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STEEL RAILING, TYPE IL-OH: MASH EVALUATION AND IMPLEMENTATION GUIDANCE

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Over the past few years, the Midwest Roadside Safety Facility worked with the Illinois and Ohio Departments of Transportation to develop a new side-mounted, steel tube bridge rail to satisfy the American Association of State Highway and Transportation Officials (AASHTO) Manual for Assessing Safety Hardware, Second Edition (MASH) Test Level 4 (TL-4) safety performance criteria. The new bridge rail was designed to be adaptable to multiple bridge deck configurations, including cast-in-place concrete slabs and prestressed box-beam decks. The system was designed to remain crashworthy after the placement of a future roadway overlay up to 3 in. thick. The final design incorporated three HSS tube rails supported by W6x15 posts spaced at 8 ft on-center.
Additionally, a transition from the Midwest Guardrail System (MGS) to the new side-mounted, steel tube bridge rail was developed to satisfy MASH TL-3 criteria. The approach guardrail transition (AGT) incorporated the MGS upstream stiffness transition and 34-in. tall nested thrie-beam to account for future overlays. A distance of 9 ft was established between the last AGT post and the first bridge rail post in order to span over any structures that may prohibit post installation (abutments, wing walls, and/or drainage features) Specialized HSS end rails were developed to extend between the adjacent bridge and AGT posts and provide the strength required to redirect errant vehicles.
The development and full-scale testing of both the bridge rail and the transition were documented in previous reports. This report, the fourth in the series, summarizes the development and evaluation of the systems, contains CAD details and component variation descriptions, and provides implementation guidance for the Steel Railing Type IL-OH and its associated transition.</td></tr>
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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 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 and Problem Statement

Over the past several decades, the Illinois and Ohio Departments of Transportation (DOTs) have often installed steel, side-mounted, beam-and-post bridge rails on their elevated bridge superstructures. In 1993, the Illinois Side-Mounted Bridge Railing was successfully tested according to the *American Association of Highway Transportation Officials (AASHTO) 1989 Guide Specifications for Bridge Railings* Performance Level 2 (PL-2) [1-4]. This bridge rail consisted of W6x25 steel posts spaced at 6 ft – 3 in. on-center with two steel tube rails mounted to the front flange of the posts, as shown in Figure 1. Starting in 1999, the Ohio Twin Steel-Tube Bridge Rail was implemented utilizing W6x25 steel posts with dual steel tube rails, as shown in Figure 2 [5-6]. The Ohio rail was deemed to be acceptable under the Test Level 4 (TL-4) safety criteria found in *National Cooperative Highway Research Program (NCHRP) Report 350* [6-7]. Both systems are now considered *NCHRP Report 350* TL-4 compliant. They were designed without a curb to allow water to drain off the sides of a bridge, and the posts were mounted to the side of the bridge deck to maximize the traversable width of the bridge.

The *Manual for Assessing Safety Hardware* (MASH) [8, 9] is the current guideline for the evaluation of roadside safety hardware. Few side-mounted steel tube bridge rails have been tested to MASH evaluation criteria. Thus, it was desired to develop a MASH TL-4 side-mounted bridge rail similar to the sponsors' existing side-mounted bridge rails.

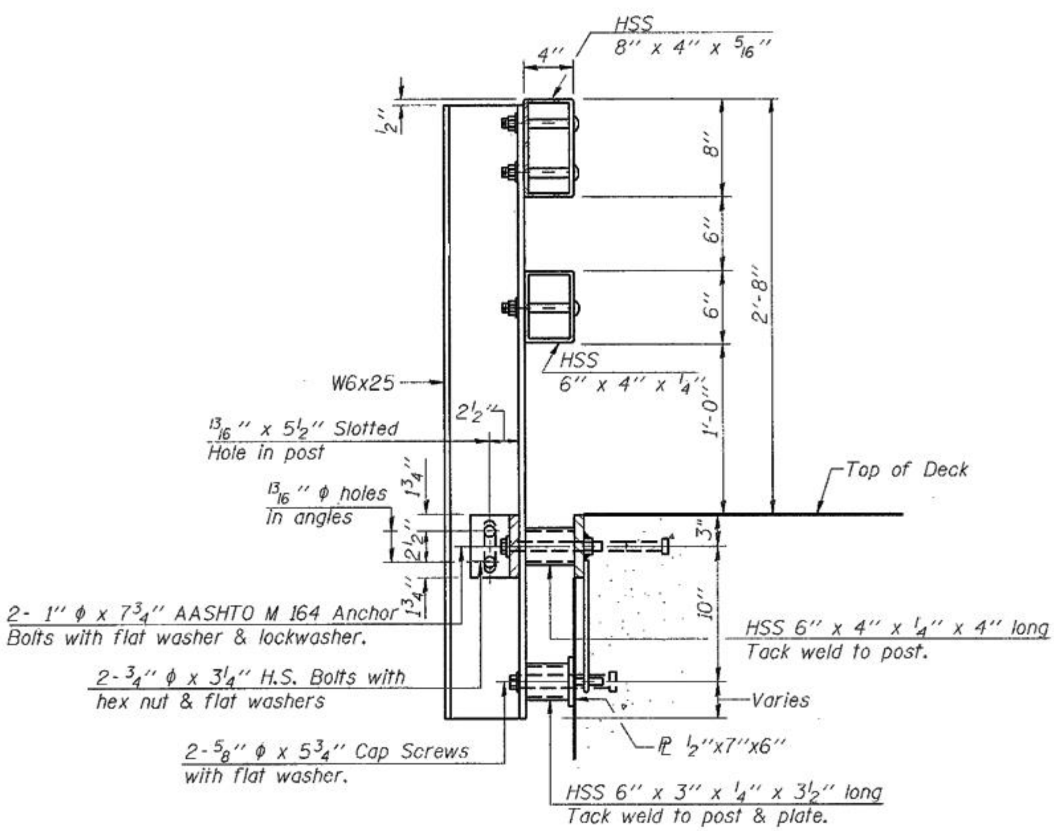


Figure 1. Illinois Side-Mounted Bridge Railing [2-4]

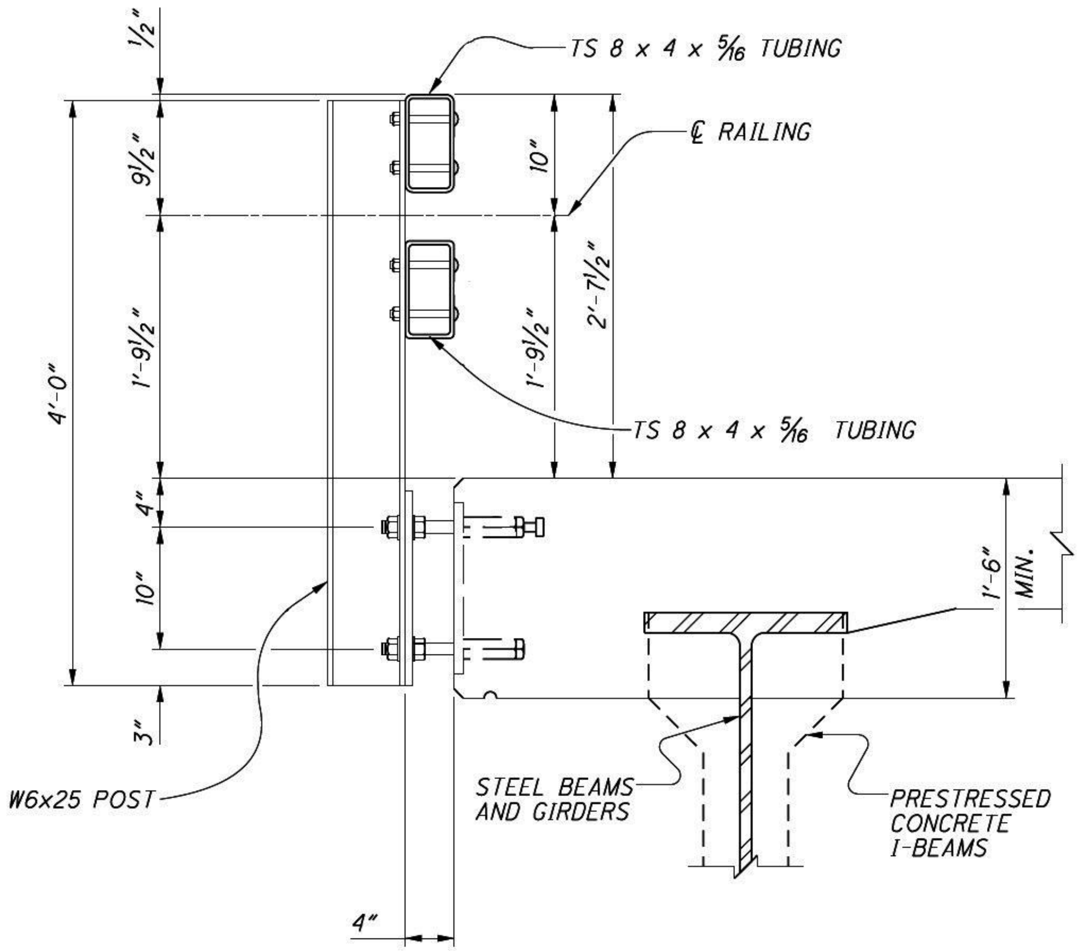


Figure 2. Ohio Twin Steel-Tube Bridge Rail [5-6]

Both Illinois DOT and Ohio DOT attached bridge rails to the side of their bridge decks. However, depending on the specific bridge, the posts may be attached to the side of either a thick concrete slab or a pre-stressed concrete box-beam girder. The Illinois DOT bridge deck configurations utilized slab bridges and concrete box-beam girders. The slab bridges had thickened deck edges that reduced to a thinner slab for the inner deck superstructure. The precast concrete box-beam girders had various widths and depths. The post anchorages for the box-beam girder had two installation options: (1) with the top anchors in the concrete wearing surface on top of the box-beam girder and the bottom anchors in the box-beam girders and (2) with the anchors connected to the box-beam girders, as shown in Figure 3. Note that either option can feature an additional asphalt wearing surface.

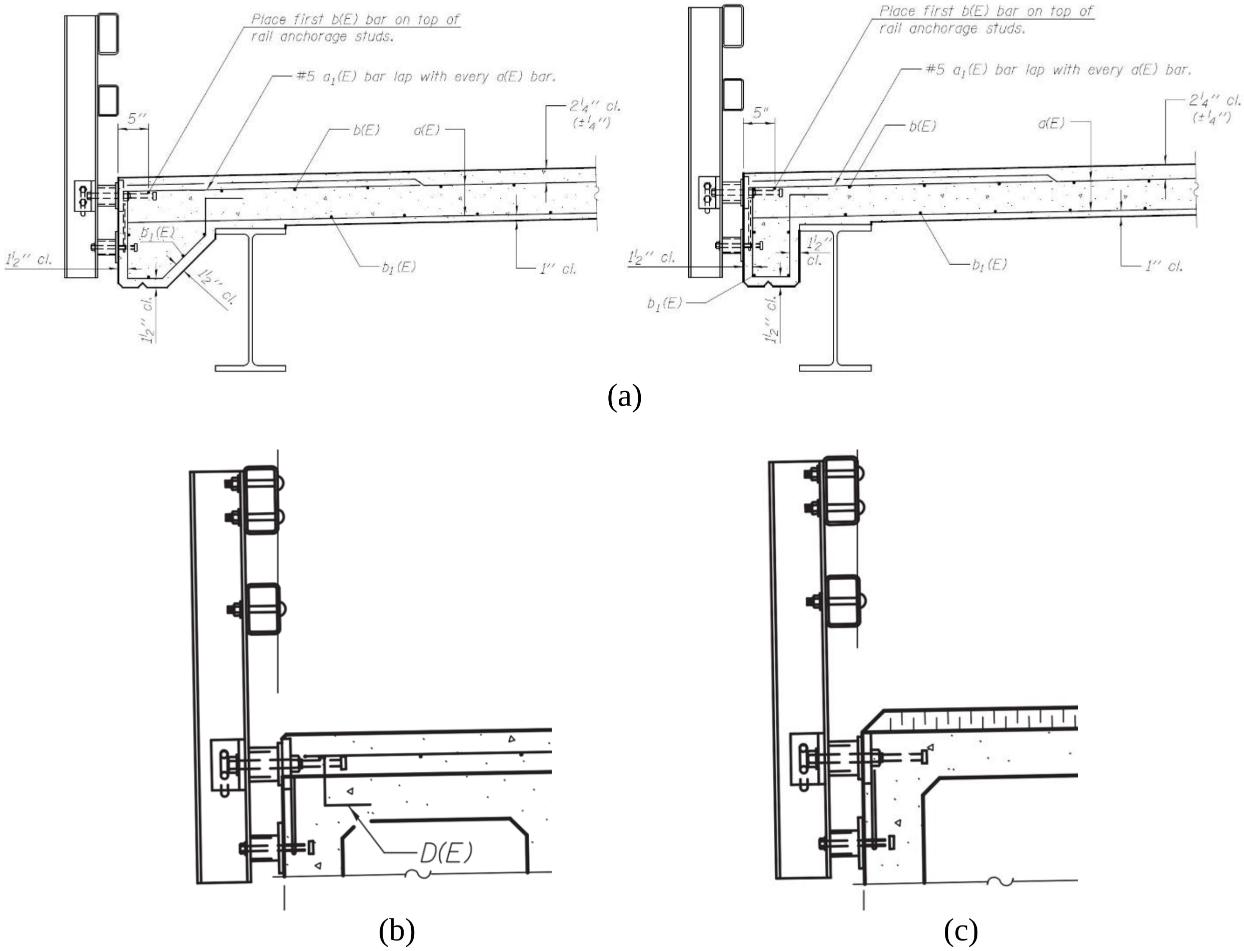


Figure 3. Illinois DOT (a) Bridge Slab, (b) Box Girder with Concrete Wearing Surface, and (c) Box Girder with Asphalt Wearing Surface [4]

Ohio DOT bridge slabs consisted of a thickened end slab deck or continuous bridge slabs with pre-stressed concrete I-beams or steel girders. Box-beam girder bridges were either composite beams with a concrete wearing surface on top of the beam or a non-composite box-beam with asphalt overlay. When anchors were installed in the box-beam girders, all anchors were located in the box-beam girders and not in the wearing surface. Anchorage types for bridge slabs and concrete box-beam girders for Ohio DOT are shown in Figure 4. It was desired that a new bridge rail be side-mounted and could be attached to the various bridge deck configurations utilized by Illinois and Ohio DOTs.

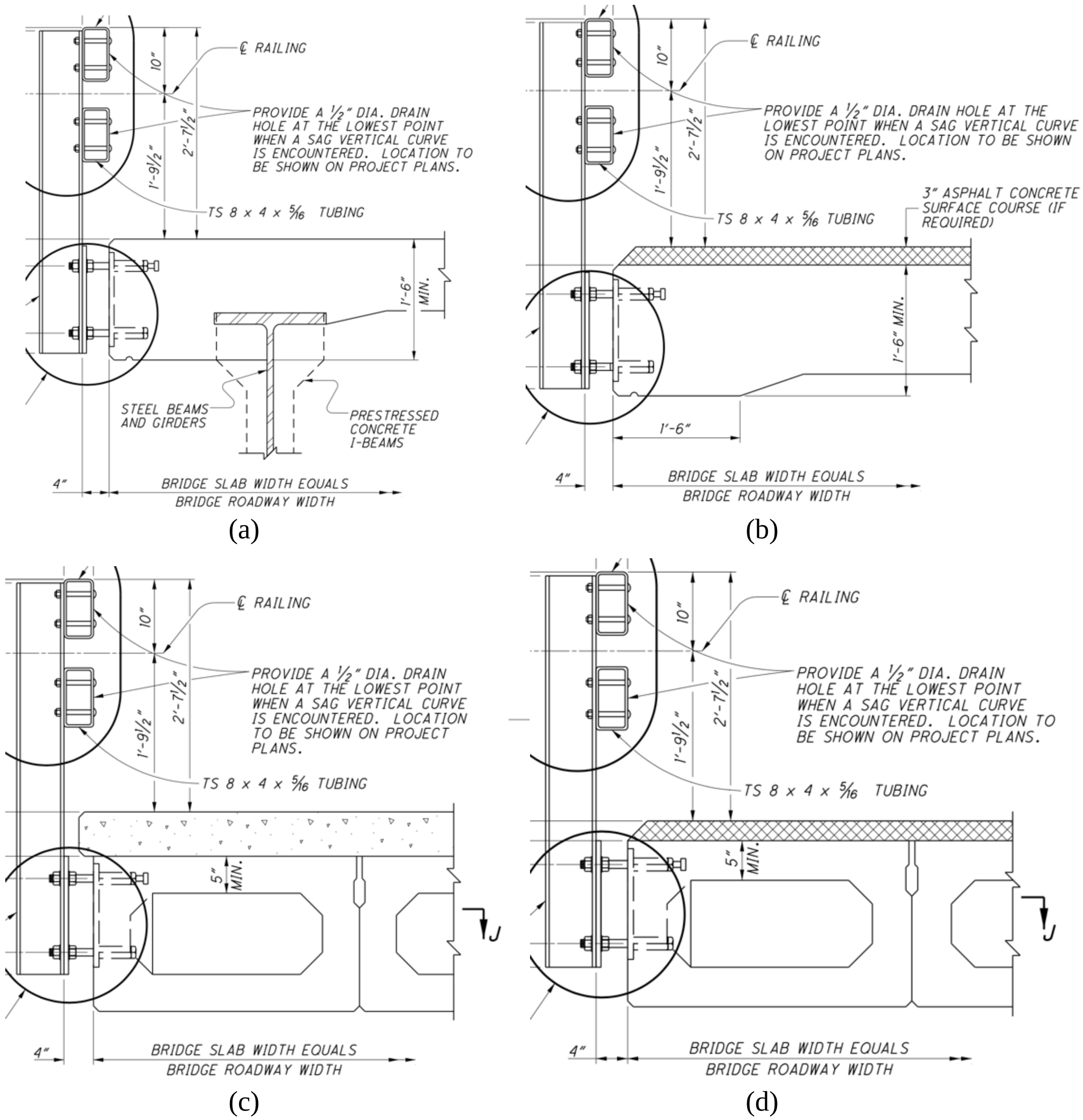


Figure 4. Ohio DOT (a) Bridge Slab, (b) Bridge Slab with Asphalt Wearing Surface, (c) Box Girder with Concrete Wearing Surface, and (d) Box Girder with Asphalt Wearing Surface [5]

1.2 Objectives

The objective of this project was to develop and evaluate a new side-mounted, steel tube bridge rail to satisfy MASH TL-4 safety performance criteria. The new bridge rail was designed to be adaptable to multiple bridge deck configurations, including cast-in-place concrete slabs and prestressed box beam decks. The system was to remain crashworthy after the placement of a future roadway overlay up to 3 in. thick. It was desired to utilize a maximum of three rail elements and to align the front faces of the steel tube rails with the edge of the bridge deck to maximize the traversable deck width. Finally, post sections smaller than W6x25 steel posts were desired to lower the impact loads transferred to the deck and mitigate bridge deck damage.

