# ADAPTATION AND DEVELOPMENT OF A MASH TL-1 LOW-HEIGHT, GLULAM TIMBER BRIDGE RAILING FOR USE ON TRANSVERSE GLULAM DECKS

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| Cover | A2LA symbol added. |
| ii | Second paragraph in "Uncertainty of Measurement Statement" removed as all tests fall within MwRSF's scope of accredited services under A2LA. |
| 23-47 | MGTR drawing set replaced with corrected version. |
| 52-70 | MGTD drawing set replaced with corrected version. |
| 71-72 | Target impact speed for test no. MGTR-1D corrected to 13 mph. |
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**16. Abstract**

In a 2009 study for the West Virginia Department of Transportation, the Midwest Roadside Safety Facility developed an AASHTO *Manual for Assessing Safety Hardware* (MASH) Test Level 1 (TL-1) low-height, glue-laminated (glulam) timber bridge railing for use on a transverse, nail-laminated deck. A series of static component tests and one full-scale vehicle crash test were performed to configure the system and determine the crashworthiness of the bridge railing and deck.

More recently, the United States Department of Agriculture – Forest Service (USDA-FS) had expressed interest in utilizing the previously-developed MASH TL-1 low-height, glulam timber bridge railing on a transverse glulam deck system instead of a transverse nail-laminated deck. To cost-effectively adapt and determine the crashworthiness of TL-1 glulam bridge railing system on the alternative timber deck, a series of physical tests were conducted on the bridge railing components installed on a transverse glue-laminated deck and a transverse nail-laminated deck. For each deck type, one static test and one dynamic test was performed on a segment of the bridge railing system supported by two scupper blocks.

From the component testing results, the glulam rail and scupper blocks installed on a transverse glulam deck resisted more lateral force, absorbed more energy, and had higher initial linear stiffness as compared to the same rail and scupper blocks installed on a transverse nail-laminated deck. Thus, the low-height, glulam timber bridge railing installed on a transverse, glulam deck will provide equal or greater safety performance as compared to the same bridge rail installed on a transverse, nail-laminated deck. As a result, the low-height, glulam timber bridge rail installed on a transverse glulam deck has also been deemed adequate for use under MASH TL-1 impact conditions.



**17. Key Words**
Glue-Laminated Timber, Glulam, Nail-Laminated Timber, Nail-Lam, Bridge Rail, Bridge Deck, Crashworthy, TL-1, Component Testing, Bogie Testing, Static, Dynamic, and MASH


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## UNCERTAINTY OF MEASUREMENT STATEMENT

The Midwest Roadside Safety Facility (MwRSF) has determined the uncertainty of measurements for several parameters involved in standard full-scale crash testing and non-standard 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.

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| SI* (MODERN METRIC) CONVERSION FACTORS | | | | |
|---|---|---|---|---|
| **APPROXIMATE CONVERSIONS TO SI UNITS** | | | | |
| **Symbol** | **When You Know** | **Multiply By** | **To Find** | **Symbol** |
| | | **LENGTH** | | |
| in. | inches | 25.4 | millimeters | mm |
| ft | feet | 0.305 | meters | m |
| yd | yards | 0.914 | meters | m |
| mi | miles | 1.61 | kilometers | km |
| | | **AREA** | | |
| $in^2$ | square inches | 645.2 | square millimeters | $mm^2$ |
| $ft^2$ | square feet | 0.093 | square meters | $m^2$ |
| $yd^2$ | square yard | 0.836 | square meters | $m^2$ |
| ac | acres | 0.405 | hectares | ha |
| $mi^2$ | square miles | 2.59 | square kilometers | $km^2$ |
| | | **VOLUME** | | |
| fl oz | fluid ounces | 29.57 | milliliters | mL |
| gal | gallons | 3.785 | liters | L |
| $ft^3$ | cubic feet | 0.028 | cubic meters | $m^3$ |
| $yd^3$ | cubic yards | 0.765 | cubic meters | $m^3$ |
| | | NOTE: volumes greater than 1,000 L shall be shown in $m^3$ | | |
| | | **MASS** | | |
| oz | ounces | 28.35 | grams | g |
| lb | pounds | 0.454 | kilograms | kg |
| T | short ton (2,000 lb) | 0.907 | megagrams (or “metric ton”) | Mg (or "t") |
| | | **TEMPERATURE (exact degrees)** | | |
| °F | Fahrenheit | 5(F-32)/9<br>or (F-32)/1.8 | Celsius | °C |
| | | **ILLUMINATION** | | |
| fc | foot-candles | 10.76 | lux | lx |
| fl | foot-Lamberts | 3.426 | candela per square meter | $cd/m^2$ |
| | | **FORCE & PRESSURE or STRESS** | | |
| lbf | poundforce | 4.45 | newtons | N |
| $lbf/in^2$ | poundforce per square inch | 6.89 | kilopascals | kPa |
| **APPROXIMATE CONVERSIONS FROM SI UNITS** | | | | |
| **Symbol** | **When You Know** | **Multiply By** | **To Find** | **Symbol** |
| | | **LENGTH** | | |
| mm | millimeters | 0.039 | inches | in. |
| m | meters | 3.28 | feet | ft |
| m | meters | 1.09 | yards | yd |
| km | kilometers | 0.621 | miles | mi |
| | | **AREA** | | |
| $mm^2$ | square millimeters | 0.0016 | square inches | $in^2$ |
| $m^2$ | square meters | 10.764 | square feet | $ft^2$ |
| $m^2$ | square meters | 1.195 | square yard | $yd^2$ |
| ha | hectares | 2.47 | acres | ac |
| $km^2$ | square kilometers | 0.386 | square miles | $mi^2$ |
| | | **VOLUME** | | |
| mL | milliliter | 0.034 | fluid ounces | fl oz |
| L | liters | 0.264 | gallons | gal |
| $m^3$ | cubic meters | 35.314 | cubic feet | $ft^3$ |
| $m^3$ | cubic meters | 1.307 | cubic yards | $yd^3$ |
| | | **MASS** | | |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.202 | pounds | lb |
| Mg (or "t") | megagrams (or “metric ton”) | 1.103 | short ton (2,000 lb) | T |
| | | **TEMPERATURE (exact degrees)** | | |
| °C | Celsius | 1.8C+32 | Fahrenheit | °F |
| | | **ILLUMINATION** | | |
| lx | lux | 0.0929 | foot-candles | fc |
| $cd/m^2$ | candela per square meter | 0.2919 | foot-Lamberts | fl |
| | | **FORCE & PRESSURE or STRESS** | | |
| N | newtons | 0.225 | poundforce | lbf |
| kPa | kilopascals | 0.145 | poundforce per square inch | $lbf/in^2$ |

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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# 1 INTRODUCTION

## 1.1 Background

In 2009, researchers at the Midwest Roadside Safety Facility (MwRSF) developed, crash tested, and evaluated a low-height, glue-laminated (glulam) timber bridge railing system that was attached to a transverse, nail-laminated deck [1-2]. This study was conducted for the West Virginia Department of Transportation (WVDOT) using the Test Level 1 (TL-1) impact conditions found in the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Assessing Safety Hardware* (MASH) [3]. For this nail-laminated deck, individual 2-in. x 6-in. dimensional lumber boards were nailed together and anchored to steel stringers until the full-length of the bridge was covered using onsite, fabrication methods. This construction process was labor intensive as it required thousands of nails to be inserted into the deck boards using a special rotating nailing pattern and epoxy adhesive lines at the outer ends of transverse boards. Further, the individual boards can often warp, lift, and create an uneven roadway surface or uneven board contact on the stringers.

While the WVDOT TL-1 low-height, glulam timber bridge railing installed on a nail-laminated deck was deemed crashworthy, other user agencies may desire to use this bridge railing system on alternative timber deck types, such as on transverse, glue-laminated (glulam) timber decks. The United States Department of Agriculture (USDA) – Forest Service (FS), National Technology and Development Program desired to utilize this MASH TL-1 bridge railing on timber deck types that are more commonly used on their transportation network. However, there existed questions as to whether the low-height, glulam timber bridge railing would be crashworthy under TL-1 impact conditions and provide sufficient structurally adequacy when installed on alternative, thin, transverse, glulam timber decks. As a result, the USDA-FS, National Technology and Development Program requested assistance to adapt the TL-1 low-height, glulam timber bridge railing system over for use on transverse, glulam timber decks and verify that it would provide equal to greater safety performance and structural adequacy as compared to that behavior observed when it was installed on a transverse, nail-laminated deck.

## 1.2 Objective

The objectives for this project included (1) the development of the necessary details to adapt the 2009 AASHTO *Manual for Assessing Safety Hardware* (MASH) [3] WVDOT TL-1 low-height bridge railing system for use on a typical USDA-FS transverse, glulam timber deck [1-2] and (2) demonstration that the TL-1 low-height, bridge rail would meet the 2016 MASH TL-1 impact safety standards by proving equivalent or greater lateral stiffness and strength when installed on the transverse, glulam deck as compared to its performance observed when installed on the as-tested, nail-laminated deck [4].

## 1.3 Research Approach

To begin this project, a literature review was conducted to identify prior research on the development, crash testing, and evaluation of low-height, bridge railings and containment barriers. Further, the investigation also focused on those systems that were attached to timber decks as well as concrete foundations. Researchers also acquired and reviewed relevant bridge railings and deck system details used across the road network within the National Technology and Development

Program, which included the applicable design charts and manuals for configuring typical transverse glulam deck systems, laying out glulam girders and diaphragms, and detailing the various connections. Using the noted information along with sponsor feedback, MwRSF configured 3-D test plans and CAD details for constructing one surrogate glulam bridge deck system and one surrogate nail-laminated bridge deck system. Each bridge system had two short glulam rail segments supported and anchored to the deck using two scupper blocks. One static and one dynamic component test was conducted on each deck type and analyzed to compare lateral stiffness, strength, energy dissipation, and overall performance between deck types. Finally, the component test results were compared to one another, and conclusions were drawn regarding the performance and crashworthiness of the low-height, glulam bridge rail installed on both transverse glulam and transverse nail-lam bridge decks.

# 2 LITERATURE REVIEW

## 2.1 Introduction

For this project, an in-depth study was conducted in order to identify previously-developed low-height, bridge railings and barriers as well as end treatments that would inform the adaptation process involving the WVDOT TL-1 low-height, glulam timber guardrail and end terminal system utilized in this study. Within the literature review, details specifically pertaining to relevant bridge railings and barriers as well as end treatments have been separated into Sections 2.2 and 2.3, respectively.

Further, relevant bridge design manuals that are used by the Missoula Technology and Development Division were reviewed to assist with the design, layout, and configuration of the surrogate glulam timber deck system. This deck system was later constructed for use in the testing and evaluation program involving the glulam bridge rail segment supported by scupper blocks. Discussion of the relevant design manuals was also provided in Section 2.4. Further, the Missoula Technology and Development Division personnel were queried on occasion to answer questions, provide additional details, assist with design guidance, help with the selection of a representative glulam timber deck system, and finalize the surrogate glulam timber bridge deck system.

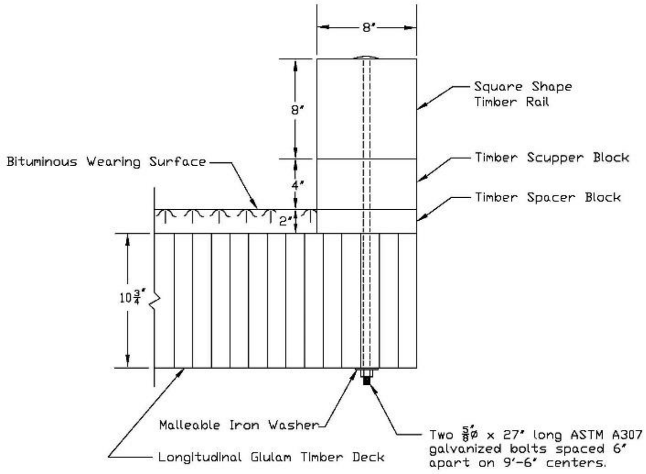
## 2.2 Low-Height Timber Bridge Rails

### 2.2.1 Low-Height, Sawn Timber Bridge Railing [5-11]

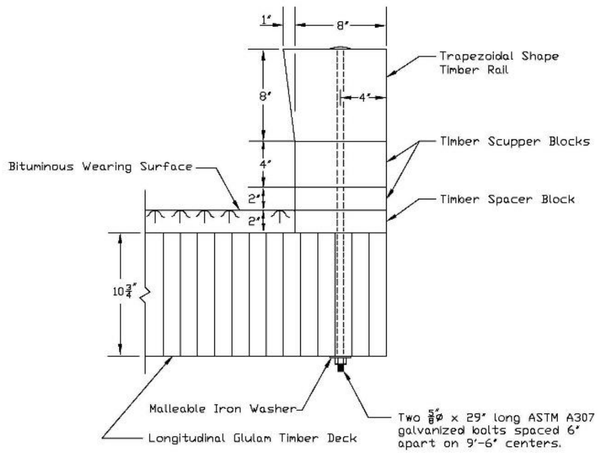
In 1993, MwRSF researchers designed a low-height, curb type, timber bridge railing for the United States Department of Agriculture – Forests Service – Forest Products Lab (USDA-FS-FPL). The project fulfilled a need to provide a low cost, low performance, bridge rail system for use on bridge decks found on low-volume roadways. As part of this study, three solid, sawn timber railing shapes were developed. These rail shapes included (1) an 8-in. x 8-in. square cross section, (2) a 9-in. x 8-in. trapezoidal cross section, and (3) a 4-in x 12-in. rectangular cross section. From the three cross sectional options, the square shape was selected to be mounted on solid sawn timber scupper blocks, thereby comprising the timber bridge railing. The selected square bridge rail supported on scupper blocks was then attached to a longitudinal glulam timber deck for full-scale crash testing. The remaining two cross sectional shapes were not examined with full-scale crash tests but rather R&D live-driver testing. However, researchers reported that the behavior of all three barrier shapes would likely perform similarly if subjected to the same full-scale crash test.

The three timber bridge rail shapes are shown in Figure 1, including details for the mounting hardware and solid sawn timber scupper blocks. Additional pictures of the crash tested timber deck and railing system have been provided in Figure 2. The bridge railing system was attached to a 10¾-in. thick longitudinal glulam timber deck system.

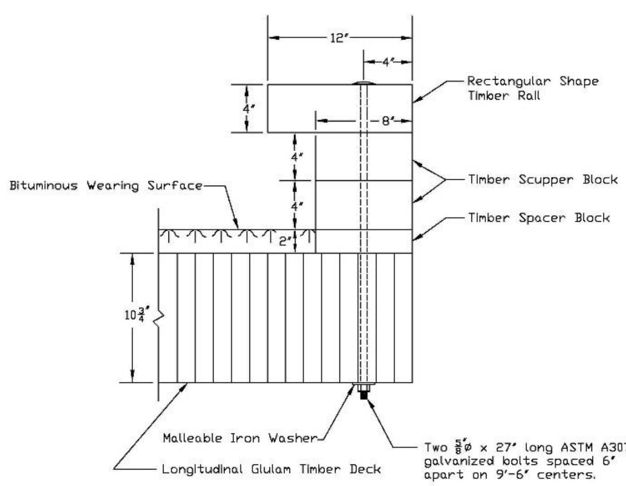
The 12-in. tall, low-height, sawn timber bridge railing was subjected to one full-scale crash test involving a 4406-lb pickup truck impacting at a speed of 14.4 mph and at an impact angle of 15 degrees. The crash test results indicated that the barrier was adequate for containing passenger vehicles sub-TL-1 impact conditions found in the NCHRP Report No. 350 impact safety standards [12].




(a) 8-in. x 8-in. Square Railing




(b) 9-in. x 8-in. Trapezoidal Railing




(c) 4-in x 12-in. Rectangular Railing

Figure 1. Design Details for NCHRP 350 Sub-TL-1 Low-Height, Timber Bridge Railing [5]

(a) Front View

(b) Lap Splice Connection

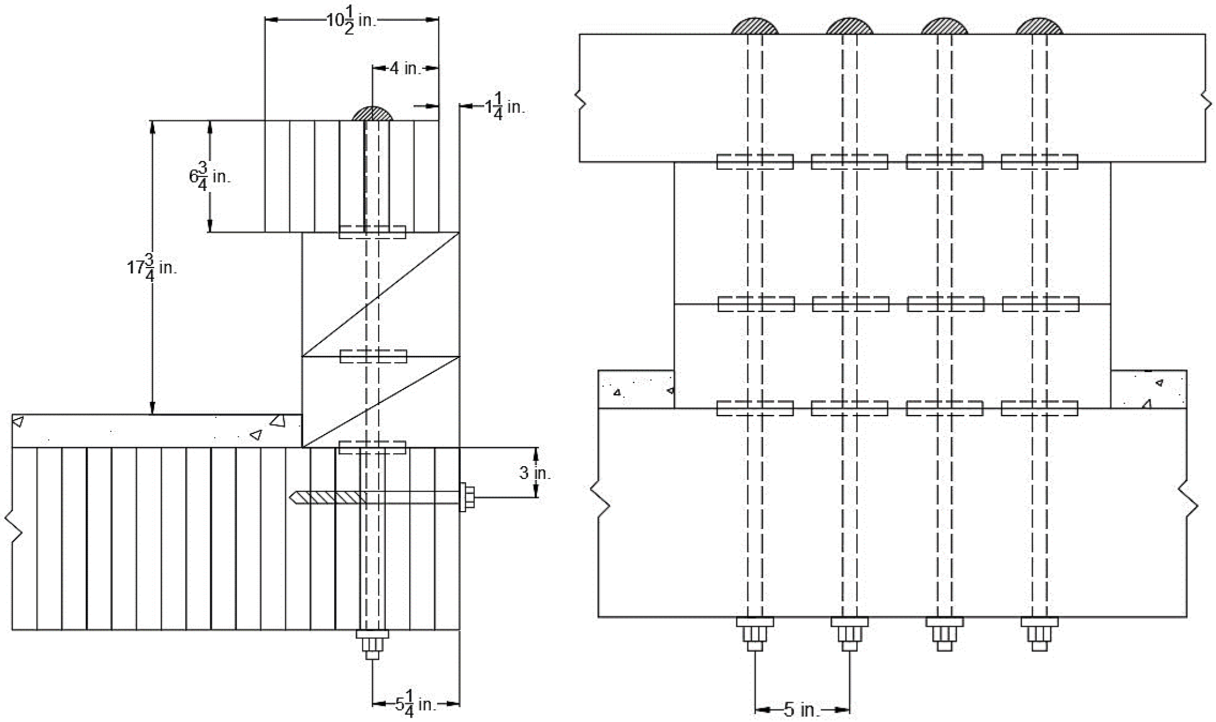
Figure 2. Low-Height, Timber Bridge Railing - 12-in. Tall, Square Rail Shape [5]

### 2.2.2 Low-Height, Glulam Timber Bridge Railing [7-14]

In 1995, MwRSF researchers and USDA-FS-FPL personnel collaborated on the development of a low-height, glulam timber bridge rail for use on low volume roads. The bridge rail consisted of two solid sawn scupper blocks stacked on top of one another with a rectangular, glulam timber railing mounted on top of the scupper blocks. The top scupper block measured 7½ in. tall x 9½ in. wide x 23 in. long, and the bottom scupper block measured 5½ in. tall x 9½ in. wide x 23 in. long. Both scupper blocks were fabricated from S4S Grade No.1 Douglas Fir material and were treated with creosote. The rail segments were 19 ft - 11½ in. long and measured 6¾ in. tall x 10½ in. wide. The material selected for the rail segments was Combination No. 2 West Coast Douglas Fir and was treated with pentachlorophenol in heavy oil. Design drawings and photographs of the crash tested bridge railing system are provided Figures 3 and 4, respectively. The bridge railing system was attached to a 10¾-in. thick longitudinal glulam timber deck system.

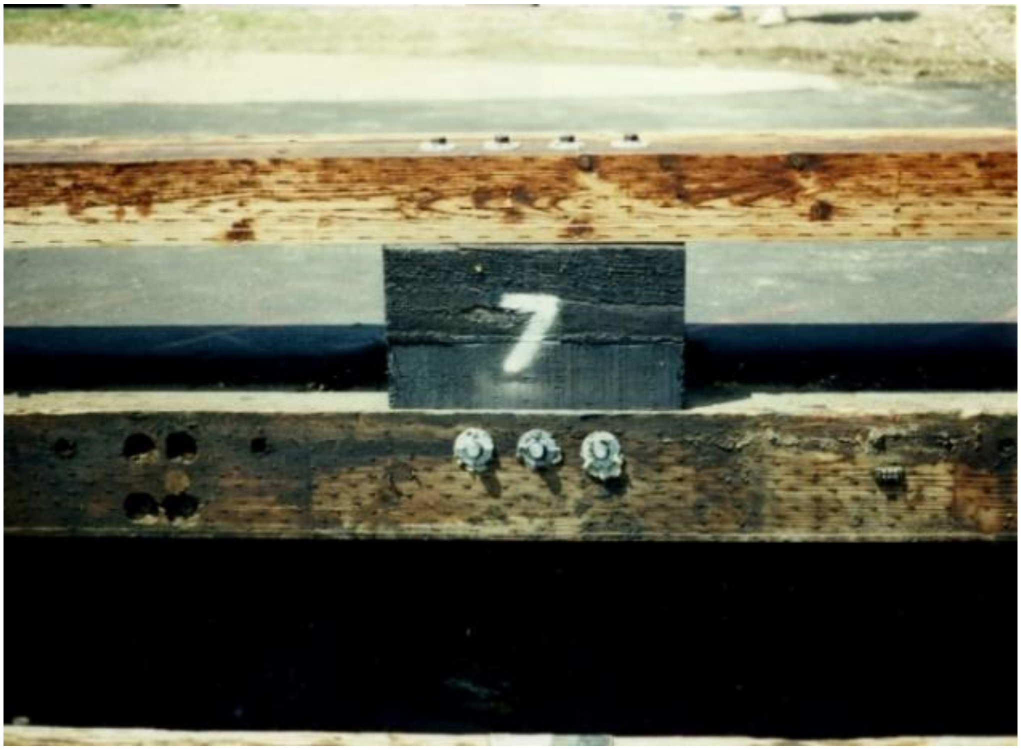
The 17¾-in. tall, low-height, glulam timber bridge railing was subjected to one full-scale crash test involving a 4435-lb pickup truck impacting at a speed of 31.6 mph and at an impact angle of 24.3 degrees. The bridge railing system was found to satisfactorily meet the TL-1 impact conditions found in NCHRP Report No. 350 [12].

Note that this bridge railing system was later modified to meet MASH TL-1 impact conditions when anchored to a nail-laminated timber bridge deck for the WVDOT. Discussion of this follow-on investigation is provided in much greater detail in Section 2.2.3 of this report.




(a) Side View (b) Back View

Figure 3. Schematic of NCHRP 350 TL-1 Low-Height, Glulam Timber Bridge Railing – (a) Side View and (b) Back View [13]




(a) Scupper Block and Glulam Rail Connection

(b) Upstream End View

Figure 4. NCHRP 350 TL-1 Low-Height, Glulam Timber Bridge Railing – (a) Scupper Block and Glulam Rail Connection and (b) Upstream End View [13]

### 2.2.3 WVDOT Low-Height, Glulam Timber Bridge Railing [1-3]

In 2009, MwRSF developed a MASH TL-1 low-height, timber glulam bridge railing for use on a transverse, nail-laminated timber bridge deck with a sloped end treatment for the WVDOT. The bridge rail consisted of two sawn timber scupper blocks stacked on top of one another with a rectangular, glulam timber railing mounted on top of the scupper blocks. The top scupper block measured 7½ in. tall x 9½ in. wide x 23 in. long, and the bottom scupper block measured 7½ in. tall x 9½ wide x 23 in. long. Both scupper blocks were fabricated from S4S Grade No.1 Southern Yellow Pine material and were treated with pentachlorophenol in heavy oil. The rail segments were 19 ft – 11¼ in. long and measured 6¾ in. tall x 12⅜ in. wide. The material selected for the rail segments was Combination No. 48 Southern Yellow Pine and was treated with pentachlorophenol in heavy oil. The bridge railing system was attached to a 5½-in. thick transverse, nail-laminated timber deck system that was configured using 2-in. x 6 in. dimensional lumber.

For the research and development program, five static component tests were conducted on individual post setups that were configured using a 23-in. long segment of the glulam timber rail supported by two scupper blocks, one stacked on top of the other. The rail segment and two scupper blocks were connected to the transverse, nail-laminated timber deck using four ¾-in. diameter x 30-in. long ASTM A307 timber bolts. Component details, dimensions, and material properties for the static testing program are provided in Table 1. Figure 5 provides a schematic for the static testing components that were anchored to the transverse, nail-laminated timber deck.

Table 1 Bridge Railing Components and Parameters - Static Testing Program

| Component | Parameter | Value |
|---|---|---|
| Scupper Blocks | Length (in.) | 23 |
| | Width (in.) | 9.5 |
| | Height (in.) | 7.5 |
| | Grade, Species | No. 1, SYP |
| Rail Segment | Length (in.) | 23 |
| | Width (in.) | 12.375 |
| | Height (in.) | 6.75 |
| | Grade, Species | Combination 48, SYP |
| Vertical Timber Bolts | Length (in.) | 30 |
| | Grade/Specification | ASTM A307 |
| | Bolt Diameter (in.) | 0.75 |

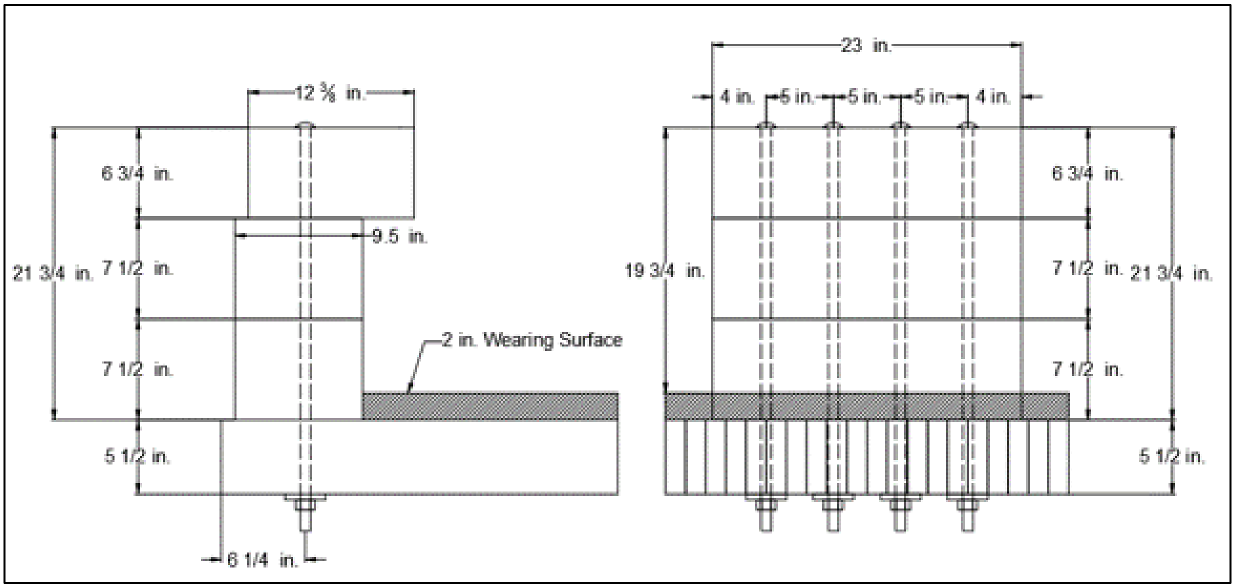



Figure 5. Bridge Railing Schematic - WVDOT Static Testing Program – Side View and Front View [1]

For test nos. WVS-1 and WVS-4, vertical timber bolts were used to connect the components. For test nos. WVS-2, WVS-3, and WVS-5, vertical timber bolts were used with either split rings or shear plates at the various interfaces. The connection details for the different post configurations are provided in Table 2. The lateral force versus deflection curves from the five static component tests were over-plotted and are shown in Figure 6. Note that Figure 6 provides two curves for test no. WVS-1. During test no. WVS-1 (1st portion), instrumentation issues occurred during the initial static loading process. As such, the static test was stopped to modify the testing apparatus. Then, test no. WVS-1 (2nd portion) was restarted to obtain the necessary results.

Table 2. Static Testing Plan and Connection Details

| Test No. | Scupper Block Shear Connection Details |
|---|---|
| WVS-1 | Timber bolts through both scupper blocks and rail segment |
| WVS-2 | Timber bolts through both scupper blocks and rail segment, split rings between scupper blocks, bottom scupper blocks and deck, & top scupper blocks and rail segment |
| WVS-3 | Timber bolts through both scupper blocks and rail segment, shear plates between scupper blocks, bottom scupper blocks and deck, & top scupper blocks and rail segment |
| WVS-4 | Timber bolts through both scupper blocks and rail segment |
| WVS-5 | Timber bolts through both scupper blocks and rail segment, & split rings between bottom scupper blocks & deck |

After completing the static component testing program, MwRSF and WVDOT personnel determined that the rail segment and scupper block connection detail utilized in test nos. WVS-1 and WVS-4 was satisfactory, cost effective, and should be subjected to a full-scale crash testing and evaluation program.

Using the preferred rail and scupper block configuration from test nos. WVS-1 and WVS-4, one full-scale vehicle crash test was conducted on a 19¾-in. tall, low-height, glulam timber bridge railing anchored to a full-size, transverse, nail-laminated bridge deck system supported by steel stringers. The crash test was performed according to the MASH TL-1 impact conditions using a 2270P pickup truck [1-3]. During the crash test, the bridge railing was subjected to a lateral dynamic deflection (D.D.) of 6.1 in. and a lateral permanent set (P.S.) of 2.4 in. Photographs of the bridge railing and deck system for the full-scale crash test are provided in Figure 7. After an analysis of the test results, it was concluded that the low-height, glulam timber bridge railing installed on a transverse, nail-laminated deck adequately met the AASHTO MASH TL-1 impact safety standards [3].

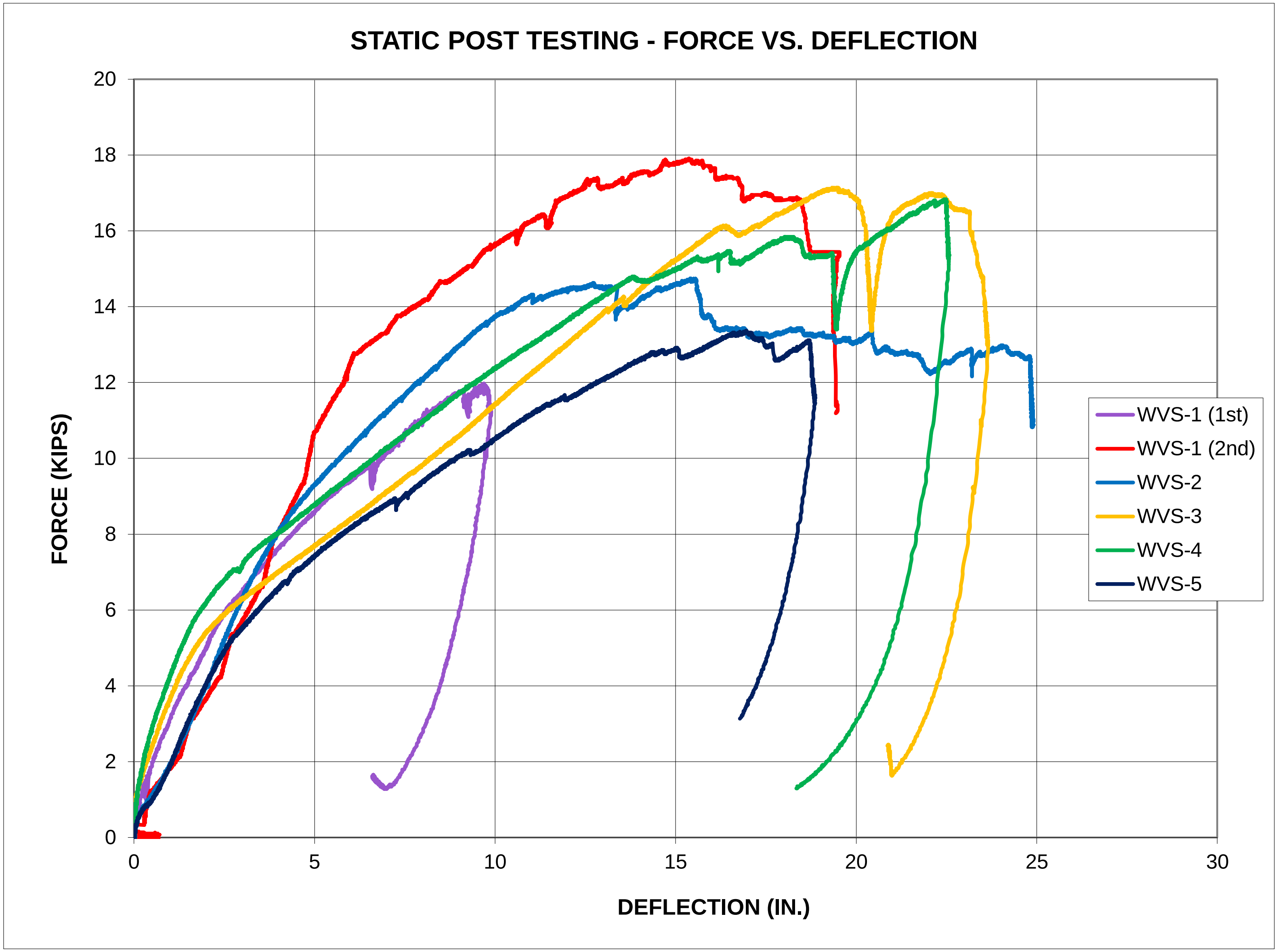



Figure 6. WVDOT Static Component Testing Results – Force Versus Deflection

(a) Front View

(b) Back View

Figure 7. MASH TL-1 Low-Height, Glulam Timber Bridge Railing on Transverse, Nail-Laminated Deck – (a) Front View and (b) Back View [1]

## 2.3 Sloped End Treatments for Low-Height Barriers

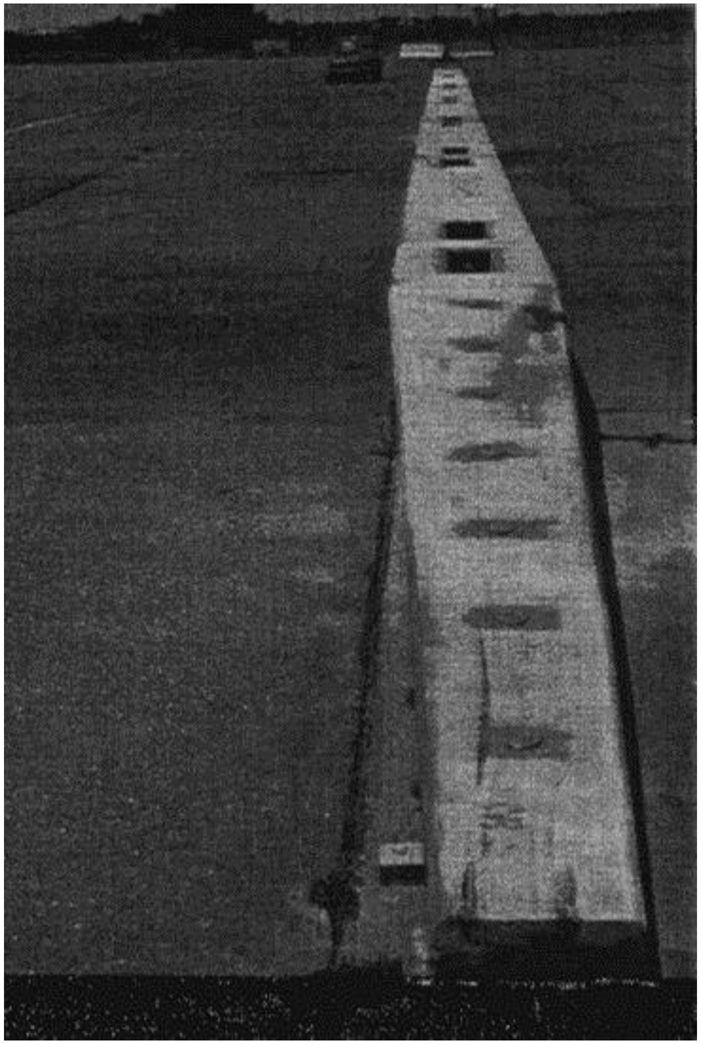
### 2.3.1 TTI Concrete End Treatment [15-16]

In 1998, the Texas A&M Transportation Institute (TTI) developed a sloped concrete end treatment for use with a low-height, concrete work-zone barrier [15]. The sloped end treatment had an upstream end configured with a 4-in. tall blunt end measuring 14.4 in. wide, while its downstream end measured 20 in. tall and 28 in. wide. Additionally, the overall length of the treatment was 15 ft. The concrete end treatment was anchored to the road surface using seven steel pins spaced 24 in. apart from one another and inserted through the segment and road surface at its centerline. The end treatment was crash tested using four small cars and three pickup trucks in accordance with Test Level 2 safety performance criteria found in the NCHRP Report No. 350 impact safety standards [12]. The seven crash test designation nos. and associated impact conditions used for the concrete end treatment are provided in Table 3. Following the completion of the full-scale crash testing program, the concrete end treatment was deemed crashworthy according to the TL-2 impact conditions published in the NCHRP Report No. 350 impact safety standards. Photographs of the TTI low-height, sloped concrete end treatment are provided in Figure 8.

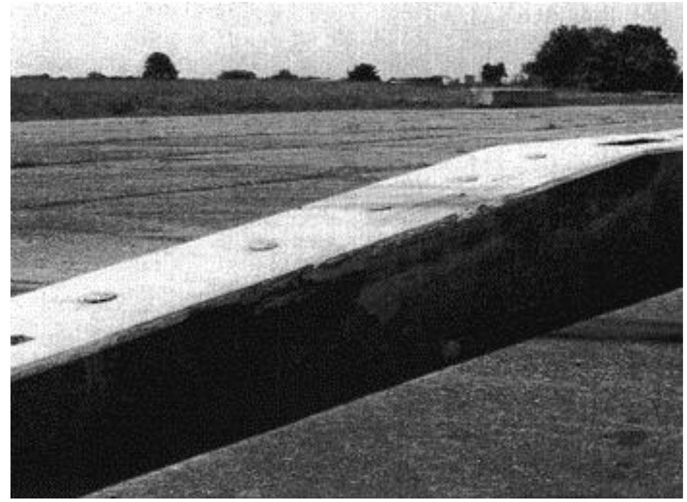
In 2013, TTI researchers modified the sloped concrete end treatment by removing the seven steel drop pins that were used to anchor the end section [16]. TTI then subjected the free-standing sloped concrete end treatment to two full-scale crash tests using the MASH TL-2 impact safety standards with a small car and a pickup truck, as summarized in Table 3 [4]. Following the completion of the full-scale crash testing program, the modified, sloped concrete end treatment was deemed crashworthy according to the MASH TL-2 impact safety standards.

Table 3. TTI Crash Tests on Low-Height, Sloped Concrete End Treatment

| Reference | Vehicle Type | Crash Test Designation No. | Target Impact Angle (degrees) | Target Impact Speed (mph) | Location of Impact |
|---|---|---|---|---|---|
| 1998 TTI [15] | Small Car | 350 2-30 | 0 | 43.5 | End of Terminal |
| | | 350 2-32 | 15 | 43.5 | End of Terminal |
| | | 350 2-34 | 15 | 43.5 | Critical Impact Point |
| | Pickup Truck | 350 2-31 | 0 | 43.5 | End of Terminal |
| | | 350 2-33 | 15 | 43.5 | End of Terminal |
| | | 350 2-35 | 20 | 43.5 | Beginning of Length of Need |
| | | 350 2-39 | 20 | 43.5 | Mid Length of Terminal |
| 2013 TTI [16] | Small Car | MASH 2-34 | 15 | 44 | Critical Impact Point |
| | Pickup Truck | MASH 2-35 | 25 | 44 | Beginning of Length of Need |



End View



Isometric View

Small Car Vehicle

Figure 8. Low-Height, Sloped Concrete End Treatment [15]

### 2.3.2 Test Level 2, Low-Profile, Concrete Bridge Railing with Sloped End Treatment [17]

In 2002, MwRSF researchers completed a Midwest Pooled Fund Program (MPFP) study to develop, test, and evaluate a low-height, reinforced concrete bridge railing to meet TL-2 impact safety standards published in NCHRP Report No. 350 [12, 17]. The 20-in. tall bridge railing was configured with a top width of 14 in. and base width of 11 in., as depicted in Figure 10. The bridge railing utilized a rectangular shape as the upper beam and a narrow, lower vertical wall to support the beam. Overall, the bridge railing generally appeared to be an upside-down "L" shape with the top section extending forward from the vertical wall, which was intended to reduce wheel climb during impact events. The concrete bridge rail was subjected to one full-scale crash test with 2000P pickup truck and resulted in satisfactory safety performance according to the TL-2 criteria found in NCHRP Report No. 350.

For the end treatment, the 20-in. tall reinforced-concrete bridge railing was configured to slope downward to the roadway surface using the same vertical slope that was utilized for the TTI sloped concrete end treatment [15-16]. The sloped, reinforced-concrete end treatment was 15 ft long with an upstream height of 4 in. and width of 14 in., as shown in Figure 10. Using the noted configuration and geometry, MwRSF researchers deemed it unnecessary to conduct additional crash testing on the sloped concrete end treatment beyond that testing already conducted by TTI researchers [15].

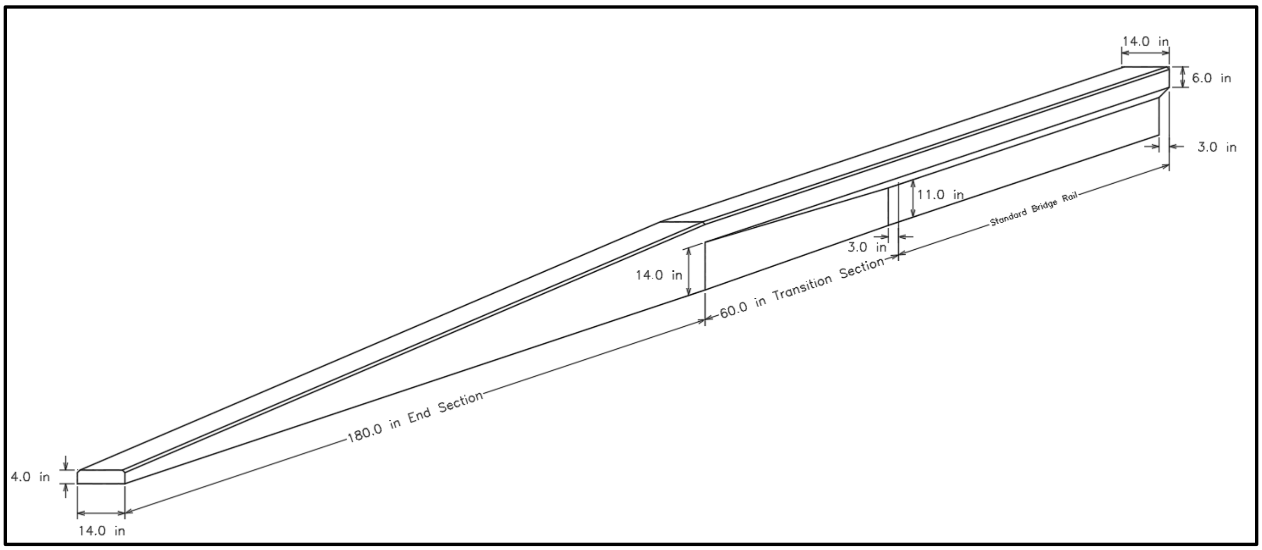



Figure 9. Original MPFP Bridge Rail with Sloped Concrete End Treatment [17]

### 2.3.3 USDA-FS-MT&D TL-1 Low-Profile, Concrete Bridge Railing with Sloped End Treatment [18]

In 2020, USDA-FS-MT&D contracted with MwRSF to develop a MASH TL-1 version of the prior MPFP NCHRP Report No. 350 TL-2 low-height, reinforced concrete bridge railing with sloped concrete end section [17] without the need for full-scale vehicle crash testing. During the recent R&D effort [18], the width of the bridge railing was reduced by 4 in. The longitudinal and vertical steel reinforcement in the bridge rail was also modified. Due to the minor modifications made to the bridge rail, no full-scale or component crash testing was required when considering that the system would only need to meet MASH TL-1 and be configured with a 20-in. top rail height.

The original sloped end treatment that was connected to the bridge rail was also modified. The width of the end treatment was also reduced by 4 in., and the steel reinforcement was modified slightly. Since the changes to the end treatment were minor, no full-scale crash testing was deemed necessary. Figure 10 provides a schematic of the adapted concrete, low-height, sloped end treatment [18].

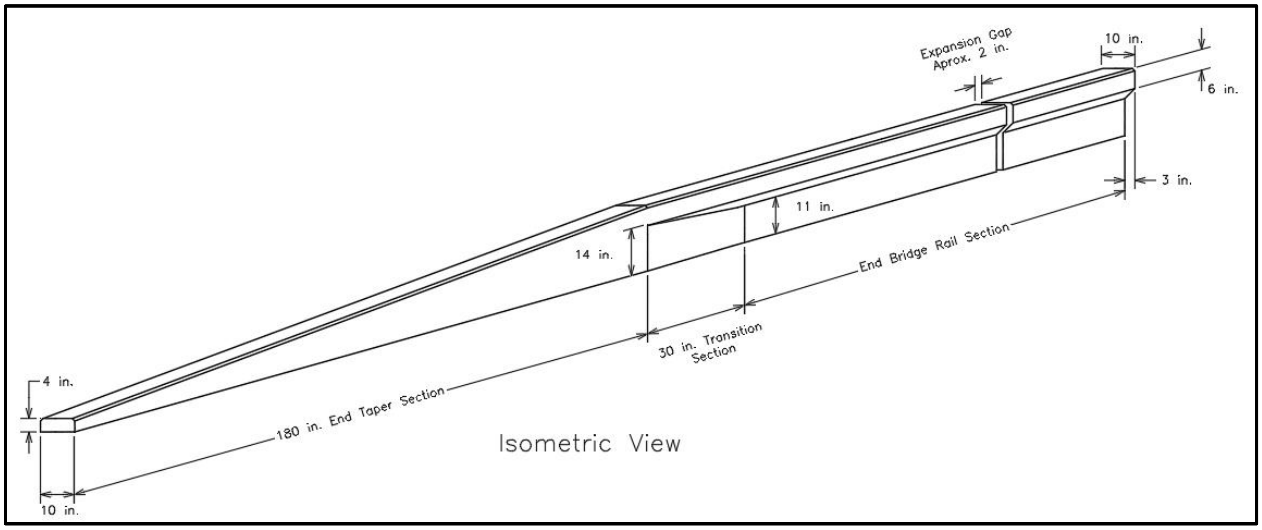



Figure 10. USDA-FS-MT&D Bridge Rail with Sloped Concrete End Treatment [18]

### 2.3.4 WVDOT Timber Sloped End Terminal [1-2]

In the West Virginia DOT study [1-2], a timber sloped end terminal was developed to properly treat the end of the bridge railing system. The geometry of the timber sloped end terminal was largely based on the geometry used in TTI's crash-tested system [15-16], which later adapted to treat the end of the low-height, concrete bridge railing [17]. The timber end treatment utilized a 35-ft long glulam rail segment that attached to the upstream end of the 19¾-in. tall, low-height, timber bridge rail. The last 15 ft of glulam timber rail was sloped downward toward the ground to create a top rail height of 4 in. above grade at the end of the treatment. To support the timber sloped end terminal beyond the bridge deck, four 6-ft long, W6x15 steel posts were attached to the underside of the glulam rail and embedded into the soil. Since this sloped end terminal was similar to the previously crash-tested TTI terminal, no crash testing was performed. Figure 11 provides multiple views of the timber sloped end terminal.

(a) Back Face

(b) Front Face

Figure 11. WVDOT Timber, Sloped End Terminal - (a) Back Face and (b) Front Face [1]

### 2.3.5 Comparison of Sloped End Terminals

In 1998, TTI researchers developed and successfully crash tested a concrete sloped end terminal under NCHRP Report 350 TL-2 impact conditions. Later, TTI's sloped end terminal was adapted for use on several different barriers and bridge railings. In each of the adapted versions, changes to TTI's original design were deemed to be minimal; therefore, researchers believed that additional crash testing was unnecessary. A comparison of the general geometries for the sloped end terminals is provided in Table 4. Other system details are available in references [1-2, 15-18].

Table 4. Comparison of Sloped End Terminals

| System Parameter | 1998 350 TL-2 TTI Concrete Treatment [15] | 2002[1] 350 TL-2 NDOT Concrete Treatment [17] | 2013 MASH TL-2 TTI Concrete Treatment [16] | 2009[1] MASH TL-1 WVDOT Timber Treatment [1-2] | 2022[1] MASH TL-1 USDA-FS-NTDP Concrete Treatment [18] |
|---|---|---|---|---|---|
| Barrier Height (in.) | 20 | 20 | 20 | 19¾ | 20 |
| Top Barrier Width (in.) | 28 | 14 | 28 | 12⅜ | 10 |
| Lower Sloped End Height (in.) | 4 | 4 | 4 | 4[2] | 4 |
| Lower Sloped End Width (in.) | 14.4 | 14 | 14.4 | 12⅜ | 10 |
| Sloped End Section Length (ft) | 15 | 15 | 15 | 15 | 15 |

NTDP – National Technology & Development Program
[1] Crash testing not performed as system geometry deemed to be similar to prior systems.
[2] Sloped end is partially buried with height above grade varying between $1^{9}/_{16}$ in. and 4 in.

## 2.4 Standard Plans for Timber Bridge Superstructures

To adapt the TL-1 low-height, glulam bridge railing system for use on standard USDA-FS-NTDP transverse, glulam timber decks, a thorough review was performed on standard plans pertaining to timber bridge superstructures, as developed by the USDA-FS-FPL, and on other MwRSF bridge railing development projects involving transverse, glulam timber decks.

The USDA-FS-FPL published a standard plan document in 2001, *Standard Plans for Timber Superstructures* [19]. This document contains information on several typical timber deck bridge types utilized by the Forest Service, including transverse, glulam deck systems. Other information includes guidelines for design loadings, component dimensions, material grades and specifications, and construction procedures. Tables provide guidance on girder sizes based on span lengths for different bridge configurations. Further information covers attachment techniques used to connect glulam deck panels to glulam or steel girders, girders to bents and abutments, and addresses diaphragm spacing and connections. In 2019, an updated standard plan document was published by the Forest Service, *Standard Plans for Glued-Laminated Timber Bridge Superstructures* [20], which contains updated guidance on the same topics in regard to the transverse, glulam timber bridges that were included in the 2001 standard plan document [19].

# 3 STATIC AND DYNAMIC TEST PLAN DEVELOPMENT

## 3.1 Test Plan Requirements

For this project, it was necessary to demonstrate that the previously-developed, low-height, bridge railing system would provide adequate strength when implemented on a transverse, glulam timber deck. As part of this effort, static testing and dynamic testing were planned on surrogate bridge rail sections that were supported by scupper blocks and anchored the two deck types. This testing layout would be similar to that used in the static testing program for the original bridge railing development [1-3]. Thus, the research team developed a series of requirements to ensure that adequate and reliable data would be acquired to observe the necessary behaviors to compare deck systems and show that the bridge rail could also be used on transverse, glulam timber decks.

The testing program needed to replicate the original research and development effort by using the same materials for all components and constructing the surrogate rail and scupper block system in the same manner. For tests conducted on the transverse, nail-laminated, timber deck, the deck construction process was identical to that used in the prior study. The deck superstructure included the same girders, girder spacing, overhang length, attachment mechanism for the girder-to-deck connection, and nailing pattern within the deck. For tests conducted on the transverse, glulam timber deck, a worst-case design scenario was necessary to ensure that the minimum or critical system strengths would be obtained and compared to the system strengths observed when implemented on a transverse, nail-laminated timber deck, as previously configured. To obtain this scenario, typical Forest Service glulam decks were reviewed, and critical design factors were considered, such as minimum deck thickness and weakest connection mechanism between girders and deck panels.

A full size, bridge railing system was not needed for this component testing program. Thus, the length of bridge system for each deck type only needed to account for the length of deck that would experience the distributed load and resulting deflections, which was likely dependent on the connection between adjacent deck members and the connection between the deck and girders. Therefore, video data from the full-scale crash test [1-2] was reviewed to determine the length of transverse, nail-laminated, timber deck assuming an estimated load distribution with associated deformations. Next, the length of transverse, glulam timber deck that was required for the component testing program was based on the number of tests, the load distribution within deck panels, and the space required to conduct impact tests with a surrogate vehicle. The final design details for each testing program are discussed in subsequent sections.

# 4 TRANSVERSE, NAIL-LAMINATED TIMBER DECK AND RAIL

## 4.1 Design Decisions

As noted previously, the deck details utilized in the WVDOT study were replicated for this study. The only design modification pertained to the length of the transverse, nail-laminated, timber deck. Since no full-scale crash testing was to be performed, a shorter bridge length was acceptable. A 10-ft centerline distance between scupper blocks was maintained. The length of the deck from each scupper block to the end of deck was determined to be at least 8 ft, thus resulting in an initial deck length of 26 ft instead of 120 ft, as used in the crash testing program. Although 26 ft was initially planned, the final constructed length of deck was 31 ft. For discussion on the construction process that resulted in the deck length increasing from 26 to 31 ft as well as its shifted position, Appendix B.

## 4.2 Superstructure and Substructure

The transverse. nail-laminated, timber bridge deck had a width of approximately 14 ft and a spanning length of 31 ft. Additionally, the nail-laminated deck had an overhang length of 50$\frac{1}{16}$ in., as measured from the edge of the deck to the vertical centerline of the exterior-most girder. The deck surface was constructed utilizing 2-in. x 6-in. x 14-ft long, grade No. 1 Southern Yellow Pine (SYP), dimensional lumber boards. For the component testing on the nail-laminated deck, the boards were treated with 0.15 lb/ft$^3$ of micronized copper azole. However, it is recommended that 0.60 lb/ft$^3$ of pentachlorophenol in heavy oil, or other similar treatment, be used to preserve the lumber boards comprising this deck type. The boards were all placed on end and were then nailed together using 20d or 20 penny “common” nails. The 20d nails were inserted perpendicular to, and through the wide face of the boards. The construction process for the transverse, nail-laminated, timber deck is shown in Figure 12. In addition, Figures 13 through 37 provide the plan drawings for the nail-laminated, timber deck. Since each board was nailed to adjacent boards, the deck was considered to be structurally continuous. A specific nail pattern was developed and repeated every four boards to prevent the nails inserted in one board from hitting the nails in adjacent boards. The special nail pattern also considered the location of vertical bolts that were used to anchor the scupper blocks to prevent nails from contacting timber bolts. The nailing pattern for the boards is shown below in Figures 19 and 20. The boards were further connected to one another using a minimum of two beads of liquid nails at the outer 3 ft of the 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 to improve load transfer between boards [1].

Steel anchor brackets were utilized to anchor the deck to the girders. The anchor brackets were placed between two adjacent boards and were connected to the boards using two 20d nails. To connect the brackets to the girders supporting the deck, the anchor brackets were slotted over either side of the inner most top flange girder, and over the inside facing top flange of the exterior facing girder. For more details on the anchor bracket pattern used, see Figures 23 and 24. The brackets were manufactured from 11-gauge, ASTM A653 G90 galvanized steel sheet. The anchor brackets were cut directly from this sheet to the dimensions shown in Figure 23.

Once the nail-laminated deck was fully assembled and anchored to the steel girders, a 2-in. thick, surrogate wearing surface was placed over the timber deck. For testing purposes, two sheets of plywood with spacer boards were laid on top of one another and placed over the deck.

This surrogate wearing surface was placed on the timber deck for the testing and evaluation program to reduce construction costs. For the long-term use of the bridge system, it is recommended that a permanent, durable material (i.e., asphalt) be used to surface and protect the timber deck.

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. To laterally brace the steel girders between the concrete supports, C-channel beams were utilized at intervals of approximately 12.5 ft along the length of the girders. As noted in Section 5.1, the transverse, nail-laminated, timber deck was utilized in a prior MwRSF research project that was conducted for the WVDOT. Since the girders, bents, and abutments from this previous study were still installed and in good condition, they were reused for this project's component testing program. In the previous study, a deck length of 120 ft was used to conduct a full-scale, MASH TL-1 crash test. However, only 26 ft of deck was needed for the necessary component testing program. Initially, the 26-ft long deck was center in a 40 ft span of the 120-ft long bridge system. Appendix B, the 26-ft long deck required an additional 5 ft and was shifted slightly. The modifications to the deck span are shown in Figure 13.

(a) Construction of Nail-Laminated Deck

(b) Anchor Brackets on Underside of Nail-Laminated Deck

(c) Side View of Nail-Laminated Deck

(d) Back View of Nail-Laminated Deck

Figure 12. Construction of Transverse, Nail-Laminated, Timber Deck

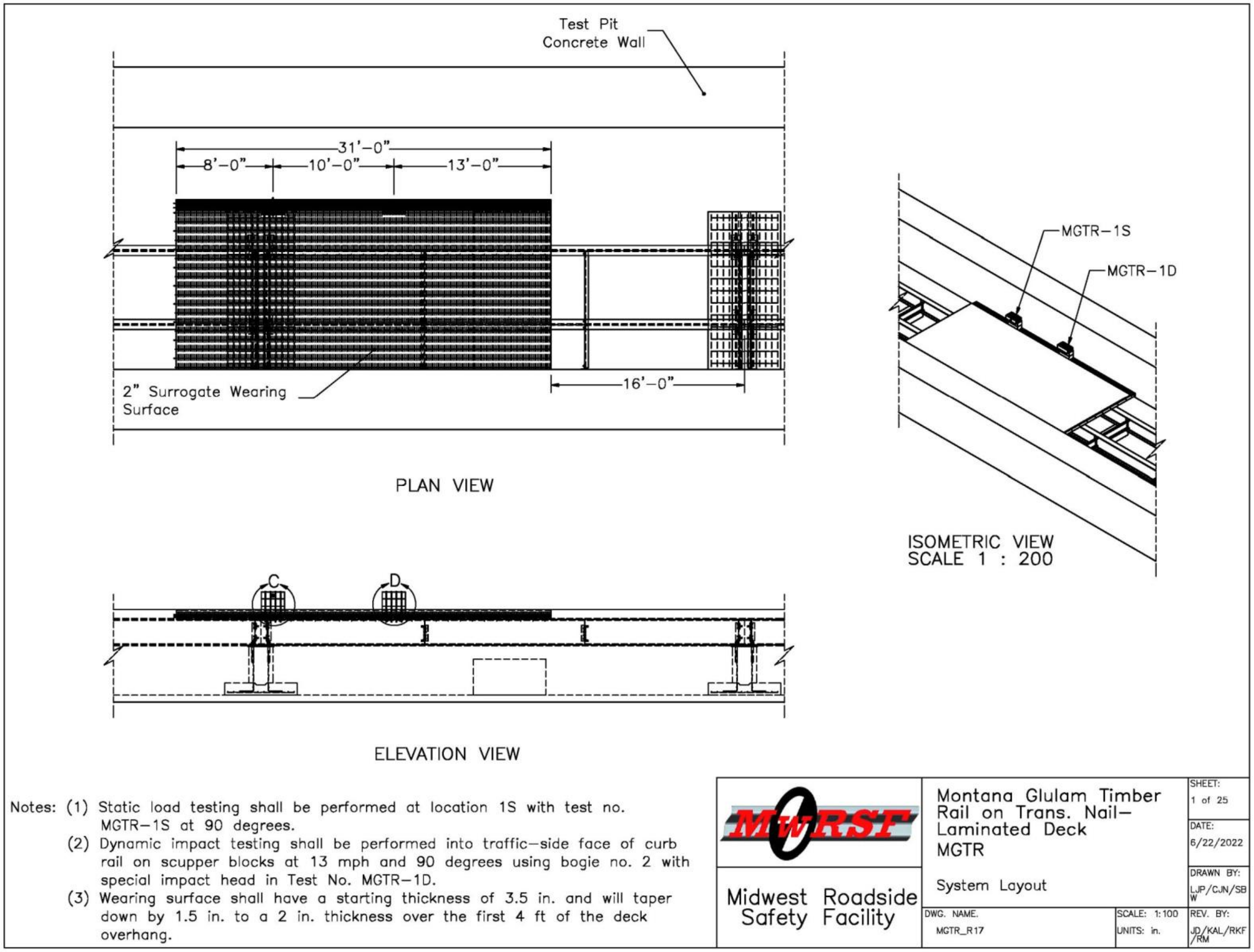



Figure 13. Transverse, Nail-Laminated, Timber Deck and Test Nos. MGTR-1S and MGTR-1D System Layout

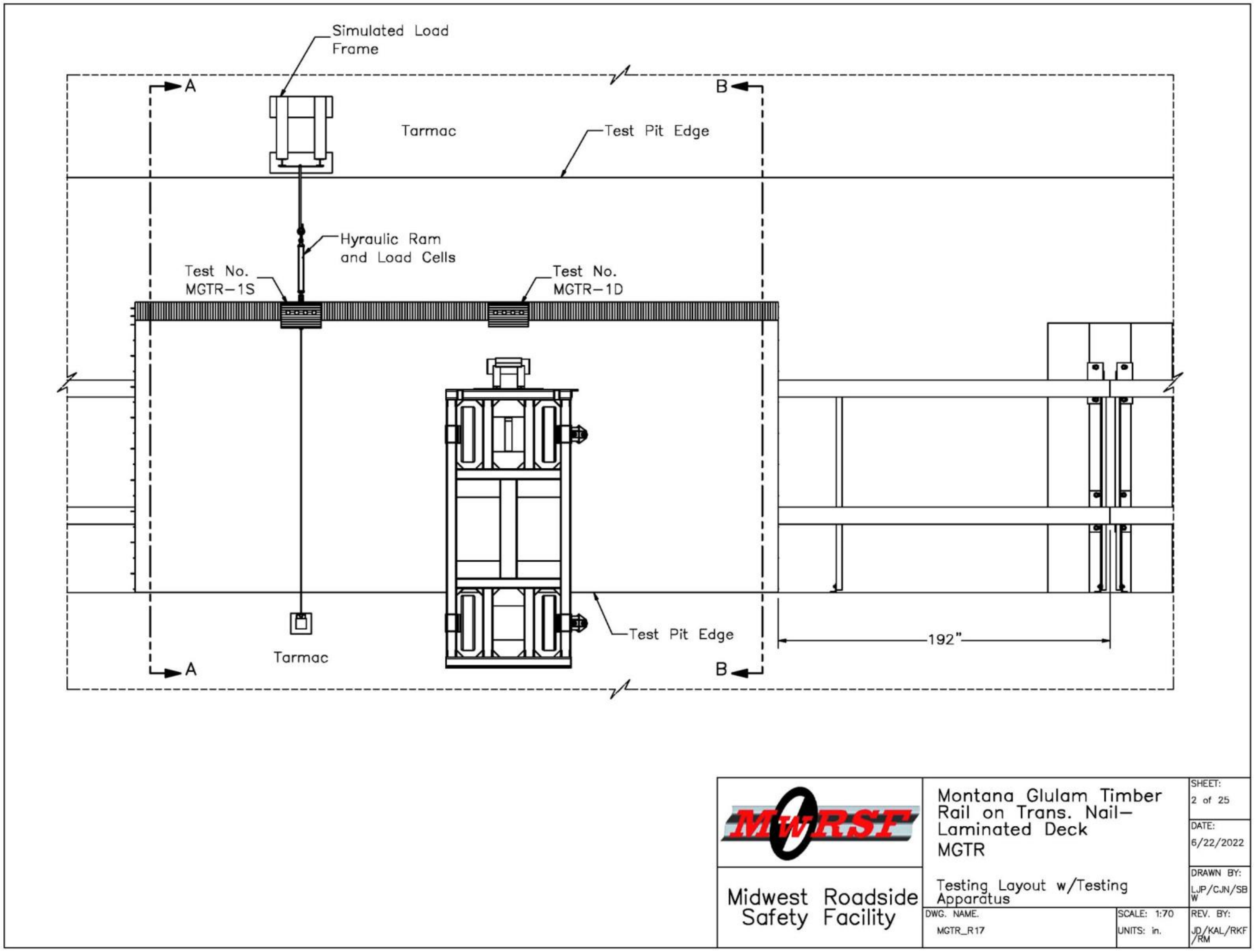
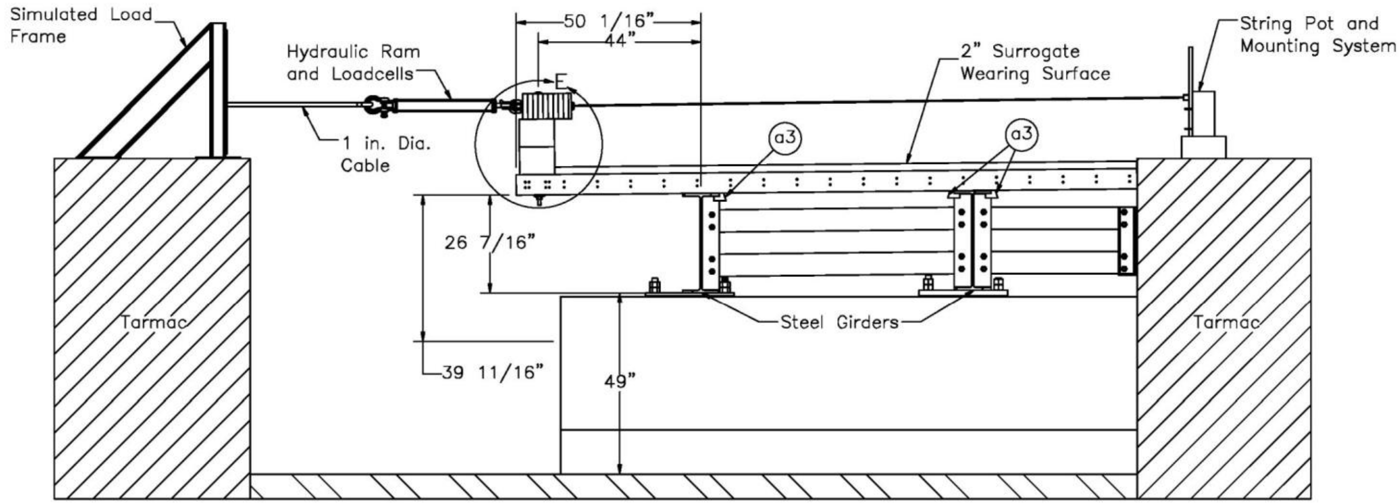



Figure 14. Test Plan and Layout, Test Nos. MGTR-1S and MGTR-1D




SECTION A–A

Notes: (1) Bottom edge of anchor plate slot (part a3) shall be in full contact with steel girders to better anchor wood deck.
(2) 2x6x14 boards shall be in full contact with top flanges of steel girders.
(3) Insert nails (part d10) into 2x6x14 boards between steel girders before inserting nails (part d10) elsewhere when construcing wood deck.
(4) 2x6x14 boards shall not be bowed and shall be plumb with steel girders.
(5) The 2x6x14 boards do not need to be flush with each other over cantilevered overhang section.

| | | |
|---|---|---|
| Midwest Roadside Safety Facility | Montana Glulam Timber Rail on Trans. Nail–Laminated Deck MGTR | SHEET: 3 of 25 |
| | | DATE: 6/22/2022 |
| | MGTR–1S Testing Layout | DRAWN BY: LJP/CJN/SBW |
| | DWG. NAME. MGTR_R17 / SCALE: 1:40 UNITS: in. | REV. BY: JD/KAL/RKF/RM |

Figure 15. Testing Layout, Test No. MGTR-1S

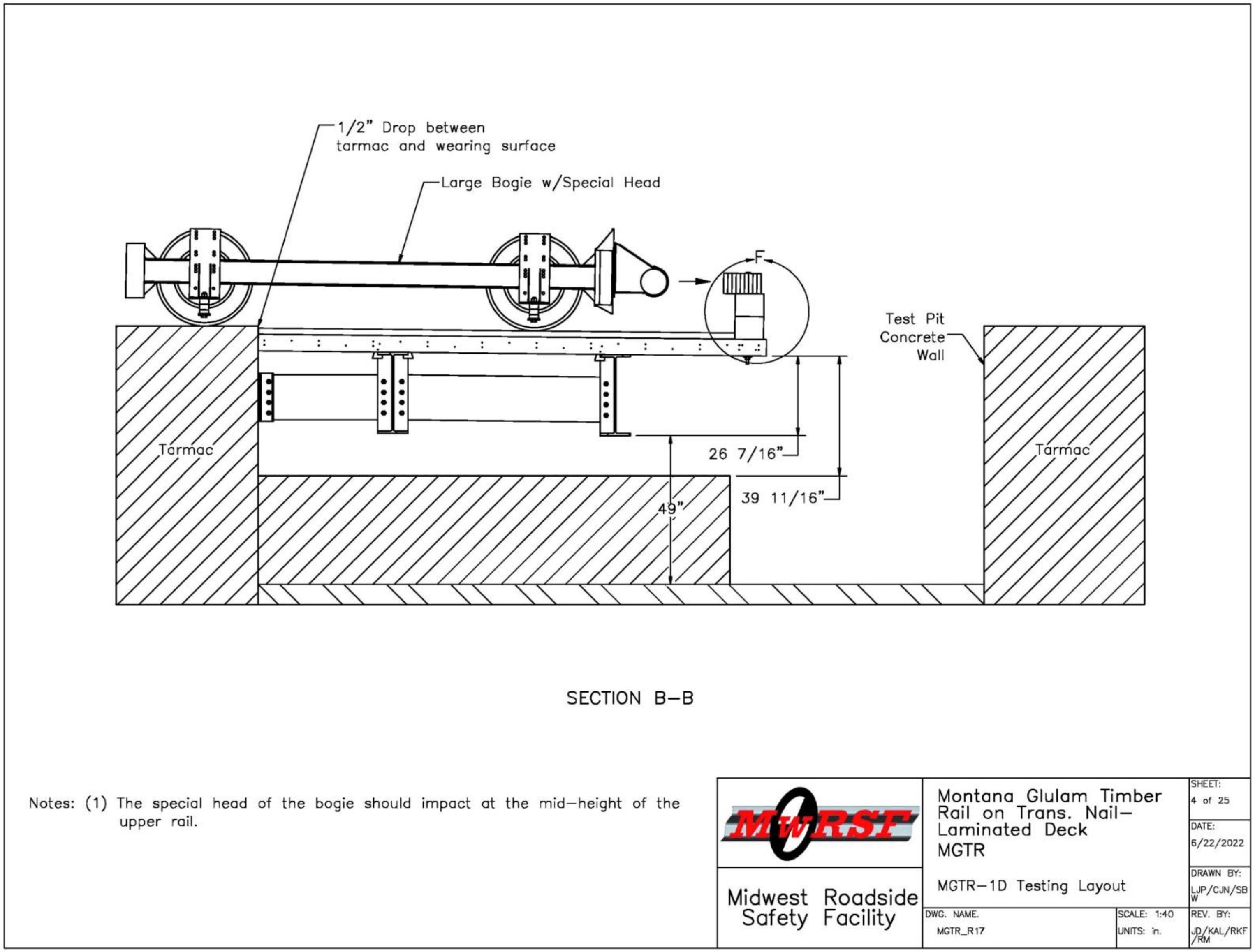



Figure 16. Testing Layout, Test No. MGTR-1D

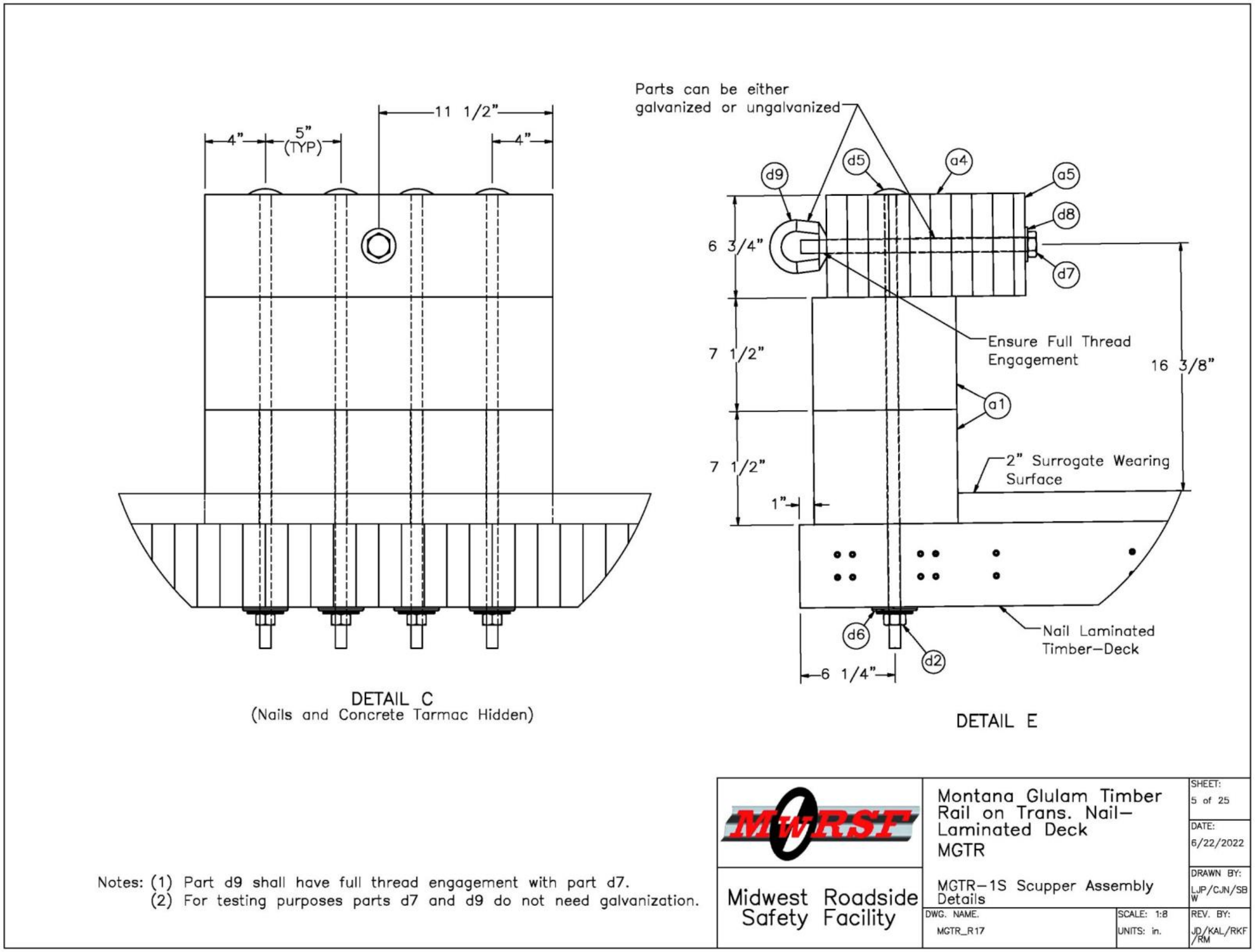



Figure 17. Scupper Assembly Details, Test No. MGTR-1S

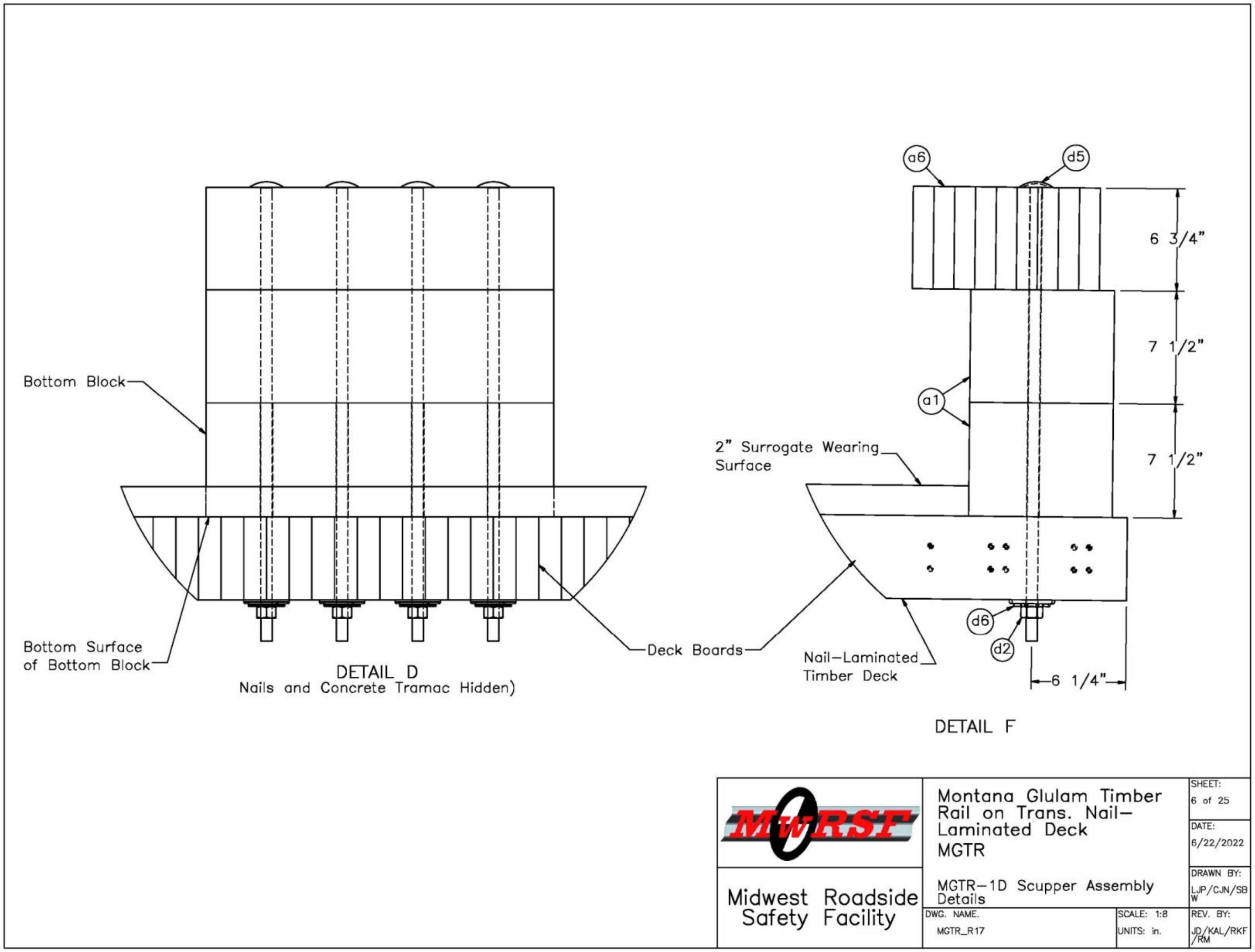
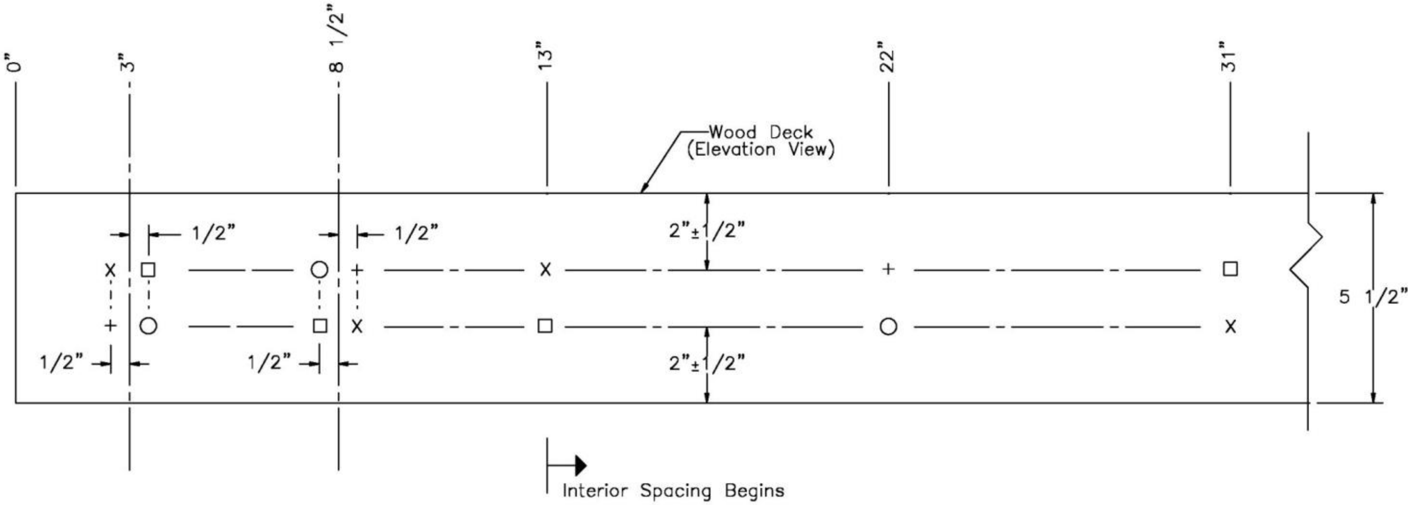



Figure 18. Scupper Assembly Details, Test No. MGTR-1D




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

Notes: (1) Timber deck boards shall be nailed together uding 20d or 20 penny "common" nails (part d10).
(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 cantilever wood deck length under bridge railing. Glue adhesive material shall consist of Liquid Nails – Heavy duty Contruction Adhesive, Item No. LN-901.

| | | |
|---|---|---|
| Midwest Roadside Safety Facility | Montana Glulam Timber Rail on Trans. Nail–Laminated Deck MGTR | SHEET: 7 of 25 |
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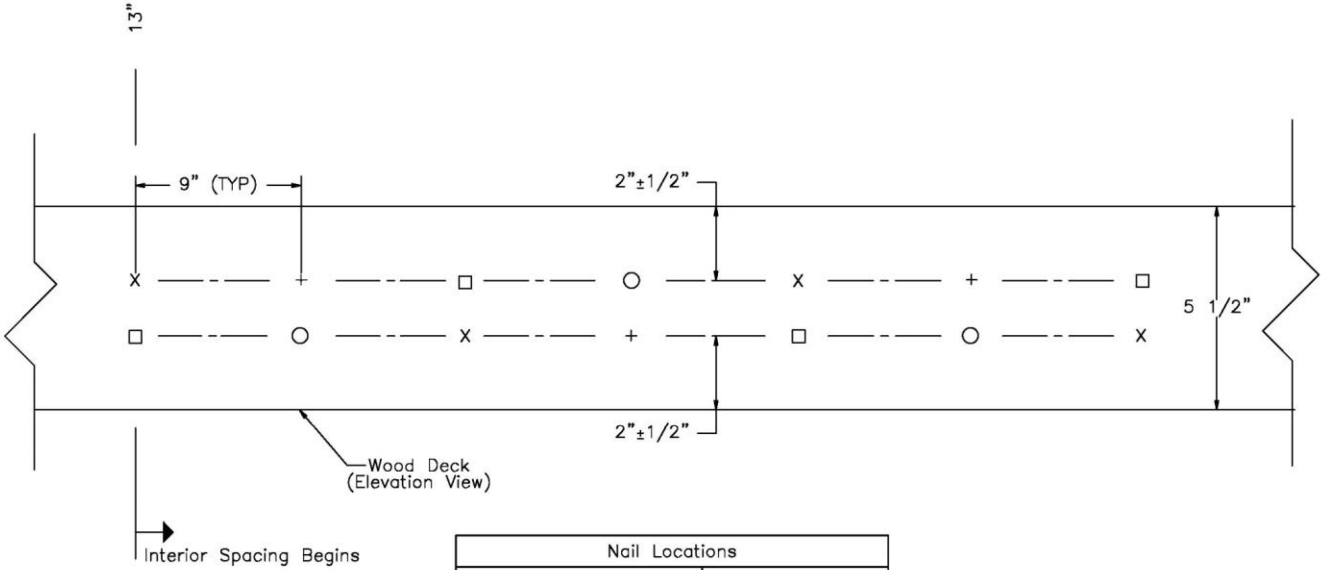
| | Exterior Nail Pattern for Timber Deck | DRAWN BY: LJP/CJN/SBW |
| | DWG. NAME. MGTR_R17 / SCALE: 1:8 UNITS: in. | REV. BY: JD/KAL/RKF/RM |

Figure 19. Exterior Nail Pattern for Timber Deck, Test Nos. MGTR-1S and MGTR-1D




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

Notes: (1) Timber deck boards shall be nailed together uding 20d or 20 penny "common" nails.
(2) Interior deck nailing pattern is shown above.

