength than that provided by Grade No. 1 Douglas Fir. Upon the successful completion of the full-scale crash test on the railing system using SYP scupper blocks, it would be deemed appropriate to allow the use for either SYP or DF scupper blocks. Final design details for the scupper blocks are provided in Chapter 5.

3.4 Rail Splice

Since the strength of the rail segments was increased, a similar increase in strength was required for the rail splices using two modifications. First, the thickness of the splice plates was increased from $^{3}/_{16}$ in. (5 mm) to ⅜ in. (10 mm). Second, a steel plate was welded orthogonal to, and at the midpoints of, the two outer steel splice plates in order to create an H-shaped connection assembly. These modifications greatly strengthened the rail splice by preventing deformations within the steel plates as well as any relative displacements between rail ends at splice locations during impact loading. Final design details for the splice plate assembly are provided in Chapter 5.

For the original curb-type, bridge railing system, the timber rails were joined together at the mid-span location of the scupper blocks using two outer steel plates and 12 structural bolts. For the new, curb-type, bridge railing system, the timber rails to joined together near the quarter-span location of the scupper blocks in order to reduce the bending loads imparted to the rail splice, decrease the loads transferred to the splice bolts, and maintain the number of splice bolts in the rail-to-rail connection at 12.

3.5 Timber Shear Connectors

For the prior, curb-type, bridge railing, shear connectors were utilized at each wood interface and at every vertical bolt location. Thus, a total of 24 shear plates (or split rings) were used for each post assembly, which quickly increased the cost of the railing system. For this study, MwRSF researchers, in agreement with WVDOT engineers, conducted static component testing in order to determine whether the timber shear connectors were necessary for configuring the rail-to-post and post-to-deck connections. From this testing, several types and quantities of timber shear connectors were evaluated, as described in Chapter 4. Design details for the bridge railing posts (i.e., scupper blocks) were finalized at the conclusion of the static testing program and are provided in Chapter 5.

3.6 Transverse Timber Deck

For this project, a transverse, nail-laminated, timber bridge deck was planned for use consisting of 2-in. x 6-in. (51-mm x 152-mm) dimensional lumber covered by a 2-in. (51-mm) thick, wearing surface. Although fabrication details existed for fastening the lumber boards together at interior locations, no installation procedures were available for nailing the boards together at exterior (or end) locations. Therefore, it was deemed necessary to determine a nailing pattern for use at both exterior and interior deck locations. The nail pattern had to ensure the following: (1) the boards were securely fastened to one another; (2) the nails would not be driven into other nails during the assembly of the deck; (3) the nails were not located where vertical holes would be drilled through the deck for use in attaching the rail segments and scupper blocks; and (4) the region of nail-laminated deck near scupper block locations would provide adequate punching shear resistance and load transfer to adjacent boards, thus reducing the potential for the fracture of individual boards.

To satisfy the criteria noted above, MwRSF researchers, in cooperation with WVDOT engineers, developed a nailing pattern which repeated every forth board. For interior regions, this pattern spaced the nails approximately 18 in. (457 mm) apart, alternating above and below the mid-planes of the boards, as shown in Chapter 5. Special care was also given to the nail pattern near the edge of the deck. The exterior nail pattern formed two tight squares, one at 3 in. (76 mm) and one at 8 ½ in. (216 mm) away from the deck edge, as depicted in Chapter 5. This end pattern ensured that the vertical bolt holes, located 6 ¼ in. (159 mm) away from the deck edge, would not coincide with the horizontal deck nails. It also ensured that at least one nail was located between the vertical bolt holes and the edge of the deck in order to provide adequate shear resistance. Also, during deck assembly, two beads of Liquid Nails adhesive were applied to the sides of each board and over the outer 3 ft (0.9 m) of deck. The adhesive was used to provide improved shear transfer between boards and prevent the end of a single board from pulling out of the deck.

3.7 End Treatment

For the bridge railing system, a safety treatment was needed to prevent blunt-end impacts into the bridge end. Full-scale crash testing was planned for the bridge railing system but not for the sloped-end treatment. As such, the research team planned to design a sloped-end terminal using the timber, curb-type railing and an acceptable geometry comparable to prior-approved, sloped-end terminals.

In 1998, the Texas Transportation Institute (TTI) successfully evaluated a sloped-end treatment for use with a low-profile concrete barrier system according to the TL-2 safety performance criteria found in NCHRP Report No. 350 [7]. The 15-ft (4.6-m) long, sloped-end treatment was constructed with a finished top height of 20 in. (508 mm) and starting top height of 4 in. (102 mm).

As noted in Section 3.3, a top railing height of 19 ¾ in. (502 mm) was selected for the curb-type, timber bridge rail, thus resulting in a barrier height that was only ¼ in. (6 mm) shorter than the TTI barrier. The timber, curb-type, sloped end section was prescribed for the end treatment. A 15-ft (4.57-m) long, glulam rail segment was attached to the upstream end of the curb-type, bridge rail and sloped down to a height of 4 in. (102 mm). Thus, the end of the sloped glulam rail was partially buried underground. The glulam rail segments used within the end treatment were supported by 6-ft (1,829-mm) long, W6x15 (W152x22.3) steel posts bolted to the bottom of the rail. These posts were placed in compacted, coarse, crushed limestone material that met Grading B of AASHTO M147-95 (1990), as found in MASH 08. With this configuration, the slope for the glulam rail was nearly identical to TTI's crashworthy, sloped-end treatment used with the low-profile concrete barrier. Therefore, the sloped, curb-type, timber rail end treatment should provide similar crashworthiness to that provided by the sloped concrete end treatment. Final design details for the sloped, curb-type, timber rail end treatment are shown in Chapter 5.

4 STATIC POST TESTING

4.1 Static Testing Setup

Static testing was conducted on multiple bridge railing posts in order to determine the lateral force versus deflection characteristics for the built-up posts and various connections between the glulam rail, scupper blocks, and timber bridge deck. For the original curb-type bridge railing system, the timber posts were statically tested and found to have an average maximum strength of 14 kips (62 kN) when loaded through the middle of the rail [4]. Thus, the new bridge railing posts should provide similar lateral resistance under static loading in order for the barrier system to be capable of redirecting the 5,000-lb (2,268-kg) pickup truck at the TL-1 impact condition.

The static testing matrix and components are provided in Figures 2 through 6. For this testing, a 23-in. (584-mm) long, timber glulam rail segment was supported by two timber scupper blocks and bolted to the nail-laminated, timber bridge deck using four ¾-in. (19-mm) diameter bolts. A lateral load was applied to each built-up post through a ⅞-in. (22-mm) diameter, steel rod placed through the center of the glulam rail segment. An eye nut was attached to the bolt on the back side of the rail, while a ½-in. (13-mm) thick plate washer was used to distribute the load to the front face of the rail segment.

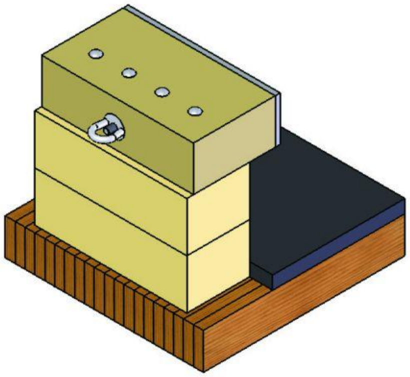
The static tests were also used to guide the selection of the appropriate shear connector configuration for the final bridge railing design. Each of the static tests utilized various types and quantities of shear connectors (i.e. shear plates, split rings, or none) for each bolt location at the timber interfaces. The test matrix shown in Figure 2 lists the number of static tests, the shear connector type, as well as the location for the shear connectors. For test nos. WVS-1 and WVS-4, no shear connectors were utilized, as shown in Figure 3. For test no. WVS-2, and as depicted in

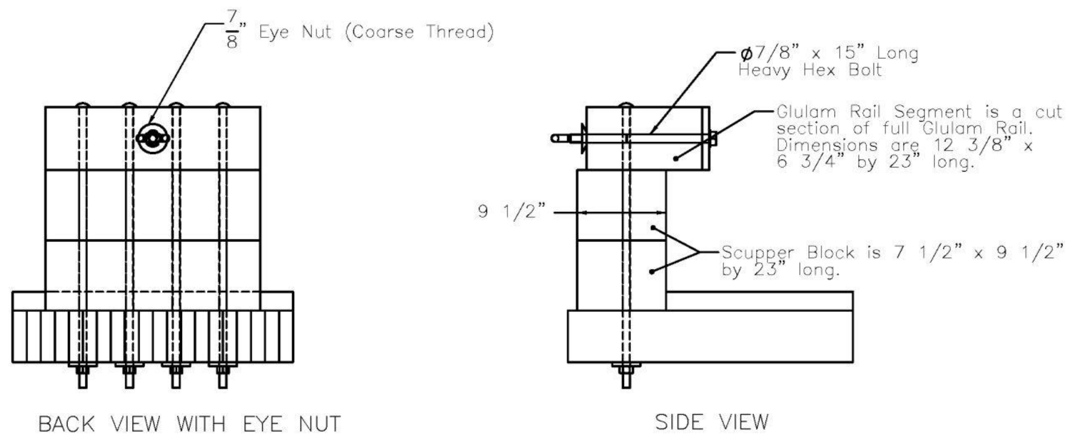
Figure 4, split rings were used at each timber interface: (1) deck to scupper; (2) scupper to scupper; and (3) scupper to rail. Test no. WVS-3 utilized shear plates at every timber interface, as shown in Figure 5. Finally, test no. WVS-5 was configured with split rings located only between the top deck surface and the bottom of the lower scupper block, as shown in Figure 6.

For static test no. WVS-1, the load was applied to the test component with a 9,000-lb (40,000-N) capacity Dayton winch combined with a 2:1 pulley system. The winch was fastened to a 7,000-lb (3,175-kg) pickup truck, and the truck was anchored to a 25,000-lb (11,300-kg) Hyster to ensure that the pickup truck and winch remained stationary during testing. A 50,000-lb (222,000-N) capacity tension load cell and a string potentiometer were used to measure the load and displacement, respectively. The load cell was attached between the 2:1 cable pulley and the eye nut on the rear face of the glulam rail segment. The string potentiometer was anchored to a heavy block located directly in front of the glulam rail segment and tied to the front face of the rail segment. The static test setup is shown in Figure 7.

During test no. WVS-1, the winch reached its maximum load before the test component failed. As such, the method for applying the load required modification. The winch and 2:1 cable pulley system were replaced by a 50,000-lb (222,000-N) capacity hydraulic ram. The ram was attached to a steel anchor frame which was bolted down to the tarmac. The locations for the load cell and string potentiometer remained the same. This modified static test setup, as shown in Figure 8, was utilized to finish test no. WVS-1 as well as for static test nos. 2 through 5.

Test No.	Shear Connection Details
1	Scupper Block with No Connection
2	Scupper Block with Timber Split Ring Grooves
3	Scupper Block with Shear Plate Grooves
4	Scupper Block with No Connection
5	Split Ring Grooves between Bottom Scupper Block and Deck





Midwest Roadside Safety Facility

West Virginia TL–1 Curb–Type Timber Rail Static Test Matrix

General Test Setup

SHEET: 1 of 5

DATE: 3/11/2009

DRAWN BY: EMA/CDB/RJ

DWG. NAME. wvbridge rail_static test matrix R4

SCALE: None

UNITS: Inches

REV. BY: RKF/SKR/KA

Figure 2. Static Testing Matrix and Setup

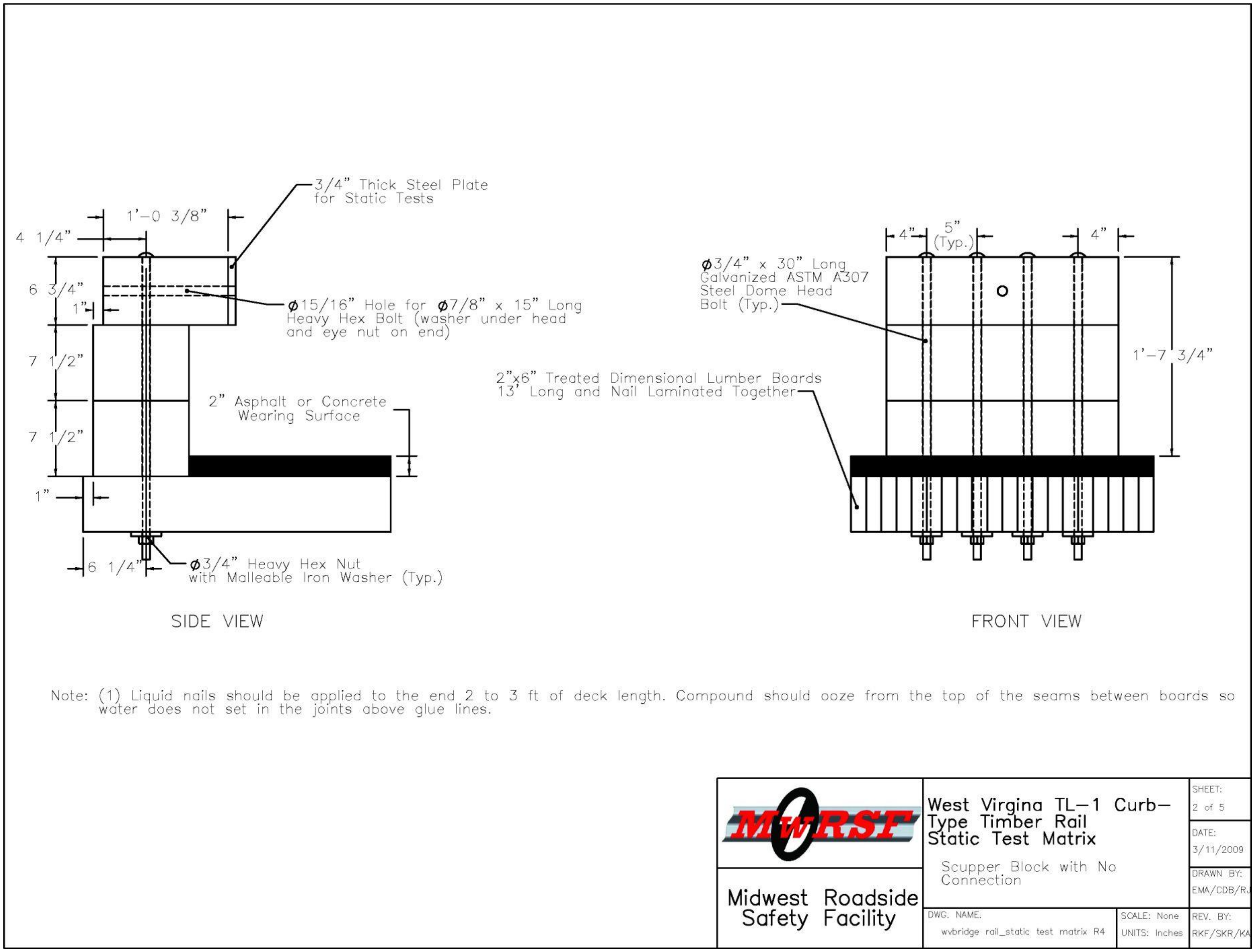


Figure 3. Static Test Component Details, Test Nos. WVS-1 and 4

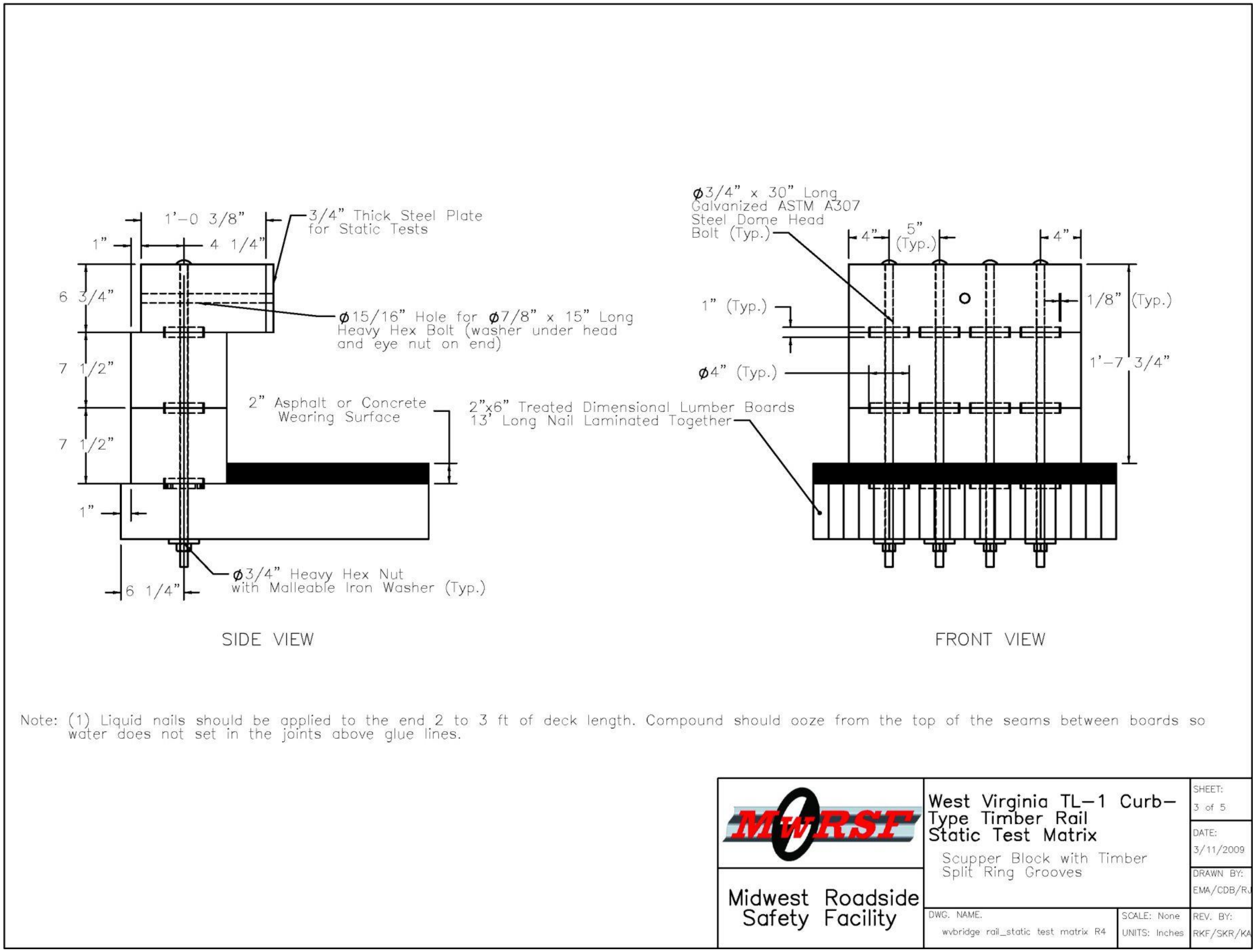


Figure 4. Static Test Component Details, Test No. WVS-2

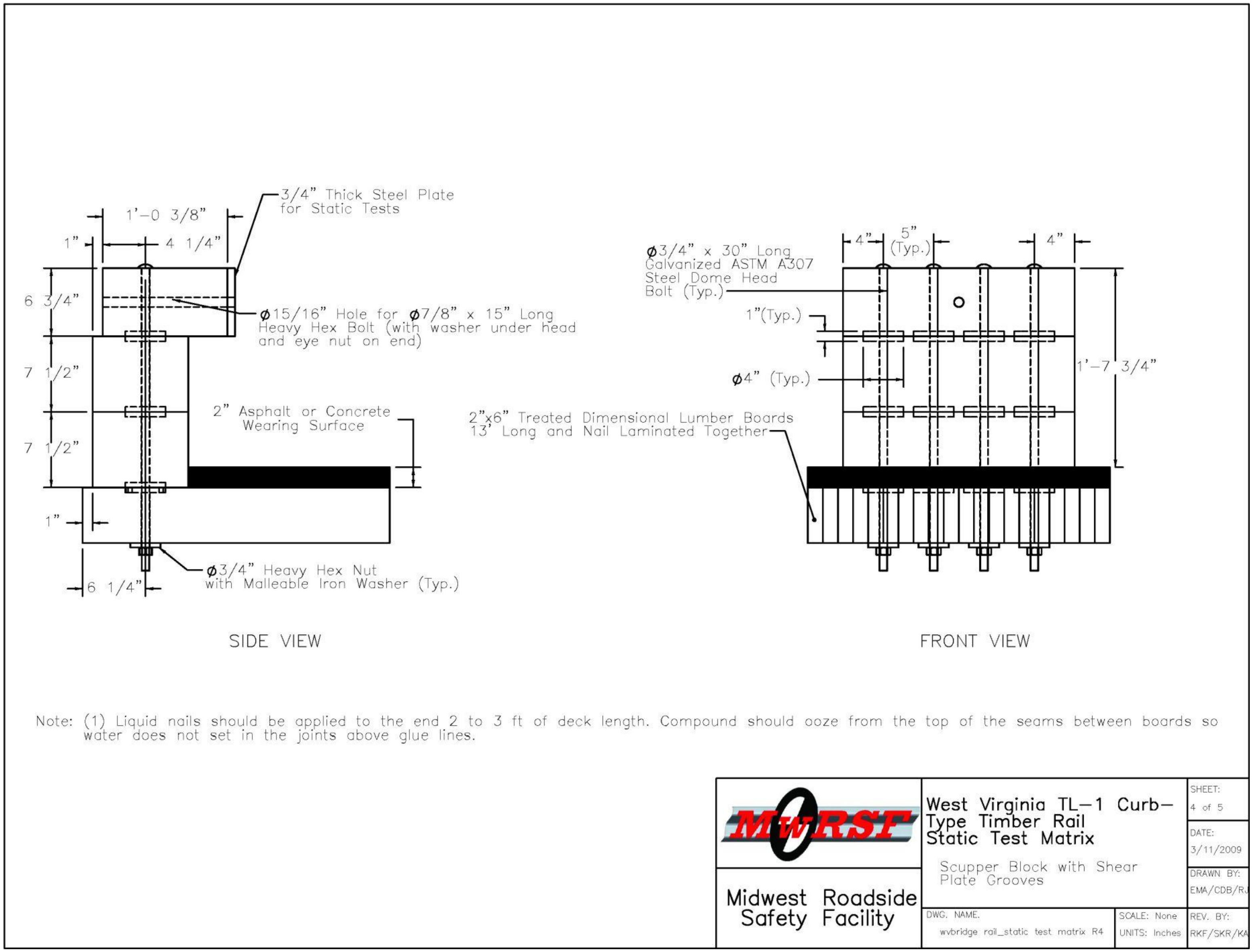


Figure 5. Static Test Component Details, Test No. WVS-3

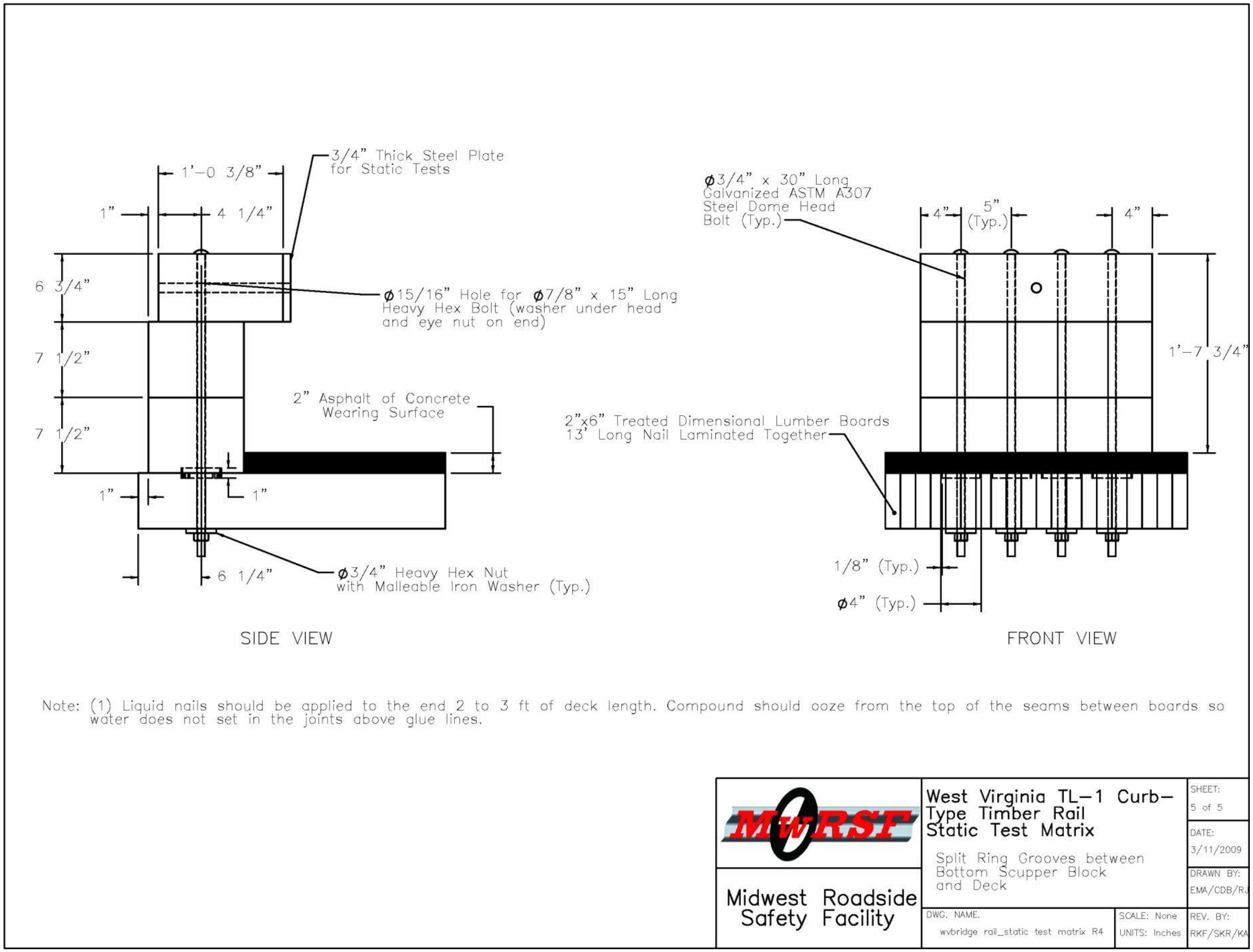


Figure 6. Static Test Component Details, Test No. WVS-5

Figure 7. Static Test Setup - Test No. WVS-1

Figure 8. Modified Static Test Setup - Test Nos. WVS-1 through 5

4.2 Static Testing Results

4.2.1 Test No. WVS-1 – No Shear Connectors

As previously stated, test no. WVS-1 began with the use of the winch and a 2:1 cable pulley system, but it was completed using a hydraulic ram to load the test component to failure. During the test, the post assembly leaned backward, thus opening a gap between the front of the lower scupper block and the bridge deck. At the same time, the edge of the deck deflected downward. A maximum lateral force of 17.9 kips (79.7 kN) was observed at 14.9 in. (378 mm) of deflection. The force versus displacement curve for test no. WVS-1 is shown in Figure 9, while photographs for the deflected post are shown in Figure 10.

Damage to the post assembly and bridge deck was minimal, as shown in Figures 10 and 11. The back edge of the lower scupper block was compressed, thus causing some fibers to splinter off. The top-back edge of the rail segment was partially removed as a result of the loading to the steel rod near the conclusion of the test. All four of the connection bolts were bent near the scupper-deck interface, and all four of the malleable iron washers on the underside of the deck had fractured. It was later revealed that the washers were placed upside-down. The transverse boards in the deck were pulled laterally approximately 1 in. (25 mm) at the post location, and the vertical bolt holes drilled through the deck were elongated due to bearing-type failures in the timber deck. Finally, six of the transverse deck boards had longitudinal cracks near the edge of the deck. However, none of the deck boards had completely fractured.

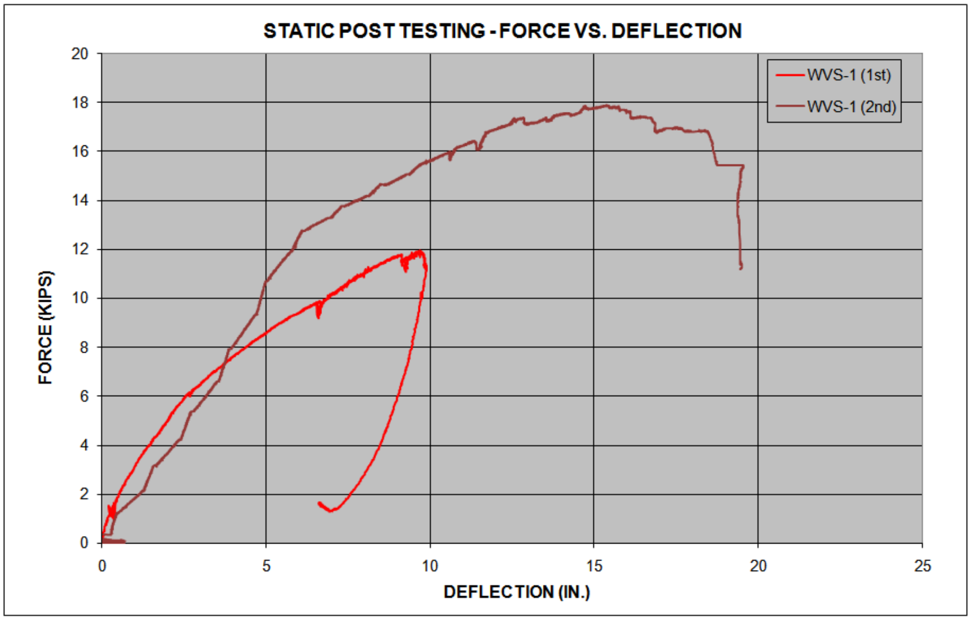


Figure 9. Force vs. Deflection Curve, Test No. WVS-1

Figure 10. Component Damage and Permanent Set Deflection, Test No. WVS-1

Figure 11. Bridge Deck Damage, Test No. WVS-1

4.2.2 Test No. WVS-2 – Split Rings in Each Interface

During test no. WVS-2, the post assembly leaned backward, thus opening a gap between the front of the lower scupper block and the bridge deck. At the same time, the edge of the deck deflected downward. A maximum force of 14.7 kips (65.4 kN) was observed at 15.5 in. (394 mm) of deflection. The force versus displacement curve for test no. WVS-2 is shown in Figure 12, while photographs for the deflected post are shown in Figure 13.

Damage to the post assembly and bridge deck was minimal, as shown in Figures 13 and 14. The back edge of the lower scupper block was compressed, and the top-back edge of the glulam rail segment was partially removed as a result of the loading to the steel rod near the conclusion of the test. All four of the connection bolts were bent near the scupper-deck interface, and three malleable iron washers on the underside of the deck had fractured. The transverse boards in the deck were pulled slightly outward, and three of the deck boards had longitudinal cracks at the deck edge. Two deck boards were fractured near the bolt line. Finally, wood pieces had chipped away from the deck surface between the bolt holes and the split ring grooves.

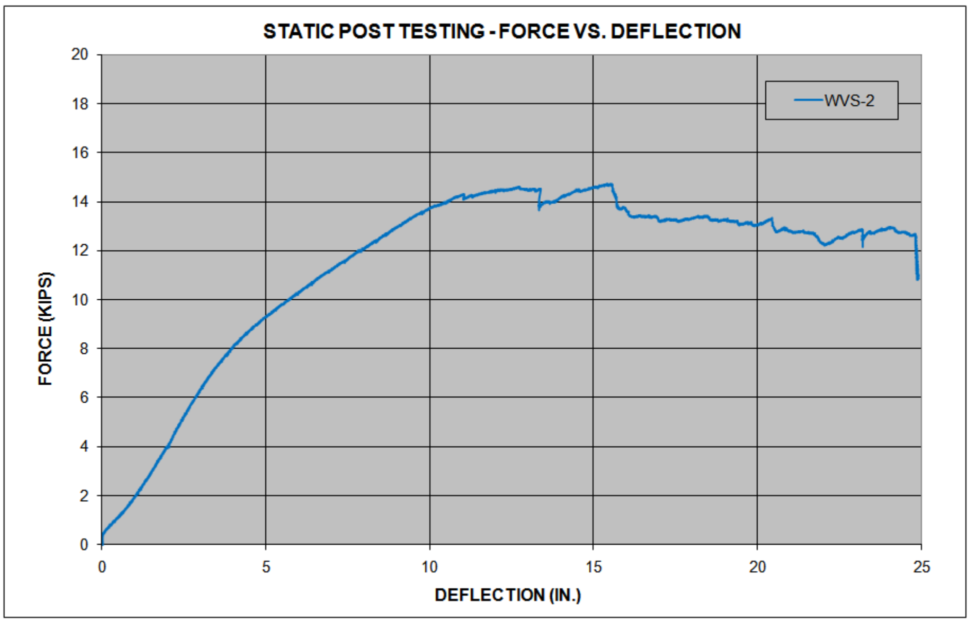


Figure 12. Force vs. Deflection Plot, Test No. WVS-2

Figure 13. Component Damage and Permanent Set Deflection, Test No. WVS-2

Figure 14. Bridge Deck Damage, Test No. WVS-2

4.2.3 Test No. WVS-3 – Shear Plates in Each Interface

During test no. WVS-3, the post assembly leaned backward, thus opening a gap between the front of the lower scupper block and the bridge deck. At the same time, the edge of the deck deflected downward. A maximum force of 17.1 kips (76.1 kN) was observed at 19.3 in. (490 mm) of deflection. The force versus displacement curve for test no. WVS-3 is shown in Figure 15, while photographs for the deflected post are shown in Figure 16.

Damage to the post assembly and bridge deck was minimal, as shown in Figures 16 and 17. The back edge of the lower scupper block was compressed, thus resulting in deformation and cracking of the block. The top-back edge of the rail segment was partially removed as a result of the loading to the steel rod near the conclusion of the test. All four of the connection bolts were bent near the scupper-deck interface, and one of the malleable iron washers on the underside of the deck fractured. Eight of the transverse deck boards had longitudinal cracks at the deck edge, and five of the deck boards appeared to be fractured near the bolt line. Finally, wood pieces had chipped off of the deck surface between the bolt holes and the shear plate grooves.

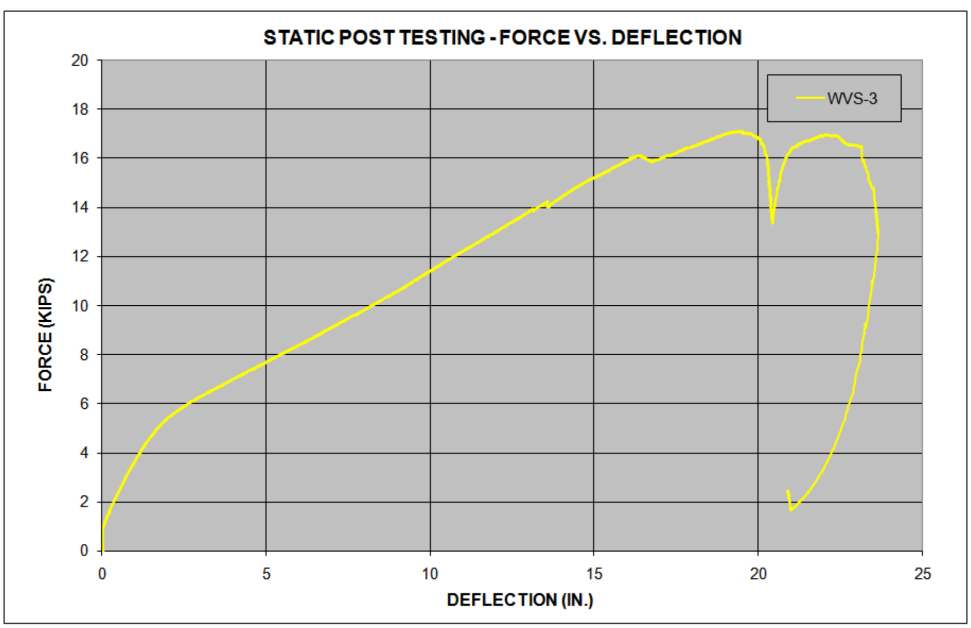


Figure 15. Force vs. Deflection Plot, Test No. WVS-3

Figure 16. Component Damage and Permanent Set Deflection, Test No. WVS-3

Figure 17. Bridge Deck Damage, Test No. WVS-3

4.2.4 Test No. WVS-4 – No Shear Connectors

During test no. WVS-4, the post assembly leaned backward, thus opening a gap between the front of the lower scupper block and the bridge deck. At the same time, the edge of the deck deflected downward. A maximum force of 16.8 kips (74.8 kN) was observed at 22.5 in. (572 mm) of deflection. The force versus displacement curve for test no. WVS-4 is shown in Figure 18, while photographs for the deflected post are shown in Figure 19.

Damage to the post assembly and bridge deck was minimal, as shown in Figures 19 and 20. The back edge of the lower scupper block was compressed, and the top-back edge of the rail segment was partially removed as a result of the loading to the steel rod near the conclusion of the test. All four of the connection bolts were bent near the scupper-deck interface, and two of the malleable iron washers on the underside of the deck fractured. The transverse boards in the deck were pulled slightly outward, and six of the boards had longitudinal cracks at the deck edge. Two of the deck boards appeared to be fractured near the bolt line.

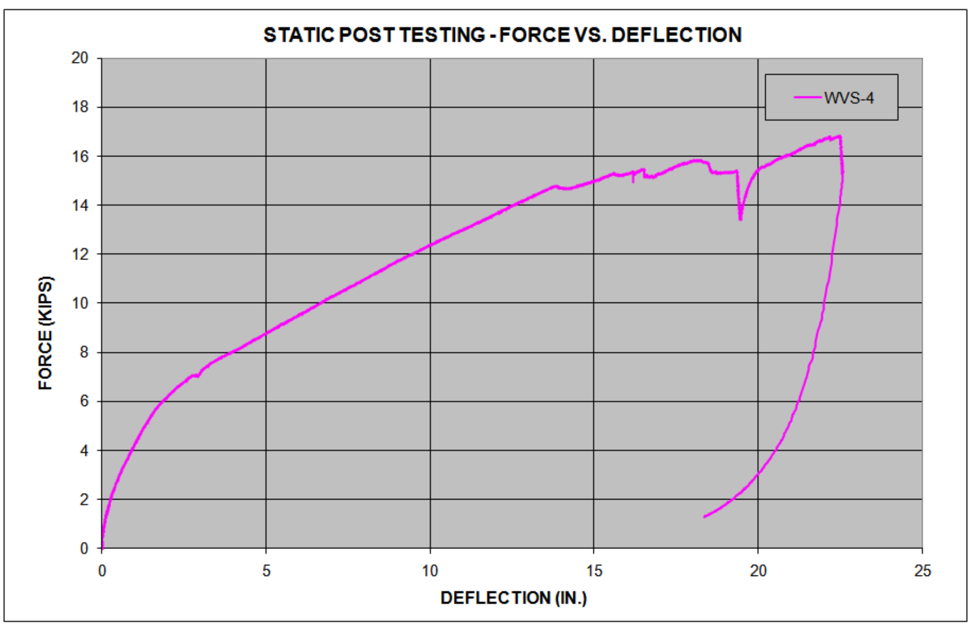


Figure 18. Force vs. Deflection Plot, Test No. WVS-4

Figure 19. Component Damage and Permanent Set Deflection, Test No. WVS-4

Figure 20. Bridge Deck Damage, Test No. WVS-4

4.2.5 Test No. WVS-5 – Split Rings in Lower Interface Only

During test no. WVS-5, the post assembly leaned backward, thus opening a gap between the front of the lower scupper block and the bridge deck. At the same time, the edge of the deck deflected downward. A maximum force of 13.4 kips (59.6 kN) was observed at 17.0 in. (432 mm) of deflection. The force versus displacement curve for test no. WVS-5 is shown in Figure 21, while photographs of the deflected post are shown in Figure 22.