Additionally, a transition from the Midwest Guardrail System (MGS) to the new side-mounted, steel tube bridge rail was desired that would satisfy MASH TL-3 criteria. The transition was to utilize components from a previously-developed 34-in. tall, thrie-beam transition [10] from MGS to a rigid parapet with modification to attach the guardrail to the steel tube rails.

1.3 Project Scope and Summary

The development of the MASH TL-4 bridge rail and associated guardrail transition was conducted through a two-phase research effort. Phase I focused on the development and testing of the steel tube bridge railing and the post-to-deck anchorage connections [11-13], while Phase II consisted of the design and testing of an approach guardrail transition [14].

A post-to-deck connection was developed to be compatible with three deck configurations and to mitigate damage to the deck during an impact event [12]. Two critical deck configurations were identified for use during the design and evaluation of the bridge rail, post-to-deck attachment hardware, and bridge deck. Efforts were made to ensure that the attachment could transfer a load equivalent to the plastic bending strength of the posts to the deck while minimizing deck damage during impact events. Several concepts for the post-to-deck attachment were developed and evaluated through dynamic component testing.

Several rail locations, design impact loads, and critical deck configurations were evaluated to optimize the post and rail sections [11]. Bridge rail design methodologies were investigated to identify a suitable design process for the new bridge rail, and the AASHTO Post-and-Beam method was used to evaluate the strength of the railing [15]. Bridge railing configurations that mitigate the potential for vehicle snag and provide adequate capacity were developed. The bridge rail was then subjected to three full-scale crash tests and satisfied all MASH TL-4 safety criteria.

An approach guardrail transition from the MGS to the side-mounted, steel tube bridge rail was developed [14]. The transition was to utilize similar components from a previously-developed 34-in. tall, thrie-beam transition [10] and incorporate the previously-developed upstream stiffness transition [16-17]. LS-DYNA finite element analysis software was utilized to explore transition options and to select critical impact points for MASH testing. The transition was then successfully crash tested to both required tests in the MASH TL-3 evaluation matrix. A summary of the development and evaluation of the Steel Railing, Type IL-OH and its associated transition are contained herein along with final drawing details and implementation guidance.

2 POST-TO-DECK ATTACHMENT

2.1 Design

The post-to-deck connections previously used by the Illinois and Ohio DOTs to attach steel bridge railings to the side of their bridge decks were similar. Both designs incorporated a 4-in. offset between the edge of the deck and the face of the post, and both used four 1-in. diameter bolts that spanned across this offset to attach the posts to anchorage assemblies embedded into the deck edge. The anchorage assemblies were also similar in that both used six ¾-in. diameter by 6-in. long headed studs to anchor the upper tension bolts to the deck. However, these relatively shallow concrete anchor studs could not provide the strength required to support the full capacity of the W6x25 posts, and high-energy vehicle impacts often resulted in concrete anchorage failure and significant damage to the edge of the bridge deck. Examples of such damage are shown in Figure 5.

Figure 5. Post-to-Deck Anchorage Failure of Previous Connection Design

One of the design objectives for this project was to develop post-to-deck connection hardware that would mitigate potential damage to the deck. This was accomplished through two design modifications. First, the post size was reduced from a W6x25 to a W6x15, which reduced the plastic bending strength of the post by 43 percent. The reduced post strength would impart a reduced load to the anchorage in the deck and reduce the risk of deck damage while still providing enough strength for the steel bridge rail to satisfy MASH TL-4 conditions. Second, the six 6-in. long studded anchors were replaced with coupling nuts and threaded rods. The coupling nuts were used to directly transfer tension loads from the attachment bolts to the threaded rod anchors. As such, minimal loads would be imparted to the anchor plates. The threaded rods were extended 34 in. into the deck and more than doubled the concrete breakout strength of the previous 6-in. long anchor studs. Thus, the threaded rod anchorage system could support the full bending capacity of the W6x15 posts. Only the upper two bolts/anchors in the post-to-deck connection were expected to be subjected to high tensile loads during impacts. As such, the threaded anchor rods were only required at the upper two coupling nut locations. The lower coupling nuts were anchored via anchor plates and short bolts. Sketches of the anchorage hardware are shown in Figures 6 and 7.

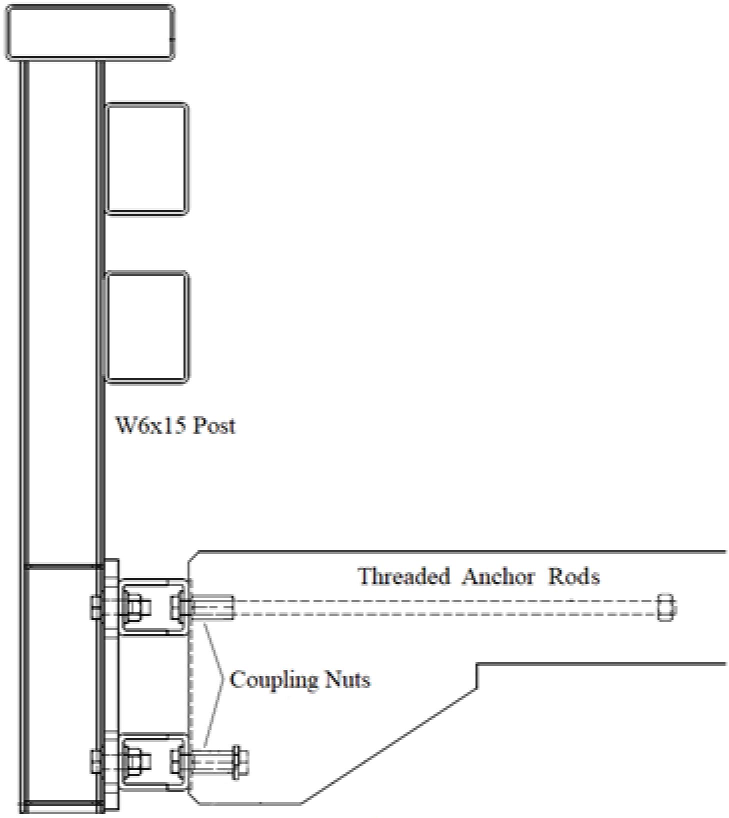


Figure 6. Anchorage Hardware Shown within Bridge Deck

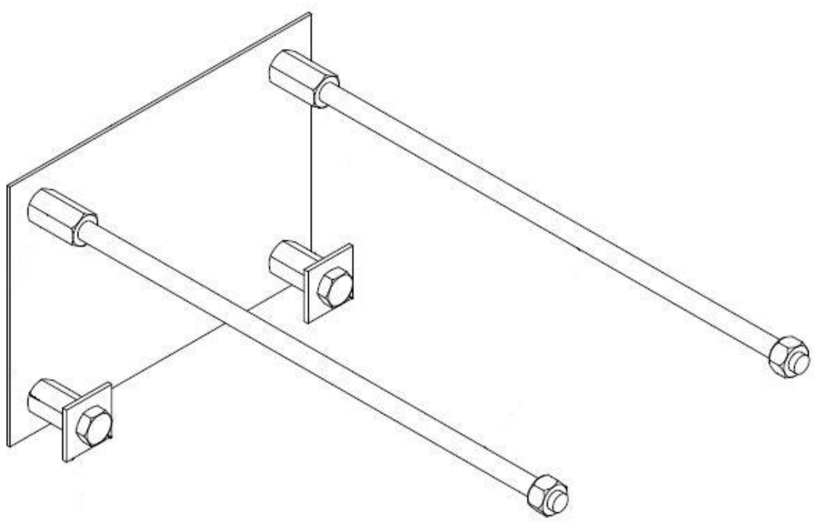


Figure 7. Isometric View of Post-to-Deck Anchorage Hardware

The new steel bridge railing was to be designed for use on both cast in place (CIP) bridge decks and prestressed concrete box beam decks. Further, a singular post-to-deck attachment configuration was desired for use on both deck types, as shown in Figure 8. As such, the vertical distance between the upper and lower anchor rods/bolts needed to be optimized to fit within all possible deck types. The optimum distance would maximize the vertical distance, which would minimize the magnitude of the tensile load applied to the deck while ensuring the upper anchor rods were placed below the top reinforcement steel in the deck and the bottom anchors were located at least 3 in. from the bottom of the deck. Although the typical thicknesses of CIP decks were much thinner, both Illinois and Ohio DOTs indicated that they would thicken the outer edges of their CIP decks to a minimum of 18 in. for attachment of the new side-mounted steel bridge rail. The prestressed concrete box beams in both states ranged in height from 17 in. to 42 in. Ultimately, a vertical distance of 11 in. was selected as optimum distance between the anchor rods/bolts that would accommodate all deck configurations.

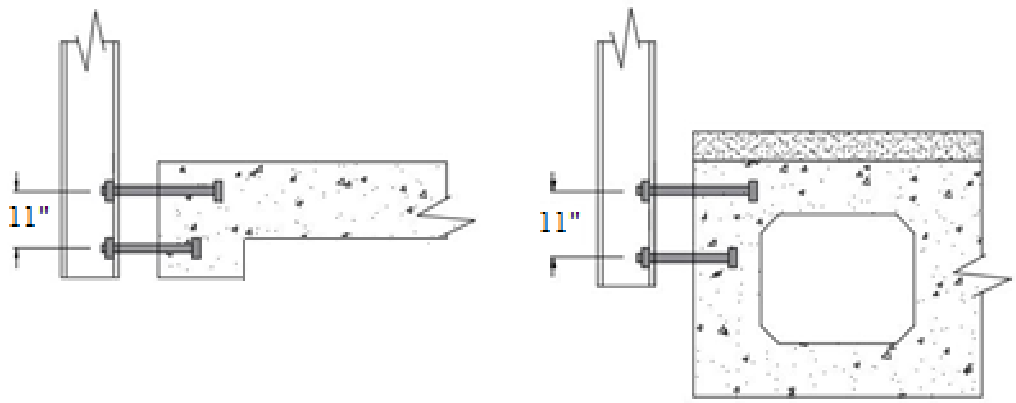


Figure 8. Same Anchorage Configuration for CIP and Prestressed Box Beam Decks

It was also desired for the face of the new steel bridge rail to be flush with the edge of the bridge deck to maximize the traversable width of the bridge. The new bridge rail design incorporated 6-in. deep HSS steel tubes attached to the face of the posts, so the post had to be offset 6 in. from the edge of the deck. Bolts extending across this 6-in. gap could be problematic as vertical loads in the bridge rail would result in bending loads applied to the bolts in addition to the high tensile and shear loads in the bolts. Thus, rectangular HSS sections were used as horizontal spacer tubes to fill the gap and transfer loads between the post and the bridge deck. The inner face of the HSS tube was bolted to the deck edge and the outer face was bolted an attachment plate welded to the face of the post, as shown in Figure 9. The ends of the HSS spacer tubes were cut at 45-degree angles to allow installation of the attachment bolts.

Figure 9. Photos of Assembled Post-to-Deck Connection [12]

An extensive analysis, which included structural analysis, LS-DYNA simulations, and dynamic component testing, was used to finalize the various components and hardware of the post-to-deck connection. A 17¾-in. x 13-in. x 1-in. thick attachment plate was welded to the front face of the W6x15 post, and ¼-in. thick gusset plates were welded at the top and bottom of the attachment plate and on both sides of the post to strengthen the post-to-plate attachment. Two HSS 5x4x½ steel tubes were used to space the post 6 in. from the edge of the deck. Four 1-in. diameter bolts were used to attach the post assembly to the HSS spacer tubes. The attachment plate contained 4¼-in. long vertical slots at each bolt location to allow for vertical construction tolerances in the post during assembly. Four additional 1-in. diameter bolts were used to attach the HSS spacer tubes to the coupling nuts embedded into the side of the deck. Threaded anchor rods extended 34 in. into the deck from the top two coupling nuts. The lower two coupling nuts were bolted to ¼-in. thick anchor plates. Further details on the post-to-deck connection are shown in Appendix A and discussed in detail within the previous report by Mauricio [12].

2.2 Component Testing and Evaluation

As mentioned in the previous section, dynamic component tests were conducted on various post-to-deck connection configurations as part of the design and evaluation of the connection. Tests were conducted with the post assemblies bolted to the sides of a 35-ft long prestressed concrete box beam measuring 42 in. tall and 36 in. wide. This prestressed box beam was selected as the critical deck configuration for three reasons:

1. The top slab of the box beams used in Illinois and Ohio was only 5½ in. thick, which was thinner than the 8 in. to 10 in. thickness of typical CIP decks. A reduced slab thickness would provide reduced anchorage capacity for the threaded anchor rods.

Thus, the critical slab thickness would be the thinnest of the possible deck configurations.

2. The concrete clear cover in the box beams was smaller than the clear cover used in CIP decks in Illinois and Ohio. The combination of clear cover and rebar sizes in the two deck types resulted in the threaded anchor rods being centered 3 in. from the top surface of a box beam and 4 in. from the top surface of CIP deck. The smaller distance would be more critical as the threaded anchor rods would be more likely to break out of the top of the deck.

3. The 5½-in. thick side walls of the box beams were susceptible to damage and crushing from the compression forces in the lower attachment bolts/HSS tubes as the post is loaded laterally. The effect of this crushing would be magnified as the height of the box beam and the side walls increased. The maximum height of prestressed concrete box beam deck girders in Illinois and Ohio was 42 in. Thus, the 42-in. tall box beam was the critical size to evaluate the potential for damage and/or crushing of the side wall.

Eight different post anchorage assemblies were embedded with the 35-ft long prestressed box beam so multiple dynamic component tests could be conducted on the same critical box beam. Each dynamic component test was conducted with a bogie vehicle impacting the post with a targeted speed of 20 mph at a height of 28 in. above the surface of the box beam. This height corresponded to the height of the middle rail of the system when installed without a wearing surface on the deck. Although box beam decks are not likely to be installed on actual bridges without a wearing surface, the lack of a wearing surface meant the posts would be shorter and the impact point would be lower and closer to the anchorage hardware. Thus, the impact loads required to plastically bend the posts would be maximized and represented the critical loading to the post-to-deck connection and anchorage hardware. The test setup is shown in Figure 10.

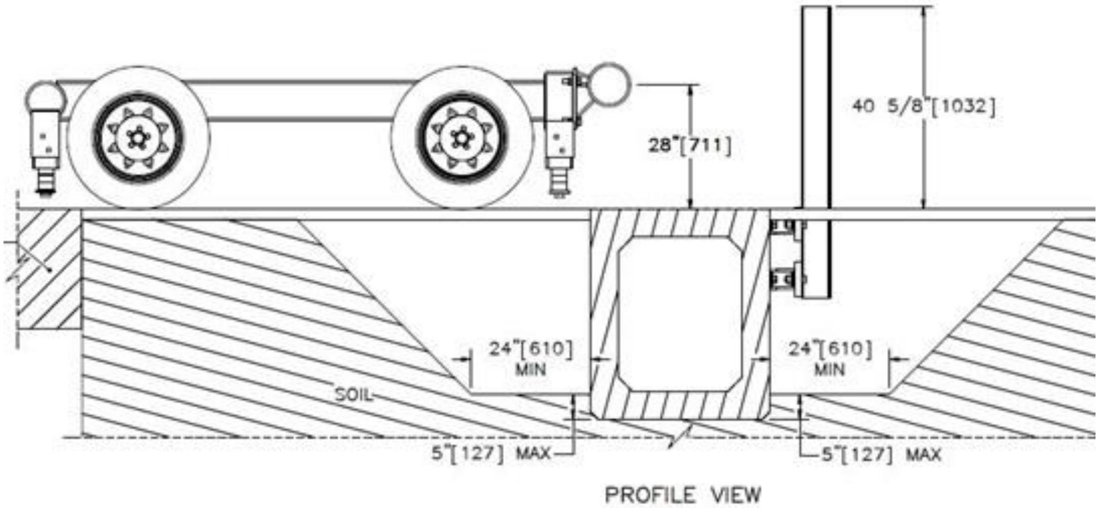


Figure 10. Test Setup for Dynamic Component Testing

The desired outcome for these dynamic component tests was to achieve plastic bending in the post without failure of any of the connection components, anchorage hardware, or the concrete box beam. A total of seven tests were conducted on various combinations of post assemblies and anchor configurations, and the post-to-deck connection was optimized through an iterative approach based on knowledge gained from previous tests. As shown in Figure 11, the finalized post-to-deck connection configuration (i.e., 1-in. thick attachment plate, four ¼-in. gussets, and 1-in. diameter attachment bolts and anchor rods) resulted in a plastic hinge forming in the W6x15 post just above the top of the attachment plate and the upper gusset plates. No damage or deformations were observed to the attachment bolts, HSS spacer tubes, or the anchorage hardware embedded in the side of the concrete box beam.