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Montana Glulam Timber Rail on Trans. Nail—Laminated Deck
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Interior Nail Pattern for Timber Deck

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| DWG. NAME. | MGTR_R17 |
| SCALE: | 1:4 |
| UNITS: | in. |

Figure 20. Interior Nail Pattern for Timber Deck, Test Nos. MGTR-1S and MGTR-1D

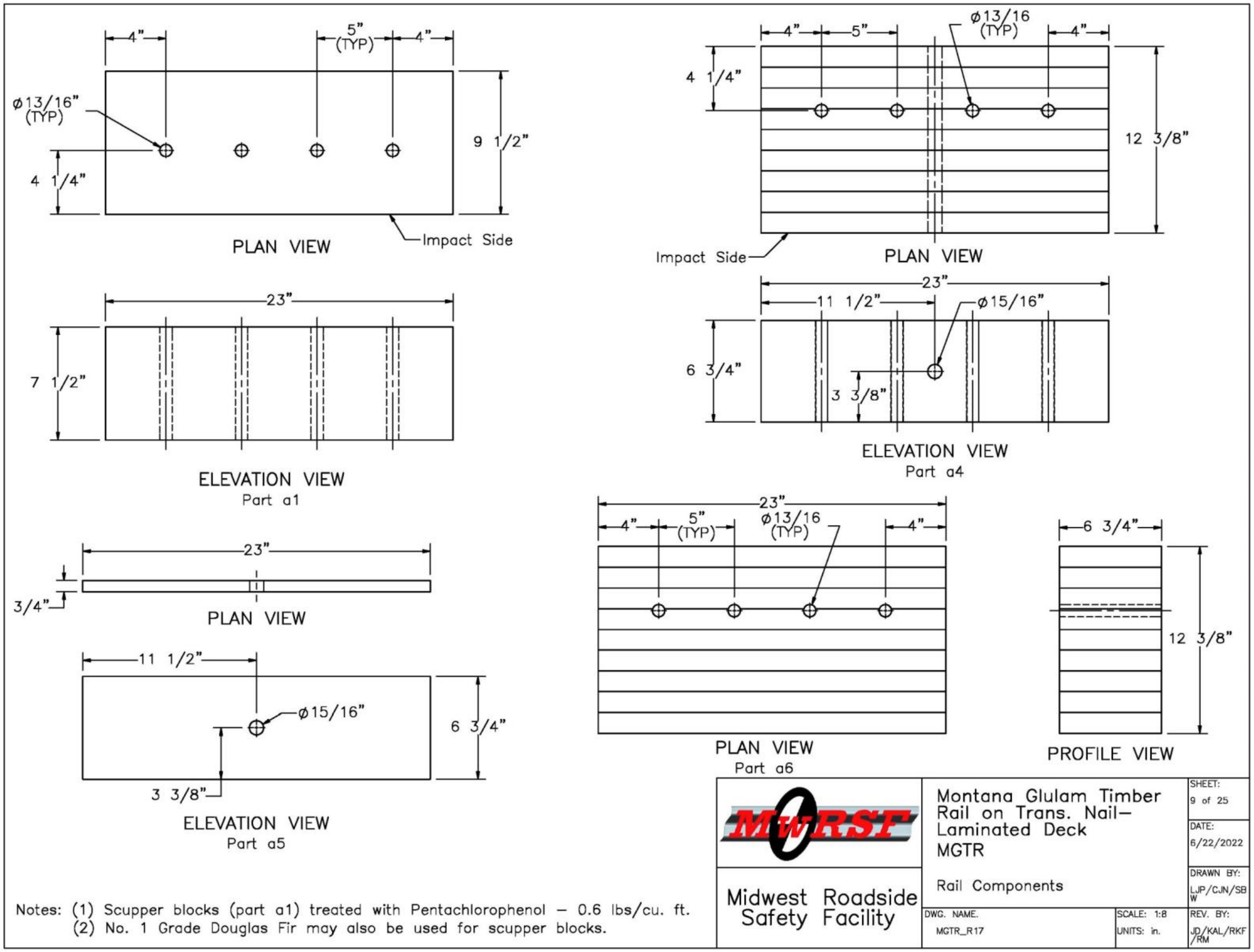
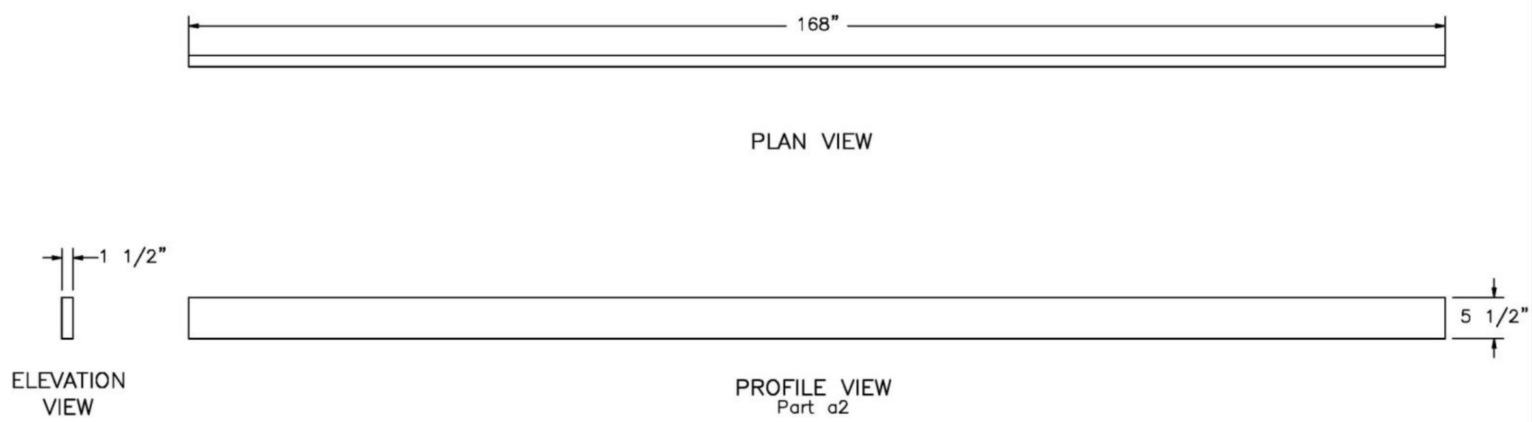


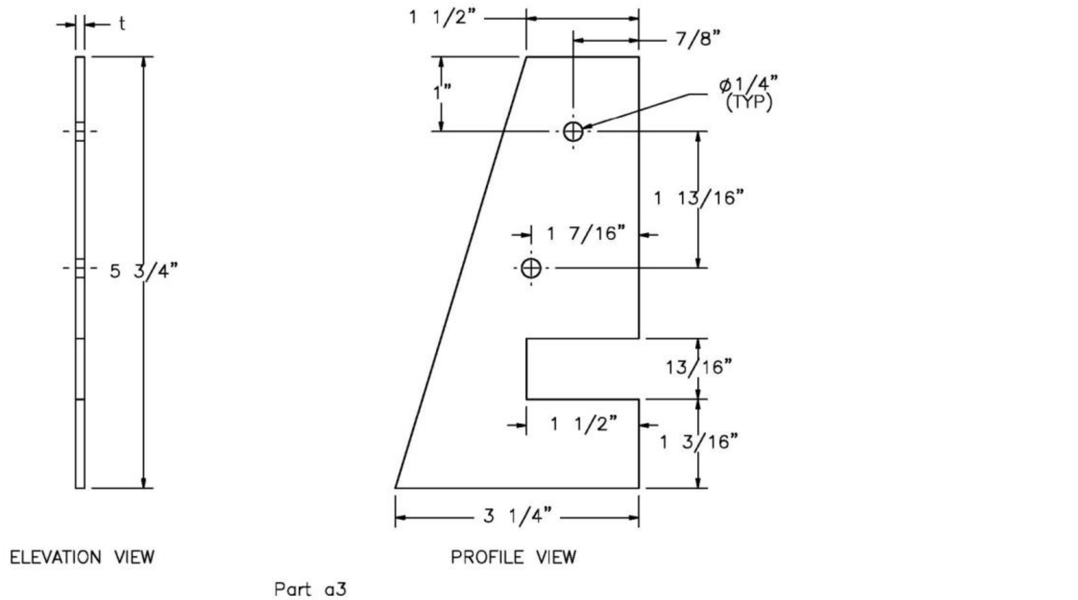

Figure 21. Rail Components, Test Nos. MGTR-1S and MGTR-1D




Notes: (1) Treated, dimensional lumber deck boards with 0.40 lbs/cu. ft retention for testing purposes only.
(2) Final Recommended Design: 2"x6", No. 1 Grade, Southern Yellow Pine, treated, dimensional lumber deck boards with 0.60 lbs/cu. ft retention.
(3) Liquid Nails adhesive should be applied using 2 beads minimum to the end 3 ft of cantilever wood deck length under the bridge railing. Glue adhesive material shall consist of Liquid Nails – Heavy Duty Construction Adhesive, Item No. LN-901.

| | | |
|---|---|---|
| MwRSF<br>Midwest Roadside Safety Facility | Montana Glulam Timber Rail on Trans. Nail–Laminated Deck MGTR | SHEET: 10 of 25<br>DATE: 6/22/2022 |
| | Nail–Laminated Timber Bridge Deck Lumber Details | DRAWN BY: LJP/CJN/SBW |
| | DWG. NAME. MGTR_R17 / SCALE: 1:4 UNITS: in. | REV. BY: JD/KAL/RKF/RM |

Figure 22. Nail-Laminated, Timber Bridge Deck Lumber Details, Test Nos. MGTR-1S and MGTR-1D




| 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 A123 (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.

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Montana Glulam Timber Rail on Trans. Nail–Laminated Deck MGTR

Deck Anchor Bracket Details

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DWG. NAME. MGTR_R17
SCALE: 1:384
UNITS: in.

Figure 23. Deck Anchor Bracket Details, Test Nos. MGTR-1S and MGTR-1D

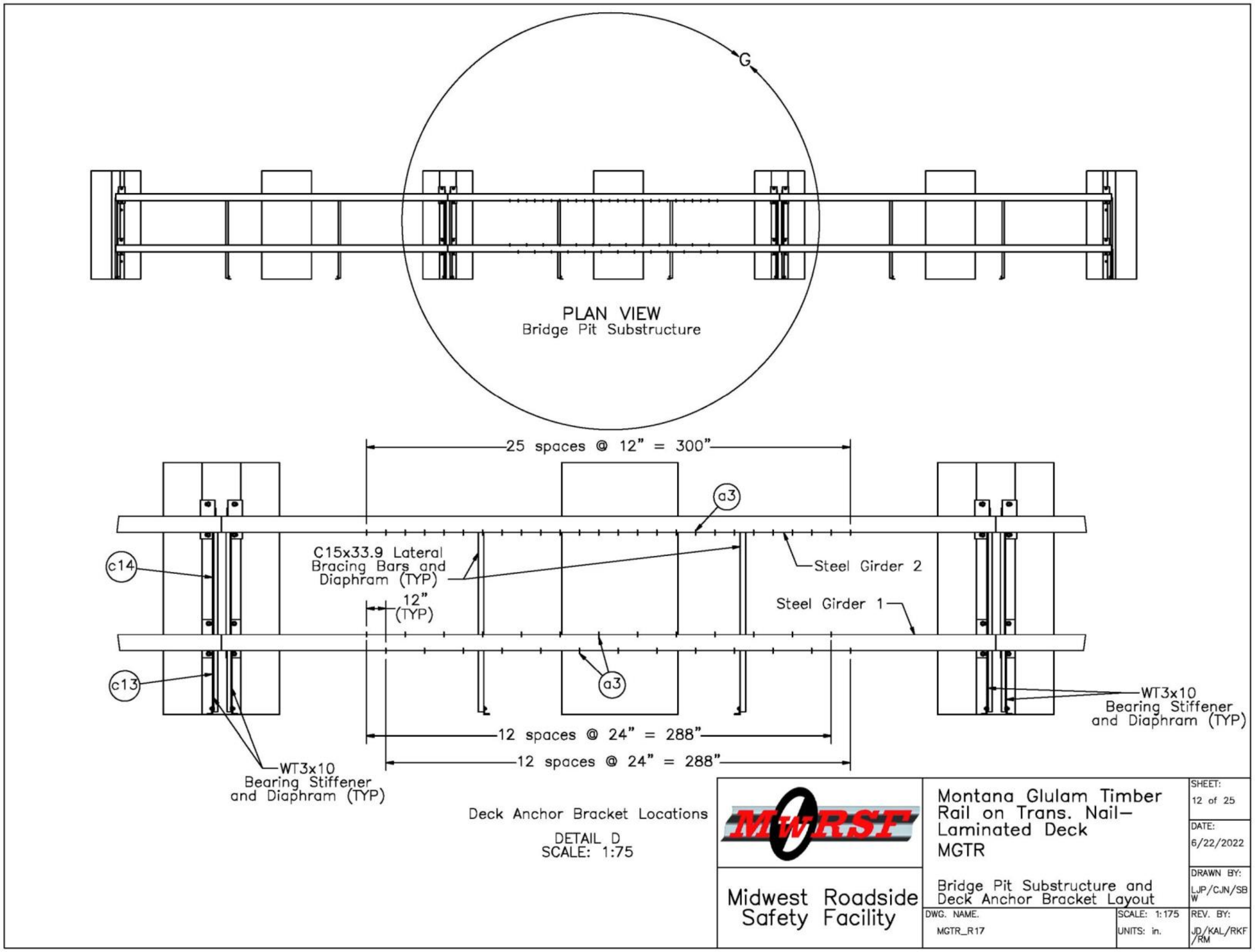
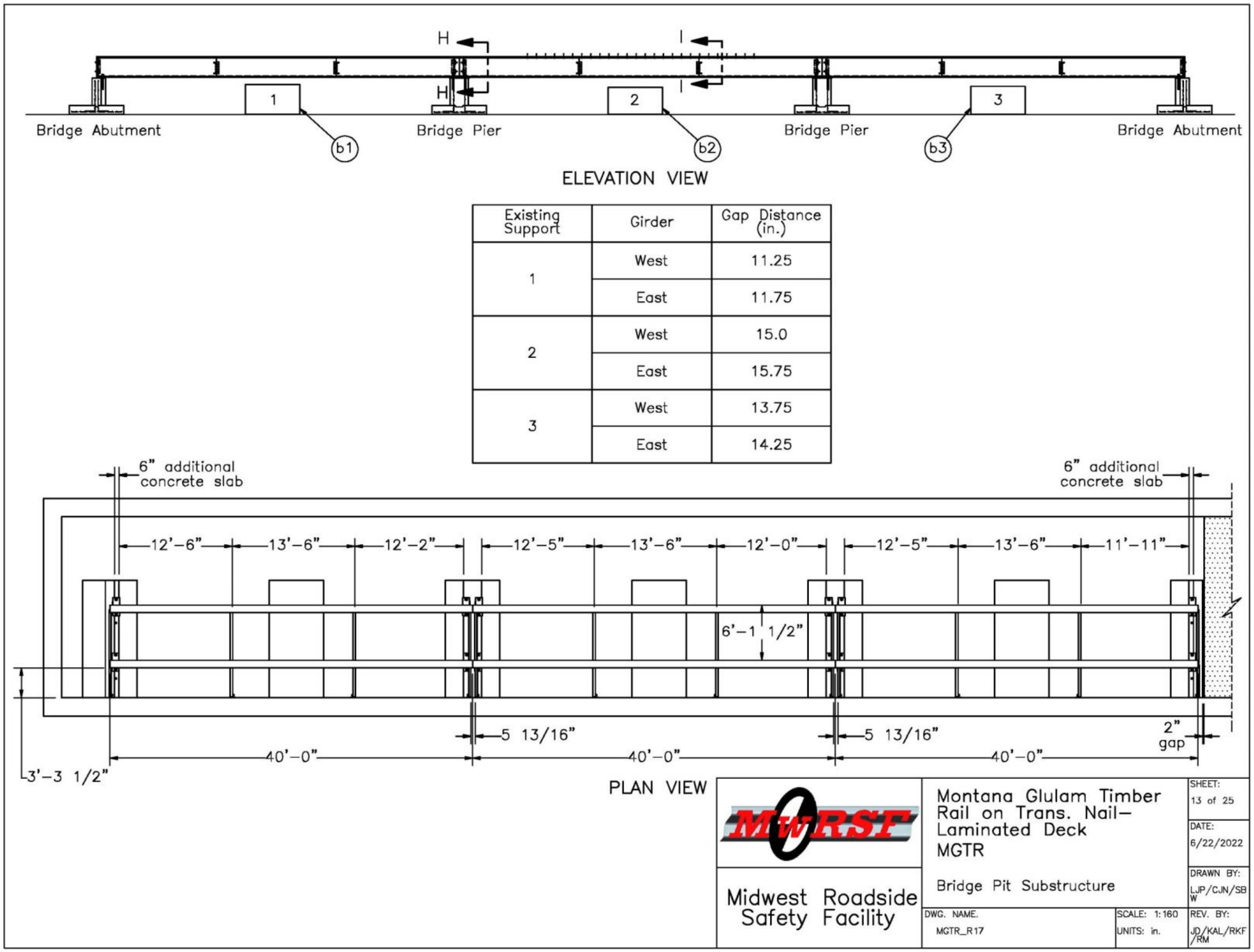



Figure 24. Bridge Pit Superstructure, Substructure, and Deck Anchor Bracket Layout, Test Nos. MGTR-1S and MGTR-1D




| Existing 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 25. Bridge Pit Superstructure and Substructure, Test Nos. MGTR-1S and MGTR-1D

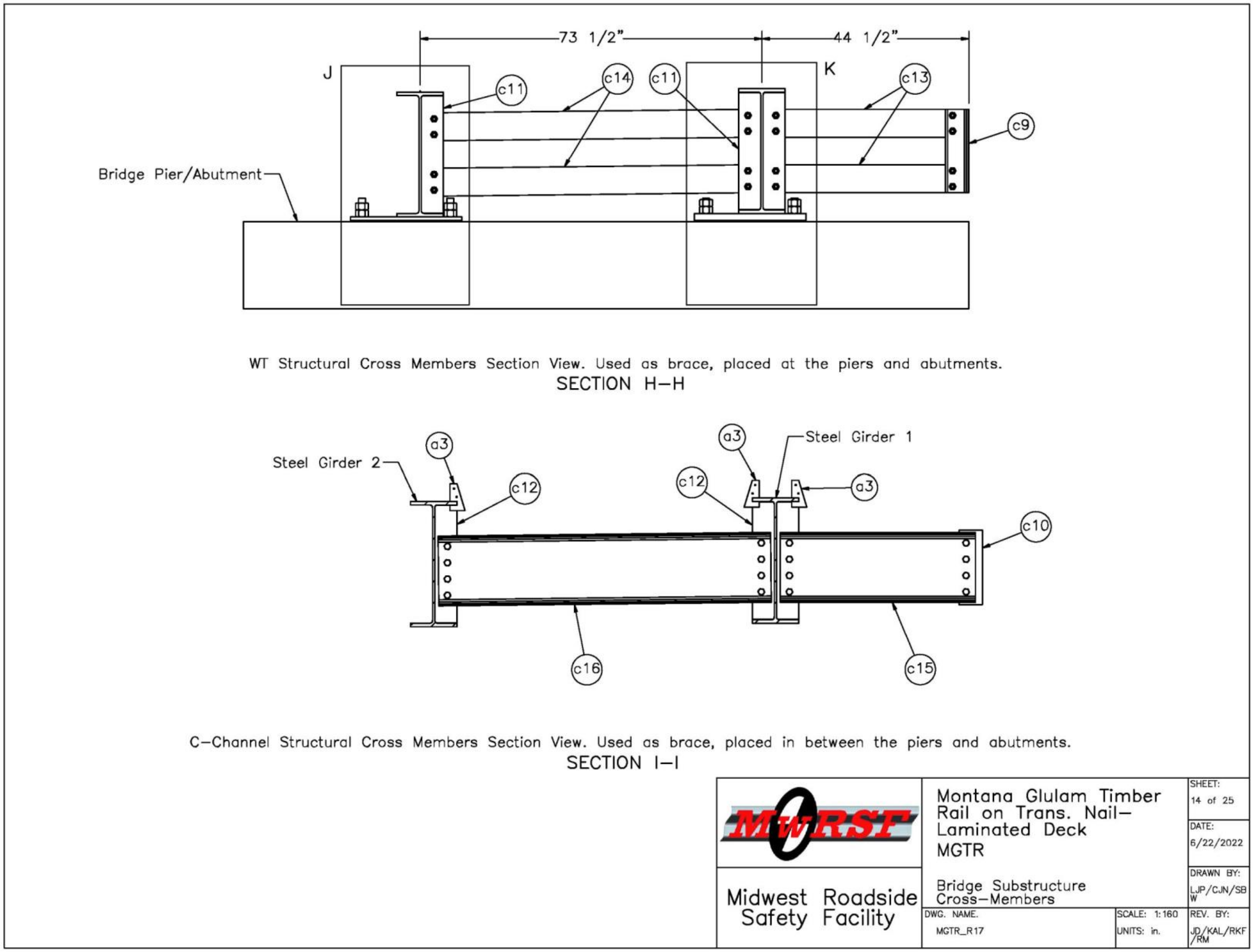



Figure 26. Bridge Superstructure and Substructure, Test Nos. MGTR-1S and MGTR-1D

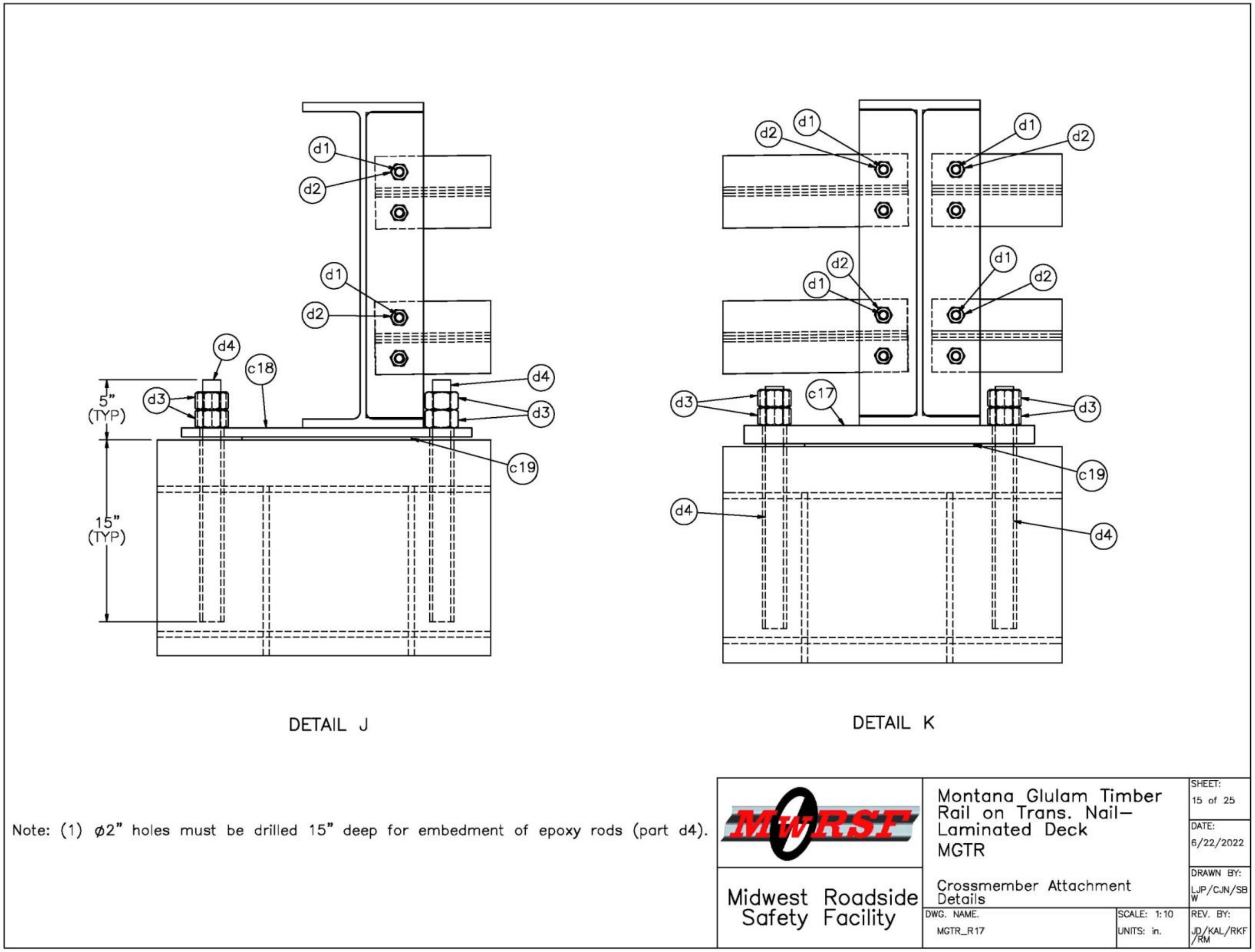
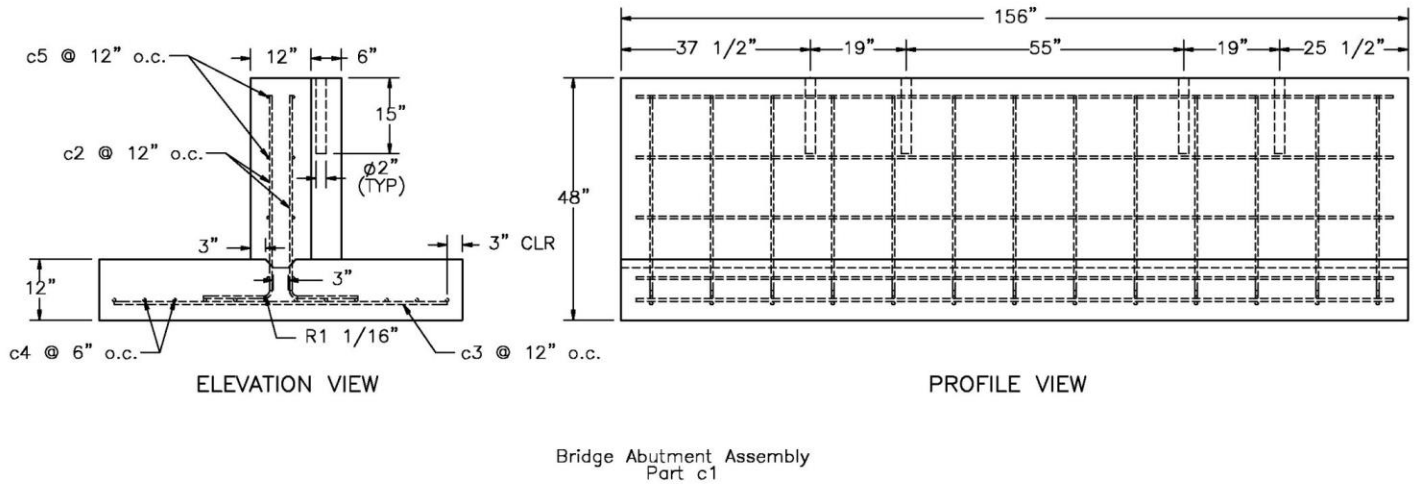



Figure 27. Crossmember Attachment Details, Test Nos. MGTR-1S and MGTR-1D




| Item No. | QTY. | Description | Material Specification |
|---|---|---|---|
| c1 | – | Bridge Abutment Assembly | Min f'c = 4,000 psi NE Mix 47 BD |
| c2 | 26 | #4 Bent Rebar, 52 3/4" Unbent Length | ASTM A615 Gr. 60 |
| c3 | 13 | #5 Rebar 5'–6" Long | ASTM A615 Gr. 60 |
| c4 | 12 | #5 Rebar 12'–6" Long | ASTM A615 Gr. 60 |
| c5 | 8 | #4 Rebar 12'–6" Long | ASTM A615 Gr. 60 |

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Bridge Abutment Assembly

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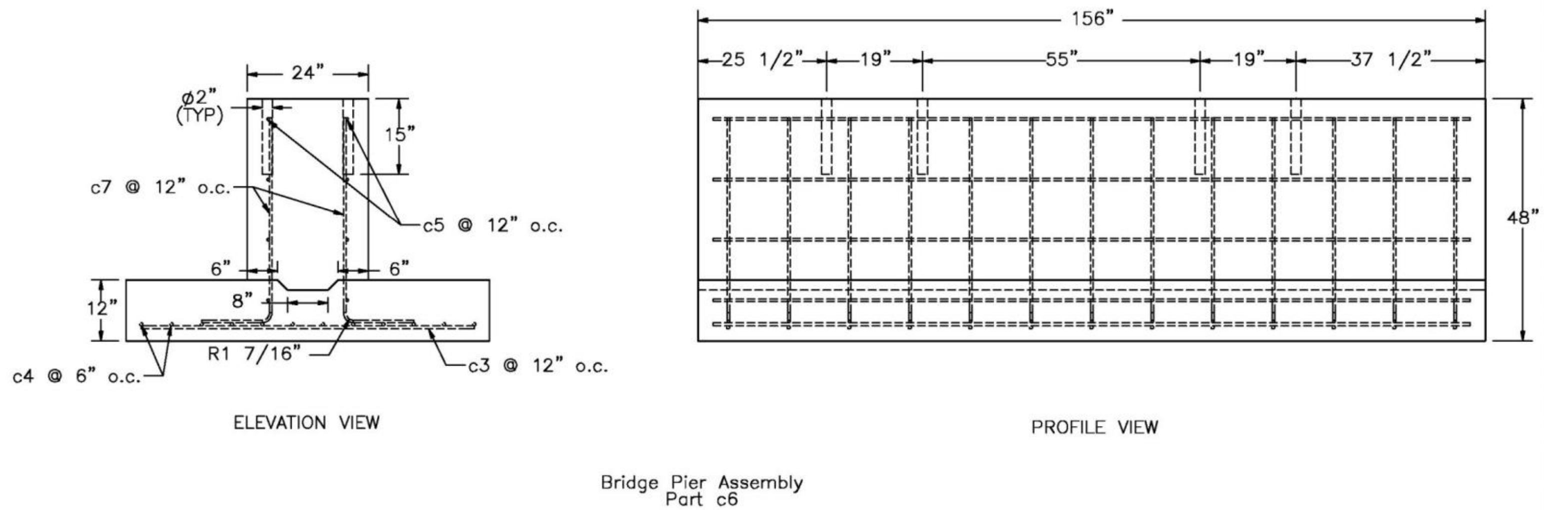
DWG. NAME. MGTR_R17
SCALE: 1:30
UNITS: in.
REV. BY: JD/KAL/RKF/RM

Figure 28. Bridge Abutment Assembly, Test Nos. MGTR-1S and MGTR-1D




| Item No. | QTY. | Description | Material Specification |
|---|---|---|---|
| c6 | – | Bridge Pier Assembly | Min f'c = 4,000 psi NE Mix 47 BD |
| c3 | 13 | #5 Rebar 5'–6" Long | ASTM A615 Gr. 60 |
| c4 | 12 | #5 Rebar 12'–6" Long | ASTM A615 Gr. 60 |
| c5 | 8 | #4 Rebar 12'–6" Long | ASTM A615 Gr. 60 |
| c7 | 26 | #4 Bent Rebar, 52 1/4" Unbent Length | ASTM A615 Gr. 60 |

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Montana Glulam Timber Rail on Trans. Nail–Laminated Deck
MGTR

Bridge Pier Assembly

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DWG. NAME. MGTR_R17
SCALE: 1:30
UNITS: in.
REV. BY: JD/KAL/RKF/RM

Figure 29. Bridge Pier Assembly Test Nos. MGTR-1S and MGTR-1D

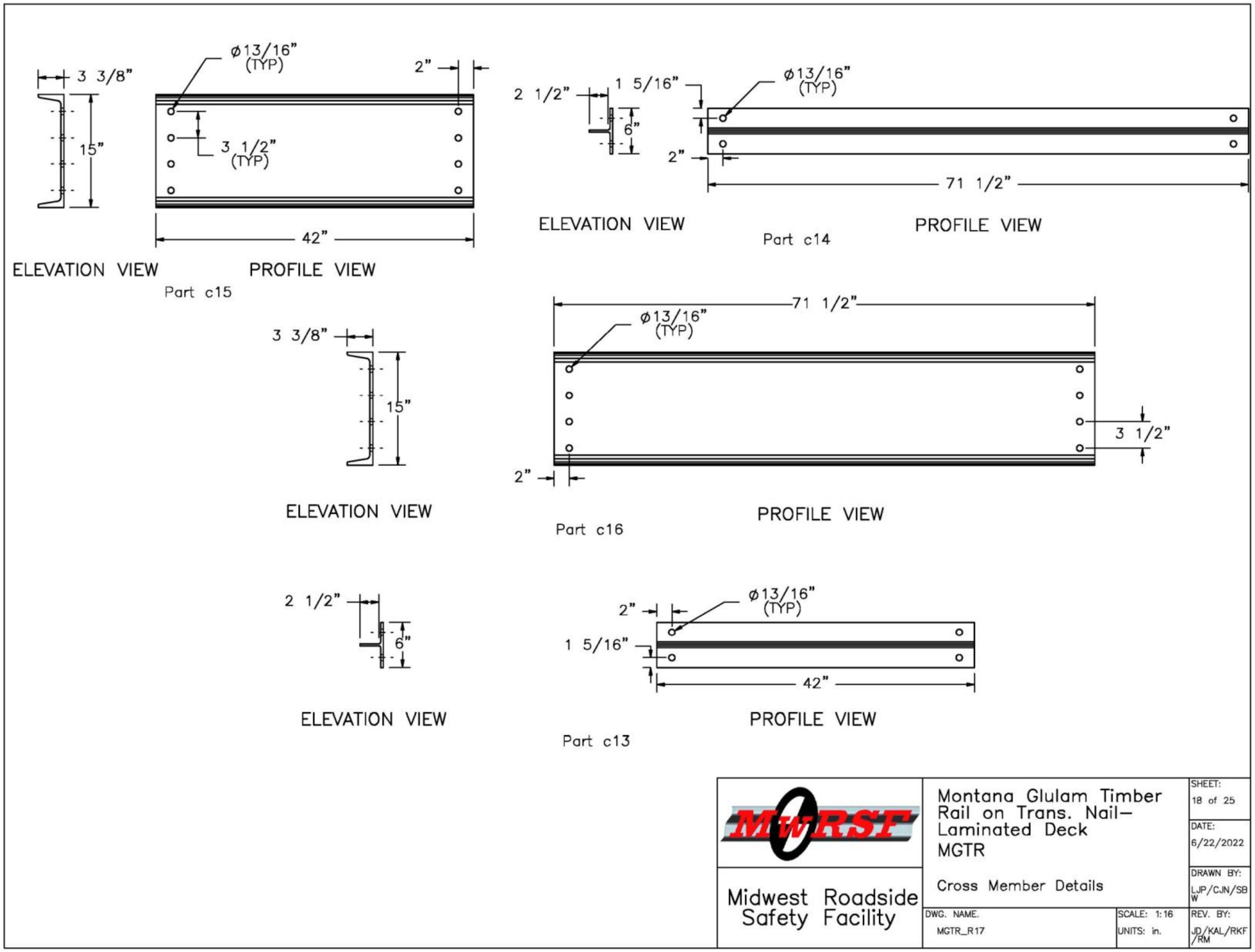



Figure 30. Cross Member Details, Test Nos. MGTR-1S and MGTR-1D

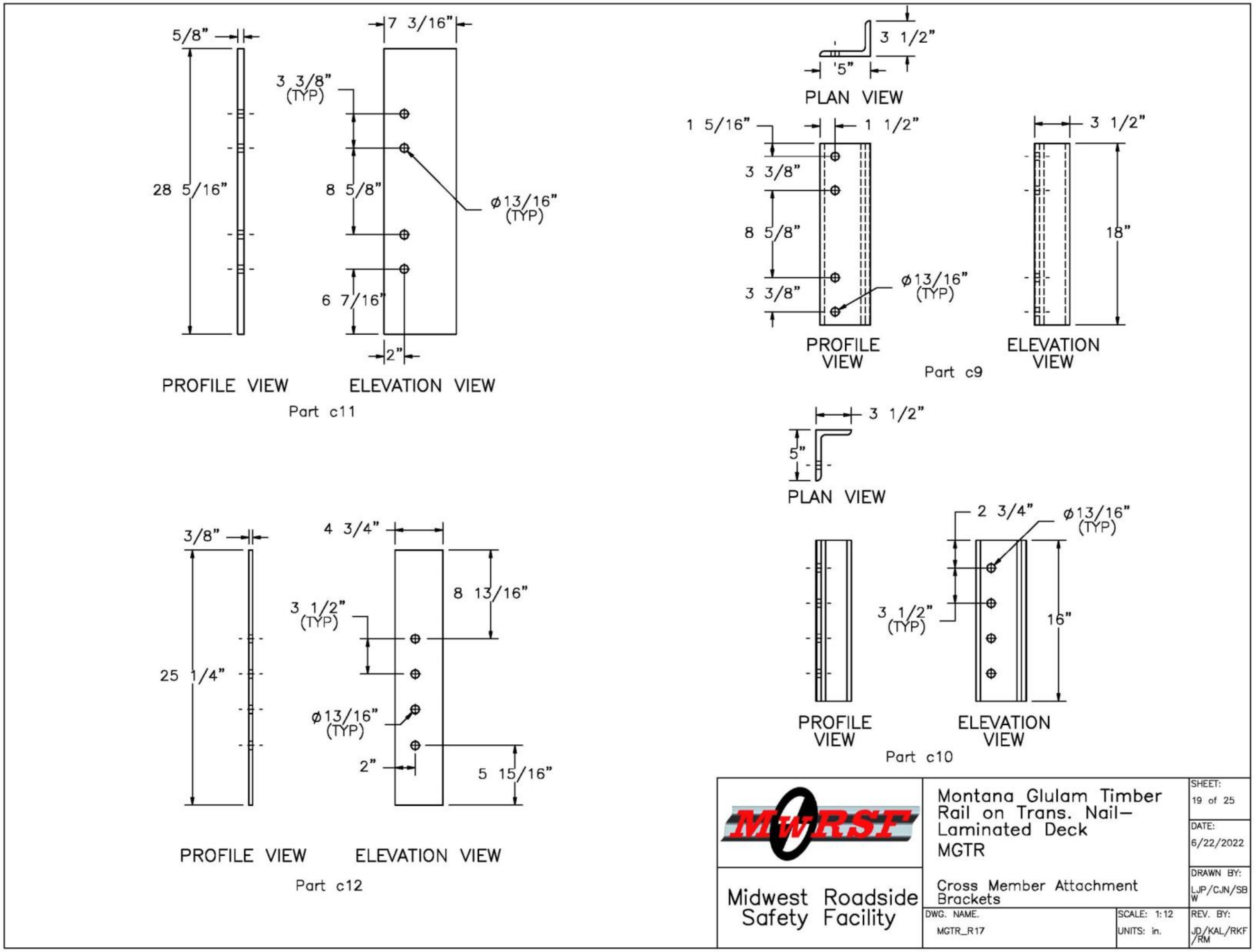



Figure 31. Cross Member Attachment Brackets, Test Nos. MGTR-1S and MGTR-1D

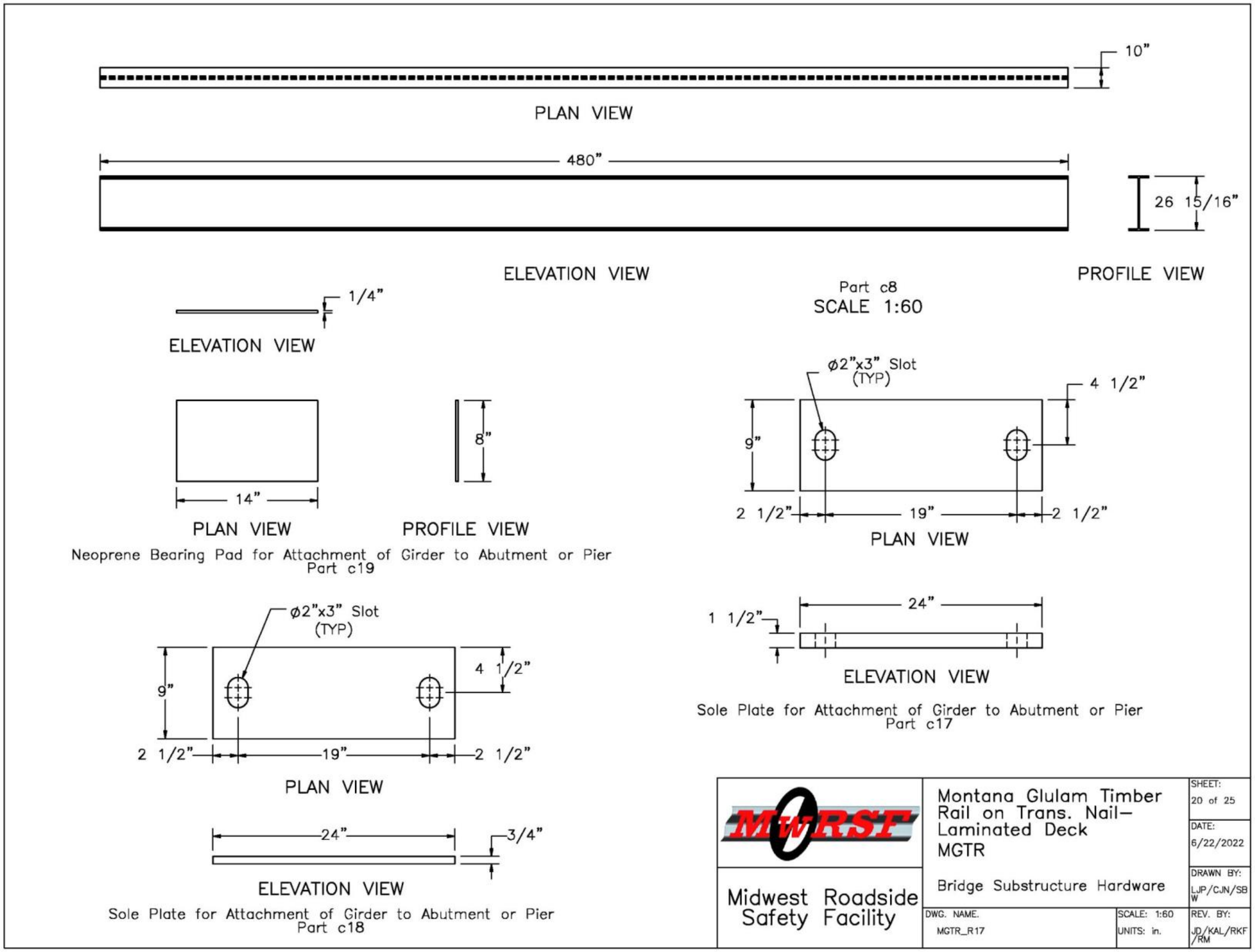



Figure 32. Bridge Substructure Hardware, Test Nos. MGTR-1S and MGTR-1D

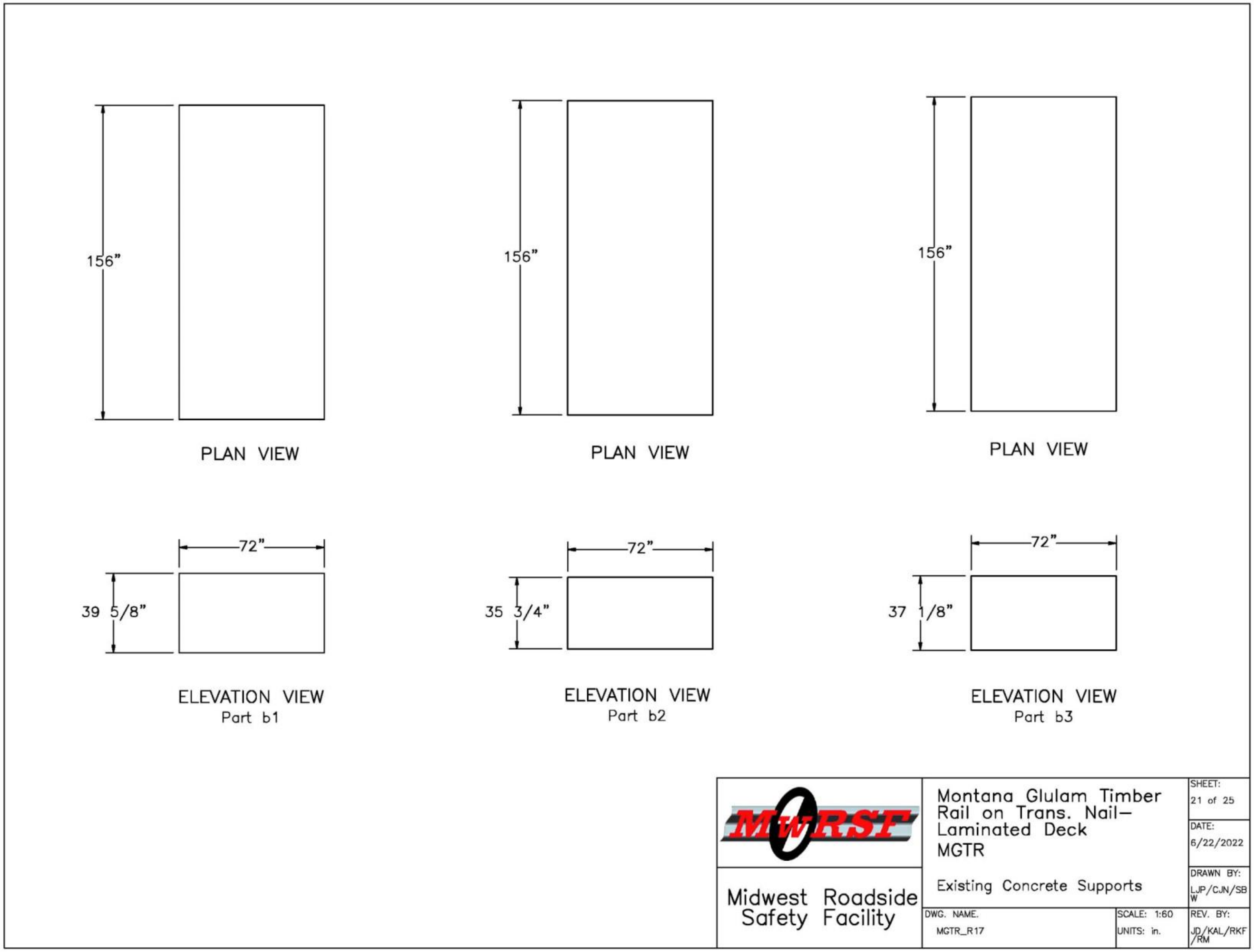
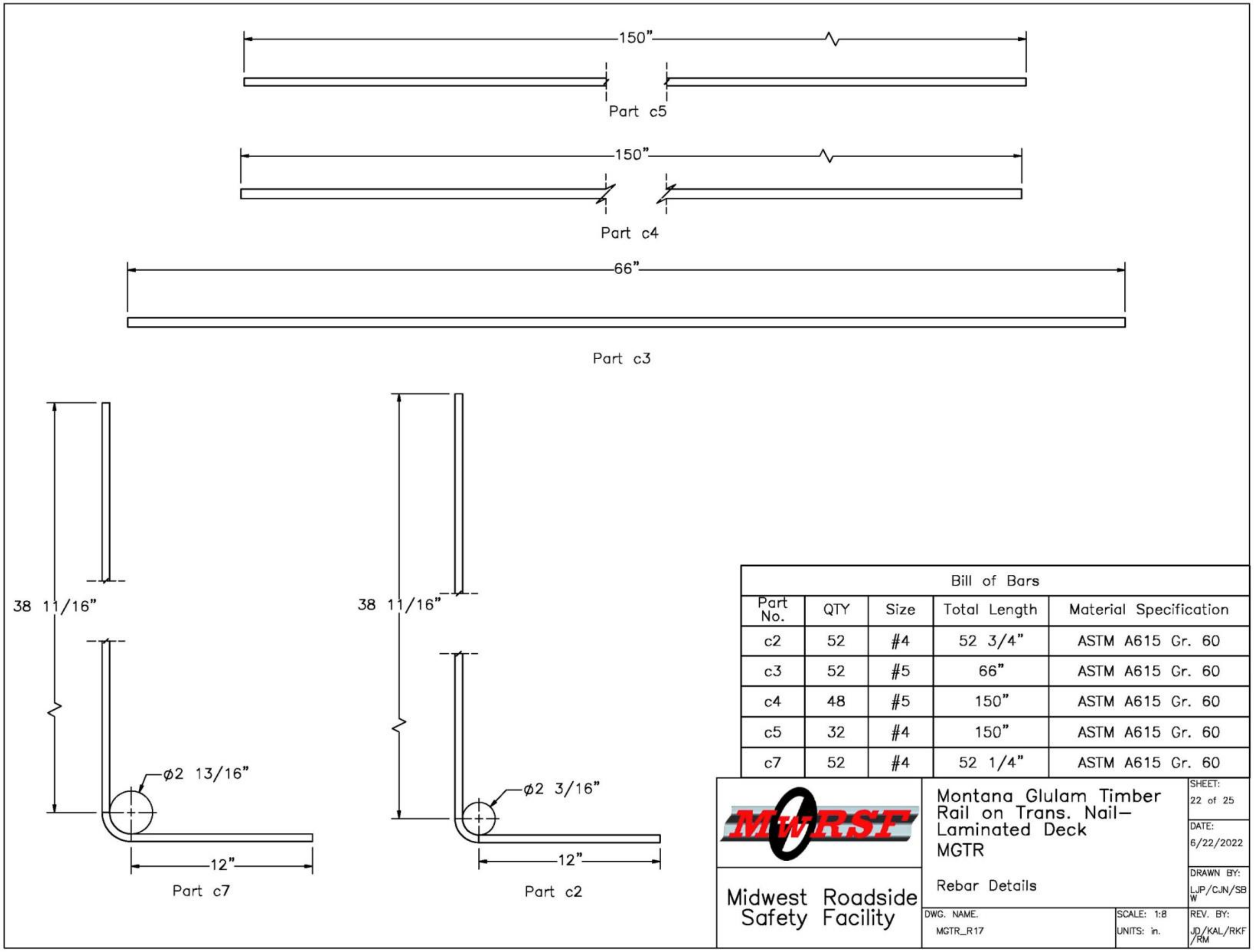



Figure 33. Existing Concrete Supports, Test Nos. MGTR-1S and MGTR-1D




Bill of Bars

| Part No. | QTY | Size | Total Length | Material Specification |
|---|---|---|---|---|
| c2 | 52 | #4 | 52 3/4" | ASTM A615 Gr. 60 |
| c3 | 52 | #5 | 66" | ASTM A615 Gr. 60 |
| c4 | 48 | #5 | 150" | ASTM A615 Gr. 60 |
| c5 | 32 | #4 | 150" | ASTM A615 Gr. 60 |
| c7 | 52 | #4 | 52 1/4" | ASTM A615 Gr. 60 |

Figure 34. Rebar Details, Test Nos. MGTR-1S and MGTR-1D

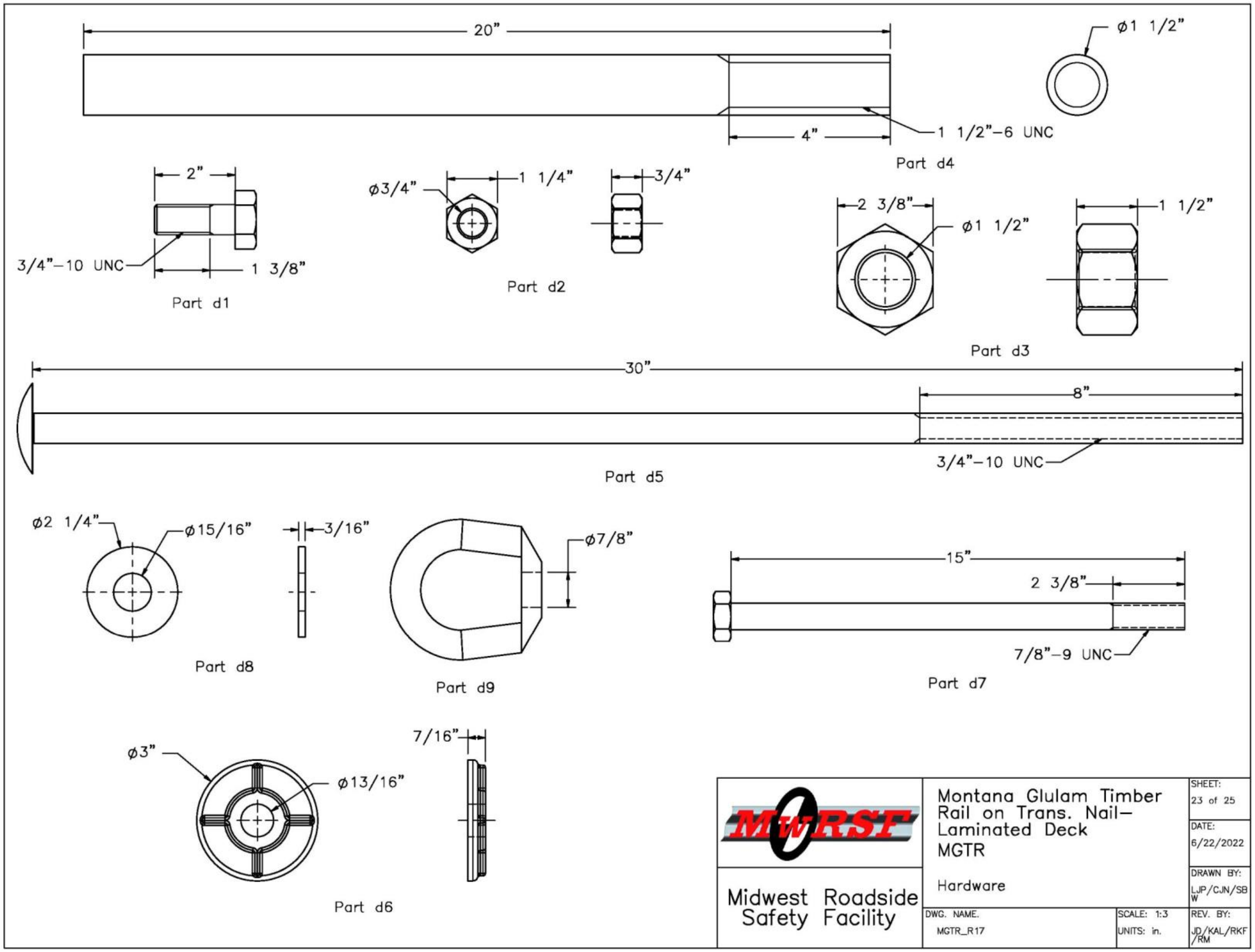



Figure 35. Hardware, Test Nos. MGTR-1S and MGTR-1D

BOM Table

| Item No. | QTY. | Description | Material Specification | Treatment Specification | Hardware Guide |
|---|---|---|---|---|---|
| a1 | 4 | 9 1/2"x23"x7 1/2" Scupper Block | Grade No. 1 Southern Yellow Pine or Douglas Fir | Pentachlorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a2 * | 248 | 2"x6"x14' Long Treated, Dimensional Lumber | Grade No. 1 Southern Yellow Pine | Pentachlorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a3 | 52 | Deck Anchor Plate | See Sheet 11 | – | – |
| a4 | 1 | 12 3/8"x23"x6 3/4" Glulam Rail Segment | Southern Yellow Pine Combination No. 48 | Pentachorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a5 | 1 | 23"x6 3/4"x3/4" Static Test Plate | ASTM A36 | ASTM A123 | – |
| a6 | 1 | 12 3/8"x23"x6 3/4" Glulam Rail Segment | Southern Yellow Pine Combination No. 48 | Pentachorophenal in Heavy Oil 0.6lbs/cu. ft Retention | – |
| b1 | 1 | Concrete Support 1 | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| b2 | 1 | Concrete Support 2 | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| b3 | 1 | Concrete Support 3 | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| c1 | 2 | Bridge Abutment Assembly | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| c2 | 52 | #4 Bent Rebar, 52 3/4" Unbent Length | ASTM A615 Gr. 60 | – | – |
| c3 | 52 | #5 Rebar 5'–6" Long | ASTM A615 Gr. 60 | – | – |
| c4 | 48 | #5 Rebar 12'–6" Long | ASTM A325 | – | – |
| c5 | 32 | #4 Rebar 12'–6" Long | ASTM A615 Gr. 60 | – | – |
| c6 | 2 | Bridge Pier Assembly | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| c7 | 52 | #4 Bent Rebar, 52 1/4" Unbent Length | ASTM A615 Gr. 60 | – | – |
| c8 | 6 | W27x94, 40' Long Steel Girder | ASTM A36 | Painted | – |
| c9 | 6 | L5x3.5x0.5, 18" Long | ASTM A36 | Painted | – |
| c10 | 6 | L5x3.5x0.5, 16" Long | ASTM A36 | Painted | – |
| c11 | 18 | 30 5/16"x7 3/16"x5/8" Plate | ASTM A36 | Painted | – |
| c12 | 18 | 30 5/16"x7 3/16"x3/8" Plate | ASTM A36 | Painted | – |
| c13 | 12 | WT3x10, 42" Long | ASTM A36 | Painted | – |

* For testing purposes, the lumber boards were acquired with an alternative preservative treatment. The treatment needed for this part is detailed under treatment specifications.

MwRSF

Midwest Roadside Safety Facility

Montana Glulam Timber Rail on Trans. Nail–Laminated Deck MGTR

Bill of Materials

SHEET: 24 of 25

DATE: 6/22/2022

DRAWN BY: LJP/CJN/SBW

DWG. NAME. MGTR_R17

SCALE: None UNITS: in.

REV. BY: JD/KAL/RKF/RM

Figure 36. Bill of Materials, Test Nos. MGTR-1S and MGTR-1D

| BOM Table | | | | | |
|---|---|---|---|---|---|
| Item No. | QTY. | Description | Material Specification | Treatment Specification | Hardware Guide |
| a1 | 4 | 9 1/2"x23"x7 1/2" Scupper Block | Grade No. 1 Southern Yellow Pine or Douglas Fir | Pentachlorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a2 * | 248 | 2"x6"x14' Long Treated, Dimensional Lumber | Grade No. 1 Southern Yellow Pine | Pentachlorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a3 | 52 | Deck Anchor Plate | See Sheet 11 | – | – |
| a4 | 1 | 12 3/8"x23"x6 3/4" Glulam Rail Segment | Southern Yellow Pine Combination No. 48 | Pentachorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a5 | 1 | 23"x6 3/4"x3/4" Static Test Plate | ASTM A36 | ASTM A123 | – |
| a6 | 1 | 12 3/8"x23"x6 3/4" Glulam Rail Segment | Southern Yellow Pine Combination No. 48 | Pentachorophenal in Heavy Oil 0.6lbs/cu. ft Retention | – |
| b1 | 1 | Concrete Support 1 | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| b2 | 1 | Concrete Support 2 | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| b3 | 1 | Concrete Support 3 | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| c1 | 2 | Bridge Abutment Assembly | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| c2 | 52 | #4 Bent Rebar, 52 3/4" Unbent Length | ASTM A615 Gr. 60 | – | – |
| c3 | 52 | #5 Rebar 5'–6" Long | ASTM A615 Gr. 60 | – | – |
| c4 | 48 | #5 Rebar 12'–6" Long | ASTM A325 | – | – |
| c5 | 32 | #4 Rebar 12'–6" Long | ASTM A615 Gr. 60 | – | – |
| c6 | 2 | Bridge Pier Assembly | Min f'c = 4,000 psi NE Mix 47 BD | – | – |
| c7 | 52 | #4 Bent Rebar, 52 1/4" Unbent Length | ASTM A615 Gr. 60 | – | – |
| c8 | 6 | W27x94, 40' Long Steel Girder | ASTM A36 | Painted | – |
| c9 | 6 | L5x3.5x0.5, 18" Long | ASTM A36 | Painted | – |
| c10 | 6 | L5x3.5x0.5, 16" Long | ASTM A36 | Painted | – |
| c11 | 18 | 30 5/16"x7 3/16"x5/8" Plate | ASTM A36 | Painted | – |
| c12 | 18 | 30 5/16"x7 3/16"x3/8" Plate | ASTM A36 | Painted | – |
| c13 | 12 | WT3x10, 42" Long | ASTM A36 | Painted | – |

* For testing purposes, the lumber boards were acquired with an alternative preservative treatment. The treatment needed for this part is detailed under treatment specifications.

| | | |
|---|---|---|
| MwRSF<br>Midwest Roadside Safety Facility | Montana Glulam Timber Rail on Trans. Nail–Laminated Deck<br>MGTR<br><br>Bill of Materials | SHEET: 24 of 25<br>DATE: 6/22/2022<br>DRAWN BY: LJP/CJN/SBW |
| | DWG. NAME. MGTR_R17 | SCALE: None UNITS: in. | REV. BY: JD/KAL/RKF/RM |

Figure 37. Bill of Materials, Cont., Test Nos. MGTR-1S and MGTR-1D

# 5 TRANSVERSE, GLULAM TIMBER DECK AND RAIL

## 5.1 Design Decisions

As noted previously, the USDA-FS-FPL standard plans and USDA-FS-NTDP bridge rails, transverse glulam timber decks, and superstructures plans and details were reviewed. After completing this task, a general questionnaire was prepared and sent to the sponsor to gather additional information to better understand typical layouts and design procedures. The questionnaire responses and additional discussion were used to develop the layout for the transverse, glulam, timber deck and its supporting elements that would be used in the component testing program. The questionnaire, responses, and additional discussion points, are provided in Appendix A.

## 5.2 Superstructure and Substructure

First, the research team concluded that a surrogate bridge system could be used to complete the investigation, which could be performed utilizing smaller girder sizes to reduce construction costs, when available. The sponsor indicated that all girder sizes are used and depend on the constraints of each real-world project. With this information, a test plan was created utilizing a 20-ft long bridge segment for performing the static and dynamic component tests. Utilizing the tables provided in FPL-GTR-125 [19], which was the currently-utilized standard plan document for the USDA-FS-NTDP, the suggested girder size for 20-ft long beams spanning 19 ft was 6¾-in. wide by 16½-in. tall. This suggested girder size corresponds to a DF Glulam Combination No. 24F-V4 beam. This beam type has been used in the past research and development programs at MwRSF. Based on information from the tables and sponsor, a 4-ft wide center-to-center spacing between girders was selected along with a 2-ft long exterior cantilever or overhang distance. A three girder system created a surrogate bridge deck with symmetry, where the deck panels extended 2 ft from the centerline of the outer girders. The girders were connected to the concrete supports using guidance provided in FPL-GTR-125 [19],which denoted the use of ½-in. thick steel plate members and a ¾-in. thick elastomeric bearing pad under the girder.