Damage to the post assembly and bridge deck was minimal, as shown in Figures 22 and 23. The back edge of the lower scupper block was compressed, and the top-back edge of the rail segment was partially removed as a result of the loading to the steel rod near the conclusion of the test. All four of the connection bolts were bent near the scupper-deck interface. The transverse boards in the deck were pulled slightly outward at the post location, and four of the deck boards appeared to be fractured near the bolt line. Finally, wood pieces had chipped away from the deck surface between the bolt holes and the timber split ring groves.

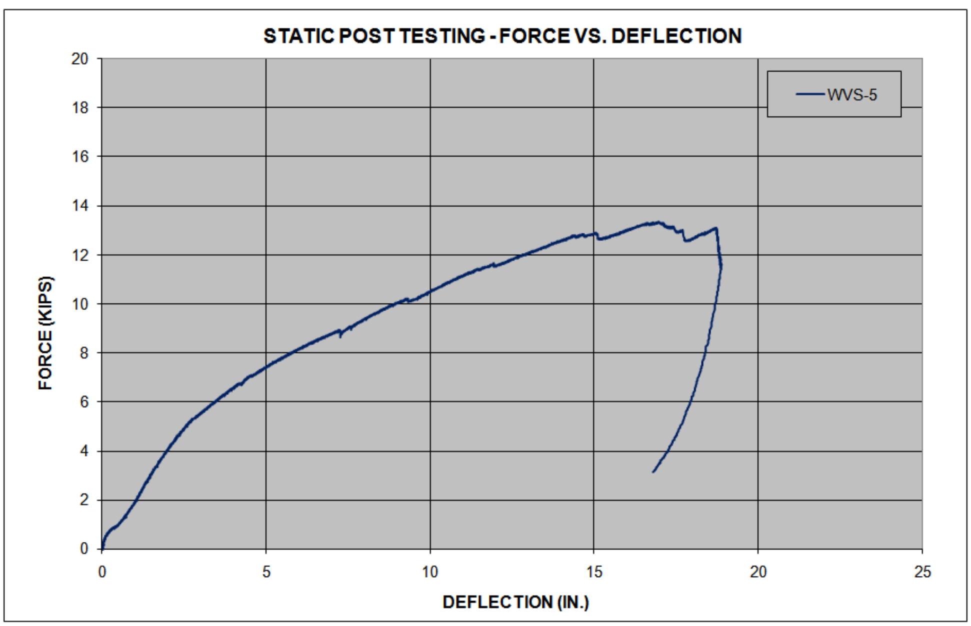


Figure 21. Force vs. Deflection Plot, Test No. WVS-5

Figure 22. Component Damage and Permanent Set Deflection, Test No. WVS-5

Figure 23. Bridge Deck Damage, Test No. WVS-5

4.3 Static Testing Summary and Conclusions

As noted previously, it was desired that the modified bridge posts be capable of resisting a lateral static load of approximately 14 kips (62 kN) or greater. As shown in Figure 24, only one post test resulted in maximum lateral load capacity below the targeted value. In static test no. WVS-5, or the system utilizing split rings between the lower scupper block and the bridge deck, a lateral static capacity of 13.4 kips (59.6 kN) was observed, or only 0.6 kips (2.7 kN) below the targeted value. For the remaining four static tests, the maximum lateral load capacities ranged between 14.7 and 17.9 kips (65.4 to 79.6 kN).

When comparing split rings usage, a 9.7 percent increase in lateral load capacity was observed for split rings used at all interfaces as compared only placed at the lower interface. However, the increase in lateral load capacity ranged between 25.4 to 33.6 percent for the case of not using shear connectors as compared to the use of split rings at the lower interface only. The option for using shear plates at all interfaces provided an increase in lateral load capacity of approximately 27.6 percent as compared to split rings placed at only the lower interface. Finally, it was observed that the no shear connector option [16.8 to 17.9 kips (74.8 to 79.6 kN)] provided a comparable maximum lateral load capacity to that provided by the option for using shear plates at all interfaces [17.1 kips (76.1 kN)].

Further, the initial stiffness observed for each of the static tests was very similar, and the variance in the force versus deflection curves occurred only in the plastic region. Thus, the use of shear connectors had little to no effect on the strength of the post assemblies. In fact, the static testing of post assemblies configured without shear connectors, test nos. WVS-1 and 4, recorded the 1st and 3rd highest lateral capacities.

The use of shear connectors did affect the damage observed during the static tests. Although the post assembly damage in each test was observed to be very similar, the deck damage in each test was substantially different. When shear connectors were placed between the deck and the lower scupper block, grooves had to be cut into the deck surface to accommodate the connectors. These grooves weakened the strength of the deck boards and led to chipping away of pieces between the grooves and the bolt holes as well as the partial fracture of multiple boards through this weakened cross section. As a result, it was determined more deck damage was observed in static tests configured with shear connectors as compared to static tests not involving shear connectors.

From the static testing program, the research team determined that shear connectors: (1) provided no or only limited increased lateral strength for the post assemblies; (2) correlated with an increased amount of deck damage; and (3) increased the labor and materials costs for the railing system. Therefore, it was concluded that no shear connectors would be specified for the timber bridge railing system.

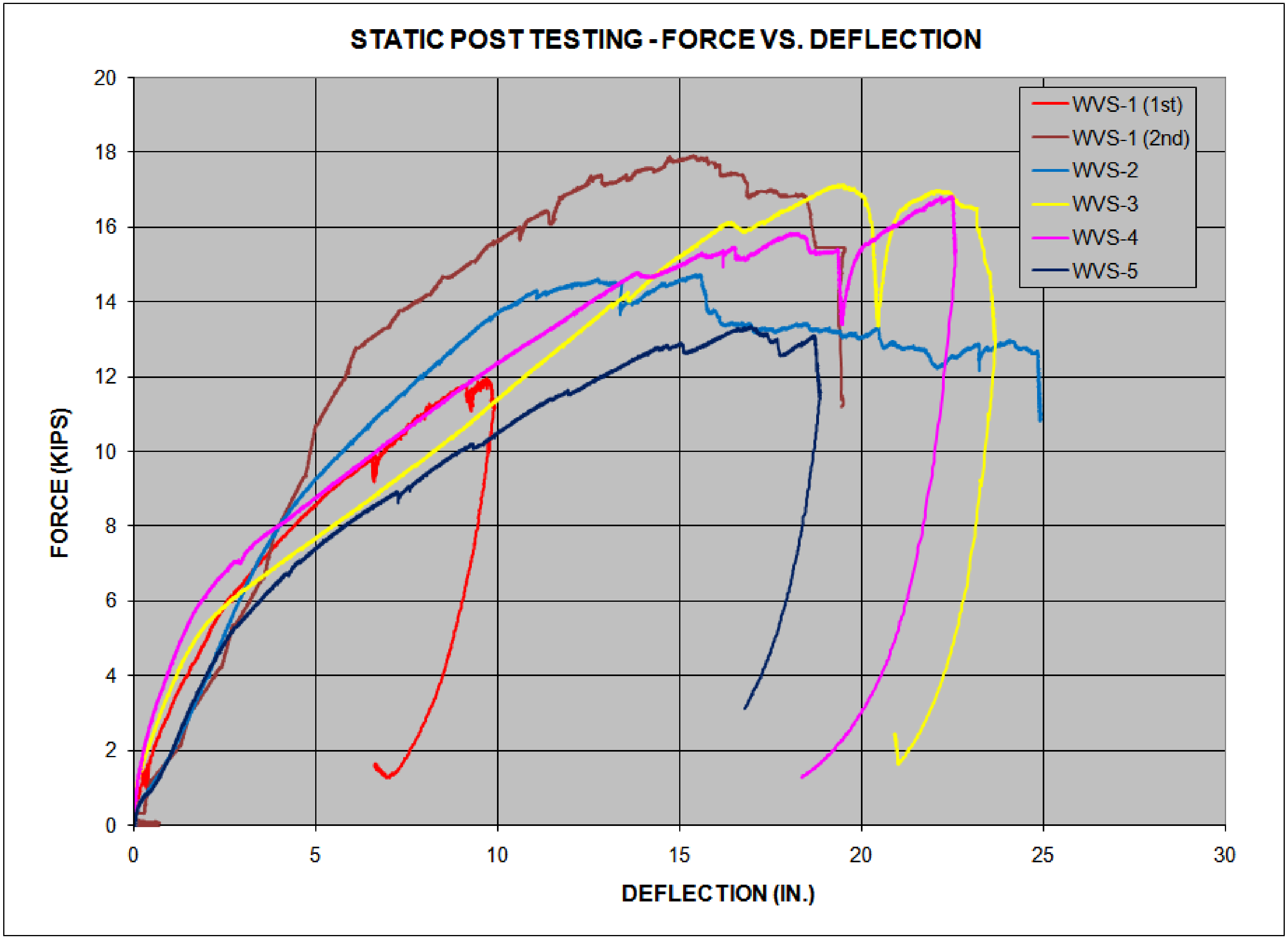


Figure 24. Force vs. Deflection Comparison for All Five Static Tests

5 DESIGN DETAILS

5.1 System Overview

The test installation consisted of three major sub-systems: (1) a 120-ft (36.6-m) long, nail-laminated, timber bridge deck placed on wide-flange, steel girders; (2) a curb-type, timber bridge railing system; and (3) a 35-ft (10.7-m) long, sloped, safety treatment located on the upstream end of the bridge railing. The total length of the test installation was 155 ft (47.2 m). Final design details are provided in Figures 25 through 56. Photographs of the test installation are also shown in Figures 57 through 60.

5.2 Timber, Curb-Type Bridge Railing

The bridge railing system consisted of three major structural components: (1) a longitudinal, glulam timber rail; (2) steel H-splice plates; and (3) post assemblies consisting of sawn lumber scupper blocks. The assembled bridge railing is shown in Figures 25 through 27 and Figure 57.

The timber rail consisted of 19.9-ft (6.08-m) long, glulam rail segments with a 6 ¾ in. x 12 ⅜ in. (171 mm x 314 mm) cross section, as shown in Figure 36. The glulam rails were manufactured from Combination No. 48 Southern Yellow Pine and were treated with pentachlorophenol in heavy oil to a minimum net retention of 0.60 lbs/ft^3 (9.61 kg/m^3) conforming to the American Wood Preserver's Association (AWPA) use category UC4A [8]. The ends of each rail segment were narrowed to a width of 11 ⅝ in. (295 mm) in order to accept the steel H-splice plates and allow the outer plate surface to be flush with the gross rail section.

The steel H-splice plates were to be fabricated from three ASTM A572 Grade 42 steel plates that were welded to one another, as shown in Figures 38 and 39. Due to an oversight, the steel plates were not ordered with much advance notice prior to the scheduling of the full-scale crash test. As such, MwRSF experienced difficulty in finding the original plate material on such short notice, thus requiring an adjustment to the design. Instead, the steel plates were fabricated from ASTM A656 Grade 50 Type 7 material. The two side plates were 34 ¾-in. long by 6 ¾-in. wide by ⅜-in. thick (883 mm x 171 mm x 9.5 mm) with twelve 1 ⅛-in. (29-mm) diameter holes. For the H-splice, the center connecting plate was installed orthogonal to the outer two plates and measured 11 ⅝-in. long by 6 ¾-in. wide by ⅜-in. thick (295 mm x 171 mm x 9.5 mm). The H-splice plates connected adjacent glulam rail segments end to end using six 1-in. (25.4-mm) diameter by 14-in. (356-mm) long, ASTM A307 galvanized dome-head bolts in each rail end, as shown in Figure 31. A photograph of an assembled rail-to-rail connection with H-splice plate is shown in Figure 59. For the testing program, the H-splice plate assemblies were not galvanized. However, these splice plate assemblies must be galvanized when used in actual field installations.

The bridge rail post assemblies consisted of two timber scupper blocks stacked on top of each other. Each scupper block was fabricated from Grade No. 1 Southern Yellow Pine sawn lumber and measured 23 in. long by 9 ½ in. wide, and 7 ½ in. tall (584 mm x 241 mm x 191 mm). Four $^{13}/_{16}$-in. (21-mm) diameter, bolt holes were drilled in the scuppers at 5-in. (127-mm) spacing intervals, as shown in Figure 35. All wooden scupper blocks were treated with pentachlorophenol in heavy oil to a minimum net retention of 0.60 lbs/ft^3 (9.61 kg/m^3) satisfying AWPA U1, UC4A.

The scupper block post assemblies were placed 1 in. (25 mm) from the outer edge of the bridge deck and spaced 10 ft (3.05 m) on centers, as shown in Figure 27. The glulam rail segments were placed on the scuppers such that the back of the rail was offset 1 in. (25 mm) from the back edge of the supper blocks, and the centerline of the joint between two adjacent rail ends was located 35 ⅞ in. (911 mm) from the centerline of the nearest post. The height to the top of the bridge railing was 19 ¾ in. (502 mm) above the concrete wearing surface placed on the timber deck. Four 30-in.

(762-mm) long by ¾-in. (19.0-mm) diameter, ASTM A307 galvanized dome-head bolts were used to connect the glulam rail and scupper blocks to the bridge deck surface, as shown in Figures 27 and 31. Galvanized, malleable iron washers were installed on the bottom side of the deck surface, as shown in Figure 44. Photographs of assembled posts are shown in Figure 58.

5.3 End Treatment

A rigid, end treatment was attached to the bridge rail system and consisted of two glulam rail segments, four steel posts, and an angled H-splice plate. The end treatment had a length of 35 ft (10.7 m) and was attached to the upstream end of the curb-type, bridge railing system, as shown in Figure 32. Photographs of the end treatment are shown in Figure 60.

The first glulam rail segment in the end treatment was exactly the same as the bridge rail segments described previously and was placed adjacent to the first bridge rail segment. The second glulam rail segment in the end treatment was similar to the first glulam rail and bridge rail segments, except for two changes. First, the downstream end of the second glulam rail segment was cut to an 85-degree angle so that the interior ends of the end treatment rails would be flush with one another. In addition, the second glulam rail segment in the end treatment was cut to a total length of 15 ft (4.6 m), as shown in Figure 37. All other dimensions, material properties, and preservative treatment were identical to that used for the glulam rail found in the bridge railing system. The sloped, end rail segment connected to the standard rail segment at a height of 19 ¾ in. (502 mm) and was sloped downward with the upstream end partially buried in the ground. The maximum height for the exposed upstream end of the sloped rail segment was 4 in. (102 mm) above grade.

The two end treatment rail segments were supported by ASTM A36, steel W6x15 (W152x22.3) posts with welded top mounting plates, as shown in Figures 41 and 42. For the testing program, the post assemblies were not galvanized. However, these post assemblies must be galvanized when used in actual field installations. The four steel posts measured 6 ft (1,829 mm) long and were placed in compacted, coarse, crushed limestone material that met Grading B of AASHTO M147-95 (1990), as found in MASH 08. Two of the posts were cut at 85-degree angles at the top in order to match the slope of the end rail segment. The ASTM A36 steel top mounting plates were the same for all four posts and measured 19 in. long by 8 ½ in. wide by ⅜ in. thick (483 mm x 216 mm x 10 mm). Once again, four $^{13}/_{16}$-in. (21-mm) diameter, bolt holes were placed in the plate at 5 in. (127 mm) intervals. The top of the posts were welded to the center of the plates, as shown in Figure 41. The rail segments were connected to the posts with four ¾-in. (19-mm) diameter by 10-in. (254-mm) long, ASTM A307 galvanized dome-head bolts, as shown in Figure 34.

One steel H-splice plate was used to connect the two rail segments within the end treatment region and was modified for use in accepting the sloped end rail segment. The materials and dimensions for this modified H-splice plate was similar to those used with the bridge railing system, except for the addition of a 5 degree bend between the upstream and downstream end of the side plates. This bend matched the slope of the end rail segment, and the appropriate dimensions are shown in Figures 38 and 39. The adjacent rail ends were attached to the angled splice plates using six 1-in. (25-mm) diameter by 14-in. (356-mm) long, ASTM A307 galvanized dome-head bolts in each rail end. A photograph of the installed angled H-splice plate is shown in Figure 59.

5.4 Transverse, Nail-Laminated Timber Deck

For the test bridge, the timber deck measured approximately 14 ft (4.3 m) wide and 120 ft (36.6 m) long. The bridge deck was fabricated with 2-in. x 6-in. (51-mm x 152-mm) by 14-ft (4.3-

m) long, dimensional lumber boards covered with a 2-in. (51-mm) thick, concrete wearing surface. The timber boards were manufactured from Grade No. 1 Southern Yellow Pine and treated with ACQ-D to a minimum net retention of 0.40 lbs/ft^3 (6.41 kg/m^3) satisfying AWPA U1, UC4A. For actual bridge installations, it is recommended that the dimensional lumber boards be treated to a net retention of 0.60 lbs/ft^3 (9.61 kg/m^3) satisfying AWPA U1, UC4B, as shown in Figure 40. The boards were placed on end and nailed together through and perpendicular to the wide face of the board using 20d or 20 penny "common" nails. A specific nail pattern, which repeated every four boards, was used to ensure that a nail did not contact a previously driven nail. Special care was given to the nail pattern near the deck edge to ensure the nails did not occupy space where the vertical bolt holes for the bridge rail would later be drilled. During deck assembly, two beads of Liquid Nails Heavy Duty Construction Adhesive (Item No. LN-901) were applied to the sides of the boards and over the outer 3 ft (0.9 m) of deck. The adhesive was used to provide additional punching shear resistance in the deck as well as improved load transfer between boards. The nailing patterns are shown in Figures 29 and 30.

Steel deck anchor brackets were sandwiched between adjacent deck boards and were used to attach the bridge deck to the steel girders. The deck anchor brackets were fabricated from 11-gauge (3.04-mm thick), ASTM A36 G90 galvanized steel sheet and were cut to the dimensions shown in Figure 43. The anchor brackets hooked onto the top flange of the steel bridge girders and were nailed to the adjacent deck boards using two 20d or 20 penny common nails. The anchor brackets were installed on 1-ft (305-mm) centers on both girders. The brackets on the exterior girder were all placed on the top-inside flange, while the brackets on the interior girder alternated sides, as shown in Figure 46.

5.5 Bridge Substructure

The support structure for the bridge deck consisted of two rows of wide-flange, steel girders, four transverse concrete supports (two bents and two abutments), and lateral bracing between girders, as shown in Figures 46 through 55. The two rows of three girders were positioned along the entire length of the 120-ft (36.58-m) long, bridge deck. The girders were supported by simulated bridge abutments at each end and two simulated bridge piers spaced approximately 40 ft (12.2 m) apart. In addition to these four rigid supports, three intermediate concrete platform supports with wood shim blocks were used to vertically support the steel girders at the midpoint of each 40 ft (12.2 m) span. Finally, steel C-channel diaphragms were used as lateral bracing for the girders and spaced at approximately 12.5-ft (3.8-m) intervals.

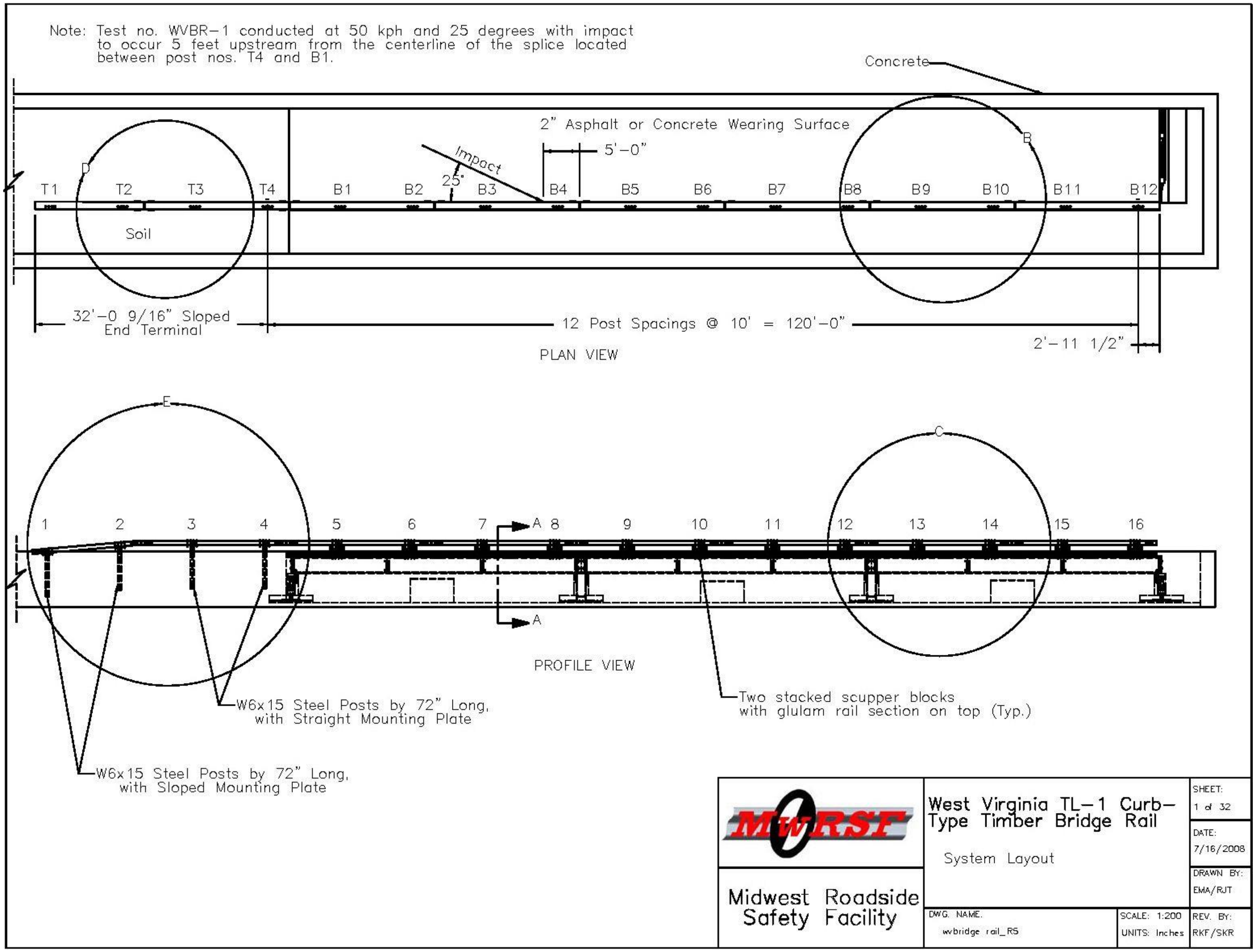


Figure 25. Test Installation Layout, Test No. WVBR-1

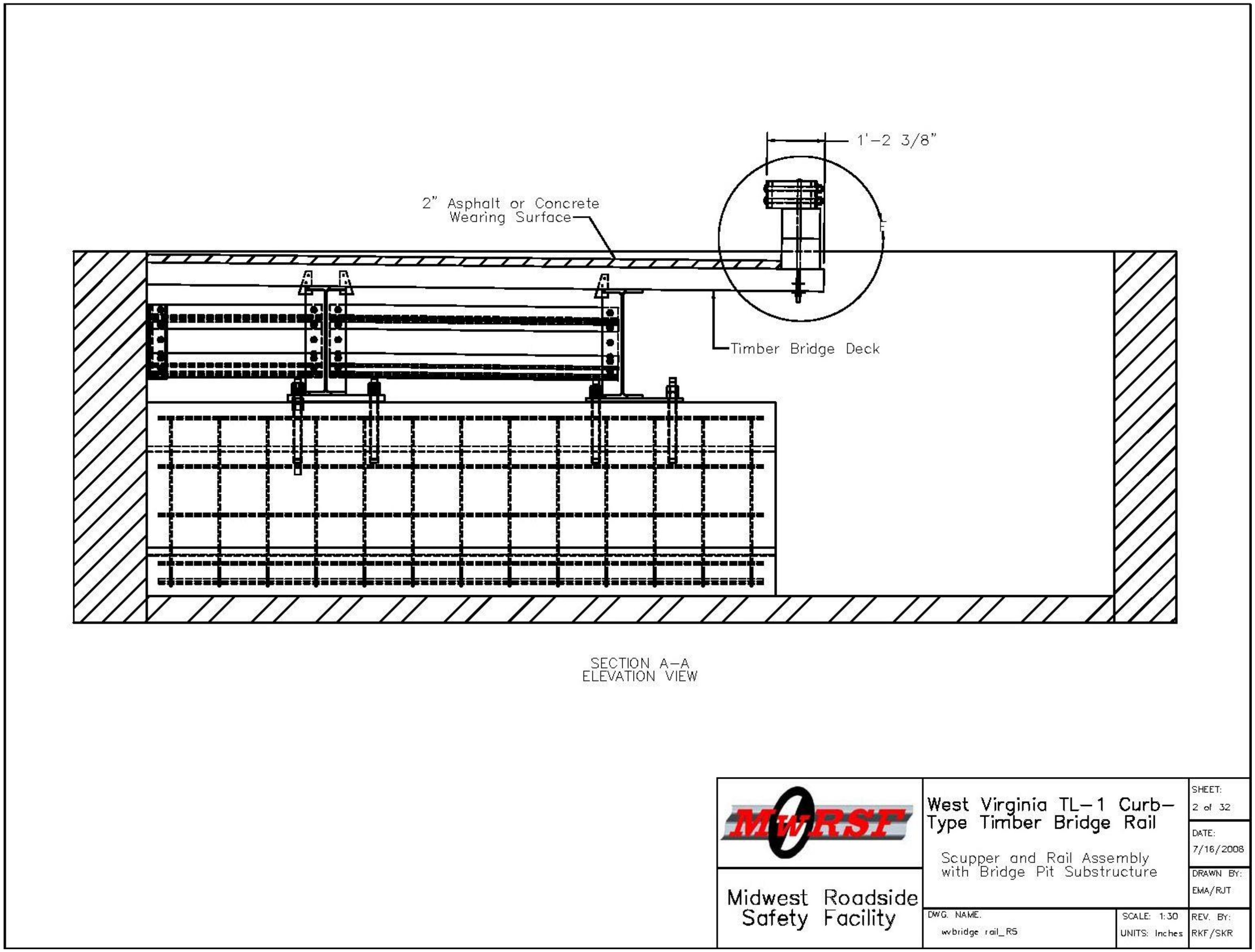


Figure 26. Test Installation and Substructure, Test No. WVBR-1

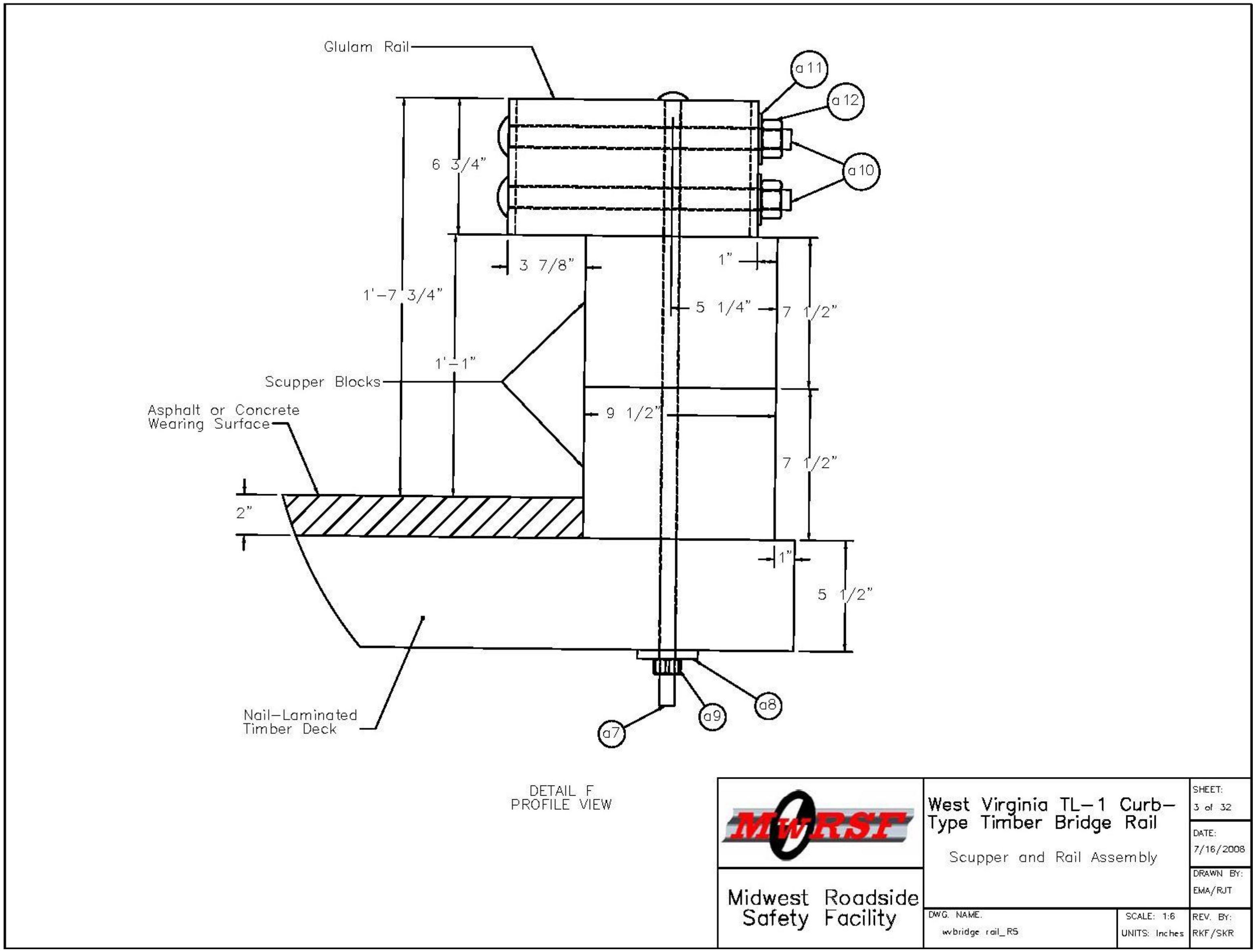


Figure 27. Curb-Type, Timber Bridge Rail Cross Section, Test No. WVBR-1

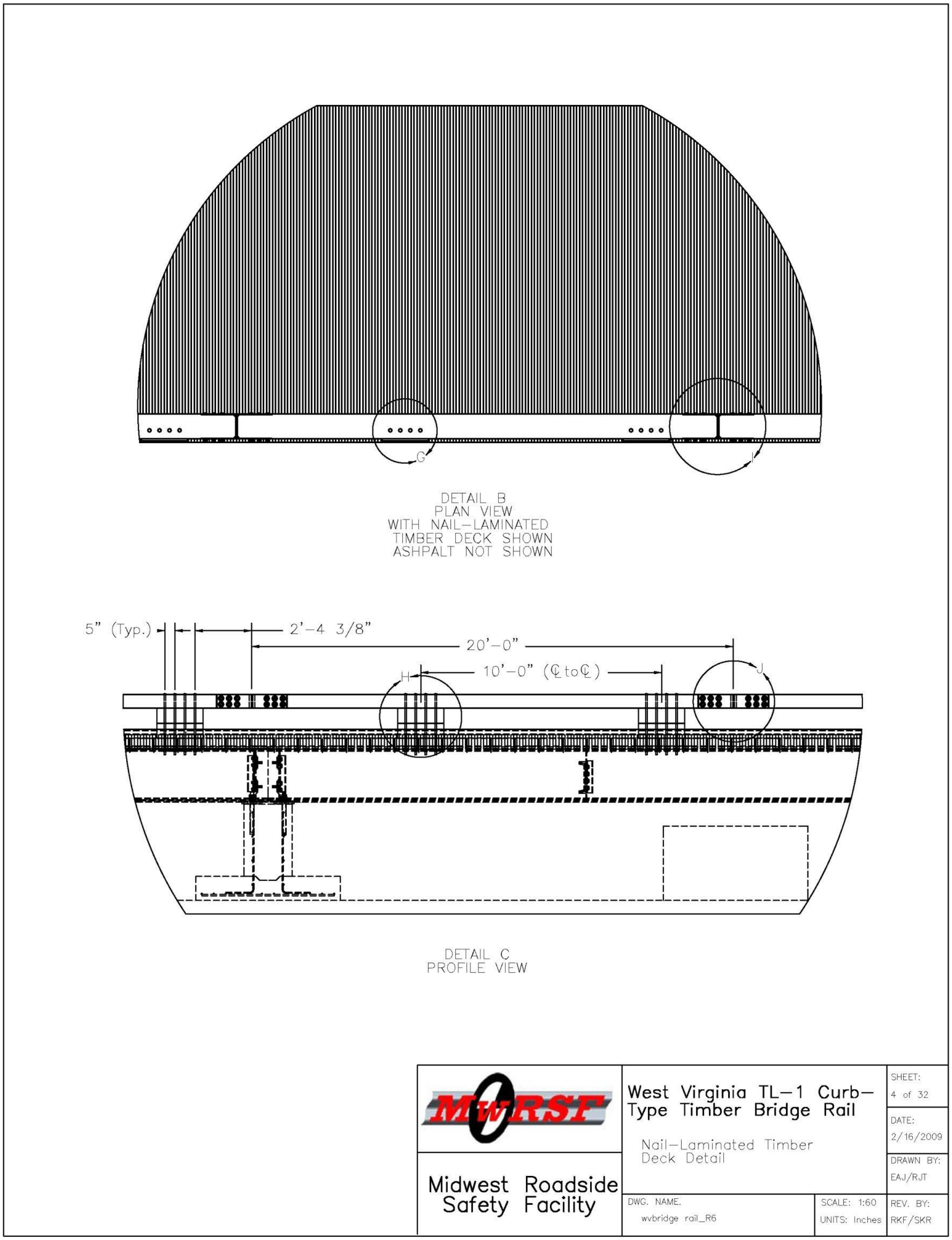
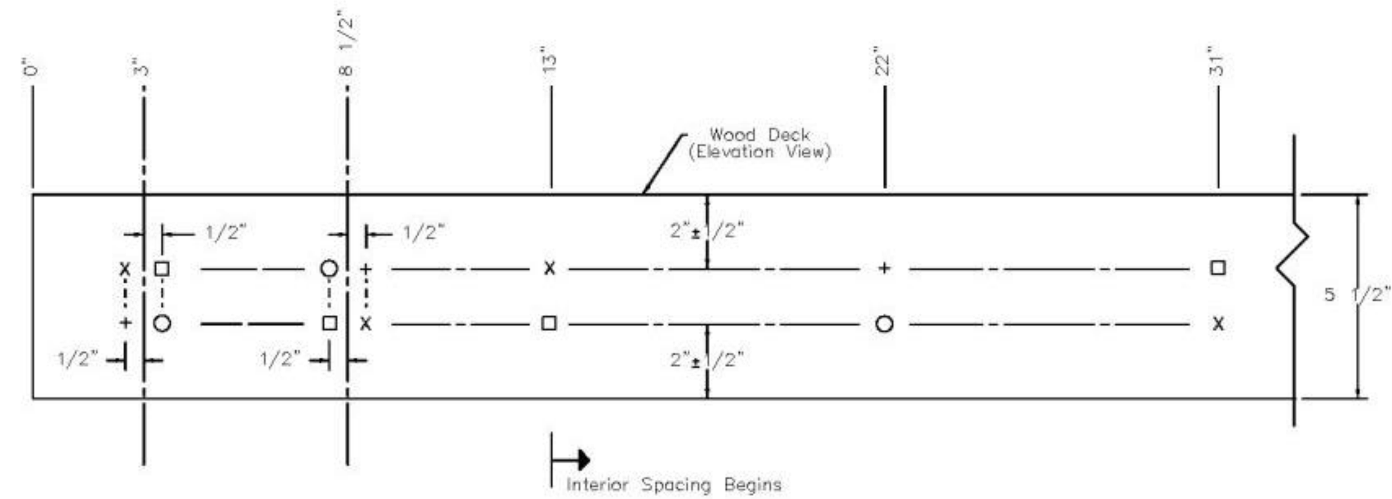


Figure 28. Timber Bridge Rail, Top and Front Views, Test No. WVBR-1



Nail Locations	
Piece of Wood	Symbol
1st	+
2nd	X
3rd	O
4th	□
5th	+

Notes:

(1) Timber deck boards shall be nailed together using 20d or 20 penny "common" nails.

(2) Exterior deck nailing pattern is shown above.

(3) Liquid Nails adhesive should be applied using 2 beads minimum to the end 3 ft of exterior wood deck length. Glue adhesive material shall consist of Liquid Nails – Heavy Duty Construction Adhesive, Item No. LN-901.