Figure 11. Post-Test Photographs of Selected Post-to-Deck Connection

Recall, a critical deck configuration was used during the component tests that minimized concrete cover around the threaded anchor rods and maximized the potential for crushing of the side wall of the box beam. Smaller box beams and CIP decks would provide greater resistance to side wall crush, and the increased slab thickness and clear cover of CIP decks would provide increased anchorage strength. Thus, the post-to-deck connection design was deemed appropriate for use with both the CIP decks and the prestressed concrete box beam decks commonly used by the Illinois and Ohio DOTs.

During a couple of the dynamic component tests, concrete spalling and cracking occurred around the two lower anchors. This unexpected damage was found to occur when the bogie overrode the top of the posts and the elastic strain in the post caused the post to snap forward. The forward motion of the post resulted in tensile loads being applied to the lower anchors of the test articles, which only included nuts and two concrete shear studs. To prevent this damage from occurring in the future, the bolts and anchorage plates shown previously in Figures 6 and 7 were added to the anchorage design. Note, these bolts and plates were included in the full-scale test article.

Finally, it was noted that anchor rods may have to be shortened at the ends of skewed bridges due to geometry constraints. As such, the last two component tests were conducted with shorter threaded anchor rods embedded within the solid end sections of the concrete box beam to evaluate the minimum required anchor length in solid sections. Test no. ILOH4-7 illustrated that threaded anchor rods extending 17 in. into the sides of the box beam end section provided sufficient strength to anchor the posts and may be used on the ends of skewed bridges.

Further details on the development, testing, and evaluation of the post-to-deck attachment design were provided in the research report by Mauricio [12].

3 BRIDGE RAIL

3.1 Design

Section 13 of the *AASHTO LFRD Bridge Design Specifications*, *8th Edition* [15] provides design loads for traffic barriers based on test level. For a TL-4 barrier, the transverse impact load, F_t, is 54 kips. However, this design load was originally determined for the TL-4 impact conditions specified by *NCHRP Report 350* [7], and Section 13 of the *AASHTO LFRD Bridge Design Specifications* has not yet been revised to include design loads for MASH barriers. The weight of the MASH TL-4 single-unit truck (SUT) increased by 4,400 lb and the impact speed increased by 6 mph as compared to NCHRP Report 350 TL-4 impact conditions. Thus, MASH TL-4 design loads were significantly higher than those used previously under NCHRP Report 350.

Researchers at the Texas A&M Transportation Institute recently conducted an LS-DYNA simulation study to evaluate barrier design loads under MASH impact conditions as part of NCHRP Project 22-20(2) [18]. MASH TL-4 impacts were simulated with a 10000S SUT impacting rigid barriers ranging in height from 36 in. to 90 in. As barrier height increased, the amount of roll experienced by the TL-4 truck decreased while the magnitude and effective height of the impact force increased. Subsequently, different TL-4 design loads were recommended for 36-in. tall barriers (designated TL-4-1) and barriers taller than 36 in. (designated TL-4-2), as shown in Table 1.

Recent studies have determined 36 in. to be the MASH TL-4 minimum barrier height to contain an SUT [19-20]. However, the new TL-4 bridge rail was also required to be crashworthy after future roadway overlays, which would effectively reduce the height of the bridge rail. Thus, the height to the top of the upper rail was required to be 39 in. at the time of initial installation and would result in a 36 in. height after a 3-in. overlay was applied to the bridge. Thus, 39 in. was selected as the desired barrier height.

The design loads for the barrier were based on those specified under designation TL-4-2 in Table 1 for a 39-in. tall barrier. However, those design loads were estimated from simulated impacts into rigid barriers. The steel beam-and-post bridge rail with reduced strength posts and an optimized rail configuration was predicted to deflect approximately 10 in. During barrier deflection, impact energy is absorbed, the impact pulse duration elongates, and the impact force magnitude is reduced. Therefore, the transverse design impact load was reduced to 65 to 70 kips in an aggressive design approach to develop an optimized bridge rail.

From the onset of the project, the new TL-4 bridge rail was envisioned to consist of three steel tube rails supported by W6x15 posts. Many of the existing TL-4 bridge rails incorporated W6x25 or larger posts. However, these larger posts used in other TL-4 bridge rails transferred high forces to the deck and often resulted in deck damage or anchor pullout during impact events. The use of W6x15 posts would reduce the force transferred to the deck while also reducing post weight and cost.

Table 1. Recommended MASH Design Impact Loads for Traffic Barriers [18]

Design Forces and Designations	TL-3	TL-4-1	TL-4-2
Rail Height, H (in.)	32	36	>36
F_t Transverse (kips)	70	70	80
F_L Longitudinal (kips)	18	22	27
F_v Vertical (kips)	4.5	38	33
L_t and L_L (ft)	4	4	5
L_v (ft)	18	18	18
H_e (in.)	24	25	30

F_t = Transverse force applied perpendicular to the barrier
F_L = Longitudinal force applied by friction along barrier's direction
F_v = Vertical force applied downward on the top of the barrier
L_t = Length of the transverse force
L_L = Length of the longitudinal force
H_e = Height of the peak force from ground level
L_v = Length of the vertical distributed design load

Mounting the upper rail on top of the W6x15 posts was desired as it provided multiple benefits over attaching the rail to the front of the post. First, a top-mounted rail would create a continuous surface along the top of the bridge rail and prevent vehicle snag on the top of the posts, a concern witnessed in prior testing of bridge rails with front-mounted upper rails [21]. Second, a top-mounted upper rail could have a deeper cross section than a front-mounted upper rail. Thus, a larger steel tube section could be utilized for the upper rail, which would result in a stronger and more cost-efficient bridge railing. Finally, it was desired to offset the upper rail 1 in. behind the face of the lower two rails to mitigate the possibility of the vehicle's passenger side window contacting the test article. Test article contact resulting in side-window fracture is a test failure according to MASH. A top-mounted upper rail can easily accommodate the offset.

The heights for the lower two rails were established to ensure vehicle stability and prevent vehicle snag on the posts. Simulations of MASH impacts have indicated that a minimum barrier height of 29 in. should be utilized to contain the 2270P pickup truck and maintain stability throughout the impact event [22]. Thus, a 32 in. top height was desired for the middle rail, which results in a 29 in. height after a 3-in. overlay, as it was desired for the two lower rails to redirect the pickup truck without contact to the top rail.

The height of the lower rail was determined by establishing a maximum gap below the lower rail to reduce snag potential on the posts. Table A13.1.1-2 in the *AASHTO LRFD Bridge Design Specifications* [15] as well as recent MASH crash tests involving the 1100C small car were reviewed, and a 12-in. maximum gap height was desired. Note, this gap was the height below the lower rail at initial installation. Future roadway overlays would reduce this height while also reducing snag potential on the posts. Both lower two rails were to be bolted directly to the front flanges of the posts without the use of support brackets.

Design of the bridge rail was conducted with an iterative approach utilizing standard steel sections and various post spacings. The three rails consisted of rectangular HSS steel tubes. It was desired that the lower two rails use identical sections between 4 in. and 6 in. deep (lateral distance), while the upper rail could be up to 12 in. deep and between 4 in. and 6 in. tall. The HSS rails were desired to have a minimum wall thickness of ¼ in. to prevent localized deformations and crushing. Post spacings of 6 ft, 8 ft, and 10 ft on-center were investigated. Finally, it was desired to limit the maximum weight of a single railing component to 500 lb, thus limiting the need for large construction equipment during installation.

Hundreds of possible railing configurations were considered, and the strength of each configuration was analyzed using the Post-and-Beam method described in Section 13 of the *AASHTO LRFD Bridge Design Specifications* [15]. The critical strength of the posts was determined with the post configured for attachment to a box beam deck with a 6-in. wearing surface and an additional 3-in. overlay. This critical configuration maximized the length of the post and the moment arm from the applied load to the post-to-deck attachment, thereby minimizing the force necessary to plastically deform the post. It was assumed that the post-to-deck attachment remained rigid and a plastic hinge formed in the post near the upper anchors. Additionally, the strength of the rails was reduced according to the loss of section from the bolt holes required to attach the rails to the posts. The results of this analysis were compared against the targeted design load of 65 to 70 kips.

Configurations were evaluated on both capacity and total weight, which was used as an indication of installation costs [11]. The selected bridge rail configuration consisted of W6x15 posts spaced at 8 ft, an HSS12x4x¼ upper rail, and two HSS8x6x¼ lower rails, as shown in Figures 12 and 13. The tube rails were spliced together using internal splice tubes, which were fabricated to fit tightly with the rails and transfer both shear and moment across the joints. Final system drawings are shown in Appendix A. The selected bridge rail configuration had an estimated capacity of 67 kips.

Figure 12. Steel Railing, Type IL-OH Photographs

Figure 13. Steel Railing, Type IL-OH Photographs

3.2 MASH Evaluation

Longitudinal barriers, such as steel bridge railings, must satisfy impact safety standards in order to be eligible for reimbursement by the Federal Highway Administration for use on the National Highway System. For new hardware, these safety standards consist of the guidelines and procedures published in MASH [8]. According to TL-4 of MASH, longitudinal barrier systems must be subjected to three full-scale vehicle crash tests, as shown in Table 2.

Table 2. MASH TL-4 Crash Test Conditions for Longitudinal Barriers

Test Article	Test Designation No.	Test Vehicle	Vehicle Weight (lb)	Impact Conditions		Evaluation Criteria [1]
				Speed (mph)	Angle (deg.)	
Longitudinal Barrier	4-10	1100C Small Car	2,420	62	25	A,D,F,H,I
	4-11	2270P Pickup Truck	5,000	62	25	A,D,F,H,I
	4-12	10000S Single Unit Truck	22,000	56	15	A,D,G

[1] Evaluation criteria explained in MASH [8]

Evaluation criteria for full-scale vehicle crash testing are based on three appraisal areas: (1) structural adequacy; (2) occupant risk; and (3) vehicle trajectory after collision. These evaluation criteria are defined in greater detail in MASH [8].

The TL-4 steel bridge rail described herein was designed to be compatible with multiple deck configurations as well as future roadway overlays. As a result, multiple configurations of the bridge rail can exist with various post lengths and rail heights relative to the roadway. Thus, critical bridge rail configurations needed to be identified for full-scale crash testing.

MASH test designation no. 4-12 with the 10000S SUT would result in the highest impact loads to the barrier and likely cause the most rail deformation. Thus, the structurally weakest configuration represented the worst-case scenario. The bridge railing configuration for placement on a box-beam girder with a 6-in. wearing surface and a 3-in. roadway overlay resulted in the longest post configuration and the longest moment arm from the impact load to the post-to-deck attachment. This configuration, which resulted in a top rail height 36 in. above the roadway surface and a 12 in. distance from the roadway surface to the center of the upper anchor bolts, had the weakest posts and the lowest capacity. Subsequently, this configuration was selected as the critical configuration for MASH test designation no. 4-12. If the weakest configuration proved crashworthy, the other design configurations on other analyzed deck configurations should also be crashworthy.

Impacts with the 2270P vehicle were also expected to produce high impact loads with the design loads for TL-3 impacts being within 10 kips of the TL-4 impact loads, as shown previously in Table 1. Thus, a weak configuration would also be critical for MASH test designation no. 4-11.

Additionally, the lower rail heights relative to the roadway resulting from an asphalt overlay may lead to vehicle roll and instability. Thus, the critically weak railing configuration selected for MASH test designation no. 4-12 was also selected as the critical configuration for MASH test designation no. 4-11.

Impacts with the 1100C small car typically have lower magnitude impact loads compared to the other MASH vehicle impacts. However, the small car is more susceptible to extending under the lower rail and snagging on system posts, which may lead to vehicle instabilities or excessive decelerations. Thus, bridge rail configurations without the overlay and with the largest vertical opening gap were critical in the evaluation of the system with the small car, as they maximized the gap below the lower rail. Further, the strongest post configuration, or the configuration with the shortest distance between the post-to-deck anchor bolts and the roadway surface, would cause the highest decelerations from vehicle snag. Thus, the railing configuration for use on a slab deck without an overlay (i.e., a 39 in. top rail height and a 4 in. distance between the roadway surface and the center of the upper anchor bolts) was selected as the critical configuration for MASH test designation no. 4-10.

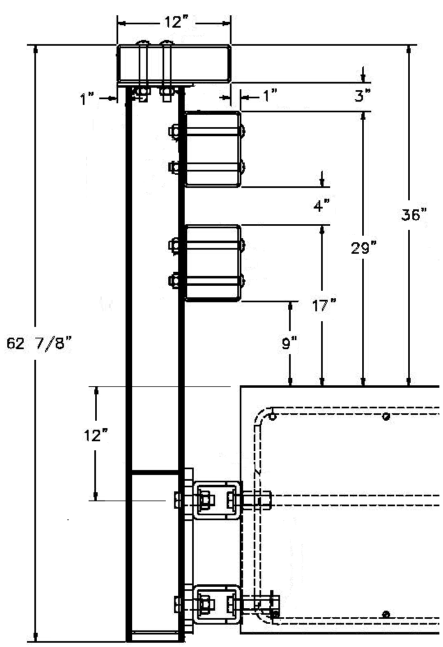
MASH specifies that post-and-beam longitudinal barriers may have two potential critical impact points, one associated with wheel snagging and pocketing on a post (i.e., hard point) and another that induces a maximum loading to a critical portion of the system, such as a rail splice [8]. When splices are coincident with a hard point, a single test can be conducted to evaluate both critical points. Since the rail splices within the new bridge rail were centered 2 ft away from the centerline of the posts, it was believed that vehicle snagging on a post and/or splice as well as maximum loading on a splice could be evaluated with one test of each of the two passenger vehicle types.

For the small car and pickup truck crash tests, previous testing and computer simulations of impacts into post and beam systems have demonstrated that critical impact points are often controlled by the wheel snagging on a post. MASH provides charts for determining critical impact points for test designation nos. 4-10 and 4-11 based on the strengths of the posts and rails comprising the barrier. Subsequently, MASH Figures 2-14 and 2-17 were used to determine the critical impact points for test designation nos. 4-10 and 4-11, respectively. From those charts, the small car and pickup truck critical impact points were determined to be 5 ft upstream from a post and 7 ft upstream from a post, respectively. For the SUT crash test, the impact location was chosen to maximize loading into critical railing components, such as rail splices. According to MASH Table 2-8, the critical impact points for a post-and-beam bridge rail impacted by a SUT should be 5 ft upstream from a rail splice location. With splices located 2 ft from posts, the CIP for the SUT was 7 ft upstream of a post.

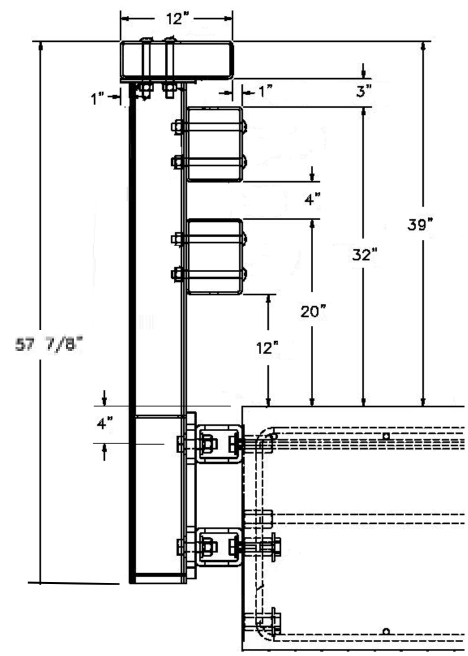
As described above, two different bridge rail configurations were deemed critical for the full-scale testing of the TL-4 steel bridge rail. MASH test designation no. 4-10 was critical with an upper rail height of 39 in. and the upper anchor bolts centered 4 in. below the roadway surface, while MASH test designation nos. 4-11 and 4-12 were critical with an upper rail height of 36 in. and the upper anchor bolts centered 12 in. below the roadway surface. As opposed to constructing two independent deck and railing systems, a surrogate hybrid deck design was constructed to accommodate both configurations. The hybrid deck was 26 in. thick to accommodate post-to-deck attachment anchors at two different heights along its free edge. One series of attachment locations had the upper anchors 4 in. below the top surface, while a second series of attachment locations had the upper anchors located 12 in. from the top surface. The attachment types alternated at 4-ft intervals along the length of the deck, which resulted in each system having an 8-ft post spacing. The surrogate deck and critical railing configurations are shown in Figure 14. As a result of using the surrogate deck, deck damage was not fully evaluated during the crash testing program. However, the testing of the post-to-deck connection hardware on a critical box-beam configuration illustrated the low risk of deck damage during impact events.

(a)

(b)



(c)



(d)

Figure 14. Test installation: (a) Photograph from Front of Bridge Rail, (b) Photograph from Back of Bridge Rail, (c) Critical Railing Configuration for MASH Test Designation Nos. 4-12 and 4-11, and (d) Critical Railing Configuration for MASH Test Designation No. 4-10

The test installations were 160 ft long and consisted of the hybrid deck, W6x15 posts, and three rectangular HSS tube rails. The posts were attached to the deck utilizing a welded attachment plate, HSS5x4x½ spacer tubes, and 1-in. diameter A325 bolts, as described previously. The HSS tube rails were bolted to the posts as shown previously in Figure 14. All the HSS rail sections were 16 ft long and spliced together with 30-in. long splice tubes. The splice tubes were fabricated from welded plates and were slid into place within the tube rails. Rail splices were located 2 ft from a post, or at a quarter span location. The surrogate bridge deck was only 108 ft long, so the system posts were top-mounted to the surface of the concrete tarmac on the downstream end of the installation. However, all crash tests occurred on the bridge deck with the side-mounted posts. The top-mounted posts were not evaluated and were only utilized to provide sufficient system length to evaluate vehicle stability.