A 20-ft long surrogate bridge system was also based on typical sizing of transverse, glulam timber deck panels, as shown in Figure 39. The sponsor indicated that a typical transverse deck panel measures 4 ft wide, which corresponds to the longitudinal direction of the bridge. Thus, the bridge system required the use five (5) panels. The transverse, glulam timber deck panels would measure 5⅛ in. thick by 12 ft long by 4 ft wide, fabricated from DF Glulam Combination No. 2. From the questionnaire, it was determined that the Forest Service does not always provide a connection between consecutive transverse deck panels. Without a mechanism connecting adjacent panels to one another, the distribution of forces and deflections from one panel to the next panel was initially assumed to be minimal. This assumption allowed for the two rail segments supported by scupper blocks to be tested on separate glulam deck panels with a reduced risk of one test setup significantly affecting the other test setup. For this reason, one component test setup was expected to be placed on the second panel from one end of the bridge system, while the second component test setup was placed on the fourth panel from the same end of the bridge system. The component test setups were planned to be centered across the width of the second and fourth panels, which assumed that only one panel would provide resistance and result in a worst-case scenario for testing and evaluation.

When using glulam girders in this type of bridge system, glulam diaphragms are also utilized. FPL-GTR-125 [19] provides guidance on typical sizes and spacings for diaphragms based on girder length. Further, guidance is provided on methods used to connect the diaphragms to the girders. For a 20-ft long bridge system, two diaphragms are used between each set of girders, spaced 14 ft apart. Each line of diaphragms is placed 3 ft from the end of the girder, but the diaphragms are offset from this line, as shown in Figure 134 [19]. Although an offset of 6 in. typical due to interference between the tie rods that connect the diaphragms to the girders and the lag screws that connect the deck panels to the girders, an offset of 9 in. was used.

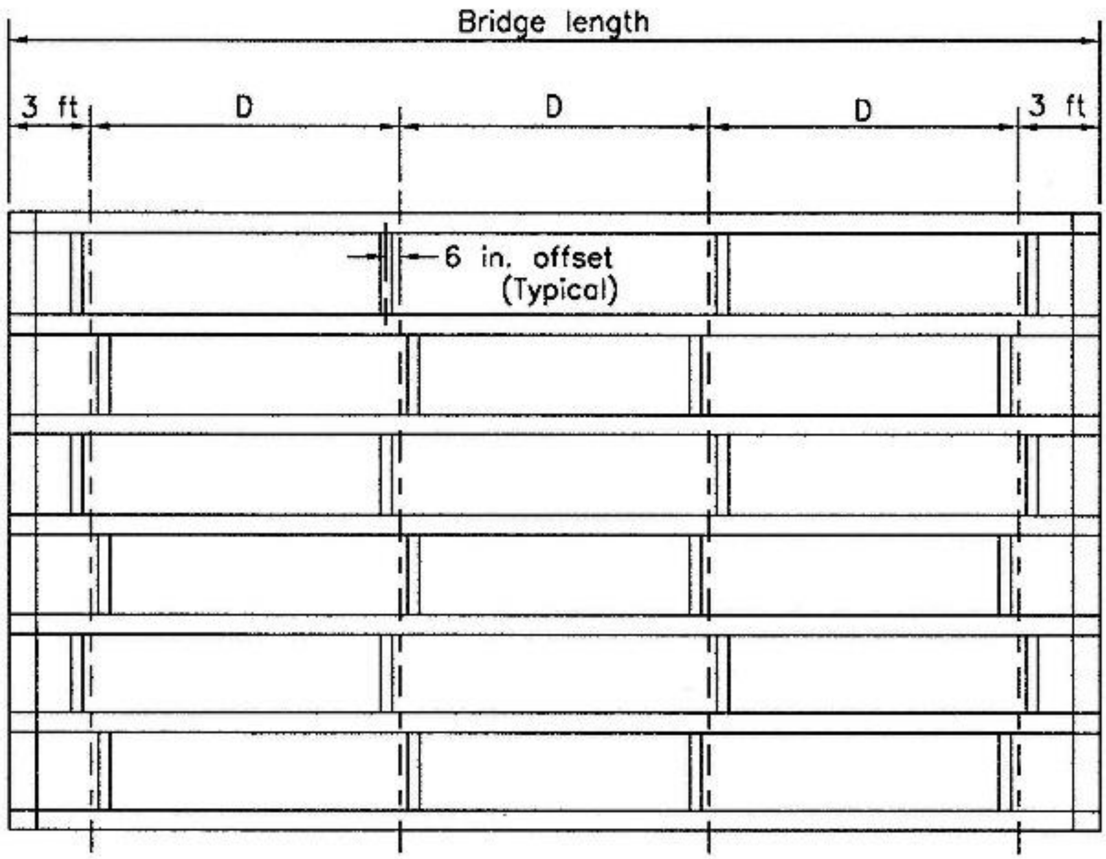



Figure 38. Typical Glulam Diaphragm Layout [19]

Per guidance from FPL-GTR-125 [19], glulam diaphragms are suggested to be 5⅛-in. thick, 8 in. shorter than the height of the girder, and ⅛ in. less than the clear span between girders. The diaphragms should be connected to the girders using two tie rods routed through the center of the diaphragm and anchored with malleable iron washers and nuts of the opposite sides of each girder. The diameter of the tie rods can be either ¾ or ⅞ in. The tie rod routes are typically centered on the diaphragm width and placed at the third glue line from the top and bottom surfaces [19]. For the previously-discussed girder configuration, the diaphragms would measure 5⅛ in. thick by 8½ in. tall by 41⅛ in. long. If an 8½-in. tall diaphragm were used, there would not be enough laminations to place two routes through the diaphragm, as this size only contains 5⅔ laminations. In order to adequately provide space for the tie rod connections within the diaphragms, a 12-in. tall diaphragm should be utilized, and the routes should be made at the second glue line from the top and bottom surfaces of each diaphragm for ¾-in. diameter tie rods.

For the final test bridge, each transverse, glulam timber deck panel measured 5⅛ in. thick by 4 ft wide by 12 ft long, spanned 20 ft (five 4-ft wide panels), and extended 2 ft beyond the vertical centerline of the two exterior girders. The deck panels were constructed from Combination No. 2 Douglas Fir (DF) and were treated using 0.60 lb/ft$^3$ of pentachlorophenol in heavy oil. To anchor the timber deck panels to the glulam girders, ¾-in. diameter x 11-in. long steel lag bolts were anchored into each of the three glulam girders using a longitudinal spacing of 12 in. As an alternative attachment mechanism, steel anchor brackets could have been used to attached to the underside of the deck and then connected to the sides of the glulam girders. However, lag bolts were utilized to test the most critical design. For further discussion on the connection design decisions for the glulam timber deck, see Appendix A. The heads of the lag bolts were countersunk into the deck panels. Again, the deck panels were placed directly adjacent to one another but were not inter-connected. The lines of holes to recess lag bolts for making the deck-to-girder connection is shown in Figure 39(d).

Once the glulam deck was anchored to the glulam girders, a 2-in. thick, surrogate wearing surface was placed over the timber deck. For testing purposes, two sheets of plywood with spacer boards were laid on top of one another and placed over the deck. This surrogate wearing surface was placed on the timber deck for the testing and evaluation program to reduce construction costs. For the long-term use of the bridge system, it is recommended that a permanent, durable material (i.e., asphalt) be used to surface and protect the timber deck.

Overall, the support for the transverse, glulam timber bridge deck system consisted of two concrete bents and three glulam girders. Glulam diaphragms provided bracing for the glulam girders, which were placed between the three girders at a longitudinal spacing of 15½ ft. The glulam girders were made from Combination No. 48 Southern Yellow Pine (SYP) material, and the diaphragm members were constructed from Combination No 2 Douglas Fir (DF) material. Additionally, all glulam members were treated with 0.60 lb/ft$^3$ of pentachlorophenol in heavy oil. For construction photographs documenting the installation of the glulam timber deck, see in Figure 39. For design and CAD details for the glulam test bridge and substructure, see Figures 40 through 58.

(a) Connection Details Between Bent to Glulam Girder

(b) 2-in. Thick Surrogate Wearing Surface on Glulam Deck

(c) Construction of Glulam Girders and Diaphragms

(d) Layout of Glulam Panels on Deck

Figure 39. Construction of Transverse, Glulam Timber Deck

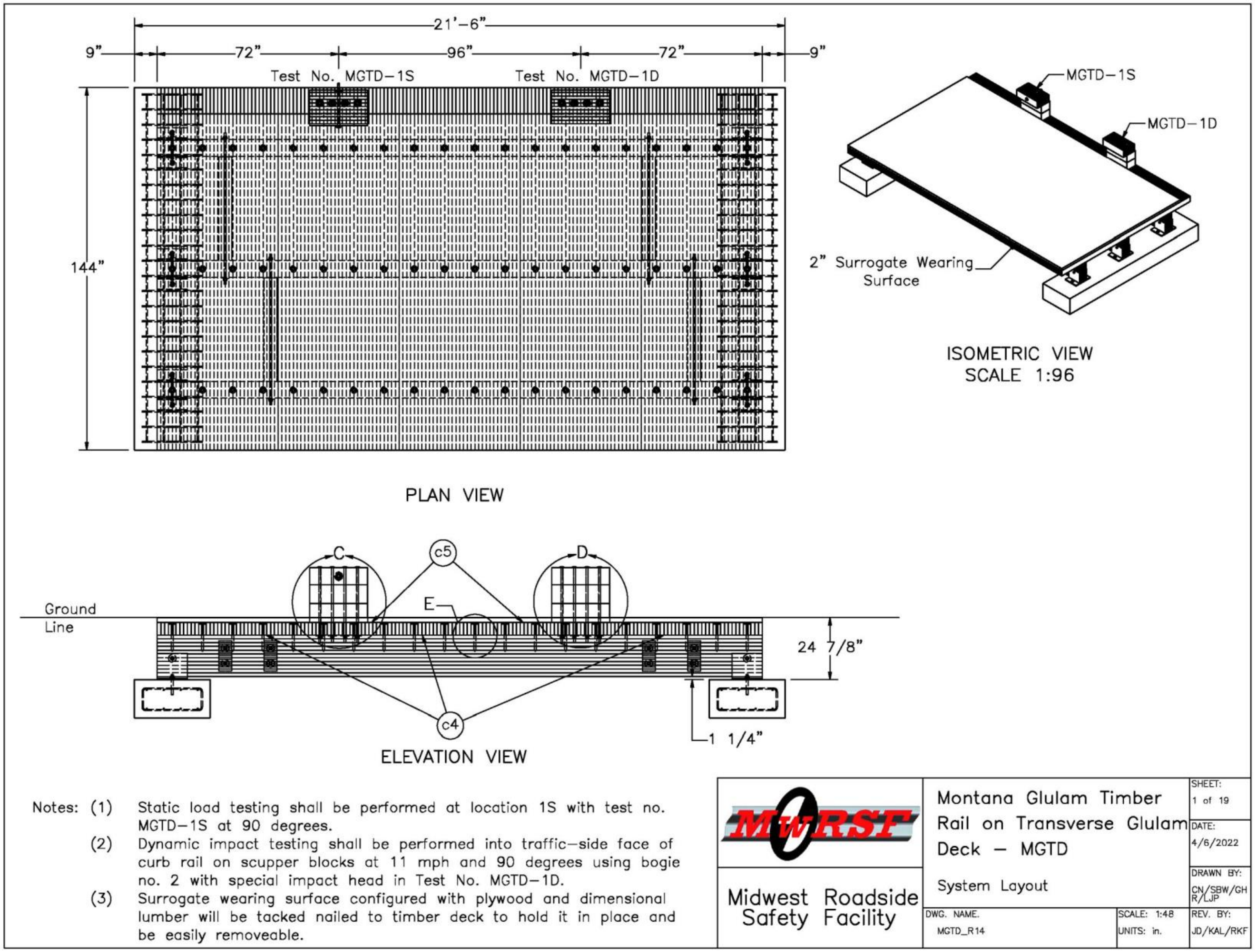



Figure 40. Transverse, Glulam Timber Deck and Test Nos. MGTD–1S and MGTD-1D System Layout

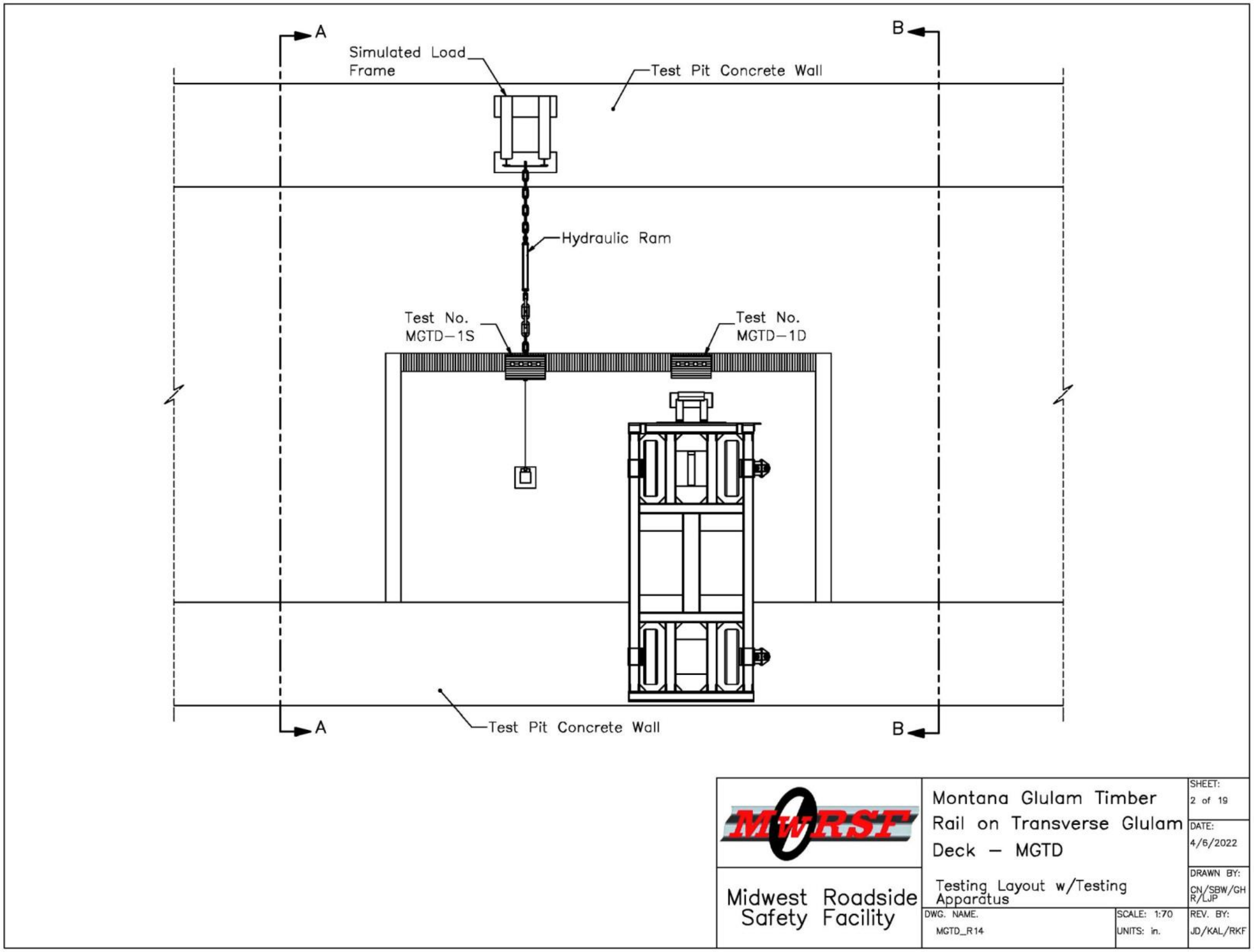



Figure 41. Test Plan and Layout, Test Nos. MGTD–1S and MGTD-1D

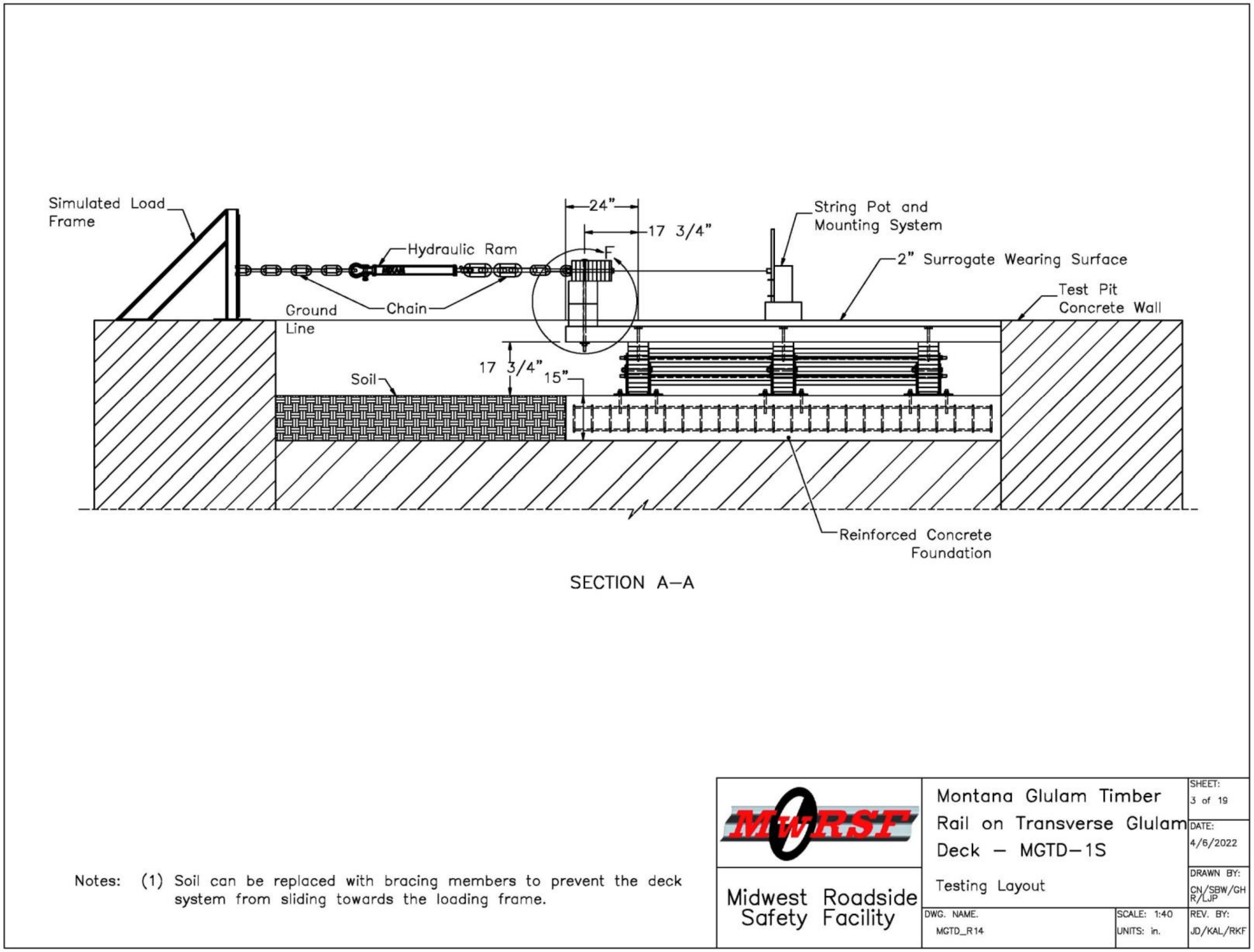
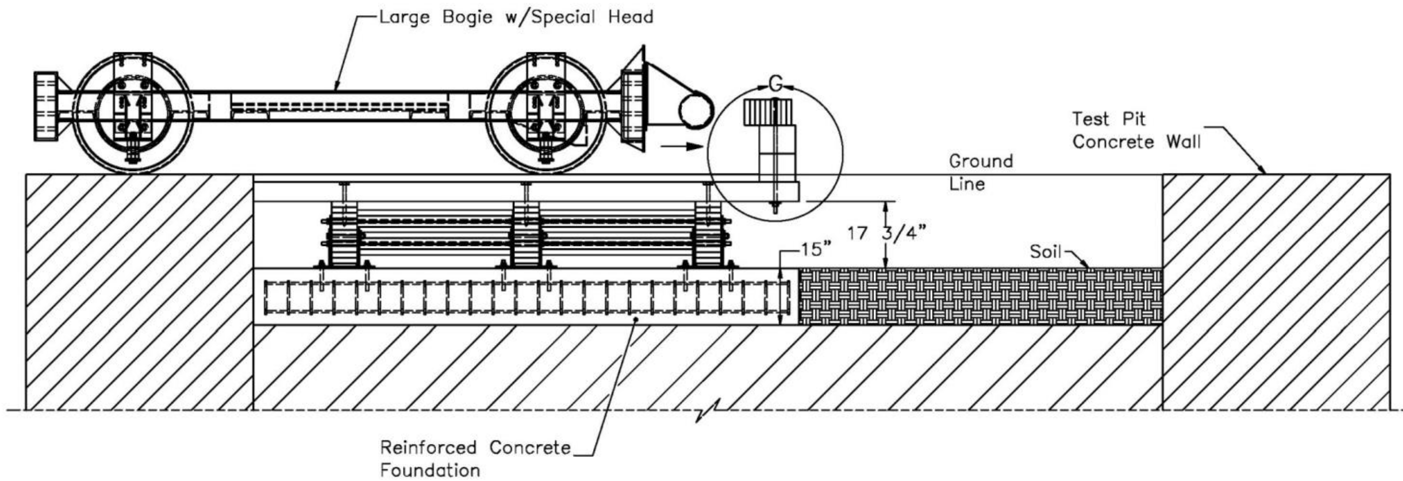



Figure 42. Testing Layout, Test No. MGTD-1S




SECTION B–B

Notes: (1) The special head of the bogie should impact at the mid–height of the upper rail.

| | | | |
|---|---|---|---|
| MwRSF | Montana Glulam Timber Rail on Transverse Glulam Deck – MGTD–1D | | SHEET: 4 of 19 |
| | | | DATE: 4/6/2022 |
| Midwest Roadside Safety Facility | Testing Layout | | DRAWN BY: CN/SBW/GH R/LJP |
| | DWG. NAME. MGTD_R14 | SCALE: 1:40 UNITS: in. | REV. BY: JD/KAL/RKF |

Figure 43. Testing Layout, Test No. MGTD-1D

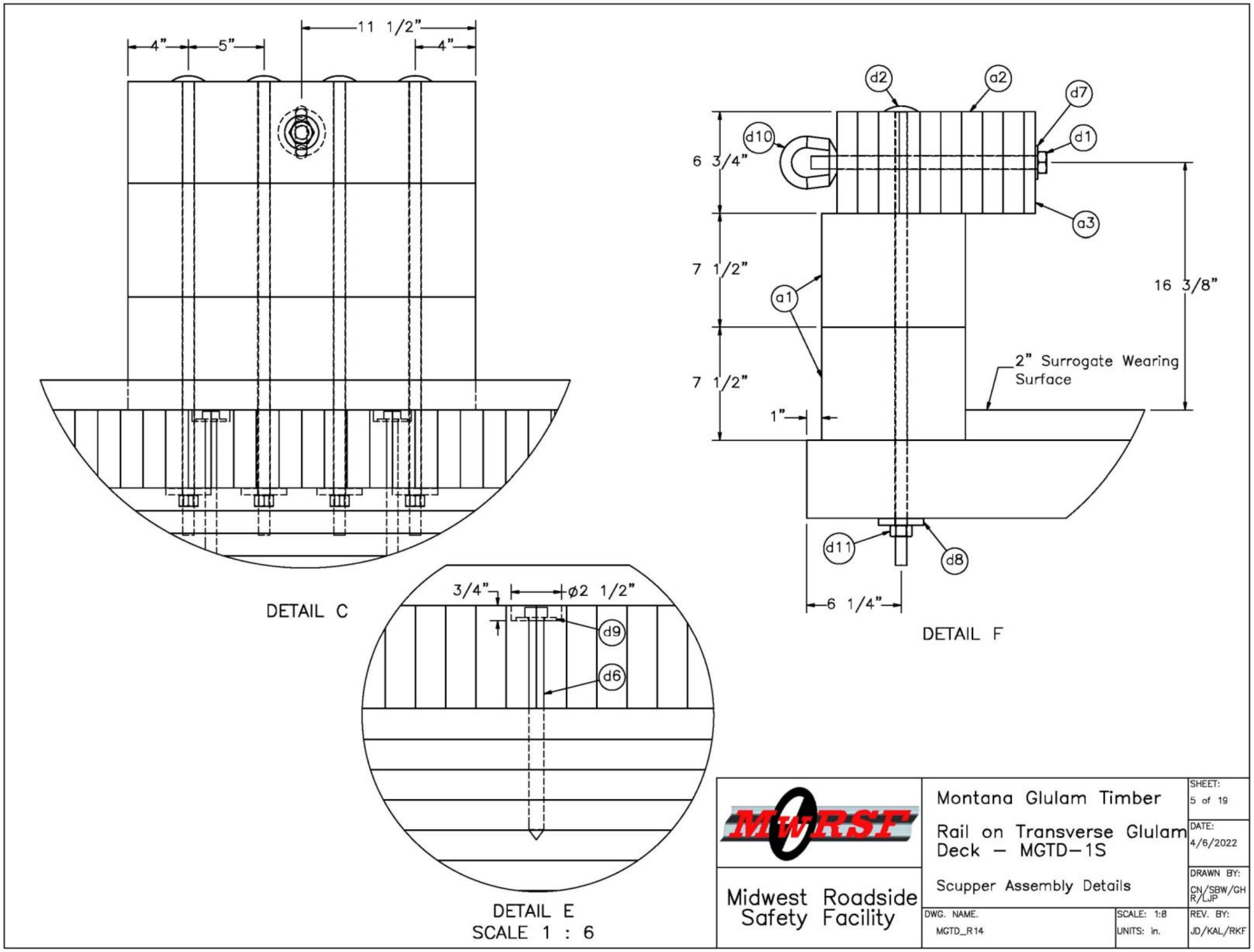



Figure 44. Scupper Assembly Details, Test No. MGTD–1S

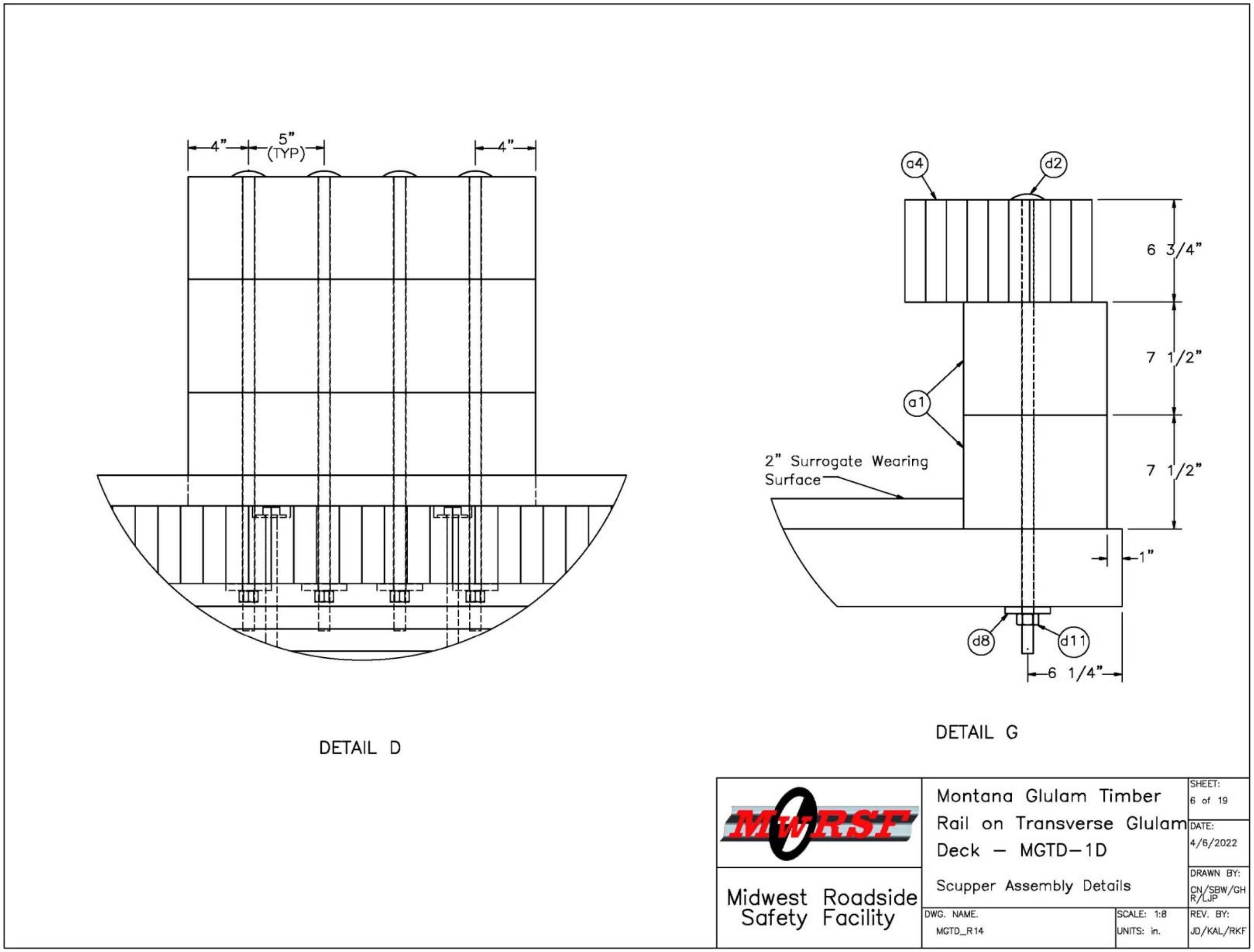



Figure 45. Scupper Assembly Details, Test No. MGTD-1D

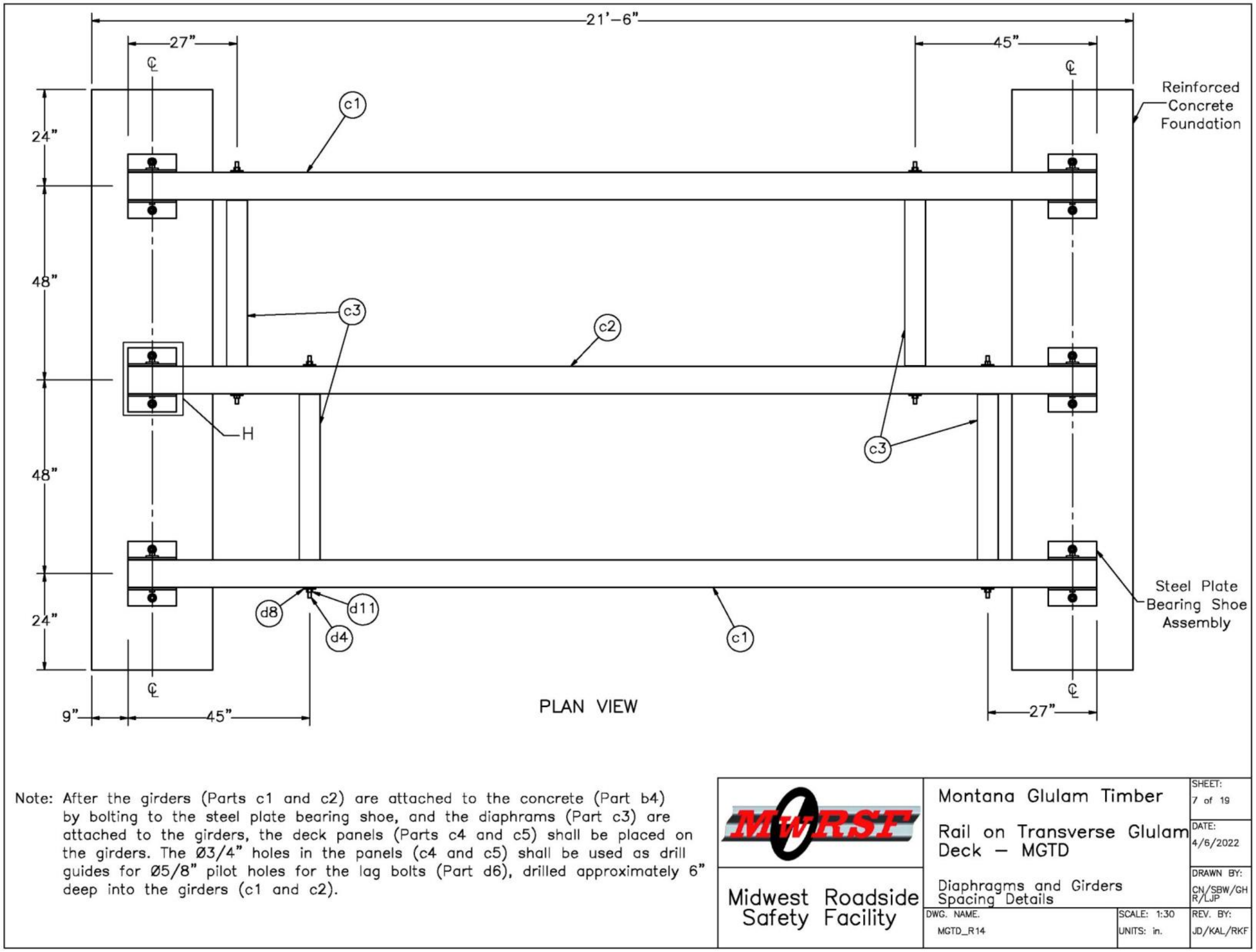



Figure 46. Diaphragms and Girders Spacing Details, Test Nos. MGTD-1S and MGTD-1D

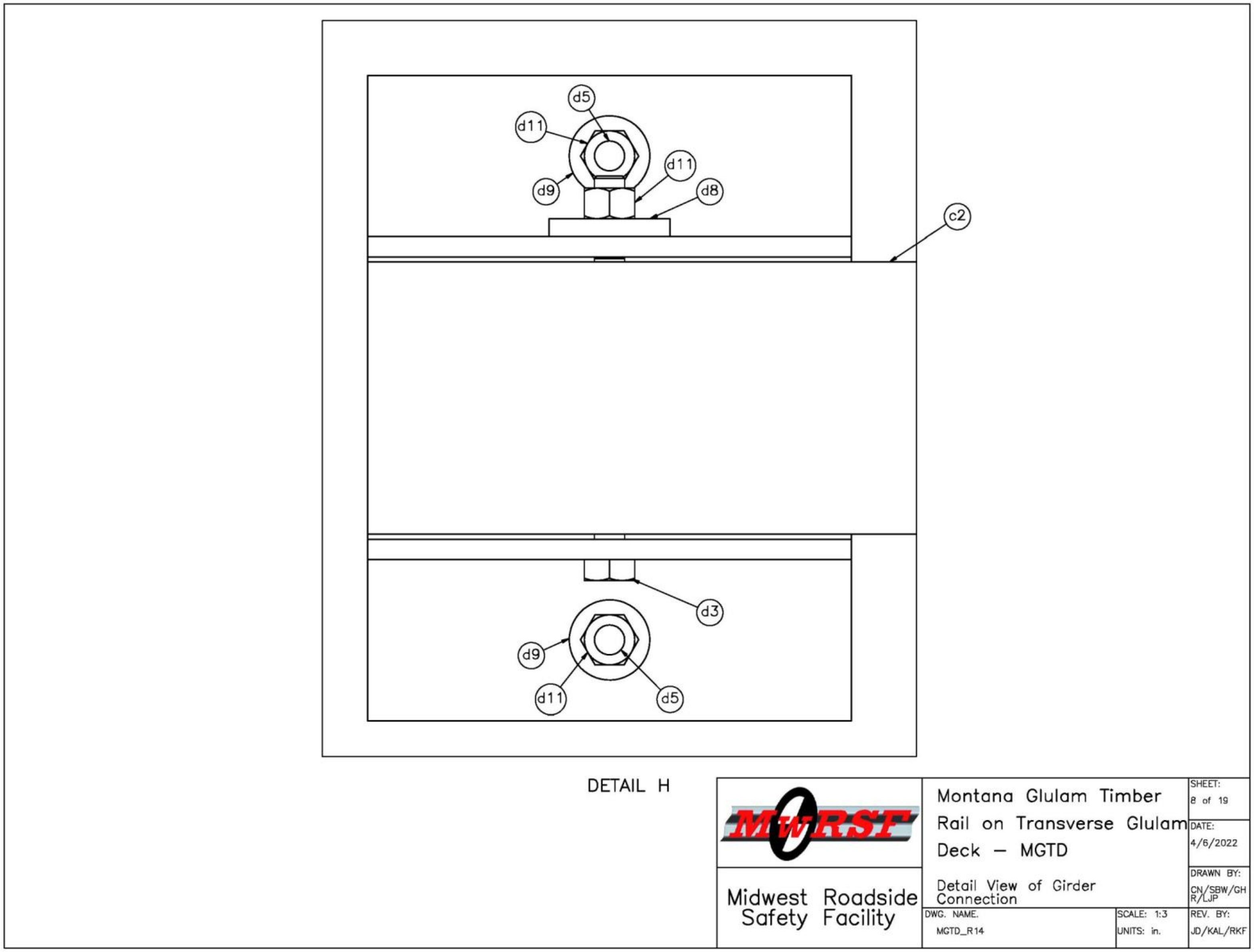



Figure 47. Detail View of Girder Connection, Test Nos. MGTD-1S and MGTD-1D

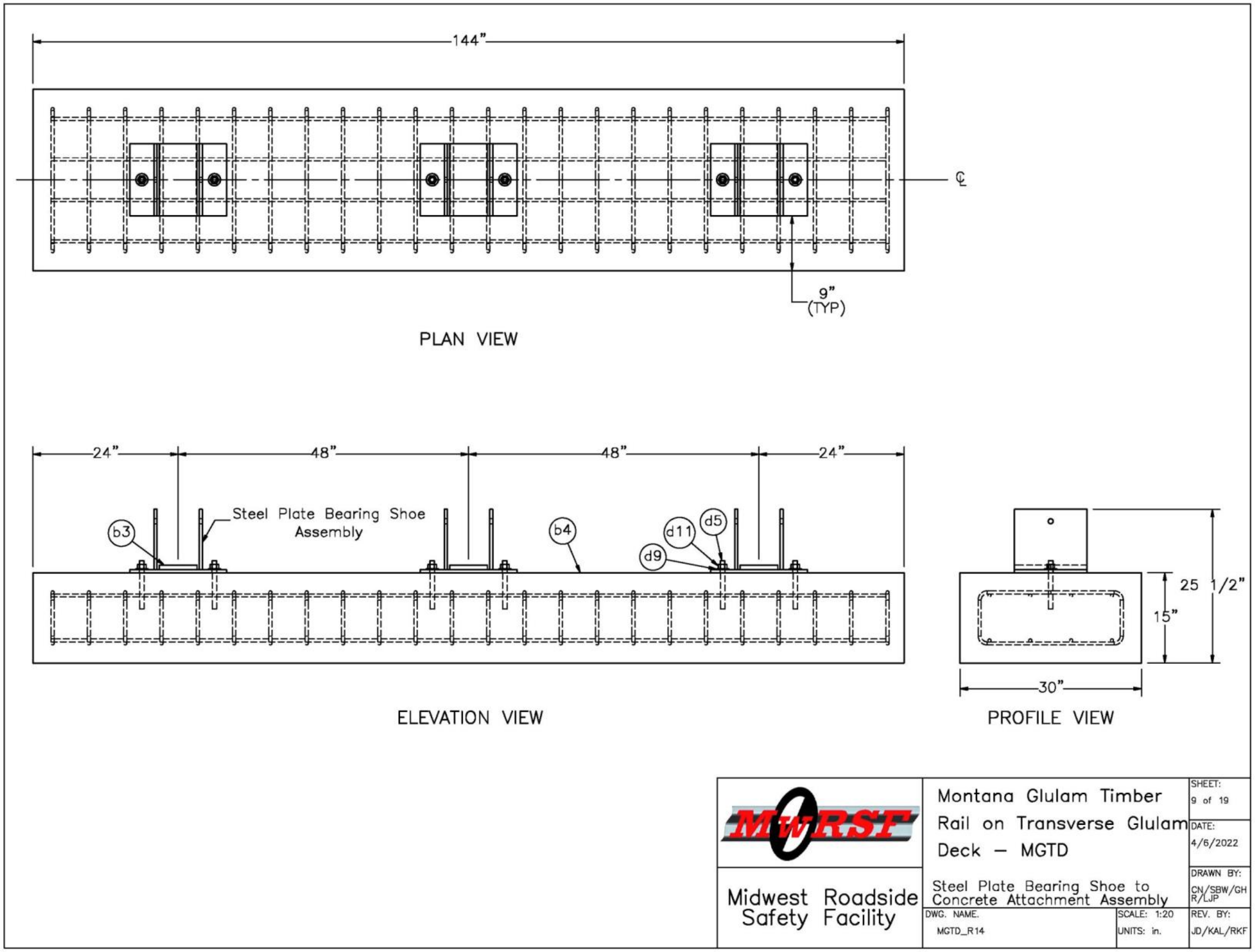



Figure 48. Steel Plate Bearing Shoe to Concrete Attachment Assembly, Test Nos. MGTD-1S and MGTD-1D

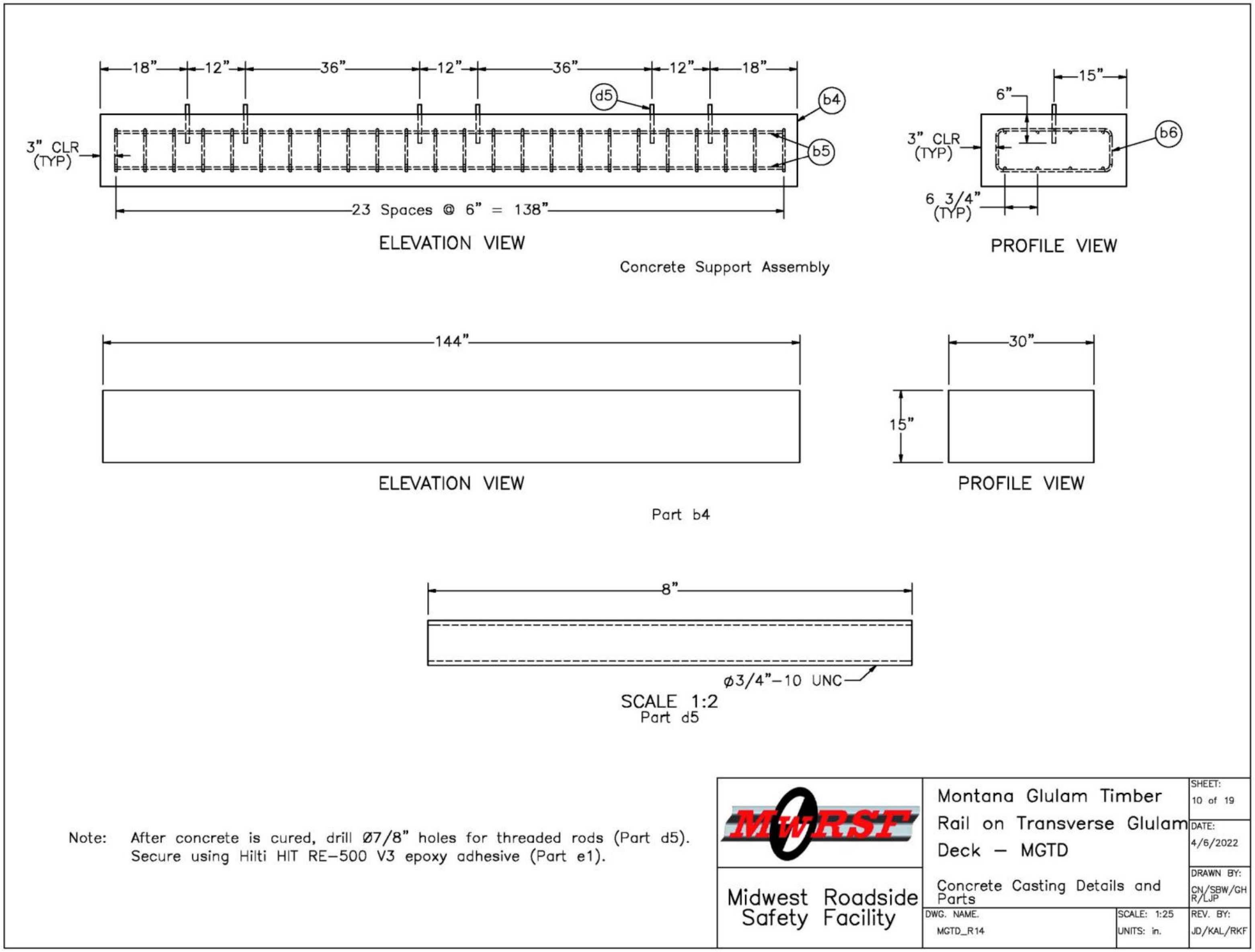
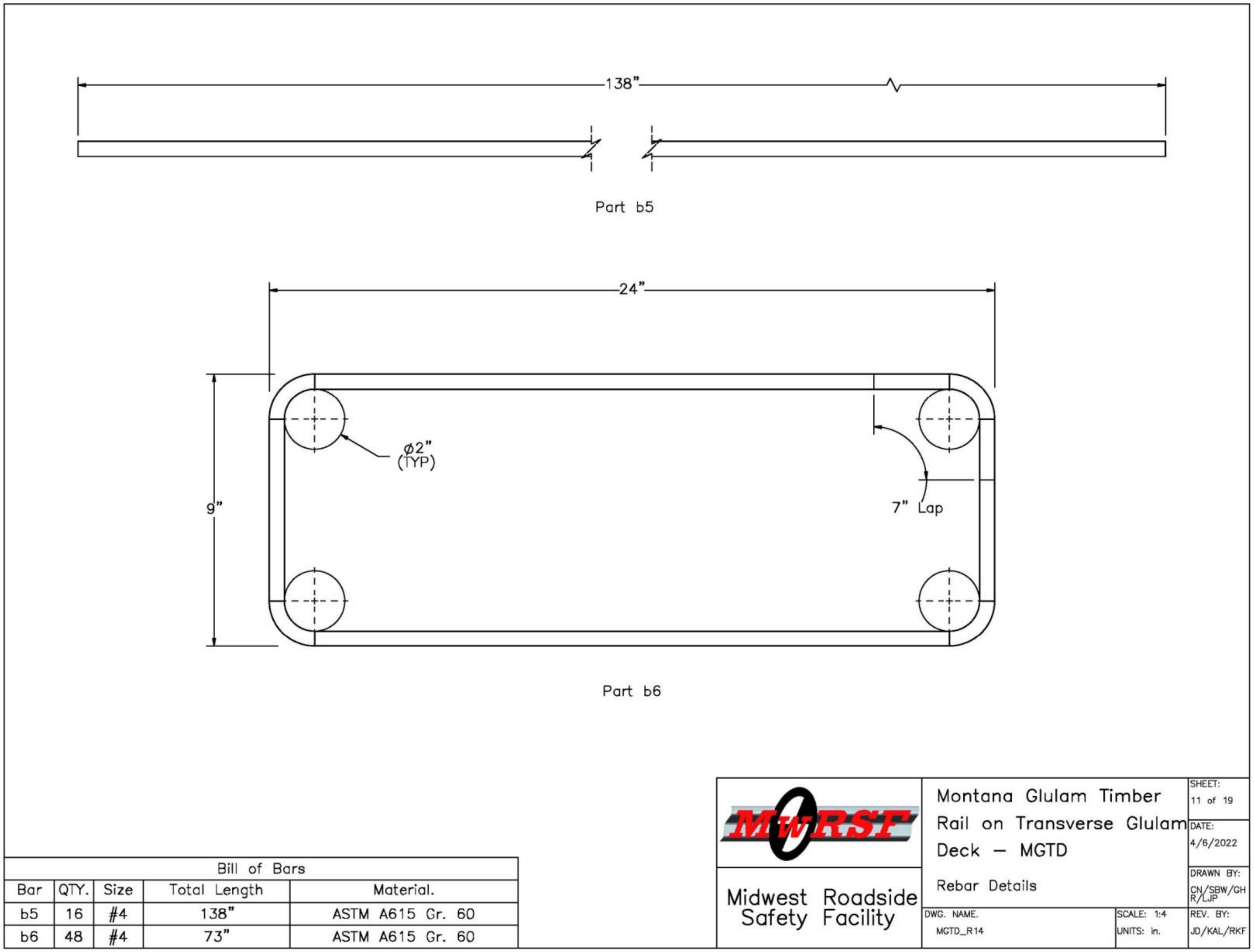
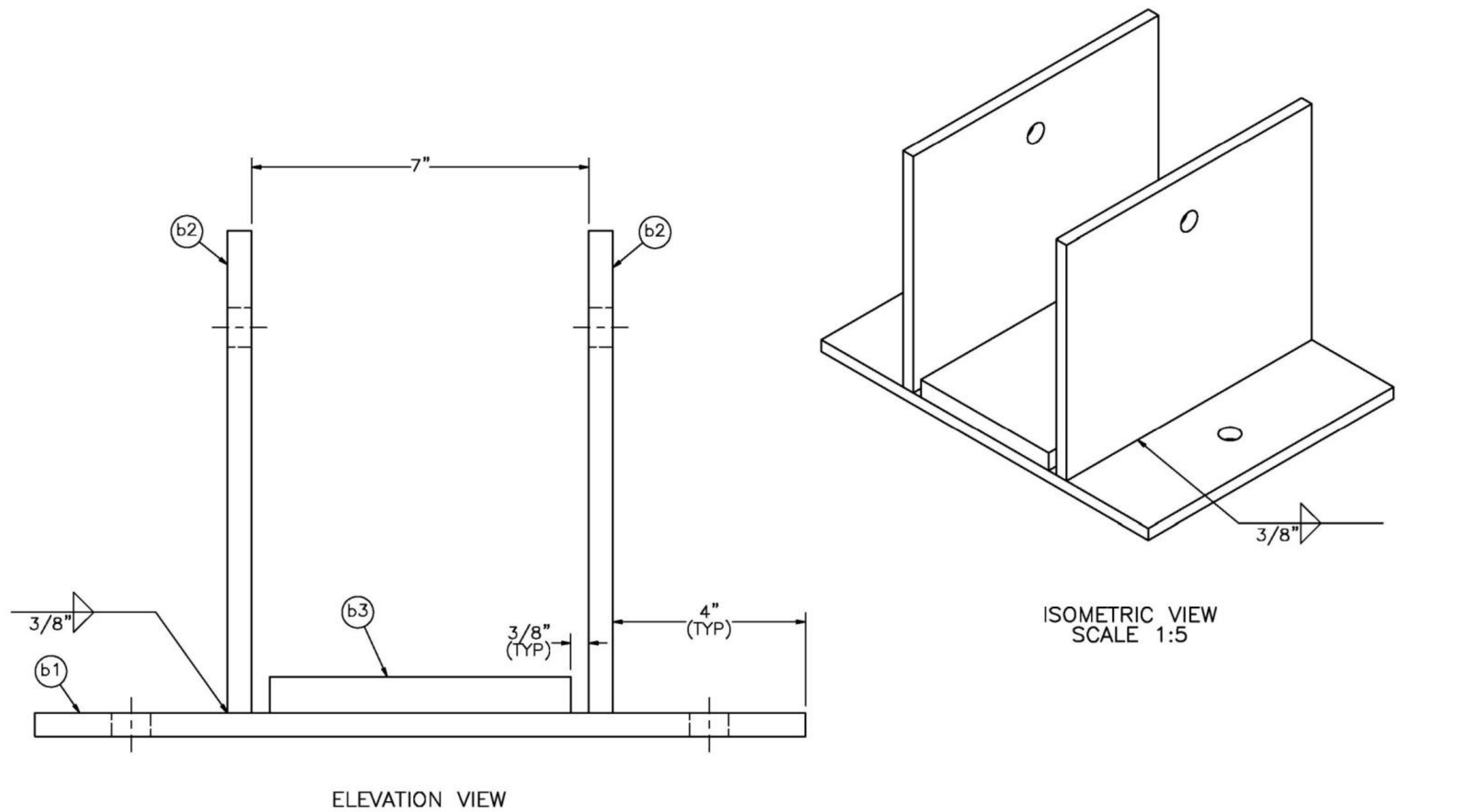



Figure 49. Concrete Casting Details and Parts, Test Nos. MGTD-1S and MGTD-1D




| Bill of Bars | | | | |
|---|---|---|---|---|
| Bar | QTY. | Size | Total Length | Material. |
| b5 | 16 | #4 | 138" | ASTM A615 Gr. 60 |
| b6 | 48 | #4 | 73" | ASTM A615 Gr. 60 |

Figure 50. Rebar Details, Test Nos. MGTD-1S and MGTD-1D




| Item No. | QTY. | Description | Material Specification | Treatment Specification |
|---|---|---|---|---|
| – | 6 | Steel Plate Bearing Shoe Assembly | – | – |
| b1 | 1 | 12"x16"x1/2" Steel Base Plate | ASTM A36 | ASTM A123 |
| b2 | 2 | 12"x10"x1/2" Side Plate | ASTM A36 | ASTM A123 |
| b3 | 1 | 12"x6 1/4"x3/4" Elastomeric Bearing Pad | Neoprene – Min. 50 Durameter | – |

Midwest Roadside Safety Facility

Montana Glulam Timber Rail on Transverse Glulam Deck – MGTD

Steel Plate Bearing Shoe Assembly

SHEET: 12 of 19
DATE: 4/6/2022
DRAWN BY: CN/SBW/GHR/LJP
DWG. NAME. MGTD_R14
SCALE: 1:3
UNITS: in.
REV. BY: JD/KAL/RKF

Figure 51. Steel Plate Bearing Shoe Assembly, Test Nos. MGTD-1S and MGTD-1D

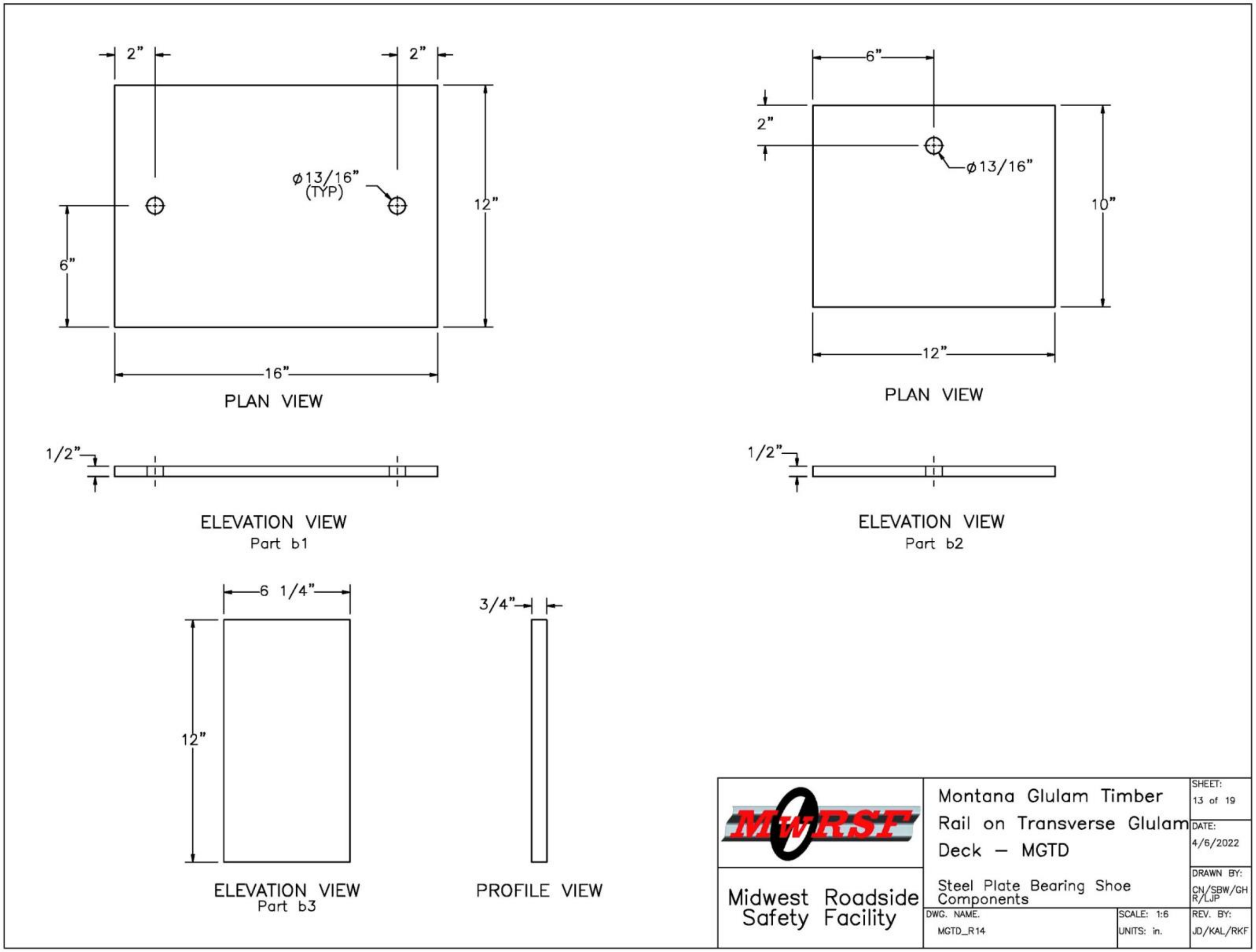



Figure 52. Steel Plate Bearing Shoe Components, Test Nos. MGTD-1S and MGTD-1D

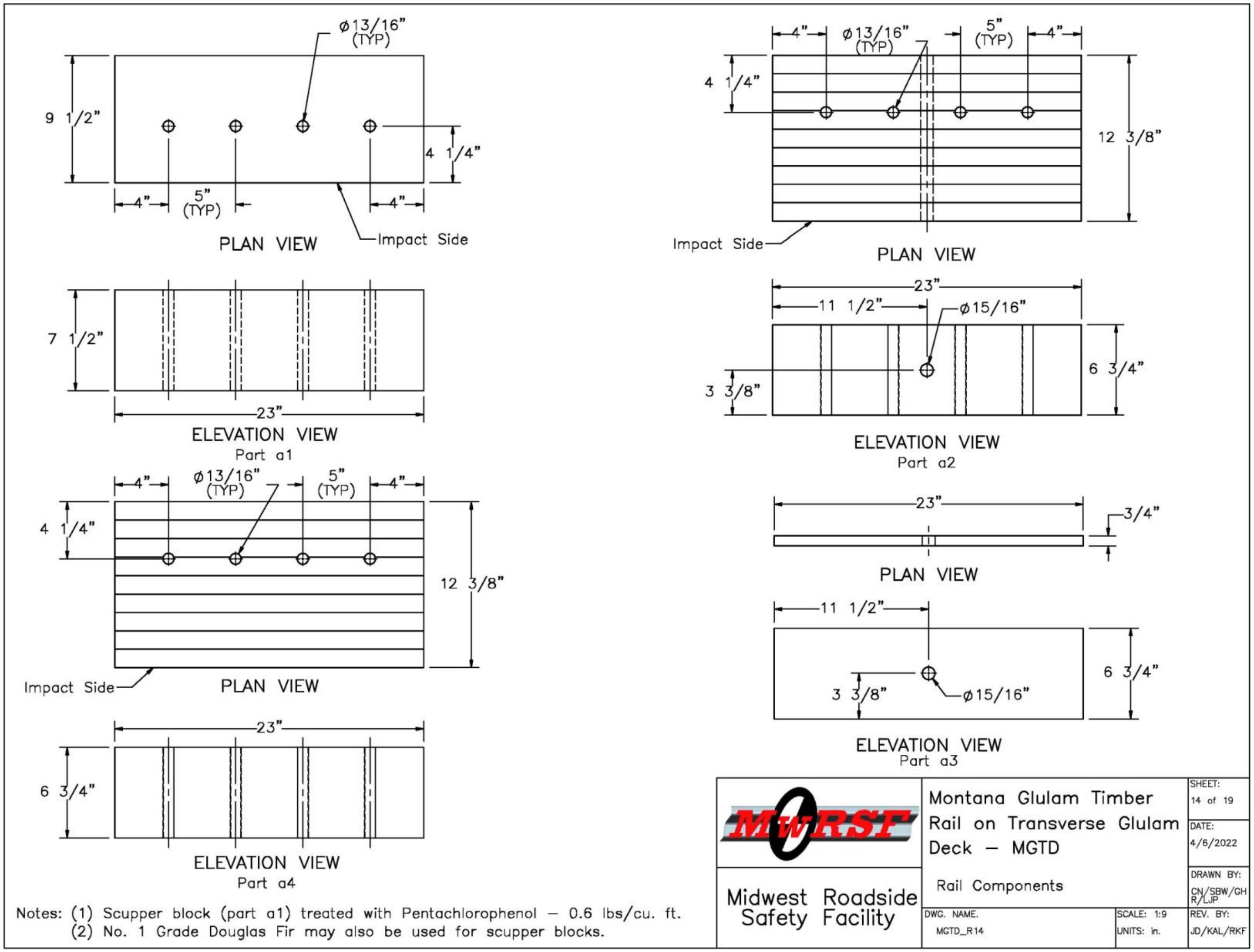



Figure 53. Rail Components, Test Nos. MGTD-1S and MGTD-1D

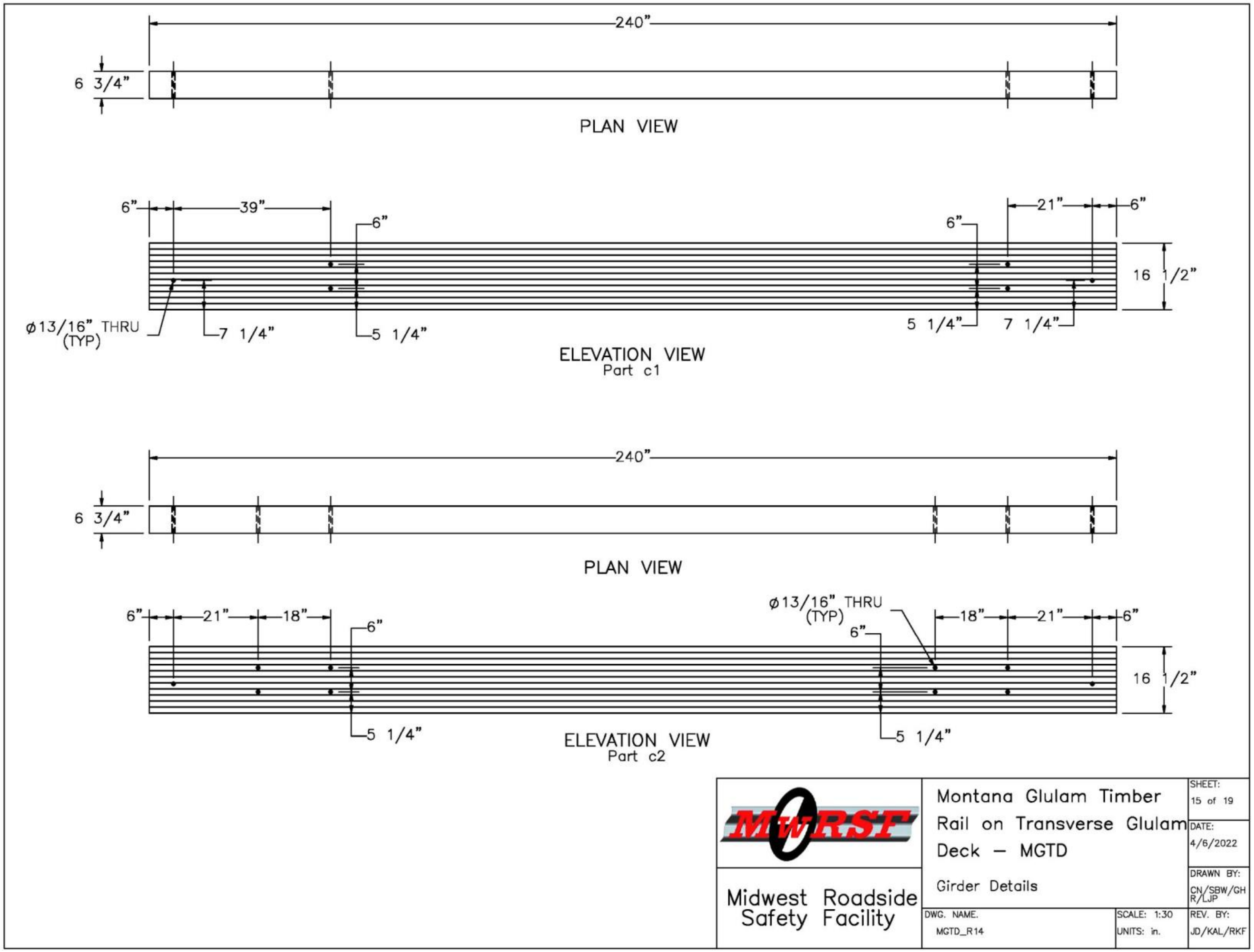



Figure 54. Girder Details, Test Nos. MGTD-1S and MGTD-1D

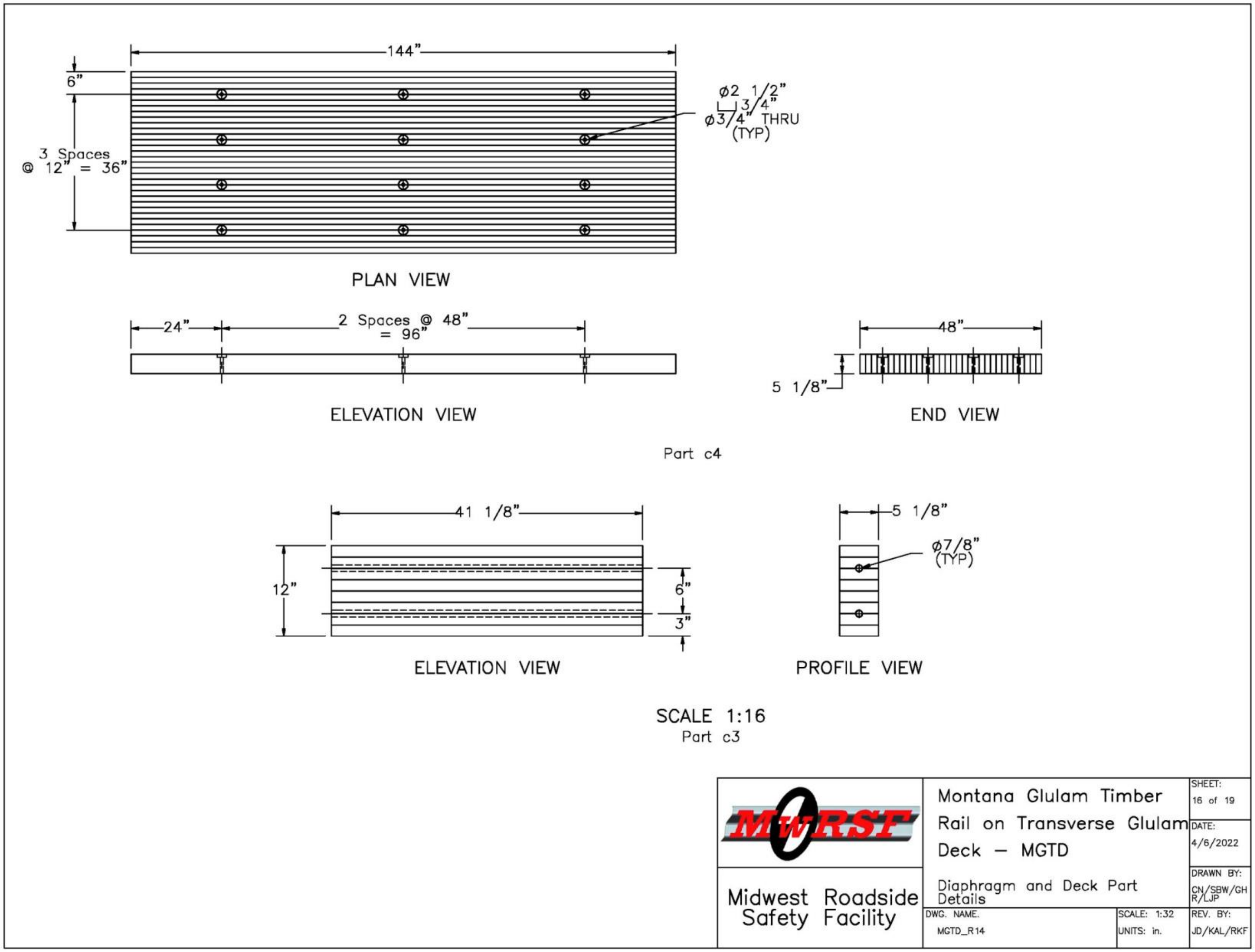


Figure 55. Diaphragm and Deck Part Details, Test Nos. MGTD-1S and MGTD-1D

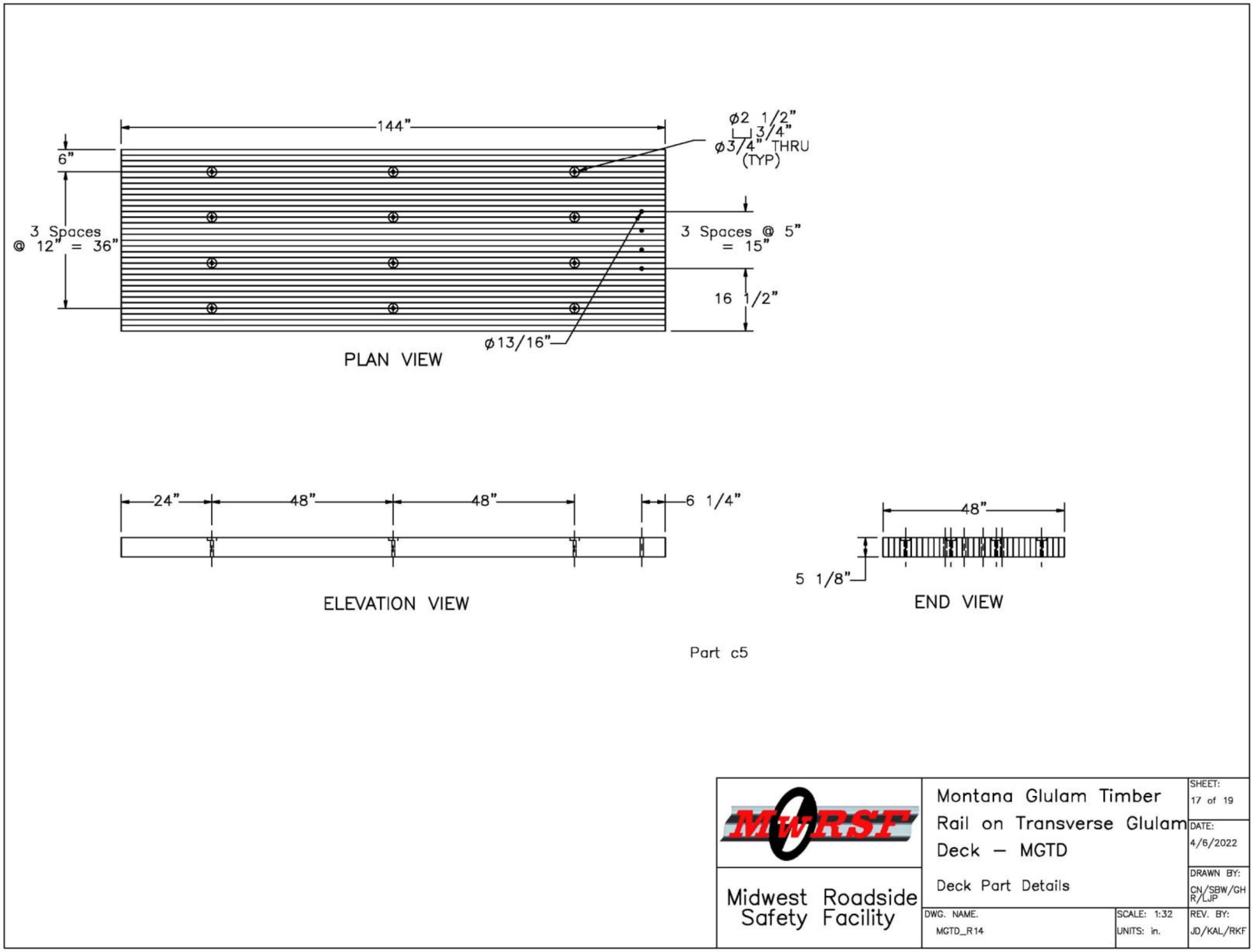



Figure 56. Deck Part Details, Test Nos. MGTD-1S and MGTD-1D

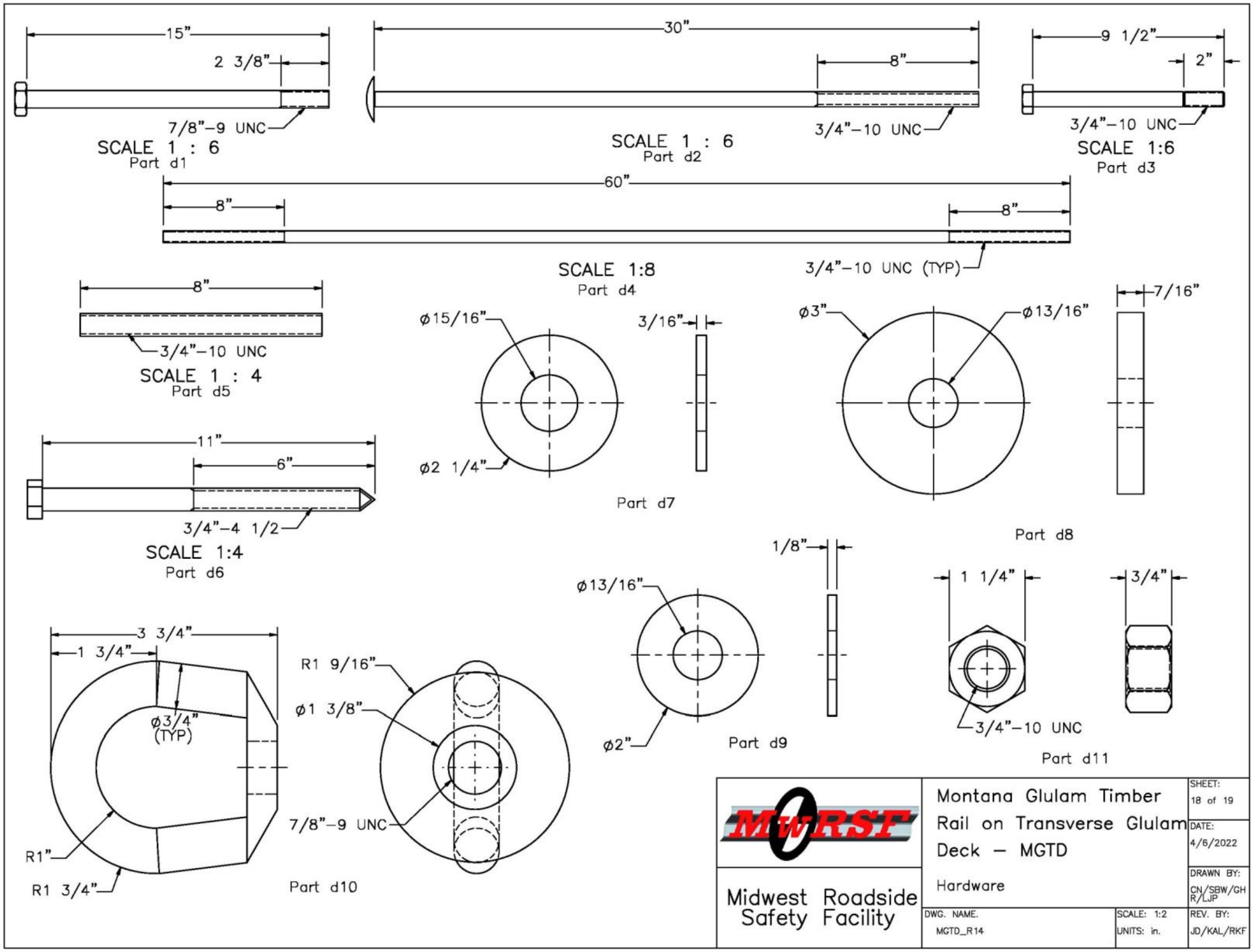



Figure 57. Hardware, Test Nos. MGTD-1S and MGTD-1D

| Item No. | QTY. | Description | Material Specification | Treatment Specification | Hardware Guide |
|---|---|---|---|---|---|
| a1 | 4 | 9 1/2"x23"x7 1/2" Scupper Block | Grade No. 1 Southern Yellow Pine or Douglas Fir | Pentachlorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a2 | 1 | 12 3/8"x23"x6 3/4" Glulam Segment | Southern Yellow Pine Combination No. 48 | Pentachlorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| a3 | 1 | 23"x6 3/4"x3/4" Static Test Plate | ASTM A36 | ASTM A123 | – |
| a4 | 1 | 12 3/8"x23"x6 3/4" Glulam Segment | Southern Yellow Pine Combination No. 48 | Pentachlorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| b1 | 6 | 12"x16"x1/2" Steel Base Plate | ASTM A36 | ASTM A123 | – |
| b2 | 12 | 12"x10"x1/2" Side Plate | ASTM A36 | ASTM A123 | – |
| b3 | 6 | 12"x6 1/4"x3/4" Elastomeric Bearing Pad | Neoprene – Min. 50 Durameter | – | – |
| b4 | 2 | 15"x30"x12' Concrete Support | Min f'c = 4,000 psi NE mix 47 BD | – | – |
| b5 | 16 | #4 Rebar, 138" Long | ASTM A615 Gr. 60 | Epoxy Coated (ASTM A775 or A934) | – |
| b6 | 48 | #4 Rebar, 73" Unbent Length | ASTM A615 Gr. 60 | Epoxy Coated (ASTM A775 or A934) | – |
| c1 | 2 | 16 1/2"x6 3/4"x20' Long Outside Glulam Girder | 24F–V4 Douglas Fir | Pentachorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| c2 | 1 | 16 1/2"x6 3/4"x20' Long Glulam Girder | 24F–V4 Douglas Fir | Pentachorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| c3 | 4 | 12"x5 1/8"x41 1/8" Long Glulam Diaphragms | Comb. No. 2 Douglas Fir | Pentachorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| c4 | 3 | 5 1/8"x4'x12' Long Glulam Deck Panel | Comb. No. 2 Douglas Fir | Pentachorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| c5 | 2 | 5 1/8"x4'x12' Long Glulam Deck Panel | Comb. No. 2 Douglas Fir | Pentachorophenal in Heavy Oil 0.6 lbs/cu. ft Retention | – |
| d1 | 1 | 7/8"–9 UNC x 15" Heavy Hex Bolt | ASTM F3125 Gr. A325 | ASTM A123 | FBX22b |
| d2 | 8 | 3/4"–10 UNC x 30" Timber Bolt w/Nubs | ASTM A307A | ASTM A123 | FBB08 |
| d3 | 6 | 3/4"–10 UNC x 9 1/2" Hex Bolt | ASTM A307A | ASTM A123 | FBX20a |
| d4 | 8 | 3/4"–10 UNC x 8" on a 60" Long Tie Rod | ASTM A307A or F1554 Gr. 36 or SAE J429 Gr. 2 | ASTM A123 | FRR28a |
| d5 | 12 | 3/4"–10 UNC x 8" Threaded Rod | ASTM A193 Gr. B7 or SAE J429 Gr. 5 | ASTM A123 | FRR20a |
| d6 | 60 | 3/4"–4 1/2 x 11" Lag Bolt | ASTM A307A | ASTM A123 | FBL20 |
| d7 | 1 | 7/8" Flat Washer | ASTM F844 | ASTM A123 or A153 or F2329 | FWC22a |
| d8 | 30 | 3/4" Malleable Iron Washer | ASTM A47 | ASTM A123 | – |
| d9 | 72 | 3/4" Flat Washer | ASTM F844 | ASTM A123 or A153 or F2329 | FWC20a |
| d10 | 1 | 7/8"–9 UNC Eye Nut | ASTM A325 | ASTM A123 | – |
| d11 | 42 | 3/4"–10 UNC Heavy Hex Nut | ASTM A536A | ASTM A123 | FNX20b |
| e1 | – | Epoxy Adhesive | Hilti HIT RE–500 V3 | – | – |

Midwest Roadside Safety Facility

Montana Glulam Timber Rail on Transverse Glulam Deck – MGTD

Bill of Materials

SHEET: 19 of 19
DATE: 4/6/2022
DRAWN BY: CN/SBW/GHR/LJP
DWG. NAME. MGTD_R14
SCALE: None
UNITS: in.
REV. BY: JD/KAL/RKF

Figure 58. Bill of Materials, Test Nos. MGTD-1S and MGTD-1D

# 6 COMPONENT TESTING PROGRAM

## 6.1 Plan

For this study, the research team was tasked with demonstrating that the MASH TL-1 low-height, glulam bridge railing system would perform in an acceptable matter when adapted from transverse, nail-laminated timber bridge decks to transverse, glulam timber bridge decks. To perform this effort, dynamic and static component tests were planned to demonstrate that the bridge railing system provides equivalent or greater lateral stiffness, strength, and energy dissipation, when installed on transverse, glulam decks as compared to transverse, nail-laminated decks.