MwRSF	West Virginia TL-1 Curb-Type Timber Bridge Rail		SHEET: 5 of 32
			DATE: 7/16/2008
Midwest Roadside Safety Facility	Exterior Nail Pattern for Timber Deck		DRAWN BY: EMA/RJT
	DWG. NAME: wvbridge rail_RS	SCALE: None UNITS: Inches	REV. BY: RKF/SKR

Figure 29. Exterior Nail Pattern for Transverse, Nail-Laminated, Timber Bridge Deck

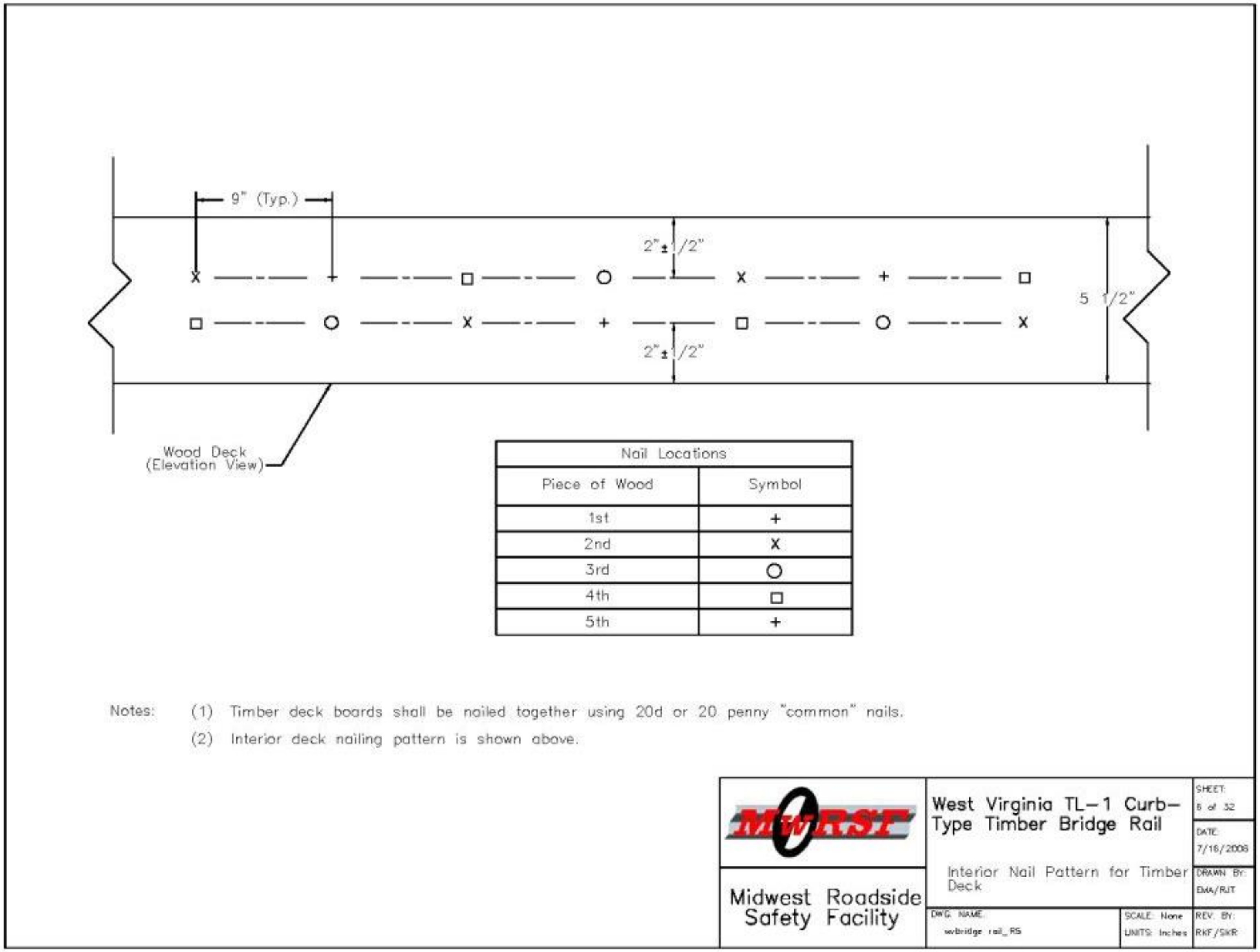


Figure 30. Nail Pattern for Transverse, Nail-Laminated, Timber Bridge Deck

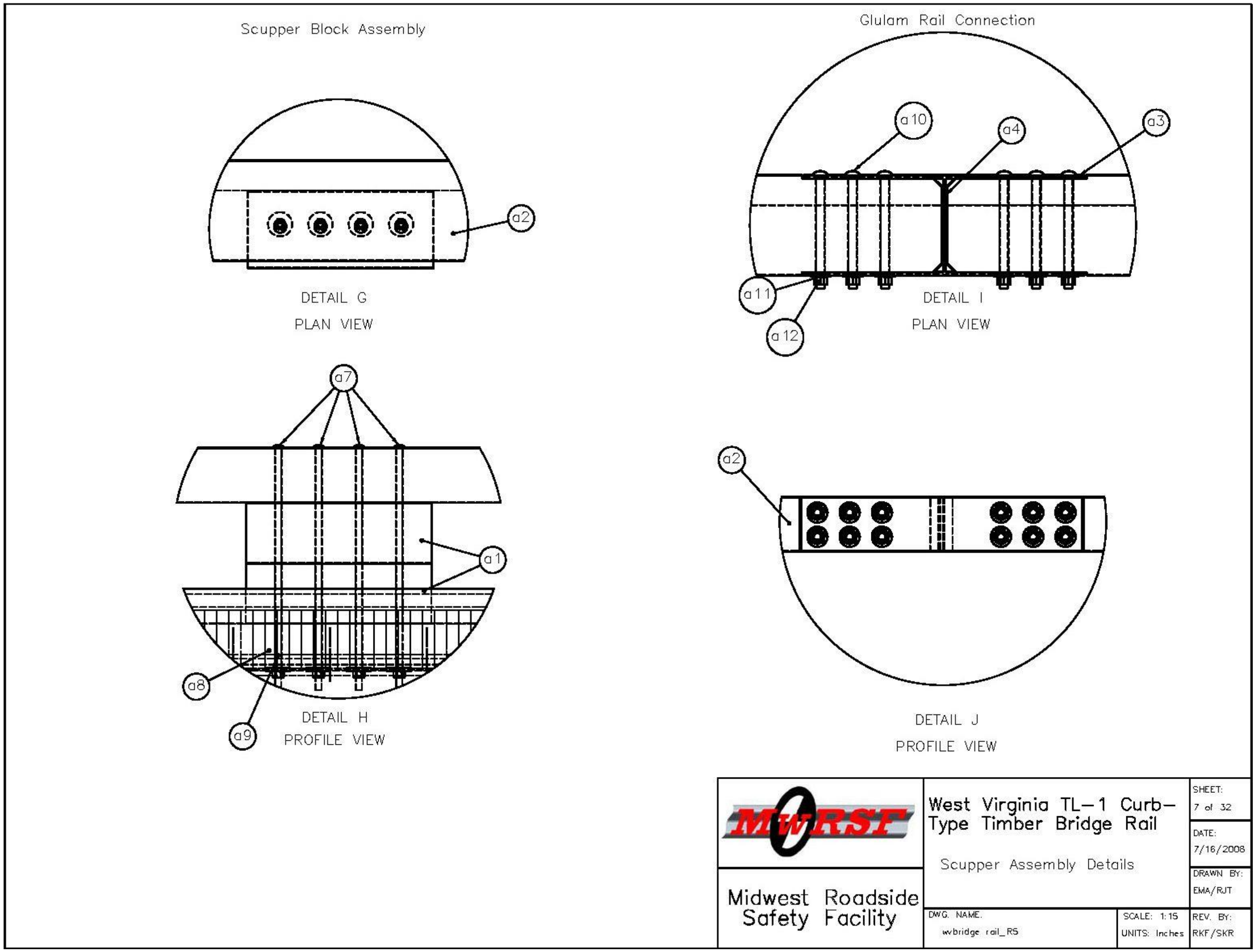


Figure 31. Rail Connection Details, Test No. WVBR-1

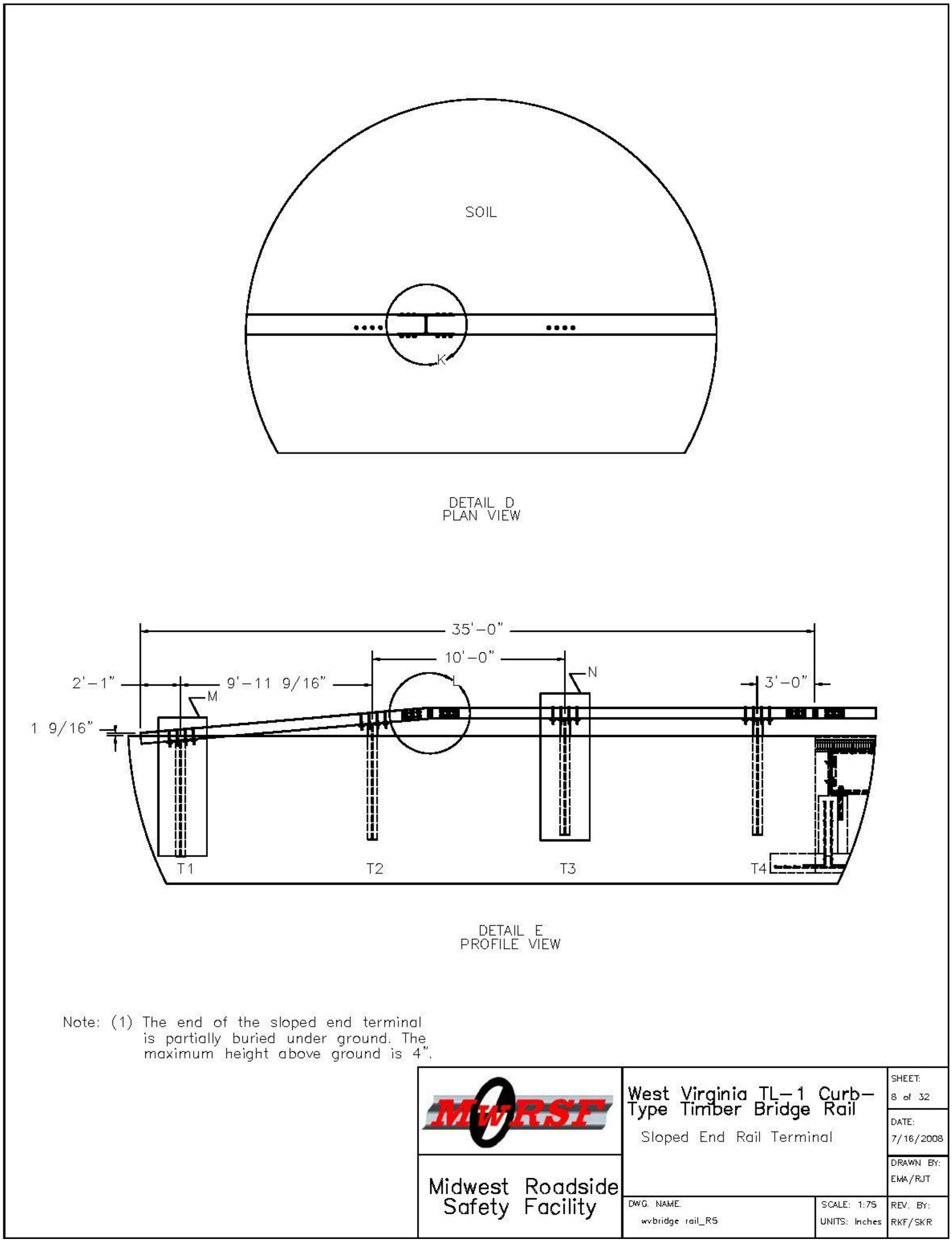


Figure 32. Rail End Treatment, Test No. WVBR-1

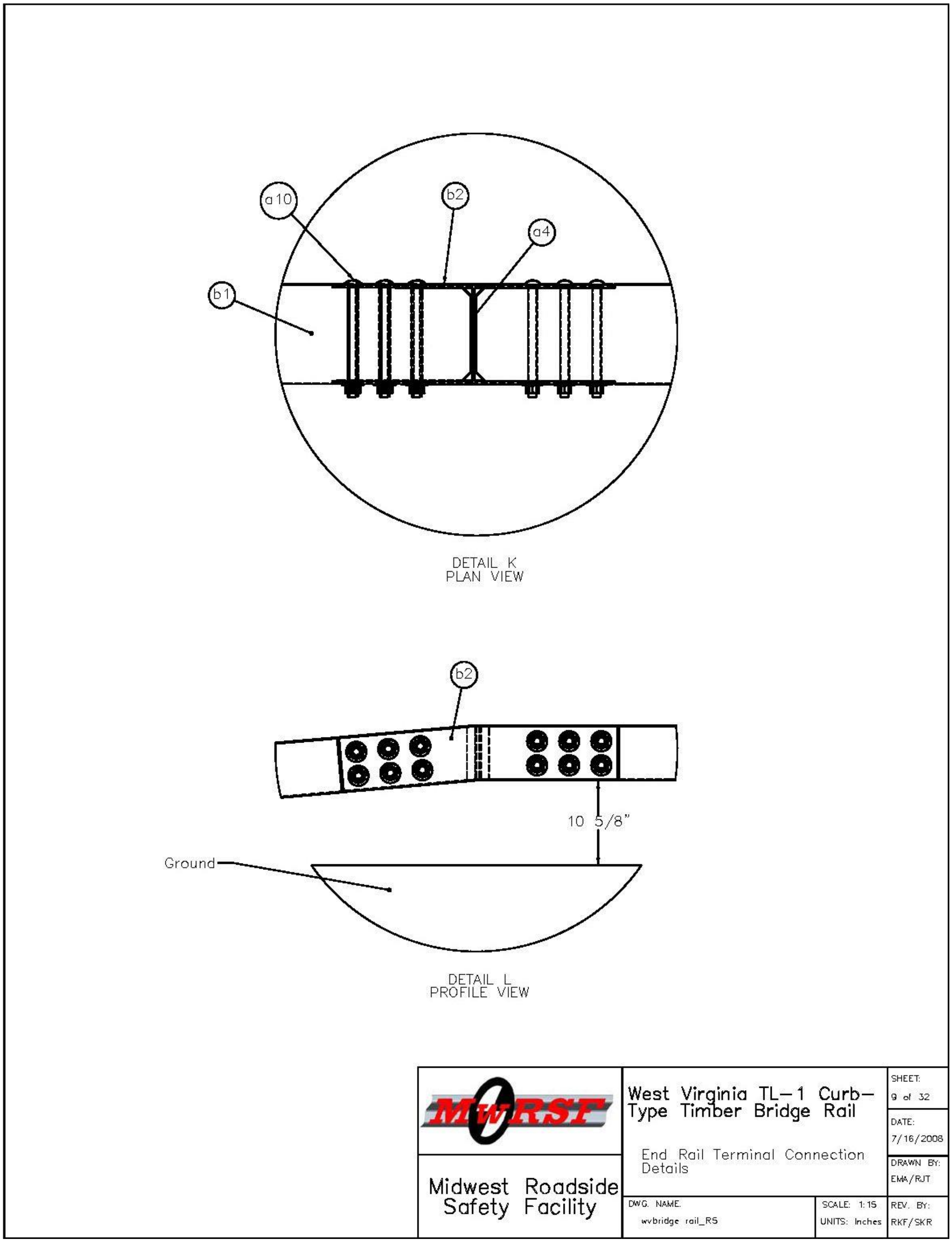


Figure 33. Rail End Treatment Connection Details, Test No. WVBR-1

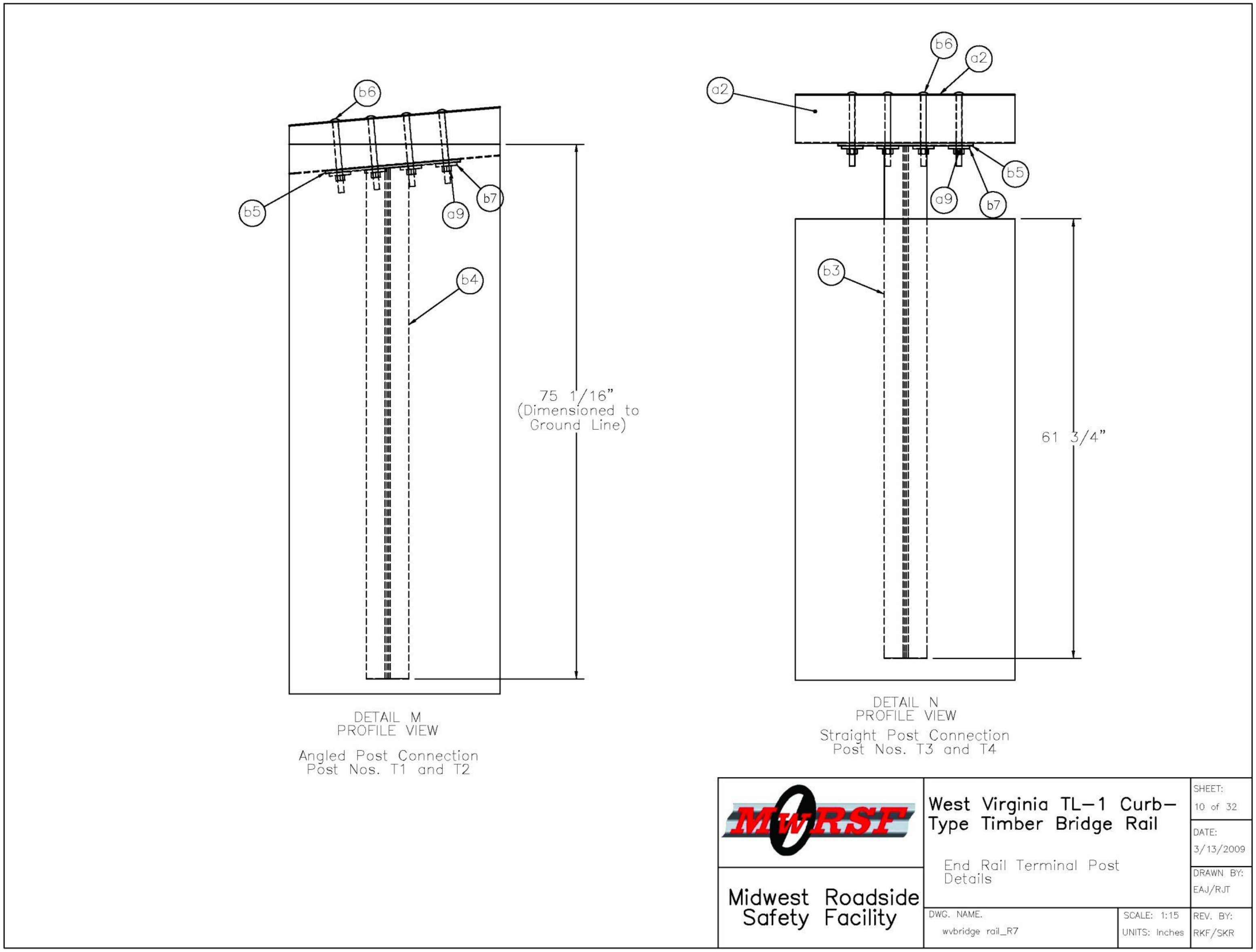


Figure 34. Rail End Section Steel Posts, Test No. WVBR-1

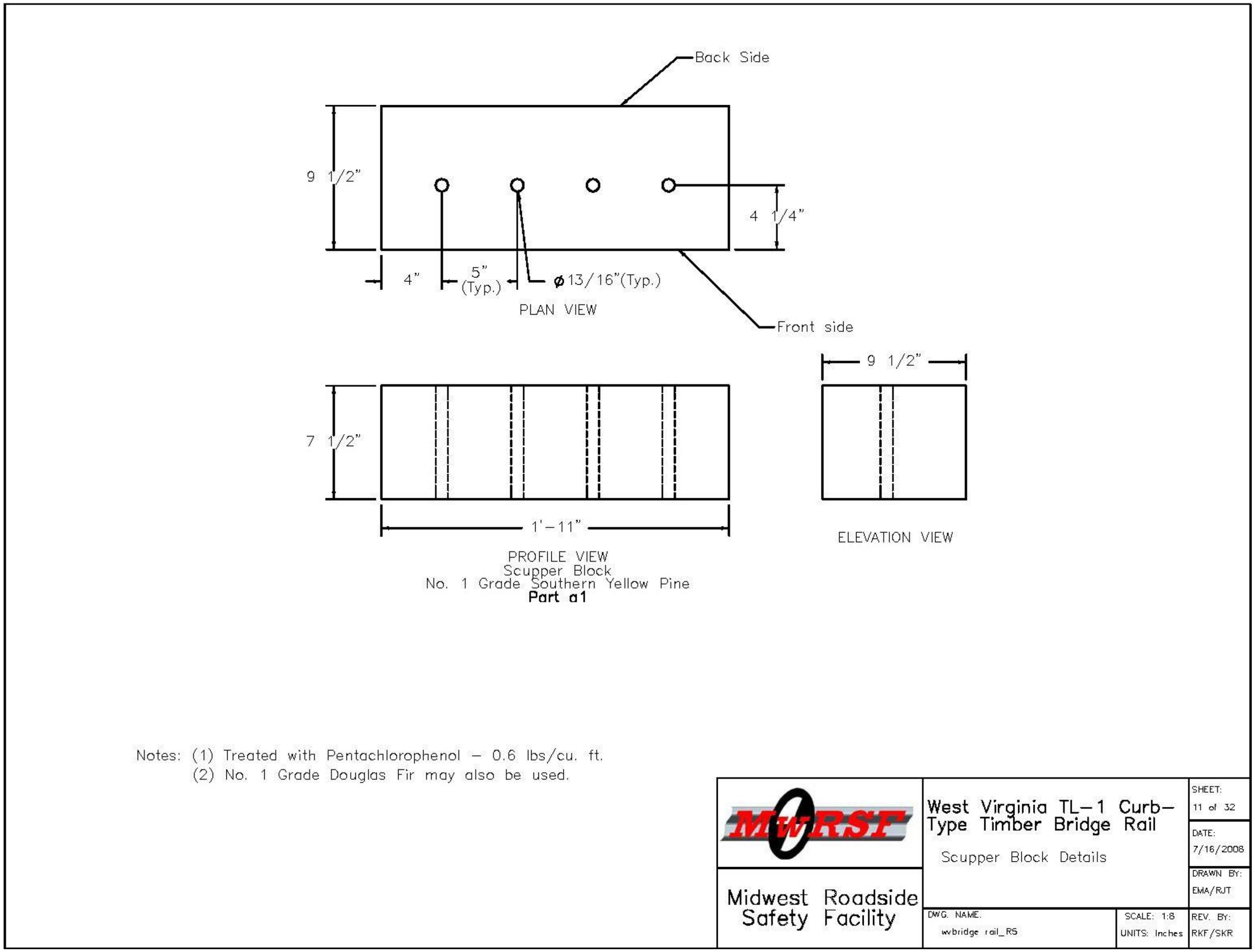


Figure 35. Scupper Block Details, Test No. WVBR-1

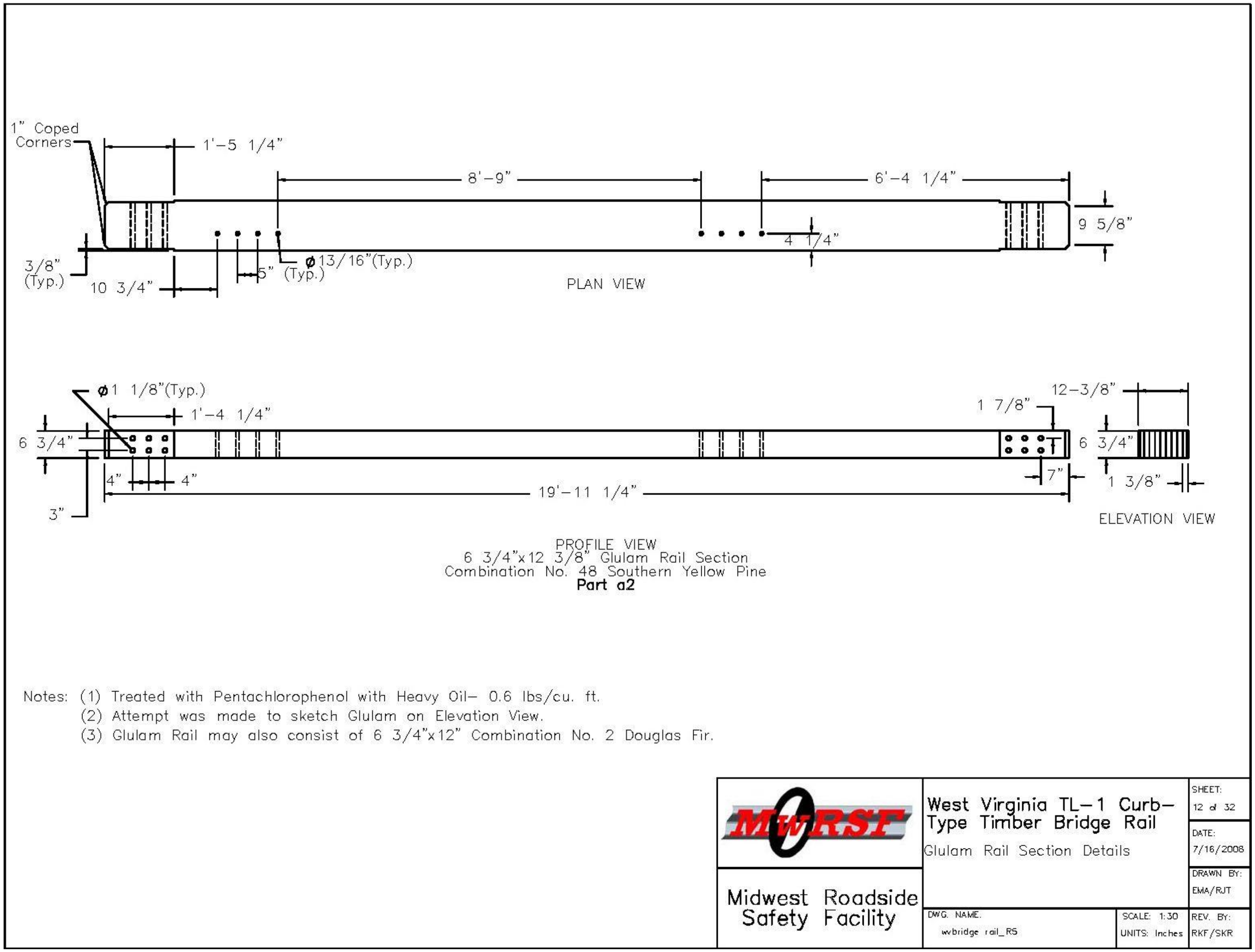


Figure 36. Glulam Rail Segment Details, Test No. WVBR-1

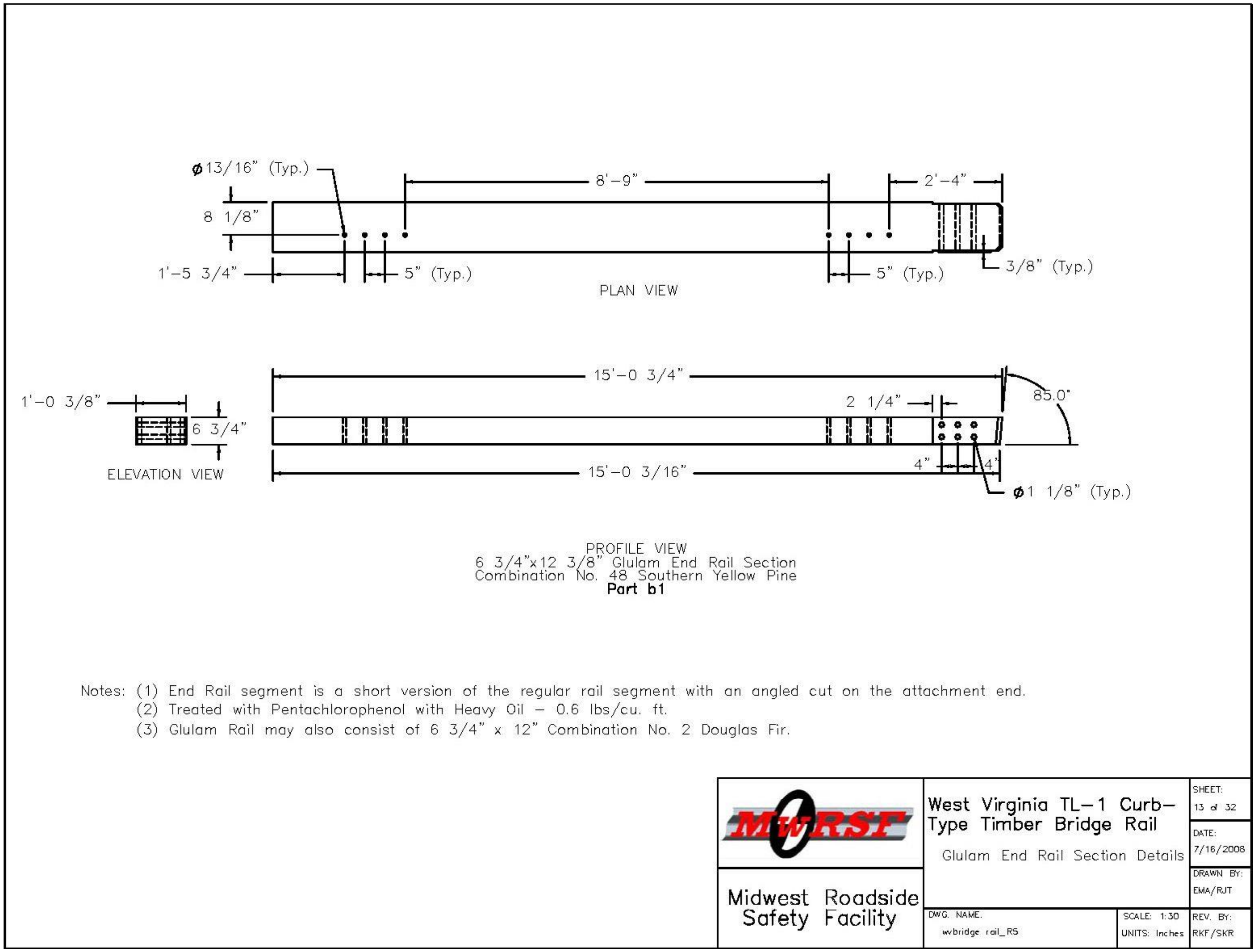


Figure 37. Glulam Rail End Segment Details, Test No. WVBR-1

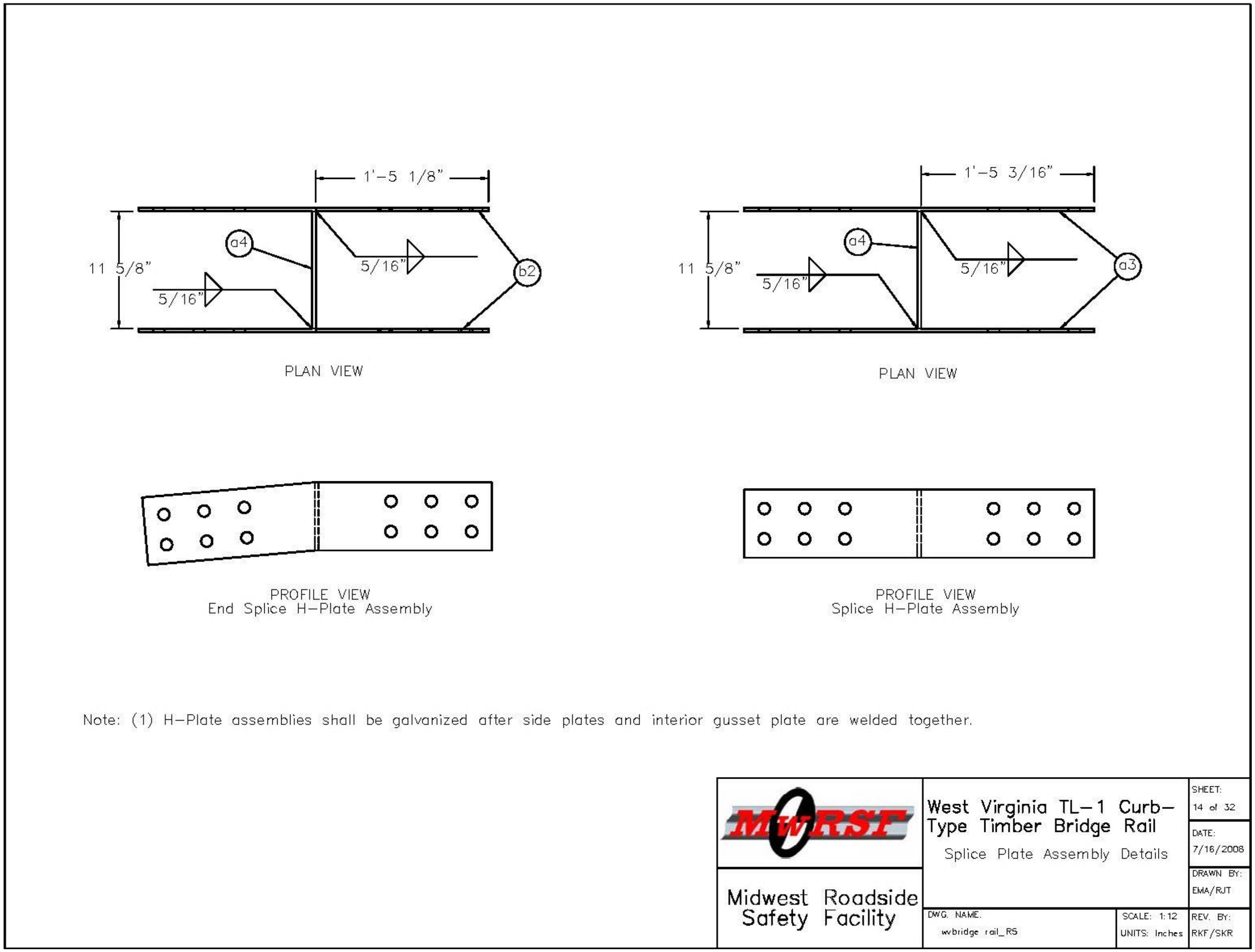


Figure 38. Splice Plate Assembly Details, Test No. WVBR-1

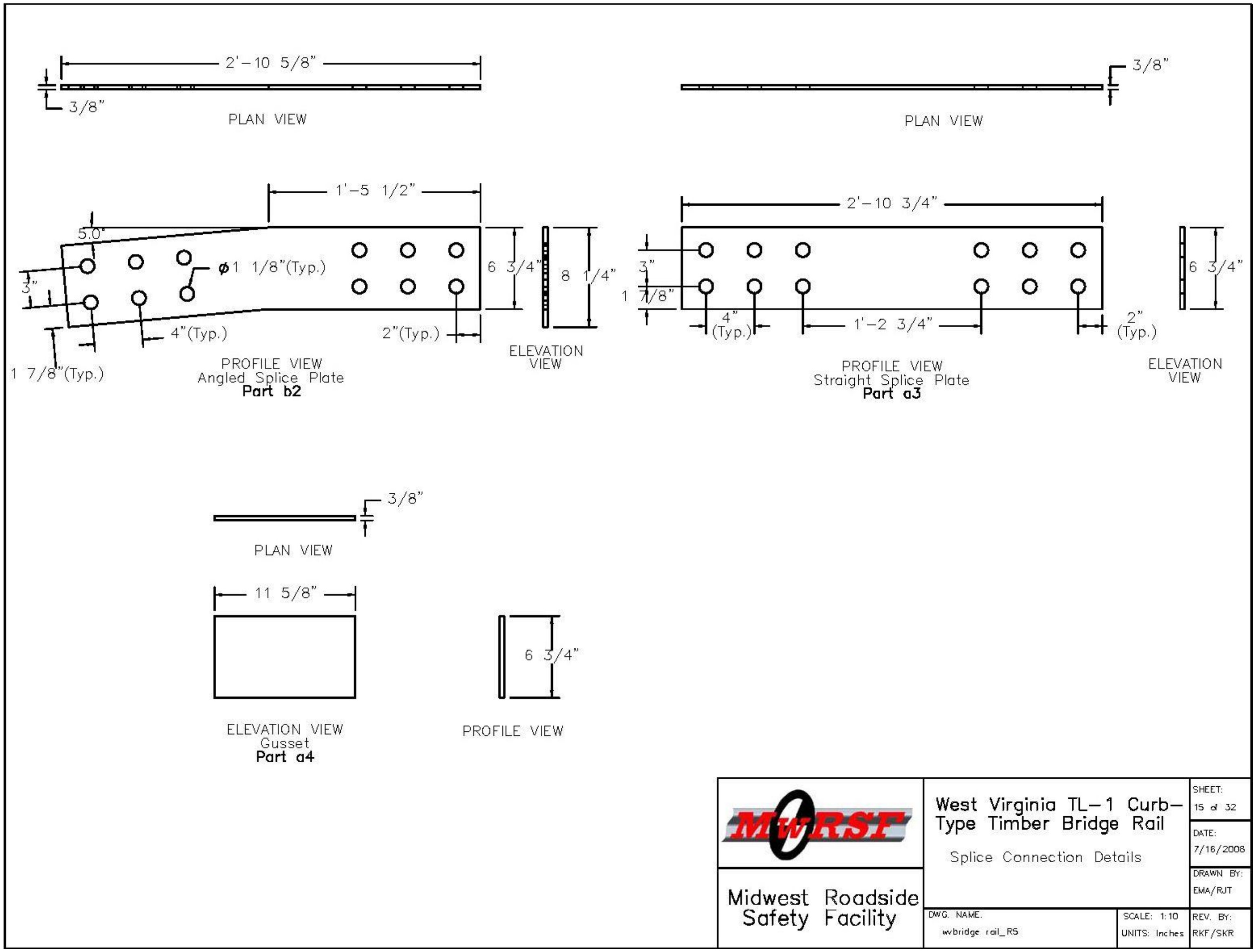


Figure 39. Splice Plate Component Details, Test No. WVBR-1

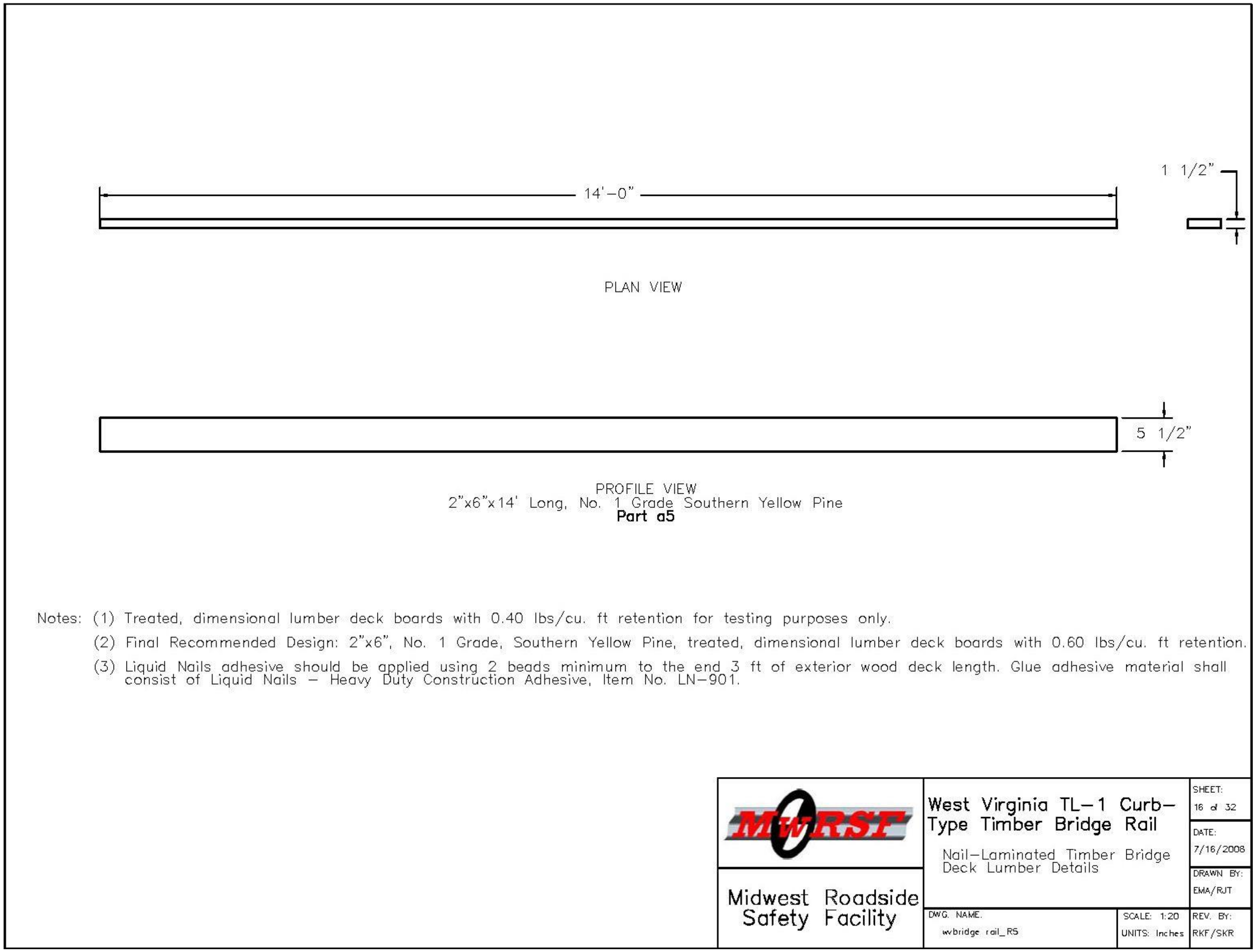


Figure 40. Dimensional Lumber Details for Timber Deck, Test No. WVBR-1

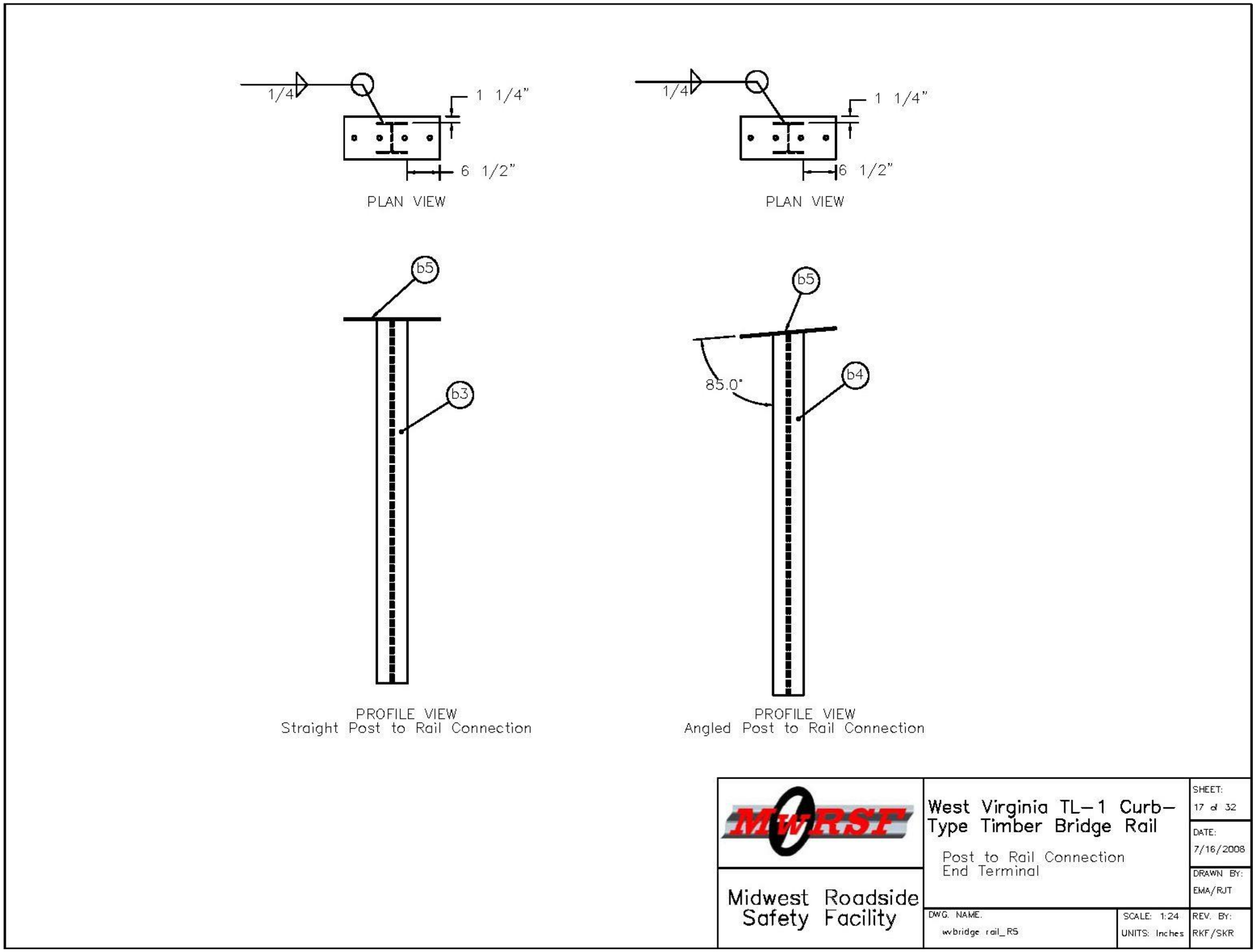


Figure 41. Steel Post Assembly Details, Test No. WVBR-1

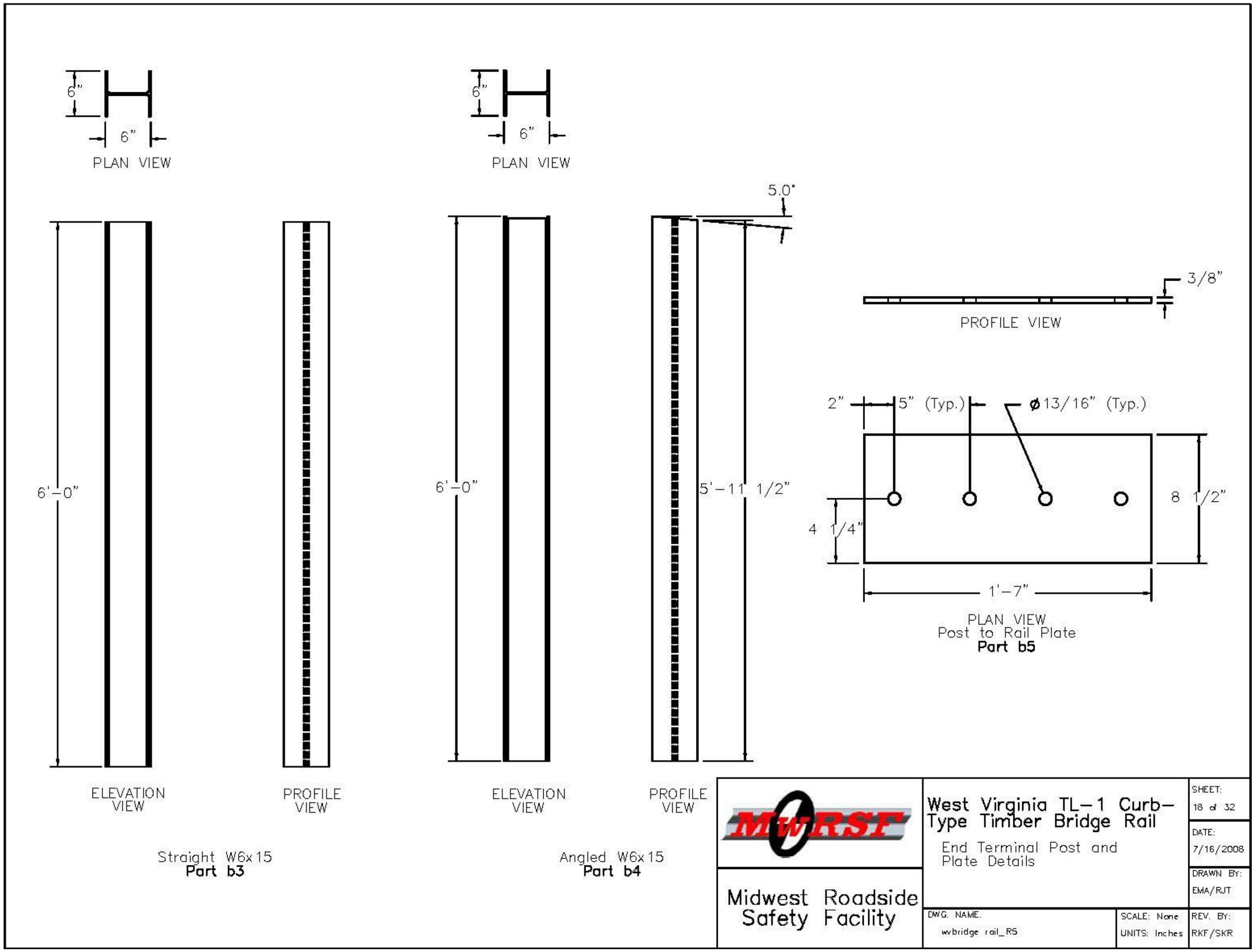
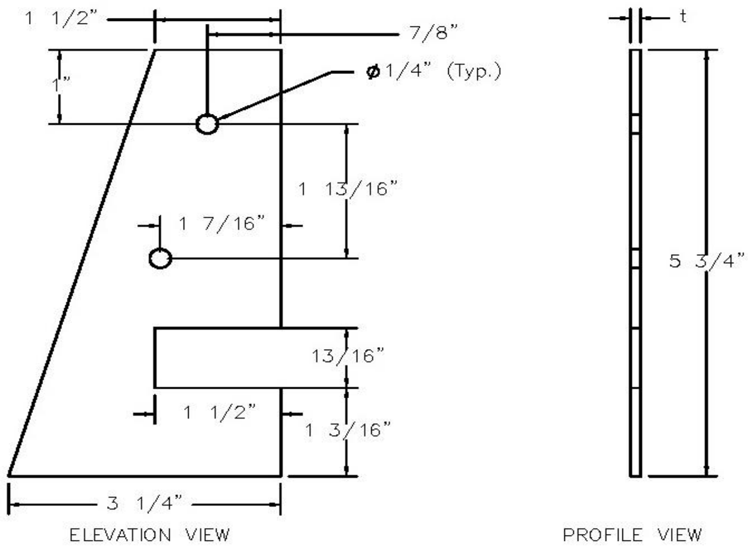


Figure 42. Steel Post Details, Test No. WVBR-1



Galvanized Deck Anchor Bracket
Part a6

Steel Specification	Galvanizing	Thickness, t
ASTM A1011 Gr. 36 Hot Rolled Sheet (U.S. Steel Designator – HR36SK53) [Fy(min) = 36 ksi, Fu(min) = 53 ksi]	ASTM A653 G90	11 Gauge
ASTM A1011 Gr. 36 Type 1 SS Hot Rolled Structural Steel Sheet [Fy(min) = 36 ksi, Fu(min) = 53 ksi]	ASTM A653 G90	11 Gauge
ASTM A1011 Gr. 36 Type 2 SS Hot Rolled Structural Steel Sheet [Fy(min) = 36 ksi, Fu(min) = 58–80 ksi]	ASTM A653 G90	11 Gauge
ASTM A36 Steel Plate [Fy(min) = 36 ksi, Fu(min) = 58–80 ksi]	Hot Dipped Galvanized ASTM ??? (0.90 oz/sq. ft)	1/8"

Notes: (1) Use 11–gauge option for crash–tested system.