In test no. STBR-1, the 22,124-lb SUT impacted the system at an angle of 14.5 degrees and a speed of 53.6 mph. According to MASH, the target impact speed is 56.0 mph with a tolerance of ± 2.5 mph, which was met, and the target impact angle is 15 degrees with a tolerance of ± 1.5 degrees, which was met. Although the test was within the limits for individual test parameters, the combination of the impact speed and the impact angle resulted in an impact severity of 133.2 kip-ft, which was below the MASH allowable limit of 142.0 kip-ft. The bridge rail properly contained and redirected the SUT. However, since the impact severity fell below the MASH requirement, the MASH test designation no. 4-12 was repeated in test no. STBR-4 to obtain a higher impact severity.

In test no. STBR-2, the 5,157-lb Dodge quad cab pickup truck impacted the bridge rail at a speed of 64.5 mph and an angle of 24.6 degrees. The critical impact point was selected to be 84 in. upstream from post 9 to maximize wheel snagging on a post and to maximize loading to a rail splice. Upon impact, posts 7 through 11 deflected backward with posts 8 and 9 plastically deforming near the upper web stiffeners. The pickup truck became parallel to the system at 0.146 seconds after impact and exited the system at 0.326 seconds after impact. The vehicle remained upright and stable throughout the impact event.

The maximum lateral dynamic barrier deflection was 7.0 in. at the upper rail splice between posts 8 and 9, and the maximum lateral permanent set of the barrier system was 3.5 in. The working width of the system was 19.0 in. Moderate concrete spalling was found at the bottom edge of the concrete deck at post 9, and hairline concrete cracks occurred at the top-left and top-right corners of the embedded plate of post 10. This spalling was believed to be caused by a tension force that occurred in the bottom anchors after plastic deformation occurred in the post and the system tried to restore after the impact event. The spalling was repaired for subsequent tests by adding a continuous plate spanning between the two lower anchors behind the stirrups and pouring new concrete. The damage to the vehicle was moderate, with damage concentrated on the left-front corner, left-front fender, and left side of the box where the impact occurred.

The analysis of the results for test no. STBR-2 showed that the system adequately contained and redirected the 2270P vehicle with controlled lateral displacements of the barrier. A summary of the test results is shown in Figure 15. All safety performance criteria for test designation no. 4-11 were within acceptable limits defined in MASH. Therefore, test no. STBR-2 was determined to be acceptable according to MASH test designation no. 4-11.

In test no. STBR-3, the 2,569-lb Kia Rio small car impacted the bridge rail at a speed of 62.0 mph and an angle of 24.8 degrees. The critical impact point was selected to be 60 in. upstream from post 7 to maximize wheel snagging on a post. Upon impact, posts 6 and 7 deflected backward and the impact-side front tire snagged on post 7. The car became parallel to the system at 0.164 seconds after impact and exited the system at 0.228 seconds after impact. The vehicle remained upright and stable throughout the impact event.

The maximum lateral dynamic barrier deflection was 2.9 in. at the upper rail between posts 6 and 7, and the maximum lateral permanent set of the barrier system was 0.6 in. The working width of the system was 15.2 in. Minimal concrete spalling and hairline cracks occurred around posts 6 and 7. Damage to the vehicle was minimal, with damage concentrated on the left front-corner of the vehicle where the impact occurred.

The analysis of the test results for test no. STBR-3 showed that the system adequately contained and redirected the 1100C vehicle with controlled lateral displacements of the barrier. A summary of the test results is shown in Figure 16. All safety performance criteria for test designation no. 4-10 were within acceptable limits defined in MASH. Therefore, test no. STBR-3 was determined to be acceptable according to MASH test designation no. 4-10.

In test no. STBR-4, the 22,152-lb 2007 Freightliner M2 106 SUT impacted the bridge rail at a speed of 56.4 mph and an angle of 14.7 degrees. The critical impact point was selected to be 60 in. upstream from the splice between posts 6 and 7 to maximize loading to the rail splice. Upon impact, posts 5 through 11 deflected backward, with posts 6 through 8 plastically deforming at the upper post web stiffeners. The SUT redirected away from the bridge rail and became parallel to the system at 0.300 seconds after impact. The truck then rolled onto its left side while remaining on the traffic side of the barrier. The cargo box landed on top of the rail and then exited the system 1.862 seconds after impact.

The maximum lateral dynamic barrier deflection was 7.9 in. at the upper rail between posts 6 and 7, and the maximum lateral permanent set of the barrier system was 7.3 in. The working width of the system was 87.7 in. due to the box extending over the rail. Minimal concrete spalling and hairline cracks occurred around posts 6 and 7. The damage to the vehicle was moderate, with damage concentrated on the left front-corner of the vehicle where the impact occurred. Occupant risk values are not required evaluation criteria for test designation no. 4-12. However, the occupant risk values were calculated with the same procedure used for the 1100C and 2270P vehicles in order to make comparisons.

The analysis of the test results for test no. STBR-4 showed that the system adequately contained and redirected the 10000S vehicle with controlled lateral displacements of the barrier. The test vehicle was contained and redirected with the box riding along the top rail of the system, and although the vehicle rolled onto its left side, it did so on the traffic side of the bridge rail, which is acceptable. A summary of the test results is shown in Figure 17. All safety performance criteria for test designation no. 4-12 were within acceptable limits defined in MASH. Therefore, test no. STBR-4 was determined to be acceptable according to MASH test designation no. 4-12.

The complete MASH TL-4 test matrix (test nos. STBR-2 through STBR-4) was successfully conducted on Steel Railing, Type IL-OH, as shown in Table 3. Therefore, Steel

Railing, Type IL-OH met all MASH safety performance criteria. Details on the full-scale crash testing of the Steel Railing, Type IL-OH were provided in a previous report by Pena [11].

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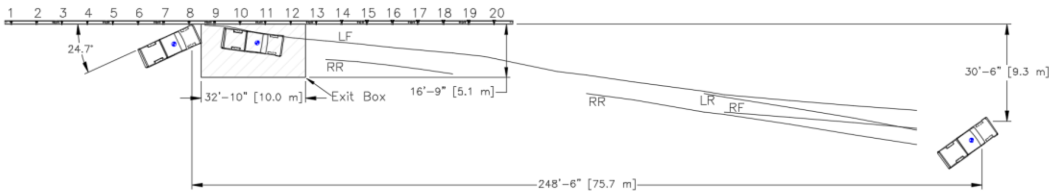
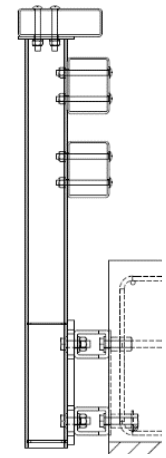
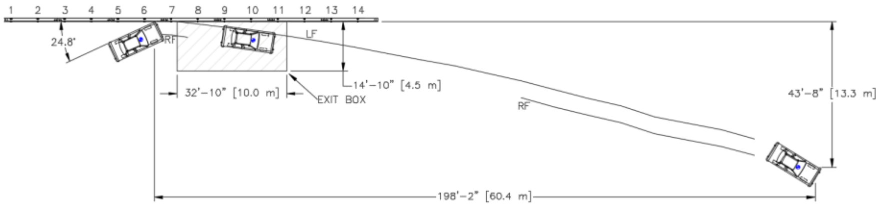


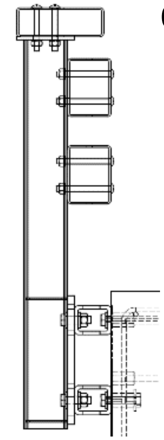


- Test Agency .... MwRSF
- Test Number STBR-2
- Date 2/22/2019
- MASH 2016 Test Designation No. 4-11
- Test Article .... Steel Railing, Type IL-OH
- Total Length .... 159 ft – 11½ in. (48.8 m)
- Key Component – Top Rail
  - Length .... 191¼ in. (4,858 mm)
  - Width .... 12 in. (305 mm)
  - Depth .... 4 in. (102 mm)
- Key Component - Post
  - Length .... 58½ in. (1,486 mm)
  - Width .... 6 in. (152 mm)
  - Spacing .... 8 ft (2.4 m)
- Vehicle Make /Model .... 2011 Dodge Ram 1500
  - Curb .... 4,938 lb (2,240 kg)
  - Test Inertial .... 4,992 lb (2,264 kg)
  - Gross Static .... 5,157 lb (2,339 kg)
- Impact Conditions
  - Speed .... 64.5 mph (103.8 km/h)
  - Angle .... 24.7 deg.
  - Impact Location .... 6 ft – 10 in. (2.1 m) upstream from post no. 9
- Impact Severity 120.9 kip-ft (163.9 kJ) > 105.6 kip-ft (143.1 kJ) limit from MASH 2016
- Exit Conditions
  - Speed .... 53.1 mph (85.4 km/h)
  - Angle .... 6.8 deg.
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 248 ft – 6 in. (75.7 m) DS from impact
  30 ft – 6 in. laterally in front
- Vehicle Damage .... Moderate
  - VDS [23] 11-LFQ-5
  - CDC [24] 11-LFAW-5
  - Maximum Interior Deformation .... 0.9 in. (23 mm)
- Test Article Damage .... Minimal
- Maximum Test Article Deflections
  - Permanent Set .... 3.5 in. (89 mm)
  - Dynamic .... 7.0 in. (178 mm)
  - Working Width .... 19.0 in. (483 mm)
- Transducer Data

Evaluation Criteria		**Transducer**		**MASH 2016 Limit**
		SLICE-1	**SLICE-2 (primary)**	
OIV ft/s (m/s)	Longitudinal	-14.50 (-4.42)	-14.27 (-4.34)	±40 (12.2)
	Lateral	26.32 (8.02)	28.61 (8.72)	±40 (12.2)
ORA g's	Longitudinal	3.70	-3.64	±20.49
	Lateral	20.35	17.62	±20.49
MAX ANGULAR DISP. deg.	Roll	-23.6	-20.0	±75
	Pitch	-4.0	-5.2	±75
	Yaw	32.8	32.3	Not required
THIV – ft/s (m/s)		30.54 (9.31)	32.40 (9.88)	Not required
PHD – g's		20.35	17.62	Not required
ASI		0.93	0.61	Not required

Figure 15. Summary of Test Results and Sequential Photographs, Test No. STBR-2 [11]





- Test Agency ....MwRSF
- Test Number..STBR-3
- Date..3/1/2019
- MASH 2016 Test Designation No...4-10
- Test Article.................................Steel Railing, Type IL-OH
- Total Length ....111 ft – 11¼ in. (34.1 m)
- Key Component – Top Rail
  - Length ....191¼ in. (4,858 mm)
  - Width..12 in. (305 mm)
  - Depth..4 in. (102 mm)
- Key Component - Post
  - Length ....58½ in. (1,486 mm)
  - Width..6 in. (152 mm)
  - Spacing..8 ft (2.4 m)
- Vehicle Make /Model..............................2009 Kia Rio
  - Curb..2,456 lb (1,114 kg)
  - Test Inertial..................................2,408 lb (1,092 kg)
  - Gross Static..................................2,569 lb (1,120 kg)
- Impact Conditions
  - Speed..62.0 mph (99.8 km/h)
  - Angle..24.8 deg.
  - Impact Location.........................61.3 in (1.6 m) upstream from post no. 7
- Impact Severity ....54.5 kip-ft (73.9 kJ) > 51.0 kip-ft (69.7 kJ) limit from MASH 2016
- Exit Conditions
  - Speed..45.1 mph (72.6 km/h)
  - Angle ....4.6 deg.
- Exit Box Criterion..Pass
- Vehicle Stability..Satisfactory
- Vehicle Stopping Distance ....198 ft – 2 in. (60.4 m) downstream from of impact
  43 ft – 8 in. (13.3 m) laterally in front
- Vehicle Damage..Moderate
  - VDS [23] 11-LFQ-6
  - CDC [24]..11-LFAW-6
  - Maximum Interior Deformation ....0.9 in. (23 mm)
- Test Article Damage ....Minimal
- Maximum Test Article Deflections
  - Permanent Set ....0.6 in. (15 mm)
  - Dynamic..2.9 in. (74 mm)
  - Working Width..................................15.2 in. (386 mm)
- Transducer Data

Evaluation Criteria		Transducer		MASH 2016 Limit
		SLICE-1 (primary)	**SLICE-2**	
OIV ft/s (m/s)	Longitudinal	-18.46 (-5.63)	-18.70 (-5.63)	±40 (12.2)
	Lateral	33.19 (10.12)	31.48 (9.59)	±40 (12.2)
ORA g's	Longitudinal	-16.82	-15.76	±20.49
	Lateral	-14.77	-13.31	±20.49
MAX ANGULAR DISP. deg.	Roll	-7.9	-4.6	±75
	Pitch	-3.6	-4.4	±75
	Yaw	33.7	32.7	Not required
THIV – ft/s (m/s)		41.82 (12.75)	39.77 (12.12)	Not required
PHD – g's		19.13	18.37	Not required
ASI		2.33	2.17	Not required

Figure 16. Summary of Test Results and Sequential Photographs, Test No. STBR-3 [11]

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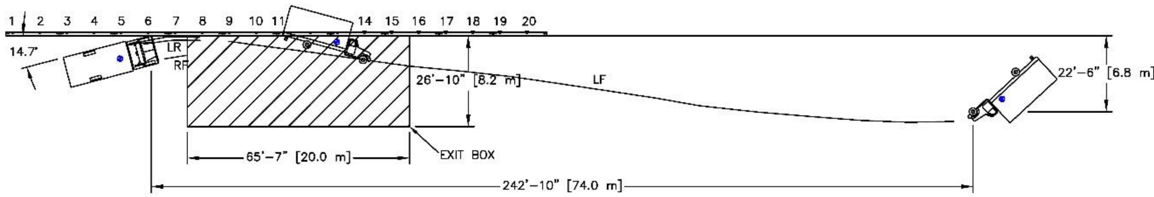

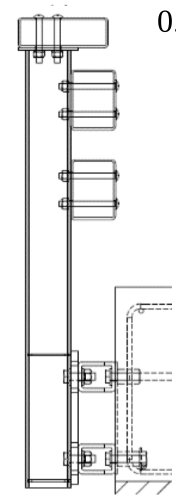

- Test Agency .... MwRSF
- Test Number STBR-4
- Date 6/6/2019
- MASH 2016 Test Designation No. 4-12
- Test Article .... Steel Railing, Type IL-OH
- Total Length .... 159 ft – 11¼ in. (48.7 m)
- Key Component – Top Rail
  - Length .... 191¼ in. (4,858 mm)
  - Width .... 12 in. (305 mm)
  - Depth .... 4 in. (102 mm)
- Key Component - Post
  - Length .... 58½ in. (1,486 mm)
  - Width .... 6 in. (152 mm)
  - Spacing .... 8 ft (2.4 m)
- Vehicle Make /Model .... 2007 Freightliner M2 106
  - Curb .... 13,884 lb (6,298 kg)
  - Test Inertial .... 22,152 lb (10,048 kg)
  - Gross Static .... 22,314 lb (10,121 kg)
- Impact Conditions
  - Speed .... 56.4 mph (90.8 km/h)
  - Angle .... 14.7 deg.
  - Impact Location .... 53.2 in (1.4 m) US from the splice between post nos. 6 and 7
- Impact Severity 151.7 kip-ft (205.7 kJ) > 142.0 kip-ft (192.5 kJ) limit from MASH 2016
- Exit Conditions
  - Speed .... 36.0 mph (58.0 km/h)
  - Angle .... N/A
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 242 ft – 10 in. (74.0 m) DS from impact
  22 ft – 6 in. (6.8 m) laterally in front
- Vehicle Damage .... Moderate
  - VDS [23] 11-LFQ-6
  - CDC [24] 11-LFAW-6
  - Maximum Interior Deformation .... 4.5 in. (114 mm)
- Test Article Damage .... Moderate
- Maximum Test Article Deflections
  - Permanent Set .... 7.3 in. (185 mm)
  - Dynamic .... 7.9 in. (201 mm)
  - Working Width .... 87.7 in. (2,228 mm)
- Transducer Data

Evaluation Criteria		Transducer		MASH 2016 Limit
		SLICE-1 (primary)	SLICE-2	
OIV ft/s (m/s)	Longitudinal	-6.72 (-2.05)	-5.00 (-1.52)	Not required
	Lateral	11.16 (3.40)	18.26 (5.56)	Not required
ORA g's	Longitudinal	-8.49	-4.31	Not required
	Lateral	18.50	-7.34	Not required
MAX ANGULAR DISP. deg.	Roll	-95.2	-93.3	Not required
	Pitch	-9.1	-8.5	Not required
	Yaw	81.7	80.1	Not required
THIV – ft/s (m/s)		19.28 (5.88)	19.12 (5.83)	Not required
PHD – g's		18.50	7.87	Not required
ASI		0.59	0.77	Not required

Figure 17. Summary of Test Results and Sequential Photographs, Test No. STBR-4 [11]