Thus, the research team configured 3-D test plans and CAD details for constructing one surrogate glulam bridge deck system and one surrogate nail-laminated bridge deck system. Each bridge system had two short glulam rail segments supported and anchored to the deck using two scupper blocks. One static and one dynamic component test was conducted on each deck type and analyzed to compare lateral stiffness, strength, energy dissipation, and overall performance between deck types. Upon completion of the component tests, the test results would be compared to one another, and conclusions would be made regarding the performance and crashworthiness of the low-height, glulam bridge rail installed on both transverse glulam and transverse nail-lam bridge decks.

## 6.2 Dynamic Testing Setup

Two dynamic component tests were planned on short sections of the bridge rail where the rail segment was supported by two scupper blocks and anchored to the timber decks. The first dynamic test was planned for the nail-laminated, timber deck, and the second dynamic test was planned for the glulam timber deck. The short rail section consisted of a 23-in. long, timber glulam rail segment, which was supported by two glulam scupper blocks. The rail segment was fabricated using the originally-requested Combination No. 48 SYP timber. Originally, the scupper block material for the dynamic component testing program was to be manufactured using Grade No. 1 SYP. However, due to supply chain issues, the scupper block material was replaced with a glulam substitute. The scupper blocks and rail segments were treated with 0.60 lb/ft$^3$ of pentachlorophenol in heavy oil. The scupper blocks and rail segments were connected and anchored to each deck type using four ¾-in. diameter x 10 UNC by 30-in. long ASTM A307 Grade A timber bolts. The top railing height was 21¾ in. above the timber deck and 19¾ in. above the surrogate wearing surface. The timber bolts were then fastened to the bottom of each deck type using ¾-in. diameter, malleable iron washers with nuts.

For the glulam deck, the target impact speed of the bogie was set for 11 mph at an impact angle of 90 degrees. For the nail-laminated deck, the target impact speed of the bogie was 13 mph at an impact angle of 90 degrees. The midpoint of the bogie's rigid head was targeted to impact the centerline of the glulam rail at 16⅜ in. above the 2-in. thick, surrogate wearing surface. The test matrix for the two dynamic tests is shown in Table 5 and the test setup for the dynamic tests performed on the nail and glue laminated deck is shown in Figures 13 and 15 and Figures 41 and 43 respectively. Additionally, material specifications, mill certifications, and certificates of conformity for the two bridge railing and deck systems are shown in Appendix C and Appendix D.

Table 5. Bogie Testing Matrix

| Test No. | Deck Type | Target Impact Speed (mph) | Impact Height (in.) |
|---|---|---|---|
| MGTD-1D | Glulam | 11 | 16⅜ |
| MGTR-1D | Nail-Laminated | 13 | 16⅜ |

## 6.3 Dynamic Testing Equipment and Instrumentation

Equipment and instrumentation utilized to collect and record data for the dynamic bogie tests included a bogie vehicle, a test jig, accelerometers, a retroreflective speed trap, high-speed and standard-speed digital video, and still cameras.

### 6.3.1 Accelerometers

Two accelerometer systems were mounted on the bogie vehicle near its center of gravity (c.g.) to measure the acceleration in the longitudinal, lateral, and vertical directions. However, only the longitudinal acceleration was processed and reported.

The SLICE-1 and SLICE-2 units were modular data acquisition systems manufactured by Diversified Technical Systems, Inc. of Seal Beach, California. Triaxial acceleration and angular rate sensor modules were mounted inside the bodies of custom-built SLICE 6DX event data recorders equipped with 7GB of non-volatile flash memory and recorded data at 10,000 Hz to the onboard microprocessor. The accelerometers had a range of ±500g's in each of three directions (longitudinal, lateral, and vertical) and a 1,650 Hz (CFC 1000) anti-aliasing filter. The SLICE MICRO Triax ARS had a range of 1,500 degrees/sec in each of three directions (roll, pitch, and yaw). The raw angular rate measurements were downloaded, converted to the proper Euler angles for analysis, and plotted. The "SLICEWare" computer software program and a customized Microsoft Excel worksheet were used to analyze and plot both the accelerometer and angular rate sensor data.

### 6.3.2 Bogie Vehicle

A rigid-frame bogie was used to impact the low-height, glulam timber rail and scupper blocks. The same impact head was used for both dynamic tests. The bogie head was constructed of 8-in. diameter, ½-in. thick standard steel pipe, with ¾-in. neoprene belting wrapped around the pipe to prevent local damage to the post from the impact. The impact head was bolted to the bogie vehicle, creating a rigid attachment with an impact height of 16⅜ in. for test nos. MGTD-1D and MGTR-1D. The bogie vehicle with impact head is shown in Figure 59. The weight of the bogie with the impact head and accelerometers was 5,220 lb for both test nos. MGTD-1D and MGTR-1D.

The bogie tests were conducted using a steel corrugated beam guardrail to guide the tires of the bogie vehicle. A pickup truck was used to push the bogie vehicle to the required impact velocity. After reaching the target velocity, the push vehicle braked, allowing the bogie to be free rolling as it came off the track before impacting the bridge rail segment. A radio-controlled brake system was installed on the bogie, allowing it to be brought safely to rest after the test.

(a) Test No. MGTD-1D (Glulam)

(b) Test No. MGTR-1D (Nail-Lam)

Figure 59. Bogie Vehicle for Dynamic Component Tests – (a) Test No. MGTD-1D (Glulam) and (b) Test No. MGTR-1D (Nail-Lam)

### 6.3.3 Retroreflective Optic Speed Trap

For tests nos. MGTD-1D and MGTR-1D, a retroreflective optic speed trap was used to determine the speed of the bogie vehicle before impact. In both tests, three retroreflective targets, spaced at approximately 18-in. intervals, were applied to the side of the bogie vehicle. When the emitted beam of light was reflected by the targets and returned to the Emitter/Receiver, a signal was sent to the data acquisition computer, recording at 10,000 Hz, as well as the external LED box activating the LED flashes. The speed was then calculated using the spacing between the retroreflective targets and the time between the signals. LED lights and high-speed digital video analysis are used as a backup if vehicle speeds cannot be determined from the electronic data.

### 6.3.4 Digital Photography

Three AOS high-speed digital video cameras, six GoPro digital video cameras, and three Panasonic digital cameras were used to document each test. The AOS high-speed cameras had a frame rate of 500 frames per second, the GoPro video cameras had a frame rate of 240 frames per second, and the Panasonic digital video cameras had a frame rate of 120 frames per second. The cameras were placed laterally from the post, with a view perpendicular to the bogie’s direction of travel. A digital still camera was also used to document pre and post-test conditions for all tests.

## 6.4 Dynamic Testing End of Test Determination

When the impact head initially contacts the test article, the force exerted by the bogie vehicle is directly perpendicular. However, as the post rotates, the bogie’s orientation and path move away from perpendicular. This behavior introduces two sources of error: (1) the contact force between the impact head and the post has a vertical component and (2) the impact head slides upward along the test article. Therefore, only the initial portion of the accelerometer trace should be used; since, variations in the data can be significant as the system rotates, and the bogie overrides the system. Additionally, guidelines were established to define the end of test time using the high-speed video of the impact. The first occurrence of either of the following events was used to determine the end of the test: (1) the test article fractured or (2) the bogie overrode or lost contact with the test article.

## 6.5 Dynamic Testing Data Processing

The electronic accelerometer data obtained in dynamic testing was filtered using the SAE Class 60 Butterworth filter conforming to the SAE J211/1 specification [21]. The pertinent acceleration signal was extracted from the bulk of the data signals. The processed acceleration data was then multiplied by the mass of the bogie to get the impact force using Newton’s Second Law. Next, the acceleration trace was integrated to find the change in velocity versus time. Initial velocity of the bogie, calculated from the retroreflective optic speed trap data, was then used to determine the bogie’s velocity and the calculated velocity trace was integrated to find the bogie’s displacement. This displacement is also the displacement of the post. Combining the previous results, a force versus deflection curve was plotted for each test. Finally, integration of the force versus deflection curve provided the energy versus deflection curve for each test.

## 6.6 Static Testing Setup

Two static component tests were also planned on short sections of the bridge rail where the rail segment was supported by two scupper blocks and anchored to the timber decks. The first static test was planned for the nail-laminated, timber deck, and the second static test was planned for the glulam timber deck. The short rail section consisted of a 23-in. long, timber glulam rail segment, which was supported by two glulam scupper blocks. The rail segment was fabricated using the originally-requested Combination No. 48 SYP timber. Originally, the scupper block material for the dynamic component testing program was to be manufactured using Grade No. 1 SYP. However, due to supply chain issues, the scupper block material was replaced with a glulam substitute. The scupper blocks and rail segments were treated with 0.60 lb/ft$^3$ of pentachlorophenol in heavy oil. The scupper blocks and rail segments were connected and anchored to each deck type using four ¾-in. diameter x 10 UNC by 30-in. long ASTM A307 Grade A timber bolts. The top railing height was 21¾ in. above the timber deck and 19¾ in. above the surrogate wearing surface. The timber bolts were then fastened to the bottom of each deck type using ¾-in. diameter, malleable iron washers with nuts.

The two rail section were loaded using a ⅞-in. diameter, steel rod that was transversely inserted through the mid-height of the glulam rail segment. An eye nut was attached to the bolt on the back side of the rail, and a ½-in. thick, 6¾-in. by 23-in. steel plate was used to distribute the load to the front face of the rail segment. The test matrix for the two static tests is shown in Table 6. The static testing setup for the nail-laminated deck is shown in Figures 13 and 14 and the static testing for the glulam deck is shown in Figures 41 and 42. Additionally, material specifications, mill certifications, and certificates of conformity for the two bridge railing and deck systems are shown in Appendix C and Appendix D.

Table 6. Static Testing Matrix

| Test No. | Deck Type |
|---|---|
| MGTD-1S | Glulam |
| MGTR-1S | Nail-Laminated |

## 6.7 Static Testing Equipment and Instrumentation

For static test nos. MGTD-1S and MGTR-1S, the eye nut attached to the back face of the rail segment was connected to a 50,000-lb capacity hydraulic ram. The hydraulic ram was attached to a steel anchor frame, which was bolted down to the concrete tarmac. To measure the load and displacement of the rail and scupper block system, two load cells were attached to the hydraulic ram, and a string potentiometer was connected to the front head of the ⅞-in. diameter steel bolt that was used to load the two rail and scupper block systems.

It should be noted that the ⅞-in diameter, horizontal loading bolt fractured in static component test no. MGTR-1S at a tension load of approximately 11.7 kips. As a result, the static test was repeated as test no. MGTR-1SB. Before beginning this repeat test, the damaged malleable iron washers under the deck were replaced, and the vertical attachment bolts that anchored the rail and scupper blocks to the deck were re-tightened. The test setup for test nos. MGTD-1S and MGTR-1S are shown in Figures 60 and 61 respectively.


(a) Front View

(b) Back View

Figure 60. Setup of Static Component Test on Glulam Deck – (a) Front View and (b) Back View


(a) Front View


(b) Back View

Figure 61. Setup of Static Component Test on Nail-Laminated Deck – (a) Front View and (b) Back View

# 7 COMPONENT TESTING RESULTS AND DISCUSSION

## 7.1 Dynamic Bogie Testing Results

Two dynamic component tests were conducted with a bogie vehicle impacting the low-profile, glulam timber railing system on two transverse timber deck types. A description and details for each test, including sequential and post-test photographs, are contained in the following sections. The accelerometer data for each test was processed to obtain acceleration, velocity, and deflection curves, as well as force versus deflection and energy versus deflection curves. Although the individual transducers produced similar results, the values reported herein were calculated from the SLICE-2 data curves to provide a common basis for comparing results from multiple tests. Test results for all transducers are provided in Appendix E.

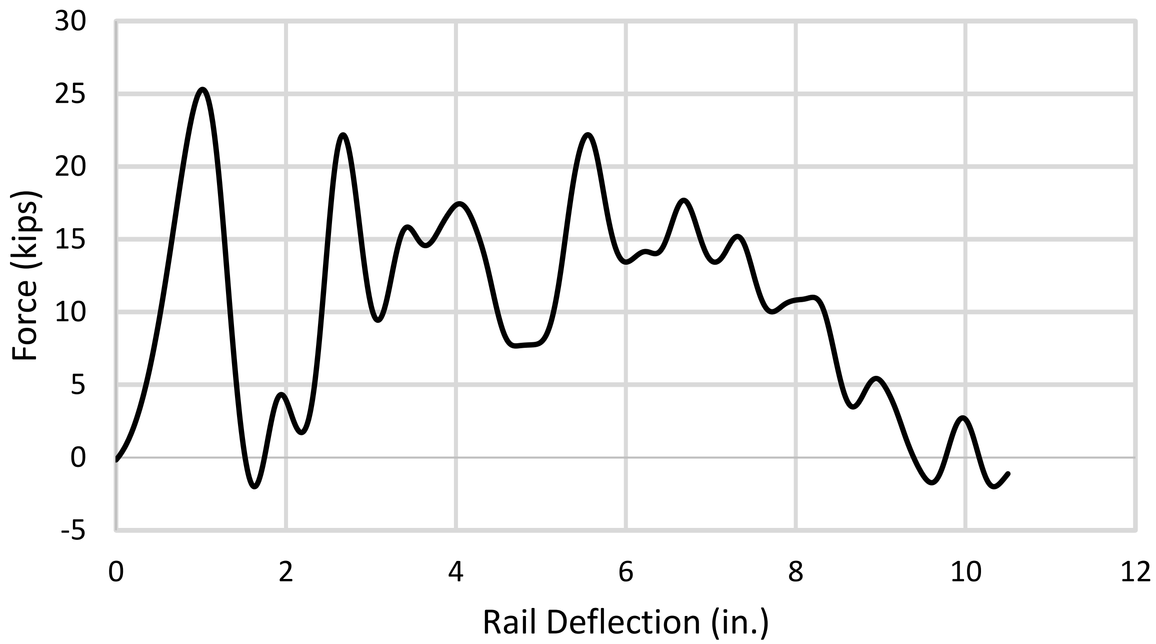
### 7.1.1 Test No. MGTD-1D (Transverse, Glulam Timber Deck)

#### 7.1.1.1 Force vs. Deflection and Energy vs. Deflection Responses

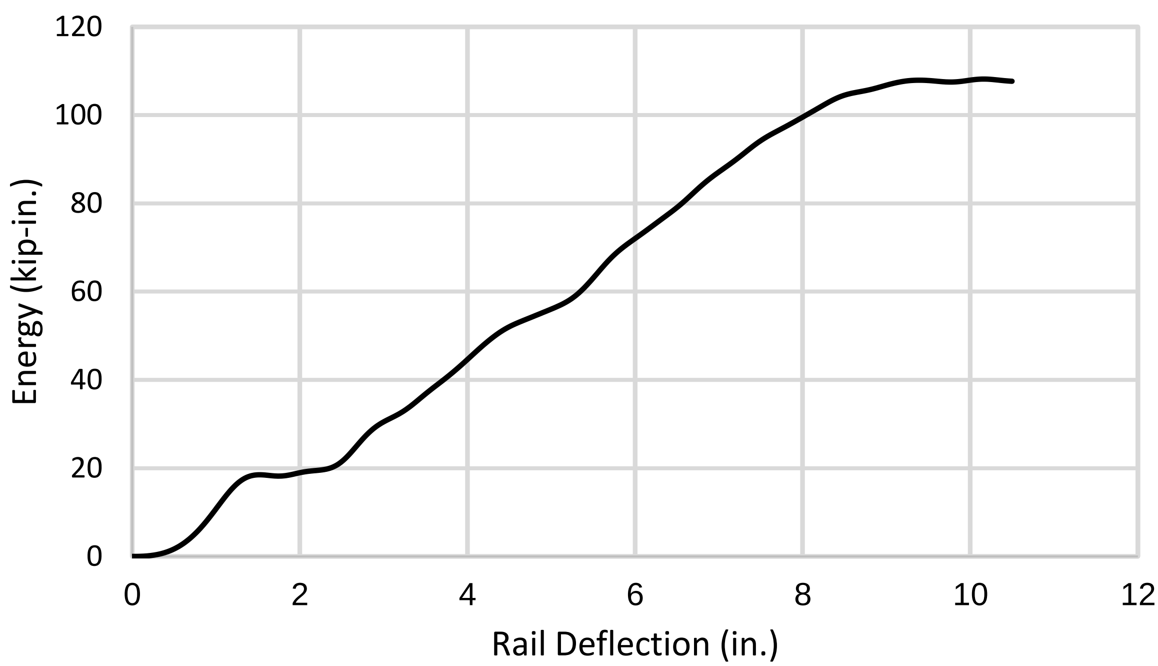
Test No. MGTD-1D was conducted with a bogie impacting the low-profile, glulam timber railing system on a transverse, glulam timber deck at the height of $16^3/_8$ in. (measured from the top of the wearing surface to the bogie head centerline) and an angle of 90 degrees and a speed of 13.1 mph. The impact caused rotational bending of the railing system and vertical deflection on the glulam timber deck. The lateral rail deflection reached approximately 10½ in. and occurred before tensile rupture of the glulam rail.

Findings from this dynamic bogie impact test were in the form of experimental data that define the general behavior of the glulam timber railing system on a transverse, glulam timber deck subjected to lateral impact force. The most important results were plots of lateral resistive force as a function of lateral rail deflection and energy dissipated (work performed) as a function of lateral rail deflection. Force vs. deflection and energy vs. deflection curves were created from the SLICE-2 accelerometer data and are shown in Figure 62. Work performed by the low-profile, glulam rail, scupper blocks, and deck was equal to the change in kinetic energy of the bogie vehicle. Energy absorption or work done on the timber bridge railing and deck system was determined by integrating the lateral rail force vs. lateral rail deflection curve. The energy dissipation during lateral impacts into the bridge railing and-deck system can be elastic but may also be inelastic. For inelastic events, the bogie's kinetic energy is transformed into other forms of energy, i.e., elastic strain-energy and non-recoverable plastic work. The transformation and dissipation of plastic work are related to permanent deformation and damage within the timber bridge railing system.

The shape of the force vs. deflection curve is the result of the impact velocity. At higher impact speeds, the peak force is expected to be larger. Inertial effects resulted in a peak force of 25.1 kips, which occurred at a deflection of 1.1 in., and an average force of 10.8 kips at 10 in. of rail deflection, as depicted in Figure 62. The average force was calculated by dividing energy by rail displacement at the impact height. At the rail's maximum lateral rail deflection of 10½ in., the rail and scupper block assembly on a transverse, glulam timber deck absorbed 107.7 kip-in. of energy. Two typical characteristics of the lateral resistive force vs. lateral rail deflection curve are: (1) the inertial peak force caused by the momentum transfer from the impact bogie vehicle to the bridge rail and deck system and (2) a fluctuating impact force followed by a rapid decline caused by the rail, scupper blocks, and deck system subjected to rotational bending resistance and propagation of a pre-existing crack in the bridge railing system. This crack continued to grow until approximately 7.5 in. of rail deflection. Time sequential and post-impact photographs are shown in Figure 63. Details regarding rail splitting is provided in Section 7.1.1.3.




(a)




(b)

Figure 62. Test No. MGTD-1D Results for Transverse, Glulam Timber Deck: (a) Lateral Force vs. Lateral Deflection and (b) Energy vs. Lateral Deflection Responses

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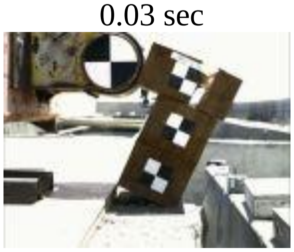

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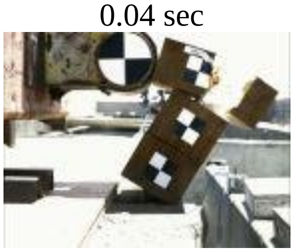

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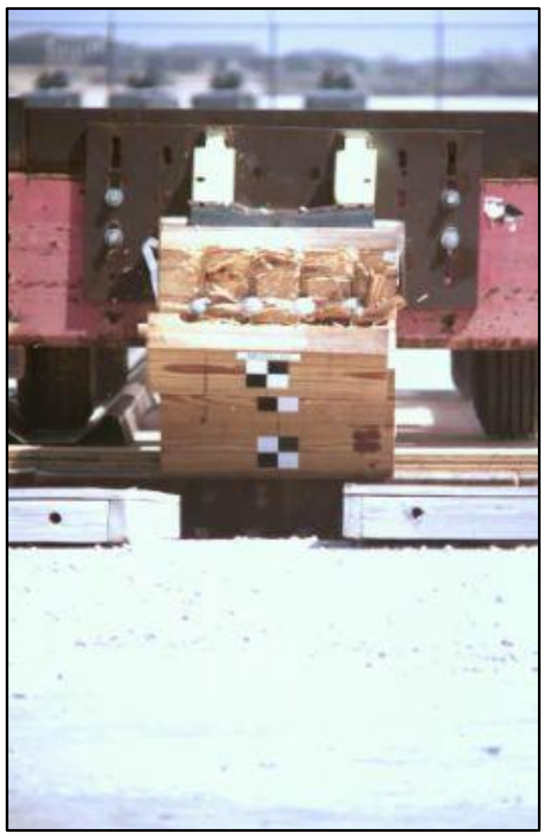


Figure 63. Time-Sequential and Post-Impact Photographs, Test No. MGTD-1D

#### 7.1.1.2 Dynamic Vertical Deck Deflection

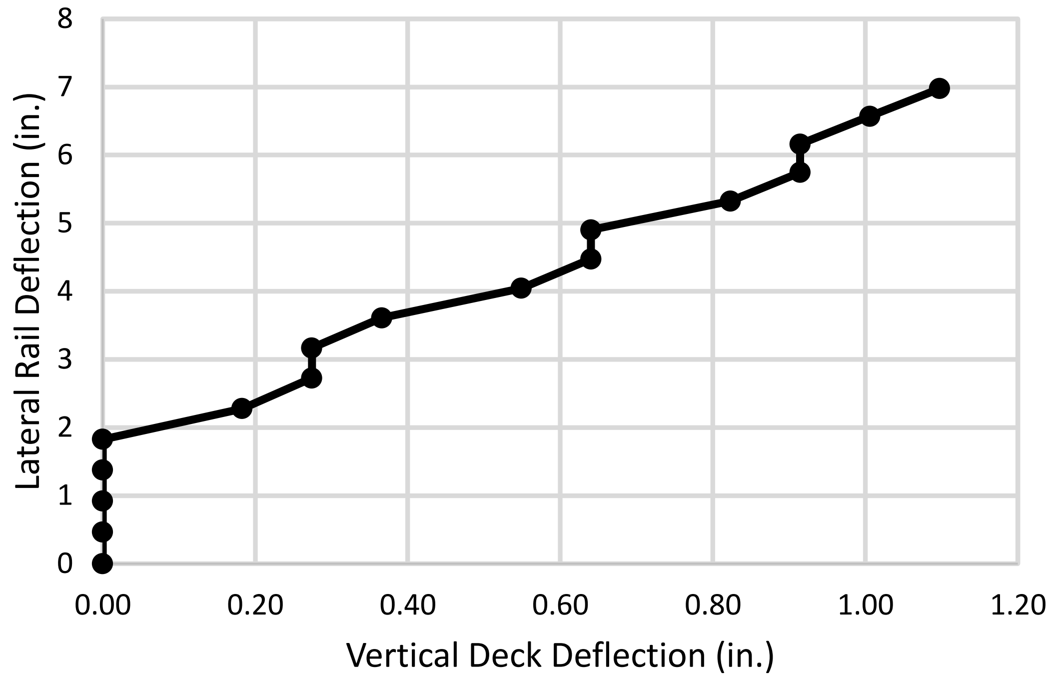
In order to obtain additional data for discerning the crashworthiness of the transverse, glulam timber deck with glulam bridge railing system for MASH TL-1 impact conditions, the vertical deck deflection at its outer edge was determined during the dynamic component testing program. Since determining the vertical deck deflection is not a standard analysis procedure performed for a typical dynamic component testing program, a brief discussion is provided herein regarding the procedure utilized to obtain the vertical deck deflection.

Before plotting the vertical deck deflection against lateral resistive force or energy, the vertical deflection versus time was obtained from the analysis of video footage taken during the dynamic bogie test. An AOS studio software was utilized to determine the vertical deck deflection versus time plots. Within this software, the time was recorded in frames per second (fps). Therefore, the recording frame rate of the impact test was utilized to compute the time duration within that video. The objects in the video were scaled against the actual sizes of the objects used during the physical component test to determine the vertical deck deflection at various time states. The ratio between the objects in the video and the objects in the actual test could be determined using an object whose actual dimensions were known. This knowledge allowed the vertical deck deflection that occurred within the video to be scaled to the actual deflection during the physical impact test.

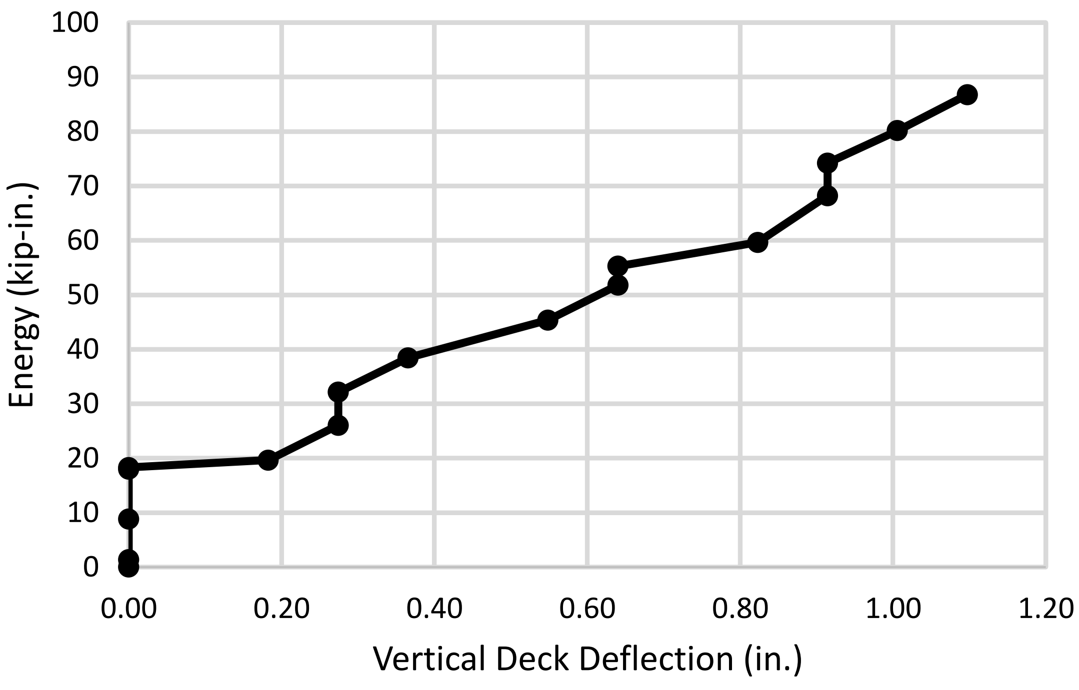
After obtaining the plot between vertical deck deflection and time, the next step was to cross-plot the vertical deck deflection against the lateral rail deflection and the energy absorbed by the glulam rail, scupper blocks, and deck. The vertical deck deflection versus lateral rail deflection and energy absorbed by the glulam railing, scupper blocks, and deck system versus vertical deck deflection plots are provided in Figure 64. It should be noted that the times obtained from the plots of the rail's lateral deflection and energy absorbed were matched up against the times associated with vertical deck deflections.

The glulam timber deck did not experience any vertical deflection for the first 2 in. of lateral rail and scupper block deflection. The vertical deck deflection increased as lateral rail deflection increased. The deck experienced a maximum dynamic vertical defection of 1.1 in. This peak vertical deflection corresponded to a lateral rail deflection of 7.0 in., as measured at the impact height. It appeared that the correlation between the lateral rail deflection vs. vertical deck deflection as well as energy absorbed by the rail, scupper blocks, and deck system vs. vertical deck deflection were approximately linear after the first 2 in. of lateral rail deflection, as shown in Figure 64.

It should be noted that the sample rate of the video data relative to the sample rate of the accelerometer data was much lower in magnitude. As a result, the number of lateral resistive force and energy-dissipated data points for cross-plotting was limited by the number of sampling points gathered from the video analysis. Because of this lower video sampling rate, the cross-plots between the vertical deck deflection and the lateral resistive force and energy-dissipated parameters should be deemed as approximate and largely used to show trends and make comparisons.




(a)




(b)

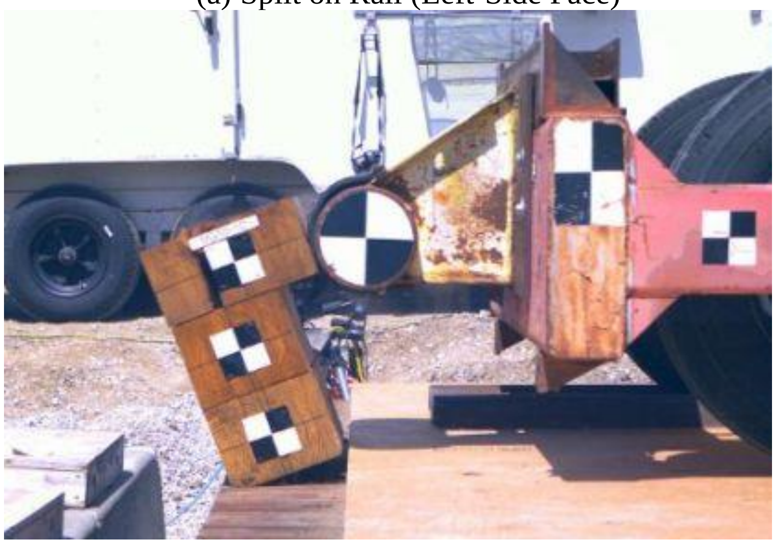
Figure 64. Results from Test No. MGTD-1D: (a) Lateral Rail Deflection vs. Vertical Deck Deflection, and (b) Energy Absorbed by the Rail, Scupper blocks, and Deck System vs. Vertical Deck Deflection

#### 7.1.1.3 Discussion on Rail Splitting

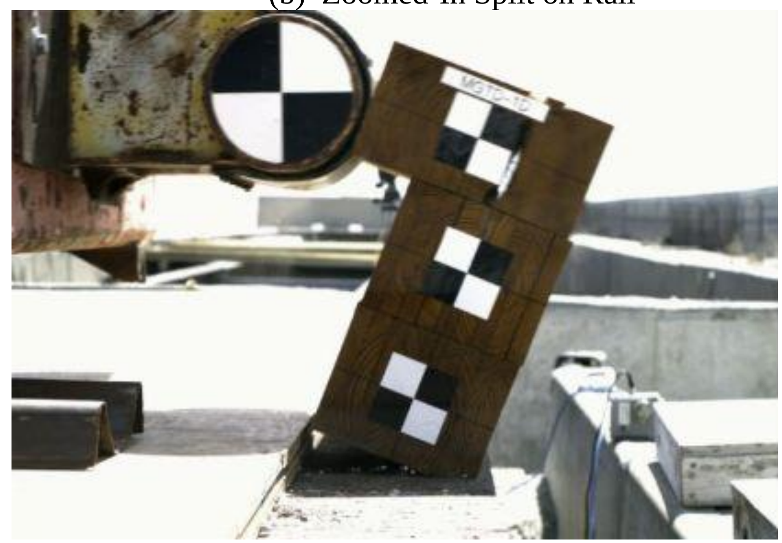
As previously-stated, after the first peak force occurred, two slightly smaller peaks were developed over approximately 4 in. of lateral rail deflection, as measured at the impact height and shown in Figure 62. There was a noticeable decrease in impact force resisted by the railing system following the third peak force. This impact force decline occurred at lateral rail deflection of approximately 7½ in. A high-speed video analysis revealed that the decline in lateral resistive force correlated to the beginning of the splitting of the glulam rail system at the location of a pre-existing crack. This pre-existing crack most likely occurred during the manufacturing process of the rail segment, or possibly when the rail segment was being installed at the test site. Images of this pre-existing crack on the rail and splitting that occurred during the impact event are provided in Figure 65. The splitting, which originated on the right-side end of the glulam rail segment, quickly propagated over the length of the rail segment, resulting in the eventual tensile fracture of the rail component. Once the rail segment split off from the scupper blocks, the timber bolts could not properly connect the scupper blocks and damaged the rail segment to the deck, resulting in a significant decline in the lateral resistive force of the rail and scupper blocks that were anchored to the deck.


(a) Split on Rail (Left-Side Face)


(b) Zoomed-In Split on Rail



(b) Left-End View of Rail Splitting



(d) Right-End View of Rail Splitting

Figure 65. Rail Segment Splitting from Pre-Existing Crack in Test No. MGTD-1D on Transverse, Glulam Timber Deck

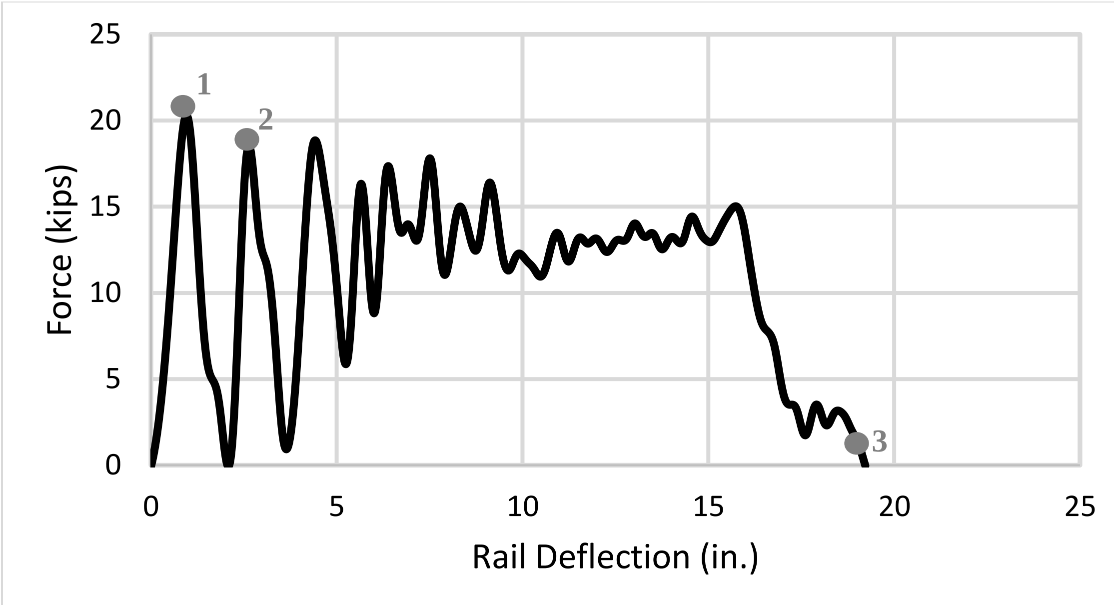
### 7.1.2 Test No. MGTR-1D (Transverse, Nail-Lam Timber Deck)

#### 7.1.2.1 Force vs. Deflection and Energy vs. Deflection Responses

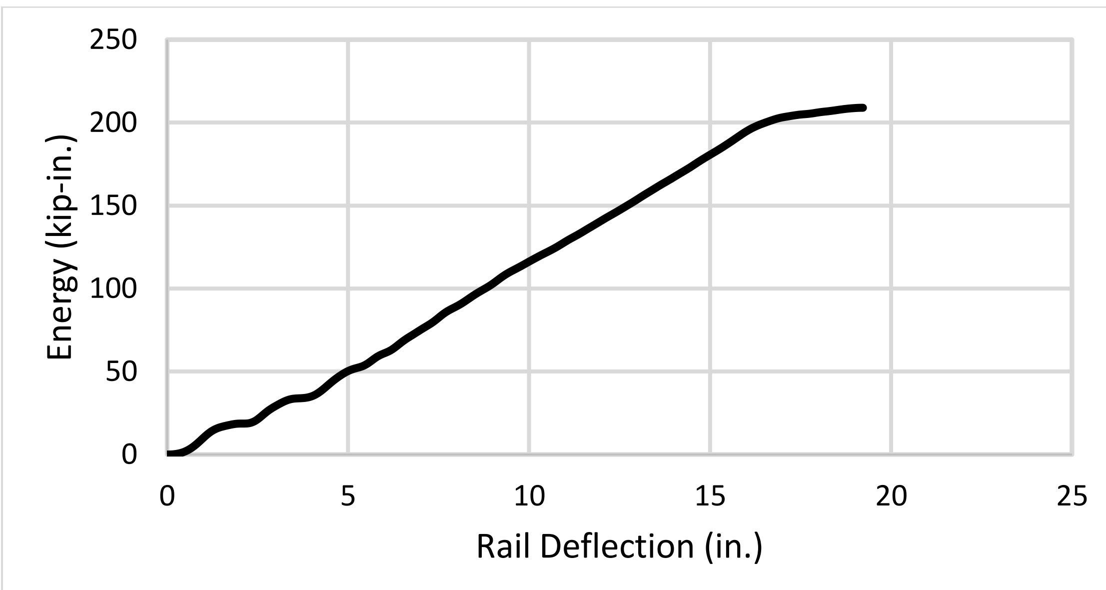
Test No. MGTR-1D was conducted with a bogie impacting the low-profile, glulam timber railing system on a transverse, nail-laminated timber deck at the height of 16⅜ in. (measured from the top of the wearing surface to bogie head centerline) and an angle of 90 degrees, with a target speed of 13 mph. The target impact speed was adjusted from 11 mph to 13 mph to produce more comparable results with those of test MGTD-1D, which had an actual impact speed of 13.1 mph. The actual impact speed for test no. MGTR-1D was 13.7 mph. The impact caused rotational bending of the railing system and vertical deflection on the nail-laminated timber deck. The lateral rail deflection reached approximately 19.2 in. before the tensile rupture of the glulam rail.

The lateral resistance force as a function of lateral rail deflection and energy dissipated as a function of lateral rail deflection were created from the impact test and are shown in Figure 66. Inertial effects resulted in a peak force of 20.1 kips, which occurred at a deflection of 1.0 in., and an average force of 11.6 kips at 10 in. of rail deflection, as depicted in Figure 66. The average force was calculated by dividing energy by rail displacement at the impact height. At the rail's maximum lateral rail deflection of 19.2 in., the rail and scupper block assembly on a transverse, nail-laminated timber deck absorbed 209.0 kip-in. of energy.

The response can be broken down into three phases. First, a rapid loading phase corresponding to the initial velocity increase of the glulam rail system. The first point (1) corresponds to the time when the glulam rail system's velocity reached its maximum value. The second phase, i.e., the plastic phase, corresponds to the plastic bending of the glulam rail and scupper blocks. In this phase, the signal showed fluctuations or oscillations. The fluctuations were significantly irregular and occurred over short time scales in test no. MGTR-1D as compared to test no. MGTD-1D. Therefore, these fluctuations could be related to the timber deck type used during dynamic bogie testing (i.e., nail-laminated deck vs. glulam deck. The second point (2) corresponded to the time when the rail and scupper block's velocity gradually decreased. The third point (3) corresponded to the time when the rail and scupper block's deflection was maximum and the rail and scupper block's velocity was at its minimum. In this phase, the rail and scupper block assembly failed (at approximately 19.2 in. of rail deflection) due to tensile rupture. Time sequential and post-impact photographs are shown in Figure 67.




(a)




(b)

Figure 66. Test No. MGTR-1D Results for Transverse, Nail-Lam Timber Deck: (a) Lateral Force vs. Lateral Deflection and (b) Energy vs. Lateral Deflection Responses

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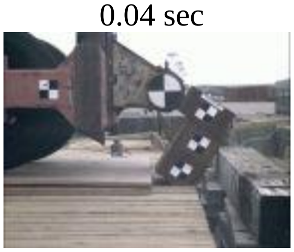

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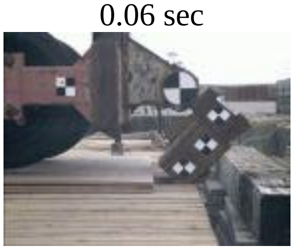

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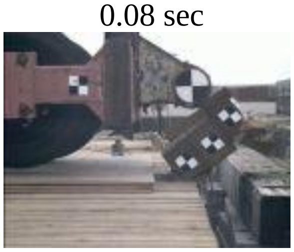


0.0966 sec

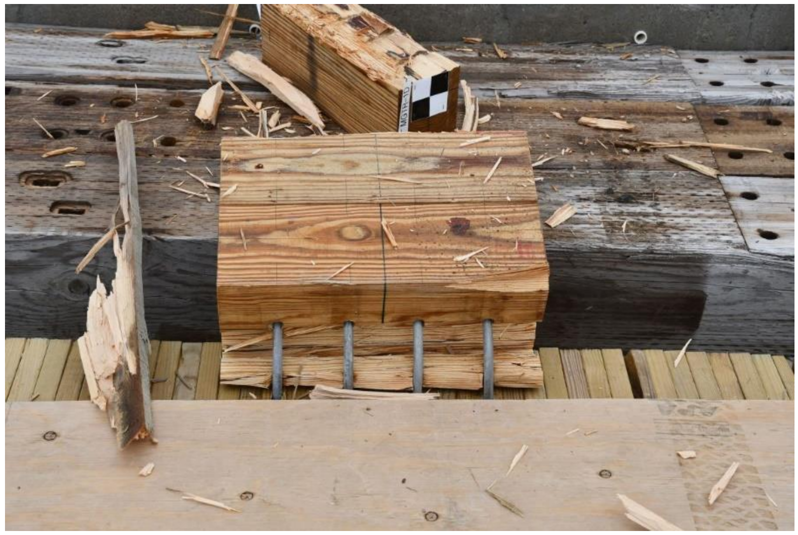
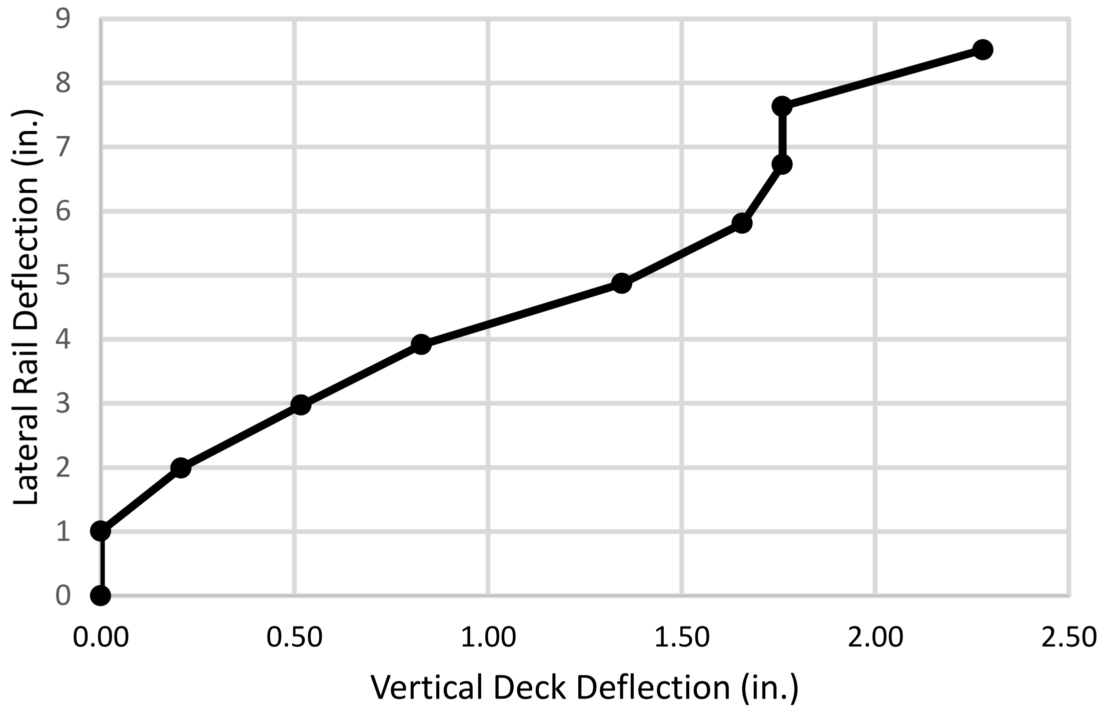


Figure 67. Time-Sequential and Post-Impact Photographs, Test No. MGTR-1D

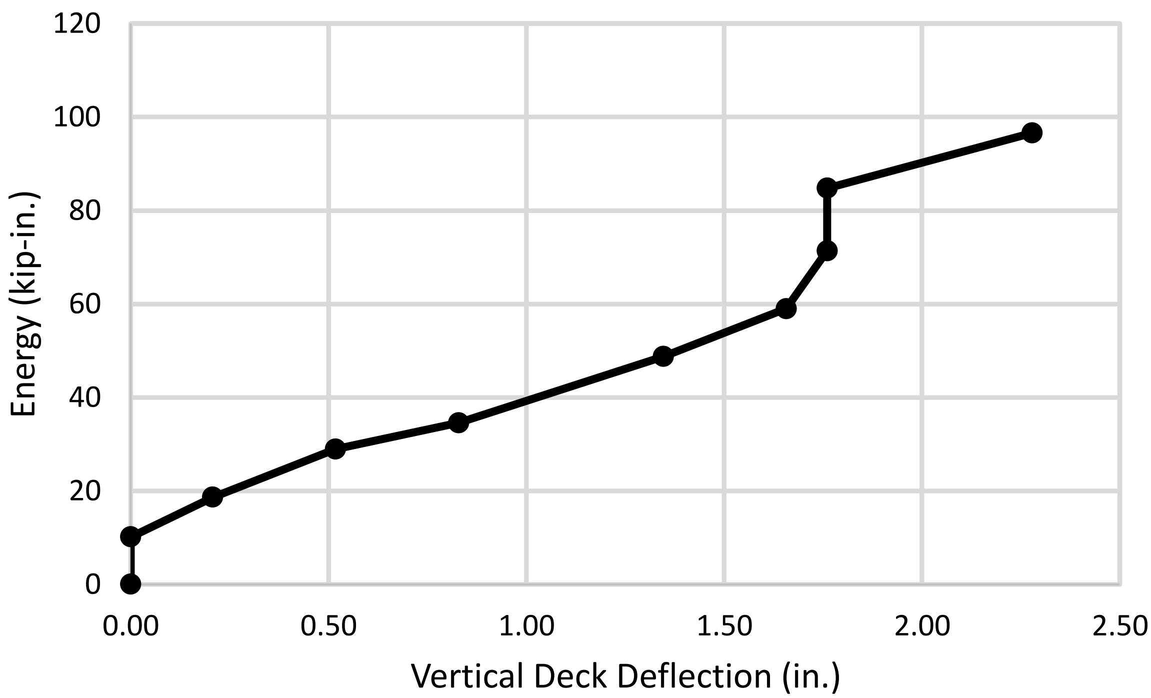
#### 7.1.2.2 Dynamic Vertical Deck Deflection

In order to obtain additional data for understanding the crashworthiness of the transverse, nail-laminated timber deck and glulam bridge rail system for MASH TL-1 impact conditions, the vertical deck deflection at its outer edge was determined during the dynamic component testing program using similar procedures to those outlined in Section 7.1.1.2. The vertical deck deflection vs. lateral rail deflection and energy absorbed by the rail, scupper blocks, and deck system vs. vertical deck deflection were plotted and are provided in Figure 68.

The nail-laminated timber deck did not experience any vertical deck deflection for the first 1 in. of lateral rail and scupper block deflection. The vertical deck deflection increased as lateral rail deflection increased. The deck experienced a maximum dynamic vertical defection of 2.3 in. This peak vertical deflection corresponded to a lateral rail deflection of 8.5 in., as measured at the impact height. It appeared that the correlation between the lateral rail deflection vs. vertical deck deflection as well as energy absorbed by the rail, scupper blocks, and deck system vs. vertical deck deflection were approximately linear after the first 1 in. of lateral rail deflection, as shown in Figure 68.




(a)




(b)

Figure 68. Results from Test No. MGTR-1D: (a) Lateral Rail Deflection vs. Vertical Deck Deflection, and (b) Energy Absorbed by the Rail, Scupper Blocks, and Deck System vs. Vertical Deck Deflection

## 7.2 Static Testing Results

Static testing was conducted on two bridge rail and scupper block systems in order to determine the lateral rail force vs. lateral rail deflection response when connected to two transverse, timber deck types. For test no. MGTD-1S, a glue-laminated timber deck was utilized, while for test no. MGTR-1S, a nail-laminated timber deck was used. A description and details for each test, including analysis of the lateral force vs. lateral deflection, lateral force vs. vertical deck deflection, and lateral rail deflection vs. vertical deck deflection curves as well as system damages, are contained in the following sections. A tension load cell and a string potentiometer were used to measure the lateral load and lateral displacement, respectively. Load cell and sting potentiometer data for each static test are provided in Appendix F.

### 7.2.1 Test No. MGTD-1S (Transverse, Glulam Timber Deck)

#### 7.2.1.1 Force vs. Deflection Response

Test no. MGTD-1S began with the use of a 50,000-lb capacity hydraulic ram to apply lateral load to test components (i.e., the rail, scupper blocks, and deck system) to failure. As the lateral load was applied to the 23-in. long glulam rail segment, the rail and scupper blocks rotated backward, thus opening a gap between the front of the lower scupper block and the bridge deck. At the same time, the bridge deck system deflected downward. A maximum lateral force of 23.1 kips was observed at 17.6 in. of lateral rail deflection. The lateral rail force versus lateral rail deflection curve for test no. MGTD-1S is shown in Figure 69.

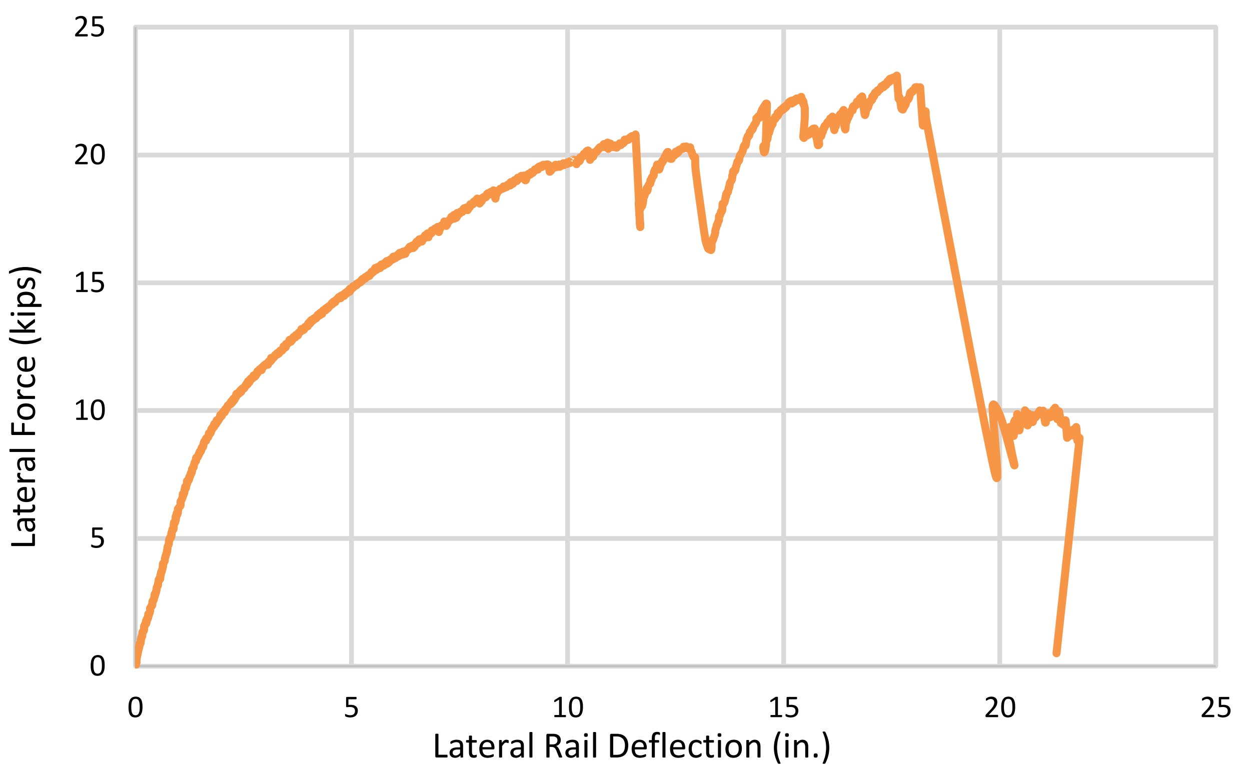
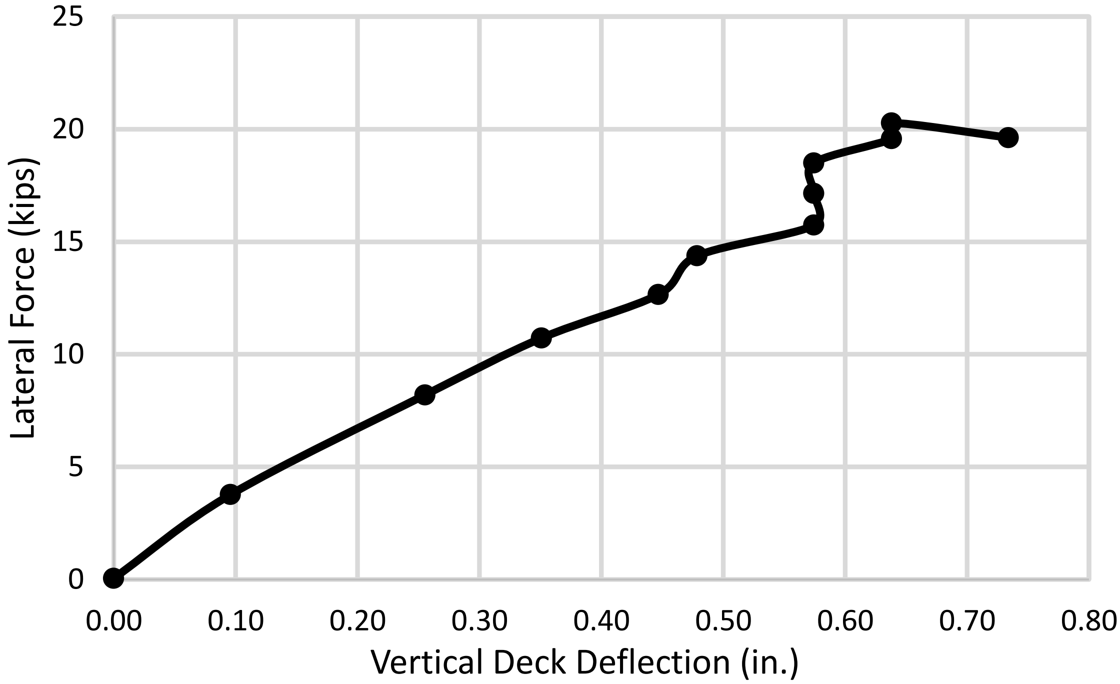



Figure 69. Lateral Rail Force vs. Lateral Rail Deflection Curve, Test No. MGTD-1S

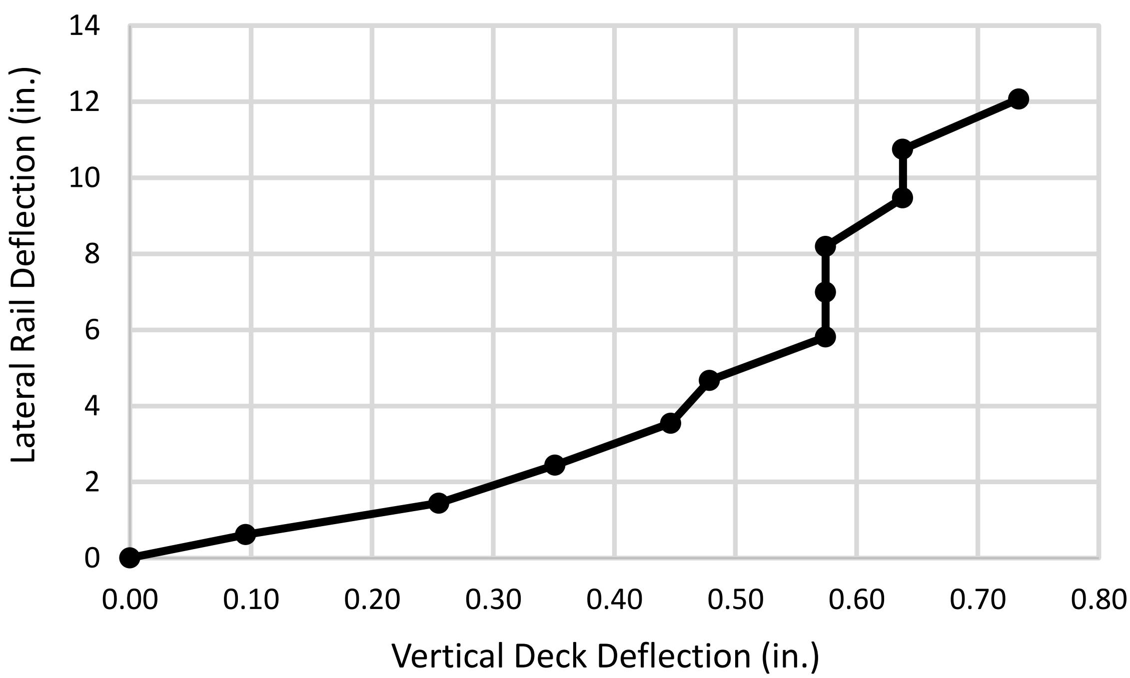
#### 7.2.1.2 Vertical Deck Deflection

In order to obtain additional data for understanding the behavior of the glue-laminated deck and timber railing system during test no. MGTD-1S, the vertical deck deflection was determined using similar procedures outlined in Section 7.1.1.2. The lateral rail force versus vertical deck deflection and lateral rail deflection versus vertical deck deflection plots are provided in Figure 70.

The vertical deck deflection increased as lateral rail deflection and lateral rail force increased, as depicted in Figure 70. The glue-laminated timber deck experienced a maximum vertical deck deflection of approximately 0.7 in. This peak vertical deck deflection corresponded to a lateral rail deflection of 12.1 in. measured at the impact height. The initial linear stiffness of the rail, scupper blocks, and deck system was determined to be 6.2 kip/in. based on a load of 6.2 kips and a lateral deflection of 1.0 in. The lateral rail deflection vs. vertical deck deflection behavior was nonlinear. The lateral rail force vs. vertical deck deflection behavior was approximately linear through the observed test deflection, as shown in Figure 70.




(a)




(b)

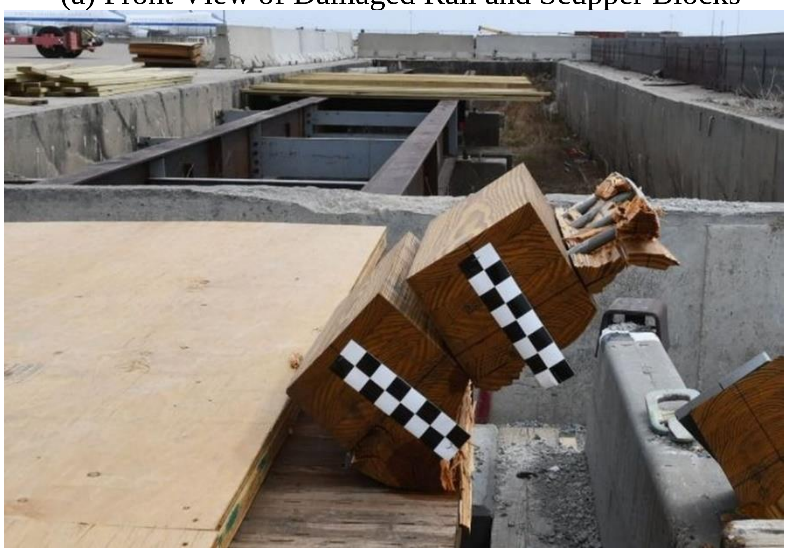
Figure 70. Results from Static Component Test, Test No. MGTD-1S: (a) Lateral Rail Force vs. Vertical Deck Deflection and (b) Lateral Rail Deflection vs. Vertical Deck Deflection

#### 7.2.1.3 System Damage

Significant damage to the rail and scupper block assembly was sustained during the loading event. As the rail and scupper block system rotated backward during loading, the back face of the bottom scupper block began to crush against the glulam timber deck. At the top of the rail and scupper block assembly, the bolt heads began to bear into the rail segment, ultimately leading to the rail splitting along the length of the rail segment. The malleable iron washers that fastened the rail and scupper block assembly to the deck sustained no damage. Some of the washers did begin to bear into the deck’s underside. Images of the scupper block crush, rail segment splitting, and the condition of the washers after static test no. MGTD-1S can be seen in Figure 71.

(a) Front View of Damaged Rail and Scupper Blocks


(b) Left-End View of Damaged Rail and Scupper Blocks



(c) Right-End View of Damaged Rail and Scupper Blocks

(d) Damage to Malleable Iron Washers

Figure 71. Damage to Rail and Scupper Blocks on Glulam Timber Deck, Static Test No. MGTD-1S

### 7.2.1 Test No. MGTR-1S (Transverse, Nail-Lam Timber Deck)

#### 7.2.1.1 Force vs. Deflection Response

Test no. MGTR-1S began with the use of a 50,000-lb capacity hydraulic ram to apply lateral load to test components (i.e., the rail, scupper blocks, and deck system) to failure. During test no. MGTR-1S, the lateral load was applied to the 23-in. long glulam rail segment, which caused the rail and scupper blocks to rotate backward, thus opening a gap between the front of the lower scupper block and the bridge deck. At the same time, the bridge deck system deflected downward. Before test no. MGTR-1S could be completed, the ⅞-in. diameter steel bolt that was used to load the assembly ruptured. Bolt rupture occurred when the rail segment had laterally deflected 11.7 in. due to an applied load of 17.1 kips. The lateral rail force versus lateral rail deflection curve for test no. MGTR-1S is shown in Figure 72.

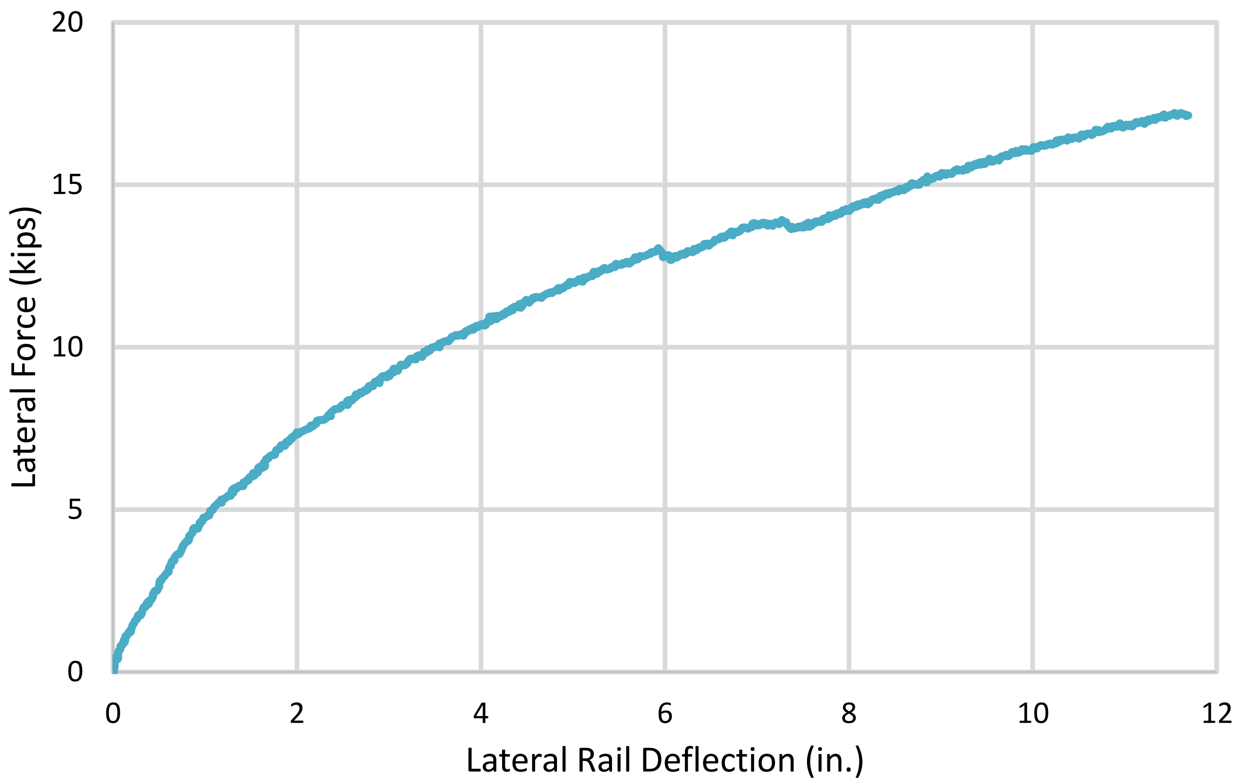



Figure 72. Lateral Rail Force vs. Lateral Rail Deflection Curve, Test No. MGTR-1S

As previously stated during test no. MGTR-1S, the horizontal rail bolt that was used to load the rail and scupper blocks failed before the assembly was allowed to fully deflect. Thus, test no. MGTR-1S was repeated as test no. MGTR-1SB. The procedure for test no. MGTR-1SB remained the same as was used for test nos. MGTR-1S and MGTD-1S. In this repeat test (test no. MGTR-1SB), the rail, scupper blocks, and four steel timber bolts were not replaced from those used in test no. MGTR-1S. The only parts that were replaced included the ⅞-in. diameter horizontal steel loading bolt and one malleable iron washer that was used to fasten the rail and scupper block assembly to the deck. During test no. MGTR-1S, the loading caused the timber bolts to plastically elongate and pull the washers and bolt heads into the wood, which resulted in a loose connection to the deck. As a result, prior to test no. MGTR-1SB, the rail and scupper blocks were fastened down tight against the deck using the vertical bolts, nuts, and washers. The lateral rail load vs. lateral rail deflection curve obtained from test no. MGTR-1SB was cross-plotted with lateral rail load vs. lateral rail deflection from test no. MGTR-1S, as shown in Figure 73. The maximum rail force observed during test no. MGTR-1SB was 20.5 kips, which corresponded to a lateral rail deflection of 14.7 in.