(2) Deck anchor brackets shall be installed using 20d or 20 penny "common" nails.

Midwest Roadside Safety Facility

West Virginia TL–1 Curb–Type Timber Bridge Rail

Deck Anchor Bracket Details

SHEET: 19 of 32
DATE: 7/16/2008
DRAWN BY: EMA/RJT
DWG. NAME. wvbridge rail_R5
SCALE: 1:2
UNITS: Inches
REV. BY: RKF/SKR

Figure 43. Deck Anchor Bracket Details, Test No. WVBR-1

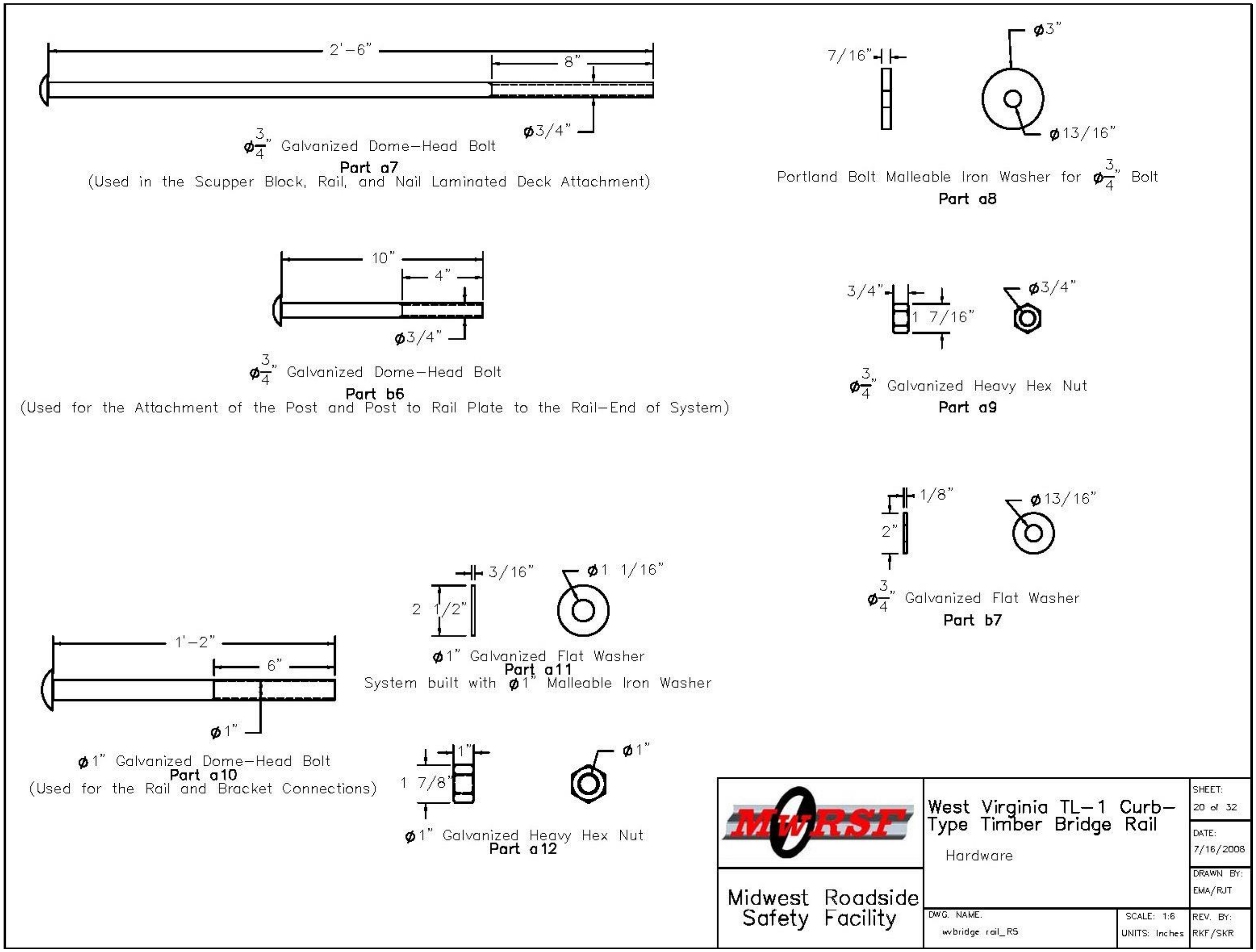


Figure 44. Connection Hardware, Test No. WVBR-1

West Virginia TL–1 Curb–Type Timber Bridge Rail			
Item No.	QTY.	Description	Material Specifications
a1	24	Scupper Block	Southern Yellow Pine No. 1
a2	7	Glulam Rail Section	Southern Yellow Pine Combination No. 48
a3	12	Straight Splice Plate	Design–A572 Gr. 42; Used–A656 Gr. 50 Ty. 7
a4	7	Splice Gusset	Design–A572 Gr. 42; Used–A656 Gr. 50 Ty. 7
a5	960	2"x6"x14' Long Treated, Dimensional Lumber (0.60 lbs retention)	Southern Yellow Pine No. 1
a6	240	Deck Anchor Plate	See Page 18 of 31
a7	48	3/4" Dome Head Bolt 30" Long	Galvanized A307
a8	64	3/4" Malleable Iron Washer	–
a9	64	3/4" Heavy Hex Nut	Galvanized A307
a10	84	1" Dome Head Bolt 14" Long	Galvanized A307
a11	84	1" Flat Washer (as built–Malleable Iron)	Galvanized A307
a12	84	1" Heavy Hex Nut	Galvanized A307
b1	1	Glulam End Rail Section	Southern Yellow Pine Combination No. 48
b2	2	Angled End Splice Plate	Design–A572 Gr. 42; Used–A656 Gr. 50 Ty. 7
b3	2	Straight W6x15	Galvanized A36
b4	2	Angled W6x15	Galvanized A36
b5	4	Post to Rail Plate	Galvanized A36
b6	16	3/4" Dome Head Bolt 10" Long	Galvanized A307
b7	16	3/4" Flat Washer	Galvanized A307
c11	18	0.625x4.75x25.375 Plate	A36
c12	18	0.375x4.75x25.375 Plate	A36
c14	12	WT3x10x66.6" Long	A36
c16	6	C15x33.9x66.5" Long	A36
c17	6	Sole Plate 1 1/2" Thick	A36
c18	6	Sole Plate 3/4" Thick	–
c19	6	W27x94	–

MwRSF Midwest Roadside Safety Facility	West Virginia TL–1 Curb–Type Timber Bridge Rail Bill of Materials for Bridge Rail		SHEET: 21 of 32 DATE: 3/13/2009 DRAWN BY: EAJ/RJT
	DWG. NAME. wvbridge rail_R7	SCALE: None UNITS: Inches	REV. BY: RKF/SKR

Figure 45. Bill of Materials, Test No. WVBR-1

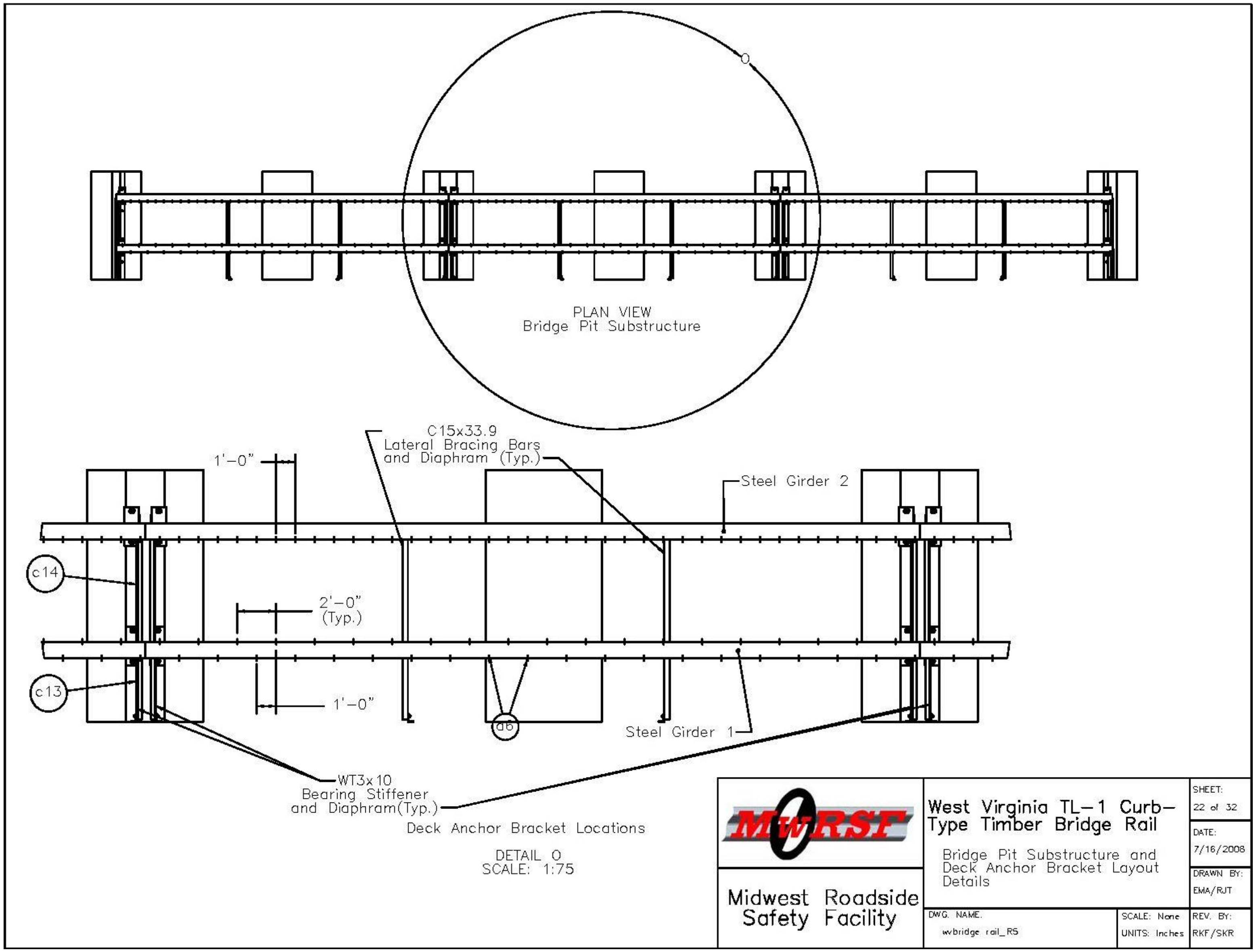
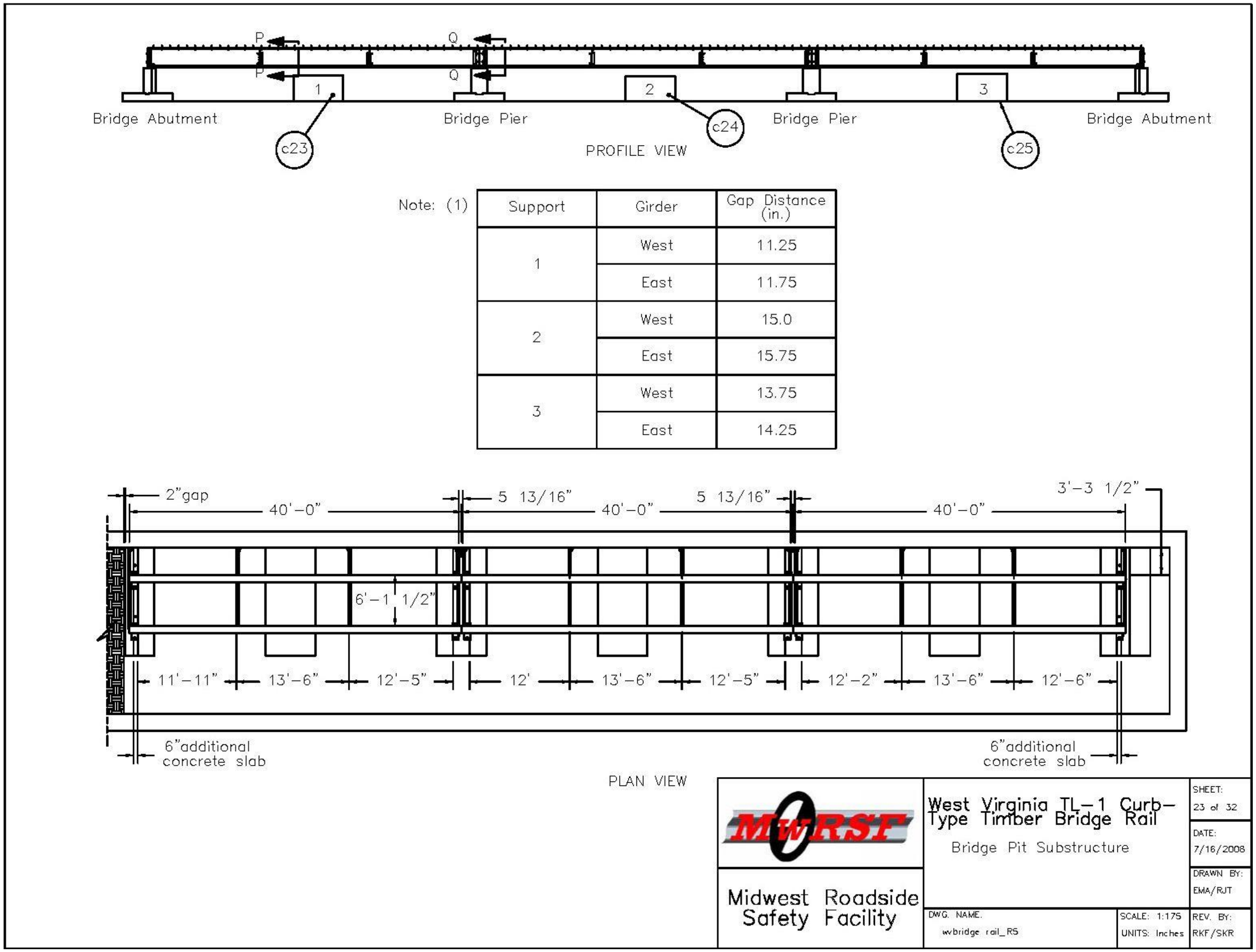


Figure 46. Bridge Substructure and Deck Anchor Bracket Layout, Test No. WVBR-1



Support	Girder	Gap Distance (in.)
1	West	11.25
	East	11.75
2	West	15.0
	East	15.75
3	West	13.75
	East	14.25