Table 3. Summary of Bridge Rail Safety Performance Evaluation

<table>
<tr><th>Evaluation Factors</th><th colspan="2">Evaluation Criteria</th><th>Test No. STBR-2</th><th>Test No. STBR-3</th><th>Test No. STBR-3</th></tr>
<tr><td>Structural Adequacy</td><td>A.</td><td>Test article should contain and redirect the vehicle or bring the vehicle to a controlled stop; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.</td><td>S</td><td>S</td><td>S</td></tr>
<tr><td rowspan="6">Occupant Risk</td><td rowspan="2">D.</td><td>1. Detached elements, fragments or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone.</td><td>S</td><td>S</td><td>S</td></tr>
<tr><td>2. Deformations of, or intrusions into, the occupant compartment should not exceed limits set forth in Section 5.2.2 and Appendix E of MASH 2016.</td><td>S</td><td>S</td><td>S</td></tr>
<tr><td>G.</td><td>It is preferable, although not essential, that the vehicle remain upright during and after collision.</td><td>NA</td><td>NA</td><td>S</td></tr>
<tr><td>F.</td><td>The vehicle should remain upright during and after collision. The maximum roll and pitch angles are not to exceed 75 degrees.</td><td>S</td><td>S</td><td>NA</td></tr>
<tr><td>H.</td><td>Occupant Impact Velocity (OIV) (see Appendix A, Section A5.2.2 of MASH 2016 for calculation procedure) should satisfy the following limits:<table><tr><th colspan="3">Occupant Impact Velocity Limits</th></tr><tr><th>Component</th><th>Preferred</th><th>Maximum</th></tr><tr><td>Longitudinal and Lateral</td><td>30 ft/s</td><td>40 ft/s</td></tr></table></td><td>S</td><td>S</td><td>NA</td></tr>
<tr><td>I.</td><td>The Occupant Ridedown Acceleration (ORA) (see Appendix A, Section A5.2.2 of MASH 2016 for calculation procedure) should satisfy the following limits:<table><tr><th colspan="3">Occupant Ridedown Acceleration Limits</th></tr><tr><th>Component</th><th>Preferred</th><th>Maximum</th></tr><tr><td>Longitudinal and Lateral</td><td>15.0 g’s</td><td>20.49 g’s</td></tr></table></td><td>S</td><td>S</td><td>NA</td></tr>
<tr><td colspan="3">MASH 2016 Test Designation No.</td><td>4-11</td><td>4-10</td><td>4-12</td></tr>
<tr><td colspan="3">Final Evaluation (Pass or Fail)</td><td>Pass</td><td>Pass</td><td>Pass</td></tr>
</table>

S – Satisfactory U – Unsatisfactory NA – Not Applicable

4 APPROACH GUARDRAIL TRANSITION

4.1 Design

An approach guardrail transition (AGT) was developed to safely connect the new Steel Railing, Type IL-OH to the Midwest Guardrail System (MGS) located on the adjacent roadway. Although the bridge rail was MASH TL-4 compliant, the AGT was only required to satisfy MASH TL-3 criteria, matching the test level of the adjacent MGS. The AGT was designed to prevent snag on the bridge rail during both conventional- and reverse-direction (traveling from bridge to roadway) impacts. Additionally, efforts were made to maximize the distance between the last AGT post and the first bridge rail post to avoid post installation obstacles, such as bridge abutments and wing walls. Finally, similar to the new bridge rail, the AGT was to remain crashworthy both before and after roadway overlays up to 3 in. thick.

The Nebraska DOT 34-in. tall AGT was selected as the basis for the new AGT to steel-tube bridge rail. This AGT utilized nested thrie beam rails supported by W6x15 posts spaced at 37.5 in. on-center. The upstream end of the AGT incorporated W6x8.5 posts at various spacings corresponding to the MASH-crashworthy MGS stiffness transition. The AGT had an initial top mounting height of 34 in. to account for future overlays up to 3 in. thick. After an overlay, the symmetric W-to-thrie transition segment would be replaced with an asymmetric W-to-thrie transition segment and the W-beam in the upstream MGS region would be raised 3 in. on the guardrail posts. Thus, the effective nominal height for the entire system would become the standard 31 in. after a 3-in. thick overlay and these minor adjustments.

Specialized transition tube rails were configured to attach the thrie-beam AGT to the steel-tube bridge rail. Two 119⅝-in. long, HSS8x6x¼ steel tube rails were used to connect the thrie beam terminal connector to the bridge rail. These transition tube rails included a 2-in. height transition near their middle to match up with the heights of the AGT rail and the bridge rail tubes. A 6H:1V vertical taper was used on the height transition to reduce snag severity. A 36-in. long, HSS6x4x¼ tube was sandwiched between the lower and middle transition rails and incorporated a 3:1 lateral taper on the downstream end to mitigate vehicle snag on the terminal connector during reverse-direction impacts. The top transition tube rail assembly was 44¼ in. long and consisted of HSS12x4x¼ segments and a ¼-in. thick bent plate. The top transition tube rail was sloped downward at a 2H:1V slope and welded to the bent plate. The bent plate fit against the top and back sides of the middle transition rail and was secured with two ¾-in. diameter bolts. The three transition tube rails were connected to the bridge rail tubes using the same hardware as the bridge rail splices.

Finally, to avoid post installation obstacles like wingwalls and abutments, the last transition post was longitudinally offset 9 ft from the first bridge post, which was more than double the post distances of other MASH AGTs. Final system drawings of the AGT and connection to steel-tube bridge railing are shown in Appendix A. Photographs of the AGT are shown in Figures 18 and 19.

Figure 18. Photographs of the AGT to Steel Railing, Type IL-OH

Figure 19. Photographs of the AGT to Steel Railing, Type IL-OH

4.2 MASH Evaluation

Longitudinal barrier transitions must satisfy impact safety standards in order to be eligible for reimbursement by the Federal Highway Administration for use on the National Highway System. For new hardware, these safety standards consist of the guidelines and procedures published in MASH [8]. According to TL-3 of MASH, longitudinal barrier transitions must be subjected to two full-scale vehicle crash tests, as summarized in Table 4.

Table 4. MASH TL-3 Crash Test Conditions for Longitudinal Barrier Transitions

Test Article	Test Designation No.	Test Vehicle	Vehicle Weight (lb)	Impact Conditions		Evaluation Criteria [1]
				Speed (mph)	Angle (deg.)	
Transition	3-20	1100C Small Car	2,420	62	25	A,D,F,H,I
	3-21	2270P Pickup Truck	5,000	62	25	A,D,F,H,I

[1] Evaluation criteria explained in MASH [8]

Recent testing of AGTs has illustrated the importance in evaluating two different transition regions along the length of the AGT: (1) the downstream transition where the thrie beam connects to the bridge rail and (2) the upstream stiffness transition where the MGS transitions to a stiffer thrie beam guardrail. Additionally, the 34-in. tall AGT described herein was designed for use both before and after roadway overlays up to 3 in. thick, which effectively changes the barrier height relative to the roadway surface. The combination of these MASH tests, different transition regions, and pre- and post-overlay barrier configurations resulted in a total of eight possible tests, but not all of them were considered critical or necessary to evaluate the performance of the new AGT.

The upstream stiffness transition of the AGT was specifically designed to replicate the MASH-crashworthy MGS stiffness transition [16-17]. Upon initial installation, the only difference between the two systems was that the 34-in. tall AGT utilized a symmetric W-to-thrie transition rail instead of an asymmetric transition rail. Since the W-beam upstream from the transition rail was mounted at its nominal 31-in. height, vehicles impacting this region of the barrier should not extend over the rail and roll excessively. Additionally, the bottom of the symmetric transition rail segment has a shallower slope, which would produce less snag as a small vehicle tries to wedge underneath the rail. Thus, there were no concerns about vehicle stability and/or snag on the upstream stiffness transition of the 34-in. tall AGT prior to a roadway overlay.

After the roadway overlay, the symmetric rail segment would be replaced by an asymmetric segment, and the W-beam of the adjacent MGS would be raised 3 in. on the posts to maintain its nominal 31-in. mounting height. Previous studies have concluded that guardrail can be raised up to 4 in. on the support posts and the system will remain crashworthy [25-27]. Thus, after an overlay, the upstream stiffness transition is essentially identical to the MASH-tested MGS stiffness transition. Since the MGS stiffness transition was previously subjected to and successfully passed MASH TL-3 criteria, the upstream stiffness transition within the new AGT to a steel-tube bridge rail would be MASH TL-3 crashworthy as well. Therefore, all crash testing of the upstream stiffness transition, both before and after an overlay, was deemed non-critical

At the downstream end of the AGT, there were concerns for rail pocketing within the 9-ft unsupported span length adjacent to the bridge rail as well as vehicle snag on the transition tube rails and bridge posts. Rail pocketing issues would be the same regardless of the presence of an overlay as an overlay would not affect the strength of the system. However, an overlay would reduce the gap below the rail, thereby reducing the likelihood that vehicle bumpers and wheels would extend under the rail and snag on system components. Accordingly, the system configuration without an overlay would present the worst-case scenario for vehicle snag. Thus, only two full-scale tests on the initial 34-in. tall configuration were recommended to evaluate the crashworthiness of the new AGT to MASH TL-3 criteria.

LS-DYNA computer simulations were conducted to identify critical impact points for both MASH TL-3 full-scale crash tests. Multiple impacts were simulated on the AGT with both vehicles to identify the impact point that would maximize vehicle snag and thereby maximize the potential for excessive decelerations, occupant compartment crush, and/or vehicle instabilities. The critical impact point for test designation no. 3-21 was determined to be 17 in. upstream from the last W6x15 AGT post to maximize occupant risk values and the potential for snagging on the sloped end of the upper transition tube and the first bridge railing post. The critical impact point for test designation no. 3-20 was determined to be 30 in. upstream from the last AGT post to maximize wedging of the small car tire underneath the sloped transition tube rails and the potential for snagging on posts.

LS-DYNA computer simulations were also conducted to evaluate the AGT during reverse-direction impacts. The simulated reverse-direction impacts showed no indication of significant pocketing or snag on the thrie beam terminal connector or its associated attachment hardware. Additionally, these reverse-direction simulations on the AGT showed similar vehicle behavior, accelerations, and system deflections as observed during the actual full-scale crash tests conducted on the interior sections of the bridge railing, test nos. STBR-2 and STBR-3. Since MASH testing on the bridge rail was successful, any reverse-direction crash tests on the transition from the bridge rail to the thrie-beam AGT should also be successful. Thus, reverse-direction testing was deemed non-critical, and only conventional direction impacts were conducted in the full-scale testing and evaluation of the new AGT connection.

A full-scale test installation of the AGT to steel-tube bridge rail was constructed and test nos. STBRT-1 and STBRT-2 were conducted on the test article in accordance with MASH test designation nos. 3-20 and 3-21, respectively. In test no. STBRT-1, the 2,404-lb small car impacted the steel-tube bridge rail system 21.3 in. upstream from the last AGT post at a speed of 64.6 mph and an angle of 25.2 degrees, resulting in an impact severity of 60.9 kip-ft. The vehicle was successfully contained and smoothly redirected with moderate damage to both the barrier system and the vehicle. After impacting the transition, the vehicle exited the system at a speed of 46.2 mph and an angle of −7.4 degrees. All vehicle decelerations, ORAs, and OIVs fell within the recommended safety limits established in MASH. Therefore, test no. STBRT-1 satisfied the safety criteria of MASH test designation no. 3-20. A summary of the test results and sequential photographs of test no. STBRT-1 are shown in Figure 20.

In test no. STBRT-2, the 5,007-lb pickup truck impacted the steel-tube bridge rail system 15.9 in. upstream from the last AGT post at a speed of 62.7 mph and an angle of 24.9 degrees, resulting in an impact severity of 116 kip-ft. The vehicle was successfully contained and smoothly redirected with moderate damage to both the bridge rail system and the vehicle. After impacting the barrier, the vehicle exited the system at a speed of 49.6 mph and an angle of −11.4 degrees. All vehicle decelerations, ORAs, and OIVs fell within the recommended safety limits established in MASH. Therefore, test no. STBRT-2 was successful according to the safety criteria of MASH test designation no. 3-21. A summary of the test results and sequential photographs of test no. STBRT-2 are shown in Figure 21, and a summary of both test evaluations is shown in Table 5.

Due to the two successful full-scale crash tests, the incorporation of the upstream MGS stiffness transition, the simulation results of the reverse-direction impacts, and the recommended modifications to the AGT after an overlay as described herein, the new 34-in. AGT to steel-tube bridge rail was determined to be crashworthy to MASH TL-3 standards both before and after a 3-in. roadway overlay. Details on the full-scale crash testing of the AGT to Steel Railing, Type IL-OH were provided in a previous report by Rasmussen [14].


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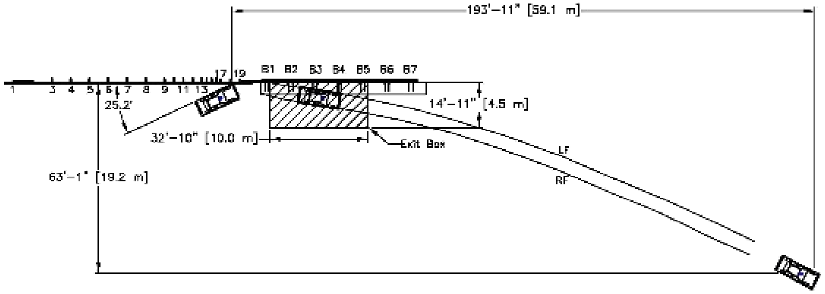
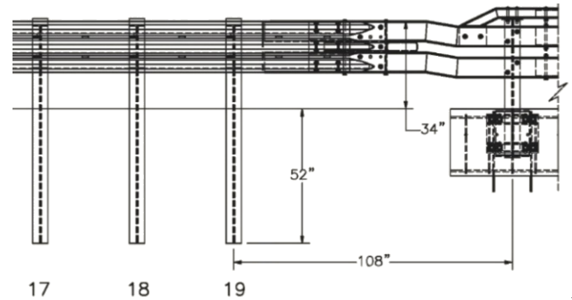


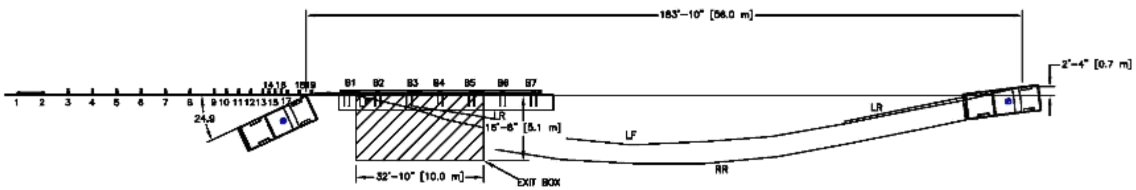


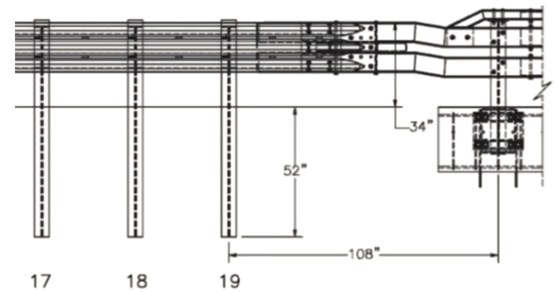
- Test Agency .... MwRSF
- Test Number STBRT-1
- Date 8/24/2020
- MASH 2016 Test Designation No. 3-20
- Test Article .... OH-IL Steel-Tube Bridge Rail Transition
- Total Length .... 137 ft – 9 in.
- Key Component –Rails
  - Middle and Lower Tube Rails HSS8x6x¼
  - Top Rail HSS12x4x¼
  - AGT Rail .... Nested 12-gauge Thrie Beam
  - Thrie Beam Connector .... 10-gauge Terminal Connector
- Key Component –Posts
  - Bridge Rail .... W6x15 at 8 ft on-center
  - AGT .... W6x15 at 37.5 in. on-center
  - Spacing between Adjacent AGT and Bridge Post .... 9 ft
- Vehicle Make /Model .... 2009 Hyundai Accent
  - Curb .... 2,447 lb
  - Test Inertial .... 2,404 lb
  - Gross Static .... 2,568 lb
- Impact Conditions
  - Speed .... 64.6 mph
  - Angle .... 25.2 deg.
  - Impact Location .... 21.3 in. U.S. from post no. 19
- Impact Severity .... 60.9 kip-ft > 51.1 kip-ft limit from MASH 2016
- Exit Conditions
  - Speed .... 46.2 mph
  - Angle .... -7.4 deg.
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 193 ft – 11 in. downstream
- Vehicle Damage .... Moderate
  - VDS [23] 11-LFQ-5
  - CDC [24] 11-LFEW-2
  - Maximum Interior Deformation .... 2.1 in.
- Test Article Damage .... Minimal
- Maximum Test Article Deflections
  - Permanent Set .... 2.7 in.
  - Dynamic .... 8.6 in.
  - Working Width .... 21.4 in.
- Transducer Data

Evaluation Criteria		Transducer		MASH 2016 Limits
		SLICE-1 (primary)	SLICE-2	
OIV (ft/s)	Longitudinal	-18.83	-17.79	±40
	Lateral	28.98	27.52	±40
ORA (g's)	Longitudinal	-9.44	-9.22	±20.49
	Lateral	11.40	11.30	±20.49
Maximum Angular Displacement (deg.)	Roll	-6.1	-3.3	±75
	Pitch	-3.9	-4.5	±75
	Yaw	36.3	36.0	not required
THIV (ft/s)		29.06	28.22	not required
PHD (g's)		11.53	11.35	not required
ASI		1.91	1.8	not required

Figure 20. Summary of Test Results and Sequential Photographs, Test No. STBRT-1 [14]