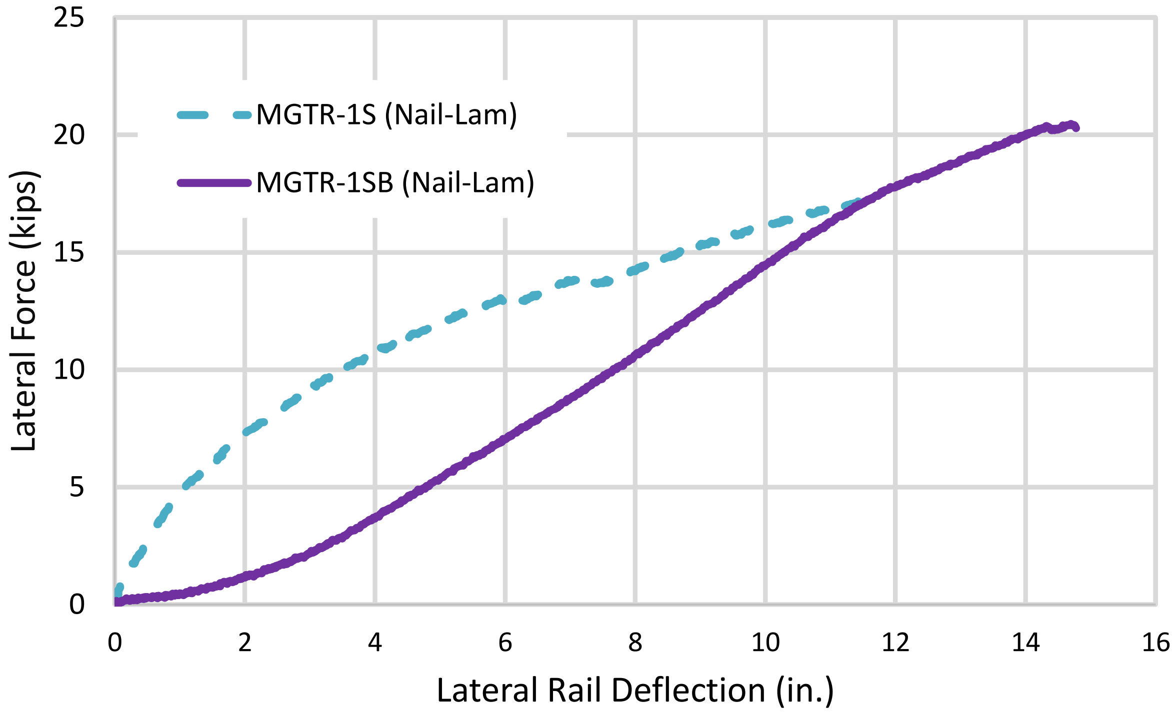


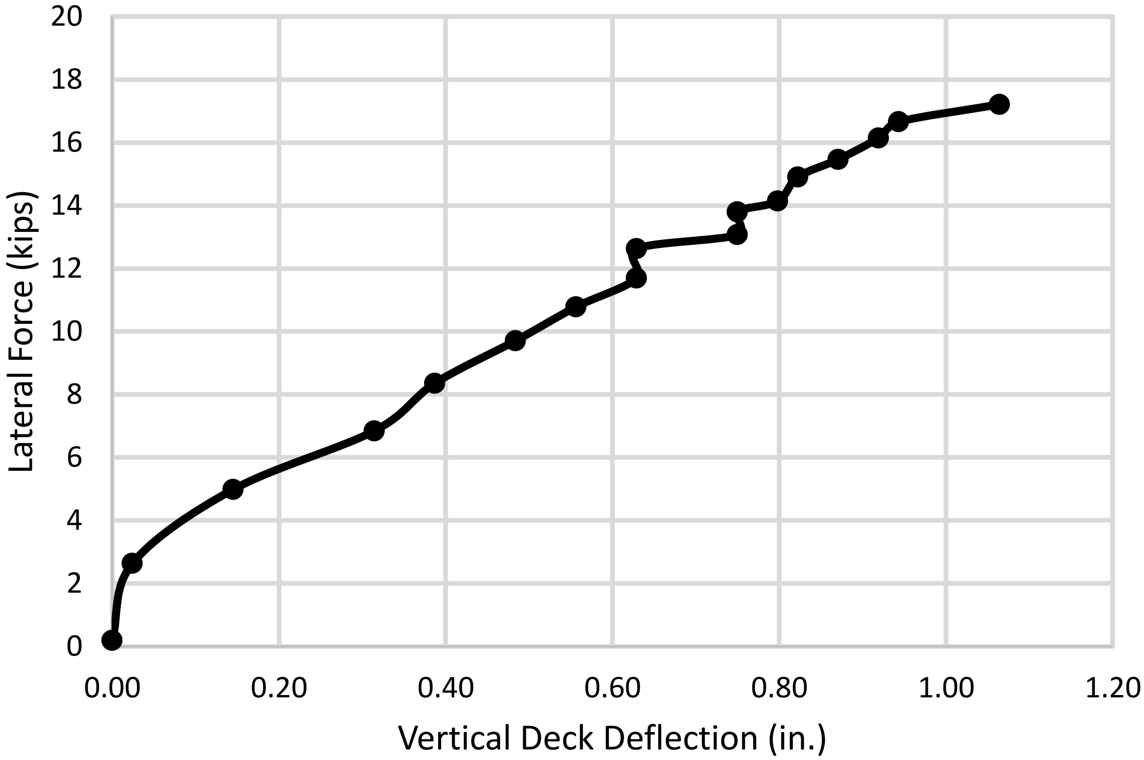

Figure 73. Lateral Rail Force vs. Lateral Rail Deflection Curves, Test Nos. MGTR-1SB and MGTR-1S

Following these tests, it was believed that the test results through the first 11.7 in. of lateral rail displacement from test no. MGTR-1S was used for the analysis of system response. For larger lateral rail displacements, the lateral rail force vs. lateral rail deflection behavior from test no. MGTR-1SB was utilized for the investigation and analysis. However, it should be noted that the results obtained from test no. MGTR-1S provided sufficient data for use in making comparisons between the nail-Lam deck (test no. MGTR-1S) and glulam deck (test no. MGTD-1S).

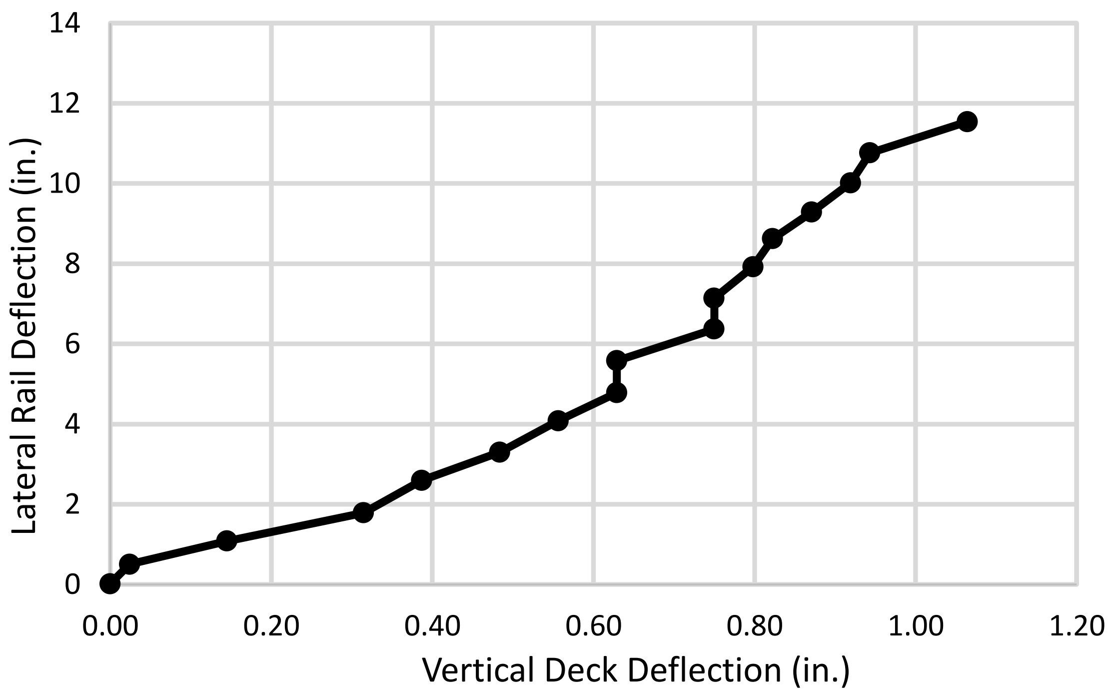
#### 7.2.1.2 Vertical Deck Deflection

In order to obtain additional data for understanding the behavior of the nail-laminated deck and timber railing system during test no. MGTR-1S, the vertical deck deflection was determined using similar procedures outlined in Section 7.1.1.2. The lateral rail force versus vertical deck deflection and lateral rail deflection versus vertical deck deflection plots are provided in Figure 74. Unlike test no. MGTR-1S, the lateral rail force versus vertical deck deflection and lateral rail deflection versus vertical deck deflection were not plotted for test no. MGTR-1SB. These plots were not included as all data pertinent to determining the adequacy of the rail and scupper blocks on a glulam deck was available from the evaluation of the nail-laminated deck in test no. MGTR-1S.

The vertical deck deflection increased as lateral rail deflection and lateral rail force increased, as depicted in Figure 74. The nail-laminated deck experienced a maximum vertical defection of 1.1 in, which corresponded to a 17.1-kip load that caused the horizontal load bolt rupture. This peak vertical deck deflection corresponded to a lateral rail deflection of 11.5 in. measured at the impact height. The lateral rail force versus vertical deck deflection and the lateral rail deflection vs. vertical deck deflection were approximately linear, as shown in Figure 74.




(a)



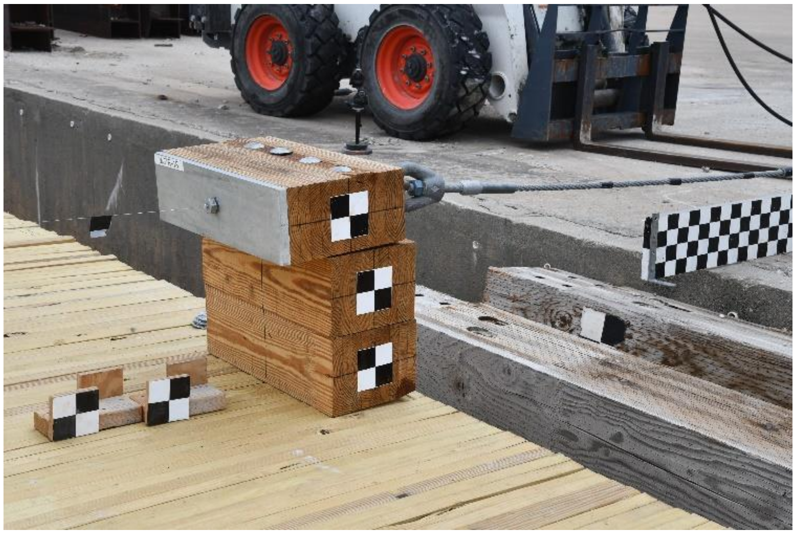

(b)

Figure 74. Results from Static Component Test, Test No. MGTR-1S: (a) Lateral Rail Force vs. Vertical Deck Deflection and (b) Lateral Rail Deflection vs. Vertical Deck Deflection

#### 7.2.1.3 System Damage

Minimal damage to the rail and scupper block assembly was sustained during the loading event in test no. MGTR-1S, as shown in Figure 75. As the rail and scupper block system rotated backward during loading, the back face of the bottom scupper block began to crush against the nail-laminated deck. After the ⅞-in. diameter horizontal load bolt ruptured, slight damage from the bolt heads bearing into the top of the rail segment was observed. Damage to one out of the four malleable iron washers occurred. Wood bearing damage occurred around all four malleable washers as they were pressed into the underside of the deck during the static loading.

In test no. MGTR-1SB, damage to the rail and scupper block assembly and timber deck was sustained as a result of the loading event, as shown in Figure 76. As the rail and scupper block system rotated backward during loading, the bottom scupper block's back face pressed against the deck's top surface. Similar to test nos. MGTD-1S and MGTR-1S, the bolt heads began to bear into the rail segment, resulting in splitting along the length of the rail segment. The nail-laminated deck and malleable iron washers also sustained significant damage. As the rail and scupper blocks rotated backward, the 2x6 boards comprising the nail-laminated deck began to crack and displace. Three of the four malleable iron washers fractured due to the loading event. After three of the four malleable iron washers fractured, the vertical steel timber bolts pulled up farther into the underside of the nail-laminated timber deck. This action resulted in the 2x6 boards directly surrounding the connection region to fracture away as the timber bolts pulled farther into the underside of the deck.



(a) Damage Sustained to Rail and Scupper Blocks

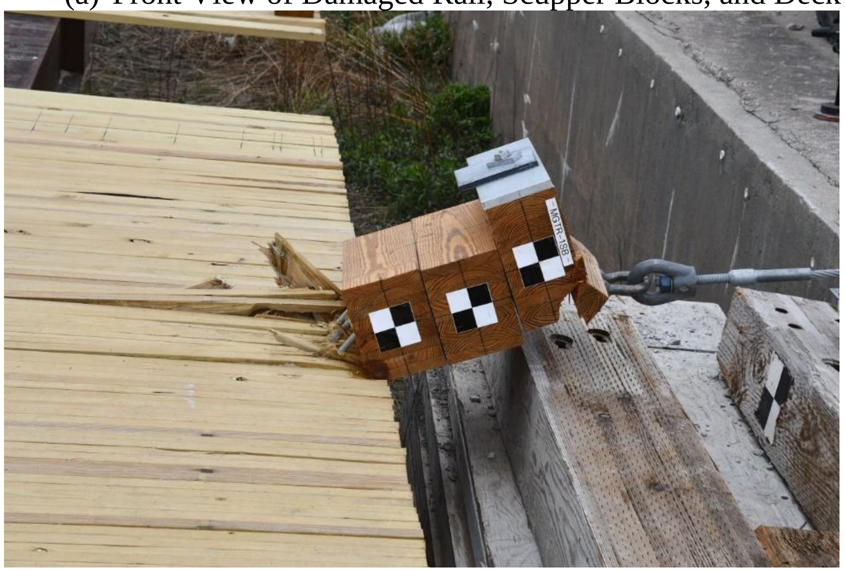
(b) Damage to Malleable Iron Washer and Deck

(c) Ruptured Bolt Used to Load Rail and Scupper Blocks

Figure 75. Component Damage, Test No. MGTR-1S

(a) Front View of Damaged Rail, Scupper Blocks, and Deck

(b) Left-End View of Damaged Rail, Scupper Blocks and Deck



(c) Right-End View of Damaged Rail, Scupper Blocks, and Deck

(d) Damage to Malleable Iron Washers and Deck

Figure 76. Damage to Rail and Scupper Blocks on Nail-Lam Deck, Static Test No. MGTR-1SB

## 7.3 Discussion and Comparison of Test Results

### 7.3.1 Dynamic Component Testing

The results from the bogie testing program are summarized in Table 7 and Table 8. The impact speeds for both dynamic bogie tests were relatively consistent as the speed only varied from 13.1 to 13.7 mph. The impact height of the bogie was 16⅜ in. for both tests. The lateral rail force vs. lateral rail deflection and energy absorbed vs. lateral rail deflection behaviors observed were similar through the first 7.5 in. of lateral rail deflection. After this deflection, the lateral resistive force for test no. MGTR-1D remained relatively constant, while the lateral resistive force for test no. MGTD-1D decreased significantly. Comparisons of the lateral rail force vs. lateral rail deflection and energy absorbed vs. lateral rail deflection for the two tests are provided in Figure 77.

For both bogie tests, inertial effects were observed at the beginning of the impact events. As illustrated in Figure 77, the data recorded from each test showed a large force spike approximately over the first 1 in. of lateral rail deflection. When comparing the lateral rail force versus lateral rail deflection between the two deck types over the first 7.5 in. of lateral rail deflection, the glulam rail and scupper blocks attached to a glulam deck resisted more force and dissipated more energy than observed with the nail-laminated deck. After 7.5 in. of lateral rail deflection, the glulam rail and scupper blocks on glulam deck quickly began to lose its ability to resist lateral force from the bogie impact. However, the glulam rail and scupper blocks attached to the nail-laminated deck sustained approximately the same force from 7.5 in. of lateral deflection through 16 in. As discussed in Section 7.1.1, this early failure of the rail and scupper block assembly on a glulam deck was likely due to a pre-existing split in the rail segment. Thus, a comparison of dynamic results between the two deck types was limited to the first 7.5 in. of lateral rail deflection. For a more in-depth discussion of the pre-existing split on the rail segment, see Section 7.1.1.

Figure 77 also compares the energy absorbed by both systems versus the lateral rail deflection. Similar to the lateral rail force versus lateral rail deflection results, the rail and scupper blocks attached to a glulam deck showed a higher capacity and energy dissipation during the first 7.5 in. of lateral rail deflection than observed with the nail-laminated deck. Again, the comparison between the two energy curves was limited to approximately the first 7.5 in. of lateral rail deflection.

The average lateral resistive forces of the rail and scupper blocks at 5 in. and 7.5 in. of lateral rail deflection were relatively higher for the glulam deck as compared to the nail-laminated deck, as shown in Table 7. Accordingly, the total energy absorbed through 7.5 in. of lateral deflection was also greater for the glulam deck as compared to the nail-laminated deck. As shown in Table 7, the initial linear stiffness for the rail and scupper blocks on the glulam deck was higher than observed for the system installed on the nail-laminated deck. A comparison of initial linear stiffness also indicated that the rail and scupper blocks attached to a glulam deck had relatively better performance as compared to the system attached to a nail-laminated deck. The peak lateral force was also higher for the glulam deck as compared to the nail-laminated deck. This difference in initial peak force was likely due to minor differences in the test setup; therefore, it is not the primary indicator of overall system performance between the two deck types.

The two deck types were also compared to one other using lateral rail displacement versus vertical deck deflection and energy absorbed versus vertical deck deflection. These plots are shown in Figure 78. Due to the limited number of data points, the graphical results should be deemed approximate and limited to providing general trends for comparing deck types. For lateral rail deflection versus vertical deck deflection, it can be observed that for the same lateral rail deflection, the vertical deflection was lower in the glulam deck as compared to the nail-laminated deck. A summary of lateral rail deflection versus vertical deck deflection results is provided in Table 8.

A comparison of the energy absorbed by the rail, scupper blocks, and deck versus vertical deck deflection revealed greater energy dissipation for the glulam deck as compared to the nail-lam deck for a given vertical deck deflection, as depicted in Figure 78.

Table 7. Dynamic Bogie Testing Results: Peak Force, Average Force, Energy Absorbed, and Initial Linear Stiffness

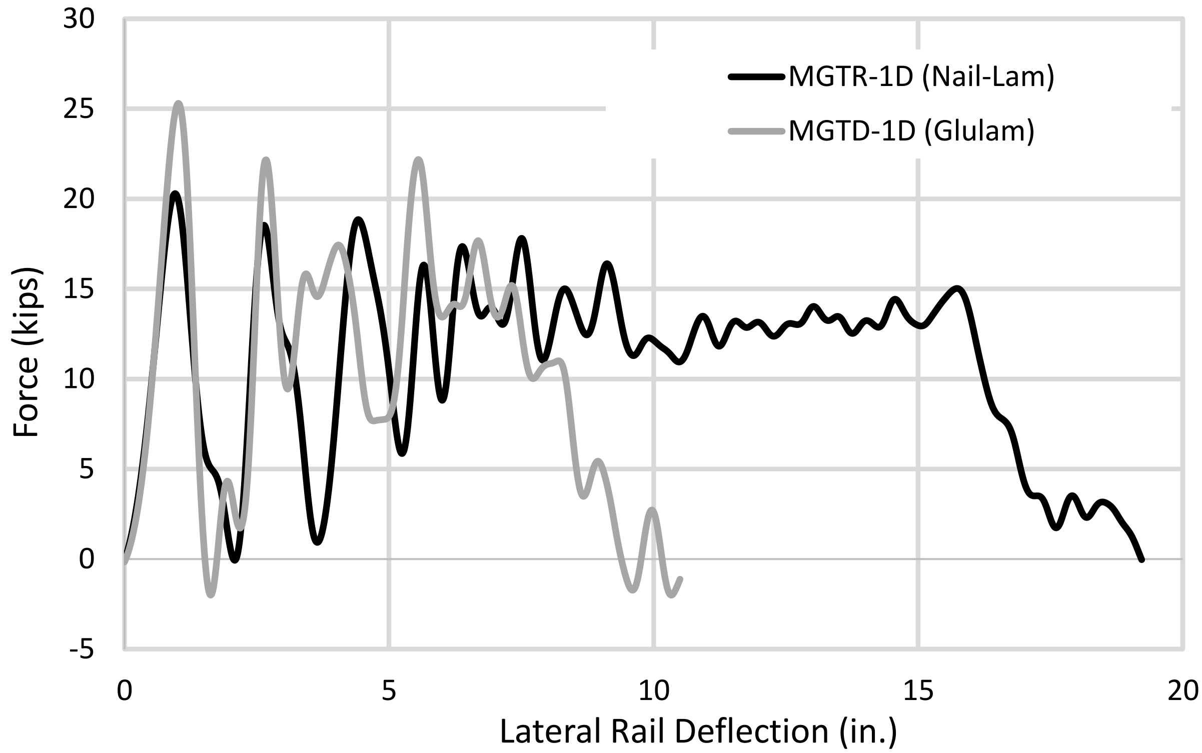
| Test No. | Deck Type | Impact Angle | Impact Speed (mph) | Peak Force (kips) | Rail/Scupper Average Lateral Force (kips) (@ Bogie Displacement)[1] | | Rail/Scupper Energy Absorbed (k-in.) (@ Bogie Displacement)[1] | | Initial Linear Stiffness kip/in. |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | @ 5 in. | @ 7.5 in. | @ 5 in. | @ 7.5 in. | |
| MGTD-1D | Glulam | 90° (Lateral) | 13.1 | 25.1 | 11.2 | 12.6 | 55.9 | 94.2 | 25.0 |
| MGTR-1D | Nail-Lam | 90° (Lateral) | 13.7 | 20.1 | 10.0 | 11.0 | 50.1 | 82.2 | 21.1 |

[1]7.5 in. of lateral rail deflection is location when glulam rail fully cracks

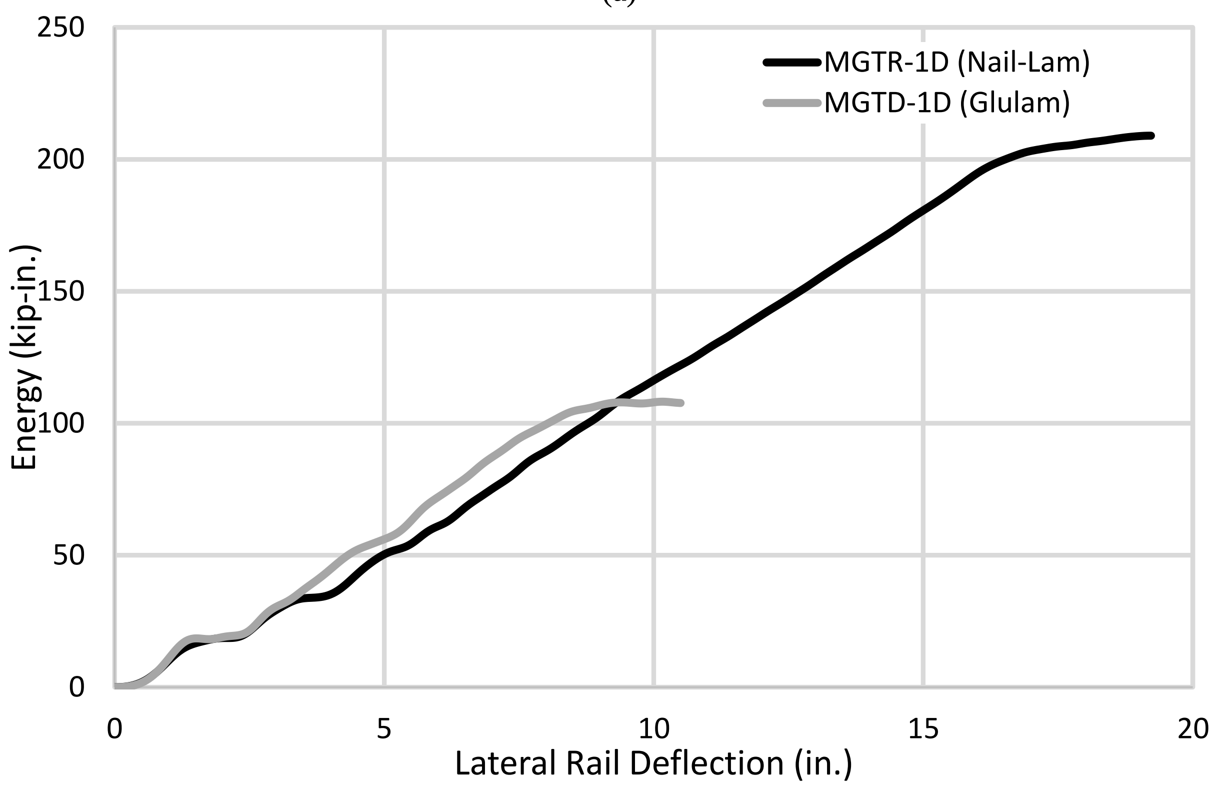
Table 8. Dynamic Bogie Testing Results: Lateral Rail Defection and Vertical Deck Deflection

| Test No. | Deck Type | Impact Angle | Impact Speed (mph) | Peak Vertical Deflection (in.) | Corresponding Lateral Rail Displacement[1] (in.) |
|---|---|---|---|---|---|
| MGTD-1D | Glulam | 90° (Lateral) | 13.1 | 1.1 | 6.98 |
| MGTR-1D | Nail-Lam | 90° (Lateral) | 13.7 | 2.3 | 8.5 |

[1]Lateral rail deflection measured at vertical impact height

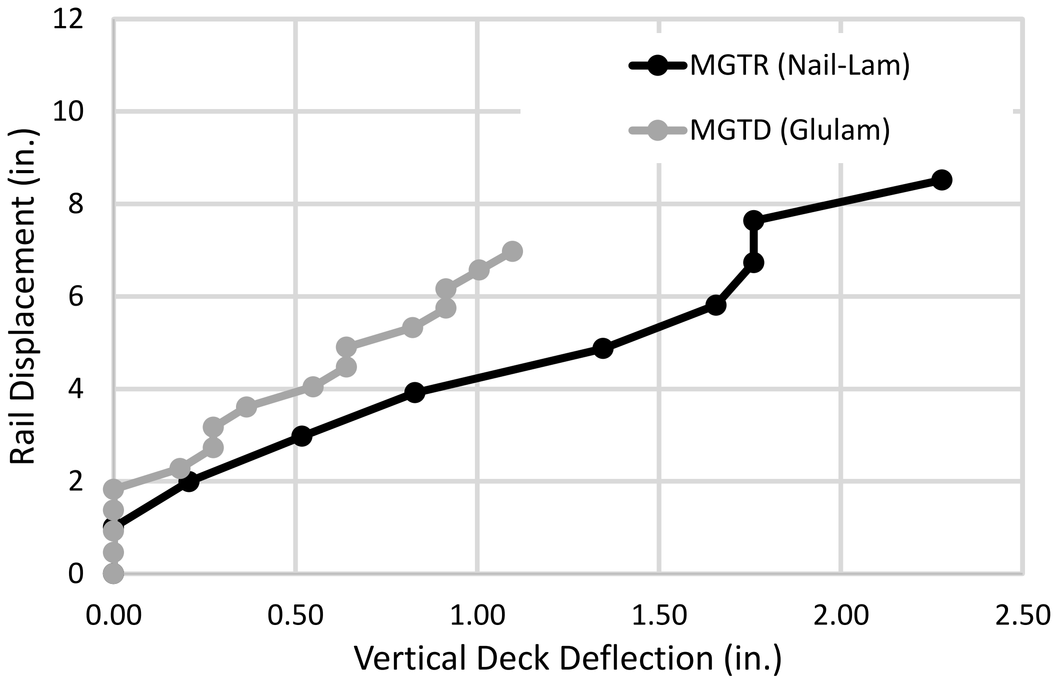



(a)

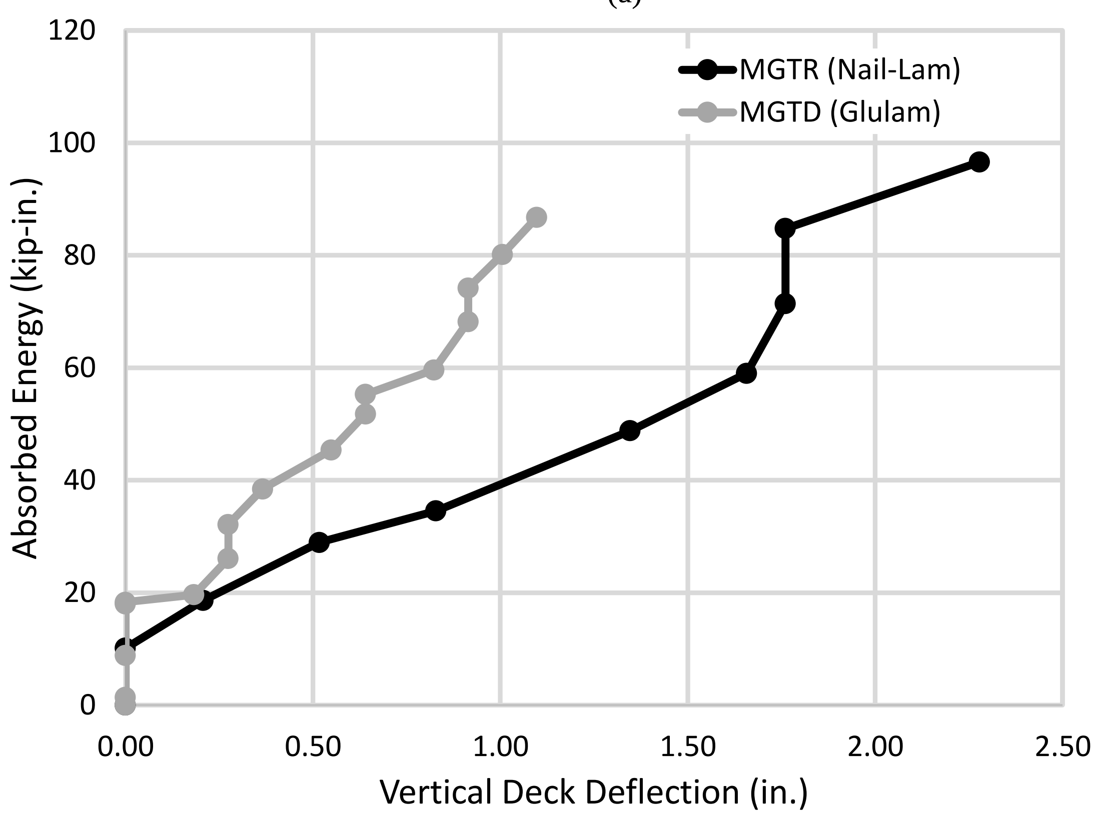



(b)

Figure 77. Comparison of Dynamic Test Nos. MGTD-1D and MGTR-1D (a) Lateral Rail Force vs. Lateral Rail Deflection and (b) Energy Absorbed vs. Lateral Rail Deflection




(a)




(b)

Figure 78. Comparison of Test Nos. MGTD-1D and MGTR-1D (a) Lateral Rail Displacement vs. Vertical Deck Deflection and (b) Energy Absorbed vs. Vertical Deck Deflection

### 7.3.2 Static Component Testing

Static testing was performed on rail and scupper block systems that were attached to glulam and nail-laminated timber decks. Test results were compared to one another to determine the relative system strength as a function of deck configuration. In the WVDOT study [1-2], two of the five static component tests on rail and scupper blocks attached to the nail-laminated deck had connection details identical to those tested during this study. As a result, these two rail and scupper block systems from the WVDOT study were also used for comparison purposes in this study. The static test results from the WVDOT study and the current study are summarized in Table 9. The force vs. deflection curves from the WVDOT study and this study are provided in Figure 79. The legend in Figure 79 denotes those two tests (test nos. WVS-1 & MGTR) were run twice. During test no. WVS-1, the original instrumentation and testing apparatus that was used to load the rail and scupper blocks did not provide sufficient force to adequately deflect the rail. As a result, the test was repeated using modified testing apparatus. The original test no. WVS-1 was designated WVS-1 (1st), and the second run was designated test no. WVS-1 (2nd). The decision to re-run test no. MGTR-1S was discussed in Section 7.2.1 of this report. Test no. WVS-4 was the other relevant static component test from the WVDOT study.

In Figure 79, the curve plotted in orange provides the results for the only rail and scupper block system installed on the glulam timber deck. When comparing this orange curve (test no. MGTD-1S) to the other curves, it can be observed that this rail and scupper blocks on a glulam timber deck provided the highest initial linear stiffness. The blue (test no. MGTR-1S) and purple (test no. MGTR-1SB) curves provided the force versus deflection behavior for the rail and scupper blocks installed on the nail-laminated deck during this study. When determining the initial linear stiffness of the timber rail and scupper blocks on the nail-laminated deck under this study, the first loading event (test no. MGTR-1S) was utilized instead of the second loading event (test no. MGTR-1SB). The remaining relevant tests that were conducted on a nail-laminated deck during the WVDOT study showed lower initial linear stiffnesses. These differences were likely due to the differing scupper block materials and setup deviations. During the WVDOT component testing program, the scupper block was fabricated from Grade No.1 SYP. Initially, the scupper blocks that were planned for use on the nail-laminated deck in this study were to be fabricated from Grade No.1 SYP. However, due to the lack of availability of Grade No 1. SYP, the scupper block material was replaced with a glulam timber material by the supplier.

For test nos. MGTR-1S, WVS-4, and WVS-1 (1st) on nail-laminated decks, the recent test (test no. MGTR-1S) using glulam versus sawn scupper blocks provided a slightly higher initial linear stiffness, as shown in Table 9. The load and deflection data shown in Table 9 were used to obtain the initial linear stiffness for the rail and scupper blocks in each test. Results from the second loading event for test no. MGTR-1SB on the nail-laminated deck are not included in Table 9. The initial linear stiffness for test no. MGTR-1SB was not considered due to the existing damage that occurred during the first static test (test no. MGTR-1S) on the nail-laminated deck under this study.

Static test results have been graphically provided for lateral rail deflections exceeding 10 in. or more. However, some of those graphical results beyond 10 in. of lateral rail deflection may not be as important for determining equivalency and adequacy of bridge rail performance when attached to alternative deck types. For the WVDOT full-scale crash test (test no. WVBR-1) on a glulam rail and scupper block assembly on a nail-laminated, timber deck, the dynamic lateral rail deflection and permanent set deflection were 6.1 in. and 2.4 in., respectively. As such, comparison of strength, deflection, stiffness, and behavior were more valuable up to a lateral rail displacement of 10 in.

For each deck type, the performance of the rail and scupper blocks were compared using lateral rail force versus vertical deck deflection and a lateral rail deflection versus vertical deck deflection, as shown in Figure 80. Due to the limited number of data points that were obtained to produce these plots, both comparisons are approximate and limited to providing general trends. As shown in Figure 80 and for the same vertical deck deflection, the rail and scupper blocks on the glulam deck resisted more lateral rail force than resisted by rail and scupper blocks on the nail-laminated deck. This trend between lateral resistive forces and vertical deck deflections does exclude the first 0.15 in. of vertical deck deflection. For lateral rail deflections through approximately 2 in., vertical deck deflections were nearly identical for both deck types. For lateral rail deflections greater than 2 in., the nail-laminated deck produced greater deflections as compared to the glulam deck. A summary of results for lateral rail deflection versus vertical deck deflection is provided in Table 10.

Table 9. Comparison of Static Tests

| **Component Test No.** | **Deck Type** | **Peak Static Load (kips)** | **Lateral Load[1] (kips)** | **Lateral Deflection[1] (in.)** | **Initial Linear Rail/Scupper Block Stiffness (kip/in.)** |
|---|---|---|---|---|---|
| MGTD-S1 | Glulam | 23.1 | 6.2 | 1.0 | 6.2 |
| MGTR-S1 | Nail-Lam | 11.7 | 3.6 | 0.7 | 5.2 |
| WVS-4 | Nail-Lam | 16.8 | 3.5 | 0.7 | 4.9 |
| WVS-1 (1st) | Nail-Lam | 12.0 | 6.1 | 0.7 | 2.3 |
| WVS-1 (2nd) | Nail-Lam | 17.5 | NA | NA | NA |

[1]Load and deflection are used to obtain values in initial linear stiffness
NA – Not Applicable

Table 10. Static Testing Results: Lateral Rail Defection and Vertical Deck Deflection

| **Test No.** | **Deck Type** | **Peak Vertical Deflection (in.)** | **Corresponding Lateral Rail Displacement[1] (in.)** |
|---|---|---|---|
| MGTD-1D | Glulam | 0.7 | 12.1 |
| MGTR-1D | Nail-Lam | 1.1 | 11.5 |

[1]Lateral rail deflection measured at vertical impact height

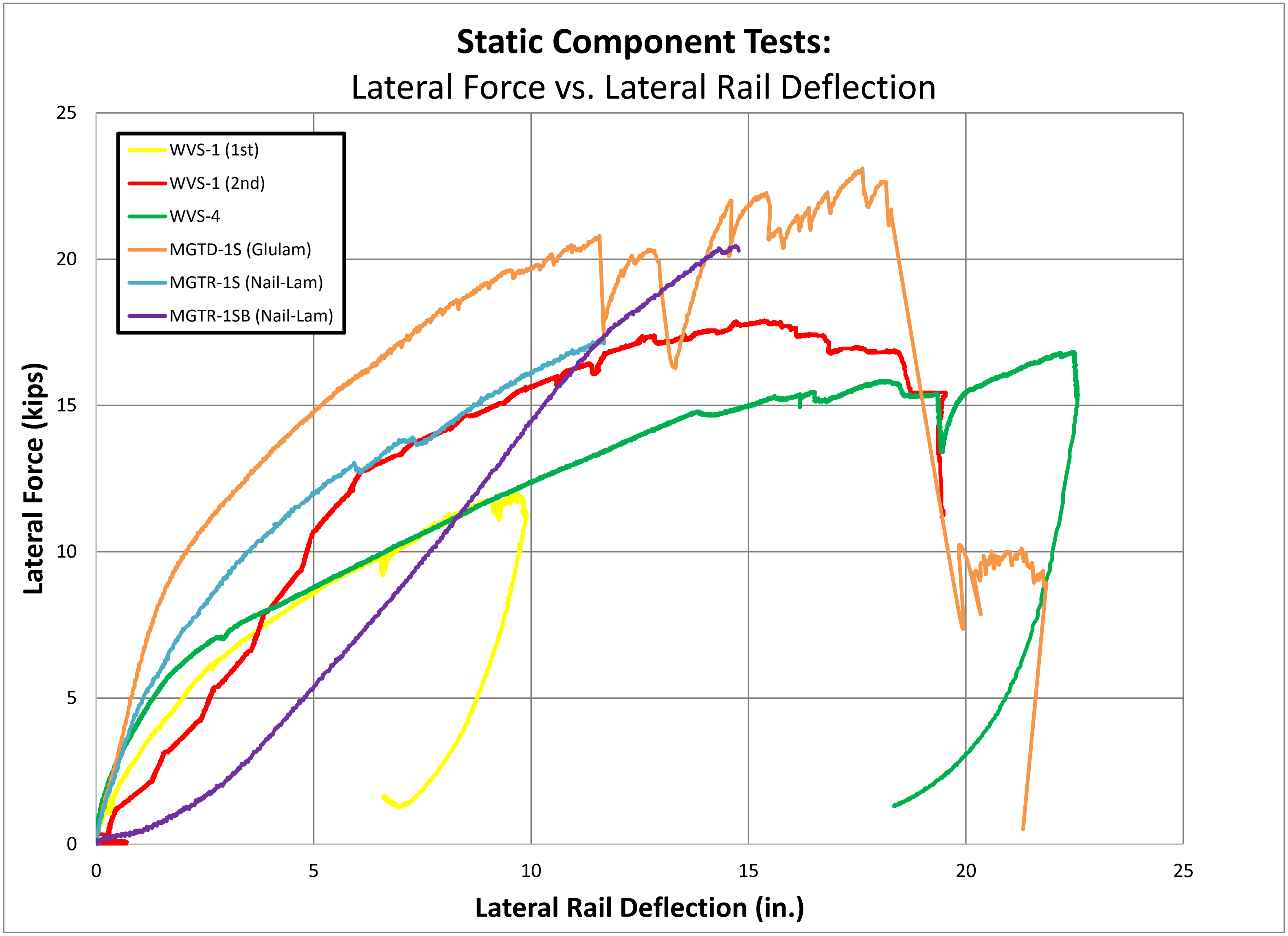
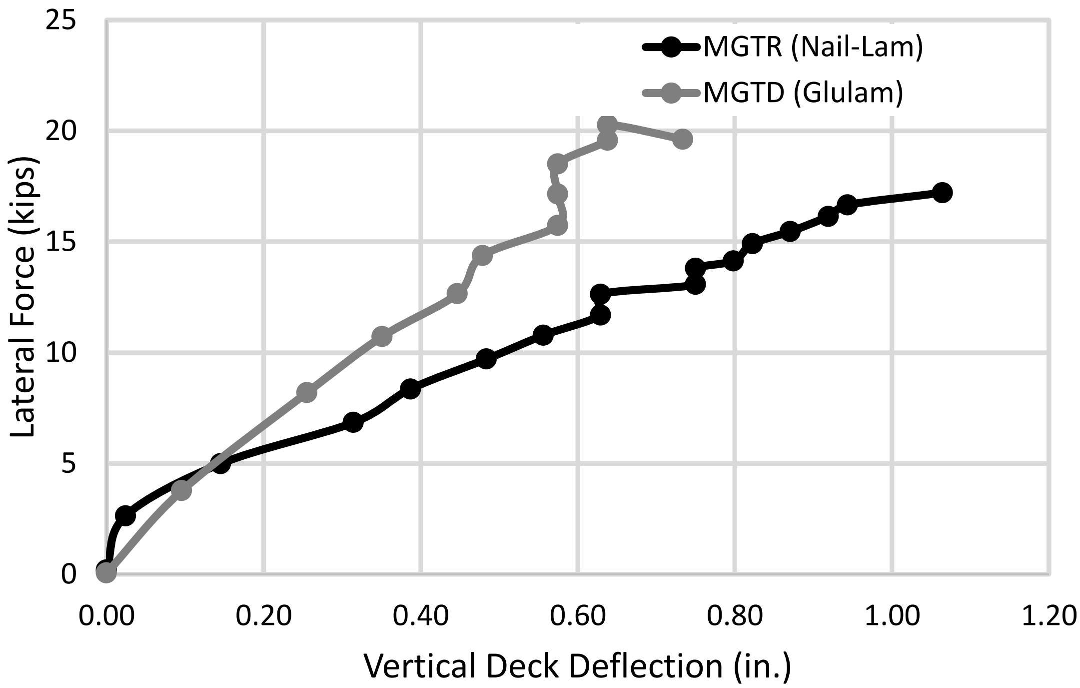
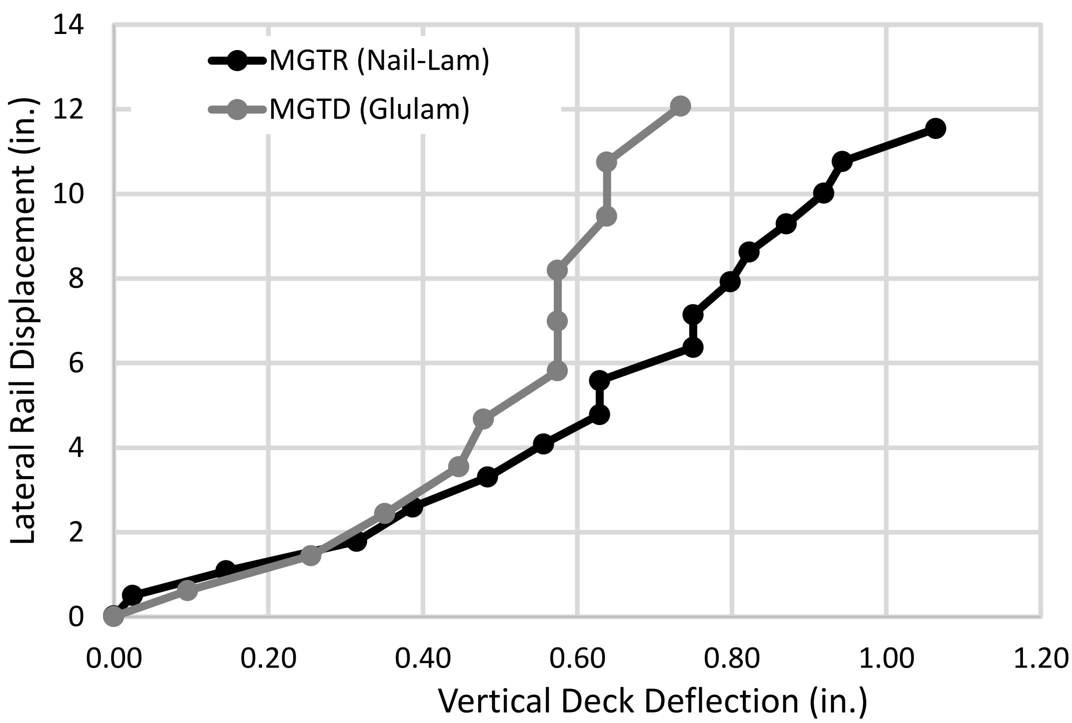



Figure 79. Comparison of Lateral Resistive Force vs. Lateral Rail Deflection, Test Nos. WVS-1 (1st), WVS-1 (2nd), WVS-4, MGTD-1S, MGTR-1S, MGTR-1SB




(a)




(b)

Figure 80. Comparison of Static Test Nos. MGTR-1S and MGTD-1S (a) Lateral Rail Force vs. Vertical Deck Deflection and (b) Lateral Rail Deflection vs. Vertical Deck Deflection

# 8 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The objectives of this project were to: (1) develop the necessary details to adapt the 2009 AASHTO MASH [3] WVDOT TL-1 low-height, glulam timber bridge railing system for use on a typical USDA-FS transverse, glulam timber bridge deck [1-2] and (2) demonstrate that the TL-1 low-height, glulam timber bridge rail would meet the 2016 MASH [4] TL-1 impact safety standards by proving equivalent or greater lateral stiffness and strength when installed on the transverse, glulam deck as compared to its performance observed when installed on the as-tested, nail-laminated timber deck [1-2].

Several tasks were completed to accomplish these objectives. First, an in-depth literature review was conducted to identify previously-developed, low-height, bridge railings, barriers, and end treatments that would inform the adaptation process involving the WVDOT TL-1 low-height, glulam timber bridge rail and end terminal system that was utilized in this study. Within the literature review, details specifically pertaining to relevant bridge railings, barriers, and end treatments were documented. In addition, relevant bridge design manuals that were used by the Forest Service National Technology and Development Division were reviewed to assist with the design, layout, and configuration of the surrogate glulam timber deck system. This deck system was later constructed for use in the component testing and evaluation program involving the glulam bridge rail segment supported by scupper blocks. A discussion of the relevant design manuals was provided. Further, the Forest Service National Technology and Development Division personnel were queried on occasion to answer questions, provide additional details, assist with design guidance, help with selecting a representative glulam timber deck system, and finalize the surrogate glulam timber bridge deck system.

Next, the research team was tasked with demonstrating that the MASH TL-1 low-height, glulam timber bridge railing system would perform in an acceptable matter when adapted from transverse, nail-laminated timber bridge decks to transverse, glulam timber bridge decks. In order to perform this effort, dynamic and static component tests were conducted to demonstrate that the bridge railing system provided equivalent or greater lateral stiffness, strength, and energy dissipation, when installed on transverse, glulam decks as compared to transverse, nail-laminated decks. The research team configured 3-D test plans and CAD details to construct one surrogate glulam bridge deck system and one surrogate nail-laminated bridge deck system. Each bridge system had two short glulam rail segments supported and anchored to the deck using two scupper blocks. One static and one dynamic component test was conducted on each deck type and analyzed to compare lateral stiffness, strength, energy dissipation, and overall performance between deck types.

Upon completion of the component tests, the results were compared to one another. This comparison revealed that the glulam rail and scupper block system resisted more lateral force and absorbed more energy when installed on a glulam timber deck as compared to a nail-laminated deck. Additionally, the glulam rail and scupper block system had a higher initial linear stiffness when installed on a glulam timber deck as compared to a nail-laminated timber deck.

These findings led to the following conclusions regarding the performance and crashworthiness of the low-height, glulam timber rail and scupper block system installed on the transverse, glue-laminated, timber bridge deck:

- The results from the component tests indicated that the MASH TL-1 glulam rail and scupper block on a transverse, glue-laminated, timber bridge deck would provide equal or greater performance by providing equivalent or greater lateral stiffness, strength, and energy dissipation as compared to the same glulam bridge rail installed on a transverse, nail-laminated, timber bridge deck, which was observed during one full-scale crash test [1-2].

- The component testing results demonstrated that the MASH TL-1 glulam timber bridge rail and scupper block system installed on a transverse, glue-laminated, timber bridge deck would laterally deflect by an equal or reduced amount as compared to the same glulam bridge rail installed on a transverse, nail-laminated timber deck when subjected to the same MASH TL-1 impact conditions.

- Based on the higher performance in component testing of the glulam bridge rail installed on the transverse, glue-laminated, timber bridge deck versus installed on the transverse, nail-laminated, timber bridge deck, it was concluded that the low-height, glulam timber rail and scupper block system installed on the glulam timber bridge deck would satisfactorily meet the 2016 AASHTO MASH TL-1 impact safety criteria.

Based on the successful static and dynamic component testing of the low-height, glulam timber rail and scupper block system installed on both deck types and the prior successful MASH full-scale crash test, MwRSF researchers believe that the comparison of static and dynamic component tests provide a valid indicator of the safety and structural performance of the glulam rail and scupper block system installed on transverse, glulam timber decks. Thus, MwRSF researchers recommend that the 2009 AASHTO MASH WVDOT TL-1 low-height, bridge railing system can be adapted for use on typical USDA-FS-NTDP transverse, glulam timber deck bridges using the design details and provided herein, as shown in Figure 81 through Figure 106.

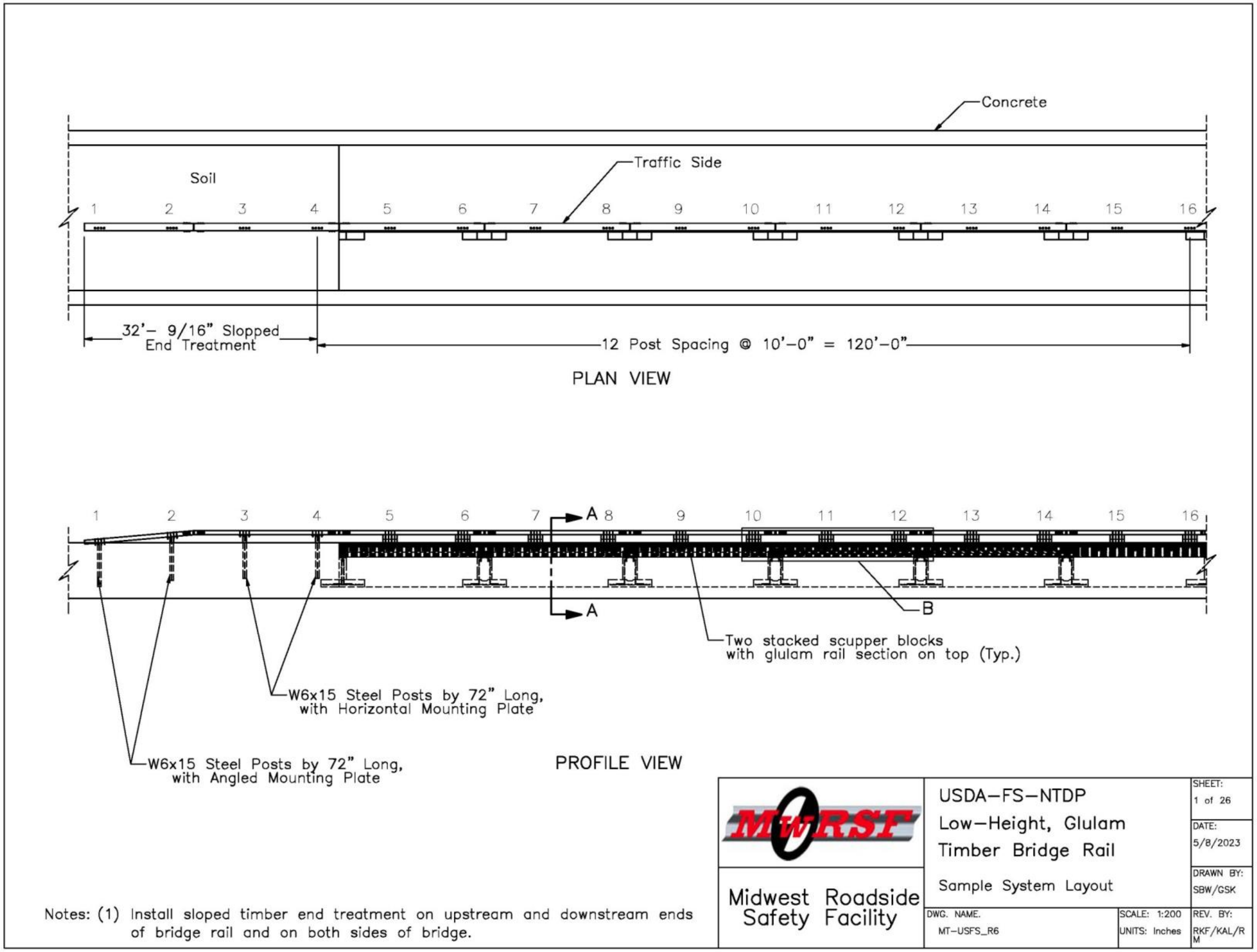



Figure 81. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Sample System Layout

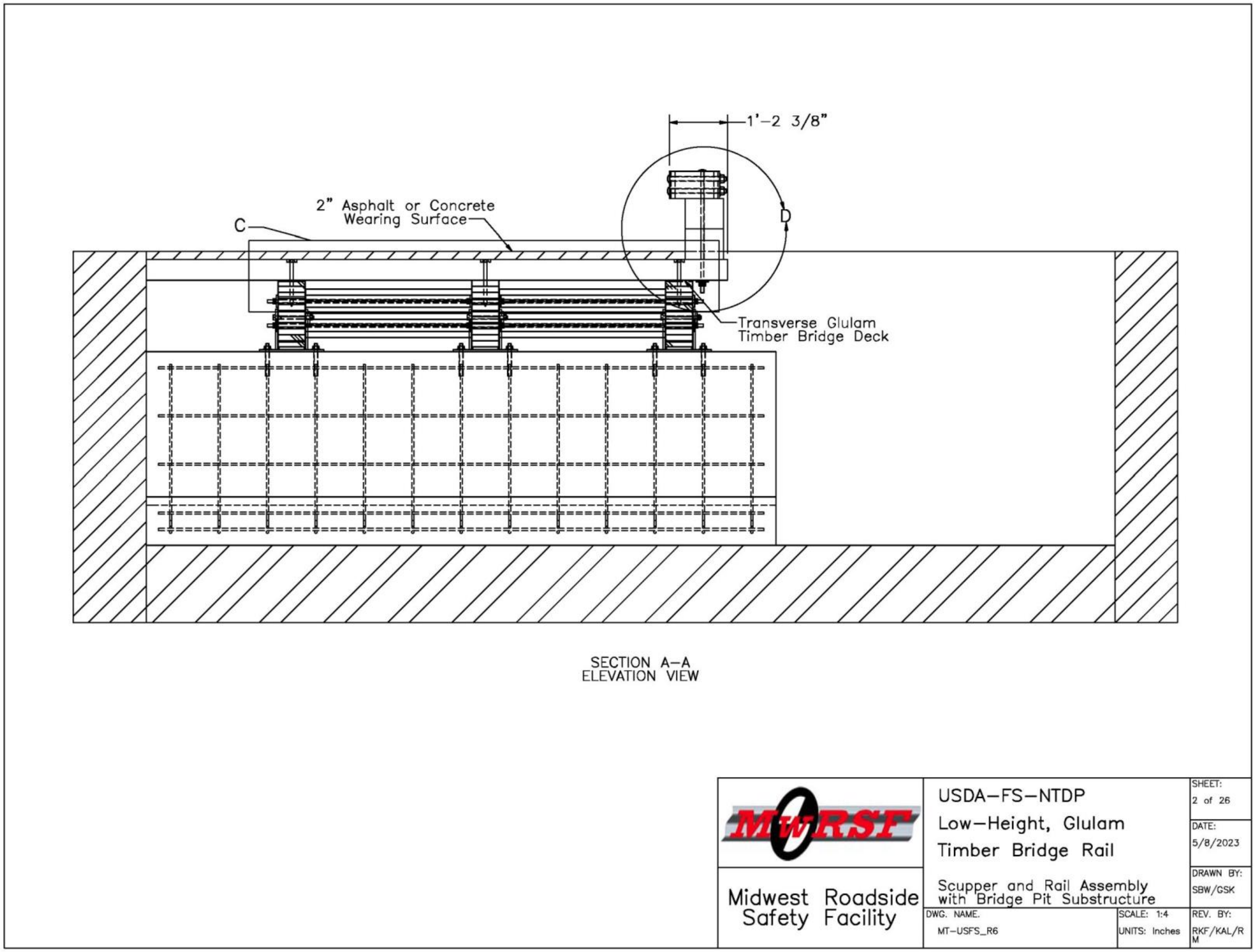
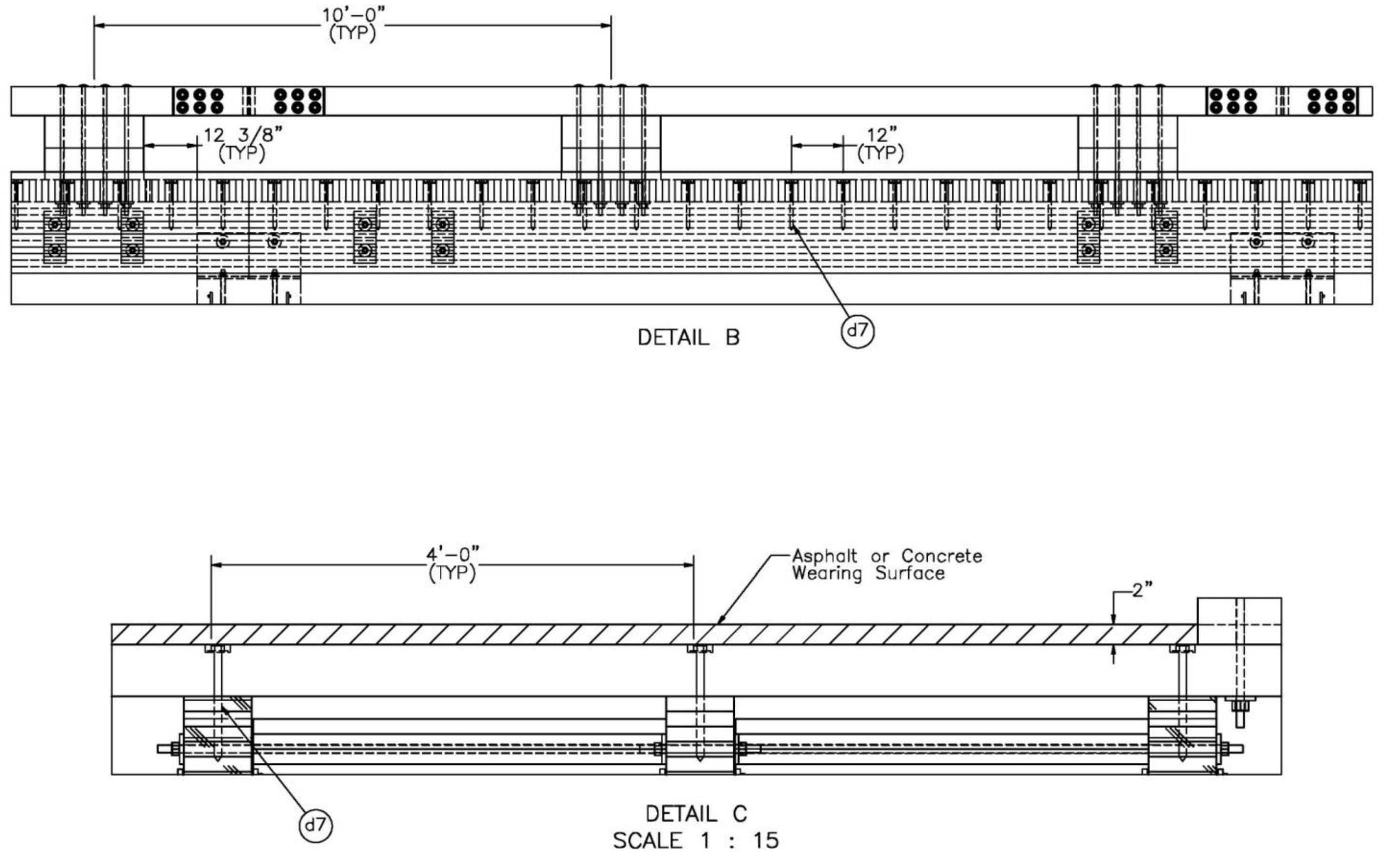



Figure 82. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Scupper and Rail Assembly with Bridge Pit Substructure




Notes: (1) Lag Screws (Part No. d7) to be centered on Bridge Girders (Part No. ).
(2) Before installation of Lag Screw a ϕ2 1/2"x3/4" deep hole shall be drilled.
(3) All field cuts, bore holes, and damages shall be treated with material acceptable to the engineer prior to installation.

| | | | |
|---|---|---|---|
| MwRSF | USDA–FS–NTDP Low–Height, Glulam Timber Bridge Rail | | SHEET: 3 of 26 |
| | | | DATE: 5/8/2023 |
| Midwest Roadside Safety Facility | Bridge Deck Details | | DRAWN BY: SBW/GSK |
| | DWG. NAME. MT–USFS_R6 | SCALE: 1:35 UNITS: Inches | REV. BY: RKF/KAL/RM |

Figure 83. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Bridge Deck Details

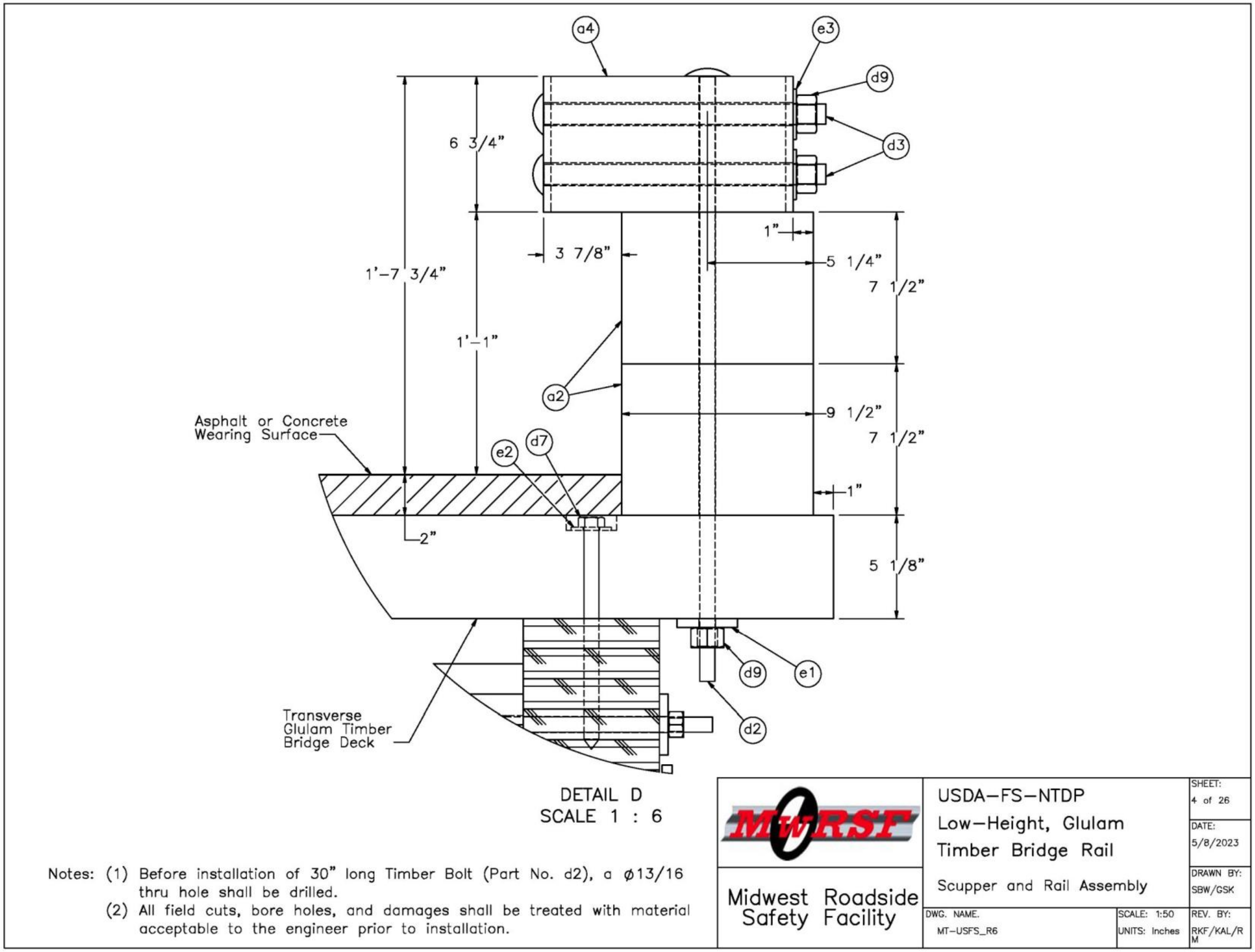



Figure 84. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Scupper and Rail Assembly

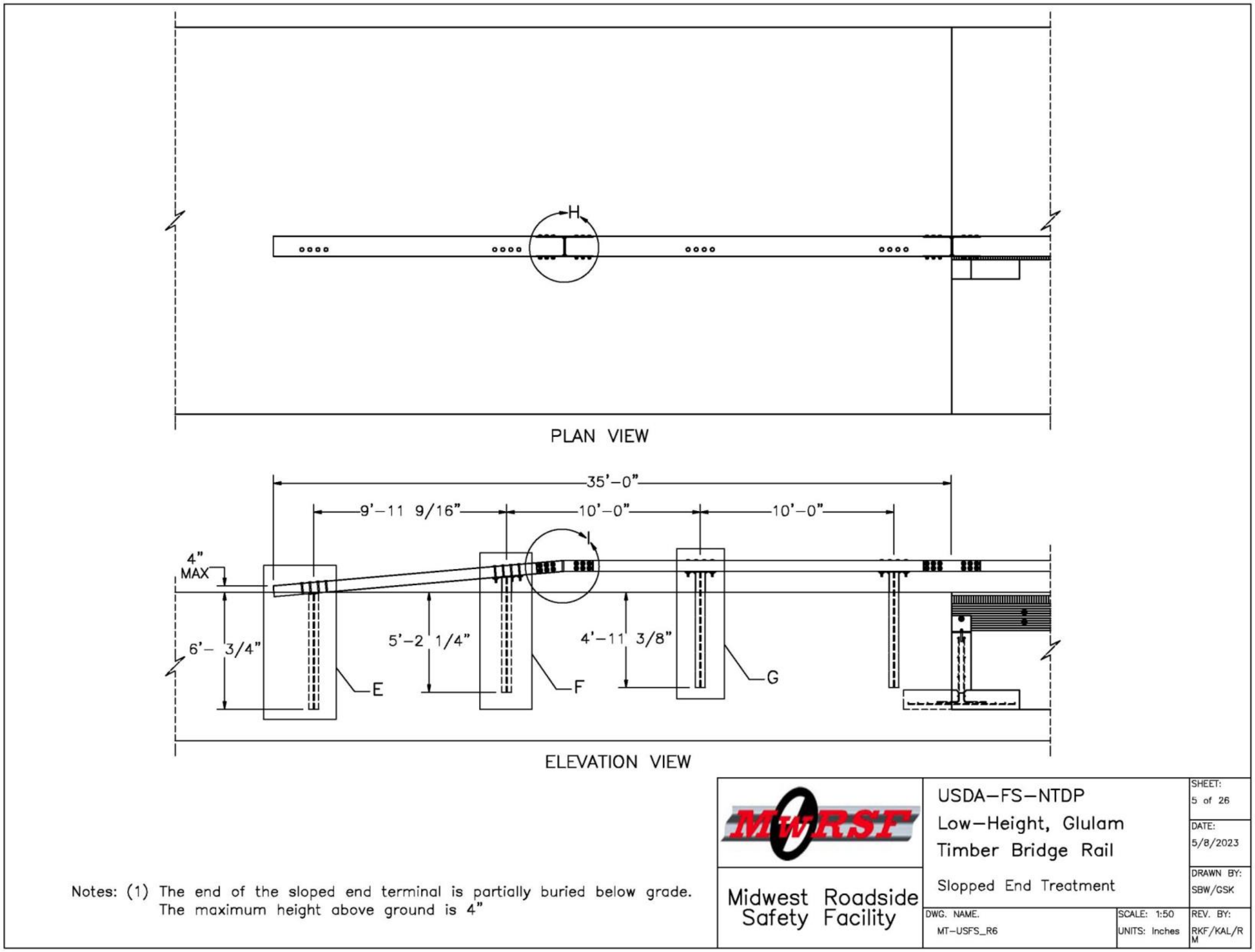



Figure 85. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Slopped End Treatment

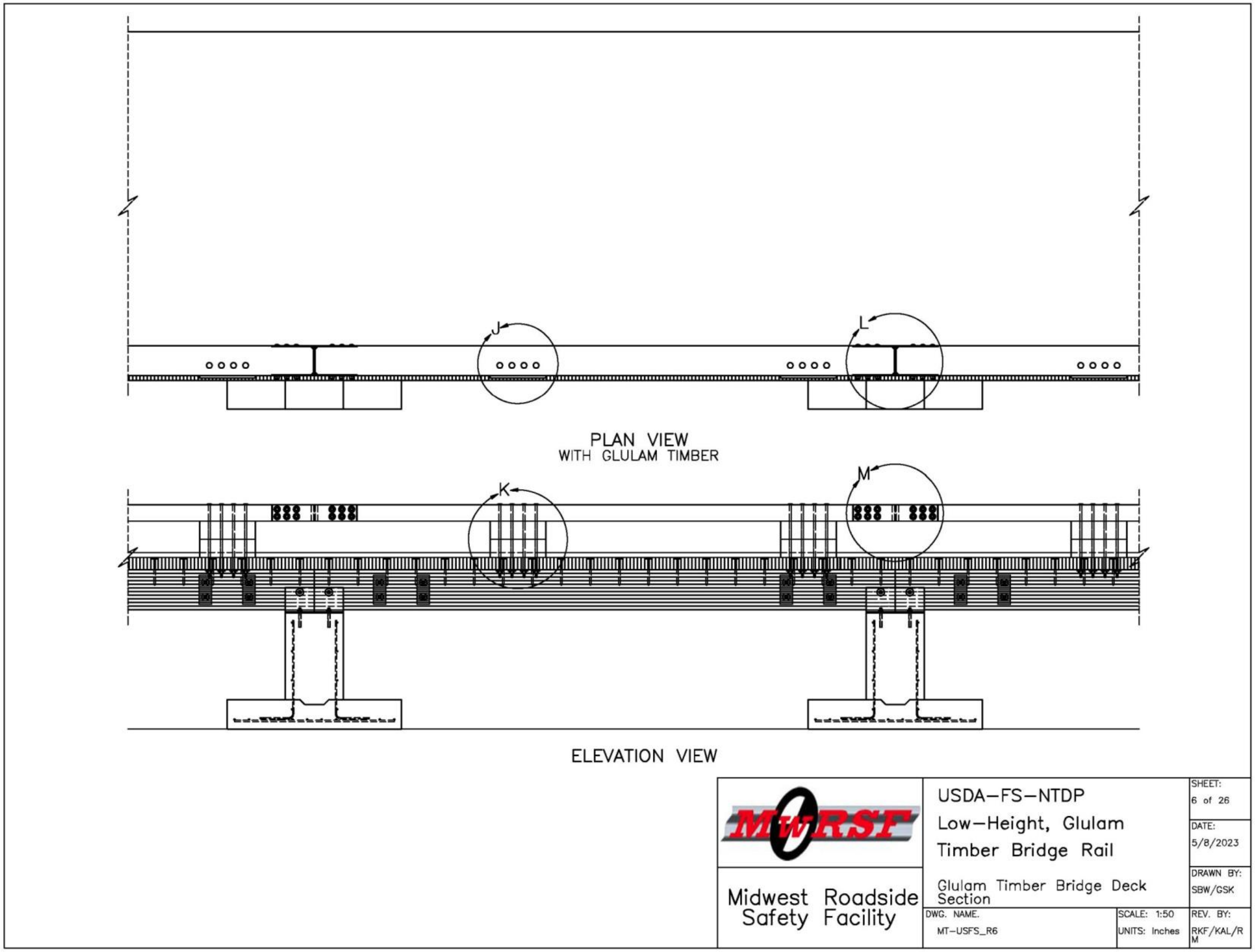



Figure 86. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Glulam Timber Bridge Deck Section