Figure 47. Bridge Pit Substructure, Test No. WVBR-1

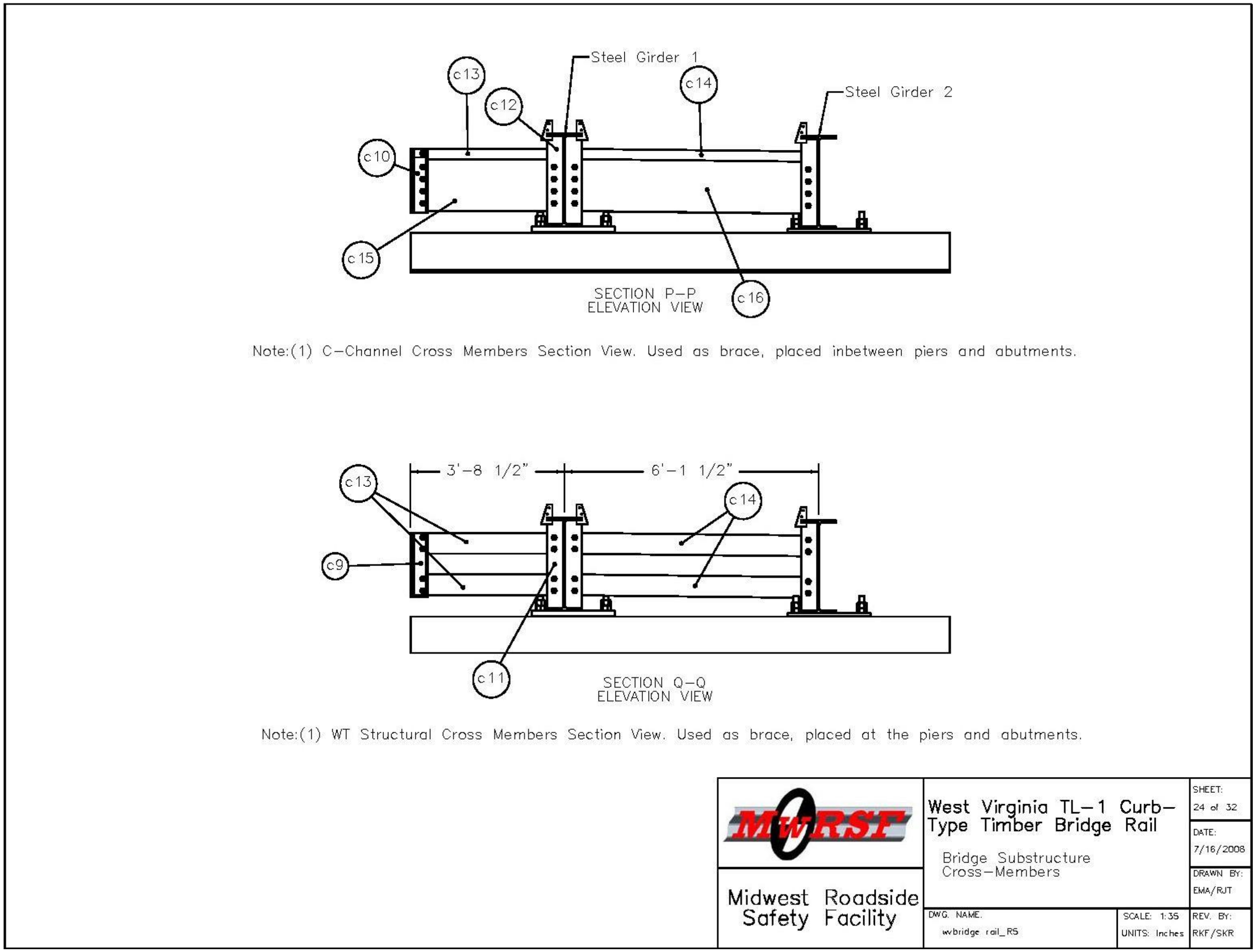


Figure 48. Bridge Girder Diaphragms, Test No. WVBR-1

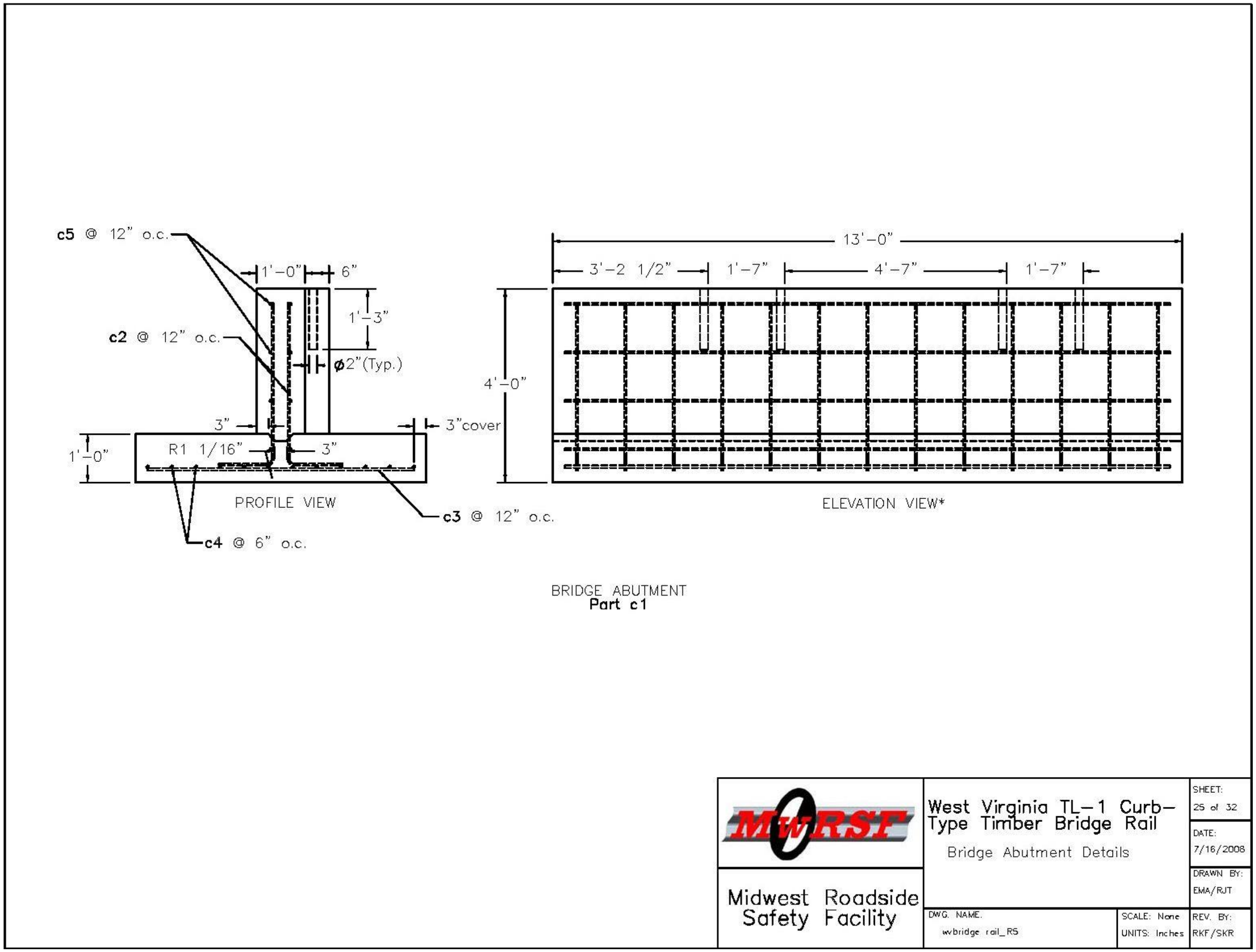


Figure 49. Bridge Abutment Details, Test No. WVBR-1

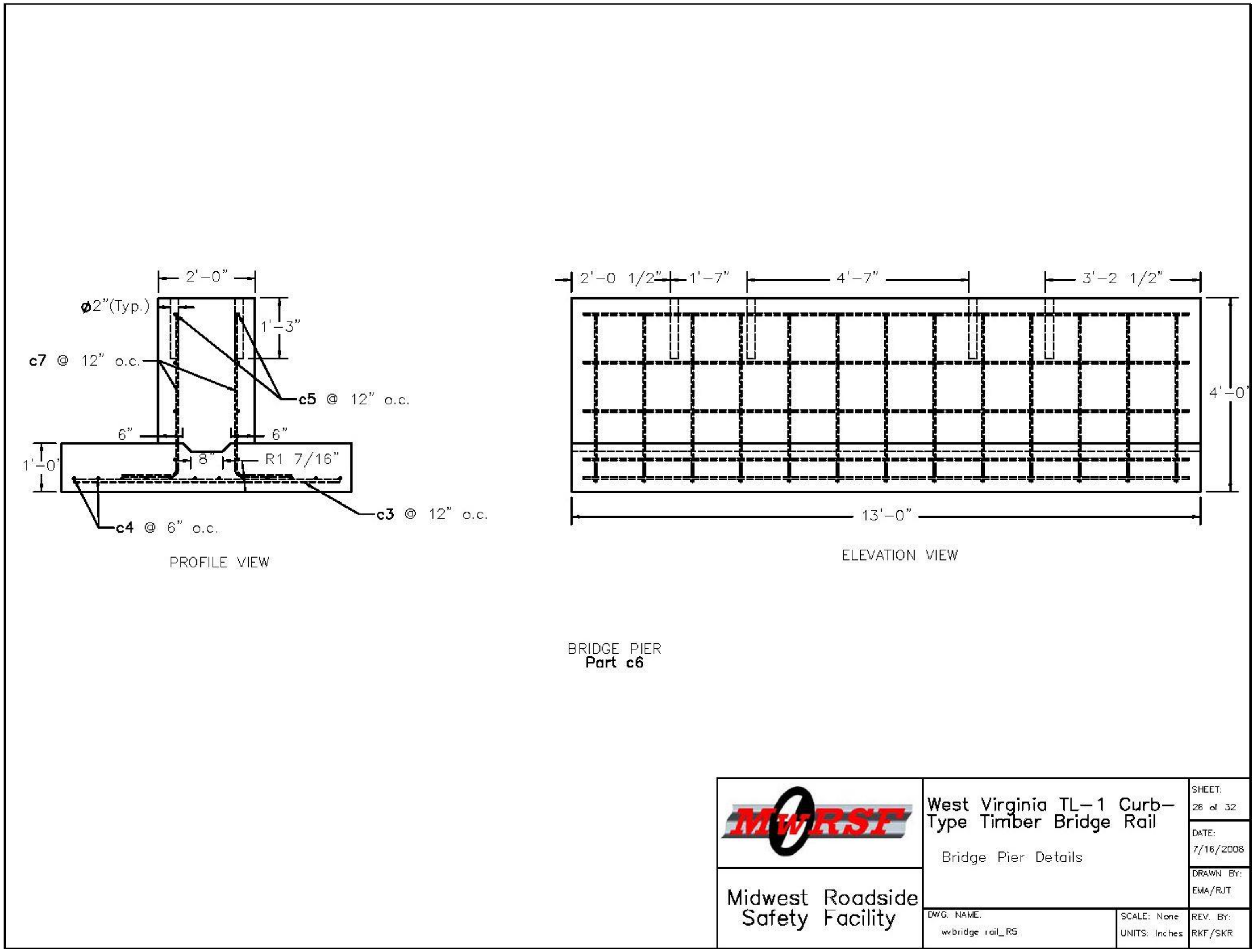


Figure 50. Bridge Bent Details, Test No. WVBR-1

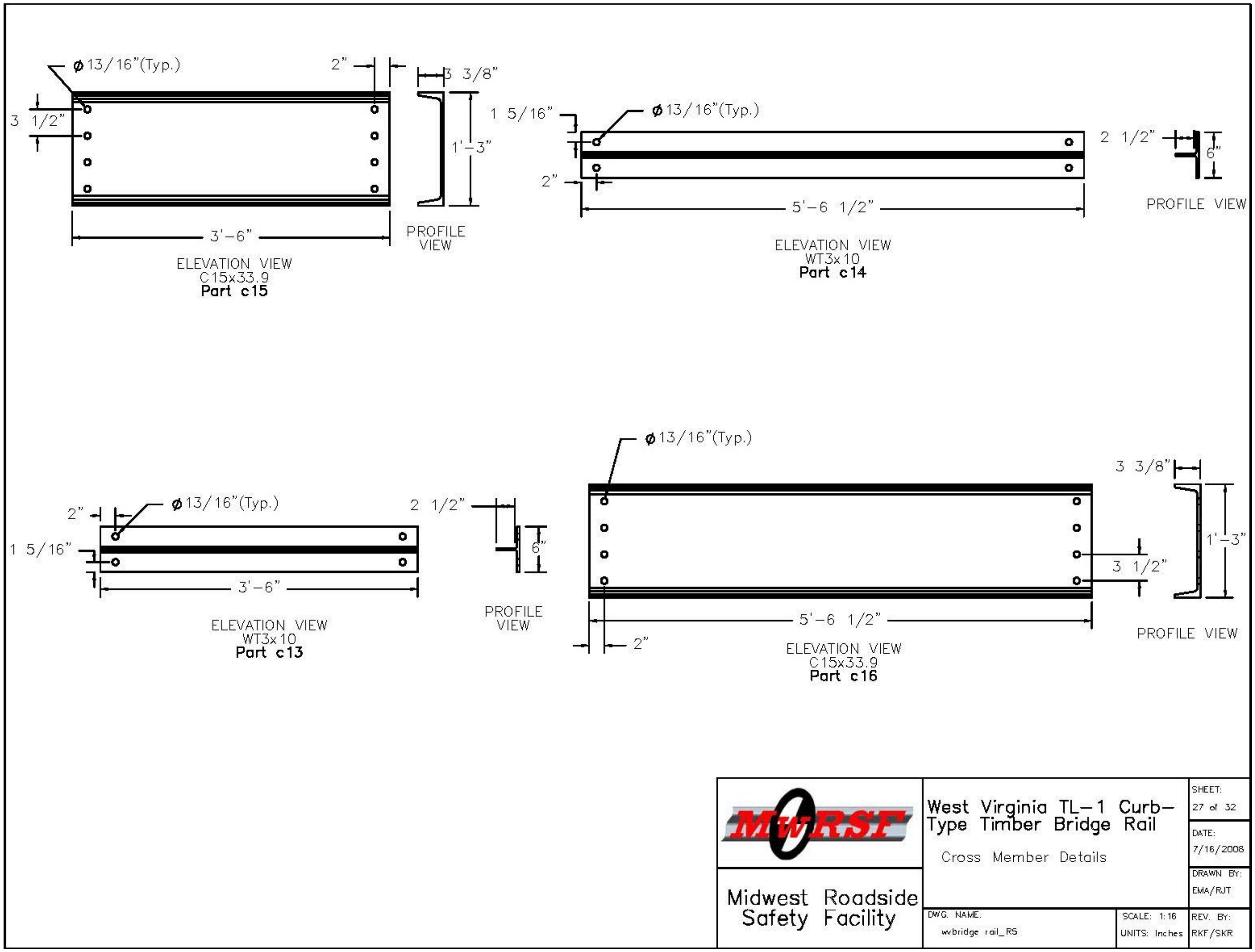


Figure 51. Diaphragm Details, Test No. WVBR-1

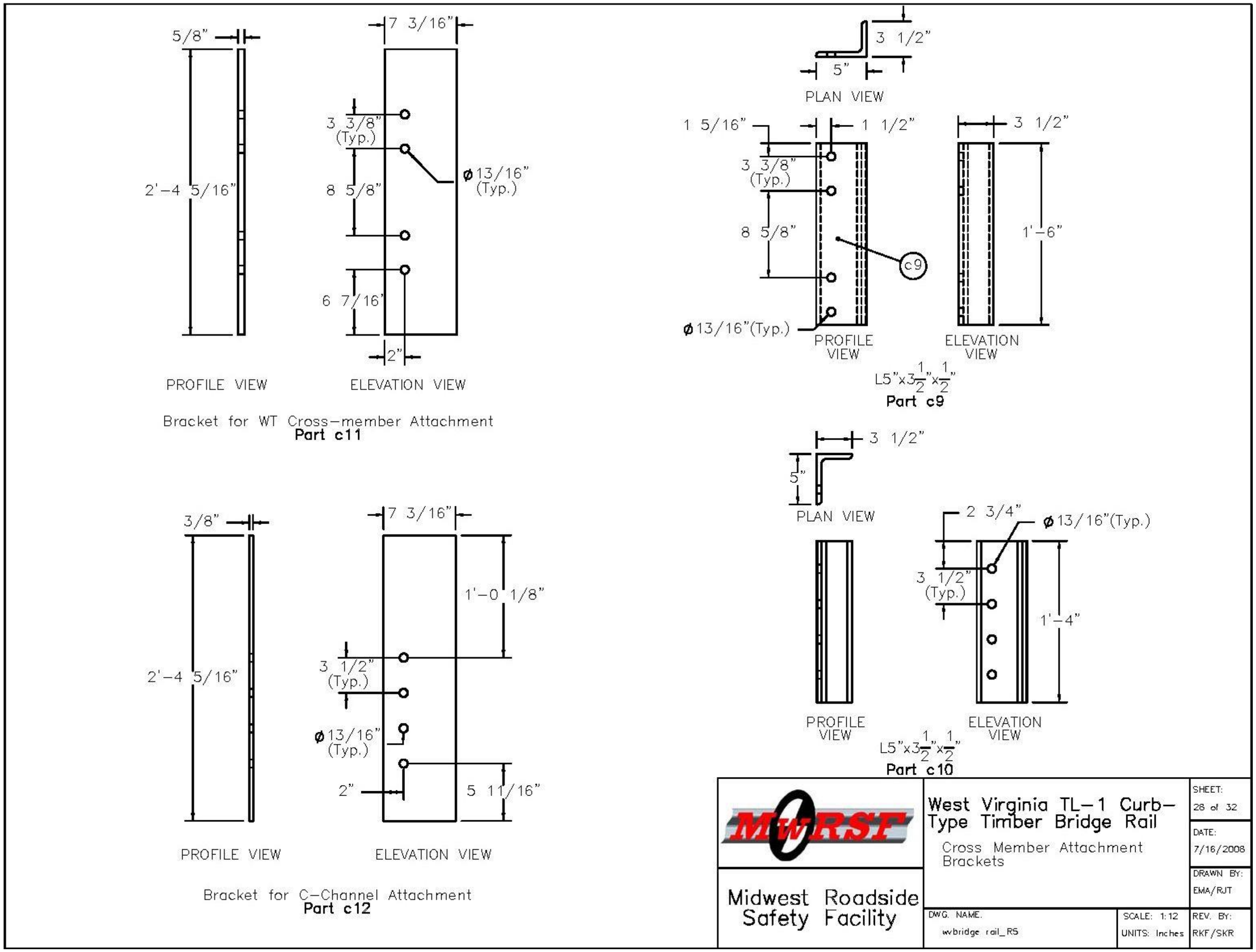


Figure 52. Diaphragm Attachment Brackets, Test No. WVBR-1

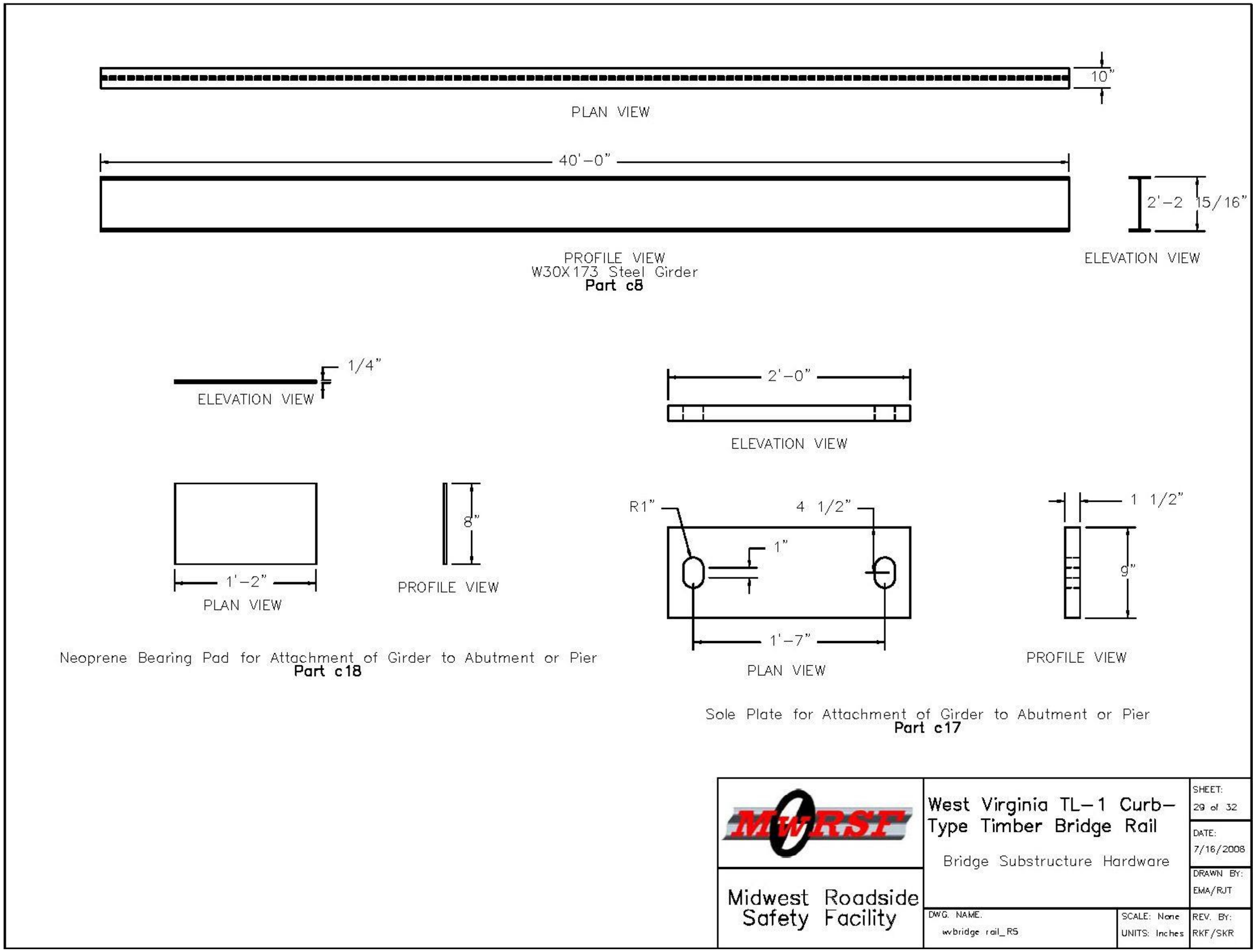


Figure 53. Bridge Substructure Hardware, Test No. WVBR-1

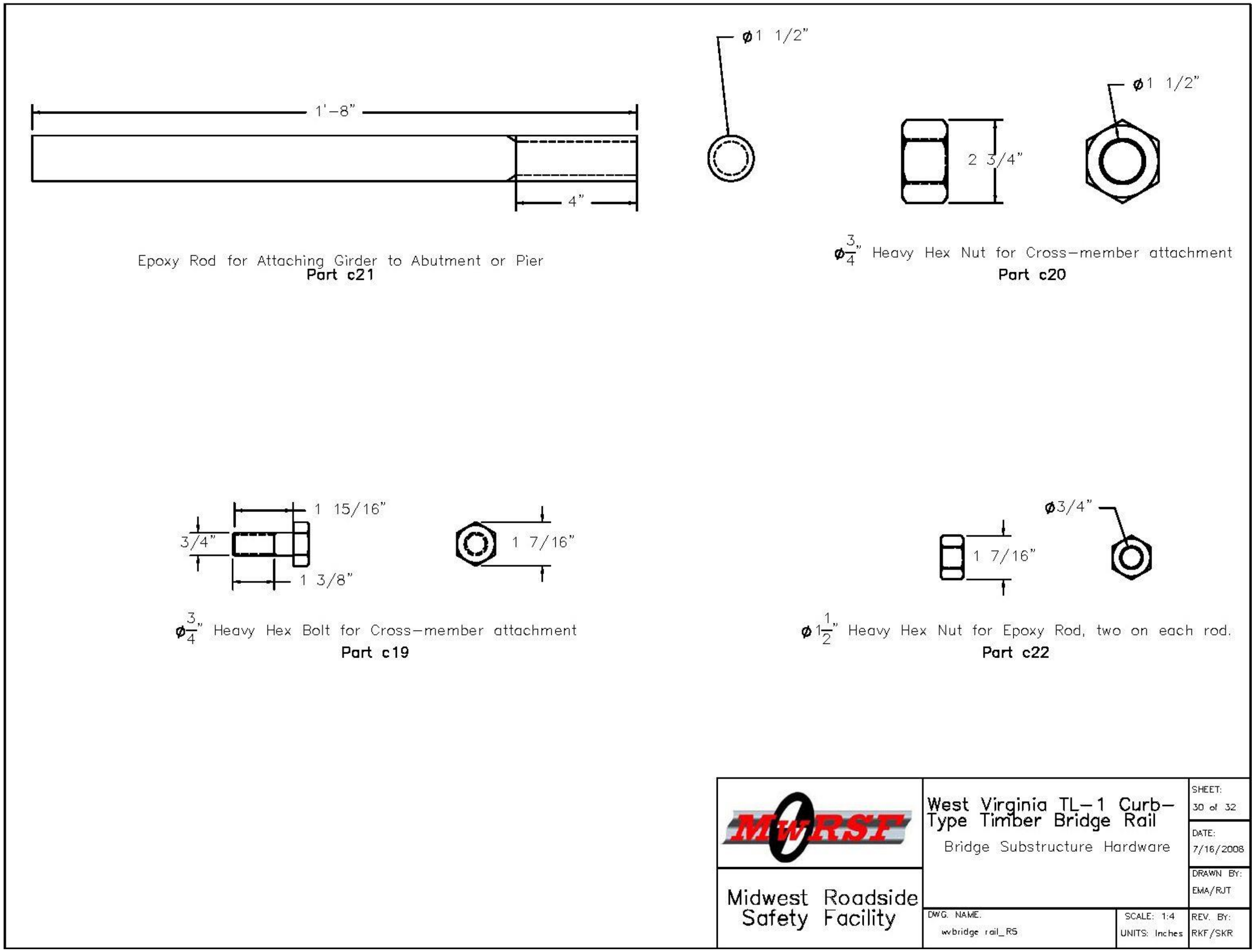


Figure 54. Bridge Substructure Hardware, Test No. WVBR-1

Midwest Roadside Safety Facility Bridge Pit Substructure			
Item No.	QTY.	Description	Material Spec
c1	2	Bridge Abutment	Concrete
c2	13	Abutment #4 Bent Rebar	Grade 60
c3	13	#5 Rebar 5'–6" Long	Grade 60
c4	12	#5 Rebar 12'–6" Long	Grade 60
c5	8	#4 Rebar 12'–6" Long	Grade 60
c6	2	Bridge Pier	Concrete
c7	26	Pier #4 Bent Rebar	Grade 60
c9	6	L5x3.5x0.5x18" Long	A36
c10	6	L5x3.5x0.5x16" Long	A36
c13	12	WT3x10x42" Long	A36
c15	6	C15x33.9x42" Long	A36
c18	12	Bearing Pad	Neoprene
c19	198	HHBOLT 0.75–10x1.75x1.375–C	Grade 5
c20	192	HHNUT 0.75–10	Grade 5
c21	24	20" Epoxy Rod	Grade 60
c22	48	HHNUT 1.5–6	Grade 5
c23	1	Concrete Support 1	Concrete
c24	1	Concrete Support 2	Concrete
c25	1	Concrete Support 3	Concrete
a5	960	2"x6"x14' Long Treated, Dimensional Lumber (0.60 lbs retention)	Southern Yellow Pine No. 1

MwRSF
Midwest Roadside Safety Facility

West Virginia TL–1 Curb–Type Timber Bridge Rail
Bill of Materials for Bridge Substructure

SHEET: 31 of 32
DATE: 7/16/2008
DRAWN BY: EMA/RJT
DWG. NAME. wvbridge rail_RS
SCALE: None
UNITS: Inches
REV. BY: RKF/SKR

Figure 55. Substructure Bill of Materials, Test No. WVBR-1

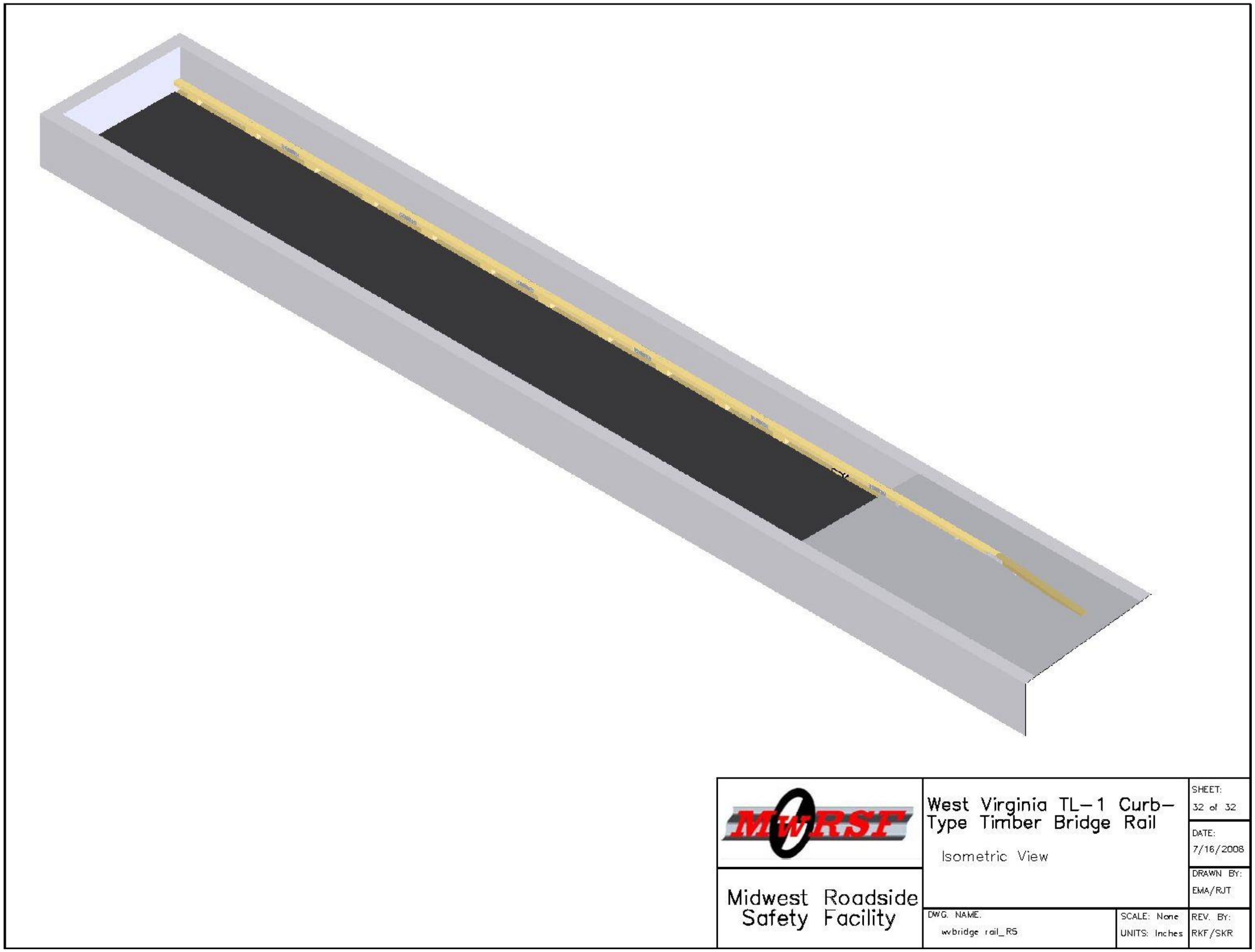


Figure 56. Isometric View of Timber Bridge Rail and End Treatment

Figure 57. Curb-Type Bridge Railing System, Test No. WVBR-1

Figure 58. Rail, Post, and Bridge Deck Connection Details, Test No. WVBR-1

Figure 59. Splice Plate Connections for Glulam Rail, Test No. WVBR-1

Figure 60. Sloped End Treatment, Test No. WVBR-1

6 TEST REQUIREMENTS AND EVALUATION CRITERIA

6.1 Test Requirements

Longitudinal barriers, such as timber bridge rails, must satisfy impact safety standards provided in MASH-08 [6] in order to be accepted by the Federal Highway Administration (FHWA) for use on National Highway System (NHS) new construction projects or as a replacement for existing designs not meeting current safety standards. According to Test Level 1 (TL-1) of MASH-08, longitudinal barrier systems must be subjected to two full-scale vehicle crash tests. The two full-scale crash tests are as follows:

I. Test Designation 1-10 consisting of a 2,420-lb (1,100-kg) small car impacting the timber bridge rail at a nominal speed and angle of 31 mph (50.0 km/h) and 25 degrees, respectively.

II. Test Designation 1-11 consisting of a 5,000-lb (2,268-kg) pickup truck impacting the timber bridge rail at a nominal speed and angle of 31 mph (50.0 km/h) and 25 degrees, respectively.

The test conditions of TL-1 longitudinal barriers are summarized in Table 1.

Table 1. MASH-08 Test Level 1 Crash Test Conditions

Test Article	Test Designation	Test Vehicle	Impact Conditions			Evaluation Criteria[1]
			Speed		Angle (deg)	
			mph	km/h		
Bridge Railing	1-10	1100C	31	50.0	25	A,D,F,H,I
	1-11	2270P	31	50.0	25	A,D,F,H,I

[1] - Evaluation criteria explained in Table 2.

Although the small car test is a requirement of the TL-1 safety performance criteria provided in MASH-08, it was believed that this test was not critical nor needed to garner FHWA acceptance for the curb-type, timber, bridge railing system. The geometry of the bridge rail was designed to mitigate any propensity for small car wheel snag on the railing or post components.

First, only a 13-in. (330-mm) gap existed between the bottom of the curb rail and the top of the concrete wearing surface. Second, a 4-in. (102-mm) lateral offset was provided between the front face of the rail and the front face of the scupper blocks. These two design features mitigated any concerns for small car wheel snag on this railing system. Therefore, the small car test, test designation no. 1-10, was deemed unnecessary for this project and was not performed.

6.2 Evaluation Criteria

Evaluation criteria for full-scale vehicle crash testing are based on three appraisal areas: (1) structural adequacy; (2) occupant risk; and (3) vehicle trajectory after collision. Criteria for structural adequacy are intended to evaluate the ability of the bridge railing to contain and redirect impacting vehicles. Occupant risk evaluates the degree of hazard to occupants in the impacting vehicle. Vehicle trajectory after collision is a measure of the potential for the post-impact trajectory of the vehicle to become involved in secondary collisions with other vehicles or fixed objects. These evaluation criteria are summarized in Table 2 and defined in greater detail in MASH-08. The full-scale vehicle crash test was conducted and reported in accordance with the procedures provided in MASH-08.

Table 2. MASH-08 Test Level 1 Evaluation Criteria for Longitudinal Barriers

<table>
<tr><td>Structural Adequacy</td><td>A.</td><td colspan="3">Test article should contain and redirect the vehicle or bring the vehicle to a controlled stop; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.</td></tr>
<tr><td rowspan="9">Occupant Risk</td><td>D.</td><td colspan="3">Detached elements, fragments or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformations of, or intrusions into, the occupant compartment should not exceed limits set forth in Section 5.3 and Appendix E of MASH-08.</td></tr>
<tr><td>F.</td><td colspan="3">The vehicle should remain upright during and after collision. The maximum roll and pitch angles are not to exceed 75 degrees.</td></tr>
<tr><td rowspan="4">H.</td><td colspan="3">Occupant Impact Velocities (OIV) (see Appendix A, Section A5.3 of MASH-08 for calculation procedure) should satisfy the following limits:</td></tr>
<tr><td colspan="3">Occupant Impact Velocity Limits, ft/s (m/s)</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>30 ft/s
(9.1 m/s)</td><td>40 ft/s
(12.2 m/s)</td></tr>
<tr><td rowspan="4">I.</td><td colspan="3">The Occupant Ridedown Acceleration (ORA) (see Appendix A, Section A5.3 of MASH-08 for calculation procedure) should satisfy the following limits:</td></tr>
<tr><td colspan="3">Occupant Ridedown Acceleration Limits (g’s)</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>15.0 g’s</td><td>20.49 g’s</td></tr>
</table>

7 TEST CONDITIONS

7.1 Test Facility

The testing facility is located at the Lincoln Air Park on the northwest side of the Lincoln Municipal Airport and is approximately 5 miles (8.0 km) northwest of the University of Nebraska-Lincoln.

7.2 Vehicle Tow and Guidance System

A reverse-cable tow system, configured with a 1:2 mechanical advantage, was used to propel the test vehicle. The tow vehicle's travel distance and speed were one-half of those parameters for the test vehicle. The test vehicle was released from the tow cable before impact with the barrier system. A digital speedometer on the tow vehicle increased the accuracy of the test vehicle impact speed.

A vehicle guidance system developed by Hinch [9] was used to steer the test vehicle. A guide-flag, attached to the front-left wheel and the guide cable, was sheared off before impact with the barrier system. The ⅜-in. (10-mm) diameter guide cable was tensioned to approximately 3,500 lbf (16 kN) and supported both laterally and vertically every 100 ft (30.5 m) 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. For test no. WVBR-1 the vehicle guidance system was 227-ft (69.2-m) long.

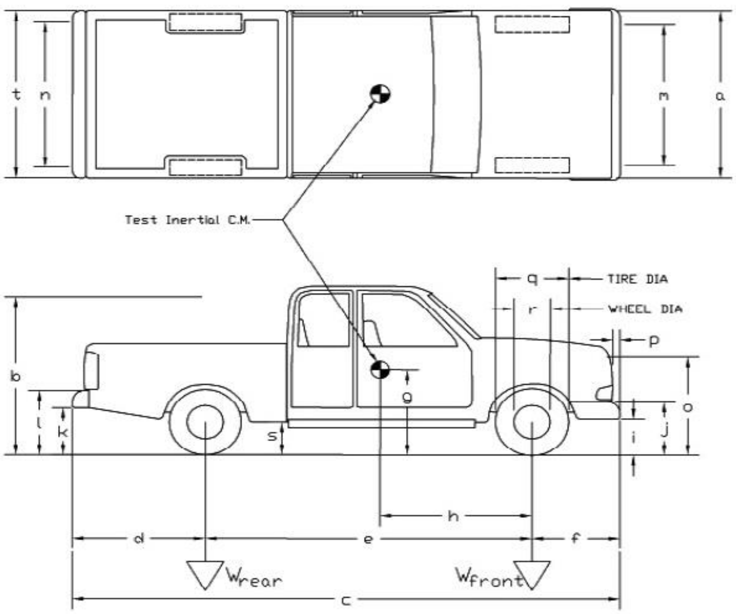
7.3 Test Vehicles

For test no. WVBR-1, a 2002 Dodge Ram 1500 Quad Cab pickup truck was used as the test vehicle. The test inertial and gross static weights were 5,007 lbs (2,271 kg) and 5,179 lbs (2,349 kg), respectively. The test vehicle is shown in Figure 61, and vehicle dimensions are shown in Figure 62.

Figure 61. Test Vehicle, Test No. WVBR-1

Date:	7/18/2008	Test Number:	WVBR-1	Model:	Ram 1500 Q.C.
Make:	Dodge	Vehicle I.D.#:	3B7HA18N12G101147		
Tire Size:	265/70 R17	Year:	2002	Odometer:	120182

*(All Measurements Refer to Impacting Side)



Vehicle Geometry -- mm (in.)

a	1974.9 (77.75)	b	1911.4 (75.25)
c	5765.8 (227.0)	d	1219.2 (48.0)
e	3562.4 (140.25)	f	984.25 (38.75)
g	714.38 (28.125)	h	1568.5 (61.75)
i	381 (15.0)	j	635 (25.0)
k	533.4 (21.0)	l	742.95 (29.25)
m	1730.4 (68.125)	n	1717.7 (67.625)
o	1098.6 (43.25)	p	88.9 (3.5)
q	787.4 (31.0)	r	469.9 (18.5)
s	400.05 (15.75)	t	1905 (75.0)

Wheel Center Height Front	368.3 (14.5)
Wheel Center Height Rear	377.83 (14.875)
Wheel Well Clearance (FR)	889 (35.0)
Wheel Well Clearance (RR)	958.85 (37.75)
Frame Height (FR)	438.15 (17.25)
Frame Height (RR)	628.65 (24.75)
Engine Type	8cyl. Gas
Engine Size	4.7L
Transmission Type:	Automatic RWD

Weights kg (lbs)	Curb	Test Inertial	Gross Static
W-front	1293.6 (2852)	1263.3 (2785)	1311.3 (2891)
W-rear	1028.3 (2267)	1007.9 (2222)	1037.8 (2288)
W-total	2321.9 (5119)	2271.1 (5007)	2349.2 (5179)

Front GVWR	3650
Rear GVWR	3900
Total GVWR	6650

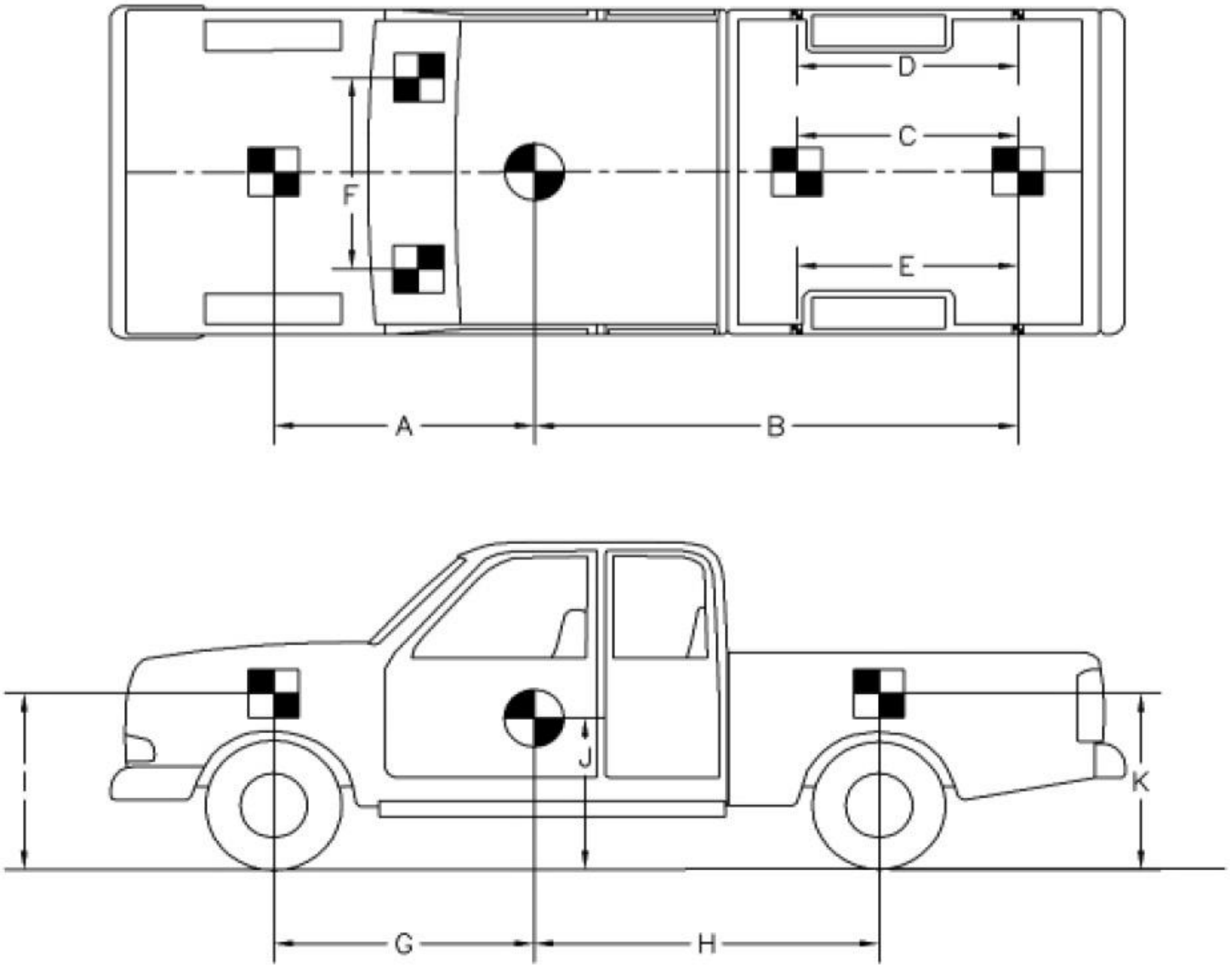
Note any damage prior to test: none

Figure 62. Vehicle Dimensions, Test No. WVBR-1

The Suspension Method [10] was used to determine the vertical component of the center of gravity (c.g.) for the pickup truck. This method is based on the principle that the c.g. 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 c.g. were established. The intersection of these planes pinpointed the location of the c.g. The longitudinal component of the c.g. was determined using the measured axle weights. The locations of the final centers of gravity are shown in Figures 62 and 63.

Square, black and white, checkered targets were placed on the vehicle to aid in the analysis of the high-speed VITcam videos, as shown in Figure 63. Round, checkered targets were placed on the center of gravity, on the left-side door, on the right-side door, and on the roof of the vehicle. The remaining targets were located for reference purposes so that they could be viewed from the high-speed cameras for video analysis.

The front wheels of the test vehicle were aligned for camber, caster, and toe-in values of zero so that the vehicles would track properly along the guide cable. A 5B flash bulb was mounted on the right-side of the vehicle's dash to pinpoint the time of impact with the barrier system on the high-speed VITcam videos. The flash bulb was fired by a pressure tape switch mounted on the front face of the bumper. A remote controlled brake system was installed in the test vehicle so the vehicle could be brought safely to a stop after the test.



TEST #: WVBR-1

TARGET GEOMETRY-- mm (in.)

A	1746	(68.75)	E	965	(38.0)	I	1003	(39.5)
B	2511	(98.875)	F	660	(26.0)	J	714	(28.125)
C	965	(38.0)	G	1568	(61.75)	K	1080	(42.5)
D	965	(38.0)	H	1994	(78.5)			

Figure 63. Target Geometry, Test No. WVBR-1

7.4 Data Acquisition Systems

7.4.1 Accelerometers

Two environmental shock and vibration sensor/recorder systems were used to measure the accelerations in the longitudinal, lateral, and vertical directions. One triaxial, piezoresistive accelerometer system, Model EDR-3, was developed by Instrumented Sensor Technology (IST) of Okemos, Michigan. The EDR-3 was configured with 256 kB of RAM memory, a range of ±200 g's, a sample rate of 3,200 Hz, and a 1,120 Hz lowpass filter. Data from the EDR-3 was analyzed and plotted using "DynaMax 1 (DM-1)", "DADiSP", as well as a customized Microsoft Excel computer software program.

The second system consisted of a two-Arm, piezoresistive accelerometers developed by Endevco of San Juan Capistrano, California. Three accelerometers were used to measure the longitudinal, lateral, and vertical accelerations at a sample rate of 10,000 Hz. Data was collected using a Sensor Input Module (SIM), Model TDAS3-SIM-16M, which was developed by Diversified Technical Systems, Inc. (DTS) of Seal Beach, California. The SIM was configured with 16 MB SRAM memory and 8 sensor input channels with 250 kB SRAM/channel. The SIM was mounted on a TDAS3-R4 module rack. The module rack was configured with isolated power/event/communications, 10BaseT Ethernet and RS232 communication, and an internal back-up battery. Both the SIM and module rack were crashworthy. Computer software programs "DTS TDAS Control", "DADiSP", and a customized Microsoft Excel worksheet were used to analyze and plot the accelerometer data from the DTS unit. All of the accelerometers were mounted near the center of gravity of the test vehicle.

7.4.2 Rate Transducers

An angle rate sensor, the ARS-1500, with a range of 1,500 degrees/sec in each of the three directions was used to measure the rates of rotation of the test vehicles. The angular rate sensor was mounted on an aluminum block inside the test vehicle near the center of gravity and recorded data at 10,000 Hz to the SIM. The raw data measurements were then downloaded, converted to the proper Euler angles for analysis, and plotted. Computer software programs "DTS TDAS Control", "DADiSP", and a customized Microsoft Excel worksheet were used to analyze and plot the angular rate sensor data.

7.4.3 Pressure Tape Switches

For test no. WVBR-1, five pressure-activated tape switches were spaced at 6.56-ft (2-m) intervals to determine the speed of the vehicle before impact. Each tape switch fired a strobe light which sent an electronic timing signal to the data acquisition system as the right-front tire of the test vehicle passed over it. Test vehicle speeds were determined from electronic timing mark data recorded using TestPoint software. Strobe lights and high-speed video analysis are only used as a backup in the event that vehicle speeds cannot be determined from the electronic data.

7.4.4 High-Speed Photography

Four high-speed AOS VITcam digital video cameras, five JVC digital video cameras, and two Canon digital video cameras were utilized to film test no. WVBR-1. Camera details, lens information, and camera operating speeds are shown in Table 3. A schematic of the camera locations is shown in Figure 64. The VITcam videos were analyzed using ImageExpress MotionPlus software. Camera speed and camera divergence factors were considered in the analysis of the high-speed videos.

Table 3. Camera and Lens Information, Test No. WVBR-1

Camera Summary

	No.	Type	Operating Speed (frames/sec)	Lens	Lens Setting
High-Speed Video	2	Vitcam CTM	500	12.5 mm fixed	-
	3	Vitcam CTM	500	sigma 50 mm fixed	-
	4	Vitcam CTM	500	sigma 24-70	24
	5	Vitcam CTM	500	sigma 70-200	100
Digital Video	1	JVC - GZ-MC500 (Everio)	29.97		
	2	JVC - GZ-MC40u (Everio)	29.97		
	3	JVC - GZ-MC500 (Everio)	29.97		
	4	JVC - GZ-MC40u (Everio)	29.97		
	5	JVC - GZ-MC40u (Everio)	29.97		
	7	Canon-ZR90	29.97		
	8	Canon-ZR90	29.97		

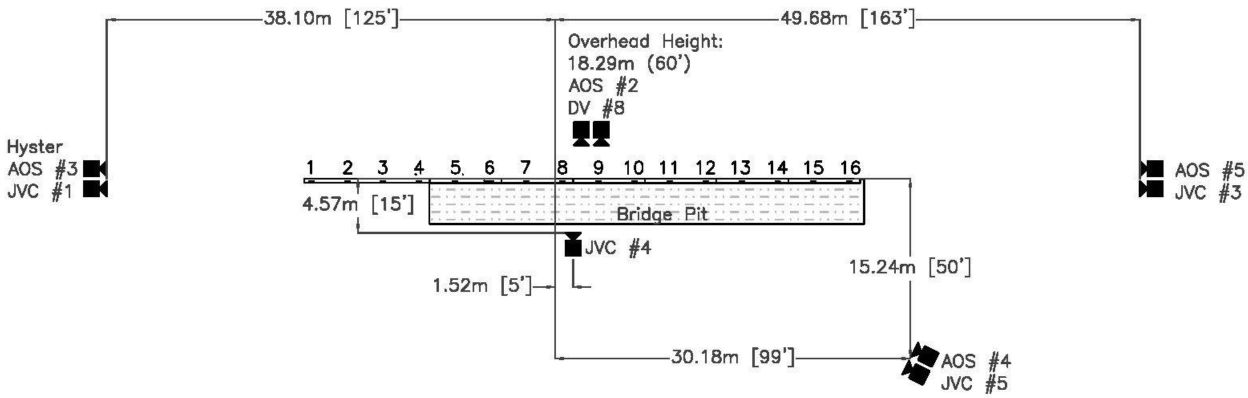


Figure 64. Camera Locations, Test No. WVBR-1

8 FULL-SCALE CRASH TEST NO. WVBR-1

8.1 Test No. WVBR-1

The 5,179-lb (2349-kg) pickup truck impacted the timber bridge rail at a speed of 30.8 mph (49.6 km/h) and at an angle of 26.1 degrees. A summary of the test results and sequential photographs are shown in Figure 65. The summary of the test results in SI units are shown in APPENDIX B. Additional sequential photographs are shown in Figures 66 through 68. Documentary photographs of the crash test are shown in Figure 69.

8.2 Weather Conditions

Test No. WVBR-1 was conducted on July 18, 2008 at approximately 12:45 pm. The weather conditions were reported as shown in Table 4.

Table 4. Weather Conditions, Test No. WVBR-1

Temperature	82° F
Relative Humidity	61%
Wind Speed	7 mph
Wind Direction	10° from True North
Sky Conditions	Partly Cloudy
Visibility	10 Statue Miles
Pavement Surface	Dry
Previous 3-Day Precipitation	2.67 in.
Previous 7-Day Precipitation	2.99 in.

8.3 Test Description

Initial impact was to occur 60 in. (1,524 mm) upstream from the centerline of the H-splice plate located between post nos. 4 and 5, as shown in Figure 70. Actual vehicle impact occurred 66.4 in. (1,687 mm) upstream from the centerline of the H-splice plate located between post nos. 4 and 5. At 0.002 seconds after impact, the right-front bumper corner deflected inward. At 0.03 sec, the right-front wheel impacted the barrier at the impact point. At 0.034 sec, the rail began to deflect backward near the impact point. At 0.042 sec, the right-front wheel impacted the rail upstream from post no. 4. At this same time, the curb rail located downstream from the impact point also began to deflect backward. At 0.064 sec, the vehicle began to roll into the barrier during redirection. At 0.134 sec, the vehicle continued to roll toward the barrier and began yawing away from the barrier. At 0.110 sec, the right-front tire lost contact with the ground. At 0.154 sec, the left-front tire became airborne, and the rail reached its maximum lateral deflection of 6.1 in. (155 mm). At 0.222 sec, the left-rear tire became airborne. At 0.36 sec, the vehicle became parallel to the barrier. At 0.370 sec, the right-front tire lost contact with the rail. At 0.388 sec, the vehicle's front end pitched downward. At 0.398 sec, the right-front tire exited the system. At 0.414 sec, the right-front tire contacted the ground, and the wheel assembly detached from the vehicle. At 0.444 sec, the vehicle reached a maximum roll angle of 8.7 degrees and started to roll away from the barrier, or toward the driver's side. At 0.464 sec, the right-rear corner of the bumper impacted the rail. At 0.470 sec, the right-rear tire impacted the rail. At 0.490 sec, the right-rear corner of the bumper lost contact with the top of the bridge rail. At 0.512 sec, the left-front tire contacted the ground. At 0.576 sec, the vehicle's front end began yawing toward the barrier. At 0.686 sec, the right-rear tire lost contact with the rail, causing the vehicle to exit the system. At this same time, the vehicle's front end began to pitch upward. At 0.698 sec, the vehicle ceased to roll. At 0.730 sec, the left-rear tire contacted the ground. At 0.836 sec, the right-rear tire contacted the ground. At 0.970 sec, the vehicle continued yawing toward the barrier and stayed on its trajectory. At 1.592 sec, the vehicle was parallel to the system again. The vehicle came to rest 122 ft (37.2 m) downstream from impact and 10 ft (3.1 m) laterally behind a line projected parallel to the traffic-side face of the bridge rail system. The trajectory and final vehicle position are shown in Figure 71.

8.4 Barrier Damage

The bridge rail suffered minor damage, as shown in Figures 72 through 75. No visible railing damage was observed upstream of the impact location. Tire and scuff marks were found on the rail from the impact point to 1 in. (25 mm) upstream from post no. 5. A 1 ½-in. (38-mm) deep by 4-in. (102-mm) long gouge was observed on the front face of the rail just upstream of post no. 4, as shown in Figure 72. Also, scrape marks were found on the rail starting at post no. 4 and extending 7 ¾ ft (2.4 m) downstream. A ½-in. (13-mm) deep gouge was found on the face of the rail 2 in. (51 mm) downstream from the centerline of post no. 4.

At the top of post nos. 4 and 5, several bolt heads were slightly pulled into the rail during impact. Also, a slight kink developed in the front face of the H-splice plate located between post nos. 4 and 5, as shown in Figure 74. Due to the permanent set deflection of the rail and rotation of the posts, a ⅜-in. (10-mm) gap formed between the concrete wearing surface and post no. 4 and the upstream end of post no. 5, as shown in Figure 75.

No visible damage occurred to the railing system between post nos. 5 through 11. No top-side or under-side deck damage was observed near the vertical bolt holes. Tire and scuff marks were found on the last 8 ft - 8 in. (2.64 m) of the railing system, as shown in Figure 73.

The permanent set deflection of the barrier system is shown in Figures 72 and 73. Post nos. 4 and 5 were deflected laterally backward, while the other posts had no permanent set deflections. The maximum lateral permanent post deflection was 2.4 in. (61 mm) at post no. 4. A maximum dynamic deflection of 6.1 in. (155 mm) was observed at the upstream end of the splice between post nos. 4 and 5. The working width for the system was 12.4 in. (314 mm).