0.000 sec | 0.100 sec | 0.150 sec | 0.250 sec | 0.350 sec





- Test Agency .... MwRSF
- Test Number STBRT-2
- Date 9/22/2020
- MASH 2016 Test Designation No. 3-21
- Test Article .... OH-IL Steel Tube Bridge Rail Transition
- Total Length .... 137 ft – 9 in.
- Key Component –Rails
  - Middle and Lower Tube Rails HSS8x6x¼
  - Top Rail HSS12x4x¼
  - AGT Rail .... Nested 12-Gauge Thrie Beam
  - Thrie Beam Terminal Connector .... 10-Gauge Terminal Connector
- Key Component –Posts
  - Bridge Rail .... W6x15 at 8 ft on-center
  - AGT .... W6x15 at 37.5 in. on-center
  - Spacing between Adjacent AGT and Bridge Post .... 9 ft
- Vehicle Make /Model .... 2014 Dodge Ram 1500
  - Curb .... 5,133 lb
  - Test Inertial .... 5,007 lb
  - Gross Static .... 5,160 lb
- Impact Conditions
  - Speed .... 62.7 mph
  - Angle .... 24.9 deg.
  - Impact Location .... 15.9 in. upstream from post no. 19
- Impact Severity .... 116 kip-ft > 106 kip-ft limit from MASH 2016
- Exit Conditions
  - Speed .... 49.6 mph
  - Angle .... -11.4 deg.
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 183 ft – 10 in. downstream
- Vehicle Damage .... Moderate
  - VDS [23] 11-LFQ-5
  - CDC [24] 11-LFEW-3
  - Maximum Interior Deformation .... 1.8 in.
- Test Article Damage .... Minimal
- Maximum Test Article Deflections
  - Permanent Set .... 8.4 in.
  - Dynamic .... 18.6 in.
  - Working Width .... 27.8 in.
- Transducer Data

Evaluation Criteria		Transducer		MASH 2016 Limits
		SLICE-1	SLICE-2 (primary)	
OIV (ft/s)	Longitudinal	-18.04	-16.27	±40
	Lateral	20.41	21.60	±40
ORA (g's)	Longitudinal	-11.28	-11.86	±20.49
	Lateral	13.62	15.81	±20.49
Maximum Angular Displacement (deg.)	Roll	-21.5	-17.7	±75
	Pitch	3.6	-5.5	±75
	Yaw	46.9	46.2	not required
THIV (ft/s)		25.56	25.80	not required
PHD (g's)		17.26	19.24	not required
ASI		1.14	1.25	not required

Figure 21. Summary of Test Results and Sequential Photographs, Test No. STBRT-2 [14]

Table 5. Summary of AGT Safety Performance Evaluation

<table>
<tr><th>Evaluation Factors</th><th colspan="4">Evaluation Criteria</th><th>Test No. STBRT-1</th><th>Test No. STBRT-2</th></tr>
<tr><td>Structural Adequacy</td><td>A.</td><td colspan="3">Test article should contain and redirect the vehicle or bring the vehicle to a controlled stop; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.</td><td>S</td><td>S</td></tr>
<tr><td rowspan="10">Occupant Risk</td><td rowspan="2">D.</td><td colspan="3">1. Detached elements, fragments or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone.</td><td>S</td><td>S</td></tr>
<tr><td colspan="3">2. Deformations of, or intrusions into, the occupant compartment should not exceed limits set forth in Section 5.2.2 and Appendix E of MASH 2016.</td><td>S</td><td>S</td></tr>
<tr><td>F.</td><td colspan="3">The vehicle should remain upright during and after collision. The maximum roll and pitch angles are not to exceed 75 degrees.</td><td>S</td><td>S</td></tr>
<tr><td rowspan="4">H.</td><td colspan="3">Occupant Impact Velocity (OIV) (see Appendix A, Section A5.2.2 of MASH 2016 for calculation procedure) should satisfy the following limits:</td><td rowspan="4">S</td><td rowspan="4">S</td></tr>
<tr><td colspan="3">Occupant Impact Velocity Limits</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>30 ft/s</td><td>40 ft/s</td></tr>
<tr><td rowspan="4">I.</td><td colspan="3">The Occupant Ridedown Acceleration (ORA) (see Appendix A, Section A5.2.2 of MASH 2016 for calculation procedure) should satisfy the following limits:</td><td rowspan="4">S</td><td rowspan="4">S</td></tr>
<tr><td colspan="3">Occupant Ridedown Acceleration Limits</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>15.0 g’s</td><td>20.49 g’s</td></tr>
<tr><td colspan="5">MASH 2016 Test Designation No.</td><td>3-20</td><td>3-21</td></tr>
<tr><td colspan="5">Final Evaluation (Pass or Fail)</td><td>Pass</td><td>Pass</td></tr>
</table>

S – Satisfactory U – Unsatisfactory NA – Not Applicable

5 SYSTEM IMPLEMENTATION

The Steel Railing, Type IL-OH and its associated guardrail transition were evaluated to and have satisfied the safety performance criteria of MASH TL-4 and TL-3, respectively. Full-system details for the 39-in. tall bridge rail and 34-in. tall AGT configuration, which were designed to accommodate a future 3-in. roadway overlay, are shown in Appendix A. The following sections provide implementation guidance and acceptable design variations for the new Steel Railing, Type IL-OH and adjacent guardrail transition.

5.1 Bridge Rail Implementation Guidance

5.1.1 Bridge Rail Configurations and Layouts

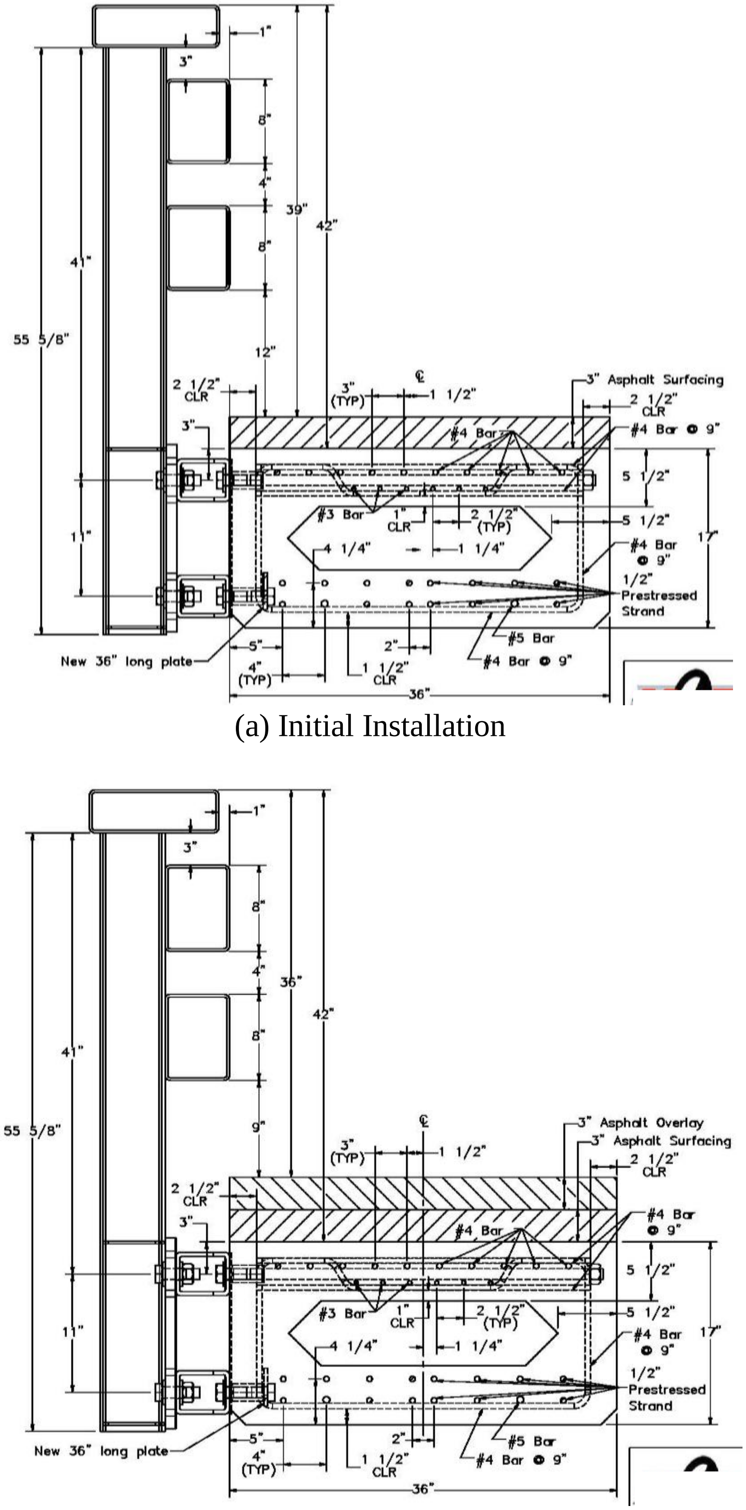
The Steel Railing, Type IL-OH was developed to be compatible with both cast-in-place (CIP) slab decks and precast box-beam girder bridge decks. However, the location of the post anchorage relative to the deck surface will differ between these deck types due to differences in concrete clear cover and reinforcement configurations. To ensure the threaded anchors are below the top reinforcing bars in the deck, the anchors were centered 4 in. from the top of a CIP slab deck and 3 in. from the top of a precast box-beam girder. Further, box beam bridge decks can be configured with a various wearing surfaces (e.g., 2-in. thick asphalt, 5-in. thick concrete, or 6-in. thick reinforced concrete). Thus, the length of the bridge rail post varies depending on the deck configuration.

Examples of the bridge rail installed on CIP slab decks, a 17-in. deep box beam deck, and a 42-in. deep box beam deck are shown in Figures 22 through 24. Each of these examples show the initial bridge rail installation with a 39-in. height and a 36-in. height after a 3-in overlay is applied. If a bridge will not be given a future overlay, the end user may elect to install the bridge rail at the nominal MASH TL-4 height of 36-in. at the time of initial installation. Note, the thrie beam AGT at the ends of the bridge would be installed at a height of 31 in. for a 36-in. bridge rail height.

As described above, the appropriate post length for each site condition is determined based on the desired top rail height (36 in. or 39 in.), thickness of the wearing surface, and location of the anchorage bolts relative to the top of the deck (4 in. for cast-in-place slab or 3 in. for box-beam girder). Combinations of these factors may result in post lengths between 53½ in. and 58½ in. Note, the top and bottom of each post assembly will remain the same since the post-to-deck attachment hardware and the HSS tube rails have specific dimensions that must be held constant. Variations in the height of the post will result in variations in the distance between the welded attachment plate and the lower HSS tube rail.

Additionally, the sponsors desired to lengthen the post by 2 in. (compared to as-tested configuration) to account for variable thicknesses in the wearing surfaces on box beam decks. Specifically, due to the camber of some pre-stressed box beams, a wearing surface may need to be thicker at pier/support locations in order to keep a flat and smooth roadway surface. Analysis of this increased post length showed a reduction in the strength of the bridge rail of less than 2 percent. As such, it was determined that the minor increase in post length would not negatively affect the crashworthiness of the bridge railing. Thus, the length of the bridge railing posts can range between 53½ in. and 60½ in., which will depend on site conditions and DOT preferences, as shown in

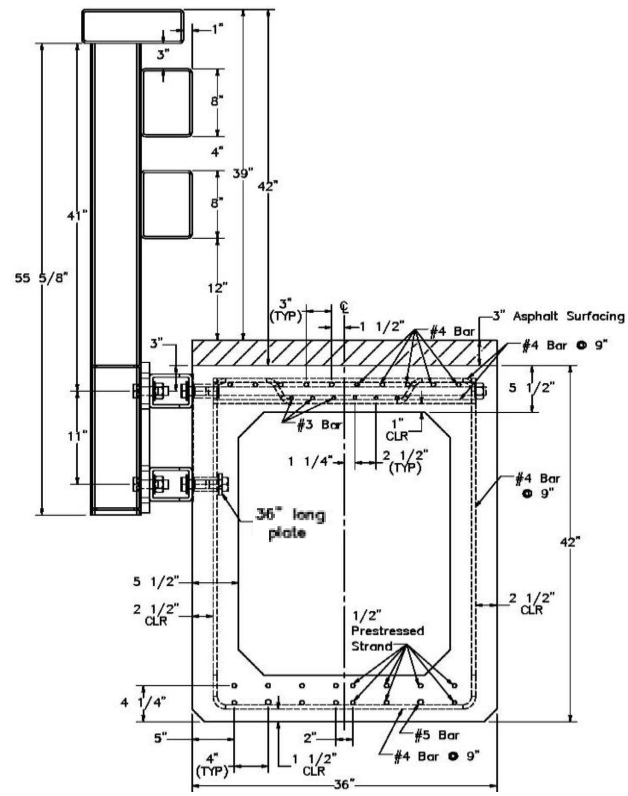
Figure A-4. Note, this distance refers only to the length of the W6x15 post and does not include the b3 mounting plate welded to the top of the post.



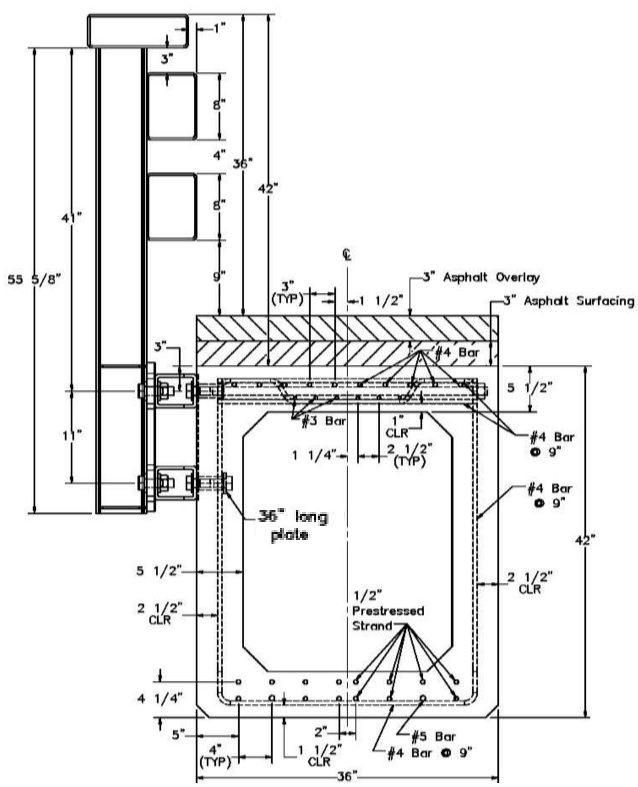
(a) Initial Installation

(b) Installation after 3-in. Overlay

Figure 22. Bridge Rail Installation on 17-in. Box Beam Girder with 3-in. Wearing Surface

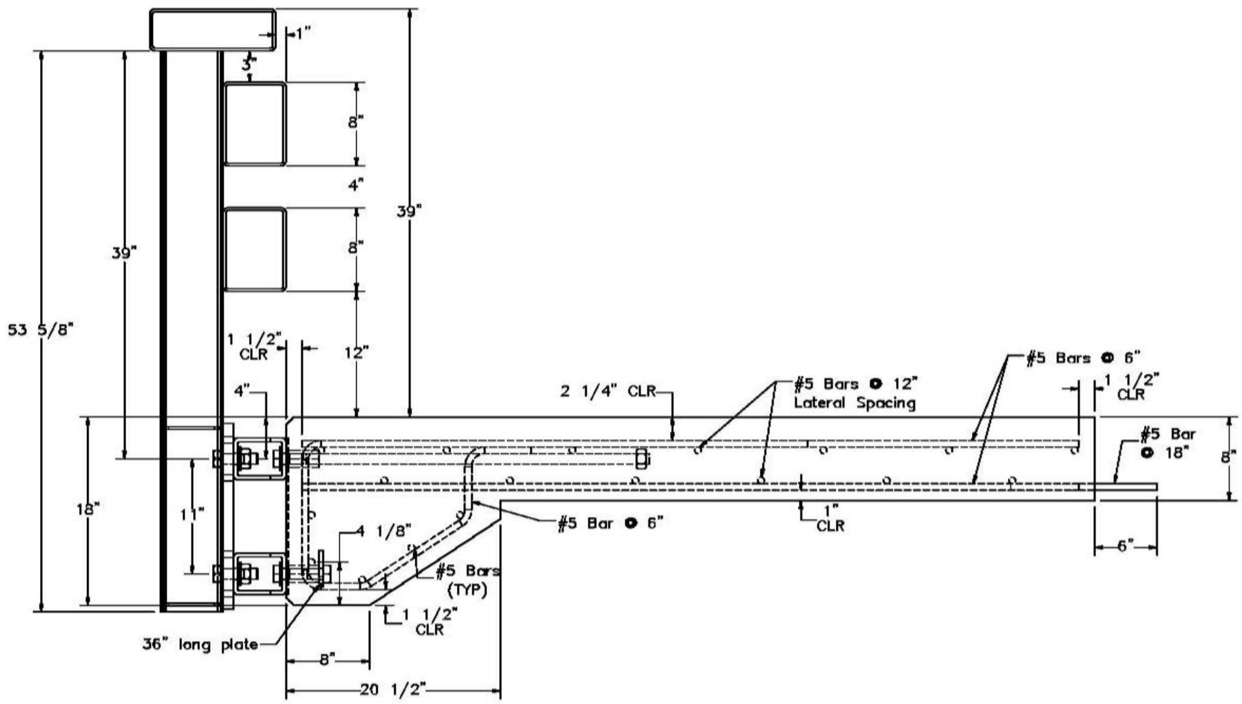


(a) Initial Installation



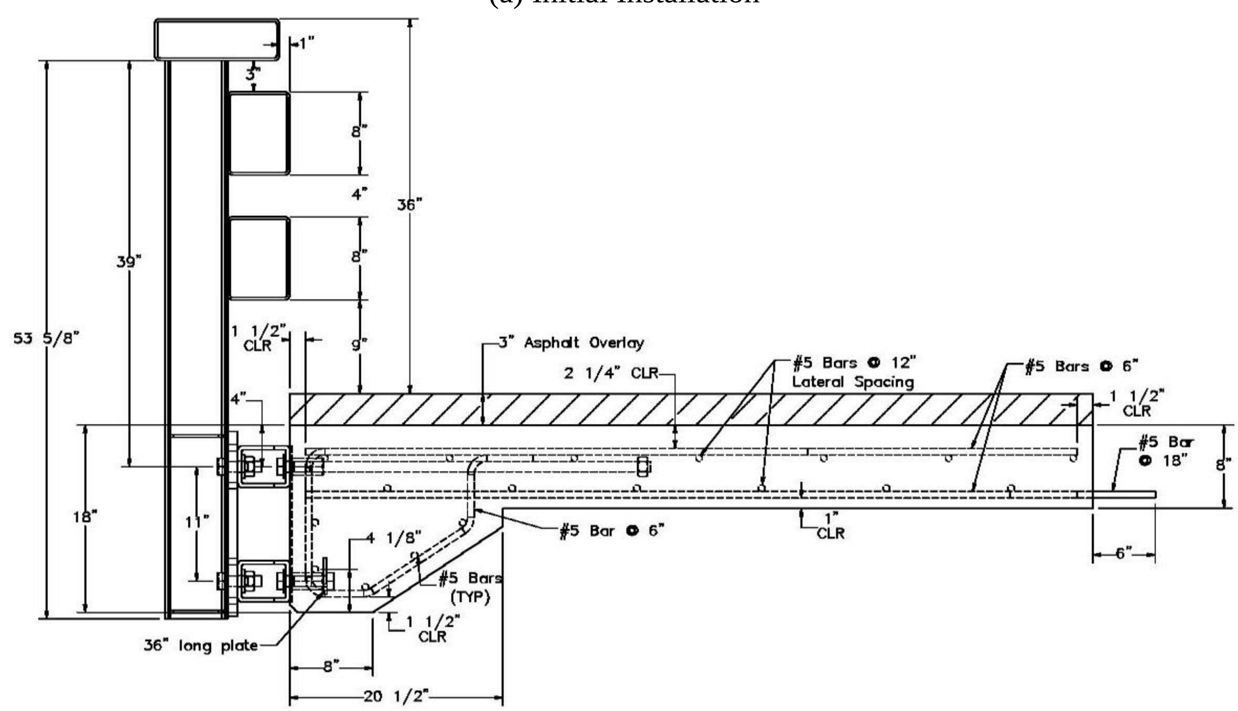
(b) Installation after 3-in. Overlay

Figure 23. Bridge Rail Installation on 42-in. Box Beam Girder with 3-in. Wearing Surface



Note: (1) The Bridge Deck shown is the Illinois Wide Overhang design and is compatible with the Illinois Narrow Overhand and the Ohio Deck Slab.