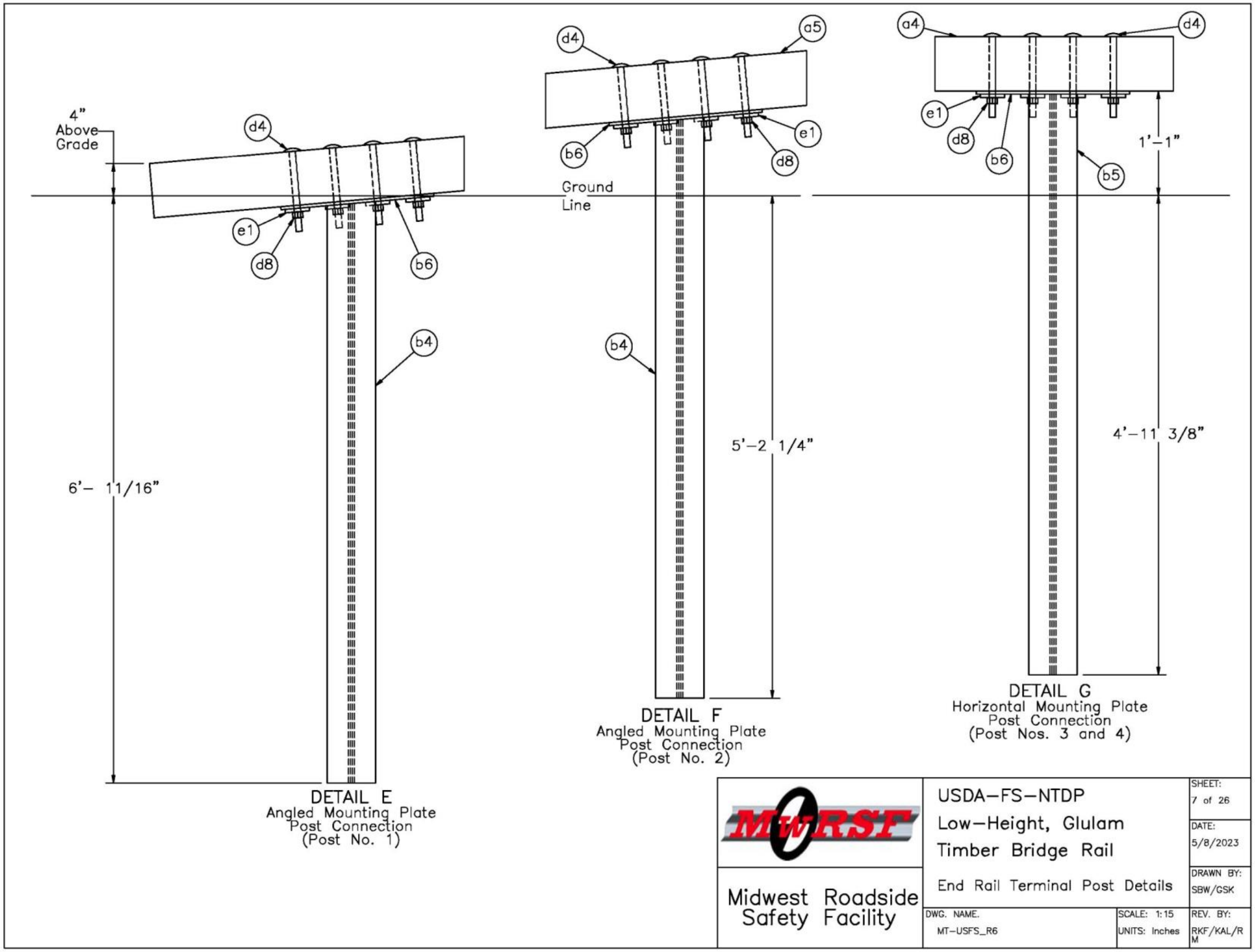



Figure 87. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: End Rail Terminal Post Details

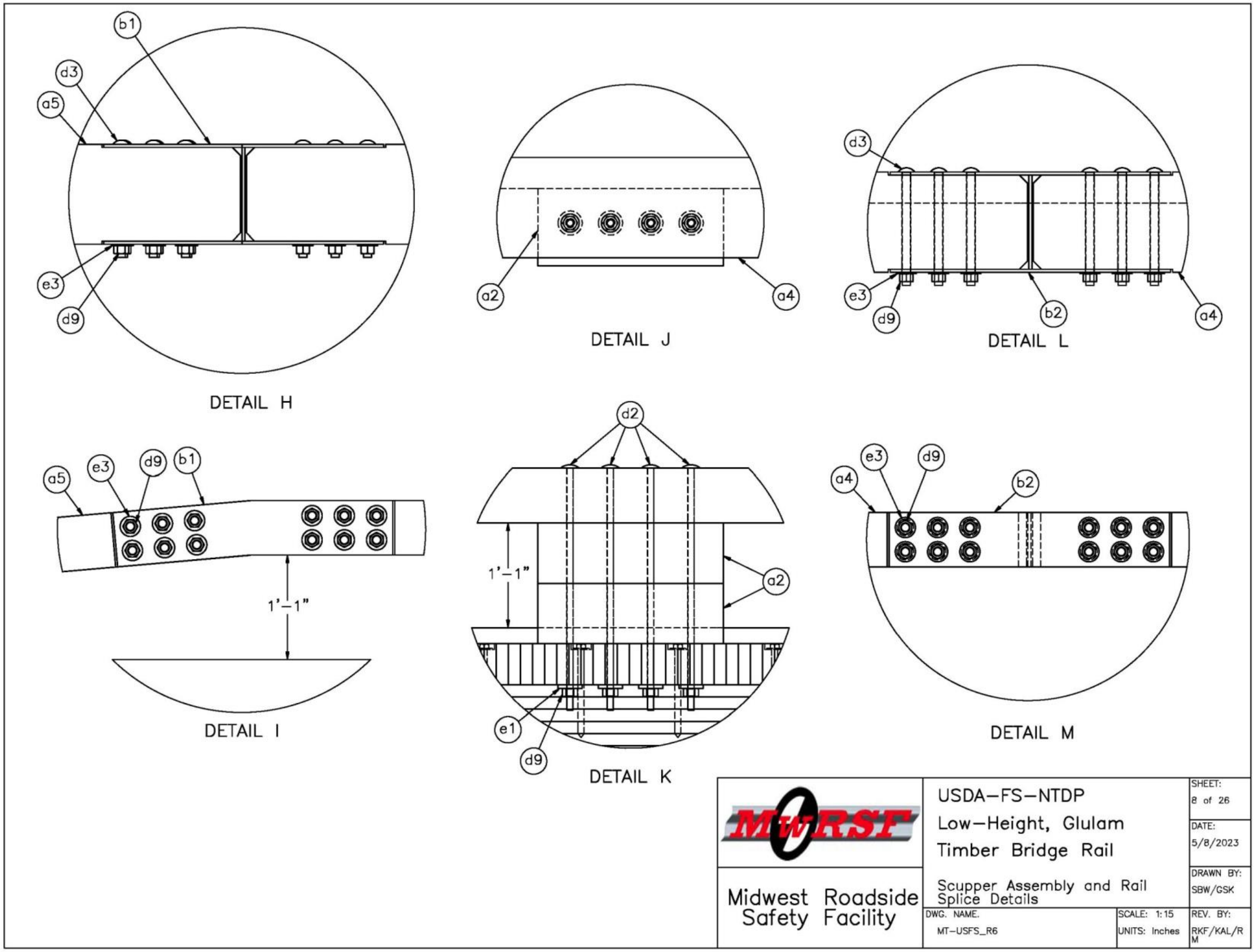



Figure 88. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Scupper Assembly and Rail Splice Details

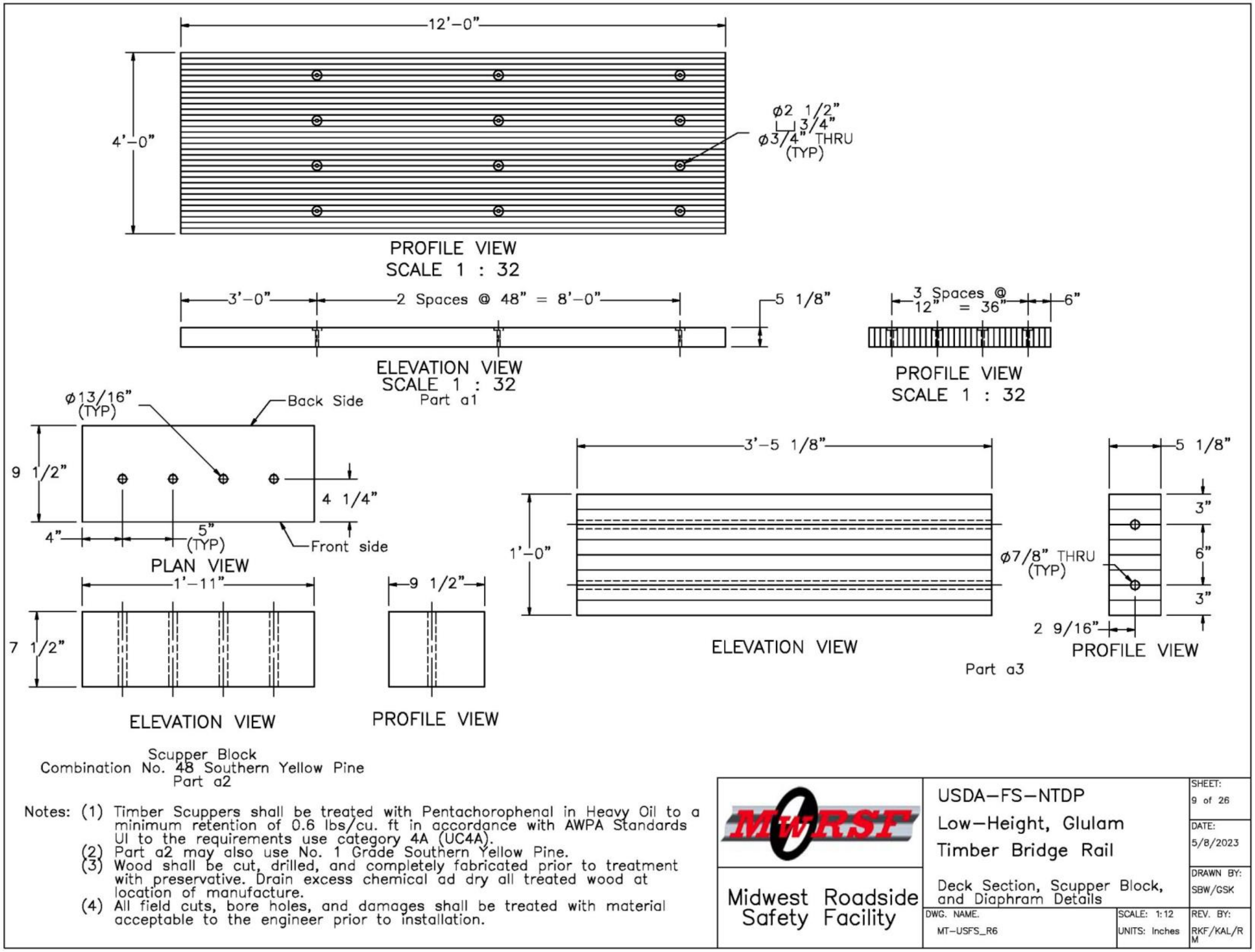


Figure 89. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Deck Section, Scupper Block, and Diaphram Details

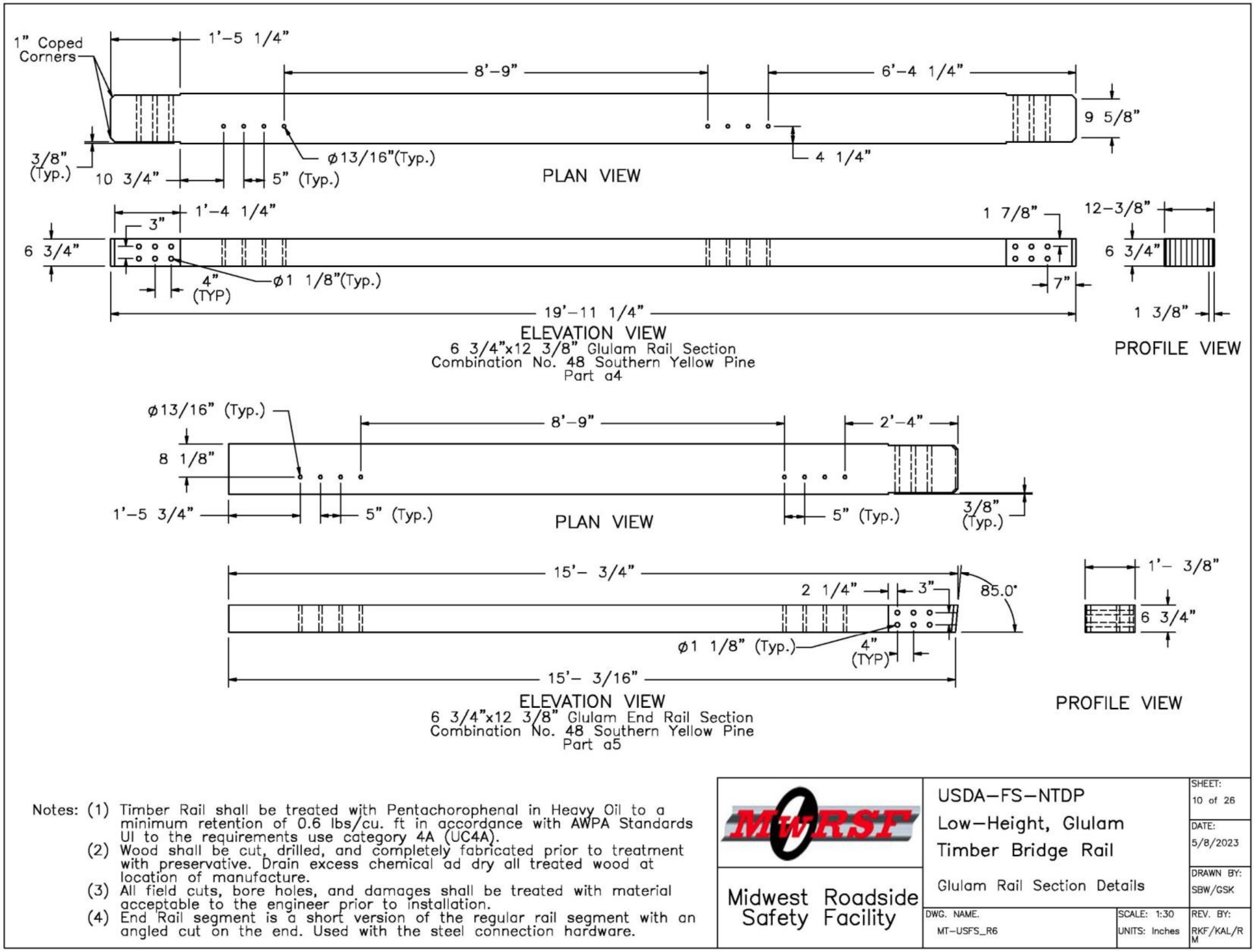
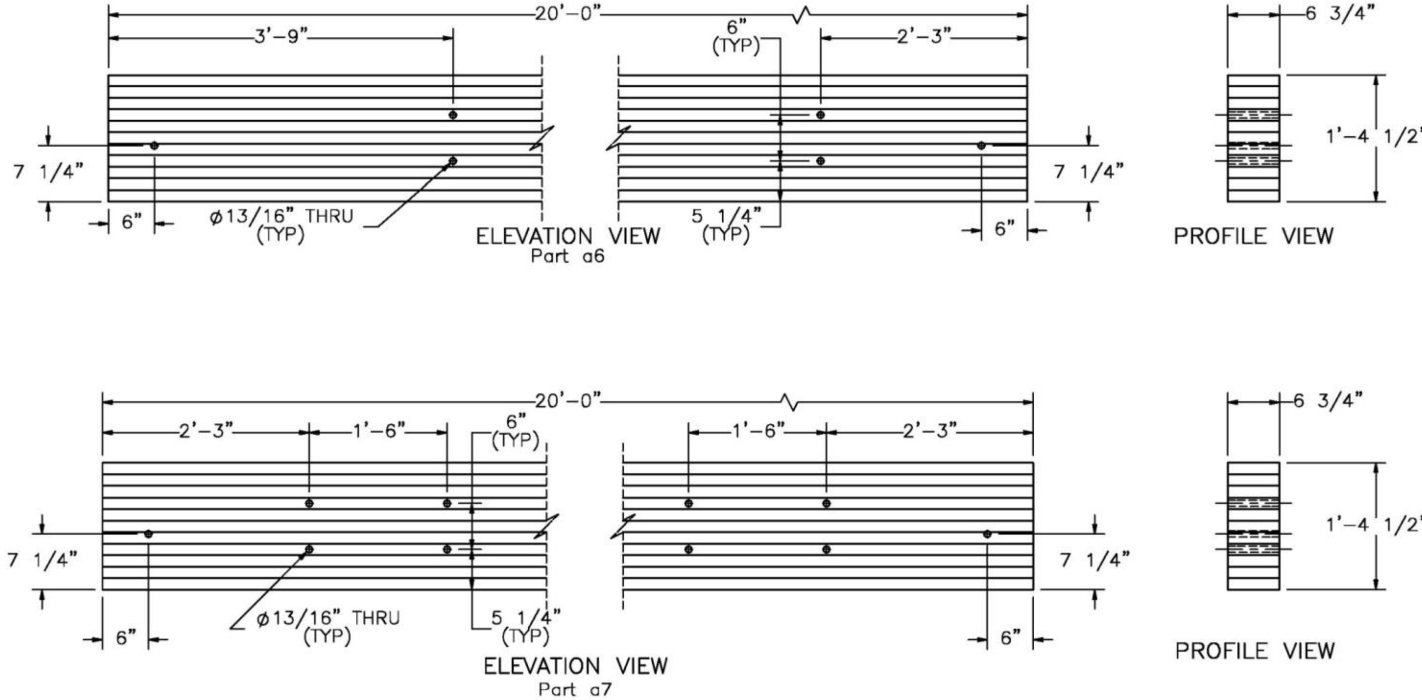


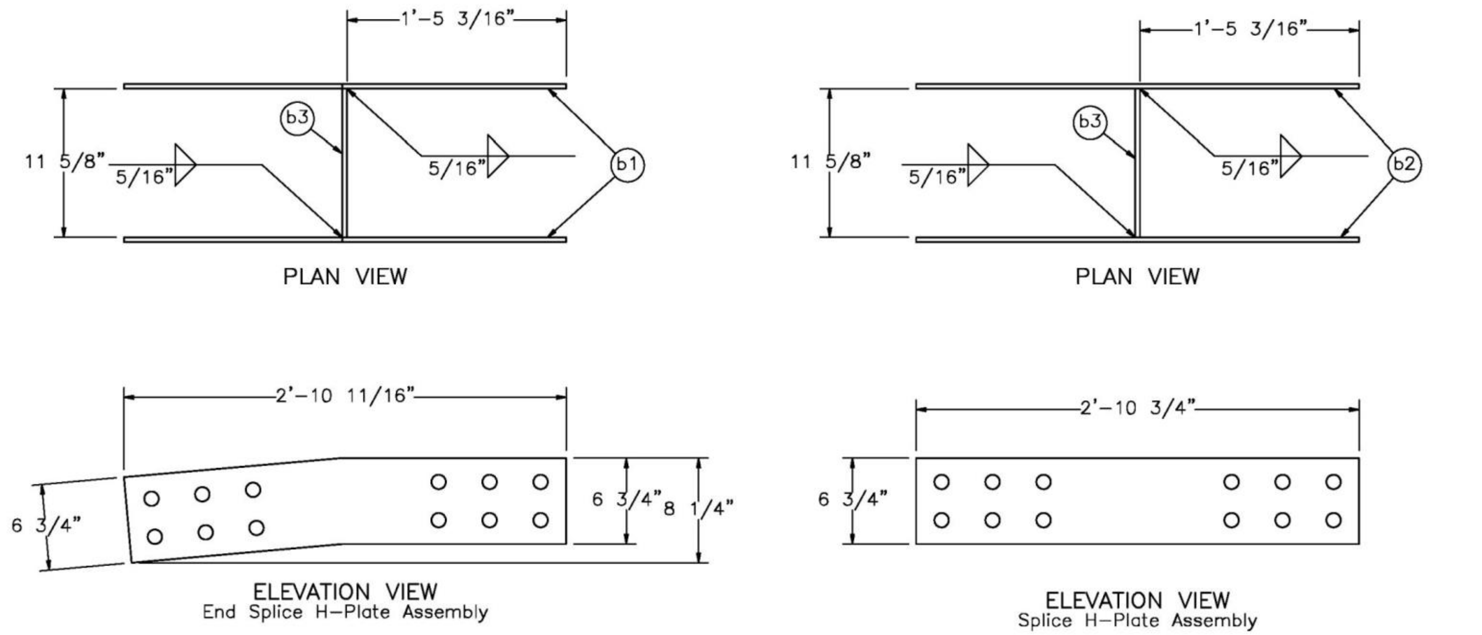

Figure 90. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Glulam Rail Section Details




Notes: (1) Timber rails shall be treated with Pentachorophenal in Heavy Oil to a minimum retention of 0.6 lbs/cu. ft in accordance with AWPA Standards UI to the requirements use category 4A (UC4A)
(2) Wood shall be cut, drilled, and completely fabricated prior to treatment with preservative. Drain excess chemical ad dry all treated wood at location of manufacture.
(3) All field cuts, bore holes, and damages shall be treated with material acceptable to the engineer prior to installation.

| | | | |
|---|---|---|---|
| MwRSF | USDA–FS–NTDP Low–Height, Glulam Timber Bridge Rail | | SHEET: 11 of 26 |
| | | | DATE: 5/8/2023 |
| Midwest Roadside Safety Facility | Glulam Girders | | DRAWN BY: SBW/GSK |
| | DWG. NAME. MT–USFS_R6 | SCALE: 1:20 UNITS: Inches | REV. BY: RKF/KAL/RM |

Figure 91. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Glulam Girders




| H-Plate Subassemblies | | | | |
|---|---|---|---|---|
| Item No. | QTY | Description | Material Specification | Treatment Specification |
| – | 1 | End Splice H-Plate | – | ASTM A123 |
| b1 | 2 | Angled, 34 3/4"x6 3/4"x3/8 Splice Plate | ASTM A572 Gr. 42 | – |
| b3 | 1 | 11 5/8"x6 3/4"x3/8" Splice Gusset | ASTM A572 Gr. 42 | – |
| | | | | |
| – | 6 | Splice H-Plate | – | ASMT A123 |
| b2 | 2 | Horizontal, 34 3/4"x6 3/4"x3/8 Splice Plate | ASTM A572 Gr. 42 | – |
| b3 | 1 | 11 5/8"x6 3/4"x3/8" Splice Gusset | ASTM A572 Gr. 42 | – |

Notes: (1) H-Plate assemblies shall be galvanized after side plates and interior gusset plate are welded together.

Midwest Roadside Safety Facility

USDA-FS-NTDP
Low-Height, Glulam Timber Bridge Rail

Splice Plate Assembly Details

| | |
|---|---|
| SHEET: | 12 of 26 |
| DATE: | 5/8/2023 |
| DRAWN BY: | SBW/GSK |
| DWG. NAME. | MT-USFS_R6 |
| SCALE: | 1:12 |
| UNITS: | Inches |
| REV. BY: | RKF/KAL/RM |

Figure 92. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Splice Plate Assembly Details

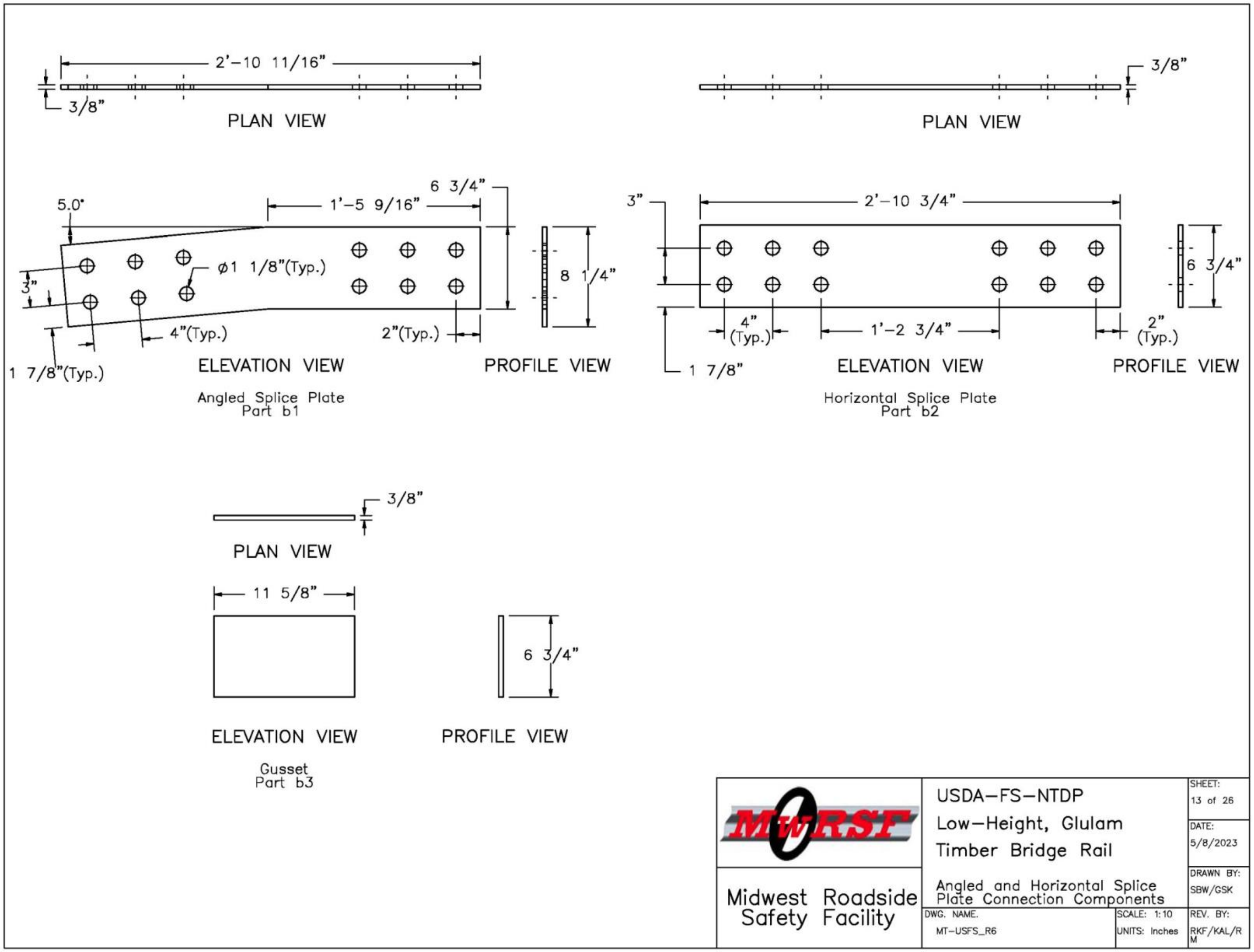
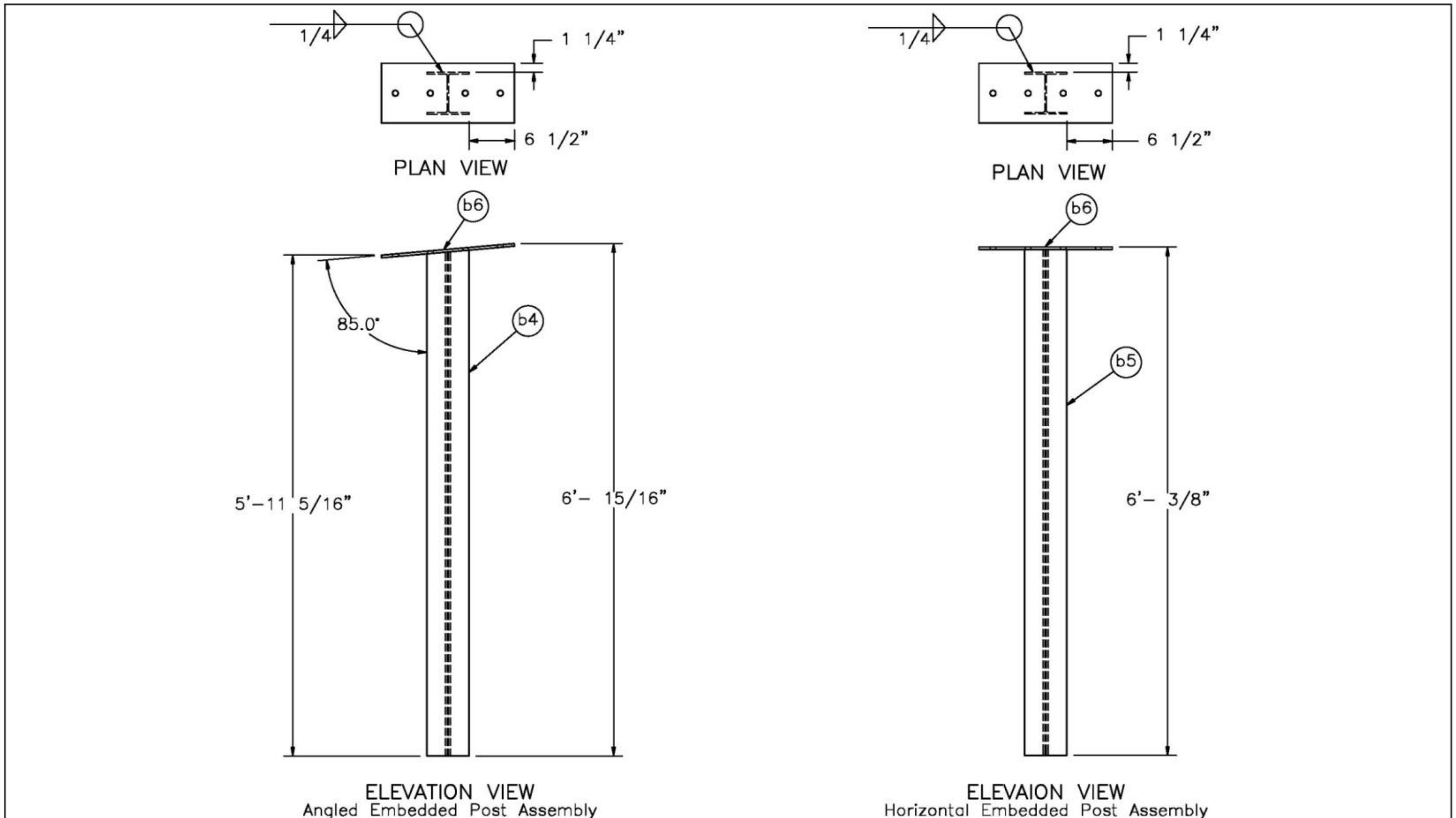



Figure 93. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Angled and Horizontal Splice Plate Connection Components




| Post Subassemblies | | | | |
|---|---|---|---|---|
| Item No. | QTY | Description | Material Specification | Treatment Specification |
| – | 2 | Angled Embedded Post Assembly | – | ASTM A123 |
| b4 | 2 | Angled W6x15, 72" Long Steel Post | ASTM A36 | – |
| b6 | 1 | 19"x8 1/2"x3/8" Post to Rail Plate | ASTM A36 | – |
| | | | | |
| – | 2 | Horizontal Embedded Post Assembly | – | ASTM A123 |
| b5 | 2 | Horizontal W6x15, 72" Long Steel Post | ASTM A36 | – |
| b6 | 1 | 19"x8 1/2"x3/8" Post to Rail Plate | ASTM A36 | – |

Midwest Roadside Safety Facility

USDA–FS–NTDP
Low–Height, Glulam Timber Bridge Rail
Soil–Embedded Posts

SHEET: 14 of 26
DATE: 5/8/2023
DRAWN BY: SBW/GSK
DWG. NAME. MT–USFS_R6
SCALE: 1:20
UNITS: Inches
REV. BY: RKF/KAL/RM

Figure 94. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Soil-Embedded Posts

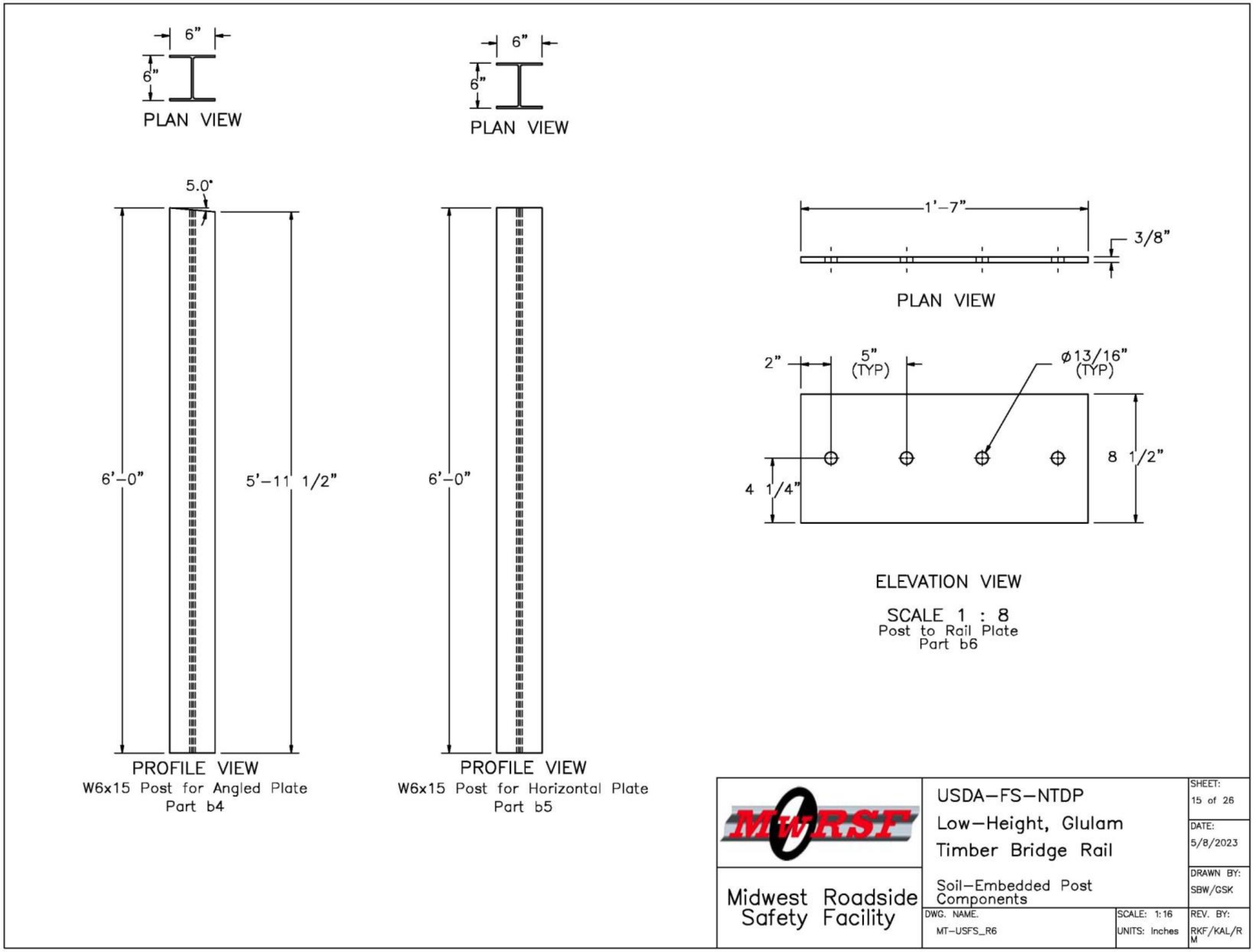
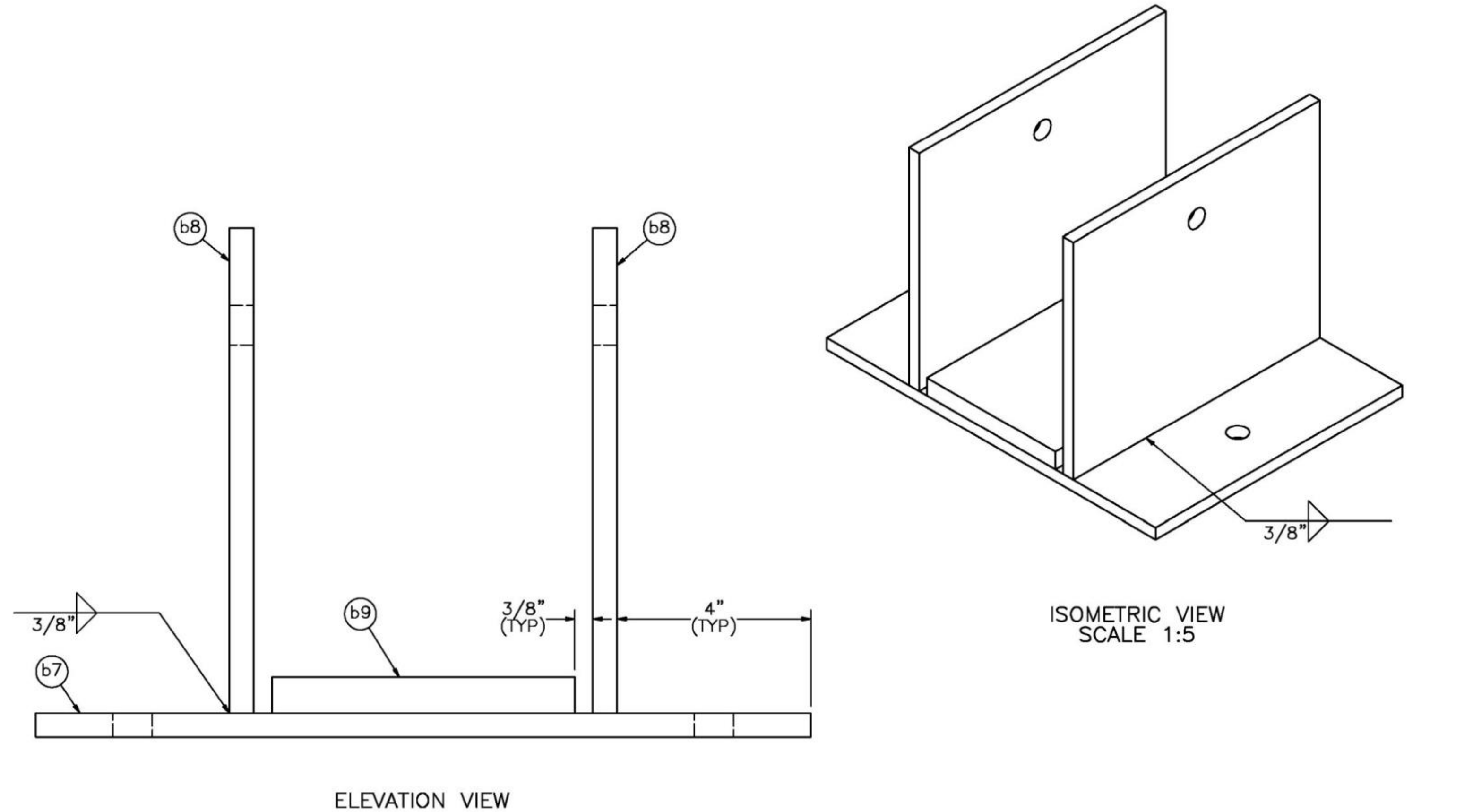



Figure 95. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Soil-Embedded Post Components




Notes: (1) Bearing Shoe assembly shall be galvanized after side plates and base plate are welded together.

| Item No. | QTY. | Description | Material Specification | Treatment Specification |
|---|---|---|---|---|
| – | 36 | Steel Plate Bearing Shoe Assembly | – | ASTM A123 |
| b7 | 1 | 12"x16"x1/2" Steel Base Plate | ASTM A36 | – |
| b8 | 2 | 12"x10"x1/2" Side Plate | ASTM A36 | – |
| b9 | 1 | 12"x6 1/4"x3/4" Elastomeric Bearing Pad | Neoprene – Min. 50 Durameter | – |

Midwest Roadside Safety Facility

USDA–FS–NTDP Low–Height, Glulam Timber Bridge Rail

Steel Plate Bearing Shoe Assembly

| | |
|---|---|
| SHEET: | 16 of 26 |
| DATE: | 5/8/2023 |
| DRAWN BY: | SBW/GSK |
| REV. BY: | RKF/KAL/RM |
| DWG. NAME. | MT–USFS_R6 |
| SCALE: | 1:3 |
| UNITS: | Inches |

Figure 96. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Steel Plate Bearing Shoe Assembly

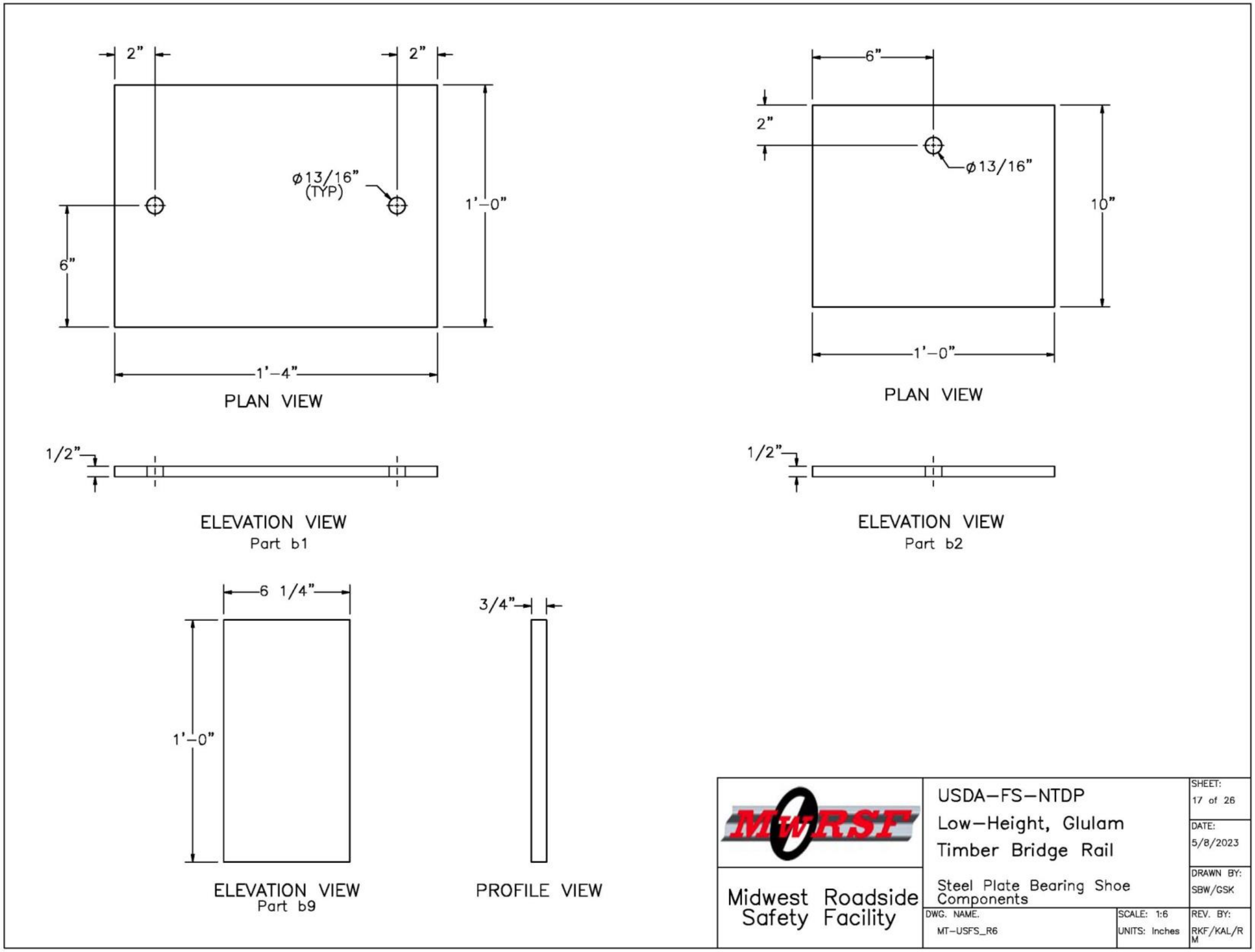



Figure 97. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Steel Plate Bearing Shoe Components

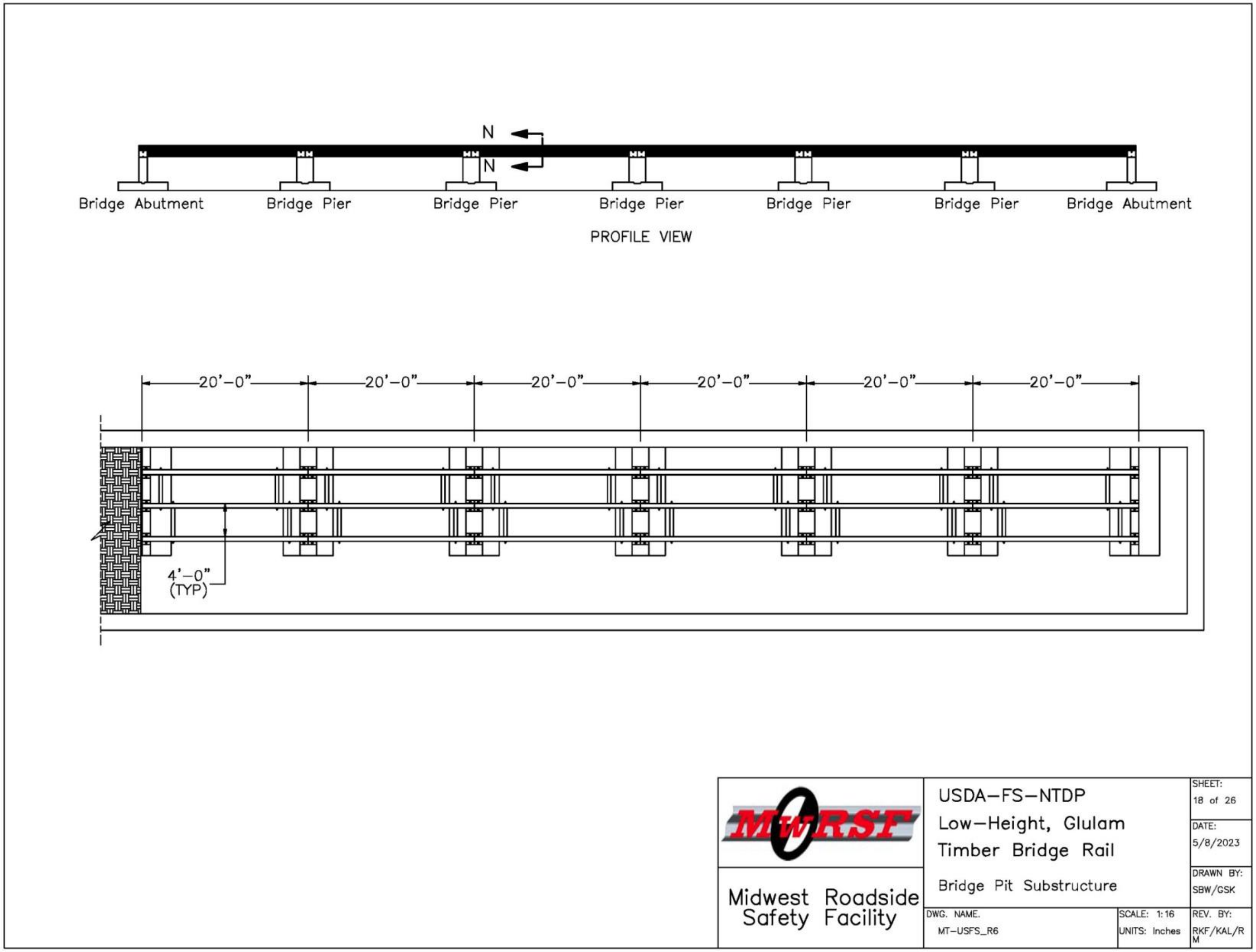



Figure 98. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Bridge Pit Substructure

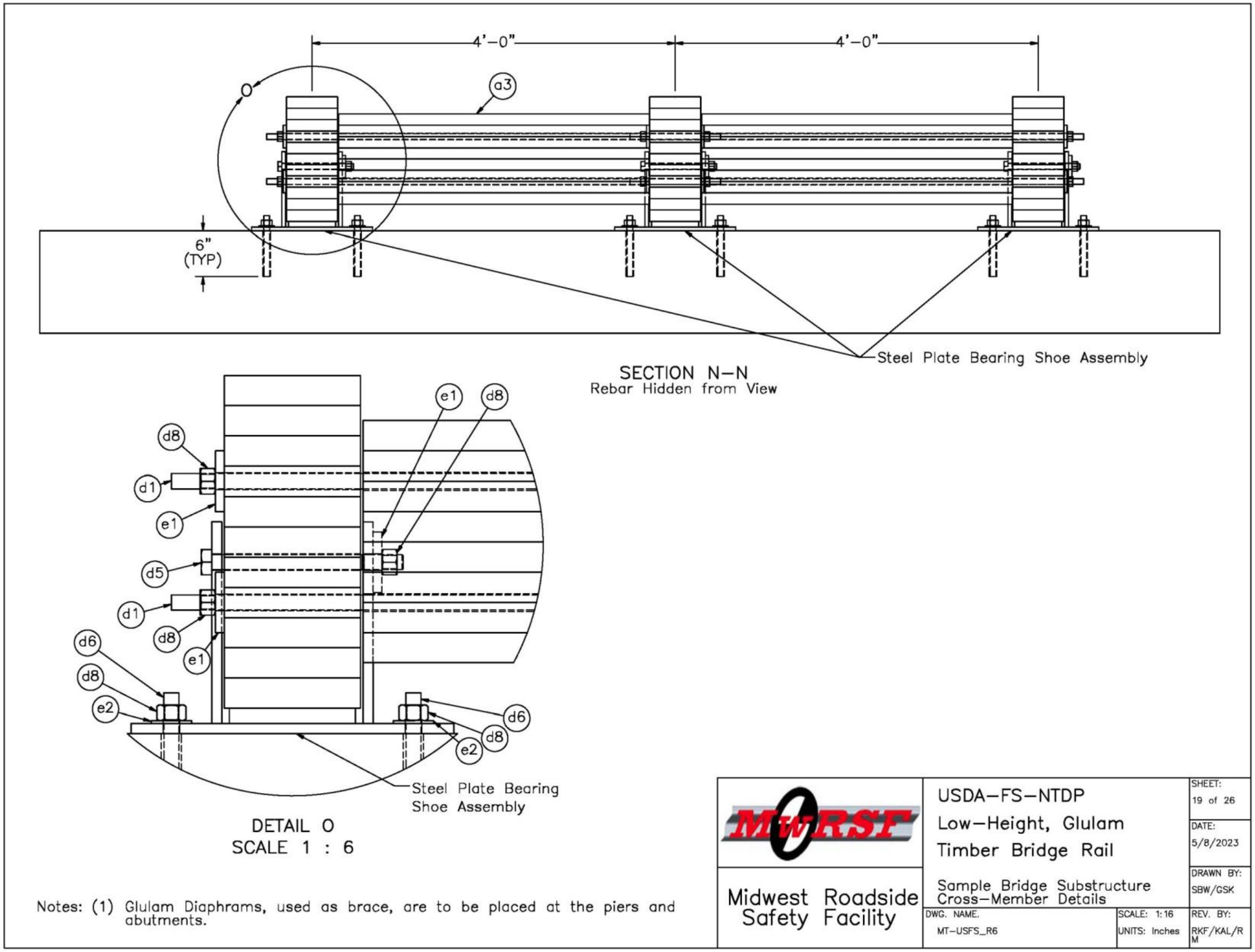



Figure 99. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Sample Bridge Substructure Cross-Member Details

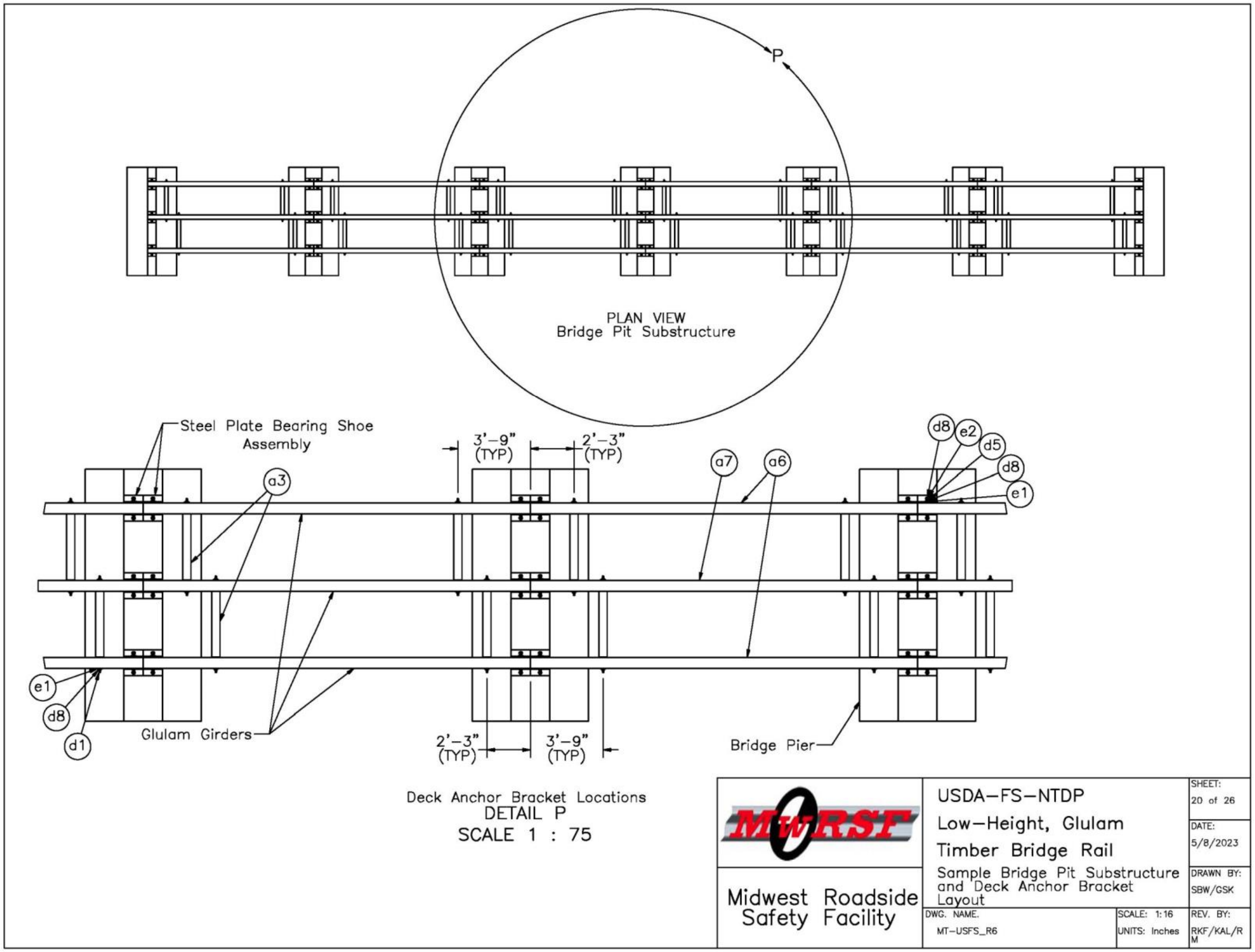
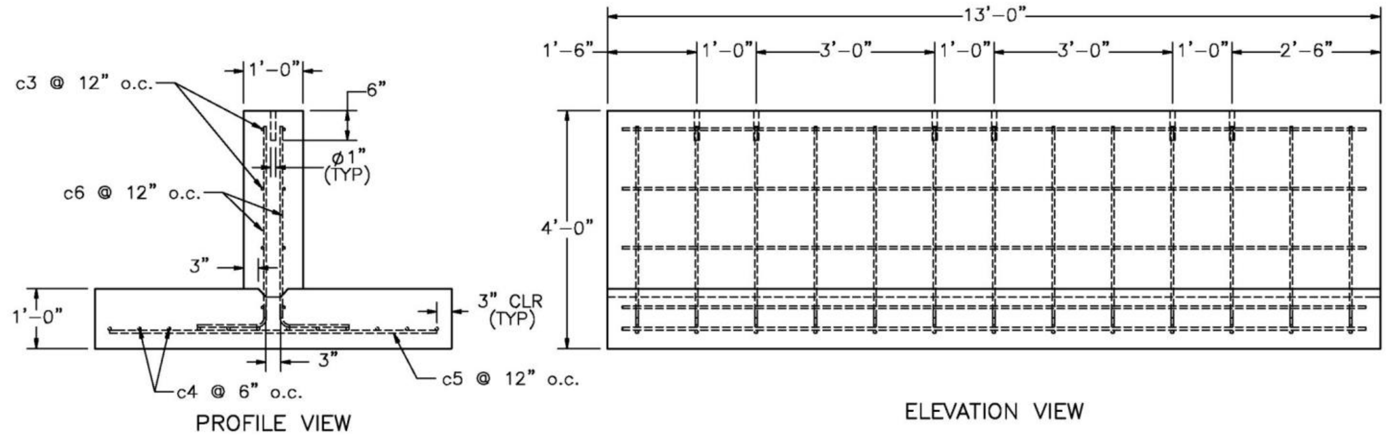



Figure 100. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Sample Bridge Pit Substructure and Deck Anchor Bracket




* rebar does not need to be epoxy–coated for testing purposes.

| Item No. | QTY. | Description | Material Specification | Treatment Specification |
|---|---|---|---|---|
| – | 2 | Abutment Assembly | – | – |
| c1 | 1 | Abutment Concrete | Min f'c = 4,000 psi | – |
| c3 | 8 | #4 Rebar, 12'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c4 | 12 | #5 Rebar, 12'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c5 | 13 | #5 Rebar, 5'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c6 | 26 | #4 Bent Rebar, 53 1/4" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |

Notes: (1) Abutment Concrete to be poured in two sections, base and then pillar.

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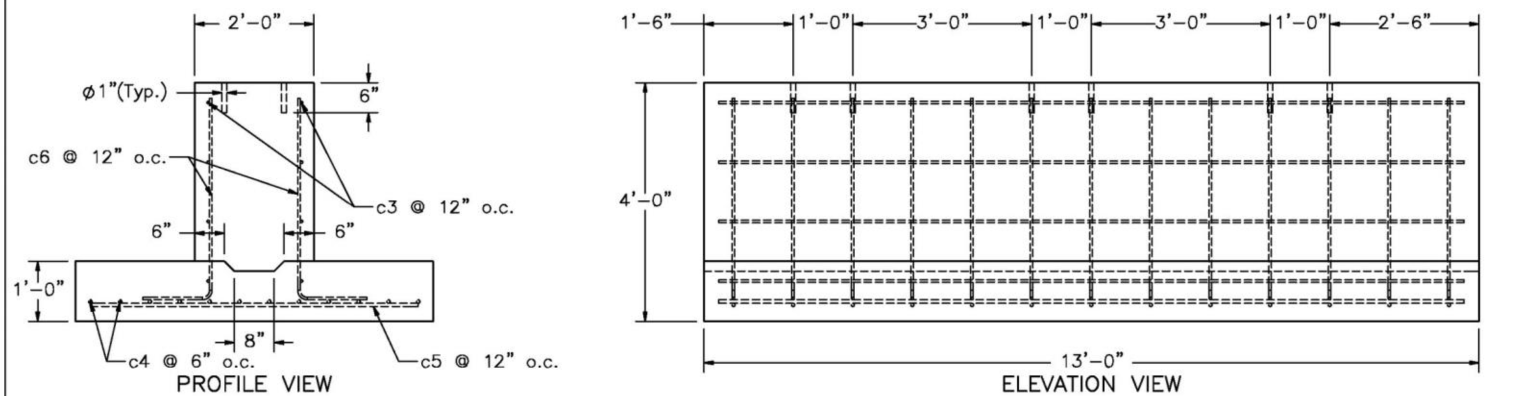
USDA–FS–NTDP
Low–Height, Glulam Timber Bridge Rail
Sample Bridge Abutment Details (Laboratory Specimens Only)

SHEET: 21 of 26
DATE: 5/8/2023
DRAWN BY: SBW/GSK
DWG. NAME. MT–USFS_R6
SCALE: 1:50
UNITS: Inches
REV. BY: RKF/KAL/RM

Figure 101. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Sample Bridge Abutment Details (Laboratory Specimens Only)




* rebar does not need to be epoxy–coated for testing purposes.

| Item No. | QTY. | Description | Material Specification | Treatment Specification |
|---|---|---|---|---|
| – | 5 | Pier Assembly | – | – |
| c2 | 1 | Pier Concrete | Min f'c = 4,000 psi | – |
| c3 | 8 | #4 Rebar, 12'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c4 | 12 | #5 Rebar, 12'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c5 | 13 | #5 Rebar, 5'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c6 | 26 | #4 Bent Rebar, 53 1/4" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |

Notes: (1) Pier Concrete to be poured in two sections, base and then pillar.

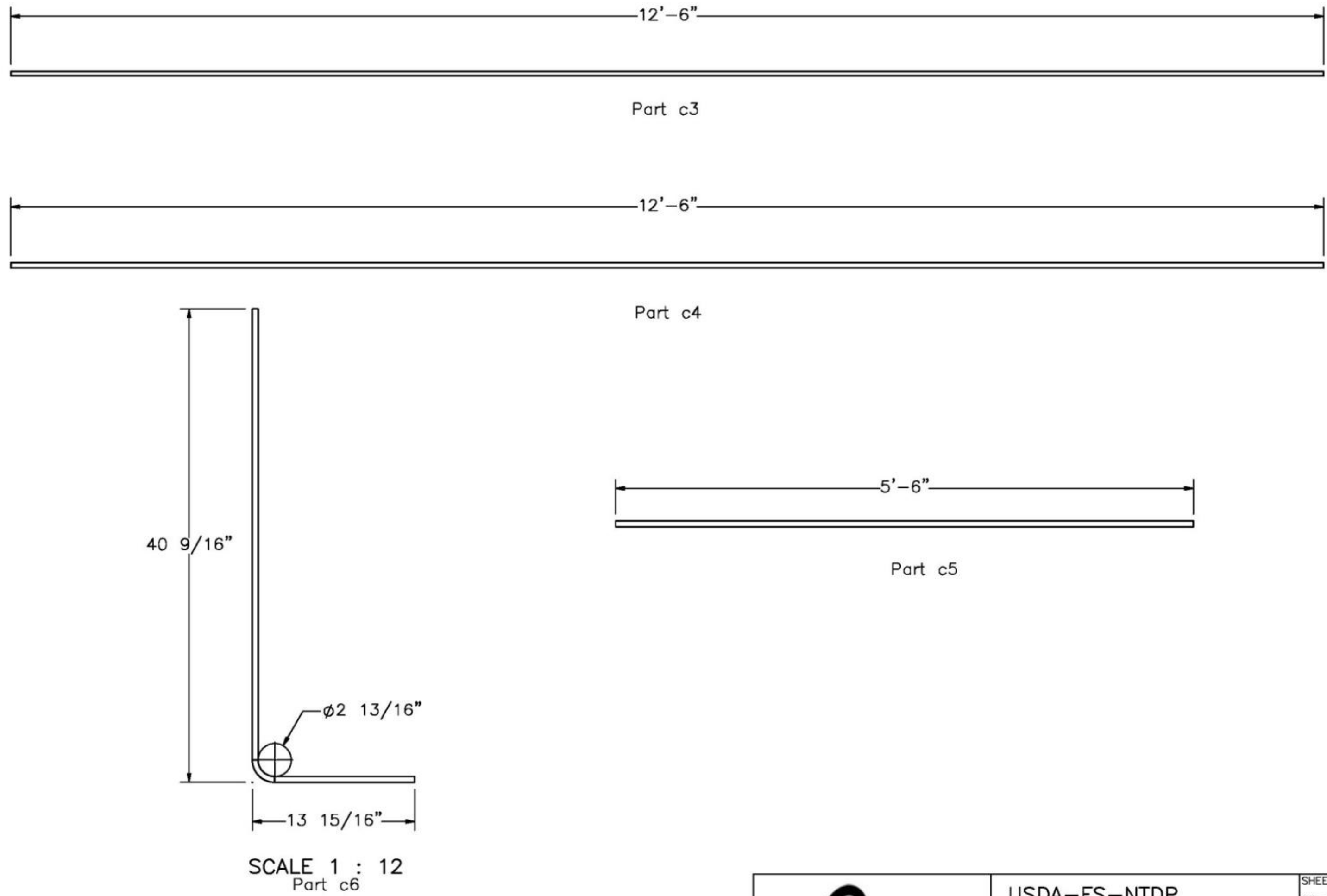
MwRSF
Midwest Roadside Safety Facility

USDA–FS–NTDP
Low–Height, Glulam Timber Bridge Rail

Sample Bridge Pier Details (Laboratory Specimens Only)

SHEET: 22 of 26
DATE: 5/8/2023
DRAWN BY: SBW/GSK
DWG. NAME. MT–USFS_R6
SCALE: 1:50
UNITS: Inches
REV. BY: RKF/KAL/RM

Figure 102. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Sample Bridge Pier Details (Laboratory Specimens Only)




| Bill of Bars | | | | | |
|---|---|---|---|---|---|
| Bar | QTY. | Size | Total Length | Material Specifications | Treatment Specifications |
| c3 | 56 | #4 | 150" | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c4 | 84 | #5 | 150" | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c5 | 182 | #5 | 53 1/4" | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c6 | 91 | #4 | 66" | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |

Midwest Roadside Safety Facility

USDA–FS–NTDP Low–Height, Glulam Timber Bridge Rail

Rebar Details

SHEET: 23 of 26
DATE: 5/8/2023
DRAWN BY: SBW/GSK
DWG. NAME. MT–USFS_R6
SCALE: 1:16
UNITS: Inches
REV. BY: RKF/KAL/RM

Figure 103. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Rebar Details

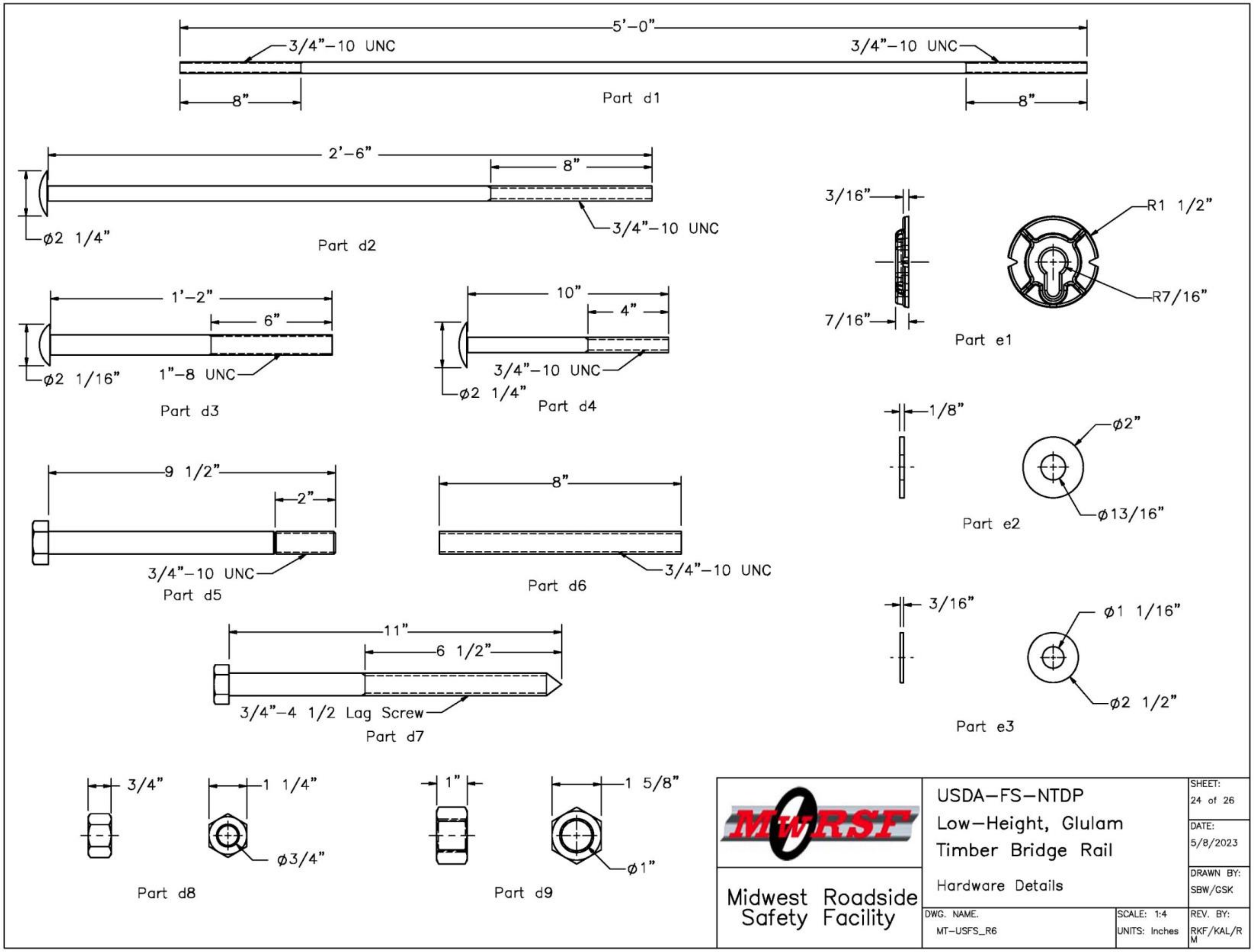



Figure 104. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Hardware Details

| BOM Table | | | | |
|---|---|---|---|---|
| Item No. | QTY. | Description | Material Specification | Treatment Specification |
| – | 1 | 2" Thick Asphalt or Concrete Wearing Surface | – | – |
| a1 | 20 | 5 1/8"x4'x12' Long, Glulam Deck Panels | Comb. No. 2 Douglas Fir | Pentachlorophenal with Heavy Oil 0.6 lbs/cu. ft Retention |
| a2 | 24 | 9 1/2"x23"x7 1/2" Scupper Block | Southern Yellow Pine Combination No. 48 | Pentachlorophenal with Heavy Oil 0.6 lbs/cu. ft Retention |
| a3 | 24 | 12"x5 1/8"x41 1/8" Long Glulam Diaphragms | Comb No. 2 Douglas Fir | Pentachlorophenal with Heavy Oil 0.6 lbs/cu. ft Retention |
| a4 | 7 | 239 1/4"x12 3/8"x6 3/4" Glulam Rail Section | Southern Yellow Pine Combination No. 48 | Pentachlorophenal with Heavy Oil 0.6 lbs/cu. ft Retention |
| a5 | 1 | 180 3/4"x12 3/8"x6 3/4" Glulam End Rail Section | Southern Yellow Pine Combination No. 48 | Pentachlorophenal with Heavy Oil 0.6 lbs/cu. ft Retention |
| a6 | 12 | 16 1/2"x6 3/4"x20' Long Outside Glulam Girder | 24F–V4 Douglas Fir | Pentachorophenal with Heavy Oil 0.6 lbs/cu. ft Retention |
| a7 | 6 | 16 1/2"x6 3/4"x20' Long Glulam Girder | 24F–V4 Douglas Fir | Pentachlorophenal with Heavy Oil 0.6 lbs/cu. ft Retention |
| b1 | 2 | Angled, 34 3/4"x6 3/4"x3/8 Splice Plate | ASTM A572 Gr. 42 | ASTM A123 |
| b2 | 12 | Horizontal, 34 3/4"x6 3/4"x3/8 Splice Plate | ASTM A572 Gr. 42 | ASTM A123 |
| b3 | 7 | 11 5/8"x6 3/4"x3/8" Splice Gusset | ASTM A572 Gr. 42 | ASTM A123 |
| b4 | 2 | Angled W6x15, 72" Long Steel Post | ASTM A36 | ASTM A123 |
| b5 | 2 | Horizontal W6x15, 72" Long Steel Post | ASTM A36 | ASTM A123 |
| b6 | 4 | 19"x8 1/2"x3/8" Post to Rail Plate | ASTM A36 | ASTM A123 |
| b7 | 36 | 12"x16"x1/2" Steel Base Plate | ASTM A36 | ASTM A123 |
| b8 | 72 | 12"x10"x1/2" Side Plate | ASTM A36 | ASTM A123 |
| b9 | 36 | 12"x6 1/4"x3/4" Elastomeric Bearing Pad | Neoprene – Min. 50 Durameter | – |

Notes:
(1) Quantities listed herein are for one bridge section.
(2) For Part No. a2 No. 1 Grade Southern Yellow Pine may also be used.
(3) Timber rails shall be treated with Pentachorophenal in Heavy Oil to a minimum retention of 0.6 lbs/cu. ft in accordance with AWPA Standards UI to the requirements use category 4A (UC4A)
(4) Wood shall be cut, drilled, and completely fabricated prior to treatment with preservative. Drain excess chemical ad dry all treated wood at location of manufacture.
(5) All field cuts, bore holes, and damages shall be treated with material acceptable to the engineer prior to installation.