8.5 Vehicle Damage

The damage to the vehicle was minor, as shown as shown in Figures 76 and 77. Damage was mostly concentrated to the right-front corner of the vehicle. The right-side bumper was scraped and deformed inward toward the engine compartment. The right-side headlight was loosened from its original mounting position. The right-side fog light detached and broke into several pieces. Scratches and scuffs were found on the right-rear tire and occurred due to the impact with the glulam rail, as evidenced by the wood fibers lodged between the tire and rim. Small dents were observed in the sheet metal behind the right-rear tire and the tailpipe. Scratches and denting was observed on the right-side panel, starting behind the right-front wheel and extending 18 in. (457 mm) onto the passenger-side door. The right-front tire detached from the vehicle due to the impact with the glulam rail, as evidenced by the wood fibers lodged between the tire and rim. Multiple tears were found in the sidewall of the right-front tire. The upper and lower ball joints on the right-front side of the vehicle were broken. Brake fluid was observed leaking on the right-front side of the vehicle. The plastic inner liner under the right-front wheel well moved downward 2 in. (51 mm) and broke near the lower-rear corner. The rear-view mirror was partially detached and hanging. All window glass remained undamaged.

Occupant compartment deformations to the right side and center of the floorboard were judged insufficient to cause serious injury to the vehicle occupants. Maximum longitudinal deformations of 1 ¼ in. (32 mm) were located near the center front of the right-side floor pan. Maximum lateral deflections of ½ in. (13 mm) were located near the center front of the right-side floor pan and the left center of right-side floor pan. Maximum vertical deflections of 1 in. (25 mm) were located near the center front of the right-side floor pan. Complete occupant compartment deformations and the corresponding locations are provided in APPENDIX C.

8.6 Occupant Risk

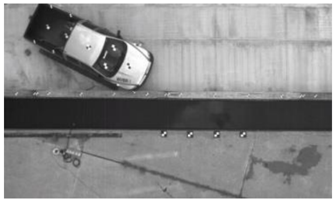
The occupant impact velocities and 0.010-sec average occupant ridedown accelerations are summarized in Table 5. It is noted that the occupant impact velocities (OIVs) and occupant ridedown accelerations (ORAs) were within the suggested limits provided in MASH-08. The results of the occupant risk, as determined from the accelerometer data, are also summarized in Figure 65. The recorded data from both the accelerometers and the rate transducer are shown graphically in APPENDIX D.

Table 5. Summary of OIV, ORA, THIV, and PHD Values, Test No. WVBR-1

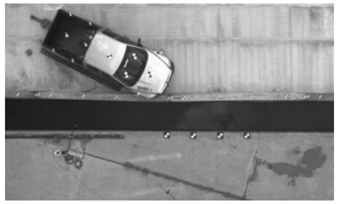
Evaluation Criteria		Transducer	
		EDR-3	**DTS**
OIV ft/s (m/s)	Longitudinal	-11.81 (-3.60)	-10.95 (-3.34)
	Lateral	-12.61 (-3.84)	-13.00 (-3.96)
ORA g's	Longitudinal	-5.33	-4.74
	Lateral	-2.69	-3.23
THIV ft/s (m/s)		NA	15.72 (4.79)
PHD g's		NA	5.25

8.7 Discussion

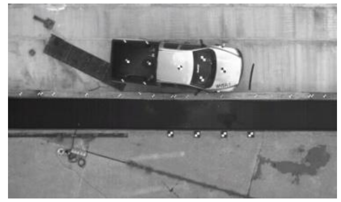
The analysis of the test results for test no. WVBR-1 showed that the low-height, curb-type, bridge railing adequately contained and redirected the ½-ton, Dodge Ram Quad Cab pickup truck with controlled lateral displacements of the barrier system. No detached elements or fragments showed the potential for penetrating the occupant compartment or presented undue hazard to other traffic. Deformations of, or intrusion into, the occupant compartment that could have caused serious injury did not occur. The test vehicle did not penetrate or ride over the low-height, curb-type, bridge railing, and it remained upright during and after collision. Vehicle roll, pitch, and yaw angular displacements were deemed acceptable. After collision, the vehicle's trajectory revealed only minimum intrusion into adjacent traffic lanes. Therefore, test no. WVBR-1 was determined to be acceptable according to the TL-1 safety performance criteria found in MASH-08. A summary of the safety performance evaluation is provided in Table 6.



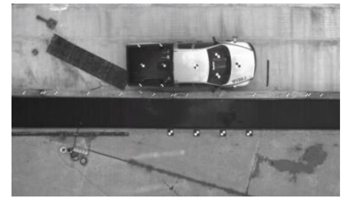
0.000 sec



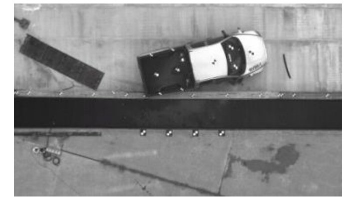
0.152 sec



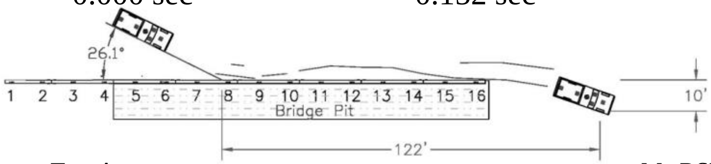
0.350 sec

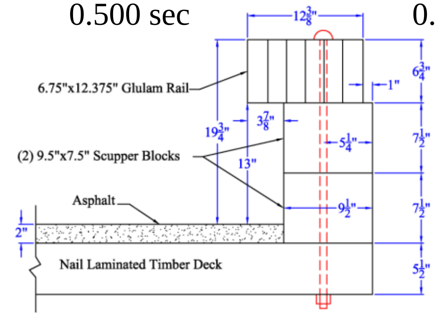


0.500 sec



0.768 sec





- Test Agency .... MwRSF
- Test Number WVBR-1
- Date 7/18/08
- MASH-08 Test Designation 1-11
- Test Article .... Low-Profile, Curb-Type, Timber Bridge Rail
- Total Length .... 155 ft
- Scupper Block or Post Spacing .... 10 ft
- Key Component - Glulam Bridge Rail Segment
  - Length .... 20 ft
  - Width .... 6 ¾ in.
  - Depth .... 12 ⅜ in.
- Key Component - Scupper Block
  - Length .... 23 in.
  - Width .... 7 ½ in.
  - Depth .... 9 ½ in.
- Vehicle Model .... 2002 Dodge Ram 1500 Quad Cab
  - Curb .... 5,119 lbs
  - Test Inertial .... 5,007 lbs
  - Gross Static .... 5,179 lbs
- Impact Conditions
  - Speed .... 30.8 mph
  - Angle .... 26.1 deg
  - Impact Location .... 30 ½ in. Upstream from Post No. 4
- Exit Conditions
  - Speed .... 19.1 mph
  - Angle .... 12.6 deg
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 120 ft Downstream from Impact
- Occupant Ridedown Acceleration (DTS)
  - Longitudinal .... -4.74 g's < 20.49 g's
  - Lateral .... -3.23 g's < 20.49 g's
- Occupant Impact Velocity (DTS)
  - Longitudinal .... -10.95 ft/s < 40.0 ft/s
  - Lateral .... -13.00 ft/s < 40.0 ft/s
- Occupant Ridedown Acceleration (EDR-3)
  - Longitudinal .... -5.33 g's < 20.49 g's
  - Lateral .... -2.69 g's < 20.49 g's
- Occupant Impact Velocity (EDR-3)
  - Longitudinal .... -11.81 ft/s < 40.0 ft/s
  - Lateral .... -12.61 ft/s < 40.0 ft/s
- Maximum Roll Angle .... 8.7 degrees
- THIV .... 15.72 ft/s < 39.4 ft/s
- PHD .... 5.25 g's < 20 g's
- Vehicle Damage .... Minimal
  - TAD[11] 1-RFQ-3
  - SAE[12] 01-RFLW1
  - Maximum Deformation .... 1 ¼ in. Near Front Floor Pan
- Test Article Damage .... Minimal
- Maximum Rail Deflections
  - Permanent Set .... 2.4 in.
  - Dynamic .... 6.1 in.
- Working Width .... 12.4 in.

Figure 65. Summary of Test Results and Sequential Photographs, Test No. WVBR-1


0.000 sec


0.030 sec


0.096 sec


0.242 sec


0.388 sec


0.470 sec


0.596 sec


0.820 sec

Figure 66. Additional Sequential Photographs, Test No. WVBR-1


0.000 sec


0.504 sec


0.074 sec


0.590 sec


0.166 sec


0.814 sec


0.390 sec


0.994 sec

Figure 67. Additional Sequential Photographs, Test No. WVBR-1


0.000 sec


0.068 sec


0.154 sec


0.360 sec


0.464 sec


0.686 sec


0.836 sec


1.028 sec

Figure 68. Additional Sequential Photographs, Test No. WVBR-1

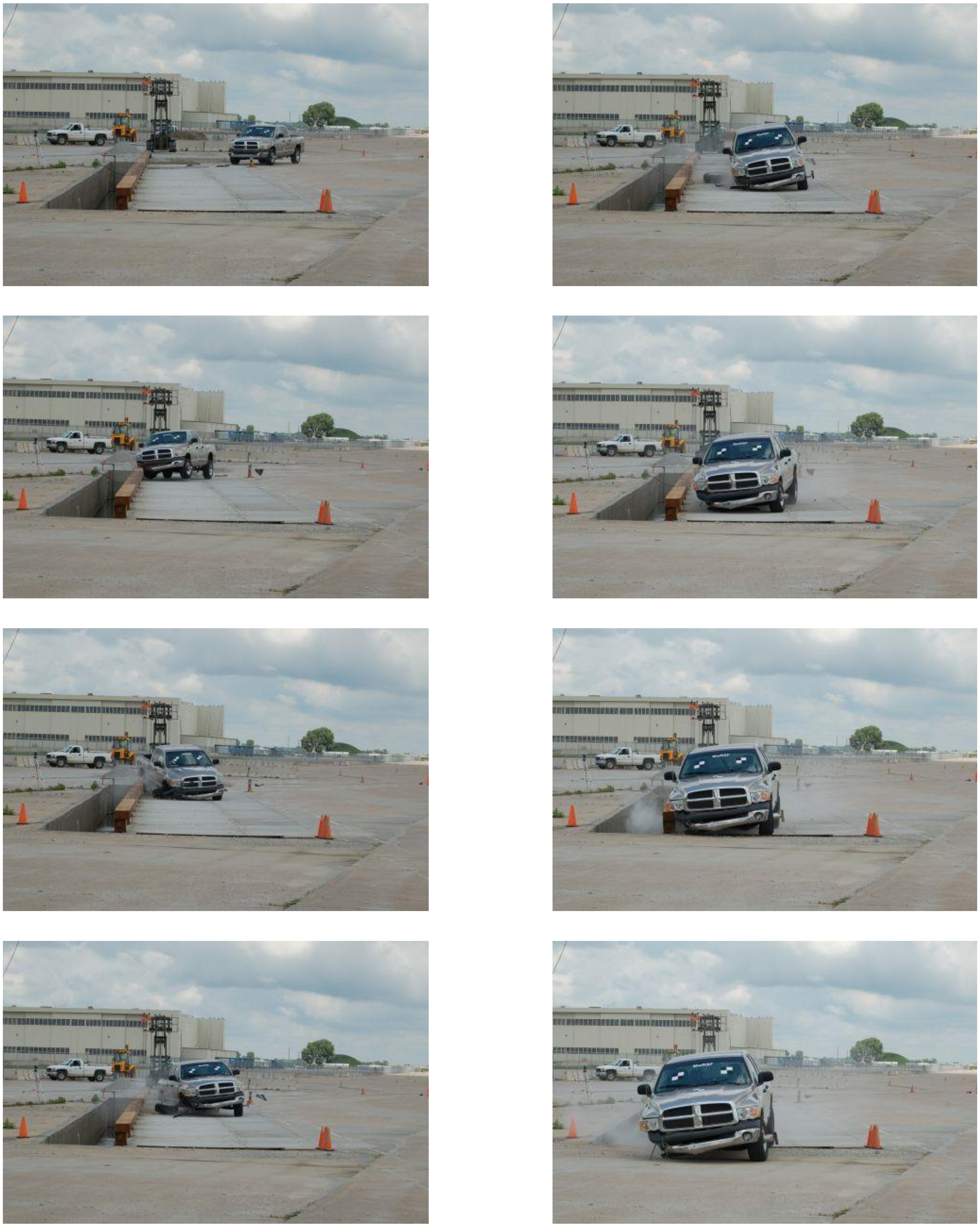


Figure 69. Documentary Photographs, Test No. WVBR-1

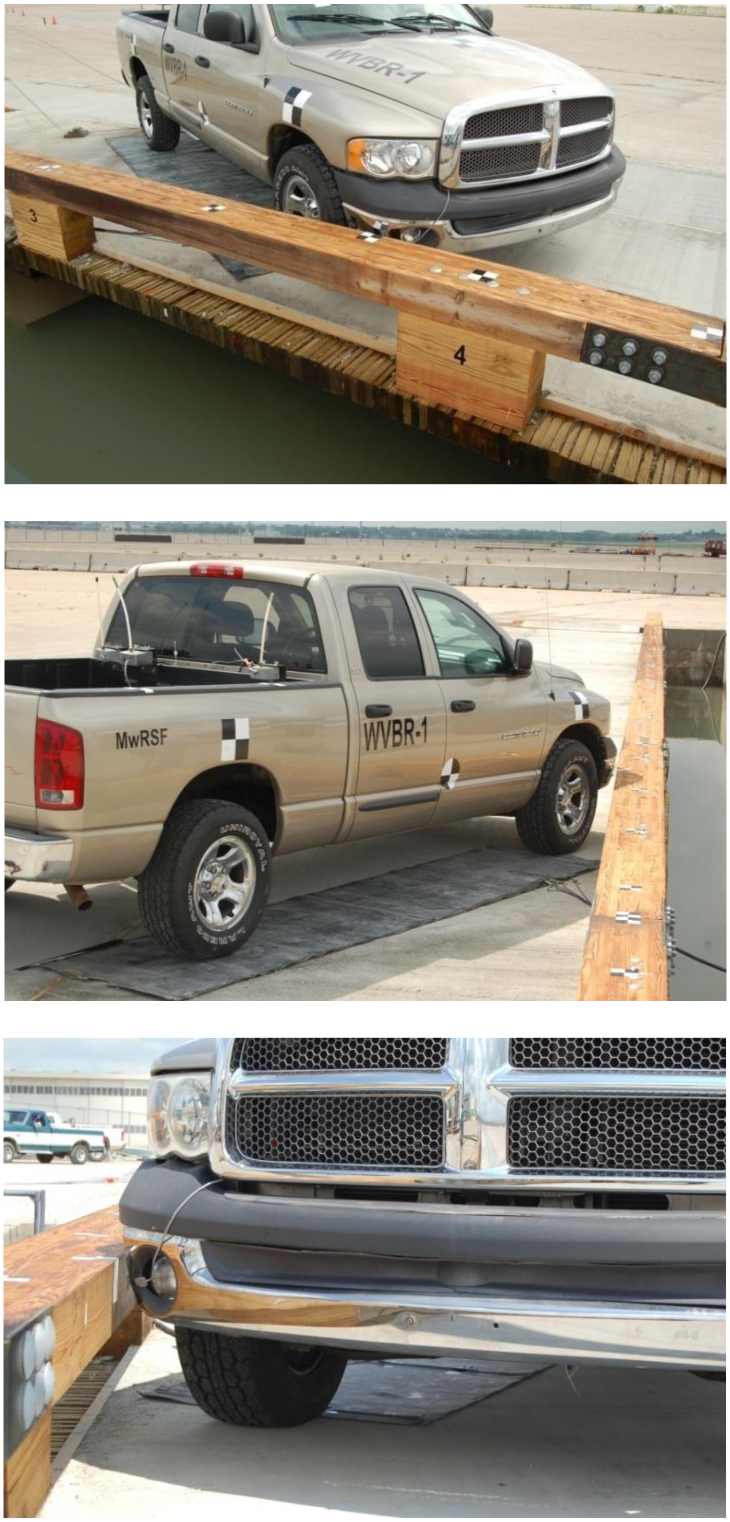


Figure 70. Target Impact Location, Test No. WVBR-1

Figure 71. Vehicle Final Position and Trajectory Marks, Test No. WVBR-1

Figure 72. System Damage, Test No. WVBR-1

Figure 73. Rail Damage and Displacement, Test No. WVBR-1

Figure 74. System Damage at Joint Between Post Nos. 4 and 5, Test No. WVBR-1

Figure 75. Concrete Gaps at Post Nos. 4 and 5, Test No. WVBR-1

Figure 76. Vehicle Damage, Test No. WVBR-1

Figure 77. Vehicle Damage at Right-Front Corner, Test No. WVBR-1

9 SUMMARY, CONCLUSIONS, AND RECCOMENDATIONS

For this study, an existing, NCHRP Report No. 350 Test Level 1, curb-type, timber, bridge railing system was modified for use on transverse, nail-laminated, timber bridges. The modified bridge railing was required to meet the new TL-1 impact safety standards found in the Update to NCHRP Report No. 350, now referred to as the MASH-08 guidelines.

Due to the larger vehicle size and the anticipated increase in lateral impact loading, several changes were made to the original bridge railing. A larger glulam timber rail was chosen in order to provide increased strength. The height of the lower scupper block was increased by 2 in. (51 mm) to match that of the upper scupper block, thus resulting in a top railing height of 19 ¾ in. (502 mm) as compared to 17 ¾ in (451 mm) for the original railing system. Also, the steel rail splices were strengthened by both increasing the plate thickness and welding an orthogonal plate between front and back plates in order to create an H-splice shape.

For the transverse, nail-laminated, timber deck, a new nailing pattern was developed for strengthening the end region of the deck boards, including the application of a construction adhesive. In addition, a material specification was selected for the steel anchor brackets used to attach the dimensional lumber boards to the flanges of the bridge girders. Several alternatives were developed for the brackets, including steel thickness, material specification, and galvanization method.

Five static tests were performed in order to evaluate the effectiveness of shear connectors placed at the interfaces between the deck, scupper blocks, and rail segments. Different types and quantities of shear connectors were examined. From the static testing program, the research team determined that shear connectors: (1) provided no or only limited increased lateral strength for the post assemblies; (2) correlated with an increased amount of deck damage; and (3) increased the labor and materials costs for the railing system. Therefore, it was concluded that no shear connectors would be specified for the timber bridge railing system.

One full-scale vehicle crash test, test no. WVBR-1, was conducted on the new bridge railing system attached to a transverse, nail-laminated, timber deck according to the TL-1 test conditions found in MASH-08. The barrier system successfully contained and redirected the 2270P pickup truck with only minimal damage to the rail and no visual damage to the deck. Therefore, the bridge railing system was determined to be acceptable according to the TL-1 safety performance criteria presented in MASH-08. A summary of the safety performance evaluation is provided in Table 6.

For the static and full-scale crash testing programs, all of the timber components were fabricated from Southern Yellow Pine (SYP) material. However, MWRSF researchers are confident that successful barrier performance would also have been obtained had the curb-type, timber, bridge railing system been fabricated using Douglas Fir (DF) material. In fact, the sawn lumber scupper blocks and glulam rail segments manufactured from Douglas Fir have higher nominal strengths than the comparable Southern Yellow Pine blocks and rails. Thus, the Douglas Fir, curb-type, timber, bridge railing would provide equivalent or greater capacity than that provided by a comparable Southern Yellow Pine system. It should be noted that the SYP and DF wood specifications and component dimensions are described in the system drawings presented in Chapter 5.

A rigid, end treatment was also developed for the curb-type, timber, bridge railing system in order to prevent blunt impacts into the end of the low-height railing. For this system, the geometry for the end treatment was adopted from a prior TL-2 end treatment used with a low-profile, concrete barrier. Thus, the upstream end of the timber bridge rail was sloped downward to the ground with the same geometry as used with the approved rigid, concrete end treatment after the railing extended off of the bridge. The end rail segments, including the sloped section, were mounted to steel W6x15 (W152x22.3) posts in order to provide a structurally-adequate foundation for the end treatment.

For the bridge railing system, steel H-splice brackets connected the rail ends to one another in order to transfer tension, shear, and moment across the joints. The H-splice brackets were to be fabricated from three welded, ASTM A572 Grade 42 [σ_y=42 ksi, σ_u=60 ksi] steel plates. As previously noted, the steel plates were not ordered with much advance notice prior to the scheduling of the full-scale crash test. As such, MwRSF experienced difficulty in finding the original plate material on such short notice, thus requiring an adjustment to the design. Instead, the steel plates were manufactured from ASTM A656 Grade 50 Type 7 [σ_y=50 ksi, σ_u=60 ksi] material. In the original development of TL-1 curb-type rail according to NCHRP Report No. 350, the steel splice plates were fabricated with 3/16-in. (4.8-mm) thick, ASTM A572 Grade 42 [σ_y=42 ksi, σ_u=60 ksi] material. For this project and using the MASH-08 guidelines, the steel splice plates were modified to use 3/8-in. (9.5-mm) thick material. Due to the increased plate thickness and satisfactory test results, MwRSF researchers believe that comparable safety performance would have been provided had the steel H-splice brackets been fabricated from ASTM A572 Grade 42 [σ_y=42 ksi, σ_u=60 ksi] or ASTM A572 Grade 50 [σ_y=50 ksi, σ_u=65 ksi] material.

Table 6. Summary of MASH-08 Safety Performance Evaluation Results (Test No. 1-11)

<table>
<tr><th>Evaluation Factors</th><th colspan="4">Evaluation Criteria</th><th>Test No. WVBR-1</th></tr>
<tr><td>Structural Adequacy</td><td colspan="4">A. Test article should contain and redirect the vehicle or bring the vehicle to a controlled stop; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.</td><td>S</td></tr>
<tr><td rowspan="11">Occupant Risk</td><td colspan="4">D. Detached elements, fragments or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformations of, or intrusions into, the occupant compartment should not exceed limits set forth in Section 5.3 and Appendix E of MASH-08.</td><td>S</td></tr>
<tr><td colspan="4">F. The vehicle should remain upright during and after collision. The maximum roll and pitch angles are not to exceed 75 degrees.</td><td>S</td></tr>
<tr><td colspan="4">H. Occupant Impact Velocities (OIV) (see Appendix A, Section A5.3 of MASH-08 for calculation procedure) should satisfy the following limits:</td><td rowspan="4">S</td></tr>
<tr><td rowspan="3"></td><td colspan="3">Occupant Impact Velocity Limits, ft/s (m/s)</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>30 ft/s (9.1 m/s)</td><td>40 ft/s (12.2 m/s)</td></tr>
<tr><td colspan="4">I. The Occupant Ridedown Acceleration (ORA) (see Appendix A, Section A5.3 of MASH-08 for calculation procedure) should satisfy the following limits:</td><td rowspan="4">S</td></tr>
<tr><td rowspan="3"></td><td colspan="3">Occupant Ridedown Acceleration Limits (g’s)</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>15.0 g’s</td><td>20.49 g’s</td></tr>
</table>

S – Satisfactory U – Unsatisfactory
M - Marginal NA - Not Available

10 REFERENCES

1. Ritter, M.A., Faller, R.K., Sicking, D.L., and Bunnell, S., *Development of Low-Volume Curb-Type Bridge Railings for Timber Bridge Decks*, Draft MwRSF Research Report No. TRP-03-31-93, Prepared for the Forest Products Laboratory, USDA-Forest Service, Performed by the Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, December 1993.

2. Faller, R.K., Rosson, B.T., Sicking, D.L., Ritter, M.A., and Bunnell, S., *Design and Evaluation of Two Bridge Railings for Low-Volume Roads*, Sixth International Conference on Low-Volume Roads - Volume 2, Conference Proceedings 6, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C., 357-372, 1995.

3. Faller, R.K., Rosson, B.T., Ritter, M.A., Lee, P.D.H., and Duwadi, S.R., *Railing Systems for Longitudinal Timber Deck Bridges*, National Conference on Wood Transportation Structures, Madison, Wisconsin, October 23-25, 1996, General Technical Report No. FPL-GTR-94, United States Department of Agriculture - Forest Service - Forest Products Laboratory and Federal Highway Administration, October 1996.

4. Faller, R.K., Soyland, K., Rosson, B.T., and Stutzman, T.M., TL-1 Curb-Type Bridge Railing for Longitudinal Glulam Timber Decks Located on Low-Volume Roads, Transportation Research Report No. TRP-03-54-96, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, April 1996.

5. Ross, H.E., Sicking, D.L., Zimmer, R.A., and Michie, J.D., *Recommended Procedures for the Safety Performance Evaluation of Highway Features*, National Cooperative Highway Research Program (NCHRP) Report No. 350, Transportation Research Board, Washington, D.C., 1993.

6. Sicking, D.L., Mak, K.K., Rohde, J.R., and Reid, J.D., *Recommended Procedures for the Safety Performance Evaluation of Highway Features*, *Draft Report*, Submitted to National Cooperative Highway Research Program, Project 22-14(2) Panel, Prepared by the Midwest Roadside Safety Facility, University of Nebraska-Lincoln, January 2008.

7. Beason, W.L., Menges, W.L., and Ivey, D.L., *Compliance Testing of an End Treatment for the Low-profile Concrete Barrier*, Research Report No. 1403-S, Texas Transportation Institute, Texas A&M University, College Station, Texas, April 1998.

8. American Wood Preserver's Association (AWPA) *U1 – Use Category System: User Specification for Treated Wood*, Birmingham, AL. 2006.

9. Hinch, J., Yang, T.L., and Owings, R., *Guidance Systems for Vehicle Testing*, ENSCO, Inc., Springfield, VA, 1986.

10. *Center of Gravity Test Code - SAE J874 March 1981*, SAE Handbook Vol. 4, Society of Automotive Engineers, Inc., Warrendale, Pennsylvania, 1986.

11. *Vehicle Damage Scale for Traffic Investigators*, Second Edition, Technical Bulletin No. 1, Traffic Accident Data (TAD) Project, National Safety Council, Chicago, Illinois, 1971.

12. *Collision Deformation Classification – Recommended Practice J224 March 1980*, Handbook Volume 4, Society of Automotive Engineers (SAE), Warrendale, Pennsylvania, 1985.

11 APPENDICES

APPENDIX A. Vehicle Center of Gravity Determination

Figure A-1. Vehicle Mass Distribution, Test No. WVBR-1

	WVBR-1		Vehicle:	2002 Dodge Ram 1500QC		
			Vehicle CG Determination			
VEHICLE	Equipment	Weight	Long CG	Vert CG	HOR M	Vert M
+	Unbalasted Truck	5119	61.75	28.2	316098.3	144355.8
+	Brake receivers/wires	5	116	51	580	255
+	Brake Frame	5	34	31	170	155
+	Brake Cylinder	22	74	29	1628	638
+	Strobe Battery	6	74	30	444	180
+	Hub	27	0	14.875	0	401.625
+	CG Plate (EDRs)	8	54	32	432	256
-	Battery	-42	-7	45	294	-1890
-	Oil	-9	8	19	-72	-171
-	Interior	-62	44	24	-2728	-1488
-	Fuel	-161	111	20	-17871	-3220
-	Coolant	-21	-18	35	378	-735
-	Washer fluid	-2	-15	35	30	-70
BALLAST	Water	85	111	20	9435	1700
	Misc. (DTS equip)	20	74	27	1480	540
	Misc.		0	0	0	0
					310298.3	140907.4
	TOTAL WEIGHT	5000			62.05965	28.18149

wheel base 140.25

NCHRP 350 Targets		CURRENT	Difference
Test Inertial Weight	5000	5000	0.0
Long CG	62	62.06	0.05965
Vert CG	28	28.18	0.18148

Note, Long. CG is measured from front axle of test vehicle

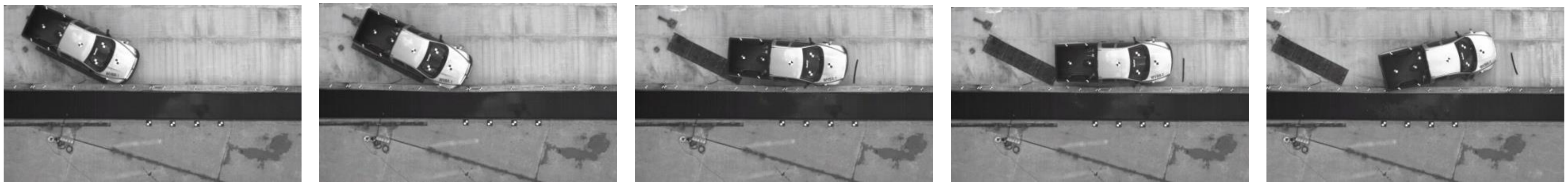
Curb Weight		
	Left	Right
Front	1477	1375
Rear	1116	1151
FRONT	2852	
REAR	2267	
TOTAL	5119	

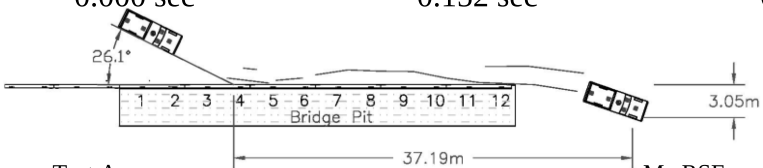
Actual test inertial weight		
	Left	Right
Front	1413	1373
Rear	1082	1140
FRONT	2786	
REAR	2222	
TOTAL	5008	

Figure A-1. Vehicle Mass Distribution, Test No. WVBR-1

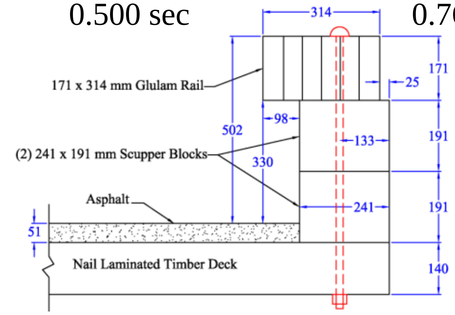
APPENDIX B. Test Summaries and Sequential Photographs (SI)

Figure B-1. Summary of Test Results and Sequential Photographs (SI), Test No. WVBR-1


0.000 sec 0.152 sec 0.350 sec 0.500 sec 0.768 sec



- Test Agency .... MwRSF
- Test Number WVBR-1
- Date 7/18/08
- MASH 08 Test Designation 1-11
- Test Article .... Low-Profile, Curb-Type, Timber Bridge Rail
- Total Length .... 47.2 m
- Scupper Post Spacing .... 3.05 m
- Key Component - Glulam Bridge Rail Segment
  - Length .... 6.1 m
  - Width .... 171 mm
  - Depth .... 314 mm
- Key Component - Scupper Block
  - Length .... 584 mm
  - Width .... 191 mm
  - Depth .... 241 mm
- Vehicle Model .... 2002 Dodge Ram 1500 Quad Cab
  - Curb .... 2,322 kg
  - Test Inertial .... 2,271 kg
  - Gross Static .... 2,349 kg
- Impact Conditions
  - Speed .... 48.6 km/h
  - Angle .... 26.1 deg
  - Impact Location .... 775 mm Upstream from Post No. 4
- Exit Conditions
  - Speed .... 30.7 km/h
  - Angle .... 12.6 deg
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 37.2 m Downstream from of Impact



- Occupant Ridedown Acceleration (DTS)
  - Longitudinal .... -4.74 g's < 20.49 g's
  - Lateral .... -3.23 g's < 20.49 g's
- Occupant Impact Velocity (DTS)
  - Longitudinal .... -3.34 m/s < 12.2 m/s
  - Lateral .... -3.96 m/s < 12.2 m/s
- Occupant Ridedown Acceleration (EDR-3)
  - Longitudinal .... -5.33 g's < 20.49 g's
  - Lateral .... -2.69 g's < 20.49 g's
- Occupant Impact Velocity (EDR-3)
  - Longitudinal .... -3.60 m/s < 12.2 m/s
  - Lateral .... -3.84 m/s < 12.2 m/s
- Maximum Roll Angle .... 8.7 degrees
- THIV .... 4.79 m/s < 12.2 m/s
- PHD .... 5.25 g's < 20 g's
- Vehicle Damage .... Minimal
  - TAD[11] 1-RFQ-3
  - SAE[12] 01-RFLW1
  - Maximum Deformation .... 32 mm Near Front Floor Pan
- Test Article Damage .... Minimal
- Maximum Rail Deflections
  - Permanent Set .... 61 mm
  - Dynamic .... 155 mm
- Working Width .... 315 mm

Figure B-1. Summary of Test Results and Sequential Photographs (SI), Test No. WVBR-1

APPENDIX C. Occupant Compartment Deformation Data

Figure C-1. Occupant Compartment Deformation Data – Set 1, Test No. WVBR-1

Figure C-2. Occupant Compartment Deformation Data – Set 2, Test No. WVBR-1

Figure C-3. Occupant Compartment Deformation Index (OCDI), Test No. WVBR-1

VEHICLE PRE/POST CRUSH INFO
Set-1

TEST: WVBR-1
VEHICLE: 2002 Dodge Ram

Note: If impact is on driver side need to enter negative number for Y

POINT	X	Y	Z	X'	Y'	Z'	DEL X	DEL Y	DEL Z
1	27	12	0	27	12	0	0	0	0
2	30	17.75	-1.25	30	17.25	-1.25	0	-0.5	0
3	31.75	23.75	-1.5	30.5	23.5	-0.5	-1.25	-0.25	1
4	29.5	29.75	-0.25	29.25	29.5	0	-0.25	-0.25	0.25
5	23.5	9.5	0.25	23.75	9.5	0	0.25	0	-0.25
6	25.25	17	-3	25.5	16.75	-3	0.25	-0.25	0
7	26.5	22.5	-6	26.25	22.25	-5.75	-0.25	-0.25	0.25
8	26	30.25	-5.5	26.25	30.5	-5.5	0.25	0.25	0
9	14.25	0.5	-2.5	14.25	0.5	-2.5	0	0	0
10	16.75	7.5	-2.25	16.75	7.5	-2.25	0	0	0
11	18.5	13	-5.5	18.5	13	-5.5	0	0	0
12	20.5	21.5	-9	20.25	22	-9.25	-0.25	0.5	-0.25
13	20.75	30	-9	20.75	30.25	-8.75	0	0.25	0.25
14	10.25	0	-3	10.25	0	-3	0	0	0
15	12	7.75	-3	12	7.5	-2.75	0	-0.25	0.25
16	16.5	16	-9.5	16.25	16	-9.25	-0.25	0	0.25
17	16.5	23.25	-9	16.5	23.25	-8.75	0	0	0.25
18	17	31	-9.5	17	31	-9.25	0	0	0.25
19	5.5	0.75	-3	5.5	0.75	-3	0	0	0
20	6	8	-3.5	6	7.75	-3.25	0	-0.25	0.25
21	10.75	16	-9	10.75	16.25	-8.75	0	0.25	0.25
22	11	23.75	-8.75	11	23.5	-8.25	0	-0.25	0.5
23	10.5	30.25	-8.5	10.5	30.25	-8	0	0	0.5
24	1.25	0.5	-2.75	1.25	0.5	-2.5	0	0	0.25
25	1.25	6.75	-2.5	1.5	6.75	-2.5	0.25	0	0
26	0.75	13	-5	0.75	13.25	-4.75	0	0.25	0.25
27	1	21.5	-4.5	1	21.75	-4.5	0	0.25	0
28	1	28.5	-4.25	1	28.5	-4	0	0	0.25
29							0	0	0

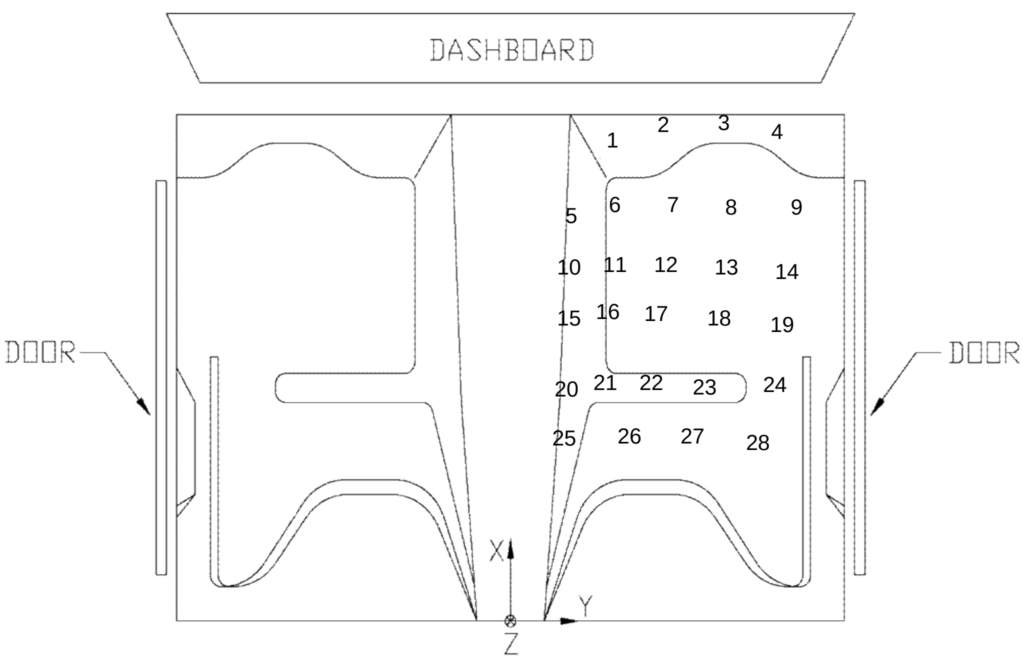


Figure C-1. Occupant Compartment Deformation Data – Set 1, Test No. WVBR-1

VEHICLE PRE/POST CRUSH INFO
Set-2

TEST: WVBR-1
VEHICLE: 2002 Dodge Ram

Note: If impact is on driver side need to enter negative number for Y

POINT	X	Y	Z	X'	Y'	Z'	DEL X	DEL Y	DEL Z
1	50	14.75	0	50	14.75	0	0	0	0
2	53	20.5	-1.25	53	20	-1.25	0	-0.5	0
3	54.75	26.5	-1.5	53.5	26.25	-0.5	-1.25	-0.25	1
4	52.5	32.5	0	52.25	32.25	0	-0.25	-0.25	0
5	46.5	12.25	0	46.75	12.25	0	0.25	0	0
6	48.25	19.75	-3	48.5	19.5	-3	0.25	-0.25	0
7	49.5	25.25	-5.75	49.25	25	-5.75	-0.25	-0.25	0
8	49	33	-5.25	49.25	33.25	-5.5	0.25	0.25	-0.25
9	37.25	3.25	-3	37.25	3.25	-3	0	0	0
10	39.75	10.25	-2.5	39.75	10.25	-2.5	0	0	0
11	41.5	15.75	-5.75	41.5	15.75	-5.75	0	0	0
12	43.5	24.25	-9.25	43.25	24.75	-9.25	-0.25	0.5	0
13	43.75	32.75	-8.75	43.75	33	-9	0	0.25	-0.25
14	33.25	2.75	-3.5	33.25	2.75	-3.5	0	0	0
15	35	10.5	-3.5	35	10.25	-3.25	0	-0.25	0.25
16	39.5	18.75	-9.5	39.25	18.75	-9.75	-0.25	0	-0.25
17	39.5	26	-9.25	39.5	26	-9.25	0	0	0
18	40	33.75	-9.5	40	33.75	-9.75	0	0	-0.25
19	28.5	3.5	-3.5	28.5	3.5	-3.5	0	0	0
20	29	10.75	-4	29	10.5	-4	0	-0.25	0
21	33.75	18.75	-9.5	33.75	19	-9.5	0	0.25	0
22	34	26.5	-9	34	26.25	-9.25	0	-0.25	-0.25
23	33.5	33	-8.75	33.5	33	-8.75	0	0	0
24	24.25	3.25	-3.25	24.25	3.25	-3.25	0	0	0
25	24.25	9.5	-3.25	24.5	9.5	-3.25	0.25	0	0
26	23.75	15.75	-5.5	23.75	16	-5.75	0	0.25	-0.25
27	24	24.25	-5.25	24	24.5	-5.25	0	0.25	0
28	24	31.25	-5	24	31.25	-5	0	0	0
29							0	0	0