(a) Initial Installation



Note: (1) The Bridge Deck shown is the Illinois Wide Overhang design and is compatible with the Illinois Narrow Overhand and the Ohio Deck Slab.

(b) Installation after 3-in. Overlay

Figure 24. Bridge Rail Installation on Cast-in-Place Concrete Slab

Dynamic testing of the post-to-deck connection was conducted with the posts placed as close as 15 in. from the end of the deck, as measured to the center of the post. Moving a post closer to the end of the deck would reduce the concrete cover of the anchor rods thereby reducing the capacity of the anchors. Therefore, it is recommended to place the first bridge rail post a minimum of 15 in. from the end of the bridge deck. Further guidance for posts located near the ends of skewed bridges is provided in Section 5.1.6.

The Steel Railing, Type IL-OH was developed with a nominal post spacing of 8 ft. However, various bridge lengths may require variations in the post spacing for the railing to span from one end to the other. In these situations, the spacing between adjacent posts may be reduced to a spacing between 4 ft and 8 ft. A desire to reduce the post spacing may be most prevalent near the end of the bridge (i.e., between bridge rail post nos. 1 and 2), as shown in Figure 25, but may also be necessary near piers for multi-span superstructures. In addition to addressing variable bridge lengths, a reduced post spacing at the end of the bridge could be used to effectively shorten the guardrail length adjacent to the bridge for installation sites with limited spaced for the guardrail. Reducing the spacing between post nos. 1 and 2 results in the first bridge rail post shifting inward and bringing the connected AGT and guardrail with it. Thus, the actual length of the AGT and guardrail doesn't change, but the distance it extends off the bridge is reduced.

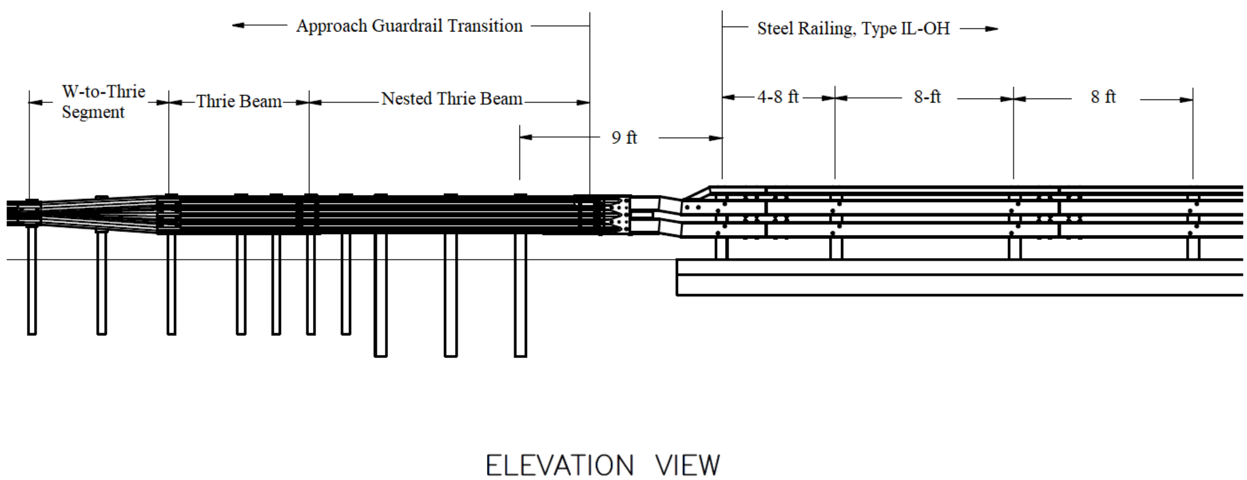


Figure 25. Variable Post Spacing at End of Bridge Railing

Bridges often contain horizontal curves to address the geometric site needs. Horizontal curves effectively increase the impact angle for a vehicle impacting the bridge rail on the outer edge. However, these curves are very shallow for high-speed roadways where vehicles would be traveling at MASH TL-4 speeds. According to Ohio DOT's standards, the maximum degree of curvature for a 60-mph roadway is 4.75 degrees. Taking this curve over a contact length of 20 ft for the 2270P pickup and 30 ft for the 10000S SUT results in angles of 1.0 degrees and 1.4 degrees. Note, both of these angles fall within the 1.5-degree tolerance window for MASH crash testing. Therefore, the large-radius horizontal curves associated with these bridges would not be expected to negatively affect the performance of the new bridge rail.

5.1.2 Bridge Rail Component Modifications

The Steel Railing, Type IL-OH was designed and tested using HSS tube rails made from ASTM A500 Grade C steel. Early in the design process, it was noted by the project sponsors and researchers that the steel industry was moving toward ASTM 1085 as the new standard for HSS sections. ASTM 1085 and A500 Grade C have very similar mechanical properties (i.e., yield and ultimate strengths), so the two steel grades should provide equivalent strength and performance. Thus, both A500 Grade C and A1085 would be appropriate for use in the HSS tube rails in the new bridge rail. Although HSS tubes are commonly made from A500 Grade B steel, this material is about 8 percent weaker than the A500 Grade C and A1085. Thus, it is not recommended to use A500 Grade B HSS tube until further research is conducted.

The details provided for the W6x15 posts include slots for the tube-to-post connections, as shown in Figure A-5. These slots were incorporated into the design to ease installation of the rail and allow for some construction tolerances. However, some installers prefer to field drill connection holes as opposed to doing it in the shop. When posts are to be field drilled, the installers may use holes instead of slots for these tube-to-rail connections.

A continuous weld was used all around the connection between the post and mounting plate, parts d5 and b4, respectively. This can create a small void between the two surfaces and within the welded area, and enclosed air voids can be problematic during a hot dipped galvanization process. Although there were not any issues experienced during the galvanization of the test articles, a small hole may be drilled into the center of the mounting plate to allow air to escape the small void, as shown in Figure 26.

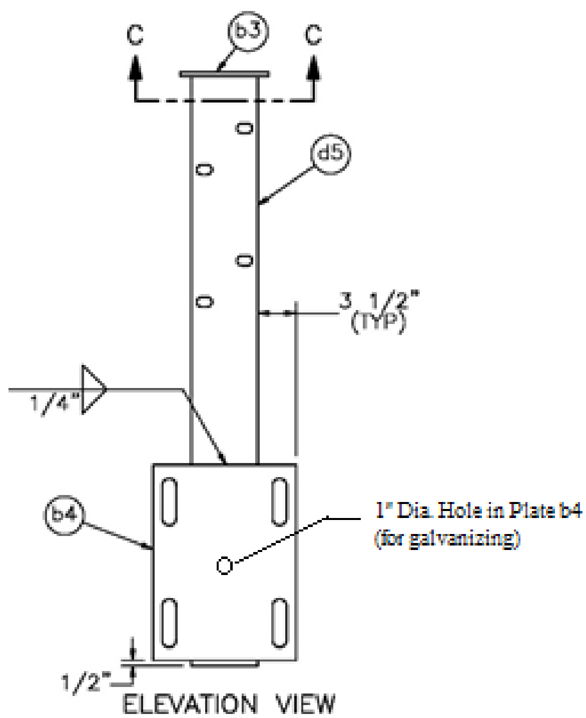


Figure 26. Example of Hole in Mounting Plate for Galvanization Purposes

5.1.3 Bridge Railing Expansion Joints

Most long bridges incorporate expansion/contraction joints to accommodate component movements caused by external loads and temperature variations. When a bridge railing spans across one of these joints, an expansion/contraction joint must also be placed within the railing. Expansion/contraction joints may be incorporated into the Steel Railing, Type IL-OH by elongating the typical splice tubes used to connect adjacent HSS segments, shown in Figures A-9 and A-10, and only bolting the splice tubes on one side of the connection. This alternative will allow the splice tubes to slide longitudinally within the HSS tube on the opposite side of the joint. The typical splice tubes were 30 in. long and should be centered within the HSS tube connection, resulting in a nominal embedded length of 14⅝ in., as shown in Figure A-8. In order to maintain this embedded length within an expansion joint, the free or unbolted end of the splice tubes should be extended a distance equal to the maximum expansion of the joint. Sketches of a typical splice tube and an expansion/contraction splice tube are shown in Figure 27.

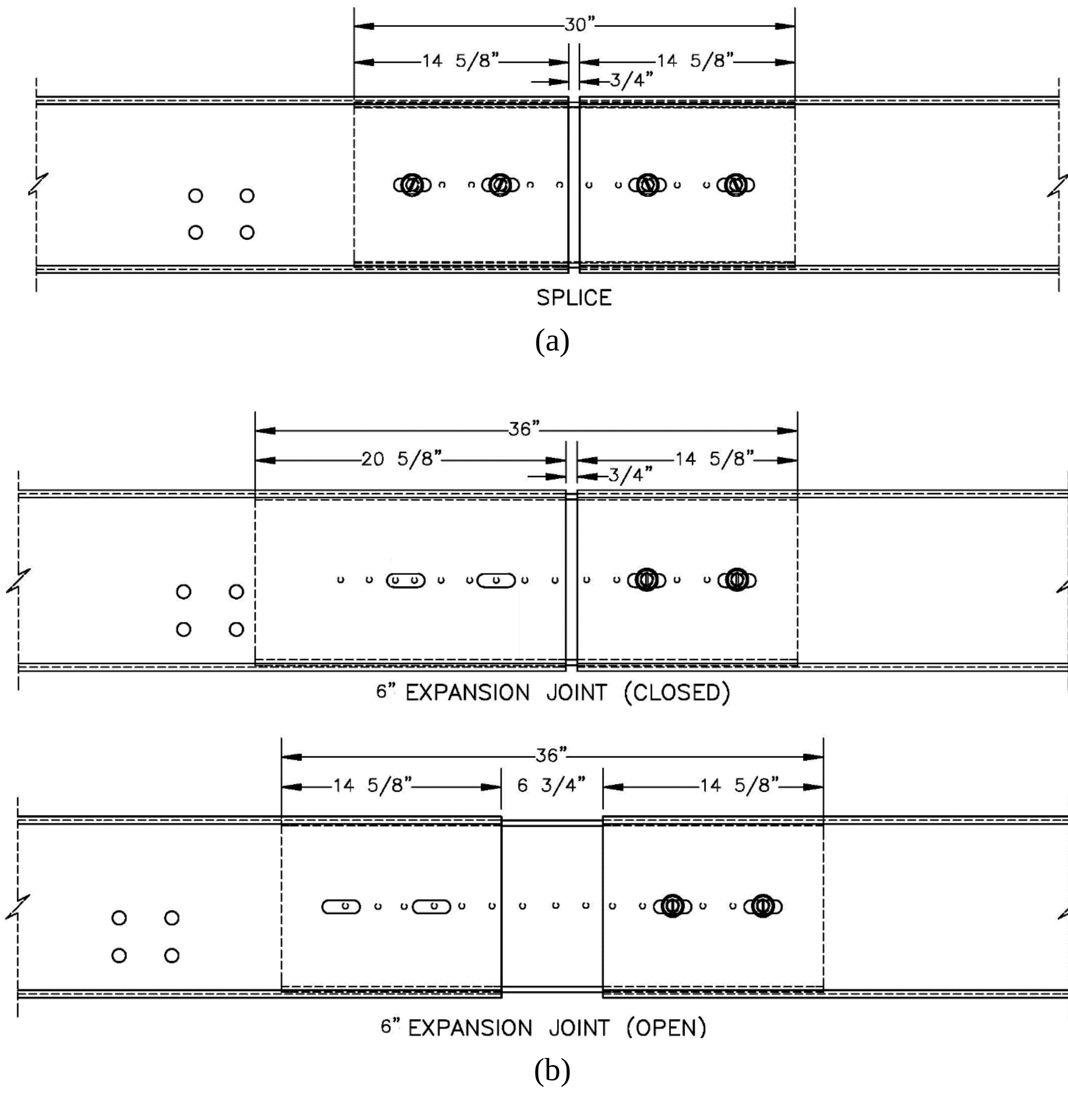


Figure 27. Top HSS Tube Rail Connections at (a) Typical Splice Joints and (b) a 6-in. Expansion/Contraction Joint

Expansion/contraction joints are often located and the ends of a bridge. However, it is not recommended to place an expansion joint in the Steel Railing, Type IL-OH within the 9-ft spacing between the last AGT post and the first bridge post. The specialized HSS transition rails designed to connect the bridge rail to the adjacent AGT were specifically designed to span across the first bridge railing post. Thus, the first HSS rail splices in the Steel Rail, Type IL-OH should always be located between the first and second post. Using the joint guidelines described above, an expansion/contraction joint could be placed between the first and second bridge railing posts. Accordingly, the first bridge railing post may be mounted to the side of the approach slab. Of course, the approach slab would have to be designed with adequate depth and the embedded anchorage hardware necessary to attach the first bridge post.

5.1.4 Bridge Railing Hardware Modifications

The length of bolt e10, used to connect the HSS spacer tubes to the deck edge, was selected to ensure proper embedment within the embedded coupling nuts. However, some installations may require washer plates to be installed between the HSS tubes and the deck in order to adjust the post's lateral position or to plumb the post. To account for the possible extra length required to span through washer plates, bolt e10 may be replaced with a slightly longer 1-in. diameter threaded rod and a heavy hex nut. The threaded rod would be threaded into the coupling nut until it contacts the threaded anchor rods, part c2, to ensure adequate threaded length and anchorage strength. The heavy hex nut would then be tightened on the threaded rod inside the HSS spacer tube. The threaded rod must have a minimum tensile strength of 120 ksi to match the ASTM F3125 Grade 325 bolt that it would be replacing (e.g., F1554 Grade 105 or A449). Note, the washer plates should match the profile of the inner face of the HSS spacer tubes to ensure proper load distribution to the deck.

The as-tested bridge railing was installed on a flat, level surrogate bridge deck. Thus, the posts were both plumb and perpendicular to the deck surface. Roadway super elevations and longitudinal slopes (e.g., vertical curves) create differences between plumb and perpendicular to the deck in the longitudinal and lateral directions, respectively. For super elevated roadways, upward slopes in front of a barrier will increase the angle between the roadways surface and the face of the barrier, and can lead to excessive roll for impacting vehicles. Thus, it is generally recommended to install barriers perpendicular to the roadway surface when on an upward slope and to install the barrier plumb when on a downward slope, as shown in Figure 28. Longitudinal slopes, or vertical grading, on roadways are typically minor and there would not be much difference between plumb and perpendicular. Embedded anchorage in the deck will likely be placed relative to the deck surface resulting in the post being perpendicular to the roadway surface. Most designers prefer the aesthetics of posts being placed perpendicular to the deck surface over having the posts at various angles to the deck along a vertical curve. Thus, it is recommended to install the posts perpendicular to the longitudinal slope of the roadway.