Midwest Roadside Safety Facility

USDA–FS–NTDP
Low–Height, Glulam Timber Bridge Rail
Bill of Materials

SHEET: 25 of 26
DATE: 5/8/2023
DRAWN BY: SBW/GSK
DWG. NAME. MT–USFS_R6
SCALE: 1:50
UNITS: Inches
REV. BY: RKF/KAL/RM

Figure 105. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Bill of Materials

| BOM Table | | | | |
|---|---|---|---|---|
| Item No. | QTY. | Description | Material Specification | Treatment Specification |
| c1 | 2 | Abutment Concrete | Min f'c = 4,000 psi | – |
| c2 | 5 | Pier Concrete | Min f'c = 4,000 psi | – |
| c3 | 56 | #4 Rebar, 12'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c4 | 84 | #5 Rebar, 12'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c5 | 91 | #5 Rebar, 5'–6" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| c6 | 182 | #4 Bent Rebar, 53 1/4" Total Length | ASTM A615 Gr. 60 | *Epoxy Coated (ASTM A775 or A934) |
| d1 | 48 | 3/4"–10 UNC, 8" on a 60" Long Tie Rod | ASTM A307A or F1554 Gr. 36 or SAE J429 Gr. 2 | ASTM A123 or A153 or F2329 |
| d2 | 48 | 3/4"–10 UNC, 30" Long Timber Bolt | ASTM A307A | ASTM A153 or F2329 |
| d3 | 84 | 1"–8 UNC, 14" Long Timber Head Bolt | ASTM A307A | ASTM A153 or F2329 |
| d4 | 16 | 3/4"–10 UNC, 10" Long Timber Bolt | ASTM A307A | ASTM A153 or F2329 |
| d5 | 36 | 3/4"–10 UNC, 9 1/2" Long Hex Bolt | ASTM A307A | ASTM A123 or A153 or F2329 |
| d6 | 72 | 3/4"–10 UNC, 8" Long Fully Threaded Rod | ASTM A193 Gr. B7 or SAE J429 Gr. 5 | ASTM A123 or A153 or F2329 |
| d7 | 360 | 3/4"–4 1/2, 11" Long Lag Bolt | ASTM A307A | ASTM A123 or A153 or F2329 |
| d8 | 220 | 3/4"–10 UNC Heavy Hex Nut | ASTM A307A | ASTM A123 or A153 or F2329 |
| d9 | 132 | 1"–8 Heavy Hex Nut | ASTM A307A | ASTM A123 or A153 or F2329 |
| e1 | 196 | 3/4" Malleable Iron Washer | ASTM A47 | ASTM A123 or A153 or F2329 |
| e2 | 432 | 3/4" Flat Washer | ASTM F844 | ASTM A123 or A153 or F2329 |
| e3 | 84 | 1" Flat Washer | ASTM A47 | ASTM A123 or A153 or F2329 |

* rebar does not need to be epoxy–coated for testing purposes.

| | | | |
|---|---|---|---|
| Midwest Roadside Safety Facility | USDA–FS–NTDP Low–Height, Glulam Timber Bridge Rail<br>Bill of Bars | | SHEET: 26 of 26<br>DATE: 5/8/2023<br>DRAWN BY: SBW/GSK |
| | DWG. NAME. MT–USFS_R6 | SCALE: 1:50<br>UNITS: Inches | REV. BY: RKF/KAL/RM |

Figure 106. USDA-FS-NTDP Low-Height, Glulam Timber Bridge Rail: Bill of Bars

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# 10 APPENDICES

# Appendix A. Correspondence between NTDP-USDA and MwRSF

In order to design the surrogate section of glulam deck for component testing, it was important that the design drawings matched closely with the decks used by MT-USFS. To meet this objective, a questionnaire was sent to the sponsor to gain their input on design details they wanted to have implemented on the glulam deck. The questions sent to the sponsors, and the sponsor's responses have been provided below.

Also included in this section of the report is a summary of the design decisions that went into selecting connection hardware to anchor the deck panels to the deck's substructure. Typically, MT-USFS utilizes bolts and bracket plates to anchor their decks to the substructure. However, due to constructability issues, the connection hardware had to be modified. To design a suitable alternative discussion between MwRSF and NTDP-USFS occurred and has been documented below.

**<u>Questions and Clarifications</u>**

**Initial Response:**
Additional Background - Many recently-installed F.S. projects currently in-place have incorporated design aides available in FPL-GTR-125, which include features and details slightly divergent from the modern design aides of FPL-GTR-260 [19-20] referenced by the Railing Team. The older aides are found here:
https://www.fpl.fs.fed.us/documnts/fplgtr/fplgtr125.pdf

Further, the referred FPL-GTR-260 is not yet adopted by F.S. bridge programming. With the variability of modern practices and the presence of a wide array of pre-existing bridges installed according to obsolete design, which are due for barrier upgrades; it is the preferable objective for this study to consider the "weakest link" of design options and indicate the applicable-range of where the proposed barrier system would be acceptable for use based on a minimum criteria, or minimum characteristics, which accommodates the application.

1. What is the shortest and typical range of bridge span lengths that will likely utilize the MASH TL-1 glulam timber curb system?

   **Response:**
   F.S. bridges range a wide array of spans, from less than 10 ft to over 100 ft or more. F.S. barrier selection is not based on span length but on traffic volume, traffic type, speed, and alignment.

2. What is the most common glulam girder width that will be used with the transverse deck panels?

   **Response:**
   F.S. does not have a standard glulam girder width, and the design of girder members varies according to the distinct project design objectives, often ranging from approx. 6-in to nearly 12-in.

   **Comments:**
   Shorter spans utilize smaller girders. We only need a short span for testing so the relatively small girders can be used in accordance with the standard plans.

3. What is the typical connection system to attach glulam timber deck panels to glulam timber girders?
   a. Lag Screws – length and diameter
   b. Nails – Refer to Proposal Timber Bridge Drawing Set
   c. Brackets – Refer to FPL-GTR-260 Figure 3.37

   **Response:**
   F.S. has not adopted a standard connection method, and the design of connections is variable according to the distinct project design objectives. Either connection method may be incorporated within the range of F.S. projects. When used, lag screws are designed according to *National Design Specifications* from American Wood Council.

**Comments:**
As an initial plan, we decided to proceed using the brackets as there are details for the brackets in the standard plans documents, and MwRSF has used them in the construction of similar decks in the past. Upon analysis and in an effort to minimize construction costs, we realized the brackets would require more excavation and that lag screws could potentially solve this issue. An analysis was done to compare the strength of the two connection types and will be discussed in a future meeting.

4. Is it common to utilize shear transfer hardware between adjacent transverse, glulam timber deck panels within the exterior cantilevered region in order to better distribute vertical/lateral impact loading?
    a. Researchers may need to consider providing comparable behavior and strength between nail-laminated and glued-laminated panels.

**Response:**
F.S. has not adopted a standard connection method for exterior cantilevered regions. The exterior region is connected similarly to the interior regions of panels for the particular project.

5. Is it common to utilize longitudinal stiffening beams between girders? Are these beams used for vertical shear transfer between panels?
    a. These beams are shown between girders, but not beyond the outer girder. Is this typical? Refer to FPL-GTR-260 Figure 3.30

**Response:**
F.S. has not adopted a standard for using stiffener beams, and the design of stiffeners is variable according to the distinct project design objectives. When used, stiffener beams provide continuity of vertical shear between panels; alternatively, some installations include dowel inserts along the mid-height of each panel interface, and others do not include inter-panel connection. When used, stiffener beams are generally within the interior bays of girders, not along the cantilevered portion of panels.

**Comments:**
Because these stiffening beams and other shear transfer devices are not used on *all* bridges, for worst-case scenario testing, our surrogate bridge will not utilize such components.

6. What is the typical range for deck overhang distance beyond the exterior glulam girder as measured the between centerline of the glulam girder to the edge of the deck?
    a. WVDOT MASH 2009 TL-1 Curb Railing System utilized 4 ft – 2 in. to the center of steel girder
    b. Proposal Timber Bridge Drawing Set shows 2 ft - 3 in. to the center of the glulam girder
    c. FPL-GTR-260 shows 1 ft - 11½ in. for a multilane bridge or 1 ft – 6 in. for single lane bridge (Figures 3.16 and 3.18), both to the center of glulam girder

**Response:**
F.S. has not adopted standard criteria for deck overhang, and the design of overhang is variable according to the distinct project design objectives, influenced by travel width and girder- size, count, and spacing, often resulting in approx. 2-ft overhang; while the FPL-GTR-260 has overhang up to 3-ft.

7. When configuring a surrogate test bridge for the component testing program, is it acceptable to use FPL-GTR-260 *Standard Plans for Glued-Laminated Timber Bridge Superstructures* to estimate girder spacings?
    a. FPL-GTR-260 shows girders at 4 ft – 5 in. for multilane bridges or 3 ft – 8 in. for single-lane bridges (Figures 3.16 and 3.18)
    b. Proposal Timber Bridge Drawing Set shows girders at 3 ft – 10 in.

**Response:**
F.S. single-lane bridges are generally 14 ft inside-barrier-face (16-foot outer width) and assuming 4-girder lines, then a spacing of approx. 4 ft is a suitable simplifying assumption.

8. What range of bridge/roadway widths, measured from barrier face to barrier face, should be considered for the MASH TL-1 glulam timber curb systems?
    a. A panel width of 4 ft (measured in the direction of traffic) is assumed.

**Response:**
F.S. bridges are generally 14 ft for single-lane bridges and 24 ft for double-lane, inside barrier faces; 4 ft panel width in the direction of travel is an acceptable assumption.

**Comments:**
As this is a surrogate bridge, we will utilize a layout that represents a portion of the roadway width, providing adequate decking to run the full testing program.

9. For testing purposes, researchers will use a surrogate 2-in. thick, wearing service of either concrete, asphalt, or timber planking material? Is this selection acceptable?

**Response:**
A surrogate wearing surface thickness of 2 in. is acceptable.

**Comments:**
In the standard plans document FPL-GTR-125, the wearing surface is shown to be 3” thick at the center of the roadway with a minimum thickness of 1.5” at the face of the barrier. Discussion is needed to clarify if a 3” overlay needs to be considered, as this affects the effective height of the barrier and its vehicle redirecting capacity.

**Deck Panel to Substructure Connection Design**

In order to design the surrogate section of the glulam deck for this project, the guidance outlined in the MT USFS standard plans was followed closely. However, in certain instances, it was discovered that following the exact guidance of these standard plans would lead to complications in the surrogate deck design. One of the instances in which following the MT-USFS design guides was not practical occurred when designing the anchorage system needed to attach the glulam deck panels to the glulam girders. In the standard plans, bracket plates, and ⅝-in. bolts are typically used to attach the panels to the girders. However, after discussion with the design team, it was concluded that installing the bracket plates would pose significant constructability issues, leading to high construction costs. As a potential solution to the constructability concerns of the typical connection design used, an alternative lag screw connection type was investigated. In this alternate design, the lag screws were to be drilled into the glulam deck panels, and then attached to the girders supporting the deck. In order to confirm that lag screws could be used as a replacement option, the relative strength of the lag screw connection to the original bracket plate and bolt connection was analyzed. If the lag screw connection resulted in a stronger connection than the bracket plate and bolt connection, the component tests to be run on the glulam deck could potentially yield results overestimating the capabilities of the low-height bridge rail when installed on the glulam deck. Therefore, a brief investigation into the relative strengths of each connection type was performed. Results documenting this investigation are provided below.

The tensile and shear capacities of ⅝-in. ASTM A307 bolts used in the bolt and bracket plate connection type were retrieved from Tables 7-1 and 7-2 in the 15th Edition of the AISC *Steel Construction Manual* [22] and checked with hand calculations. Bracket tear-out strength at the bolt hole and shear strength at the vertical plane are shown by the solid blue line and denoted A-A in Figure A-1, and were also analyzed for strength. The brackets were assumed to be cast of aluminum alloy 356 with a yield stress, $F_y$ of 24 ksi, and an ultimate stress, $F_u$ of 33 ksi, as is standard per FPL-GTR-260 [20].

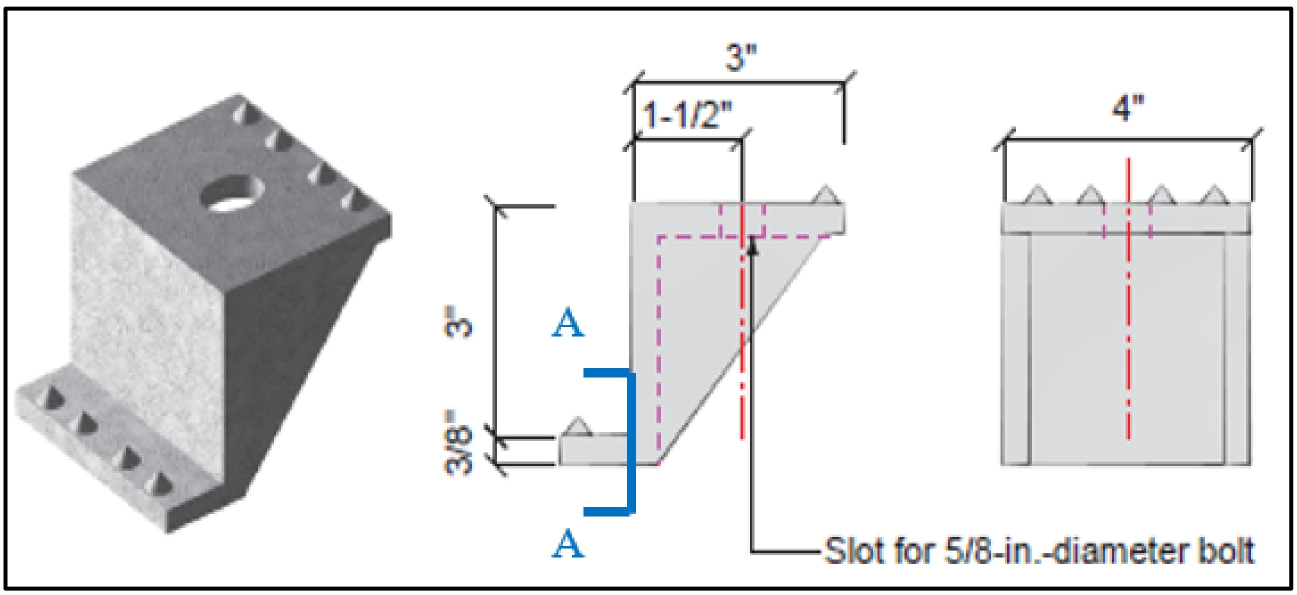



Figure A-1. Bracket Plate Used to Connect Deck Panels to Girder [20]

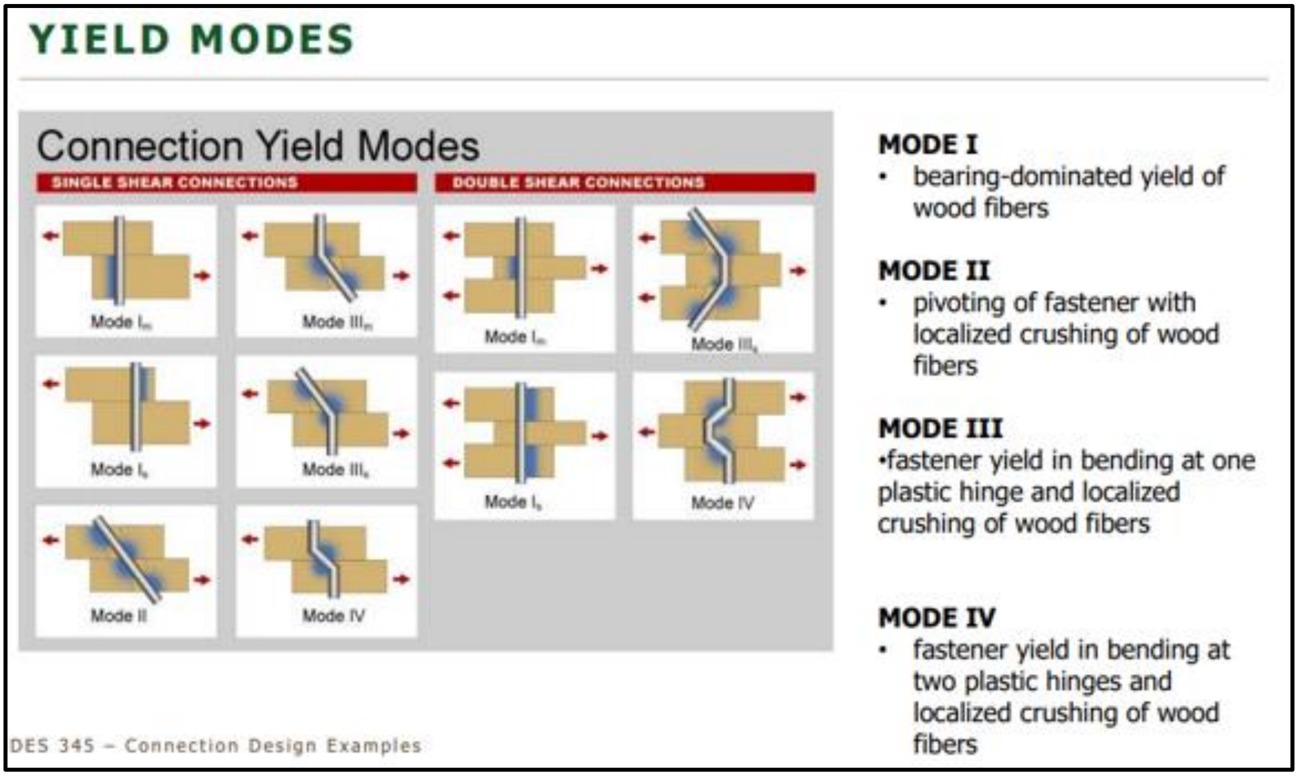



Figure A-2. Single Bolt Shear Failure Modes

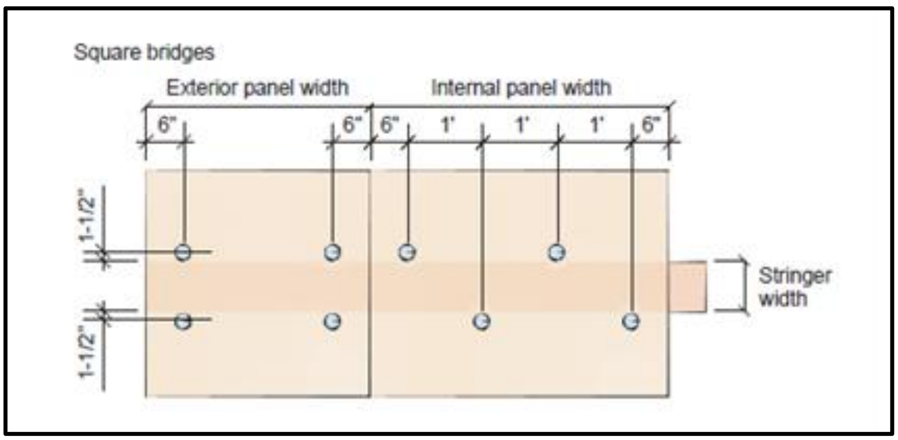
The tensile and shear capacities of ¾-in. ASTM A307 lag screws were determined by hand calculations using the cross-sectional area of the screws at the root diameter. Lag screw dimensions were retrieved from Table L2 of the 2018 NDS, assuming full-body diameter lag screws. The withdrawal and lateral strengths of a single screw were determined per the 2018 NDS, assuming Douglas Fir-Larch wood with a specific gravity, G, of 0.50, found in Table 12.3.3A. The lag screw's tensile and shear capacities and the screw's lateral failure and withdrawal limit states were determined utilizing these design guides. Each of the six failure modes shown in Figure A-2 was calculated based on the equations outlined in the 2018 NDS to determine lateral strength. It should be noted that in this figure, both single and double-shear failure modes are shown. For the connection type being utilized for this project, the single shear failure modes were the only failure types analyzed. Based on the analysis done, it was determined that failure mode IV controlled the design. The strength values associated with the lag screw have been tabulated in Table A-1. The capacities for the bolt and bracket plate connection were also calculated. These capacities include the bolt tensile and shear capacity and the horizontal and vertical bracket tear-out capacity. The capacities for this connection type have also been summarized in Table A-1.

Table A-1. Summary of Connection Hardware Strengths

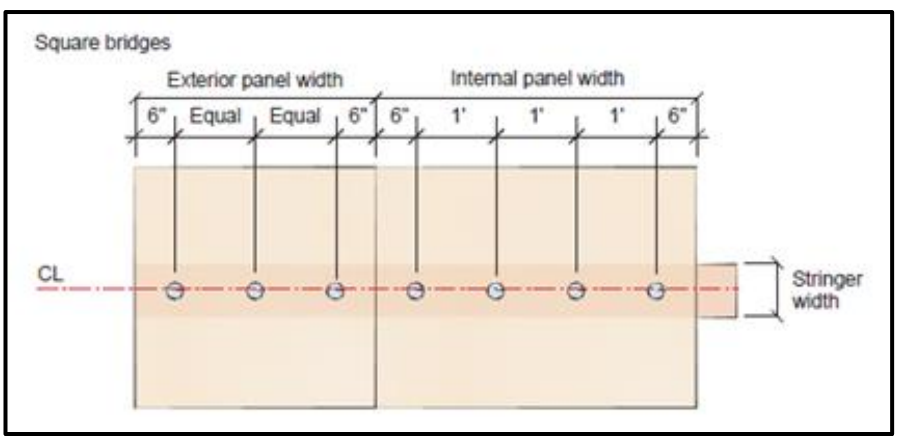
| ¾-in. Lag Screw Limit States(1) | Value | ⅝-in. Bolt and Bracket Plate Limit States | Value |
|---|---|---|---|
| Tensile Capacity (kip/screw) | 11.8 | Bolt Tensile Capacity (kips/bolt) | 10.4 |
| Shear Capacity (kip/screw) | 7.11 | Bolt Shear Capacity (kips/bolt) | 6.23 |
| Lateral Failure (kip/screw) | 2.37 | Horizontal Bracket Tearout (kips/bracket) | 15.7 |
| Withdrawal (kip/screw) | 7.61 | Vertical Bracket Shear (kips/bracket) | 22.3 |

(1) ASTM A307 Bolts and Lag Screws

After determining the capacities of individual bolts, lag screws, and bracket plates, the deck-to-girder connections were analyzed to determine the critical connectors contributing to resisting the loads from vehicle impacts. Figure A-3(a) provides the bolt-hole layout used when implementing a combination of bolts and bracket plates onto a glulam panel. Figure A-3(b) provides the bolt-hole layout necessary when using lag screws only.




(a) Bolt and Bracket Plate Connection Layout [20]




(b) Lag Screw Connection Layout [20]

Figure A-3. Connection Hardware Layout on Glulam Deck Panels

The connection design shown in Figure A-3(a) was analyzed first. This analysis concluded that on a single panel along each girder, two bolts and bracket plates would resist horizontal movement, and four bolts and bracket plates would resist vertical movement. After analyzing the scenario shown in Figure A-3(b), it was determined that the four lag screws used to anchor the panel to the girder would contribute to resisting vertical and horizontal movement from a vehicle impact. Upon determining which lag screws and bracket plates would contribute towards vertical and horizontal resistance, the strengths of the two connection types towards resisting vehicle impacts were determined. Their results were tabulated in Table A-2 and Table A-3. In addition to documenting the amount of strength resisted vertically and horizontally by the bolt and bracket plate connection type and the lag screw connection type, the number of critical connectors and the controlling limit state of each connection type towards vertical and horizontal loading are shown as well. Upon comparison of the results, it can be observed that the vertical and horizontal strengths of the deck are weaker when utilizing a lag screw connection over a bolt and bracket plate connection. Therefore, it was concluded that utilizing a lag screw connection for component testing would result in a conservative estimate of the performance of the low-height timber bridge rail when mounted on a glulam bridge rail.

Table A-2. Comparison of Total Vertical Strength between Connection Hardware

| Vertical Strength | | | |
|---|---|---|---|
| Connector | Number of Critical Connectors | Total Connection Strength (kips) | Controlling Limit State |
| Bolt and Bracket Plate | 4 | 41.6 | Bolt Tensile |
| Lag Screw | 4 | 30.4 | Screw Withdrawal |

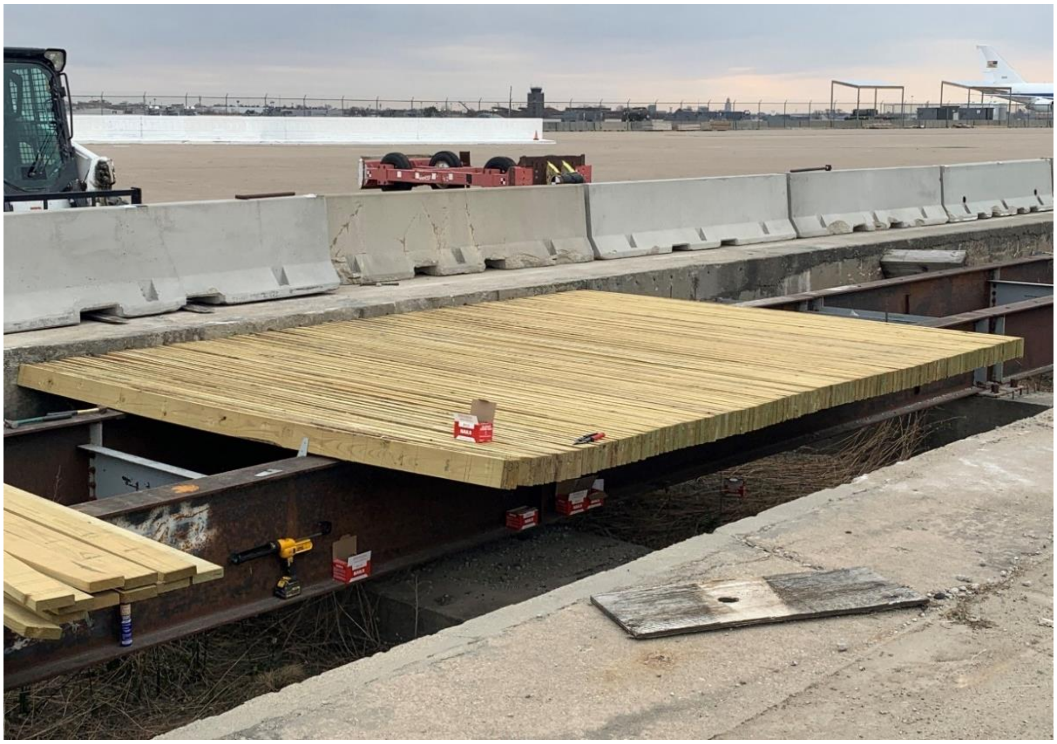
Table A-3. Comparison of Total Lateral Strength between Connection Hardware

| Horizontal Strength | | | |
|---|---|---|---|
| Connector | Number of Critical Connectors | Total Connection Strength (kips) | Controlling Limit State |
| Bolt and Bracket Plate | 2 | 12.5 | Bolt Shear |
| Lag Screw | 4 | 9.5 | Screw Lateral Failure |

# Appendix B. Nail-Laminated Deck Design Modifications

The boards were nailed together, starting at the south end of the deck and proceeding north to begin constructing the nail-laminated deck.. After approximately 19 ft of the deck had been assembled, the construction team observed that the boards on the north end, where new boards were being added, were warping significantly, and leaning onto one another, as shown in Figure B-1. As a result, the anchor brackets used to attach the deck to wide flange steel girders could not be installed in a manner that would satisfactorily secure the deck to the substructure. After the construction team discovered the deck warping, the techniques used to build and secure the deck to the girders were modified. By the time these changes to the construction methods were implemented, the affected north-end region of the deck was too far out of tolerance to continue construction.

With 19 ft of the deck already constructed, an additional length of 7 ft was required to obtain a total length of 26 ft of nail-laminated deck span. Consequently, additional boards were attached to what was designed to be the south deck edge of the nail-laminated system. However, in an attempt to minimize the use of the north end of the deck, an additional 5 ft was added to the south end, allowing for the location of the posts on the deck to be shifted away from the north end by 5 ft. The addition of 5 ft at the south end of the deck resulted in a final deck span of 31 ft. Additionally, construction was halted 7 ft short of the originally planned north deck end, and the placement of the deck system relative to the bents and abutments supporting the deck was also shifted. Figure B-2 shows a plan view of the changes to the deck span length and the change in the deck's location relative to the bents, over which the deck was to be centered.



(a) Warping of Nail-Laminated Deck Off of the Girders


(b) Leaning of Boards on Nail-Laminated Deck

Figure B-1. Warping and Leaning of Boards during Construction of Nail-Laminated Deck

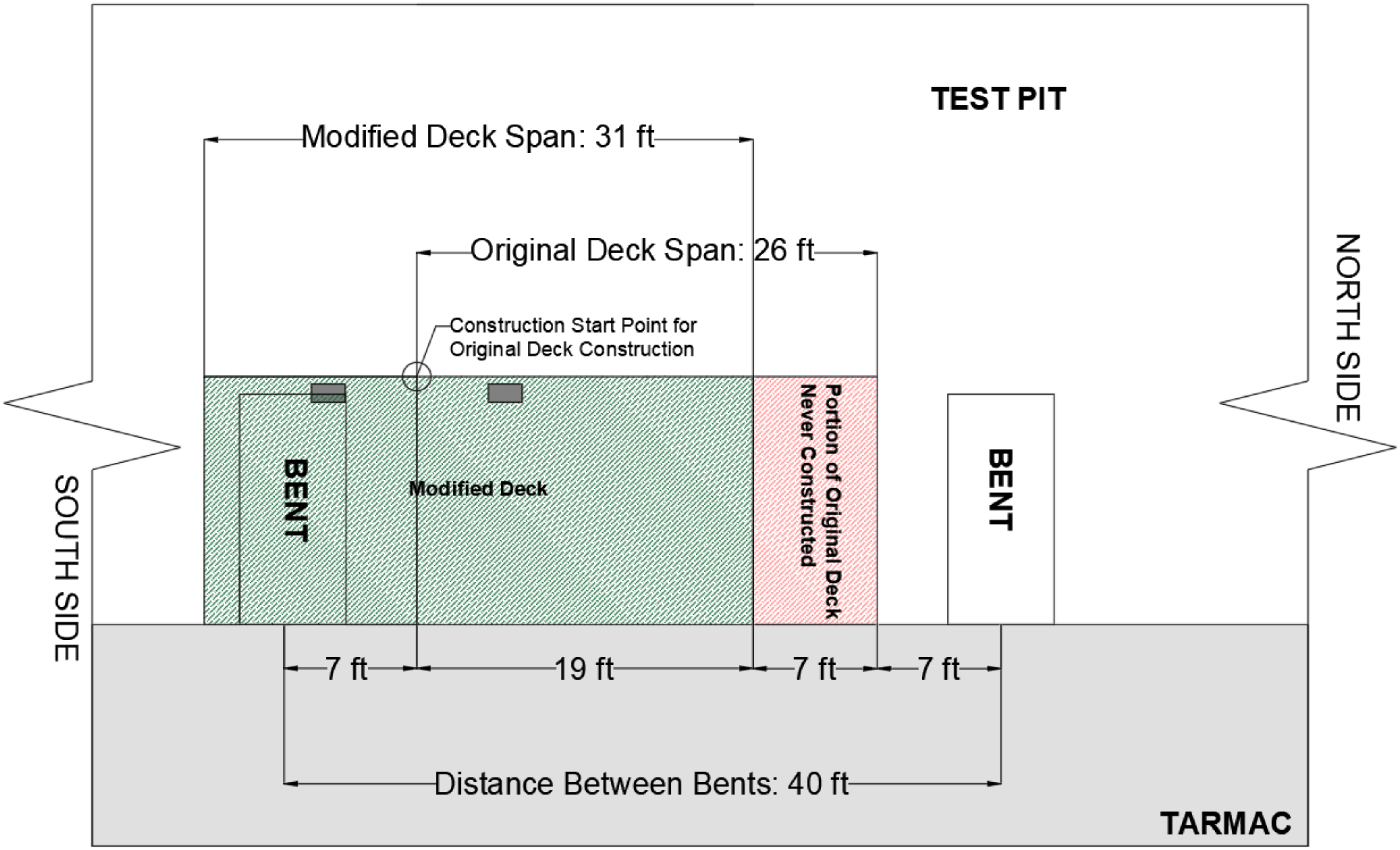



Figure B-2. Plan View Sketch of Modifications to Nail-Laminated Deck Length and Location

# Appendix C. Glulam Deck and Rail Material Specifications

Table C-1. Bill of Materials, Glulam Deck and Rail, Test Nos. MGTD-1S and MGTD-1D

| Item No. | Description | Material Specification | Reference |
|---|---|---|---|
| a1 | 9½"x23"x7½" Scupper Block | Grade No. 1 Southern Yellow Pine or Douglas Fir | Bell Lumber R#153279 |
| a2 | 12⅜"x23"x6¾" Glulam Segment | Southern Yellow Pine Combination No. 48 | Bell Lumber R#153279 |
| a3 | 23"x6¾"x¾" Static Test Plate | ASTM A36 | H#RS3132, H#RS3610, H#RS3781, H#RS3825 |
| a4 | 12⅜"x23"x6¾" Glulam Segment | Southern Yellow Pine Combination No. 48 | Bell Lumber R#153279 |
| b1 | 12"x16"x½" Steel Base Plate | ASTM A36 | H#A1A281 |
| b2 | 12"x10"x½" Side Plate | ASTM A36 | H#A1A281 |
| b3 | 12"x6¼"x¾" Elastomeric Bearing Pad | Neoprene - Min. 50 Durameter | McMaster Carr 1370N412 PO#E000869475 |
| b4 | 15"x30"x12' Concrete Support | Min f'c = 4,000 psi NE mix 47 BD | Ticket #1275083, Benesch Project #00110546.00 |
| b5 | #4 Rebar, 138" Long | ASTM A615 Gr. 60 | H#3600014740 |
| b6 | #4 Rebar, 61⅞" Unbent Length | ASTM A615 Gr. 60 | H#3600014740 |
| c1 | 16½"x6¾"x20' Long Outside Glulam Girder | 24F-V4 Douglas Fir | Bell Lumber R#153279 |
| c2 | 16½"x6¾"x20' Long Glulam Girder | 24F-V4 Douglas Fir | Bell Lumber R#153279 |
| c3 | 12"x5⅛"x41⅛" Long Glulam Diaphragms | Comb. No. 2 Douglas Fir | Bell Lumber R#153279 |
| c4 | 5⅛"x4'x12' Long Glulam Deck Panel | Comb. No. 2 Douglas Fir | Bell Lumber R#153279 |
| c5 | 5⅛"x4'x12' Long Glulam Deck Panel | Comb. No. 2 Douglas Fir | Bell Lumber R#153279 |
| d1 | ⅞"-9 UNC x 15" Heavy Hex Bolt | ASTM F3125 Gr. A325 | H#3093334 |
| d2 | ¾"-10 UNC x 30" Timber Bolt w/Nubs | ASTM A307A | Portland Bolt Order#142079 H#1202025843 |
| d3 | ¾"-10 UNC x 9½" Hex Bolt | ASTM A307A | COC P#91975 C#120306283 |
| d4 | ¾"-10 UNC x 8" on a 60" Long Tie Rod | ASTM A307A or F1554 Gr. 36 or SAE J429 Gr. 2 | H#1202027708 |
| d5 | 3/4"-10 UNC x 8" Threaded Rod | ASTM A193 Gr. B7 or SAE J429 Gr. 5 | H#18B701615 P#0186717 C#935935-1 |

Table C-2. Bill of Materials, Glulam Deck and Rail, Cont., Test Nos. MGTD-1S and MGTD-1D

| Item No. | Description | Material Specification | Reference |
|---|---|---|---|
| d6 | ¾"-4½ x 11" Lag Bolt | ASTM A307A | COC P#22492 |
| d7 | ⅞" Flat Washer | ASTM F844 | P#33187 C#170089822 L#1844804 Red Paint |
| d8 | ¾" Malleable Iron Washer | ASTM A47 | H#2019112802 P#0128540 |
| d9 | ¾" Flat Washer | ASTM F844 | L#2008905 P#1133186 C#210220089 |
| d10 | ⅞"-9 UNC Eye Nut | ASTM A325 | 3274T51 McMaster Carr |
| d11 | ¾"-10 UNC Heavy Hex Nut | ASTM A536A | H#B19120832 P#36716 C#180198094 |
| e1 | Epoxy Adhesive | Hilti HIT RE-500 V3 | COC |

BILL OF LADING - SHORT FORM - NOT NEGOTIABLE

Page 2 of 2

**BELL LUMBER & POLE COMPANY**

**153279**

P. O. Box 120736 New Brighton, MN 55112
Yard Phone: 651-633-4334 Yard Fax: 651-633-8852

Date: 1/4/2022

DESCRIPTION OF ARTICLES AND SPECIAL MARKS Received By:

**General Delivery 778 1st St NW New Brighton, MN 55112**

| Qty | Product | Framing | PO Number | Other PO | Item Number | Deck |
|---|---|---|---|---|---|---|
| 2 | PEL-6x12-1.8125 | NA | | | | |
| 2 | PEL-6x12-1.8125 | NA | | | | |
| 10 | PEL-7x9-1.8125 | NA | | | | |
| 4 | EL-5x12-3.5 | NA | | | | |
| 3 | EL-5x48-12 | NA | | | | |
| 2 | EL-5x48-12 | NA | | | | |
| 2 | EL-6x16-20 | NA | | | | |
| 1 | EL-6x16-20 | NA | | | | |
| 26 | Weight: 6981 # | | | | | |

**Wood Utility Poles**

Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of the property as follows: "The agreed or declared value of the property is specifically stated by the shipper to be not exceeding ____________ per ____________.

COD Amount: $

Fee terms: Collect ☐ Prepaid ☐ Customer check acceptable ☐

**Note: Liability limitation for loss or damage in this shipment may be applicable. See 49 USC § 14706(c)(1)(A) and (B).**

Received, subject to individually determined rates or contracts that have been agreed upon in writing between the carrier and shipper, if applicable, otherwise to the rates, classifications, and rules that have been established by the carrier and are available to the shipper, on request, and to all applicable state and federal regulations

The carrier shall not make delivery of this shipment without payment of charges and all other lawful fees.

Shipper Signature

SHIPPER Bell Lumber Pole Company

PER

Print Date 1/4/2022 Date Actual

**Trailer Loaded:**
☐ By shipper
☐ By driver

Internal Yard Doc:
NB-11496

**Freight Counted:**
☐ By shipper
☐ By driver

**Carrier Signature/ Pickup Date**

Carrier acknowledges receipt of packages and required placards. Carrier certifies emergency response information was made available and/or carrier has the DOT emergency response guidebook or equivalent documentation in the vehicle. Property described above is received in good order, except as noted.

Copies: White & Canary - Office Pink - Customer Goldenrod - Carrier

Figure C-1. Scupper Block, Glulam Rail, and Glulam Deck, Test Nos. MGTD-1S and MGTD-1D (Item Nos. a1, a2, a4, c1, c2, c3, c4, and c5)

**EVRAZ** | EVRAZ INC. NA
Evraz Oregon Steel 14400 N. Rivergate Blvd., Portland, Oregon 97203

**REPORT OF CHEMICAL/PHYSICAL TESTS**

| CERTIFICATE NO. | DATE | PAGE |
|---|---|---|
| 1769748 | Mar 26, 2021 | 1 |

| MILL ORDER NO. | DATE |
|---|---|
| 362697 | |

CUSTOMER ORDER NO.: 42834
JOB/REQ. NO.:

| SHIPPING NO. | DATE |
|---|---|
| 1769748 | 03/26/2021 |

CARRIER: BURLINGTON NORTHERN
CAR/TRUCK NO.: AOK607031

ISO 9001 REGISTERED U.S. PLATE PRODUCER

SOLD TO:
TRINITY PRODUCTS LLC
1969 W TERRA LANE
O FALLON, MO 63366
USA

TRINITY PRODUCTS LLC
425 E. 151ST STREET
EAST CHICAGO, IN 46312
USA

THIS MATERIAL HAS BEEN MANUFACTURED, TESTED AND FOUND TO MEET THE SPECIFICATIONS AND PURCHASE ORDER REQUIREMENTS
CARBON STRUCTURAL QUALITY PLATE ASTM A36-19/ASME SA36 2019 ASTM A709-18 GRADE 36. KILLED FINE GRAIN PRACTICE.

**PHYSICAL PROPERTIES**

| ITEM NO | DESCRIPTION | | HEAT NO. | SLAB | YIELD PSI X 100 | TENSILE PSI X 100 | % ELONG 8" | % ELONG 2" | % RA | HARDNESS BHN | BEND TEST | IMPACTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.7500 X 72.000ME X 240.000<br>PT# PL.050.072.240.A36 | | | | | | | | | | | |
| | 1 PC 3675 LBS | +! | RS3132 | | 545 | 655 | 32 | | | | | |
| | 2 PCS 7350 LBS | +! | RS3610 | | 625 | 710 | 28 | | | | | |
| | | | | | 675 | 735 | 21 | | | | | |
| | 4 PCS 14700 LBS | +! | RS3781 | | 660 | 795 | 24 | | | | | |
| | 5 PCS 18375 LBS | +! | RS3825 | | 550 | 675 | 26 | | | | | |
| | | | | | 515 | 680 | 29 | | | | | |

**CHEMICAL ANALYSIS**

| | HEAT NO. | C | Mn | P | S | Si | Cu | Ni | V | Cb | Al | Cr | Mo | Ti | B | N | Ca | | | CE | McQuaid Ehn Grain Size |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| +! | RS3132 | .05 | .83 | .011 | .007 | .25 | .30 | .10 | .002 | .023 | .023 | .08 | .04 | | | | | | | .24 | |
| +! | RS3610 | .05 | 1.25 | .009 | .009 | .23 | .27 | .09 | .004 | .035 | .044 | .08 | .04 | | | | | | | .31 | |
| +! | RS3781 | .06 | 1.23 | .012 | .003 | .20 | .27 | .10 | .003 | .059 | .045 | .12 | .09 | | | | | | | .33 | |
| +! | RS3825 | .05 | .83 | .011 | .001 | .25 | .32 | .11 | .003 | .023 | .043 | .10 | .03 | | | | | | | .24 | |

HEATS INDICATED WITH (!) WERE MELTED & POURED IN THE CANADA.
HEATS INDICATED WITH (+) WERE ROLLED IN THE USA.

I certify the above to be correct as contained in the records of EVRAZ INC. NA By [signature] Chemist

Figure C-2. 23-in. x 6¾-in. x ¾-in. Static Test Plate, Test No. MGTD-1S (Item No. a3)

SSAB

1770 Bill Sharp Boulevard, Muscatine, IA 52761-9412, US

# Test Certificate

⚠ **WARNING:** This product can expose you to chemicals including nickel and nickel compounds, which are known to the State of California to cause cancer. For more information go to www.P65Warnings.ca.gov.

Form TC1: Revision 4: Date 6 Feb 2019

**Customer:**
STEEL & PIPE SUPPLY
P.O. BOX 1688

MANHATTAN
KS 66502

**Customer P.O.No.:** 4500358483
**Mill Order No.** 41-628789-01
**Shipping Manifest:** MT427225

**Product Description:** ASTM A36(19)/A709(18)36/ASME SA36(19)
AASHTO M270(20)36, 0.80-1.20 MN

**Ship Date:** 19 Feb 21
**Cert Date:** 19 Feb 21
**Cert No:** 061886005
(Page 1 of 1)

**Size:** 0.500 X 96.00 X 240.0 (IN)

| Tested Pieces: | | | | Tensiles: | | | | | | Charpy Impact Tests | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Heat Id | Piece Id | Tested Thickness | Tst Loc | YS (KSI) | UTS (KSI) | %RA | Elong % 2in 8in | Tst Dir | Hardness | Abs. Energy(FTLB) 1 2 3 Avg | % Shear 1 2 3 Avg | Tst Tmp | Tst Dir | Tst Siz (mm) | BDWTT Tmp %Shr |
| A1A281 | B58 | 0.310 (DISCRT) | L | 50 | 68 | | 37 | T | | | | | | | |
| A1A281 | B59 | 0.497 (DISCRT) | L | 46 | 67 | | 37 | T | | | | | | | |

**Chemical Analysis**

| Heat Id | C | Mn | P | S | Si | Tot Al | Cu | Ni | Cr | Mo | Cb | V | Ti | B | N | IIW | ORGN |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A1A281 | .16 | .84 | .009 | .002 | .04 | .030 | .33 | .11 | .14 | .02 | .001 | .003 | .006 | .0001 | .0073 | .36 | USA |

```
KILLED STEEL
MERCURY IS NOT A METALLURGICAL COMPONENT OF THE STEEL AND NO MERCURY WAS INTENTIONALLY ADDED DURING THE MANUFACTURE
OF THIS PRODUCT.
CEV (IIW)  = C + MN/6 + (CR+MO+V)/5 + (NI+CU)/15
MTR EN 10204:2004 INSPECTION CERTIFICATE 3.1 COMPLIANT
100% MELTED, POURED, AND ROLLED IN THE USA
PRODUCTS SHIPPED:
  A1A281          B56          PCES:   6, LBS:   19602
```

(P) Cust Part #: 721696240

WE HEREBY CERTIFY THAT THIS MATERIAL WAS TESTED IN ACCORDANCE WITH, AND MEETS THE REQUIREMENTS OF, THE APPROPRIATE SPECIFICATION

Brian Wales
SENIOR METALLURGIST - PRODUCT

Figure C-3. Steel Base Plates and Side Plates, Test Nos. MGTD-1S and MGTD-1D (Item Nos. b1 and b2)

McMASTER-CARR.

# Packing List

600 N County Line Rd
Elmhurst IL 60126-2081
630-600-3600
chi.sales@mcmaster.com

University of Nebraska
Midwest Roadside Safety Facility
M W R S F
4630 Nw 36TH St
Lincoln NE 68524-1802
Attention: Shaun M Tighe
Test Site

Purchase Order
**E000869475**

Order Placed By
**Shaun M Tighe**

McMaster-Carr Number
**7470319-01**

Page 1 of 1

05/25/2021

| Line | | Product | Ordered | Shipped |
|---|---|---|---|---|
| 1 | **1370N412** | Multipurpose Neoprene Rubber Sheet with Certificate, 12" x 12", 3/4" Thick, 50A Durometer | **6 Each** | **6** |

Certificate of compliance

This is to certify that the above items were supplied in accordance with the description and as illustrated in the catalog. Your order is subject only to our terms and conditions, available at www.mcmaster.com or from our Sales Department.

Sarah Weinberg
Compliance Manager

WARCO BILTRITE
America's choice for quality rubber.™
SHEET | EXTRUSION | MOLDED | MATTING | CUSTOM
NSF a NSF Certified ISO 9001:2008 Company

1337 W. Braden Court
Orange, CA 92868
Tel: 714-532-3355
Fax: 714-532-2238

## Certification

| Product Description | Material Description |
|---|---|
| 050E1443 .750x12.000x12.000 PLAIN BACK | ASTM D 2000 M1BC507 |

### Specifications / Basic Physical Requirements

| | |
|---|---|
| Durometer | 50 +/- 5 |
| Tensile Strength (min psi) | 1000 psi |
| Temperature Range | -30 to 200 |
| Ultimate Elongation (min %) | 300 |

*The above values are not actuals

### Batch Information

| | |
|---|---|
| Batch / Lot # | 1237403 |
| Cure Date | 2Q21 |
| MFG # | 1058758 |

This is to certify that all materials on this order conform to all purchase order requirements.

David Yuong
Technical Services

Form WXM0115 Rev B Date: 2/18/19

98

Figure C-4. 12-in. x 6¼-in. x ¾-in. Elastomeric Bearing Pad, Test Nos. MGTD-1S and MGTD-1D (Item No. b3)

**Ready Mixed Concrete Company**
6200 Cornhusker Hwy, Lincoln, NE 68529
Phone: (402) 434-1844 Fax: (402) 434-1877

Customer's Signature: ____________

| PLANT | TRUCK | DRIVER | CUSTOMER | PROJECT | TAX | PO NUMBER | DATE | TIME | TICKET |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 184 | 8508 | 62461 | | NTE | | 2/18/22 | 3:07 PM | 1275083 |

| Customer | Delivery Address | Special Instructions |
|---|---|---|
| UNL-MIDWEST ROADSIDE SAFETY | 4630 NW 36TH ST | NW 38TH ST & W CUMING ST & EAST TO NW 36TH ST & SOUTH |

| LOAD QUANTITY | CUMULATIVE QUANTITY | ORDERED QUANTITY | PRODUCT CODE | PRODUCT DESCRIPTION | UOM | UNIT PRICE | EXTENDED PRICE |
|---|---|---|---|---|---|---|---|
| 4.00 | 4.00 | 4.00 | QL324504 | LNK47B1PF4000HW | yd | $148.50 | $594.00 |
| | | | | MINIMUM HAUL | | | $30.00 |
| | | | | WINTER SERVICE | | | $24.00 |

| Water Added On Job At Customer's Request: | SLUMP | Notes: | | |
|---|---|---|---|---|
| | 4.00 in | | TICKET SUBTOTAL | $648.00 |
| | | | SALES TAX | $0.00 |
| | | | TICKET TOTAL | $648.00 |
| | | | PREVIOUS TOTAL | |
| | | | GRAND TOTAL | $648.00 |

**CAUTION FRESH CONCRETE**
**KEEP CHILDREN AWAY**

Contains Portland cement. Freshly mixed cement, mortar, concrete or grout may cause skin injury. Avoid prolonged contact with skin. Always wear appropriate Personal Protective Equipment (PPE). In case of contact with eyes or skin, flush thoroughly with water. If irritation persists, seek medical attention promptly.

**Terms & Conditions**

This concrete is produced with the ASTM standard specifications for ready mix concrete. Strengths are based on a 3" slump. Drivers are not permitted to add water to the mix to exceed this slump, except under the authorization of the customer and their acceptance of any decrease in compressive strength and any risk of loss as a result thereof. Cylinder tests must be handled according to ACI/ASTM specifications and drawn by a licensed testing lab and/or certified technician.
Ready Mixed Concrete Company will not deliver any product beyond any curb lines unless expressly told to do so by customer and customer assumes all liability for any personal or property damage that may occur as a result of any such directive.
The purchaser's exceptions and claims shall be deemed waived unless made in writing within 3 days from time of delivery. In such a case, seller shall be given full opportunity to investigate any such claim. Seller's liability shall in no event exceed the purchase price of the materials against which any claims are made.

Figure C-5. Concrete Support, Test Nos. MGTD-1S and MGTD-1D (Item No. b4)

**Ready Mixed Concrete Company**
6200 Cornhusker Hwy, Lincoln, NE 68529
Phone: (402) 434-1844 Fax: (402) 434-1877

Customer's Signature: ________________

| PLANT | TRUCK | DRIVER | CUSTOMER | PROJECT | TAX | PO NUMBER | DATE | TIME | TICKET |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 184 | 8508 | 62461 | | NTE | | 2/18/22 | 3:07 PM | 1275083 |

| Customer | Delivery Address | Special Instructions |
|---|---|---|
| UNL-MIDWEST ROADSIDE SAFETY | 4630 NW 36TH ST | NW 38TH ST & W CUMING ST & EAST TO NW 36TH ST & SOUTH |

| LOAD QUANTITY | CUMULATIVE QUANTITY | ORDERED QUANTITY | PRODUCT CODE | PRODUCT DESCRIPTION | UOM | UNIT PRICE | EXTENDED PRICE |
|---|---|---|---|---|---|---|---|
| 4.00 | 4.00 | 4.00 | QL324504 | LNK47B1PF4000HW | yd | $148.50 | $594.00 |
| | | | | MINIMUM HAUL | | | $30.00 |
| | | | | WINTER SERVICE | | | $24.00 |

| Water Added On Job At Customer's Request: | SLUMP | Notes: | | |
|---|---|---|---|---|
| | 4.00 in | | TICKET SUBTOTAL | $648.00 |
| | | | SALES TAX | $0.00 |
| | | | TICKET TOTAL | **$648.00** |
| | | | PREVIOUS TOTAL | |
| | | | GRAND TOTAL | $648.00 |

**CAUTION FRESH CONCRETE**
**KEEP CHILDREN AWAY**

Contains Portland cement. Freshly mixed cement, mortar, concrete or grout may cause skin injury. Avoid prolonged contact with skin. Always wear appropriate Personal Protective Equipment (PPE). In case of contact with eyes or skin, flush thoroughly with water. If irritation persists, seek medical attention promptly.

**Terms & Conditions**

This concrete is produced with the ASTM standard specifications for ready mix concrete. Strengths are based on a 3" slump. Drivers are not permitted to add water to the mix to exceed this slump, except under the authorization of the customer and their acceptance of any decrease in compressive strength and any risk of loss as a result thereof. Cylinder tests must be handled according to ACI/ASTM specifications and drawn by a licensed testing lab and/or certified technician.
Ready Mixed Concrete Company will not deliver any product beyond any curb lines unless expressly told to do so by customer and customer assumes all liability for any personal or property damage that may occur as a result of any such directive.
The purchaser's exceptions and claims shall be deemed waived unless made in writing within 3 days from time of delivery. In such a case, seller shall be given full opportunity to investigate any such claim. Seller's liability shall in no event exceed the purchase price of the materials against which any claims are made.

Figure C-6. Concrete Support, Test Nos. MGTD-1S and MGTD-1D (Item No. b4)

benesch



# Concrete Sample Test Report
## Cylinder Compressive Strength

| | |
|---|---|
| Project Name: | Midwest Roadside Safety - Misc Testing |
| Project Number: | 00110546.00 |
| Client: | Midwest Roadside Safety Facility |
| Location: | MNPD |
| Sample: | 02182022.1 |
| Description: | Montana A |

### Field Data (ASTM C172, C143, C173/C231, C138, C1064)

| | | Property | Test Result |
|---|---|---|---|
| Supplier: | | Slump (in): | |
| Mix Name: | | Air Content (%): | |
| Ticket Number: | | Unit Weight (lb/ft³): | |
| Truck Number: | | Air Temp (°F): | |
| Load Volume (yd³): | | Mix Temp (°F): | |
| Mold Date: | 02/18/2022 | Min Temp (°F): | |
| Molded By: | | Max Temp (°F): | |
| Initial Cure Method: | | | |

### Laboratory Test Data (ASTM C39)

| | | | | | | |
|---|---|---|---|---|---|---|
| Sample Number: | 02182022.1 | | | | | |
| Set Number: | A | | | | | |
| Specimen Number: | 1 | | | | | |
| Age: | 52 | | | | | |
| Length (in): | 12 | | | | | |
| Diameter (in): | 5.99 | | | | | |
| Area (in²): | 28.18 | | | | | |
| Density (lb/ft³): | 140 | | | | | |
| Test Date: | 04/11/2022 | | | | | |
| Break Type: | 2 | | | | | |
| Max Load (lbf): | 131,642 | | | | | |
| Strength (psi): | 4,670 | | | | | |
| Spec Strength (psi): | | | | | | |
| Excl in Avg Strength: | ☐ | ☐ | ☐ | ☐ | ☐ | ☐ |

Remarks:
Set A, Specimen 1, 52-day Compressive Strength (psi): **4,670**

Sample Receive Date: 04/11/2022

Approved by:

Matt Roessler

Matt Roessler Manager


Date: 04/11/2022



825 M Street Suite 100
Lincoln, NE 68508

Alfred Benesch & Company

Version 1 Created by Matt Roessler Manager (mroessler@benesch.com) on 04/11/2022 7:43 PM CDT



Figure C-7. Concrete Support, Test Nos. MGTD-1S and MGTD-1D (Item No. b4)

benesch

Page 1 of 1

# Concrete Sample Test Report
## Cylinder Compressive Strength

| | |
|---|---|
| Project Name: | Midwest Roadside Safety - Misc Testing |
| Project Number: | 00110546.00 |
| Client: | Midwest Roadside Safety Facility |
| Location: | MNPD |
| Sample: | 02182022.2 |
| Description: | Montana B |

### Field Data (ASTM C172, C143, C173/C231, C138, C1064)

| | | Property | Test Result |
|---|---|---|---|
| Supplier: | | Slump (in): | |
| Mix Name: | | Air Content (%): | |
| Ticket Number: | | Unit Weight (lb/ft³): | |
| Truck Number: | | Air Temp (°F): | |
| Load Volume (yd³): | | Mix Temp (°F): | |
| Mold Date: | 02/18/2022 | Min Temp (°F): | |
| Molded By: | | Max Temp (°F): | |
| Initial Cure Method: | | | |

### Laboratory Test Data (ASTM C39)

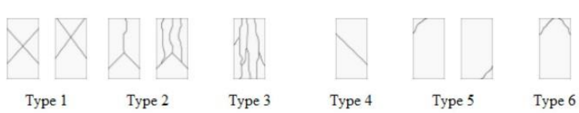
| | | | | | | |
|---|---|---|---|---|---|---|
| Sample Number: | 02182022.2 | | | | | |
| Set Number: | B | | | | | |
| Specimen Number: | 1 | | | | | |
| Age: | 52 | | | | | |
| Length (in): | 12 | | | | | |
| Diameter (in): | 5.99 | | | | | |
| Area (in²): | 28.18 | | | | | |
| Density (lb/ft³): | 144 | | | | | |
| Test Date: | 04/11/2022 | | | | | |
| Break Type: | 6 | | | | | |
| Max Load (lbf): | 146,535 | | | | | |
| Strength (psi): | 5,200 | | | | | |
| Spec Strength (psi): | | | | | | |
| Excl in Avg Strength: | ☐ | ☐ | ☐ | ☐ | ☐ | ☐ |

Remarks:
Set B, Specimen 1, 52-day Compressive Strength (psi): **5,200**

Sample Receive Date: 04/11/2022

Approved by:

Matt Roessler Manager




Date: 04/11/2022

This report shall not be reproduced, except in full, without prior approval of Alfred Benesch & Company. Results relate only to items tested.

825 M Street Suite 100
Lincoln, NE 68508

Alfred Benesch & Company

Version 1 Created by Matt Roessler Manager (mroessler@benesch.com) on 04/11/2022 7:55 PM CDT



Figure C-8. Concrete Support, Test Nos. MGTD-1S and MGTD-1D (Item No. b4)

Akzo Nobel Coatings Inc.
Powder Coatings

AkzoNobel

## CERTIFICATE OF COMPLIANCE

Product Name: RB-600 (HKF30R)
Product Description: RESICOAT® GREEN REBAR COATING

To Whom It May Concern:

This is to certify that the batch number of Resicoat RB-600 fusion bonded epoxy powder coating listed below is chemically the same material as tested by Wiss Janney Elstner Associates of Northbrook Illinois to ASTM A 775. I certify that it meets the requirements of ASTM A 775. Resicoat RB-600 also meets the requirements of ASTM D 3963, ASTM A 884, AASHTO M 254 type B and AASHTO M 284.

The following batch was manufactured in the United States and qualifies as "U.S. made end products", "domestic construction materials", and "domestic manufactured goods". When applied to steel or iron in the U.S. this coating meets the Buy America provisions set forth in FHWA 23 CFR 635.410 Section 1041(a) of the ISTEA.

Batch: _WH69479NA____ Production Date: _8/27/2020___ Batch Size: __17,480__ Kg's.

For Quality Assurance Supervisor: [signature] Signed

State/Commonwealth TN County of DAVIDSON

On this the 28th (Day) of AUGUST (Month), 2020 (Year), before me KAYDIE WILLIS (Name of Notary Public)

The undersigned Notary Public, personally appeared KENNY MCFARLAND (Name(s) of Signer(s)) ☑ Personally known to me

KAYDIE WILLIS
STATE OF TENNESSEE NOTARY PUBLIC
DAVIDSON COUNTY

To be the person(s) whose name(s) is/are subscribed to the Within instrument, and acknowledged to me that he/she/they Executed the same for the purposes therin stated.

Witness my hand and official seal

[signature] Signature of Notary Public

**CAUTION: Special safety practices should be followed when using any powder coating. For further information, please refer to the specific product Material Safety Data Sheet (MSDS). The information contained in this COC has been determined through the application of accepted engineering practice and is believed to be reliable. Since the conditions of application and use of our products are beyond our control, no warranty is expressed or implied regarding accuracy of the information, the results to be obtained from the use of the product, or that such use will not infringe on any patent. This information is furnished with the express condition that you will make your own tests to determine the suitability of the product for your particular use. RESICOAT® is a registered trademark of Akzo Nobel.**

20 Culvert Street
Nashville, TN 37210
USA

T +1 615 259 2430
F +1 615 255 7903
www.interpon.us

RESICOAT

Figure C-9. Grade 60 No. 4 Bars, Test Nos. MGTD-1S and MGTD-1D (Item Nos. b5 and b6)

NUCOR

# Mill Certification

09/02/2020

MTR#:458890-2
Lot #:360001474020
ONE NUCOR WAY
BOURBONNAIS, IL 60914 US
815 937-3131
Fax: 815 939-5599

Sold To: SIMCOTE INC
1645 RED ROCK RD
ST PAUL, MN 55119 US

Ship To: SIMCOTE INC
1645 RED ROCK RD
ST PAUL, MN 55119 US

| | | | |
|---|---|---|---|
| Customer PO | MN-3748 | Sales Order # | 36013225 - 1.31 |
| Product Group | Rebar | Product # | 2110206 |
| Grade | A615 Gr 60/AASHTO M31 | Lot # | 360001474020 |
| Size | #4 | Heat # | 3600014740 |
| BOL # | BOL-567414 | Load # | 458890 |
| Description | Rebar #4/13mm A615 Gr 60/AASHTO M31 60' 0" [720"] 6001-10000 lbs | Customer Part # | |
| Production Date | 08/12/2020 | Qty Shipped LBS | 22725 |
| Product Country Of Origin | United States | Qty Shipped EA | 567 |
| Original Item Description | | Original Item Number | |

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Country of Origin : United States    Melting Date: 08/07/2020

| C (%) | Mn (%) | P (%) | S (%) | Si (%) | Ni (%) | Cr (%) | Mo (%) | Cu (%) | V (%) | Nb (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.34 | 0.90 | 0.015 | 0.043 | 0.198 | 0.18 | 0.23 | 0.06 | 0.40 | 0.012 | 0.002 |

**Other Test Results**

Yield (PSI) : 66100    Tensile (PSI) : 99200    Average Deformation Height (IN) : 0.036

Elongation in 8" (%) : 14.5    Bend Test : Pass    Weight Percent Variance (%) : -4.00

**Comments:**

All manufacturing processes of the steel materials in this product, including melting, have occurred within the United States. Products produced are weld free. Mercury, in any form, has not been used in the production or testing of this material.

Zachary Sprintz, Chief Metallurgist

Page 1 of 1

Figure C-10. Grade 60 No. 4 Bars, Test Nos. MGTD-1S and MGTD-1D (Item Nos. b5 and b6)

**Portland Bolt**
**& MANUFACTURING COMPANY**

Phone: 800-547-6758 | Fax: 503-227-4634
3441 NW Guam Street, Portland, OR 97210
Web: www.portlandbolt.com | Email: sales@portlandbolt.com

**CERTIFICATE OF CONFORMANCE**

**For:** MIDWEST ROADSIDE SAFETY FACIL
**PB Invoice#:** 142135
**Cust PO#:** MGTR/MDTD
**Date:** 5/21/2021
**Shipped:** 5/27/2021

**We certify that the following items were manufactured and tested in accordance with the chemical, mechanical, dimensional and thread fit requirements of the specifications referenced.**

**Description:** 7/8 X 15 GALV ASTM F3125 GRADE A325 HEAVY HEX BOLT

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Heat#:** | 3093334 | | **Base Steel:** | 4140 | | **Diam:** | 7/8 | | |
| **Source:** | COMMERCIAL METALS CO | | | | | **Proof Load:** | 39,250 LBF | | |
| **C :** | .400 | **Mn:** | .810 | **P :** | .016 | **Hardness:** | 293 HBN | | |
| **S :** | .019 | **Si:** | .240 | **Ni:** | .190 | **Tensile:** | 67,180 LBF | **RA:** | .00% |
| **Cr:** | .870 | **Mo:** | .208 | **Cu:** | .320 | **Yield:** | 0 | **Elon:** | .00% |
| **Pb:** | .000 | **V :** | .024 | **Cb:** | .000 | **Sample Length:** | 0 | | |
| **N :** | .000 | | | **CE:** | .6329 | **Charpy:** | | **CVN Temp:** | |

LOT#19878

**Coatings:**
ITEMS HOT DIP GALVANIZED PER ASTM F2329/A153C

By:
Certification Department Quality Assurance
Dane McKinnon

Figure C-11. ⅞-in.- 9 UNC x 15-in. Heavy Hex Bolt, Test Nos. MGTD-1S, (Item No. d1)

**Portland Bolt**
**& MANUFACTURING COMPANY**

Phone: 800-547-6758 | Fax: 503-227-4634
3441 NW Guam Street, Portland, OR 97210
Web: www.portlandbolt.com | Email: sales@portlandbolt.com

**CERTIFICATE OF CONFORMANCE**

**For:** MIDWEST ROADSIDE SAFETY FACIL
**PB Invoice#:** 142079
**Cust PO#:** MGTD/MGTR
**Date:** 5/19/2021
**Shipped:** 5/20/2021

**We certify that the following items were manufactured and tested in accordance with the chemical, mechanical, dimensional and thread fit requirements of the specifications referenced.**

---

**Description:** 3/4 X 30 GALV ASTM A307A TIMBER BOLT

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Heat#:** | 1202025843 | **Base Steel:** | A36 | | | **Diam:** | .68 | | |
| **Source:** | NUCOR STEEL | | | | | **Proof Load:** | 0 | | |
| **C :** | .130 | **Mn:** | .680 | **P :** | .015 | **Hardness:** | 0 | | |
| **S :** | .034 | **Si:** | .230 | **Ni:** | .070 | **Tensile:** | 70,800 PSI | **RA:** | 48.00% |
| **Cr:** | .150 | **Mo:** | .020 | **Cu:** | .270 | **Yield:** | 51,300 PSI | **Elon:** | 27.00% |
| **Pb:** | .000 | **V :** | .002 | **Cb:** | .000 | **Sample Length:** | 8 INCH | | |
| **N :** | .000 | | | **CE:** | .2679 | **Charpy:** | | **CVN Temp:** | |

---

**Coatings:**
ITEMS HOT DIP GALVANIZED PER ASTM F2329/A153C

---

By: ______________________
Certification Department Quality Assurance
Dane McKinnon

Figure C-12. ¾-in.-10 UNC x 30-in. Timber Bolts with Nubs, Test Nos. MGTD-1S and MGTD-1D (Item No. d2)

FASTENAL®

# Certificate of Compliance

**Sold To:**

UNL / UNMC E-SHOP / PUNCHOUT

**Purchase Order:** E000867825

**Job:** MIDWEST ROADSIDE SAFETY

**Invoice Date:** 07/1/2021

THIS IS TO CERTIFY THAT WE HAVE SUPPLIED YOU WITH THE FOLLOWING PARTS.
THESE PARTS WERE PURCHASED TO THE FOLLOWING SPECIFICATIONS.

6 PCS 3/4"-10 x 9-1/2" ASTM A307 Grade A Hot Dipped Galvanized Hex Bolt SUPPLIED UNDER OUR TRACE NUMBER 120306283 AND UNDER PART NUMBER 91975

12 PCS 3/4"-10x8" (OAL 8-1/4") A193 B7 Hot Dipped Galvanized Fully Threaded Stud SUPPLIED UNDER OUR TRACE NUMBER 935935-1 AND UNDER PART NUMBER 0186717

60 PCS 3/4"-4.5 x 11" Grade A Hot Dip Galvanized Finish Hex Head Lag Screw SUPPLIED UNDER OUR TRACE NUMBER llne50358 AND UNDER PART NUMBER 22492

80 PCS 3/4" x 2.000" OD Low Carbon Hot Dipped Galvanized Finish Steel USS General Purpose Flat Washer SUPPLIED UNDER OUR TRACE NUMBER 210220089 AND UNDER PART NUMBER 1133186

42 PCS 3/4"-10 Grade A Hot Dip Galvanized Heavy Hex Nut SUPPLIED UNDER OUR TRACE NUMBER 180198094 AND UNDER PART NUMBER 36716

This is to certify that the above document is true and accurate to the best of my knowledge.

**Fastenal Account Representative Signature**

Ross Schall

**Printed Name**

7/1/2021

**Date**

Please check current revision to avoid using obsolete copies.

This document was printed on 07/01/2021 and was current at that time.

**Fastenal Store Location/Address**

*3201 N. 23rd Street STE 1*
*LINCOLN, NE 68521*
*Phone #: (402)476-7900*
*Fax #: 402/476-7958*

Page 1 of 1

Figure C-13. ¾-in.-10 UNC x 9½-in. Hex Bolt, Test Nos. MGTD-1S, 1D (Item No. d3)

**Portland Bolt & MANUFACTURING COMPANY**

Phone: 800-547-6758 | Fax: 503-227-4634
3441 NW Guam Street, Portland, OR 97210
Web: www.portlandbolt.com | Email: sales@portlandbolt.com

**CERTIFICATE OF CONFORMANCE**

**For:** MIDWEST ROADSIDE SAFETY FACIL
**PB Invoice#:** 142080
**Cust PO#:** MGTD
**Date:** 5/27/2021
**Shipped:** 5/28/2021

**We certify that the following items were manufactured and tested in accordance with the chemical, mechanical, dimensional and thread fit requirements of the specifications referenced.**

**Description:** 3/4 X 60 GALV ASTM A307A ROD

**Heat#:** 1202027708 **Base Steel:** A36 **Diam:** 3/4

**Source:** NUCOR STEEL **Proof Load:** 0

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **C :** | .130 | **Mn:** | .680 | **P :** | .011 | **Hardness:** | 0 | | | | |
| **S :** | .039 | **Si:** | .150 | **Ni:** | .080 | **Tensile:** | 67,600 PSI | **RA:** | 46.00% | | |
| **Cr:** | .110 | **Mo:** | .020 | **Cu:** | .240 | **Yield:** | 50,400 PSI | **Elon:** | 29.00% | | |
| **Pb:** | .000 | **V :** | .002 | **Cb:** | .000 | **Sample Length:** | 8 INCH | | | | |
| **N :** | .000 | | | **CE:** | .2637 | **Charpy:** | | **CVN Temp:** | | | |

**Other:**
ALL ITEMS MELTED & MANUFACTURED IN THE USA

By:
Certification Department Quality Assurance
Dane McKinnon

Figure C-14. ¾-in.-10 UNC x 8 on a 60-in. Long Tie Rod, Test Nos. MGTD-1S and MGTD-1D (Item No. d4)

**FASTENAL® MANUFACTURING**

7730 Pinemont Dr. Houston, TX 77040 Tel. 713-460-4381 Fax. 713-996-7342
1801 Theurer Blvd. Winona MN 55987

## Certified Material Test Report

| | | | |
|---|---|---|---|
| **Certificate Number:** | 935935-1 | **Date Issued:** | October 2, 2020 |
| **Customer Name:** | NC100CASH | **Customer Part Number:** | N/A |
| **Sold To:** | NHUB | **Customer P.O.:** | N/A |
| **Description:** | Std 3/4-10x8 | **Marking:** | FNL B7 |
| **Finish:** | HOT DIPPED GALVANIZED | **Fastenal Part Number:** | 0186717 |
| **Material:** | B7 | **Quantity Shipped Production Lot Quantity:** | 276/270 |

| **SPECIFICATION / GRADE:** In accordance with ASTM A193-20 ; ASME SA193-19 Gr. B7 | |
|---|---|
| **SURFACE QUALITY:** In accordance with ASTM F788/F788M-20 | PASS |
| **COATING:** Hot Dipped Galvanized | PASS |

**CHEMISTRY – Heat Number:** 18B701615
Heat Composition (WT% Heat Analysis)

| Element: | C | Mn | P | S | Si | Cr | Mo |
|---|---|---|---|---|---|---|---|
| **Minimum:** | 0.38 | 0.75 | - | - | 0.15 | 0.80 | 0.15 |
| **Maximum:** | 0.48 | 1.00 | 0.035 | 0.040 | 0.35 | 1.10 | 0.25 |
| **Result:** | 0.40 | 0.84 | 0.014 | 0.002 | 0.19 | 0.93 | 0.22 |

**MACROETCH (if required):** ASTM E381: S2,R2,C2

**Heat Treat Method:** Quenched and Tempered

| Attribute | Test Method | Sample Size | Requirement | Result | Acceptance |
|---|---|---|---|---|---|
| Hardness | - | 1 | Max: 35 HRC | 30 HRC | PASS |
| Reduction of Area | - | 1 | Min: 50 % | 52 % | PASS |
| Elongation (4D) | - | 1 | Min: 16 % | 17 % | PASS |
| Yield Strength (.2 % Offset) | - | 1 | Min: 105,000 PSI | 118,000 PSI | PASS |
| Tensile Strength | - | 1 | Min: 125,000 PSI | 136,000 PSI | PASS |
| Decarburization | ASTM A962 Sect.14-16 | 2 | - | PASS | PASS |

Page 1 of 2 June 25 ,2020
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Figure C-15. ¾-in.-10 UNC x 8-in. Threaded Rod, Test Nos. MGTD-1S and MGTD-1D (Item No. d5)

FASTENAL®

# Certificate of Compliance

**Sold To:**

UNL / UNMC E-SHOP / PUNCHOUT

**Purchase Order:** E000867825

**Job:** MIDWEST ROADSIDE SAFETY

**Invoice Date:** 07/1/2021

THIS IS TO CERTIFY THAT WE HAVE SUPPLIED YOU WITH THE FOLLOWING PARTS.
THESE PARTS WERE PURCHASED TO THE FOLLOWING SPECIFICATIONS.

6 PCS 3/4"-10 x 9-1/2" ASTM A307 Grade A Hot Dipped Galvanized Hex Bolt SUPPLIED UNDER OUR TRACE NUMBER 120306283 AND UNDER PART NUMBER 91975

12 PCS 3/4"-10x8" (OAL 8-1/4") A193 B7 Hot Dipped Galvanized Fully Threaded Stud SUPPLIED UNDER OUR TRACE NUMBER 935935-1 AND UNDER PART NUMBER 0186717

60 PCS 3/4"-4.5 x 11" Grade A Hot Dip Galvanized Finish Hex Head Lag Screw SUPPLIED UNDER OUR TRACE NUMBER llne50358 AND UNDER PART NUMBER 22492

80 PCS 3/4" x 2.000" OD Low Carbon Hot Dipped Galvanized Finish Steel USS General Purpose Flat Washer SUPPLIED UNDER OUR TRACE NUMBER 210220089 AND UNDER PART NUMBER 1133186

42 PCS 3/4"-10 Grade A Hot Dip Galvanized Heavy Hex Nut SUPPLIED UNDER OUR TRACE NUMBER 180198094 AND UNDER PART NUMBER 36716

This is to certify that the above document is true and accurate to the best of my knowledge.