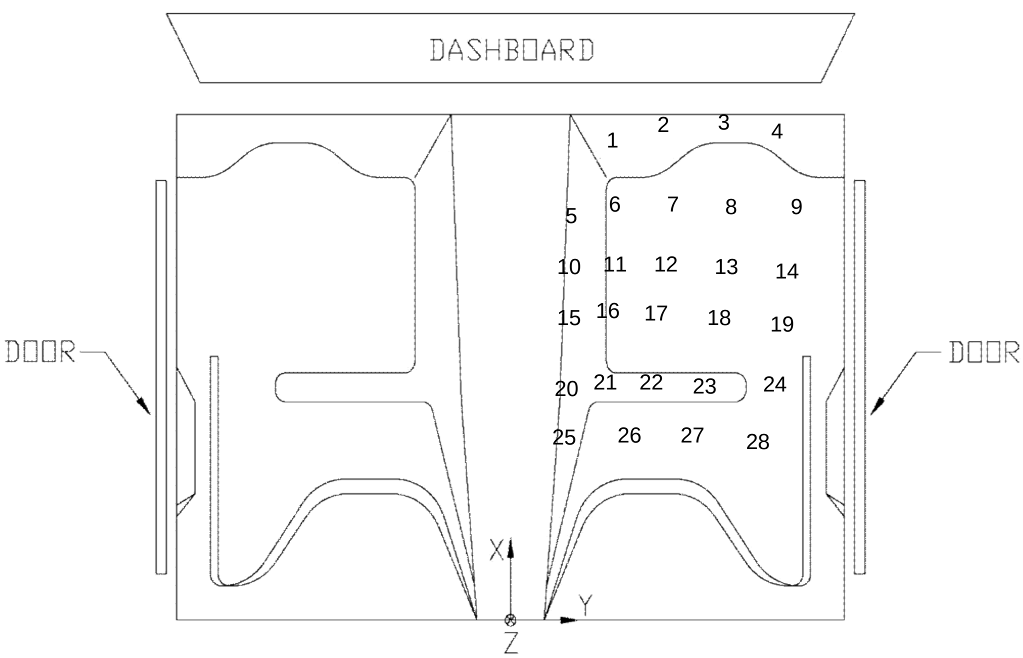


Figure C-2. Occupant Compartment Deformation Data – Set 2, Test No. WVBR-1

Occupant Compartment Deformation Index (OCDI)

Test No. WVBR-1
Vehicle Type: 2002 Dodge Ram

OCDI = XXABCDEFGHI

XX = location of occupant compartment deformation

A = distance between the dashboard and a reference point at the rear of the occupant compartment, such as the top of the rear seat or the rear of the cab on a pickup

B = distance between the roof and the floor panel

C = distance between a reference point at the rear of the occupant compartment and the motor panel

D = distance between the lower dashboard and the floor panel

E = interior width

F = distance between the lower edge of right window and the upper edge of left window

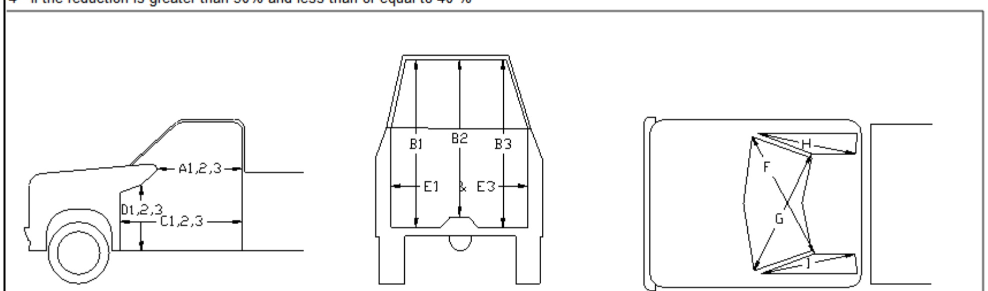
G = distance between the lower edge of left window and the upper edge of right window

H= distance between bottom front corner and top rear corner of the passenger side window

I= distance between bottom front corner and top rear corner of the driver side window

Severity Indices

0 - if the reduction is less than 3%
1 - if the reduction is greater than 3% and less than or equal to 10 %
2 - if the reduction is greater than 10% and less than or equal to 20 %
3 - if the reduction is greater than 20% and less than or equal to 30 %
4 - if the reduction is greater than 30% and less than or equal to 40 %



where,
1 = Passenger Side
2 = Middle
3 = Driver Side

Location:

Measurement	Pre-Test (in.)	Post-Test (in.)	Change (in.)	% Difference	Severity Index
A1	56.00	56.25	0.25	0.45	0
A2	50.50	50.50	0.00	0.00	0
A3	57.00	57.00	0.00	0.00	0
B1	47.25	47.00	-0.25	-0.53	0
B2	41.75	41.75	0.00	0.00	0
B3	47.50	47.50	0.00	0.00	0
C1	73.75	73.25	-0.50	-0.68	0
C2	48.00	48.00	0.00	0.00	0
C3	71.00	70.00	-1.00	-1.41	0
D1	22.75	22.50	-0.25	-1.10	0
D2	13.00	13.25	0.25	1.92	0
D3	22.50	22.50	0.00	0.00	0
E1	65.25	65.75	0.50	0.77	0
E3	65.00	65.25	0.25	0.38	0
F	59.00	59.50	0.50	0.85	0
G	59.25	59.25	0.00	0.00	0
H	40.00	40.00	0.00	0.00	0
I	40.50	40.75	0.25	0.62	0

Note: Maximum sevrity index for each variable (A-I) is used for determination of final OCDI value

	XX	A	B	C	D	E	F	G	H	I
Final OCDI:	RF	0	0	0	0	0	0	0	0	0

Figure C-3. Occupant Compartment Deformation Index (OCDI), Test No. WVBR-1

APPENDIX D. Accelerometer and Rate Transducer Data Plots

Figure D-1. 10-ms Average Longitudinal Acceleration, Test No. WVBR-1 (EDR-3)

Figure D-2. Longitudinal Occupant Impact Velocity, Test No. WVBR-1 (EDR-3)

Figure D-3. Longitudinal Occupant Displacement, Test No. WVBR-1 (EDR-3)

Figure D-4. 10-ms Average Lateral Acceleration, Test No. WVBR-1 (EDR-3)

Figure D-5. Lateral Occupant Impact Velocity, Test No. WVBR-1 (EDR-3)

Figure D-6. Lateral Occupant Displacement, Test No. WVBR-1 (EDR-3)

Figure D-7. 10-ms Average Longitudinal Acceleration, Test No. WVBR-1 (DTS)

Figure D-8. Longitudinal Occupant Impact Velocity, Test No. WVBR-1 (DTS)

Figure D-9. Longitudinal Occupant Displacement, Test No. WVBR-1 (DTS)

Figure D-10. 10-ms Average Lateral Acceleration, Test No. WVBR-1 (DTS)

Figure D-11. Lateral Occupant Impact Velocity, Test No. WVBR-1 (DTS)

Figure D-12. Lateral Occupant Displacement, Test No. WVBR-1 (DTS)

Figure D-13. Vehicle Roll, Pitch, and Yaw Angular Displacements, Test No. WVBR-1

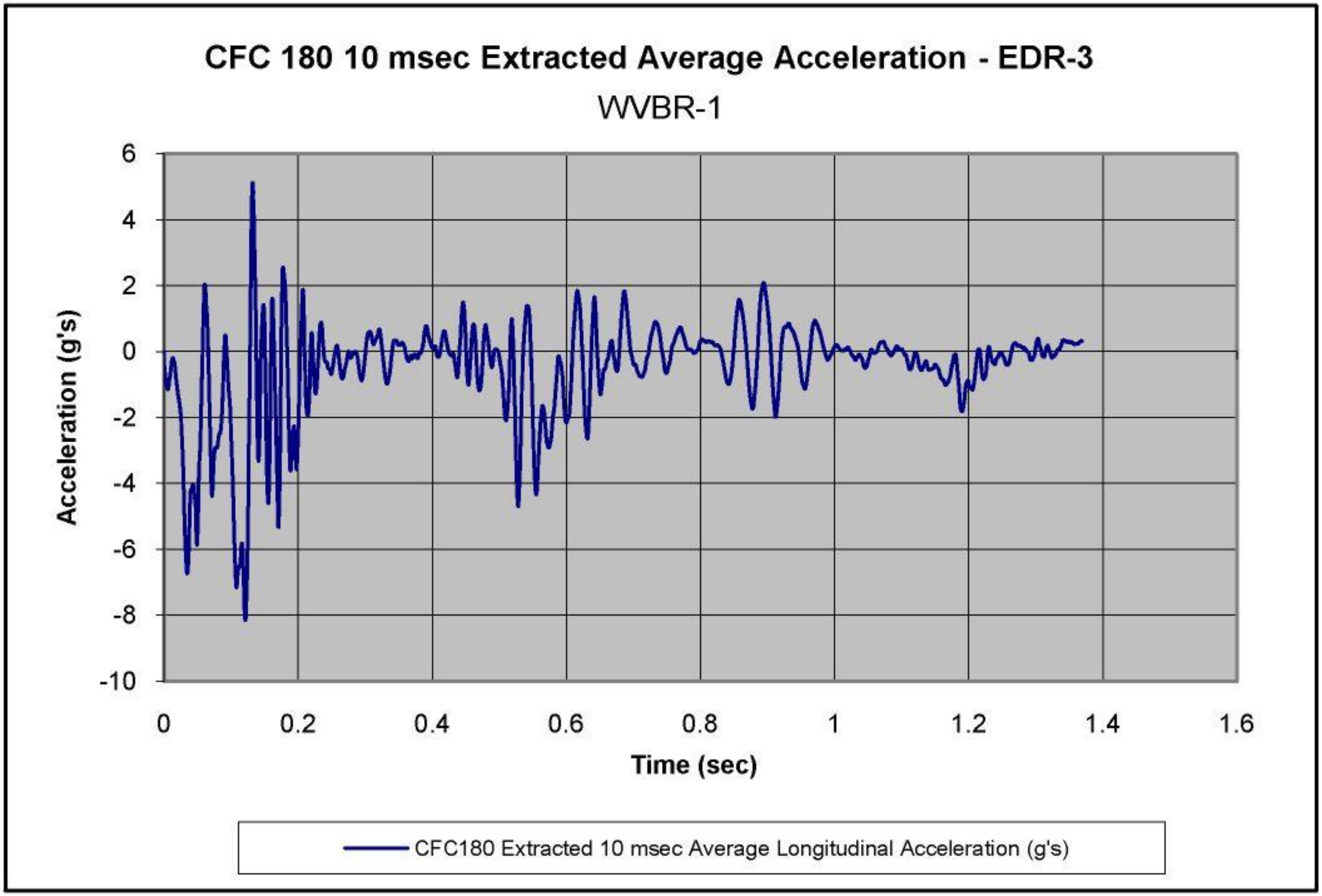


Figure D-1. 10-ms Average Longitudinal Acceleration, Test No. WVBR-1 (EDR-3)

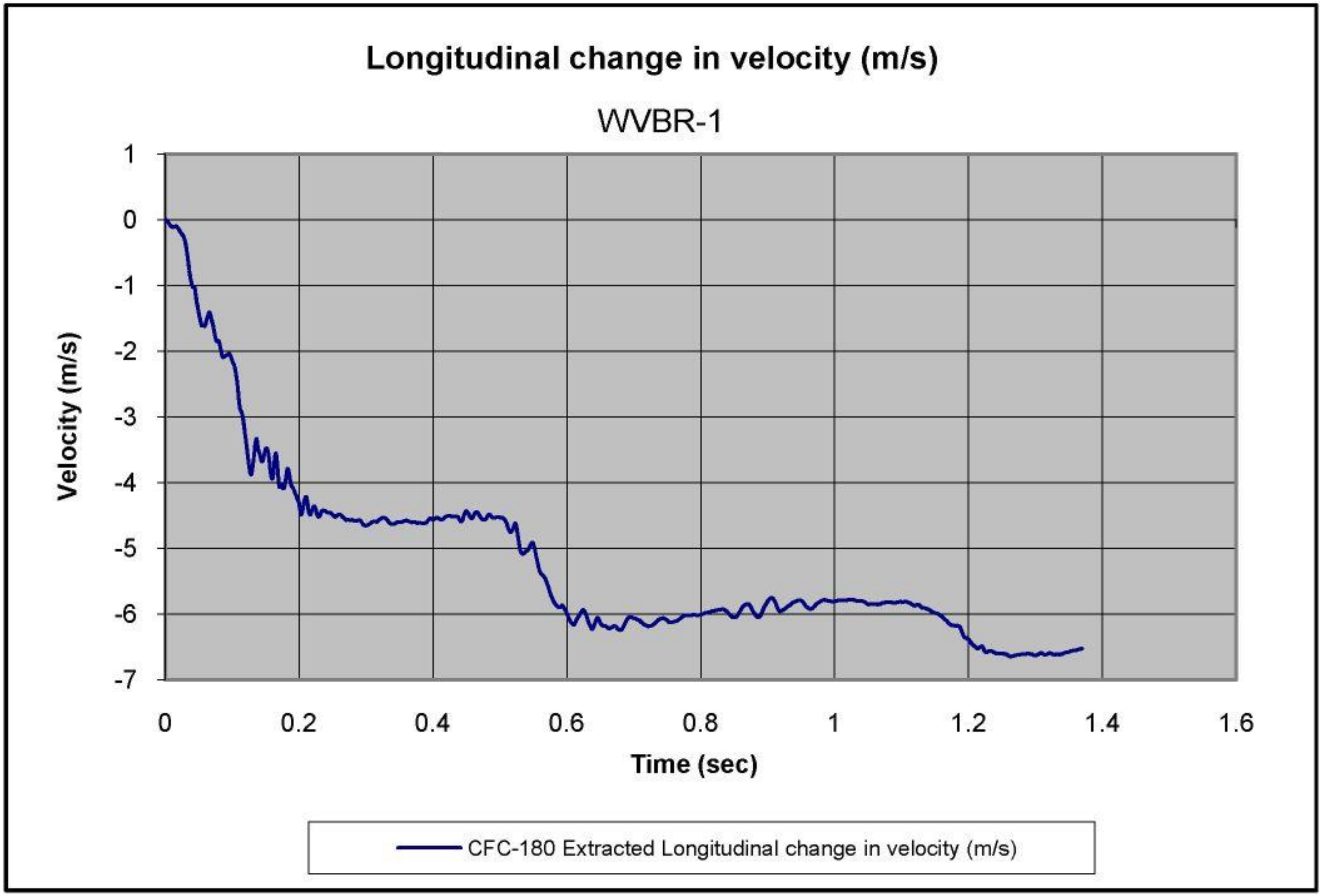


Figure D-2. Longitudinal Occupant Impact Velocity, Test No. WVBR-1 (EDR-3)

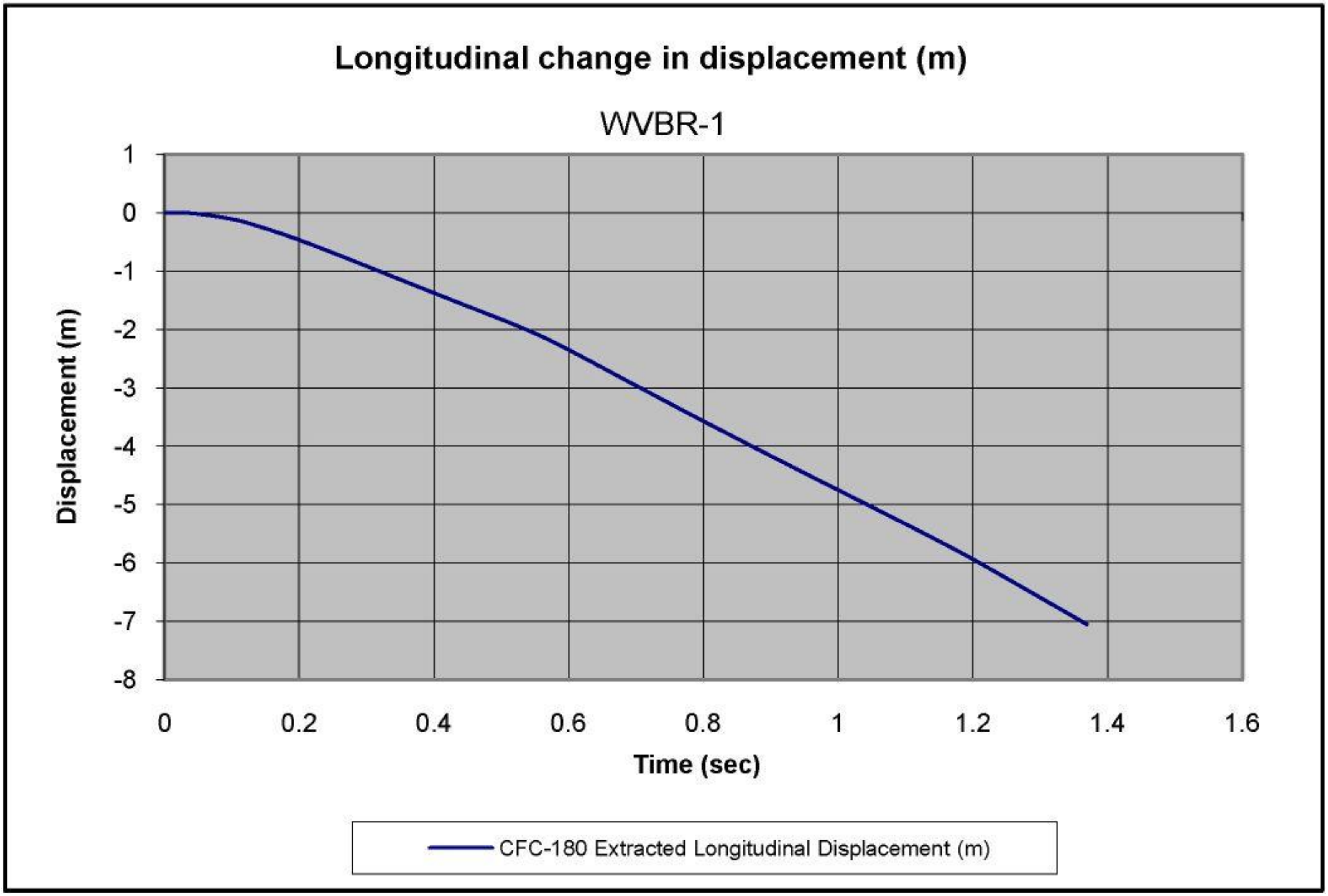


Figure D-3. Longitudinal Occupant Displacement, Test No. WVBR-1 (EDR-3)

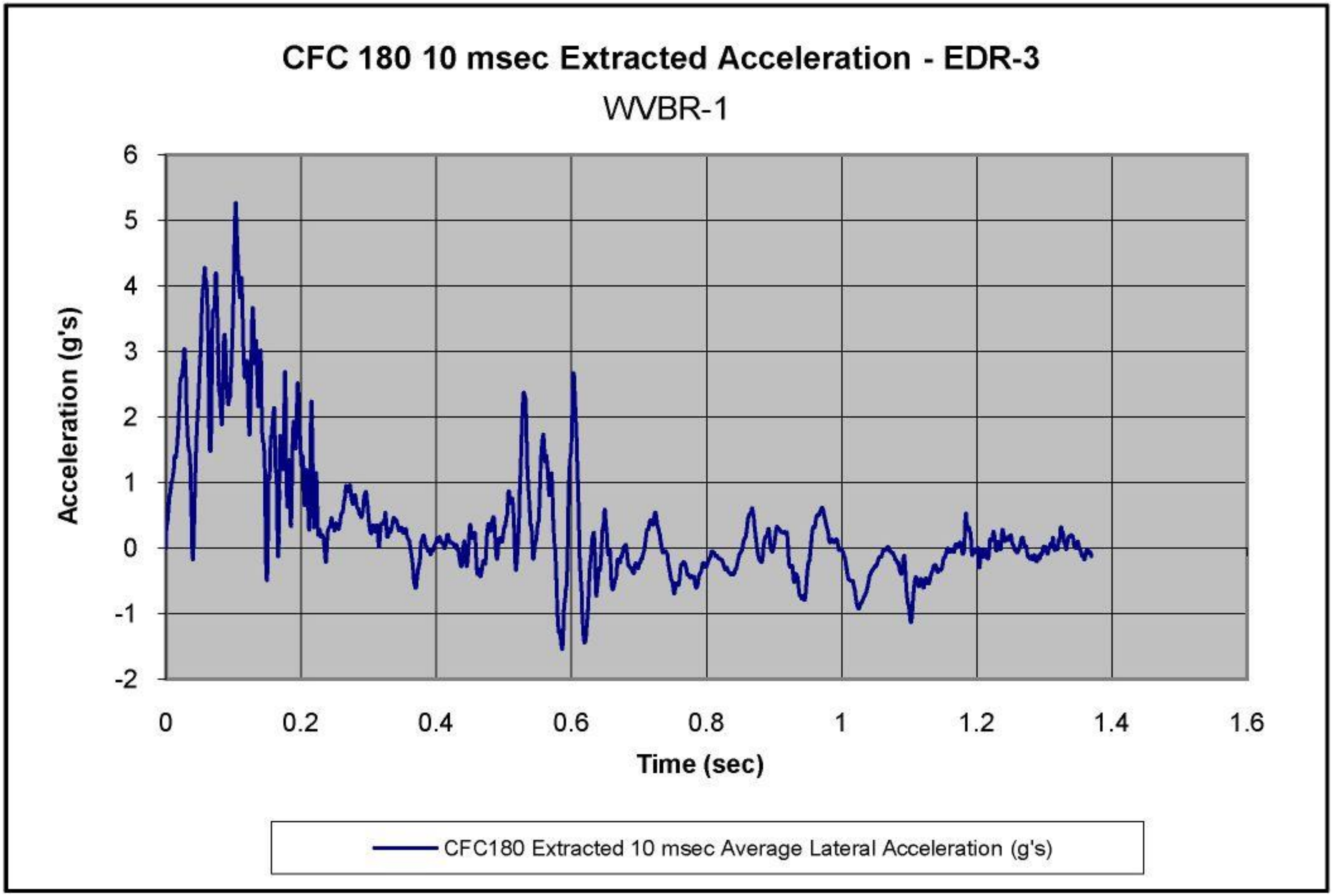


Figure D-4. 10-ms Average Lateral Acceleration, Test No. WVBR-1 (EDR-3)

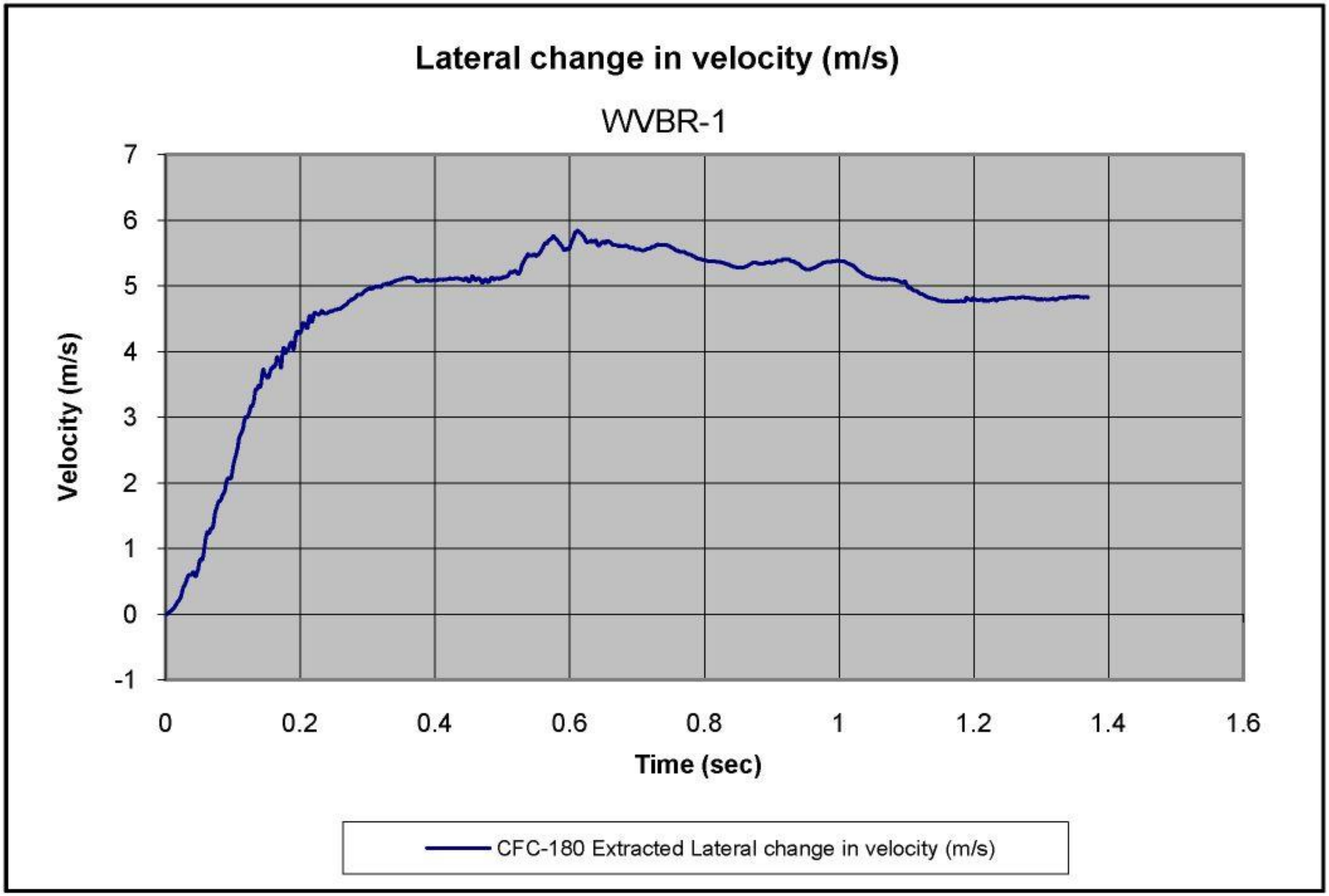


Figure D-5. Lateral Occupant Impact Velocity, Test No. WVBR-1 (EDR-3)

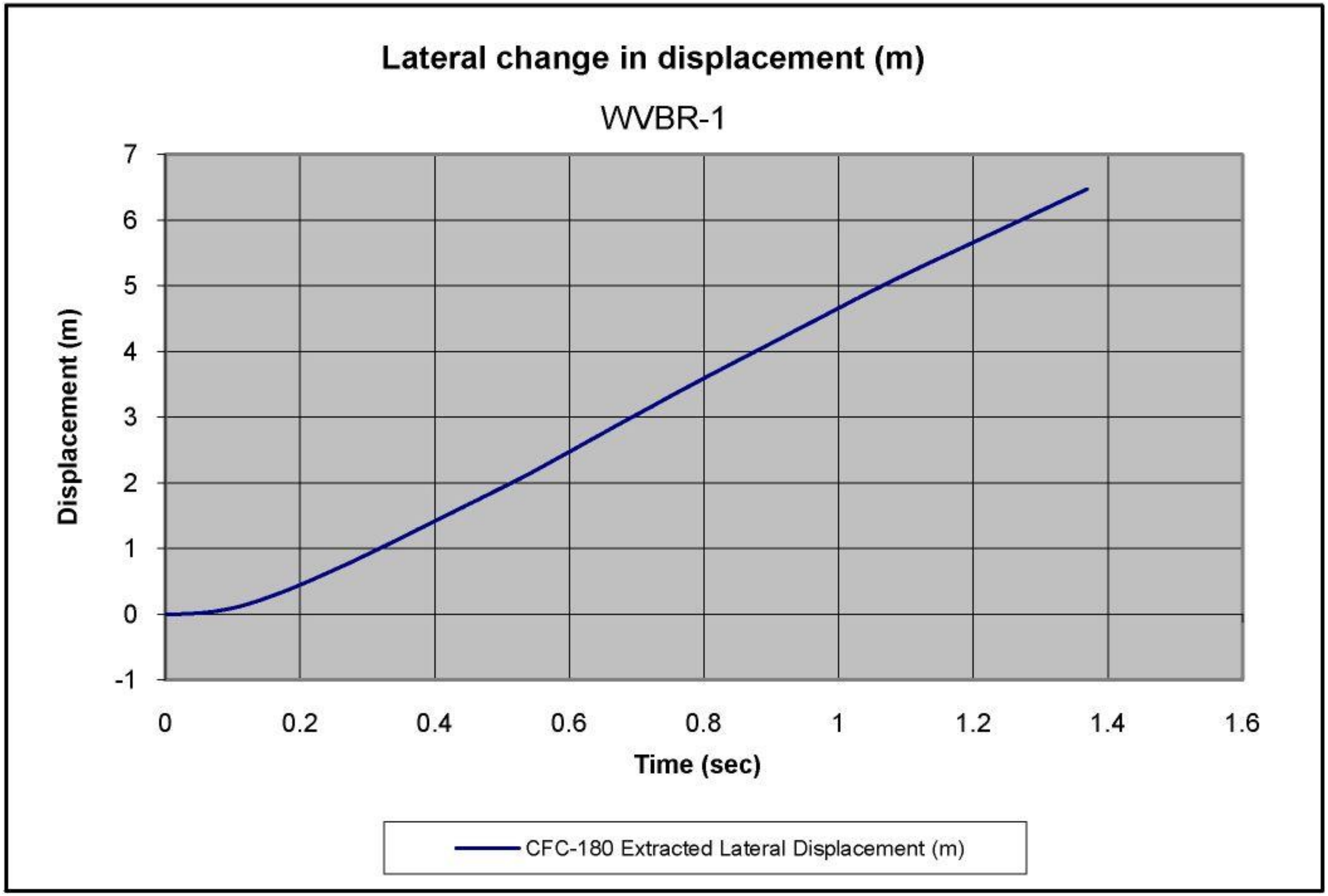


Figure D-6. Lateral Occupant Displacement, Test No. WVBR-1 (EDR-3)

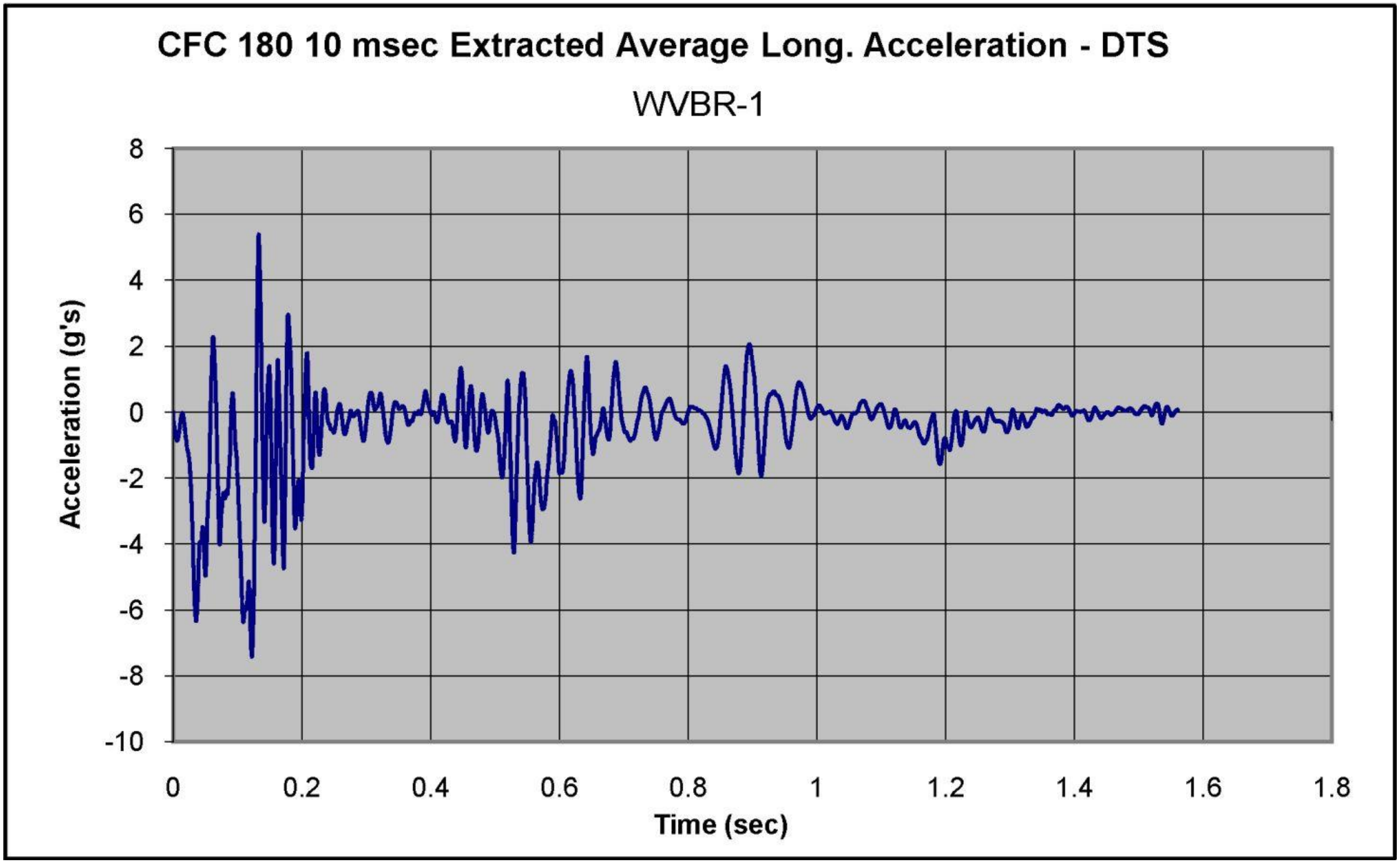


Figure D-7. 10-ms Average Longitudinal Acceleration, Test No. WVBR-1 (DTS)

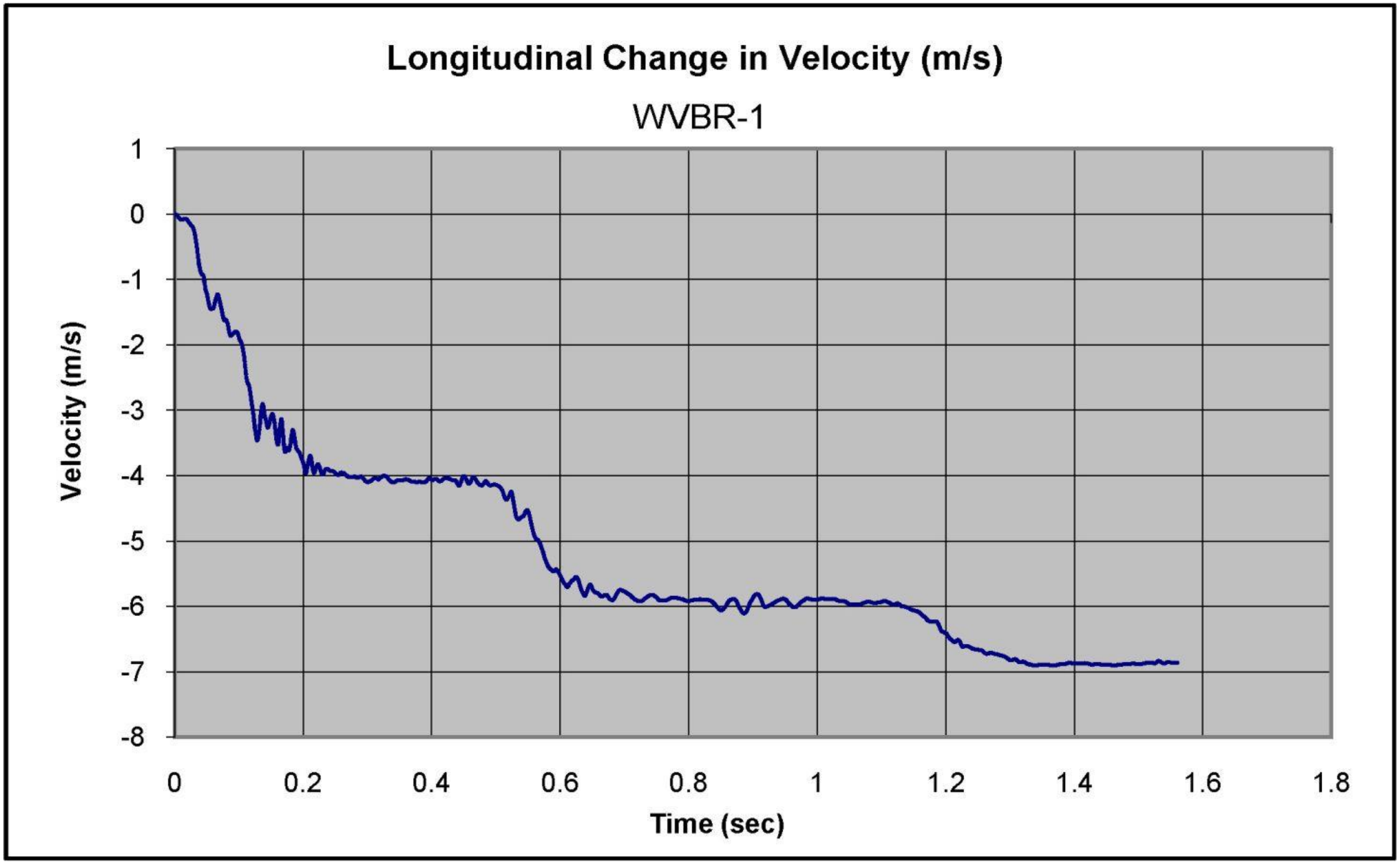


Figure D-8. Longitudinal Occupant Impact Velocity, Test No. WVBR-1 (DTS)

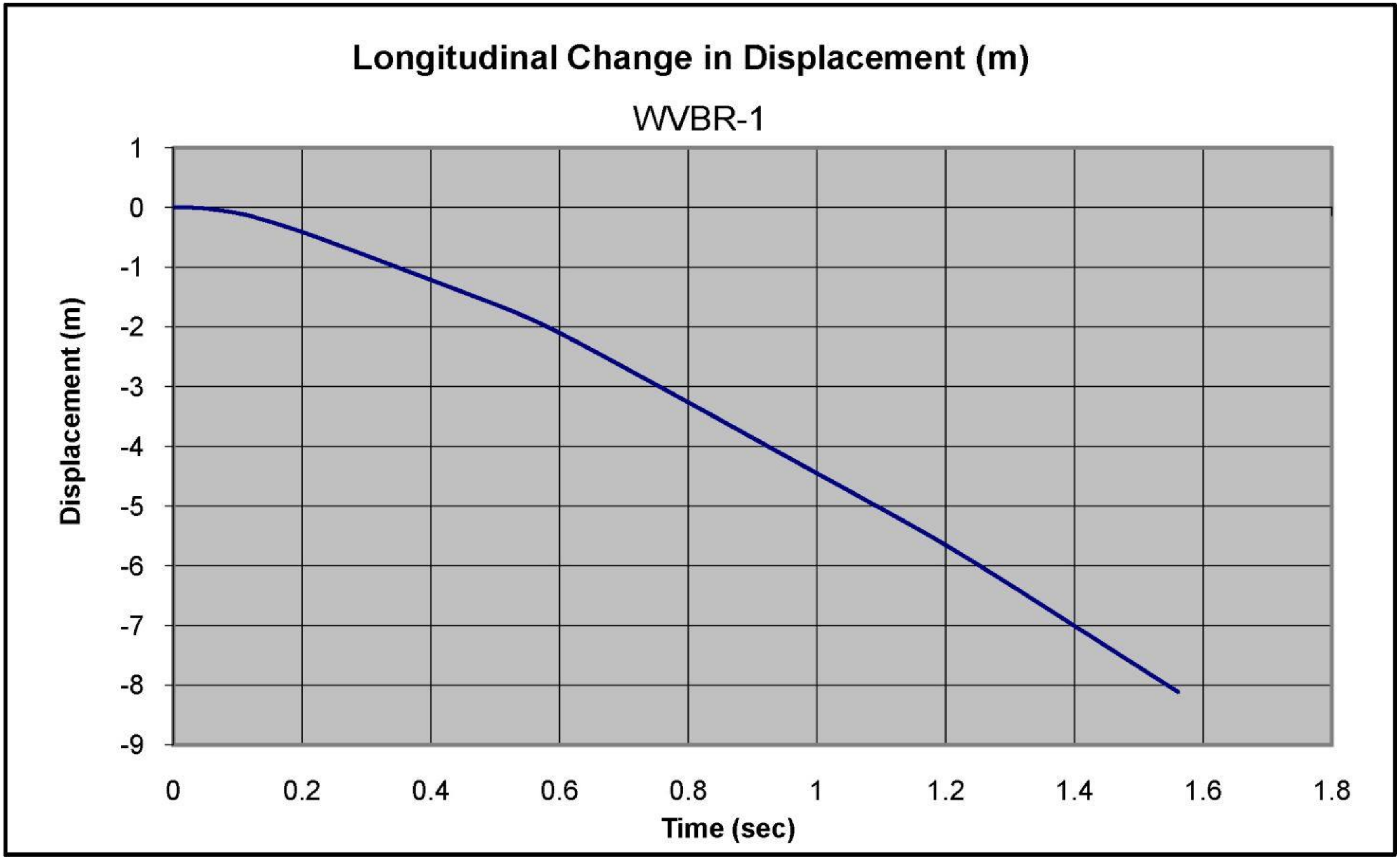


Figure D-9. Longitudinal Occupant Displacement, Test No. WVBR-1 (DTS)