[signature]
**Fastenal Account Representative Signature**

Ross Schall
**Printed Name**

7/1/2021
**Date**

Please check current revision to avoid using obsolete copies.

This document was printed on 07/01/2021 and was current at that time.

**Fastenal Store Location/Address**

*3201 N. 23rd Street STE 1*
*LINCOLN, NE 68521*
*Phone #: (402)476-7900*
*Fax #: 402/476-7958*

Page 1 of 1

Figure C-16. ¾-in. x 4½-in. x 11-in. Lag Bolt, Test Nos. MGTD-1S and MGTD-1D (Item No. d6)

P#33187 C#17089822 L#1844804 7/8" Dia. Plain Round Washer

# CERTIFIED MATERIAL TEST REPORT
# FOR USS FLAT WASHERS HDG

FACTORY: IFI & Morgan Ltd — REPORT DATE: 23/4/2019
ADDRESS: Chang'an North Road, Wuyuan Town, Haiyan,Zhejiang, China

MFG LOT NUMBER: 1844804

SAMPLING PLAN PER ASME B18.18-11 — PO NUMBER: 170089822
SIZE: USS 7/8 HDG — QNTY(Lot size): 7200PCS
HEADMARKS: NO MARK — PART NO: 33187

DIMENSIONAL INSPECTIONS — SPECIFICATION: ASTM B18.21.1-2011

| CHARACTERISTICS | SPECIFIED | ACTUAL RESULT | ACC. | REJ. |
|---|---|---|---|---|
| APPEARANCE | ASTM F844 | PASSED | 100 | 0 |
| OUTSIDE DIA | 2.243-2.280 | 2.246-2.254 | 10 | 0 |
| INSIDE DIA | 0.931-0.968 | 0.956-0.965 | 10 | 0 |
| THICKNESS | 0.136-0.192 | 0.136-0.157 | 10 | 0 |

| CHARACTERISTICS | TEST METHOD | SPECIFIED | ACTUAL RESULT | ACC. | REJ. |
|---|---|---|---|---|---|
| HOT DIP GALVANIZED | ASTM F2329-13 | Min 0.0017" | 0.0017-0.0020 in | 8 | 0 |

ALL TESTS IN ACCORDANCE WITH THE METHODS PRESCRIBED IN THE APPLICABLE ASTM SPECIFICATION. WE CERTIFY THAT THIS DAIA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY.
ISO 9001:2015 SGS Certificate # HK04/0105

IFI & MORGAN LTD.
检验专用章
QUANLITY CONTROL

(SIGNATURE OF Q.A. LAB MGR.)

Figure C-17. ⅞-in. Flat Washer, Test No. MGTD-1S and MGTD-1D (Item No. d7)

**INV#:MB19-137**

# CERTIFIED MATERIAL TEST REPORT

**Factory: SHENG DA-LI MACHINERY FACTORY** — **Date: 2019-12-12**

**Item: ROUND WASHER** — **Lot No: 9100963-02**

**Customer: BBI** — **Finish:HDG.**

**Quantity Shipped:72CTNS.** — **BBI/PO:B19100963**

**Sampling Plan per: 32510 MALLEABLE IRON** — **Part No: P39086**

**Size & Description: 3/4"** — **Heat No: 2019112802**

## Material Test Results

材 质 报 告

| Chemical Analysis（%） 化 学 成 分 | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| C | Si | Mn | P | S | | | | | |
| 2.55 | 1.52 | 0.51 | 0.053 | 0.053 | | | | | |

## Mechanical Properties Test Results

机 械 性 能 报 告

| | Standard Requirements 要求 | Test Results 检验结果 |
|---|---|---|
| Tensile Strength (Mpa) 抗 拉 强 度 | 345 | 363 |
| Yield Strength (Mpa) 屈 服 强 度 | 224 | 253 |
| Elongation (%) 延 伸 率 | 10 | 14.1 |

**All tests are in accordance with the methods prescribed in the applicable ASTM specification. We certify that this data is a true representation of information provided by the material supplier and our testing laboratory.**

**REMARK:** **1.The report is issued according to ISO16228 F3.1 (EN10204 3.1).**
**2.Test Facility: M**

王团训

**(Signature of Q.A. Lab Mgr.)**

Figure C-18. ¾-in. Malleable Iron Washer, Test Nos. MGTD-1S and MGTD-1D (Item No. d8)

# CERTIFIED MATERIAL TEST REPORT
# FOR USS FLAT WASHERS HDG

FACTORY: **IFI & Morgan Ltd** REPORT DATE: 10/5/2021
ADDRESS: **NO.12 Plant 1, Haisheng Road, Wuyuan Town, Haiyan, Zhejiang, China** MANUFACTURE DATE:
CUSTOMER:
MFG LOT NUMBER: 2008905

SAMPLING PLAN PER ASME B18.18-11 PO NUMBER: 210220089
SIZE: USS 3/4 HDG QNTY(Lot size): 11250PCS
HEADMARKS: NO MARK PART NO: 1133186

DIMENSIONAL INSPECTIONS SPECIFICATION: ASTM B18.21.1-2011

| CHARACTERISTICS | SPECIFIED | ACTUAL RESULT | ACC. | REJ. |
|---|---|---|---|---|
| ********************* | *********************************** | ****************** | ******** | ******** |
| APPEARANCE | ASTM F844 | PASSED | 100 | 0 |
| OUTSIDE DIA | **1.993-2.030** | 1.996-2.004 | 10 | 0 |
| INSIDE DIA | **0.805-0.842** | 0.830-0.839 | 10 | 0 |
| THICKNESS | **0.122-0.177** | 0.122-0.138 | 10 | 0 |

| CHARACTERISTICS | TEST METHOD | SPECIFIED | ACTUAL RESULT | | ACC. | REJ. |
|---|---|---|---|---|---|---|
| ********************* | ****************** | ****************** | ****************** | | ******** | ******** |
| HOT DIP GALVANIZED | ASTM F2329-13 | Min 0.0017" | 0.0017-0.0020 | in | 8 | 0 |

ALL TESTS IN ACCORDANCE WITH THE METHODS PRESCRIBED IN THE APPLICABLE ASTM SPECIFICATION. WE CERTIFY THAT THIS DAIA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY.
MFG ISO9002 CERTIFICATE NO. HK04/0105

IFI & MORGAN LTD. 检验专用章 QUANLTIY CONTROL

(SIGNATURE OF Q.A. LAB MGR.)
(NAME OF MANUFACTURER)

Figure C-19. ¾-in. Flat Washer, Test Nos. MGTD-1S and MGTD-1D (Item No. d9)

**McMASTER-CARR.**

# Packing List

600 N County Line Rd
Elmhurst IL 60126-2081
630-600-3600
chi.sales@mcmaster.com

University of Nebraska
Midwest Roadside Safety Facility
M W R S F
4630 Nw 36TH St
Lincoln NE 68524-1802
Attention: Shaun M Tighe
Midwest Roadside Safety Facility

Purchase Order
**E000869476**

Order Placed By
**Shaun M Tighe**

McMaster-Carr Number
**7470328-01**

Page 1 of 1

05/25/2021

| Line | | Product | Ordered | Shipped |
|---|---|---|---|---|
| 1 | **97801A111** | Steel Nails, 20D Penny Size, 4" Long, Packs of 125 | **10 Packs** | **10** |
| 2 | **3274T51** | Steel Oval Eye Nut - for Lifting, 7/8"-9 Thread Size | **2 Each** | **2** |

Certificate of compliance

This is to certify that the above items were supplied in accordance with the description and as illustrated in the catalog. Your order is subject only to our terms and conditions, available at www.mcmaster.com or from our Sales Department.

Sarah Weinberg
Compliance Manager

795

Figure C-20. ⅞-in.-9 UNC Eye Nut, Test No. MGTD-1S (Item No. d10)

YUXING FASTENER (JIAXING) CO.,LTD.

CHANGQIAN TOWN, XITANGQIAO SUB-DISTRICT, HAIYAN COUNTY, JIAXING CITY, ZHEJIANG PROVINCE, CHINA

TEL:86-573-8685-0620 FAX:86-573-86855061

CERTIFICATE OF INSPECTION

Report Date: 2020/6/6 Manufacture Date: 2020/4/11

| | | | | | | |
|---|---|---|---|---|---|---|
| Customer | | FASTENAL COMPANY PURCHASING IMPORT | | | | |
| Customer PO Number | | 180198094 | | | | |
| Customer Part Number | | 36716 | | | | |
| Customer Item Number | | HHN.GRA.HDG.01 | | | | |
| Product Description | | 3/4-10 GRADE A HEAVY HEX NUT | | | | |
| Surface Condition | | Hot Dip Galvanize per ASTM F2329/F2329M-2015 | | | | |
| Head Marking | | NO MARK | | | | |
| Manufacture quantity: 10,000 PCS | | | Shipment quantity: 9,000 PCS | | | |
| Sampling Plan | | ASME B18.18-2017/ASTM F1470-2019 | | | | |
| Material type | | ML08AL | Heat No | B19120832 | | |
| Chemical composition | Spe | C | Mn | P | S | Si | Cr |
| | | 0.55max | / | 0.120max | 0.150max | | |
| | Test | 0.06 | 0.33 | 0.016 | 0.007 | 0.03 | |

| Test item | SPEC. | Standard | Test value | Sampling size | ACC | REJ |
|---|---|---|---|---|---|---|
| Width across Flat | ASME B18.2.2-2015 | 1.250"-1.212" | 1.243"-1.235" | 4 | 4 | 0 |
| Width across Corner | ASME B18.2.2-2015 | 1.443"-1.382" | 1.435"-1.432" | 4 | 4 | 0 |
| Height | ASME B18.2.2-2015 | 0.758"-0.710" | 0.746"-0.744" | 4 | 4 | 0 |
| Thread minor | ASME B1.1-2003 | 0.683"-0.662" | 0.678"-0.673" | 15 | 15 | 0 |
| Thread | ASTM A563-2015 | Overtaped Go gauge | | 15 | 15 | 0 |
| | | Overtaped NO Go gauge | | 15 | 15 | 0 |
| Surface condition | ASTM F812-2012 | Surface discontinuities: OK | | 22 | 22 | 0 |
| Run-out tol.(FIM) | ASME B18.2.2-2015 | 0.027"max | 0.015"-0.012" | 4 | 4 | 0 |

| Mechanical properties | ASTM A563-2015 | | | | | |
|---|---|---|---|---|---|---|
| Test item | Test method | Standard | Test value | Sampling size | ACC | REJ |
| Proof load | ASTM F606-2019 | 75,000PSI | 75,000PSI | 3 | 3 | 0 |
| Core Hardness | ASTM F606-2019 | HRB68-HRC32 | HRC13-11 | 4 | 4 | 0 |
| Plating thickness | ASTM F2329-2015 | 43μm min | 49-44 μm | 15 | 15 | 0 |

FACTORY INSPECTOR: 黄伟明

宇星紧固件（嘉兴）股份有限公司
YUXING FASTENER (JIAXING) CO.,LTD.

Parts are manufactured and tested according to above specification and compliance with order,
we certify that this is a true representation of information provided by manufacturer and laboratory.
The MTRshall include a statement that the products supplied are in compliance with all the requirements of the order.
We certify this MTR comliance to DIN EN 10204.3.1 content.

¾-in.-10 UNC Heavy Hex Nut, Test Nos. MGTD-1S and MGTD-1D (Item No. d11)

P.O. Box 21148
Tulsa, OK 74121
Phone (918) 252-6000
Fax No. (918) 252-6520

Certified Quality and Environmental Systems
SQS
ISO 9001 / ISO 14001
Reg. No. 12455-03

Date: 12/13/2016

**Subject: Certificate of Conformance**

**Product: HIT RE-500 V3 Adhesive**

To Whom it May Concern:

This is to certify that the HIT-RE 500 V3 is a high-strength, slow cure two-part epoxy adhesive contained in two cartridges separating the resin from the hardener.

Additionally, this certifies that the product has been seismically and cracked concrete qualified as represented in ICC-ES report ESR- 3814.

Sincerely,

**Hilti, Inc.**

5400 South 122 East Avenue

Tulsa, Oklahoma 74146

800-879-8000

800-879-7000 fax

US-Sales@hilti.com

Figure C-21. Hilti RE-500 V3 Adhesive Certificate of Conformance, Test Nos. MGTD-1S and MGTD-1D (Item No. e1)

# Appendix D. Nail-Lam Deck and Rail Material Specifications

Table D-1. Bill of Materials, Nail-Laminated Deck and Rail, Test Nos. MGTR-1S and MGTR-1D

| Item No. | Description | Material Specification | Reference |
|---|---|---|---|
| a1 | 9½"x23"x7½" Scupper Block | Grade No. 1 Southern Yellow Pine or Douglas Fir | Bell Lumber R#153279 |
| a2 | 2"x6"x14' Long Treated, Dimensional Lumber | Grade No. 1 Southern Yellow Pine | Order #31470230 |
| a3 | Deck Anchor Plate | | H#41743950 |
| a4 | 12⅜"x23"x6¾" Glulam Rail Segment | Pentachorophenal- 0.6 lb/cu. ft Retention | Bell Lumber R#153279 |
| a5 | 23"x6¾"x¾" Static Test Plate | ASTM A36 | H#RS3132, H#RS3610, H#RS3781, H#RS3825 |
| a6 | 12⅜"x23"x6¾" Glulam Rail Segment | Southern Yellow Pine Combination No. 48 | Bell Lumber R#153279 |
| b1 | Concrete Support 1 | Min f'c = 4,000 psi NE Mix 47 BD | N/A |
| b2 | Concrete Support 2 | Min f'c = 4,000 psi NE Mix 47 BD | N/A |
| b3 | Concrete Support 3 | Min f'c = 4,000 psi NE Mix 47 BD | N/A |
| c1 | Bridge Abutment Assembly | Min f'c = 4,000 psi NE Mix 47 BD | N/A |
| c2 | #4 Bent Rebar, 52¾" Unbent Length | ASTM A615 Gr. 60 | N/A |
| c3 | #5 Rebar 5'-6" Long | ASTM A615 Gr. 60 | N/A |
| c4 | #5 Rebar 12'-6" Long | ASTM A325 | N/A |
| c5 | #4 Rebar 12'-6" Long | ASTM A615 Gr. 60 | N/A |
| c6 | Bridge Pier Assembly | Min f'c = 4,000 psi NE Mix 47 BD | N/A |
| c7 | #4 Bent Rebar, 52¼" Unbent Length | ASTM A615 Gr. 60 | N/A |
| c8 | W27x94, 40' Long Steel Girder | ASTM A36 | N/A |
| c9 | L5x3.5x0.5, 18" Long | ASTM A36 | N/A |
| c10 | L5x3.5x0.5, 16" Long | ASTM A36 | N/A |
| c11 | 30 5/16"x7 3/16"x⅜" Plate | ASTM A36 | N/A |
| c12 | 30 5/16" x7 3/16"x⅜" Plate | ASTM A36 | N/A |

N/A – Material certification not available

Table D-2. Bill of Materials, Nail-Laminated Deck and Rail, Cont., Test Nos. MGTR-1S and MGTR-1D

| Item No | Description | Material Specification | Reference |
|---|---|---|---|
| c13 | WT3x10, 42" Long | ASTM A36 | N/A |
| c14 | WT3x10, 66½" Long | ASTM A36 | N/A |
| c15 | C15x33.9, 42" Long | ASTM A36 | N/A |
| c16 | C15x33.9, 66½" Long | ASTM A36 | N/A |
| c17 | 24"x9"x1½" Sole Plate | ASTM A36 | N/A |
| c18 | 24"x9"x3/4" Sole Plate | ASTM A36 | N/A |
| c19 | Elastomeric Bearing Pad | Neoprene - Min. 50 Durometer | N/A |
| d1 | ¾"-10 UNC, 2" Long Heavy Hex Bolt | ASTM A563 Gr. 5 | N/A |
| d2 | ¾"-10 UNC Heavy Hex Nut | ASTM A563 | N/A |
| d3 | 1½"-6 UNC Heavy Hex Nut | ASTM A563 Gr. 5 | N/A |
| d4 | 1½" Dia. Epoxy Rod, 20" Long | ASTM A615 Gr. 60 | N/A |
| d5 | ¾"-10 UNC Timber Bolt, 30" Long | ASTM A307 Gr. A | H#1202025843 |
| d6 | ¾" Malleable Iron Washer | | P#0128540 H#2019112802 |
| d7 | ⅞"-9 UNC, 15" Long Heavy Hex Bolt | ASTM F3125 Gr. A325 | H#3093334 |
| d8 | ⅞" Dia. Plain Round Washer | ASTM F844 | P#33187 C#170089822 L#1844804 |
| d9 | ⅞"-9 UNC Eye Nut | ASTM A325 | 3274T51 McMaster Carr |
| d10 | 20d Nails | | 97801A111 McMaster Carr |
| d11 | Liquid Nails | | N/A |

N/A – Material certification not available

BILL OF LADING - SHORT FORM - NOT NEGOTIABLE

Page 2 of 2

**BELL LUMBER & POLE COMPANY**

153279

P. O. Box 120736 New Brighton, MN 55112
Yard Phone: 651-633-4334 Yard Fax: 651-633-8852

Date: 1/4/2022

DESCRIPTION OF ARTICLES AND SPECIAL MARKS Received By:

**General Delivery 778 1st St NW New Brighton, MN 55112**

| Qty | Product | Framing | PO Number | Other PO | Item Number | Deck |
|---|---|---|---|---|---|---|
| 2 | PEL-6x12-1.8125 | NA | | | | |
| 2 | PEL-6x12-1.8125 | NA | | | | |
| 10 | PEL-7x9-1.8125 | NA | | | | |
| 4 | EL-5x12-3.5 | NA | | | | |
| 3 | EL-5x48-12 | NA | | | | |
| 2 | EL-5x48-12 | NA | | | | |
| 2 | EL-6x16-20 | NA | | | | |
| 1 | EL-6x16-20 | NA | | | | |
| 26 | Weight: 6981 # | | | | | |

**Wood Utility Poles**

Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of the property as follows: "The agreed or declared value of the property is specifically stated by the shipper to be not exceeding ______________ per ______________.

COD Amount: $

Fee terms: Collect ☐ Prepaid ☐ Customer check acceptable ☐

**Note: Liability limitation for loss or damage in this shipment may be applicable. See 49 USC § 14706(c)(1)(A) and (B).**

Received, subject to individually determined rates or contracts that have been agreed upon in writing between the carrier and shipper, if applicable, otherwise to the rates, classifications, and rules that have been established by the carrier and are available to the shipper, on request, and to all applicable state and federal regulations

The carrier shall not make delivery of this shipment without payment of charges and all other lawful fees.

Shipper Signature

SHIPPER Bell Lumber Pole Company

PER

Print Date 1/4/2022 Date Actual

**Trailer Loaded:**
☐ By shipper
☐ By driver

Internal Yard Doc:
NB-11496

**Freight Counted:**
☐ By shipper
☐ By driver

**Carrier Signature/ Pickup Date**

Carrier acknowledges receipt of packages and required placards. Carrier certifies emergency response information was made available and/or carrier has the DOT emergency response guidebook or equivalent documentation in the vehicle. Property described above is received in good order, except as noted.

Copies: White & Canary - Office Pink - Customer Goldenrod - Carrier

Figure D-1. Scupper Block and Glulam Rail, Test Nos. MGTR-1S and MGTR-1D (Item Nos. a1, a4, a6)

# Mead Lumber

**Lincoln**
1060 N 33rd Street, Suite F
Lincoln, Nebraska 68503

## Delivery Ticket

| | |
|---|---|
| **Order No** | **31470230** |
| **Order Date** | 05/28/2021 |
| Customer | 143799 |
| Contact Name | |
| Contact Number | |
| Your Ref | (6-14' SYP #1 Treated |
| Delivery | On 06/08/2021 |
| Taken By | PARKER K |
| Sales Rep | No Sales Rep |

**Invoice Address**

UNIVERSITY OF NE LINCOLN
***MASTER ACCOUNT***
PO BOX 880623
LINCOLN, NE, 68588

**Delivery Address**

UNIVERSITY OF NE LINCOLN
4630 NW 36 ST
LINCOLN, NEBRASKA, 68524

Page 1 of 1

**Special Instructions**

DELIVERY FOR UNIVERSITY OF NEBRASKA-LINCOLN
MIDWEST ROADSIDE SAFETY FACILITY
4630 NW 36ST
LINCOLN, NE 68524

TUESDAY 6/8/2021

**Notes**

CALL DUSTIN KOTIK BEFORE DEPARTURE FOR INSTRUCTION
402-472-5881

| Line | Item | Description | Qty Delivered | Qty BackOrdered | Qty Received |
|---|---|---|---|---|---|
| 1 | 2614T1 | 2X6-14 SYP #1 TREATED | 256 EA | | |

Driver ____________________

Date ____________________

All items listed have been received in good condition.by:

Print name ____________________

Signature ____________________

Subject to our terms and conditions of sale. Further copies available on request.

Figure D-2. 2-in. x 6-in. x 14-ft Long Treated, Dimensional Lumber, Test Nos. MGTR-1S and MGTR-1D (Item No. a3)

MPH

PAGE 01 OF 01
**REV DT: APR 02, 2018**

# MATERIAL CERTIFICATION

Central Steel & Wire Company

**P.O. Box 12100 - Detroit, MI 48212-0100**
1-313-368-5000 1-800-462-1950
(FAX 1-800-292-3878)

**Cincinnati, OH 45216-2355**
1-800-621-8510
(FAX 1-773-471-3962)

**P.O. Box 1154 - Milwaukee, WI 53201-1154**
1-414-481-5000 1-800-521-8031
(Fax 1-800-292-6459)

**P.O. Box 5100 - Chicago, IL 60680-5100**
1-773-471-3800 1-800-621-8510
(FAX 1-773-471-3962)

**P.O. Box 22015 - Greensboro, NC 274202-2015**
1-336-333-2332 1-800-621-8510
(FAX 1-800-232-9279)

**SOLD TO:** KATIE SCHOMER
T M C O INC
535 J ST
LINCOLN NE 68508-2935

**DATE (ORG):** APR 02, 2018
**CSW ORDER:** 866270
**CUSTOMER PO:** 0066603-00

64570

ITEM 001 .1233 (11 GA) X 60" X 120" GALVANIZED G90 STEEL 258#
SHEET TAG PART # 848-01103

## CHEMICAL COMPOSITION (%)

SOURCE: STEEL DYNAMICS SALES HEAT NO: 41743950 SYM: -

| C | MN | P | S | SI | NI | CR | MO | CU | AL | V | CB | CA | TI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| .030 | .230 | .011 | .001 | .0300 | .050 | .050 | .020 | .120 | .029 | .001 | .001 | .0020 | .002 |

| B | N |
|---|---|
| .0056 | .0080 |

WE CERTIFY THAT THE ABOVE MATERIALS BY US WILL CONFORM TO THE REQUIREMENTS AND SPECIFICATIONS OUTLINED.
Certifications furnished to the buyer by the seller describe the materials and/or services furnished, as indicated in the seller's records.
Results of mill tests implied in such certifications are based on standard mill practices and do not indicate each piece has been tested.

WE HEREBY CERTIFY THAT THE FOREGOING DATA IS A TRUE REPRESENTATION OF THE DATA FURNISHED TO US BY THE PRODUCING MILL.

Ryan Rathbun

RYAN RATHBUN - MANAGER
METALLURGY & TEST REPORTS
Central Steel & Wire Company

Figure D-3. Deck Anchor Plates, Test Nos. MGTR-1S and MGTR-1D (Item No. a3)

**EVRAZ** | EVRAZ INC. NA
Evraz Oregon Steel 14400 N. Rivergate Blvd., Portland, Oregon 97203

**REPORT OF CHEMICAL/PHYSICAL TESTS**

| CERTIFICATE NO. | DATE | PAGE |
|---|---|---|
| 1769748 | Mar 26, 2021 | 1 |

MILL ORDER NO.: 362697
DATE:
CUSTOMER ORDER NO.: 42834
JOB/REQ. NO.:
SHIPPING NO.: 1769748 DATE: 03/26/2021
CARRIER: BURLINGTON NORTHERN
CAR/TRUCK NO.: AOK607031

ISO 9001 REGISTERED
U.S. PLATE PRODUCER

SOLD TO:
TRINITY PRODUCTS LLC
1969 W TERRA LANE
O FALLON, MO 63366
USA

TRINITY PRODUCTS LLC
425 E. 151ST STREET
EAST CHICAGO, IN 46312
USA

THIS MATERIAL HAS BEEN MANUFACTURED, TESTED AND FOUND TO MEET THE SPECIFICATIONS AND PURCHASE ORDER REQUIREMENTS
CARBON STRUCTURAL QUALITY PLATE ASTM A36-19/ASME SA36 2019 ASTM A709-18 GRADE 36. KILLED FINE GRAIN PRACTICE.

**PHYSICAL PROPERTIES**

| ITEM NO | DESCRIPTION | HEAT NO. | SLAB | YIELD PSI X 100 | TENSILE PSI X 100 | % ELONG 8" | % ELONG 2" | % RA | HARDNESS BHN | BEND TEST | IMPACTS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.7500 X 72.000ME X 240.000 PT# PL.050.072.240.A36 | | | | | | | | | | |
| | 1 PC 3675 LBS +! | RS3132 | | 545 | 655 | 32 | | | | | |
| | 2 PCS 7350 LBS +! | RS3610 | | 625 | 710 | 28 | | | | | |
| | | | | 675 | 735 | 21 | | | | | |
| | 4 PCS 14700 LBS +! | RS3781 | | 660 | 795 | 24 | | | | | |
| | 5 PCS 18375 LBS +! | RS3825 | | 550 | 675 | 26 | | | | | |
| | | | | 515 | 680 | 29 | | | | | |

**CHEMICAL ANALYSIS**

| | HEAT NO. | C | Mn | P | S | Si | Cu | Ni | V | Cb | Al | Cr | Mo | Ti | B | N | Ca | | | CE | McQuaid Ehn Grain Size |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| +! | RS3132 | .05 | .83 | .011 | .007 | .25 | .30 | .10 | .002 | .023 | .023 | .08 | .04 | | | | | | | .24 | |
| +! | RS3610 | .05 | 1.25 | .009 | .009 | .23 | .27 | .09 | .004 | .035 | .044 | .08 | .04 | | | | | | | .31 | |
| +! | RS3781 | .06 | 1.23 | .012 | .003 | .20 | .27 | .10 | .003 | .059 | .045 | .12 | .09 | | | | | | | .33 | |
| +! | RS3825 | .05 | .83 | .011 | .001 | .25 | .32 | .11 | .003 | .023 | .043 | .10 | .03 | | | | | | | .24 | |

HEATS INDICATED WITH (!) WERE MELTED & POURED IN THE CANADA.
HEATS INDICATED WITH (+) WERE ROLLED IN THE USA.

I certify the above to be correct as contained in the records of EVRAZ INC. NA By [signature] Chemist

Figure D-4. Static Test Plates, Test Nos. MGTR-1S and MGTR-1D (Item No. a5)

**Portland Bolt**
**& MANUFACTURING COMPANY**

Phone: 800-547-6758 | Fax: 503-227-4634
3441 NW Guam Street, Portland, OR 97210
Web: www.portlandbolt.com | Email: sales@portlandbolt.com

**CERTIFICATE OF CONFORMANCE**

**For:** MIDWEST ROADSIDE SAFETY FACIL
**PB Invoice#:** 142079
**Cust PO#:** MGTD/MGTR
**Date:** 5/19/2021
**Shipped:** 5/20/2021

**We certify that the following items were manufactured and tested in accordance with the chemical, mechanical, dimensional and thread fit requirements of the specifications referenced.**

**Description:** 3/4 X 30 GALV ASTM A307A TIMBER BOLT

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Heat#:** | 1202025843 | | | **Base Steel:** | A36 | **Diam:** | .68 | | |
| **Source:** | NUCOR STEEL | | | | | **Proof Load:** | 0 | | |
| **C :** | .130 | **Mn:** | .680 | **P :** | .015 | **Hardness:** | 0 | | |
| **S :** | .034 | **Si:** | .230 | **Ni:** | .070 | **Tensile:** | 70,800 PSI | **RA:** | 48.00% |
| **Cr:** | .150 | **Mo:** | .020 | **Cu:** | .270 | **Yield:** | 51,300 PSI | **Elon:** | 27.00% |
| **Pb:** | .000 | **V :** | .002 | **Cb:** | .000 | **Sample Length:** | 8 INCH | | |
| **N :** | .000 | | | **CE:** | .2679 | **Charpy:** | | **CVN Temp:** | |

**Coatings:**
ITEMS HOT DIP GALVANIZED PER ASTM F2329/A153C

By: ______________________
Certification Department Quality Assurance
Dane McKinnon

Figure D-5. ¾-in.-10 UNC Timber Bolt, 30 in. Long, Test Nos. MGTR-1S and MGTR-1D (Item No. d5)

**INV#:MB19-137**

# CERTIFIED MATERIAL TEST REPORT

**Factory: SHENG DA-LI MACHINERY FACTORY** **Date: 2019-12-12**
**Item: ROUND WASHER** **Lot No: 9100963-02**
**Customer: BBI** **Finish:HDG.**
**Quantity Shipped:72CTNS.** **BBI/PO:B19100963**
**Sampling Plan per: 32510 MALLEABLE IRON** **Part No: P39086**
**Size & Description: 3/4"** **Heat No: 2019112802**

## Material Test Results
材 质 报 告

| Chemical Analysis（%） 化 学 成 分 | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| C | Si | Mn | P | S | | | | | |
| 2.55 | 1.52 | 0.51 | 0.053 | 0.053 | | | | | |

## Mechanical Properties Test Results
机 械 性 能 报 告

| | Standard Requirements 要求 | Test Results 检验结果 |
|---|---|---|
| Tensile Strength (Mpa) 抗 拉 强 度 | 345 | 363 |
| Yield Strength (Mpa) 屈 服 强 度 | 224 | 253 |
| Elongation (%) 延 伸 率 | 10 | 14.1 |

**All tests are in accordance with the methods prescribed in the applicable ASTM specification. We certify that this data is a true representation of information provided by the material supplier and our testing laboratory.**

**REMARK:** **1.The report is issued according to ISO16228 F3.1 (EN10204 3.1).**
**2.Test Facility: M**

王 団 训

**(Signature of Q.A. Lab Mgr.)**

Figure D-6. ¾-in. Malleable Iron Washer, Test Nos. MGTR-1S and MGTR-1D (Item No. d6)

**Portland Bolt**
& MANUFACTURING COMPANY

Phone: 800-547-6758 | Fax: 503-227-4634
3441 NW Guam Street, Portland, OR 97210
Web: www.portlandbolt.com | Email: sales@portlandbolt.com

**CERTIFICATE OF CONFORMANCE**

**For:** MIDWEST ROADSIDE SAFETY FACIL
**PB Invoice#:** 142135
**Cust PO#:** MGTR/MDTD
**Date:** 5/21/2021
**Shipped:** 5/27/2021

**We certify that the following items were manufactured and tested in accordance with the chemical, mechanical, dimensional and thread fit requirements of the specifications referenced.**

**Description:** 7/8 X 15 GALV ASTM F3125 GRADE A325 HEAVY HEX BOLT

**Heat#:** 3093334 **Base Steel:** 4140 **Diam:** 7/8

**Source:** COMMERCIAL METALS CO **Proof Load:** 39,250 LBF

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **C :** | .400 | **Mn:** | .810 | **P :** | .016 | **Hardness:** | 293 HBN | | | | |
| **S :** | .019 | **Si:** | .240 | **Ni:** | .190 | **Tensile:** | 67,180 LBF | | | **RA:** | .00% |
| **Cr:** | .870 | **Mo:** | .208 | **Cu:** | .320 | **Yield:** | 0 | | | **Elon:** | .00% |
| **Pb:** | .000 | **V :** | .024 | **Cb:** | .000 | **Sample Length:** | 0 | | | | |
| **N :** | .000 | | | **CE:** | .6329 | **Charpy:** | | | | **CVN Temp:** | |

LOT#19878

**Coatings:**
ITEMS HOT DIP GALVANIZED PER ASTM F2329/A153C

By: Certification Department Quality Assurance
Dane McKinnon

Figure D-7. ⅞-in.-9 UNC, 15-in. Long Heavy Hex Bolt, Test Nos. MGTR-1S and MGTR-1D (Item No. d7)

P#33187 C#17089822 L#1844804 7/8" Dia. Plain Round Washer

CERTIFIED MATERIAL TEST REPORT
FOR USS FLAT WASHERS HDG

FACTORY: IFI & Morgan Ltd REPORT DATE: 23/4/2019
ADDRESS: Chang'an North Road, Wuyuan Town, Haiyan,Zhejiang, China

MFG LOT NUMBER: 1844804

SAMPLING PLAN PER ASME B18.18-11 PO NUMBER: 170089822
SIZE: USS 7/8 HDG QNTY(Lot size): 7200PCS
HEADMARKS: NO MARK PART NO: 33187

DIMENSIONAL INSPECTIONS SPECIFICATION: ASTM B18.21.1-2011

| CHARACTERISTICS | SPECIFIED | ACTUAL RESULT | ACC. | REJ. |
|---|---|---|---|---|
| APPEARANCE | ASTM F844 | PASSED | 100 | 0 |
| OUTSIDE DIA | 2.243-2.280 | 2.246-2.254 | 10 | 0 |
| INSIDE DIA | 0.931-0.968 | 0.956-0.965 | 10 | 0 |
| THICKNESS | 0.136-0.192 | 0.136-0.157 | 10 | 0 |

| CHARACTERISTICS | TEST METHOD | SPECIFIED | ACTUAL RESULT | | ACC. | REJ. |
|---|---|---|---|---|---|---|
| HOT DIP GALVANIZED | ASTM F2329-13 | Min 0.0017" | 0.0017-0.0020 | in | 8 | 0 |

ALL TESTS IN ACCORDANCE WITH THE METHODS PRESCRIBED IN THE APPLICABLE ASTM SPECIFICATION. WE CERTIFY THAT THIS DAIA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY.
ISO 9001:2015 SGS Certificate # HK04/0105

IFI & MORGAN LTD. 检验专用章 QUANLITY CONTROL

(SIGNATURE OF Q.A. LAB MGR.)

Figure D-8. ⅞-in. Dia. Plain Round Washer, Test Nos. MGTR-1S and MGTR-1D (Item No. d8)

**McMASTER-CARR.** Packing List

600 N County Line Rd
Elmhurst IL 60126-2081
630-600-3600
chi.sales@mcmaster.com

University of Nebraska
Midwest Roadside Safety Facility
M W R S F
4630 Nw 36TH St
Lincoln NE 68524-1802
Attention: Shaun M Tighe
Midwest Roadside Safety Facility

Purchase Order
**E000869476**

Order Placed By
**Shaun M Tighe**

McMaster-Carr Number
**7470328-01**

Page 1 of 1

05/25/2021

| Line | Product | | Ordered | Shipped |
|---|---|---|---|---|
| 1 | **97801A111** | Steel Nails, 20D Penny Size, 4" Long, Packs of 125 | **10 Packs** | **10** |
| 2 | **3274T51** | Steel Oval Eye Nut - for Lifting, 7/8"-9 Thread Size | **2 Each** | **2** |

Certificate of compliance

This is to certify that the above items were supplied in accordance with the description and as illustrated in the catalog. Your order is subject only to our terms and conditions, available at www.mcmaster.com or from our Sales Department.

Sarah Weinberg
Compliance Manager

Figure D-9. ⅞-in.-9 UNC Eye Nut and 20D Nails, Test Nos. MGTR-1S and MGTR-1D (Item Nos. d9 and d10)

# Appendix E. Bogie Test Results

The results of the recorded data from each accelerometer for every dynamic bogie test are provided in the summary sheets found in this appendix. Summary sheets include acceleration, velocity, and deflection versus time plots as well as force versus deflection and energy versus deflection plots.

# MIDWEST ROADSIDE SAFETY FACILITY

**Bogie Test Summary**

**Test Information**
- Test Description: Montana Glulam Timber-
- Test Number: MGTD-1D
- Test Date: 4/12/2022
- Failure Type: N/A

**Post Properties**
- Post Type: NA
- Post Size: NA
- Post Length: NA
- Embedment Depth: NA
- Orientation: 0 Degrees

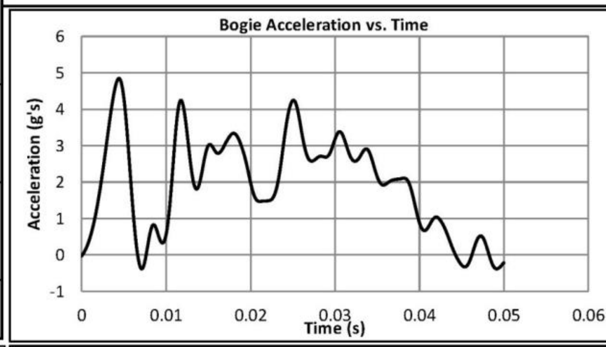
**Soil Properties**
- Gradation: N/A
- Moisture Content: N/A
- Compaction Method: N/A

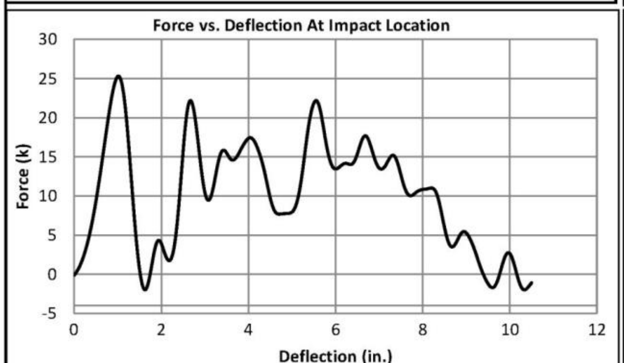
**Bogie Properties**
- Impact Velocity: 13.14 mph (19.28 ft/s)
- Impact Height: 16.375
- Bogie Mass: 5220 lb

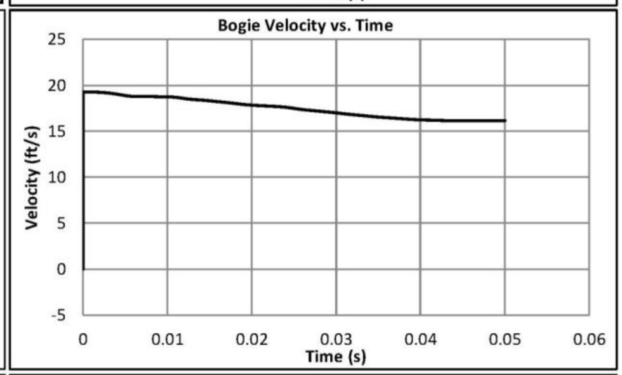
**Data Acquired**
- Accelerometer: SLICE-2
- Camera Data: AOS-8, AOS-11, AOS-12

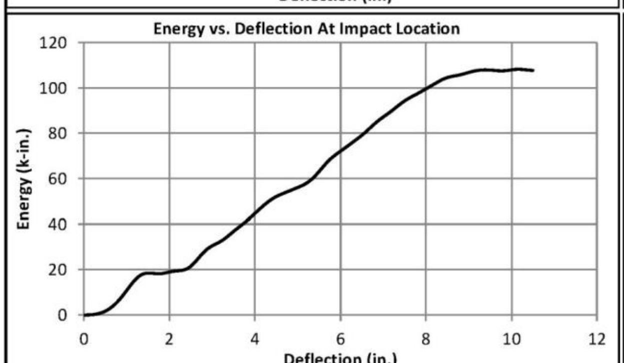
**Test Results Summary**
- Event Duration: 0.0500 sec
- Max. Deflection: 10.5 in.
- Peak Force: 25.3 k
- Initial Linear Stiffness: 25.0 k/in.
- Total Energy: 108.2 k-in.

| | @ 5" | @ 10" | @ 15" | @20" |
|---|---|---|---|---|
| Average Force (k) | 11.19 | 10.79 | NA | NA |
| Energy (k-in.) | 55.9 | 107.9 | NA | NA |



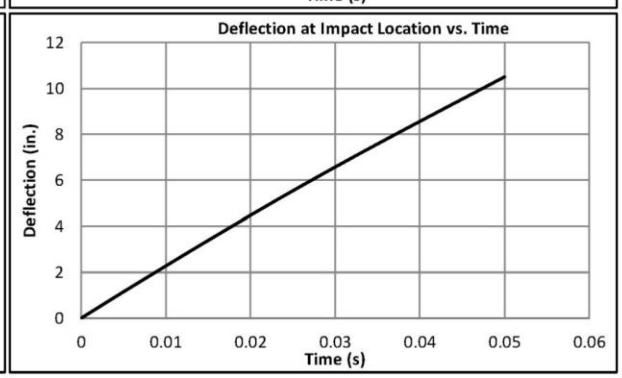




Figure E-1. Test No. MGTD-1D Results (SLICE-2)

# MIDWEST ROADSIDE SAFETY FACILITY

**Bogie Test Summary**

**Test Information**

| | |
|---|---|
| Test Description: | Montana Glulam Timber- |
| Test Number: | MGTR-1D |
| Test Date: | 5/4/2022 |
| Failure Type: | Rail fracture/splitting. |

**Post Properties**

| | |
|---|---|
| Post Type: | NA |
| Post Size: | NA |
| Post Length: | NA |
| Embedment Depth: | NA |
| Orientation: | 90 degrees - lateral impact |

**Soil Properties**

| | |
|---|---|
| Gradation: | N/A |
| Moisture Content: | N/A |
| Compaction Method: | N/A |

**Bogie Properties**

| | | |
|---|---|---|
| Impact Velocity: | 13.65 mph | (20.03 ft/s) |
| Impact Height: | 16.375 | |
| Bogie Mass: | 5220 lb | |

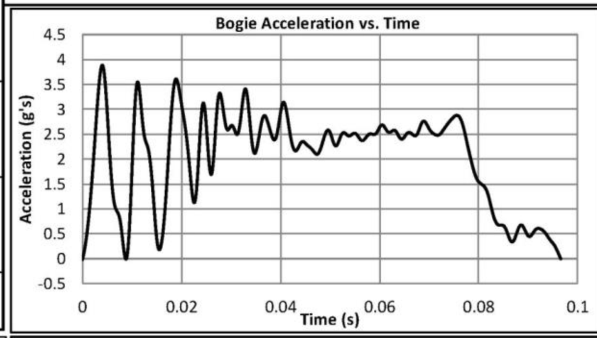
**Data Acquired**

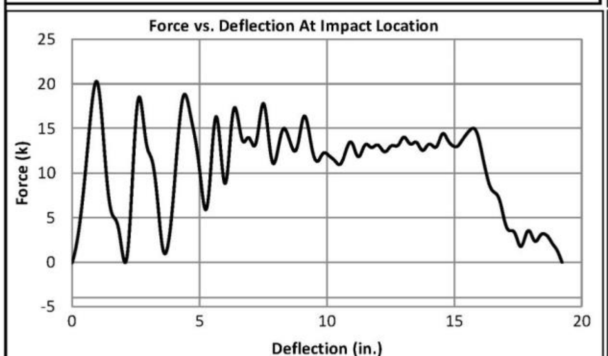
| | |
|---|---|
| Accelerometer: | SLICE-2 |
| Camera Data: | AOS-11, AOS-12 |

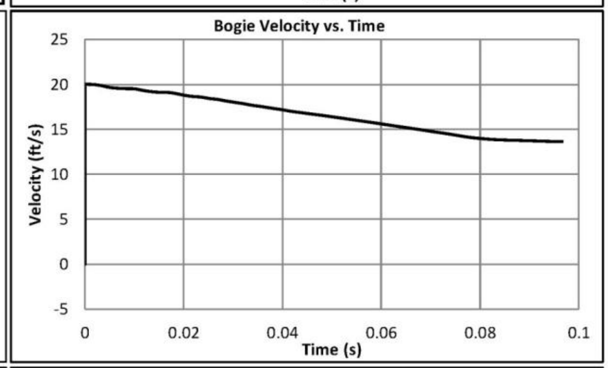
**Test Results Summary**

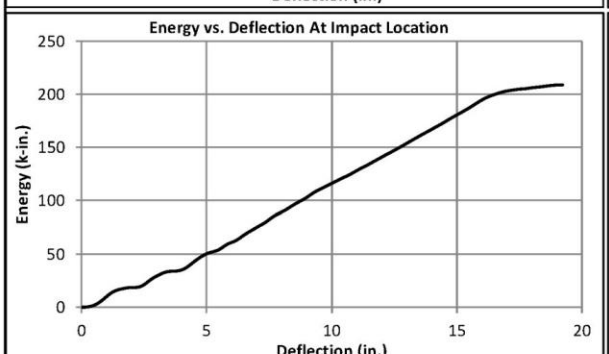
| | | |
|---|---|---|
| Event Duration: | 0.0966 | sec |
| Max. Deflection: | 19.2 | in. |
| Peak Force: | 20.3 | k |
| Initial Linear Stiffness: | 21.2 | k/in. |
| Total Energy: | 209.0 | k-in. |

| | @ 5" | @ 10" | @ 15" | @20" |
|---|---|---|---|---|
| Average Force (k) | 10.03 | 11.62 | 12.03 | NA |
| Energy (k-in.) | 50.1 | 116.2 | 180.4 | NA |



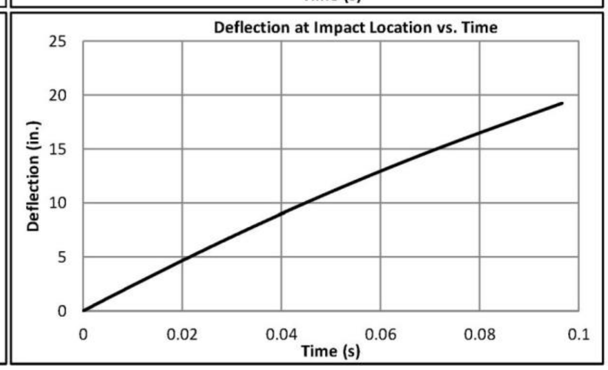




Figure E-2. Test No. MGTR-1D Results (SLICE-2)

# Appendix F. Load Cell and String Potentiometer Data – Static Testing Program

# MIDWEST ROADSIDE SAFETY FACILITY

Load Cell Summary

**Test Information:**

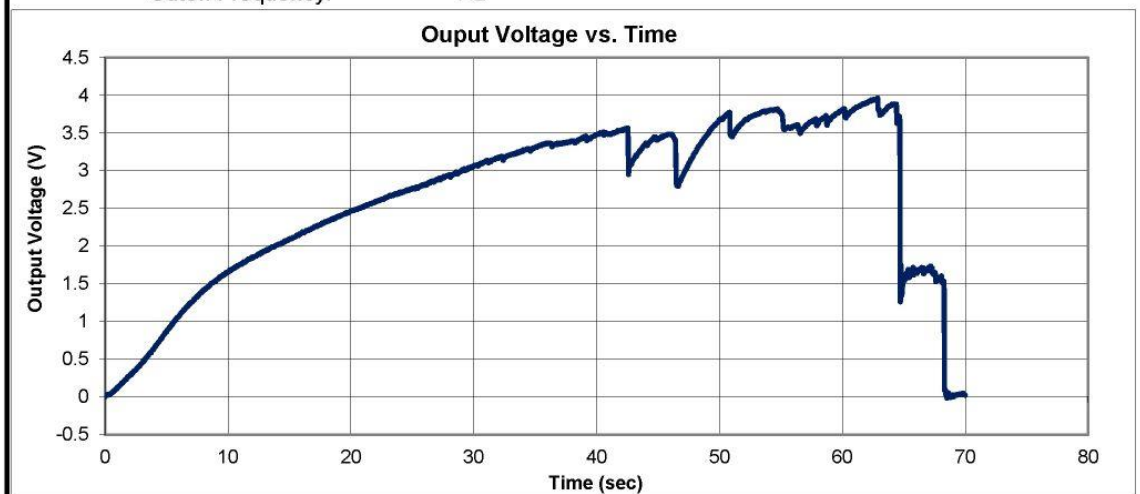
| | |
|---|---|
| Test No: | MGTD-1S |
| Date: | 4/15/2022 |
| System / Test Article: | Montana TL-1 Low-Height Glulam Rail |
| LC Location / Component: | 0 |
| Additional Notes: | |

**Load Cell Information:**

| | |
|---|---|
| Load Cell No.: | 143436 |
| Calibration Factor: | 2.1463 mv/V |
| Input Voltage (excitation): | 9.99 Volts |
| Gain: | 400 |
| Full Scale Load: | 50 kips |
| Sample Rate: | 25 Hz |
| Cutoff Frequency: | Hz |

**Results:**

| | |
|---|---|
| Preload: | 0 kips |
| Max. Load: | 23.11 kips |
| Time of Max. Load: | 62.88 sec |
| Event Duration: | 70 sec |
| Final Load: | 0.13 kips |



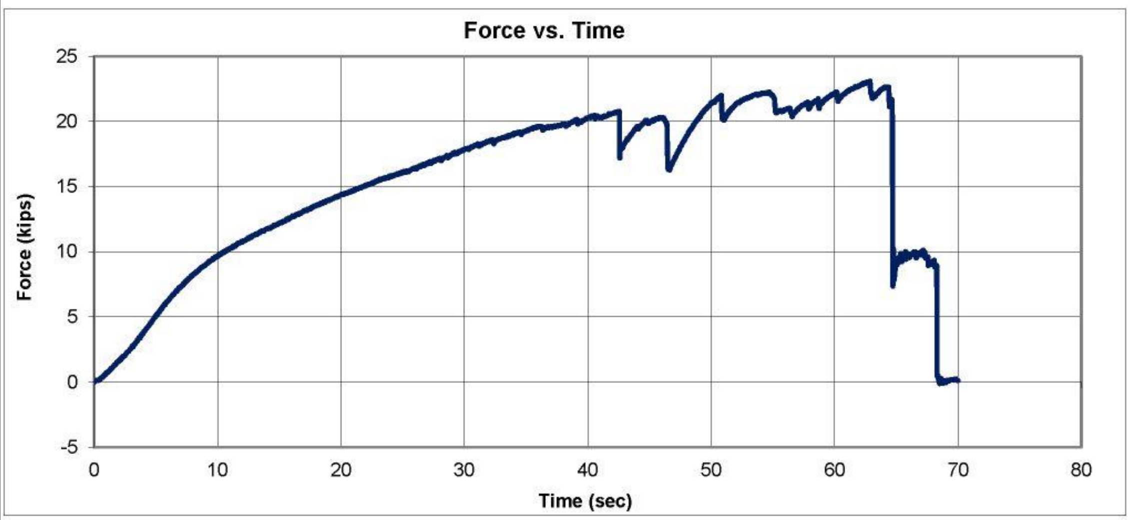




Figure F-1. Load Cell Data, Test No. MGTD-1S

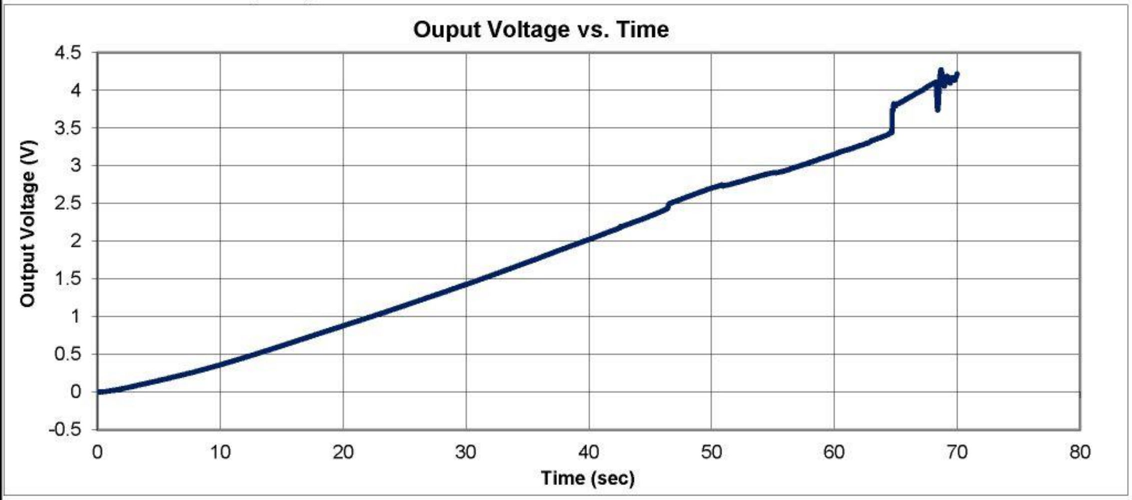
# MIDWEST ROADSIDE SAFETY FACILITY

String Potentiometer Summary

**Test Information:**

| | |
|---|---|
| Test No: | MGTD-1S |
| Date: | 4/15/2022 |
| System / Test Article: | Montana TL-1 Low-Height Glulam Rail |
| SP Location / Component: | 0 |
| Additional Notes: | |

| **String Potentiometer Information:** | | **Results:** | |
|---|---|---|---|
| String Pot No.: | 27039202 | Max. Displacement: | 22.74 in. |
| Calibration Factor: | 18.82133 mV/V/in. | Time of Max. Displacement: | 68.68 sec |
| Input Voltage (excitation): | 9.99 Volts | Event Duration: | 70 sec |
| Gain: | 1 | Final Displacement: | 22.44 in. |
| Full Scale Load: | 1 | | |
| Sample Rate: | 25 Hz | | |
| Cutoff Frequency: | Hz | | |



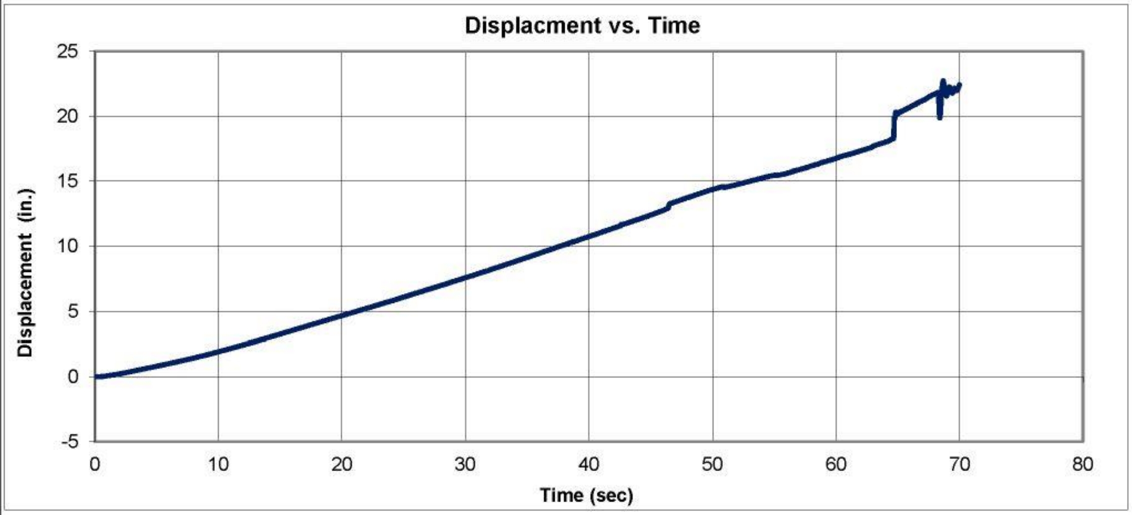




Figure F-2. String Potentiometer Data, Test No. MGTD-1S

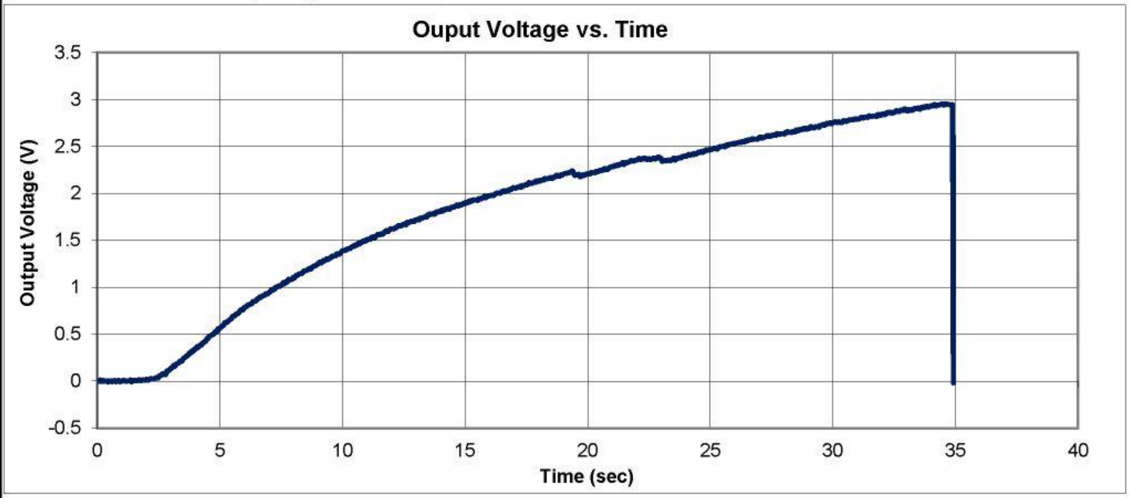
# MIDWEST ROADSIDE SAFETY FACILITY

Load Cell Summary

**Test Information:**

| | |
|---|---|
| Test No: | MGTR-1S |
| Date: | 5/3/2022 |
| System / Test Article: | Montana TL-1 Low-Height Glulam Rail |
| LC Location / Component: | 0 |
| Additional Notes: | |

| **Load Cell Information:** | | **Results:** | |
|---|---|---|---|
| Load Cell No.: | 143436 | Preload: | 0 kips |
| Calibration Factor: | 2.1463 mv/V | Max. Load: | 17.21 kips |
| Input Voltage (excitation): | 10 Volts | Time of Max. Load: | 34.48 sec |
| Gain: | 400 | Event Duration: | 34.92 sec |
| Full Scale Load: | 50 kips | Final Load: | -0.10 kips |
| Sample Rate: | 25 Hz | | |
| Cutoff Frequency: | Hz | | |



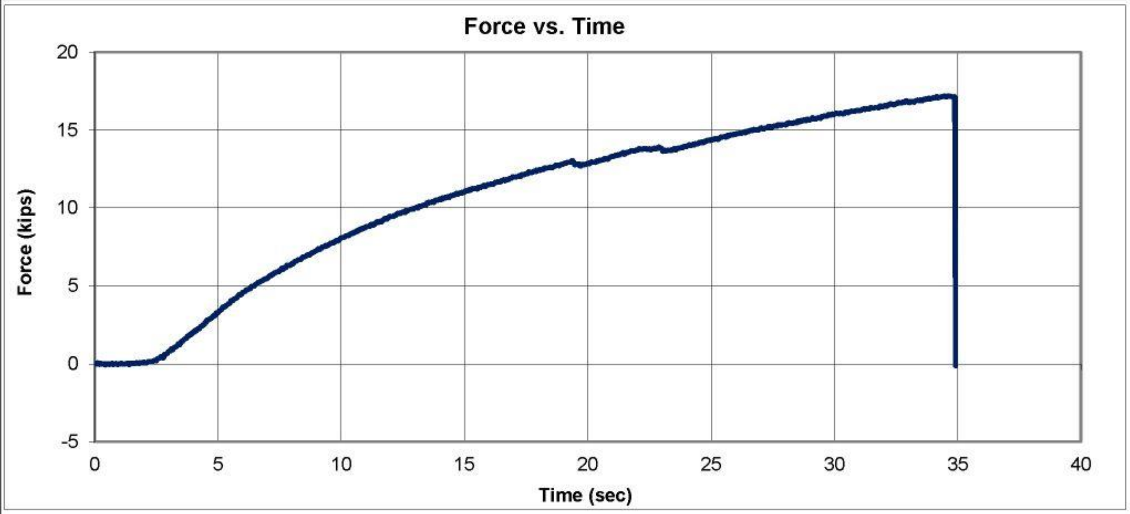




Figure F-3. Load Cell Data, Test No. MGTR-1S

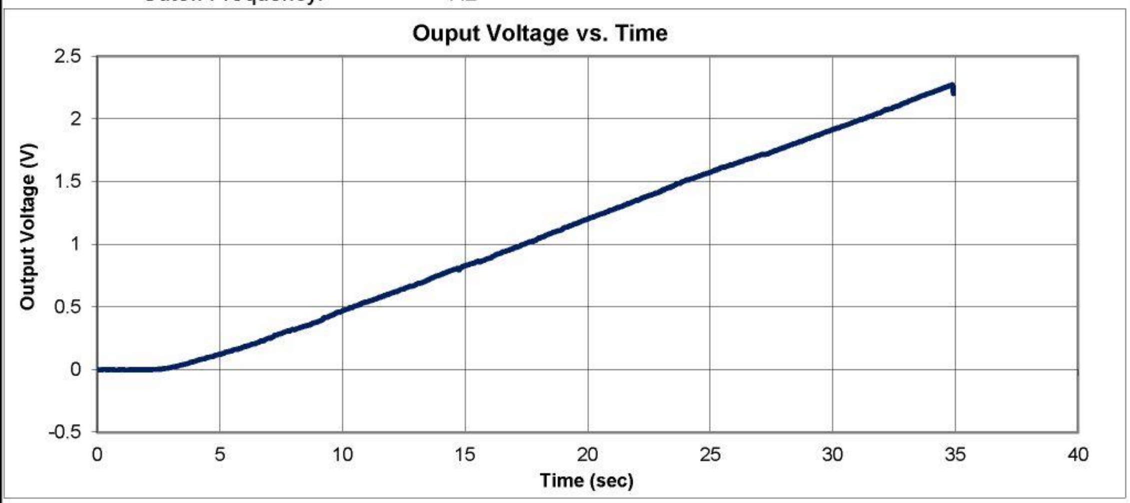
# MIDWEST ROADSIDE SAFETY FACILITY

String Potentiometer Summary

**Test Information:**

Test No: MGTR-1S
Date: 5/3/2022
System / Test Article: Montana TL-1 Low-Height Glulam Rail
SP Location / Component: 0
Additional Notes:

| **String Potentiometer Information:** | | | **Results:** | |
|---|---|---|---|---|
| String Pot No.: | 27039203 | | Max. Displacement: | 11.69 in. |
| Calibration Factor: | 19.4529 | mV/V/in. | Time of Max. Displacement: | 34.88 sec |
| Input Voltage (excitation): | 9.99 | Volts | Event Duration: | 34.92 sec |
| Gain: | 1 | | Final Displacement: | 11.34 in. |
| Full Scale Load: | 1 | | | |
| Sample Rate: | 25 | Hz | | |
| Cutoff Frequency: | | Hz | | |



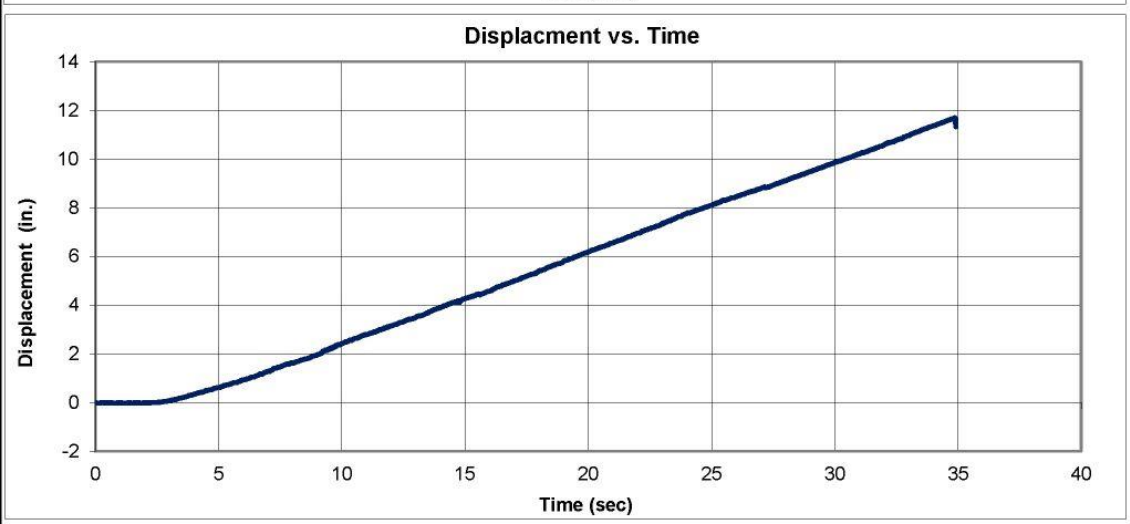




Figure F-4. String Potentiometer Data, Test No. MGTR-1S

# MIDWEST ROADSIDE SAFETY FACILITY

Load Cell Summary

**Test Information:**

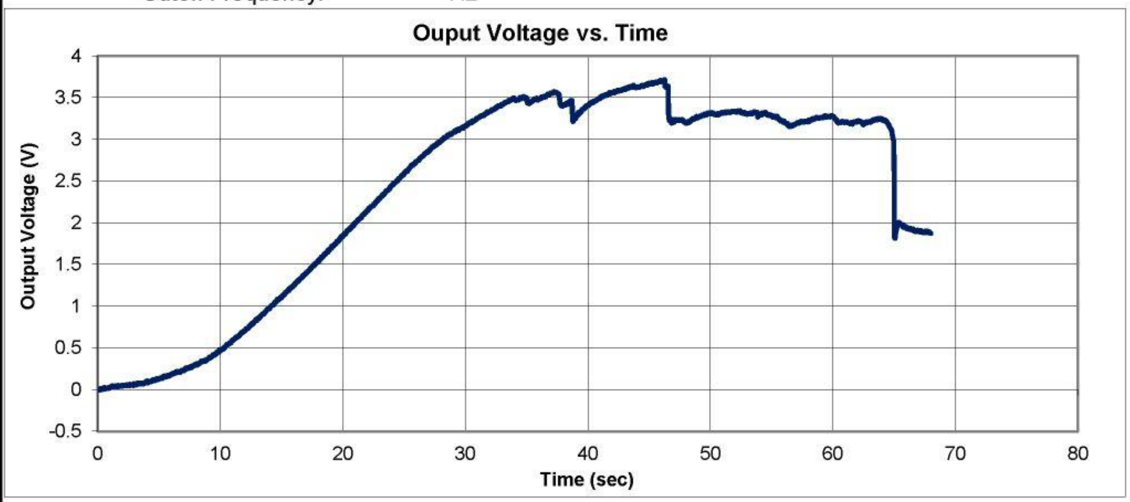
| | |
|---|---|
| Test No: | MGTR-1SB |
| Date: | 5/3/2022 |
| System / Test Article: | Montana TL-1 Low-Height Glulam Rail |
| LC Location / Component: | 0 |
| Additional Notes: | |

**Load Cell Information:**

| | |
|---|---|
| Load Cell No.: | 143436 |
| Calibration Factor: | 2.1463 mv/V |
| Input Voltage (excitation): | 10 Volts |
| Gain: | 400 |
| Full Scale Load: | 50 kips |
| Sample Rate: | 25 Hz |
| Cutoff Frequency: | Hz |

**Results:**

| | |
|---|---|
| Preload: | 0 kips |
| Max. Load: | 21.65 kips |
| Time of Max. Load: | 46.28 sec |
| Event Duration: | 68 sec |
| Final Load: | 10.91 kips |



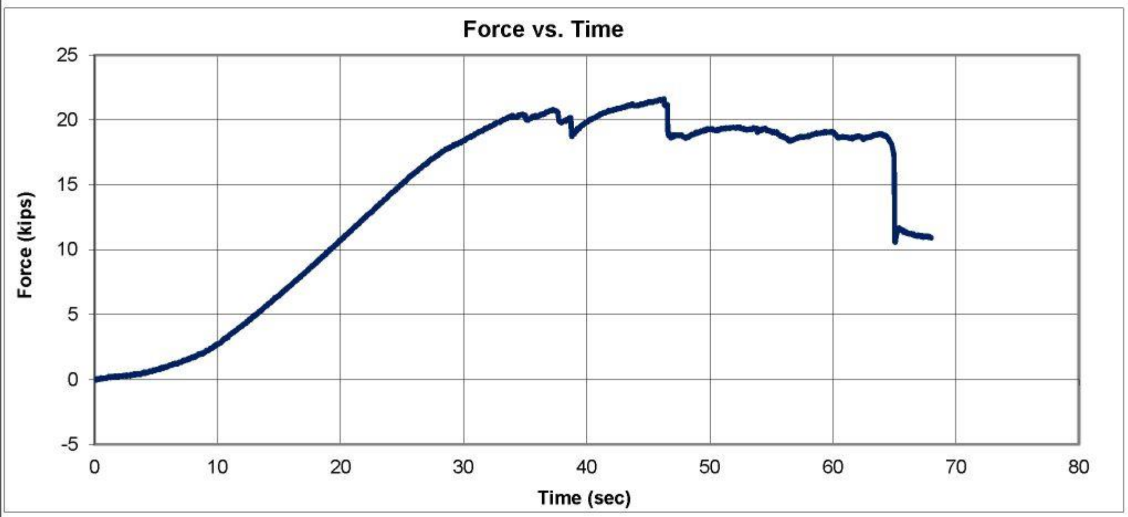




Figure F-5. Load Cell Data, Test No. MGTR-1SB

# MIDWEST ROADSIDE SAFETY FACILITY

String Potentiometer Summary

**Test Information:**

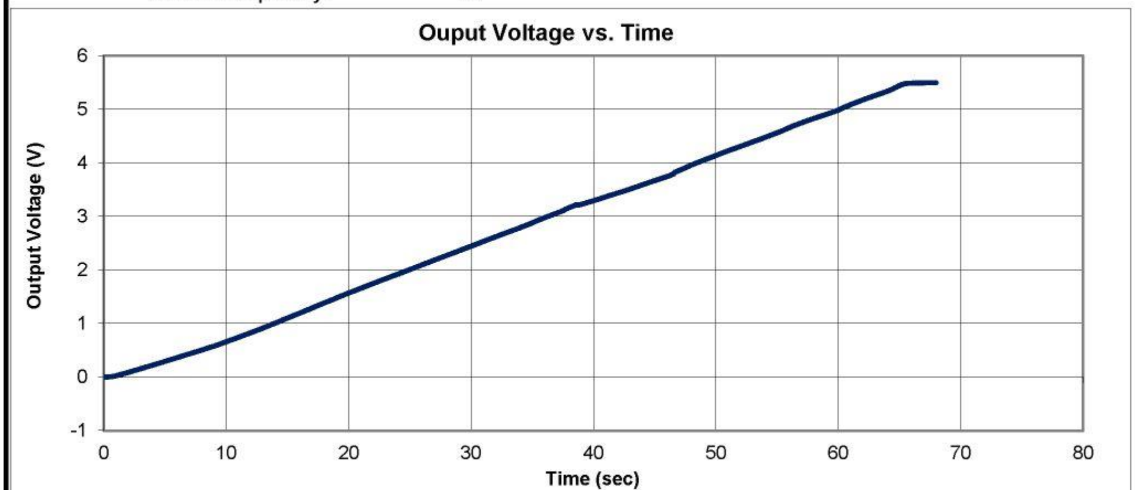
| | |
|---|---|
| Test No: | MGTR-1SB |
| Date: | 5/3/2022 |
| System / Test Article: | Montana TL-1 Low-Height Glulam Rail |
| SP Location / Component: | 0 |
| Additional Notes: | |

**String Potentiometer Information:**

| | |
|---|---|
| String Pot No.: | 27039203 |
| Calibration Factor: | 19.4529 mV/V/in. |
| Input Voltage (excitation): | 9.99 Volts |
| Gain: | 1 |
| Full Scale Load: | 1 |
| Sample Rate: | 25 Hz |
| Cutoff Frequency: | Hz |

**Results:**

| | |
|---|---|
| Max. Displacement: | 28.29 in. |
| Time of Max. Displacement: | 67.04 sec |
| Event Duration: | 68 sec |
| Final Displacement: | 28.29 in. |



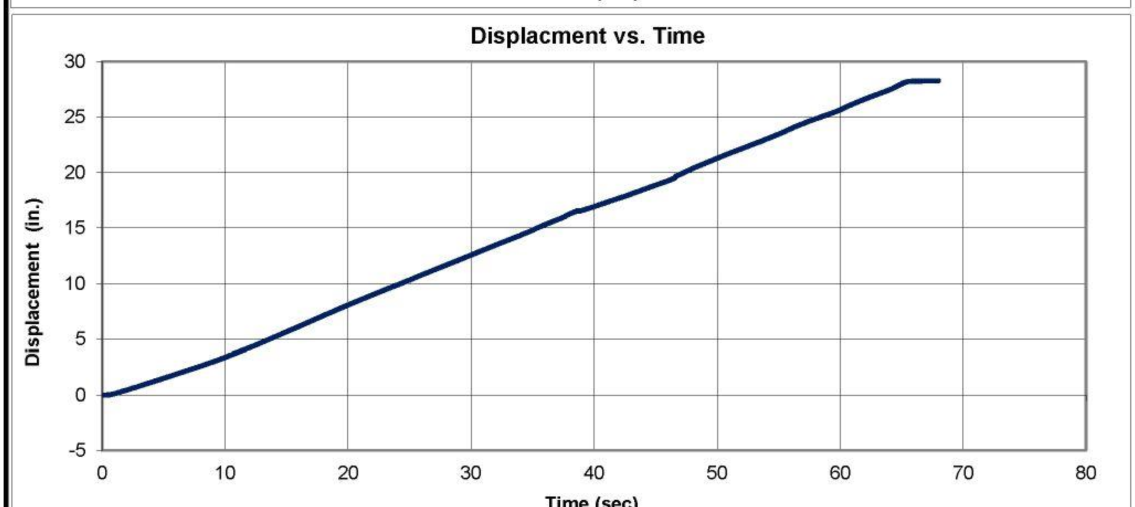




Figure F-6. String Potentiometer Data, Test No. MGTR-1SB

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