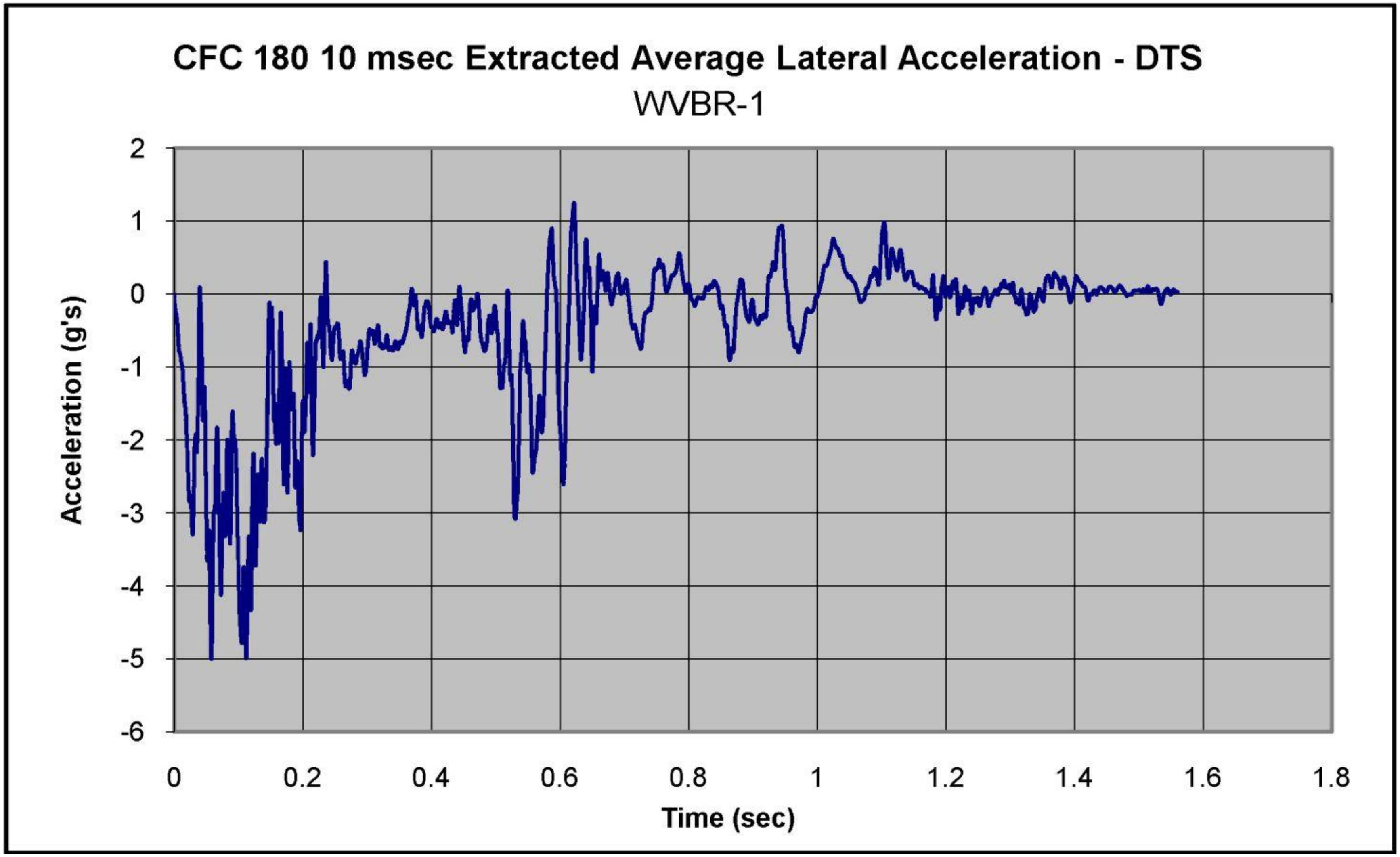


Figure D-10. 10-ms Average Lateral Acceleration, Test No. WVBR-1 (DTS)

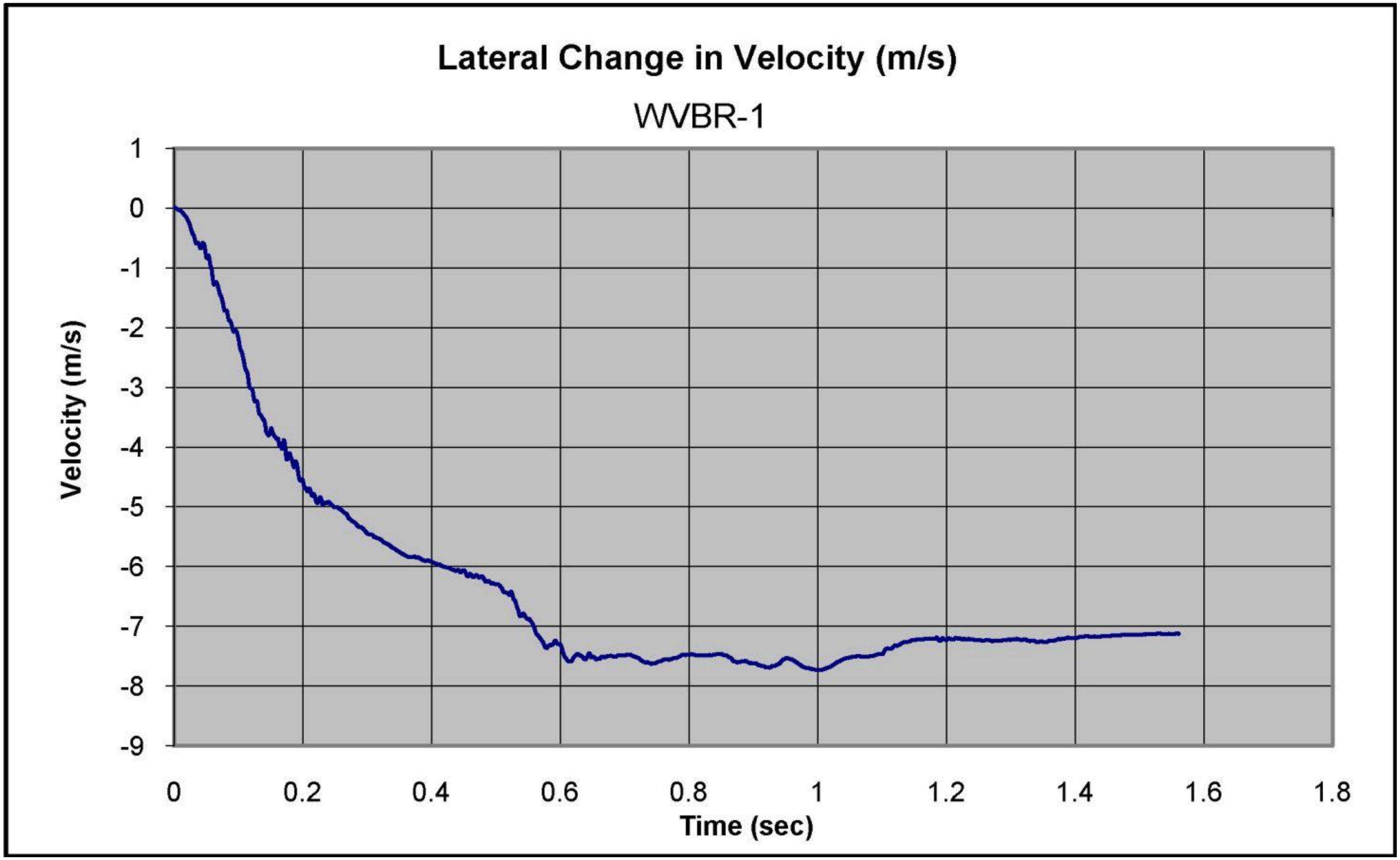


Figure D-11. Lateral Occupant Impact Velocity, Test No. WVBR-1 (DTS)

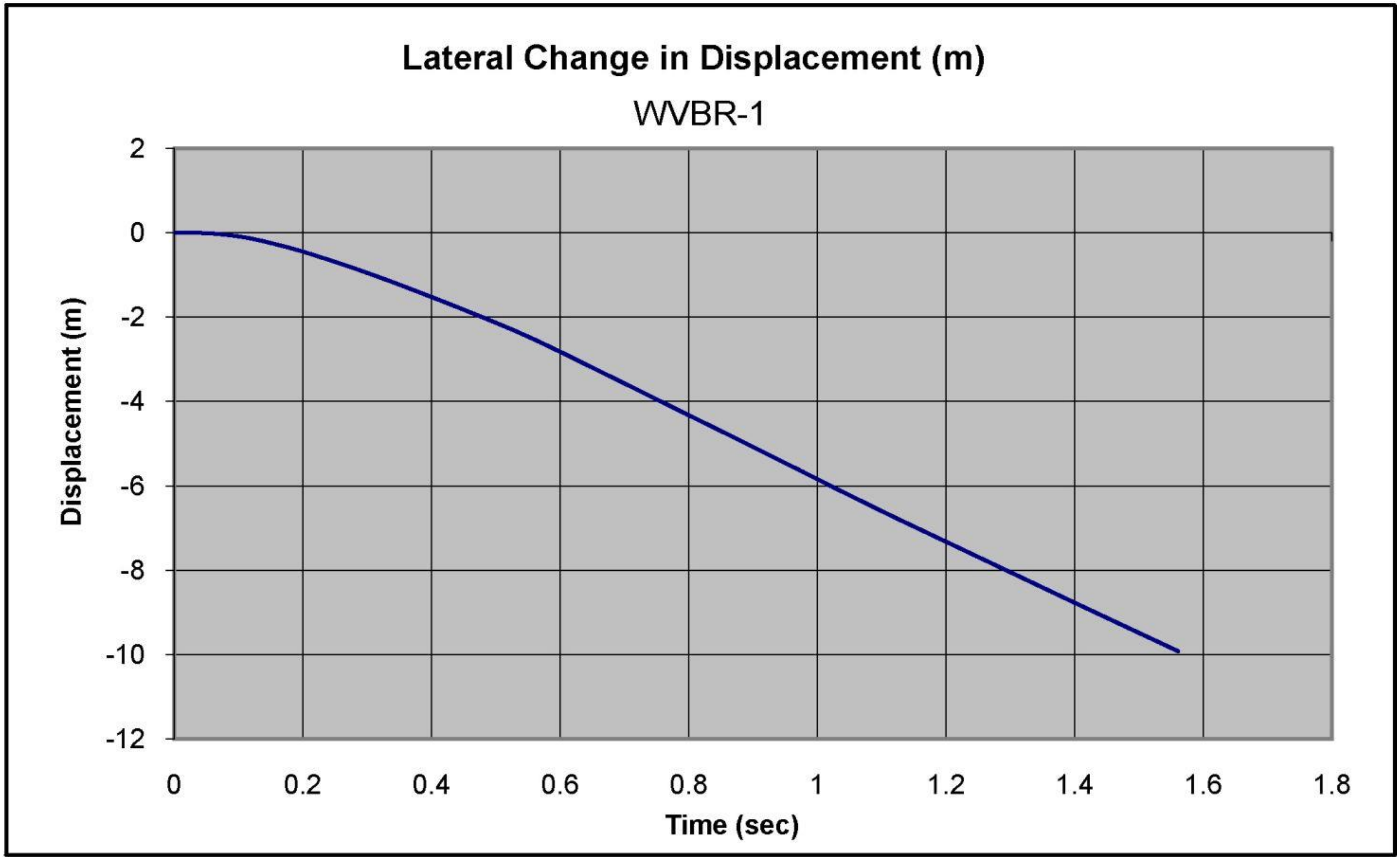


Figure D-12. Lateral Occupant Displacement, Test No. WVBR-1 (DTS)

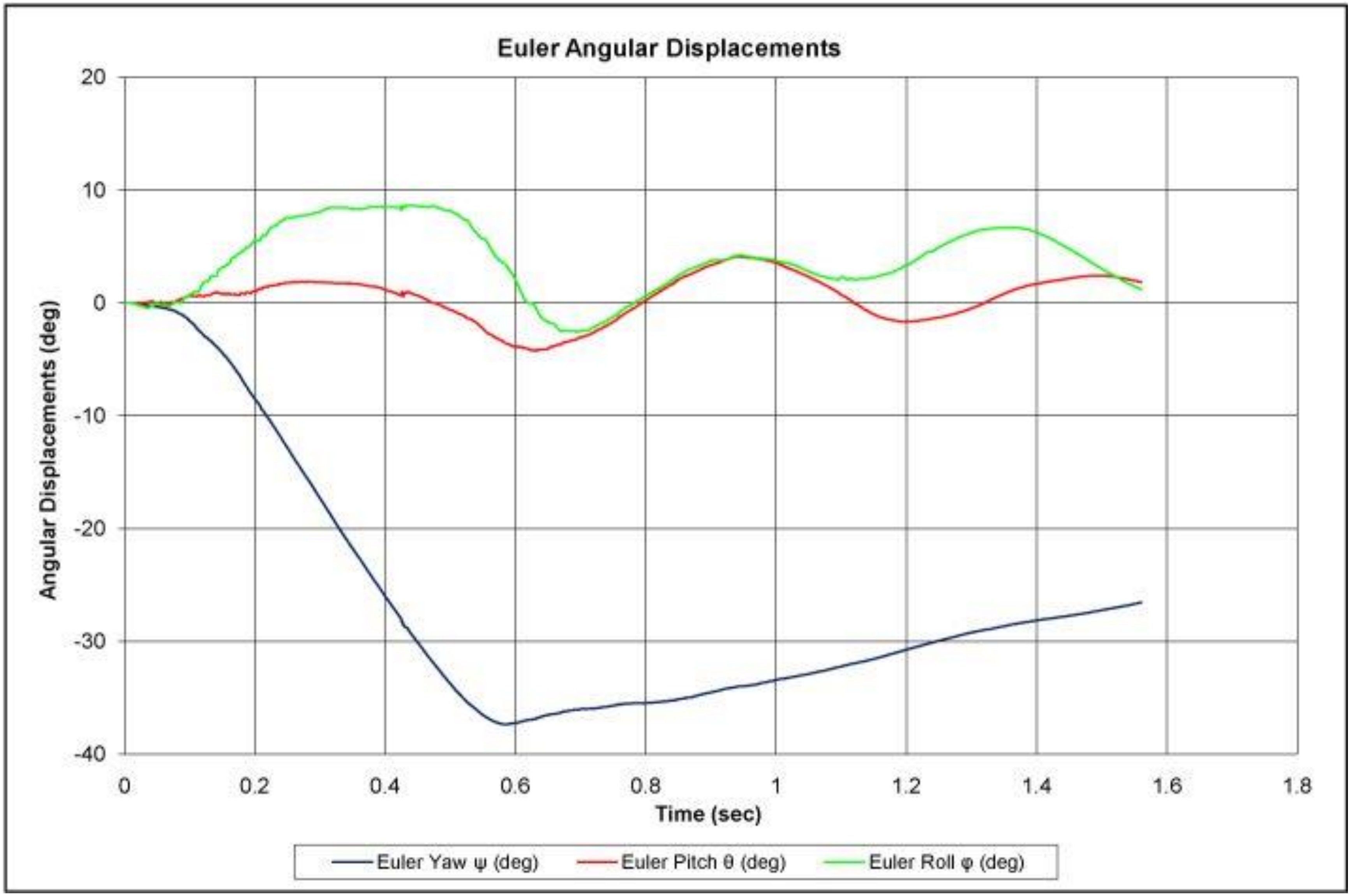


Figure D-13. Vehicle Roll, Pitch, and Yaw Angular Displacements, Test No. WVBR-1

APPENDIX E. Material Specifications and Documentation

Figure E-1. Glulam Timber Beam Invoice

Figure E-2. Timber Rail and Scupper Products Bill of Materials

Figure E-3. Timber Products Certificate of Performance

Figure E-4. Deck Lumber Invoice

Figure E-5. Deck Anchor Bracket Invoice

Figure E-6. Deck Anchor Bracket Certification

Figure E-7. Steel Connection Hardware Invoice

Figure E-8. Steel Connection Hardware Certification

Figure E-9. Steel Connection Hardware Certification Continued

Figure E-10. Splice Plate Steel Certification

Figure E-11. Splice Plate Steel Certification Continued

S0017024

ALAMCO
WOOD PRODUCTS, INC.
1410 WEST 9TH STREET
ALBERT LEA, MN 56007

INVOICE

Customer PO#	Alamco Order #	Invoice Date
4500183834	99-3824	4/22/2008
	Invoice #	
	04-372	

Sold To: University of Nebraska
Attn: **Accounting Department**
401 Canfield Admin Building
PO Box 880439
Lincoln, NE 68588-0439

Shipped To: Midwest Roadside Safety - MWRSF
4800 NW 35th Street
Lincoln, NE 68524

Remit To:

Alamco Wood Products, Inc.
1410 West Ninth Street
Albert Lea, MN 56007

Telephone: 507-373-1401
Fax: 507-373-8116

Carrier: Pro Trucking		
Status: Complete	Weight: 4,658	Date Shipped:

Pieces	Description	Board Feet	Amount
50	Laminated Wood Beams	2,588	$5,652.00

RECEIVED
APR 25 2008
UNL ACCOUNTING

Sub Total	$5,652.00
Freight	$250.00
Total Due	**$5,902.00**

Terms:

Net Due 10 Days

If any shortage or damage to contents has occurred, please have carrier's agent note same of freight bill.
Please note that a service charge of 1 1/2% per month on the unpaid balance will be made on all past due accounts per our General Terms of Sale.

Figure E-1. Glulam Timber Beam Invoice

Carrier

ALAMCO
WOOD PRODUCTS, INC.
1410 WEST 9TH STREET
ALBERT LEA, MN 56007
507-373-1401 Outside MN 800-328-8255
BILL OF MATERIALS

Customer:	U OF NEB	Sheets	1
PO #	4500183834	Sheet 1	of 1
Order No.	99-3824	Revised:	
		Revised:	
Job Name	BRIDGE RAIL		
	LINCOLN, NEB.		

QUANT.	TYPE	MARK	SIZE	LENGTH		
7	BEAM	R1	6 3/4X12 3/8	19-11 1/4		
1		R2	6 3/4X12 3/8	15-0 3/4		
1		R3	6 3/4X12 3/8	1-11		
3		R4	6 3/4X12 3/8	1-11		
1		R5	6 3/4X12 3/8	1-11		
27	SOLIDS	S1	8X10	1-11		
8		S2	8X10	1-11		
2		S3	8X10	1-11		
			TREAT AT BELL TO .6 PENTA			

RECEIVED
APR 25 2008
UNL ACCOUNTING

Figure E-2. Timber Rail and Scupper Products Bill of Materials

CERTIFICATE OF CONFORMANCE

THE UNDERSIGNED MANUFACTURER HEREBY CERTIFIES that the products identified below and on attached sheets are marked with the Collective Mark of the AMERICAN INSTITUTE OF TIMBER CONSTRUCTION (AITC) and were manufactured in conformance with applicable provisions of the latest revision of American National Standard for wood products – Structural Glued Laminated Timber, ANSI/AITC A190.1, and that such manufacture occurred at our plant in MN, which plant has a quality control system approved by the INSPECTION BUREAU OF THE AMERICAN INSTITUTE OF TIMBER CONSTRUCTION, and is audited periodically by such Bureau.

Job Name: University of Nebraska

Job Location: Lincoln, NE

Customer's Order No. 4500183834 Date: 3-12-08 Mfgr's Order No. 99-3824

Order Description:

Per attached material list

RECEIVED
APR 25 2008
UNLACCOUNTING

Signature: Company: Alamco Wood Products, Inc.

Title: Quality Control Address: 1410 W. 9th St. Albert Lea, MN 56007 Date: 3-12-08

AITC HEREBY CERTIFIES that the said company at its said plant is licensed by the AMERICAN INSTITUTE OF TIMBER CONSTRUCTION to use the AITC Collective Mark in respect of products which comply with applicable provisions of said Standard, that the adequacy of the quality control system in effect at said plant is periodically audited and verified by the AITC INSPECTION BUREAU, and that in the judgment of such Bureau, said company is capable of complying with applicable manufacturing and testing provisions of said Standard in respect of products manufactured in said plant. Conformance with the Standard in respect of any specific or particular product is the sole responsibility of the manufacturer; AITC's guarantee hereunder being only that the said company is qualified to produce a product meeting the said Standard and that its plant is periodically audited and verified by the AITC INSPECTION BUREAU.

AMERICAN INSTITUTION OF TIMBER CONSTRUCTION CORPORATE SEAL 1952 DELAWARE

AITC Certificate No. **04-112312**

AMERICAN INSTITUTE OF TIMBER CONSTRUCTION

© 2004 AMERICAN INSTITUTE OF TIMBER CONSTRUCTION

Figure E-3. Timber Products Certificate of Performance

LINCOLN LUMBER COMPANY

LUMBERYARD
932 NORTH 23rd
LINCOLN, NEBRASKA
(402) 474-4488

MAILING ADDRESS
P.O. BOX 30373
LINCOLN, NE 68503-0373
FAX NO. (402) 474-3749

LUMBER YARD

A FINANCE CHARGE will be assessed on any past due balances computed at a **PERIODIC RATE OF 2% PER MONTH (ANNUAL PERCENTAGE RATE OF 24%)** on the outstanding balance. Mechanics Liens will be filed automatically as lien date approaches.

SOLD TO

UNIVERSITY OF NEBR
ACCOUNTING
401 ADM BLDG.
LINCOLN, NE 68588-0439

SHIP TO

MIDWEST ROADSIDE SAFETY
4800 NW 35TH STREET.
MWRSF
LINCOLN, NE 68524

ACCOUNT NO.	CUSTOMER P.O. NO.	TERMS	ORDER NO.	ORDER DATE	SLSM	LUMBER INV. NO.	INVOICE DATE
U7500	45001	DUE NET 20 DAYS	28070	03/03/08	MAK	L	0 / / 0

ORDERED	BACK ORDERED	SHIPPED	✓	DESCRIPTION	U/M	U/PRICE	AMOUNT
1024	0	1024		2X6X14' .40 #1 WSYP TREATED V /TREATED LUMBER	EA		

Ken Krenk

MAR 3, 2008 13:47:22 OT:

FILLED BY | CHECKED BY | SHIPPED BY

****
* DELIVERY TICKET *
****

SHIP VIA LINCOLN LUMBER

PAGE 1 OF 1

PO# 4500185288

MERCHANDISE	
OTHER	
TAX	
FREIGHT	
TOTAL	

Figure E-4. Deck Lumber Invoice

TMCO

TMCO Inc
535 "J" Street
Lincoln, NE 68508
Phone: 402-476-0013
Fax: 402-817-2050

Quotation

Quote To: UNIVERSITY
MIDWEST ROADSIDE SAFETY
LINCOLN, NE
United States

Quote Number:	18394			**Contact:**	
Quote Date:	05/04/08	**Expires:**	06/03/08	**Inquiry:**	
Customer:	UNL001			**Terms:**	Net 30 Days
Salesman:	HOUSE SALES			**Phone:**	472-4767
Ship Via:				**FAX:**	472-8080
FOB:	Lincoln, NE 68508			**Delivery:**	Standard Delivery 4 weeks FRO.

Thank you for the opportunity to submit this quote. Please be aware that it is our standard practice to fill each order to plus or minus 10% of the ordered quantity. Please specify if exact quantities are required. All deliveries made by TMCO are subject to a delivery charge. Please contact customer service for current delivery charge.

Item	Part Number	Description	Revision	Quantity	Price
1	DECK BRACKET	WEST VIRGINIA TL-1 CURB-TYPE TIMBER BRIDGE RAIL (MADE FROM 11GA A36 GALVANIZED SHEET)		260	$2.2400/EA
				Total:	***$582.40***

By Shane Jackson
TMCO Inc

Figure E-5. Deck Anchor Bracket Invoice

NUCOR
SHEET MILL GROUP
Nucor Steel-Crawfordsville
4537 South Nucor Road
Crawfordsville, IN 47933-9450

C121380
Print Date: 05/30/2007

Page: 2 of 3

Customer Name: STATE STEEL SUPPLY CO., INC.
Customer Address: P.O. BOX 3224
SIOUX CITY IA 51102
Customer PO Number: P703308L003

Order Number: 172734 - 0001
Order Dimensions: 0.1150 X 48.0000
GVHR,CUT,A653CS,1005A1,690

Coil Number	Heat	Slab
1186606.000	272616	06

Chemical Analysis

Heat	C	Mn	P	S	Si	Cu	Sn	Ni	Cr	Mo	Al	N	V	Nb	Ti	B	Sb
272616	0.04	0.240	0.015	0.001	0.026	0.109	0.010	0.037	0.087	0.015	0.045	0.008	0.002	0.001	0.003	<0.001	0.002

Coil Number	Heat	Slab
1188198.000	272618	01

Chemical Analysis

Heat	C	Mn	P	S	Si	Cu	Sn	Ni	Cr	Mo	Al	N	V	Nb	Ti	B	Sb
272618	0.04	0.240	0.010	<0.001	0.026	0.063	0.003	0.017	0.040	0.012	0.046	0.008	0.002	0.001	0.003	<0.001	0.002

Anchor Bracket
Mill Cert.

MELTED AND ROLLED IN THE USA

THIS IS NOT A CERTIFIED TEST REPORT

Figure E-6. Deck Anchor Bracket Certification

S0013003

PORTLAND BOLT & Manufacturing Co., Inc.
3441 N.W. Guam / Portland, Oregon 97210
(503) 227-5488 / (800) 547-6758 / FAX (503) 227-4634
Established 1912 • www.portlandbolt.com
DOMESTIC MANUFACTURER OF CONSTRUCTION FASTENERS
★ *Celebrating 95 Years • Proudly made in the USA* ★

SALES ORDER NUMBER	INVOICE DATE	INVOICE NUMBER
19294	3/26/2008	19294

REMIT TO:
P.O. Box 2866
Portland, OR 97208

invoice

SOLD TO:
UNIV OF NEBRASKA
MIDW ROADSIDE SAFETY
527 NEBRASKA HALL
LINCOLN, NE 68588-0529

SHIP TO:
UNIV OF NEBRASKA
MIDW ROADSIDE SAFETY
4800 NW 35TH STREET
LINCOLN, NE 68524

FID 93-1129217

CUSTOMER NUMBER	CUSTOMER ORDER NUMBER	ORDERED BY	DATE ORDERED	DATE PROMISED	SALESMAN	B/O INVOICE NUMBER	PAGE
62290	4500184460	SCOTT	3/11/2008	3/28/2008	STEVEN W		1

SHIP VIA: FEDEX FREIGHT
TERMS: NET 30

QUANTITY ORDERED	B.O.	PRODUCT CODE	DESCRIPTION	WEIGHT	QUANTITY SHIPPED	INVENTORY SOURCE	UNIT PRICE	TOTAL
24		12405	4 X 3/4 GAL SHEAR PLATE		24	1	4.75/E	114.00
28		18686	4" DOMESTIC GAL SPLIT RING		28	1	6.06/E	169.68
75		18587	3/4 X 30 GAL ECONOMY BOLT 8" TH		75	1	6.77/E	507.75
20		18553	3/4 X 10 GAL ECONOMY BOLT 4" TH		20	1	3.46/E	69.20
90		12378	1 X 14 GALV A36 ECONOMY BOLT 6"THD R/T		90	2	11.02/E	991.80
95		11706	3/4 GAL HEX NUTS A563A		95	1		N/C
75		11988	3/4 ROUND GAL MALLEABLE WASHERS		75	1	.80/E	60.00
20		16413	3/4 DOM GAL F844 CUTWASHER		20	1	.23/E	4.60
90		11710	1 GAL HEX NUT A563A		90	1		N/C
90		11992	1 ROUND GAL MALLEABLE WASHERS		90	1		N/C
		14188	FREIGHT CHARGES MK: P008-33			2		243.75

No credit will be issued for goods returned without our permission. Special order merchandise will not be accepted for credit. No claim for shortages allowed unless reported immediately upon receipt of shipment. A service charge of one and one-half percent (1½%) per month which is an A.P.R. of eighteen percent (18%) will be charged on all past due accounts.
We hereby certify that these goods were produced in compliance with all applicable requirements of section 6, 7 and 12 of the Fair Labor Standards Act as amended and of regulations and orders of the United States Department of Labor issued under Section 14 thereof.

DATE SHIPPED	TOTAL WEIGHT	FREIGHT	INVOICE TOTAL
3/26/2008	808		2160.78

FORM NO. 403 (4/99

CUSTOMER'S COPY

Figure E-7. Steel Connection Hardware Invoice

PORTLAND BOLT
& MANUFACTURING CO., INC.
1-800-547-6758

Mailing Address: PO Box 2866 • Portland, OR 97208
Physical Address: 3441 NW Guam St. • Portland, OR 97210
Phone: 503-227-5488 • Fax: 503-227-4634
Web: www.portlandbolt.com • E-Mail: sales@portlandbolt.com

PRODUCT CERTIFICATION

For: UNIV OF NEBRASKA
PB Invoice#: 19294
Cust PO#: 4500184460
Date: 3/14/2008
Shipped: 3/28/2008

We certify the following material was supplied in accordance with your order.

Description: 3/4 X 30 GALV ASTM A36 ECONOMY BOLT

Heat#: 350907 | Base Steel: A36 | Diam: .68

Source: CASCADE STEEL RLG MILL | Proof Load: 0

C :	.18	Mn:	.70	P :	.01	Hardness:	151 HBN		
S :	.02	Si:	.19	Ni:	.09	Tensile:	71,500 PSI	RA:	41.00%
Cr:	.06	Mo:	.01	Cu:	.32	Yield:	50,500 PSI	Elon:	24.00%
Pb:	.00	V :	.00	Cb:	.00	Sample Length:	8 INCH		
N :	.00			CE:	.3149	Charpy:			

Description: 3/4 X 10 GALV ASTM A307A ECONOMY BOLT

Heat#: 417706 | Base Steel: A36 | Diam: .68

Source: CASCADE STEEL RLG MILL | Proof Load: 0

C :	.18	Mn:	.68	P :	.01	Hardness:	151 HBN		
S :	.02	Si:	.15	Ni:	.09	Tensile:	70,500 PSI	RA:	56.00%
Cr:	.08	Mo:	.01	Cu:	.24	Yield:	48,300 PSI	Elon:	25.00%
Pb:	.00	V :	.00	Cb:	.00	Sample Length:	8 INCH		
N :	.00			CE:	.3116	Charpy:			

Figure E-8. Steel Connection Hardware Certification

PORTLAND BOLT
& MANUFACTURING CO., INC.
1-800-547-6758

Mailing Address: PO Box 2866 • Portland, OR 97208
Physical Address: 3441 NW Guam St. • Portland, OR 97210
Phone: 503-227-5488 • Fax: 503-227-4634
Web: www.portlandbolt.com • E-Mail: sales@portlandbolt.com

PRODUCT CERTIFICATION

For: UNIV OF NEBRASKA
PB Invoice#: 19294
Cust PO#: 4500184460
Date: 3/14/2008
Shipped: 3/28/2008

We certify the following material was supplied in accordance with your order.

Description: 1 X 14 GALV ASTM A36 ECONOMY BOLT

Heat#: 675407 | Base Steel: A36 | Diam: .912

Source: CASCADE STEEL RLG MILL | Proof Load: 0

C :	.180	Mn:	.660	P :	.005	Hardness:	156 HBN		
S :	.024	Si:	.180	Ni:	.100	Tensile:	70,500 PSI	RA:	45.00%
Cr:	.060	Mo:	.021	Cu:	.340	Yield:	47,500 PSI	Elon:	21.00%
Pb:	.000	V :	.000	Cb:	.000	Sample Length:	8 INCH		
N :	.000			CE:	.3090	Charpy:			

Nuts:
ASTM A563A HEX

Washers:
ASTM A47 MALLEABLE
ASTM F844 CUT
ASTM D5933 SHEAR PLATE

Coatings:
ASTM A153 CL.C AND F2329, HOT DIP GALV

By: ______________________
Certification Department Quality Assurance

Figure E-9. Steel Connection Hardware Certification Continued

Date: 2/23/2008 Time: 4:22 AM To: @ 91 402 379-5403 Page: 010-012

Certified Test Report

NORTH STAR BLUESCOPE STEEL LLC
6767 County Road 9
Delta, Ohio 43515
Telephone: (888) 822-2112

Customer:
Norfolk Iron & Metal
3001 Victory Road
Norfolk, NE 68702-1129

Customer P.O.: 02-124484-12	**Order Number**	170300	**Ordered Width (mm/in)**	1219.200 / 48.000
Cust. Ref/Part # 22968	**Line Item Number**	1	**Ordered Gauge (mm/in)**	9.525 / 0.375
	Heat Number	111172	**Material Description**	For Conversion to ASTM A656-50 Type 7
	Coil Number	837182	**Production Date/Time**	Feb 10 2008 3.14AM

Heat Chemical Analysis (wt%)

Type	C	Mn	P	S	Si	Al	Cu	Cr	Ni	Mo	Sn	N	B	V	Nb	Ti	Ca
Heat	0.05	0.84	0.011	0.003	0.03	0.02	0.09	0.03	0.03	0.01	0.00	0.014	0.0000	0.055	0.001	0.003	0.002

Mechanical Test Report

All mechanical tests are performed on a sample from the tail of a coil.

Yield Strength	Tensile Strength	% Elongation in 2 inches
65,150 psi	71,840 psi	34.5%

This material has been produced and tested in accordance with each of the following applicable standards: ASTM E 1806-96, ASTM E 415-98a, ASTM A 751-01, ASTM A 370-03a, JIS Z2201:1998, JIS Z 2241:1998. This report certifies that the above test results are representative of those contained in the records of North Star BlueScope Steel LLC for the material identified in this test report and is intended to comply with the requirements of the material description. North Star BlueScope Steel LLC is not responsible for the inability of this material to meet specific applications. Any modifications to this certification as provided negates the validity of this test report. All reproductions must have the written approval of North Star BlueScope Steel. This product was manufactured, melted, cast, and hot-rolled (min. 3.1 reduction ratio), entirely within the U.S.A at North Star BlueScope Steel LLC, Delta, Ohio. This material was not exposed to Mercury or any alloy which is liquid at ambient temperature during processing or while in North Star BlueScope Steel LLC possession. Test equipment calibration certificates are available upon request. NIST traceability is established through test equipment calibration certificates which are available upon request. Uncertainty calculations are calculated in accordance with NIST standards and are maintained at a 4:1 ratio in accordance with NIST standards. Uncertainty data is available upon request.

Tim Mitchell **Manager Quality Assurance and Technology**

Date Issued: Feb 22, 2008 06:00:15
Revision#: 01

Figure E-10. Splice Plate Steel Certification

GERDAU AMERISTEEL

CARTERSVILLE STEEL MILL
384 OLD GRASSDALE RD NE
CARTERSVILLE GA 30121 USA
(770) 387-3300

Chemical and Physical Test Report

MADE IN UNITED STATES

G-110473

PRODUCED IN: CARTERSVILLE

SHIP TO	INVOICE TO	SHIP DATE
NORFOLK IRON & METAL GREELEY 804 YARD 02, RR STATION WD694 970-352-6722 GILL, CO 80632	NORFOLK IRON & METAL CO INC ATTN-ACCTS PAYABLE PO BOX 1129 NORFOLK, NE 68702	02/27/08 CUST. ACCOUNT NO 60056348

SHAPE + SIZE	GRADE	SPECIFICATION	SALES ORDER	CUST P.O. NUMBER
W6 X 20#	A57250/992	ASTM A572 GR50-07, ASTM A992 -06A	8012749-01	03059147-01

HEAT I.D.	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Nb	B	N	Sn	Al	Ti	Ca	Zn	C Eqv
G800981	.17	.90	.013	.018	.20	.30	.10	.06	.021	.000	.009	.0002	.0086	.010	.001	.00100	.00050	.00550	.363

Mechanical Test: Yield 55800 PSI, 384.73 MPA Tensile: 76700 PSI, 528.83 MPA %El: 21.9/8in, 21.9/200MM Std Dev:0 Idl Diam: .576
Customer Requirements CASTING: STRAND CAST
Mechanical Test: Yield 57400 PSI, 395.76 MPA Tensile: 78800 PSI, 543.31 MPA %El: 21.5/8in, 21.5/200MM Std Dev:0 Idl Diam: .576
Customer Requirements CASTING: STRAND CAST

SHAPE + SIZE	GRADE	SPECIFICATION	SALES ORDER	CUST P.O. NUMBER
W6 X 15#	A57250/992	ASTM A572 GR50-07, ASTM A992 -06A	7132198-08	03058056-08

HEAT I.D.	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Nb	B	N	Sn	Al	Ti	Ca	Zn	C Eqv
G800975	.19	.86	.015	.026	.20	.32	.05	.06	.016	.000	.009	.0003	.0098	.012	.001	.00100	.00130	.00490	.382

Mechanical Test: Yield 59400 PSI, 409.55 MPA Tensile: 81200 PSI, 559.85 MPA %El: 23.3/8in, 23.3/200MM Std Dev:0 Idl Diam: .63
Customer Requirements CASTING: STRAND CAST
Mechanical Test: Yield 59800 PSI, 412.31 MPA Tensile: 82000 PSI, 565.37 MPA %El: 21.5/8in, 21.5/200MM Std Dev:0 Idl Diam: .63
Customer Requirements CASTING: STRAND CAST

This material, including the billets, was produced and manufactured in the United States of America.

Bhaskar Yalamanchili
Quality Director
Gerdau Ameristeel

THE ABOVE FIGURES ARE CERTIFIED EXTRACTS FROM THE ORIGINAL CHEMICAL AND PHYSICAL TEST RECORDS AS CONTAINED IN THE PERMANENT RECORDS OF COMPANY.

Mgr. Metallurg. Svcs.
CARTERSVILLE STEEL MILL

Seller warrants that all material furnished shall comply with specifications subject to standard published manufacturing variations. NO OTHER WARRANTIES, EXPRESSED OR IMPLIED, ARE MADE BY THE SELLER, AND SPECIFICALLY EXCLUDED ARE WARRANTIES OF MERCHANTABILITY AND FITNESS FOR A PARTICULAR PURPOSE.
In no event shall seller be liable for indirect, consequential or punitive damages arising out of or related to the materials furnished by seller.
Any claim for damages for materials that do not conform to specifications must be made from buyer to seller immediately after delivery of same in order to allow the seller the opportunity to inspect the material in question.

Figure E-11. Splice Plate Steel Certification Continued

END OF DOCUMENT