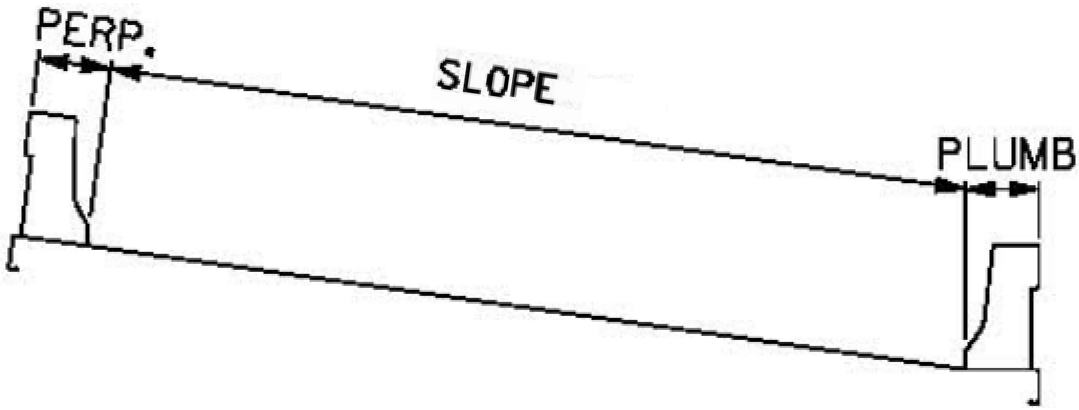


Figure 28. Plumb vs. Perpendicular Barrier Installations on Superelevated Roadways

The new Steel Railing, Type IL-OH was originally designed and tested with bolt e9, used to attach the post assembly to the HSS spacer tubes, oriented with the bolt head on the outside of the connection and the nut located inside the HSS spacer tube, as shown in Figure A-3. However, some installers may desire to flip the bolt around to place the bolt head inside the HSS spacer tube and create a more room in the tube. This minor modification to the as-tested details is acceptable as it would not affect the performance of the system.

During the assembly of the test articles at the MwRSF Outdoor Test Site, the e8 bolts used to connect the top HSS tube rail were found to have inadequate thread length to properly tighten the nut at multiple locations. As such, two f2 washers were placed adjacent to the nut to tighten the bolt. If inadequate thread length exists for either of the tube-to-post connection bolt types, bolts e8 or e7, shown in Figure A-3, double washers should be used adjacent to the nut so that the bolt can be tightened.

5.1.5 Deck Requirements

As described in Section 5.1.1, the Steel Railing, Type IL-OH was developed for use on both CIP slab decks and pre-stressed box beam decks. The new bridge railing was MASH crash tested on a surrogate bridge deck, but only after an extensive analysis was conducted on the various deck configurations used in both Illinois and Ohio as part of the post-to-deck attachment design. That analysis led to the identification of the large box beam deck as the critical deck configuration for the evaluation of the bridge railing due to limited concrete cover for the anchor rods and the limited strength of the thin side walls. Dynamic component testing on W6x15 post assemblies mounted to a critical box beam girder demonstrated that these decks provide adequate anchorage strength and adequate wall strength to prevent crushing inward at the base of the post attachment. Thus, the post-to-deck attachment design should be acceptable for use on both deck types.

However, the deck geometries and reinforcement patterns applicable for use with the Steel Railing, Type IL-OH should be limited to those analyzed during this study until further research is conducted. Pre-stressed box-beams with heights between 17 in. and 42 in. are compatible with the new bridge rail. The pre-stressed box-beams should have a minimum top slab thickness of 5½ in., and the embedded anchor rods in the top slab should be placed below components of both the longitudinal and transverse reinforcing steel to ensure anchorage strength. Similarly, the side walls should have a minimum thickness of 5½ in. and minimum reinforcement of a #4 stirrup spaced at 9 in. on-center to prevent lateral crushing.

CIP slab decks should have an edge beam that extends at least 18 in. below the surface of the deck (not counting a wearing surface) and have a width of at least 9 in. The minimum reinforcement within the deck edge beam should incorporate #5 transverse hoops/stirrups spaced at 11 in. on-center and #4 longitudinal bars at each corner of the edge beam. The slab deck should be at least 7½ in. thick, and incorporate reinforcement with a minimum of a #5 transverse bar spaced every 5½ in. in the upper mat of steel. The embedded anchor rods must be placed below the upper mat of steel reinforcement.

5.1.6 Embedded Attachment Hardware

The embedded attachment hardware that was evaluated during the full-scale crash testing program is shown in Figure A-11, Option 1. This configuration was fully evaluated and is acceptable for use. However, the concrete deck spalled at one post location during test no. STBR-2, as shown in Figure 29, and required repair for subsequent crash tests. It is believed that this undesirable concrete damage was caused by the elastic restoration and/or rebound of the bridge railing, thus placing the lower anchors in tension. To further mitigate the risk of deck damage, additional tensile capacity is needed in the lower post-to-deck anchors.

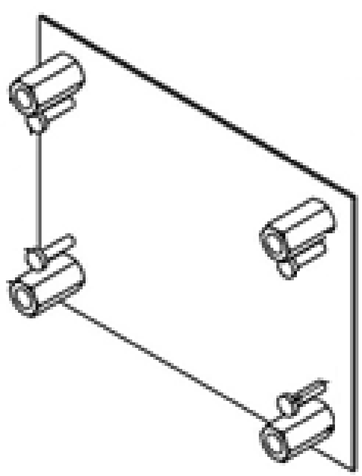
Figure 29. Concrete Damage Observed during Test No. STBR-2

Two modifications have been identified to increase the tensile strength of the lower anchors. The first is to use the embedded attachment hardware shown in Figure A-11, Option 2. The singular anchor plate running along the inside face of the coupling nuts will be enclosed by the transverse stirrups within the deck and provide significantly increased tensile strength to the lower anchors. The angular notches in the plate were designed to avoid interference with deck reinforcement in 17-in. deep box-beam girders and 18-in. deep CIP decks. These angular notches would not be required for deeper deck sections, as interference with the deck reinforcement is not a concern. Thus, a continuous, rectangular, internal plate could be used in lieu of the plate shown in Option 2 with angular notches, if desired, for deck sections deeper than 18 in.

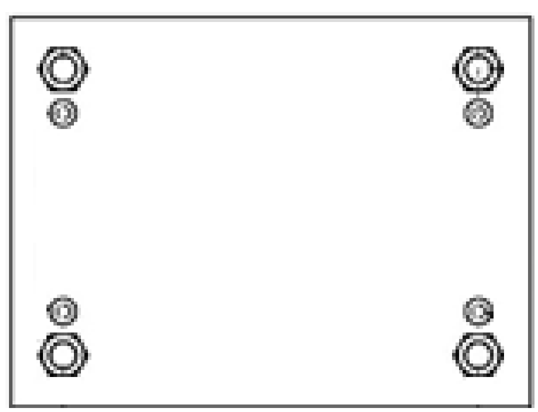
The second modification would be to increase the thickness of the deck to increase the distance between the lower anchor bolt and the bottom of the deck. This will increase the anchorage strength by eliminating edge effects, which are reductions in anchor strength due to the anchors being placed close to a free edge. Even an increase in 1-2 in. of deck thickness could result in up to a 30 percent increase in anchor strength.

In some instances, when the embedded anchorage assembly is inserted into the box-beam girder form, the bolt head of the lower anchors may interfere with the internal pre-stressing strands in the box-beam girders. In that case, the e11 bolts shown in Figure A-11 may be eliminated and the internal plate, parts c3 or c4 depending of the selected option, could be welded directly to the g4 coupling nuts to avoid interference with the pre-stressing strands.

For the construction of the surrogate concrete bridge deck, the embedded anchorage hardware was held in place by bolting through the form and into the coupling nuts rather than welding the coupling nuts to the edge plates, part c1 in Figure A-11, and tying the anchors to the deck reinforcement. When the bolts and formwork were removed, some edge plates detached from the edge of the deck. This detachment did not affect the performance of the system as the edge plates are only there as a template for the placement of the coupling nuts and to distribute compression loads into the deck edge. No tensile loads are applied to these edge plates. For future installations, it is recommended to either weld the coupling nuts to the edge plate or weld concrete shear studs to the inside face of the edge plate to hold the plate in place. Any shear studs applied to the plate should be placed along the vertical line between the coupling nuts to prevent interference with deck reinforcement, as shown in Figure 30. If the coupling nuts are welded to the edge plate, special considerations will need to be taken during galvanization so that the coupling nuts do not fill with galvanization.



Isometric View



Elevation View

Figure 30. Example of Shear Studs Applied to Inner Face of the Embedded Edge Plate

It was noted that the embedded edge plate, part c1, extends ⅝ in. below the face of 17-in. deep box beam girders, as shown Figure 31. For these situations, the bottom ⅝ in. of the edge plate can be trimmed off. This bottom strip of material coincides with the curved lower edge of the HSS spacer tubes, so this part of the plate is not in direct contact with the spacer tube. As such, the removal of a ⅝-in. strip of material from the edge plate should not affect the performance of the bridge railing.

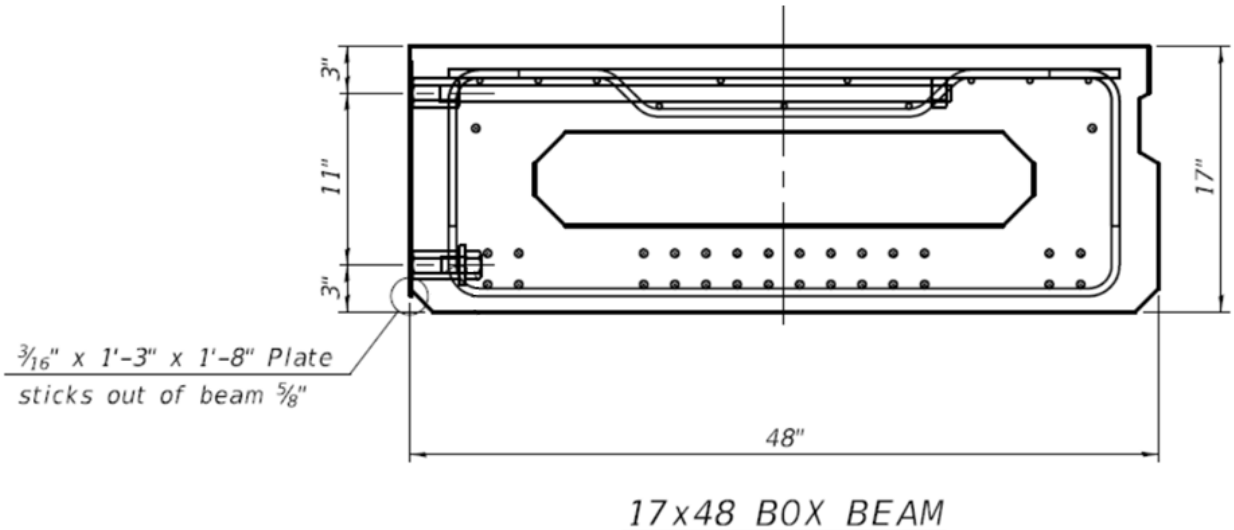


Figure 31. Edge Plate Extending from Face of 17-in. Box Beam

Note 3 on Figure A-11 specifies that the c2 threaded anchor rods are to be threaded 1¼ in. into the g4 coupling nuts. This distance is important as it controls the post-to-deck anchorage strength. If the threaded rods are not inserted far enough, the threads could be stripped and the anchorage would fail prematurely. If the threaded rod is inserted too far, it would limit the length in which the e10 bolts could be threaded into the coupling nuts and could lead to a premature failure on that end of the anchorage connection.

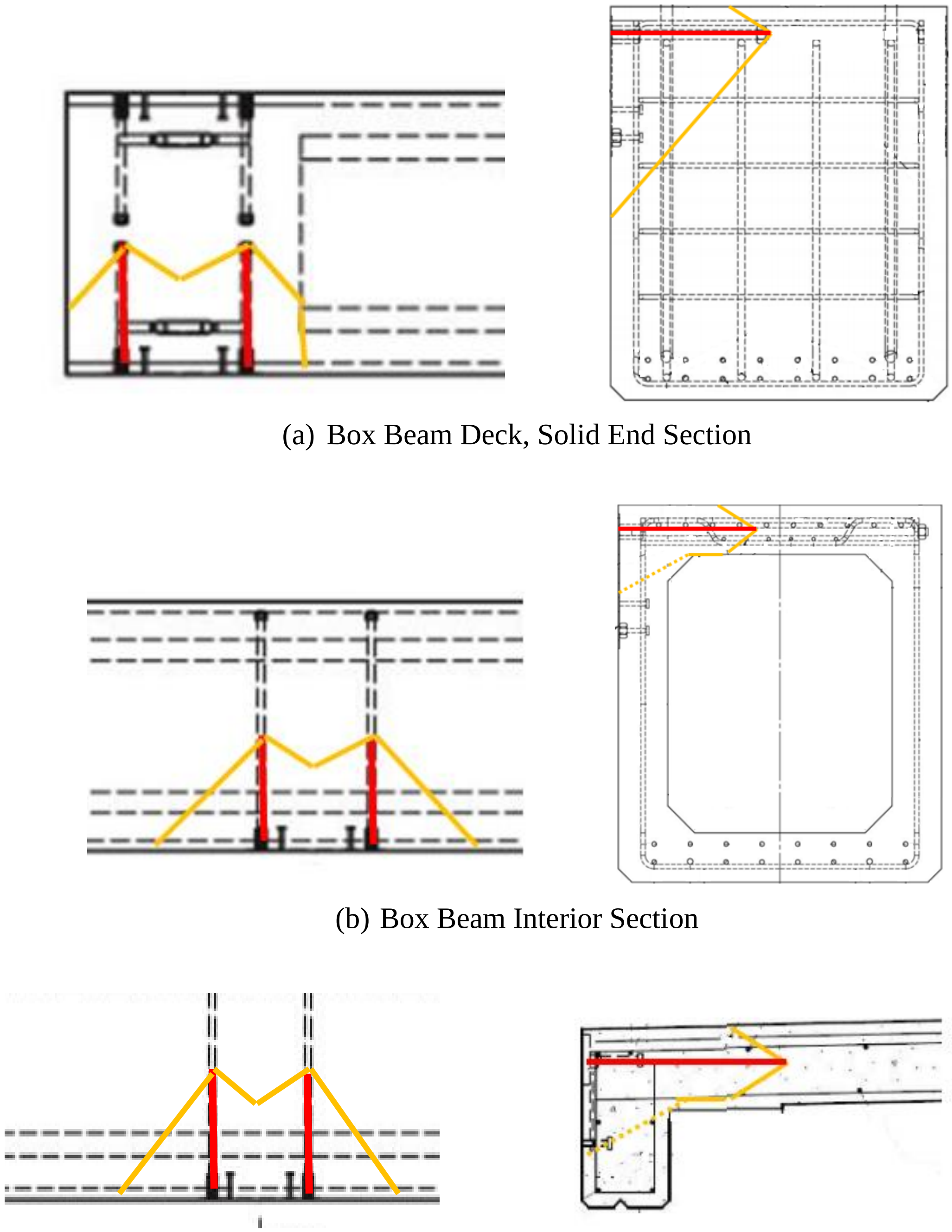
Both the component testing and the MASH crash testing was conducted using 32¾-in. long threaded anchor rods, part c2. This distance was selected, in part, to extend across the top slab of the 36-in. wide the concrete box beam and terminate above the opposite side wall. Concrete box beams wider than the 36-in. wide box beam used for component testing are often used to construct bridges in Illinois and Ohio. For these wider box beams, it is not necessary to extend the threaded anchor rods to reach the opposite side wall. The standard 32¾-in. long anchor should provide adequate strength to anchor the bridge railing posts.

At the ends of skewed bridges, the 32¾-in. long threaded anchor rods, part c2, may not be able to be properly embedded. As such, shorter anchor rods would be necessary to fit within these skewed bridge ends. During the dynamic component testing of the post-to-deck attachment hardware, anchor rods as short as 15 in. were evaluated and successfully anchored the post located within the solid section ends of the box beam. Thus, threaded anchors as short as 15 in. may be used in the solid end sections of box beams at the ends of skewed bridges.

Note, the 15-in. anchor rods have only been tested and evaluated in the solid end sections of box beams. Placement of the shorter anchors at interior locations and in the narrow top slab of a box beam would greatly reduce the concrete breakout surface area of the anchors, as shown in Figure 32, which results in a large reduction in the anchorage strength. Thus, it is not recommended to use the 15-in. long anchor rods within box beam interior sections until further evaluation and/or testing has been completed.

CIP slab decks are thicker than the top slab of box beam decks and offer some additional concrete breakout resistance, as shown in Figure 32. However, the breakout surface area is still limited by the narrow slab, so the 32¾-in. long threaded anchor rods are recommended for use in CIP slab decks. If the shorter 15-in. long anchors are desired for use in CIP slab decks, additional anchorage reinforcement must be added around the post locations to increase anchorage strength. Additional transverse reinforcement for CIP slab decks should consist of a minimum of two no. 6 bars per anchor, or four no. 6 bars per post, to develop the anchorage strength of the as-tested system. This additional steel should be hooked near the deck edge and extend far enough into the deck to ensure proper development length of the bars past the anchors. Note, ILDOT and ODOT currently place additional transverse reinforcement in their deck edges adjacent to each of the regular transverse bars in their decks (typically spaced at 6 in.). Further, these bars are already designed with hooks at the deck edge and extend over 5 ft into the deck. If these additional reinforcing bars were designated as no. 6 bars, they should provide the anchorage strength necessary to utilize the 15-in. long threaded anchors.

Even with the use of 15-in. long anchor rods, certain combinations of bridge skew angles and post spacing to the end of the bridge deck will result in the interior end of the anchors being located close to the bridge end. If this distance is too short, the concrete breakout surface will extend to the end of the deck instead of out to the edge of the deck and the anchorage strength will not be sufficient to anchor the posts. Thus, the interior end of the anchors should be a minimum of 7 in. from the skewed deck end, as shown in Figure 33. To achieve this minimum distance, the post may be shifted away from the deck end. Recall that post spacings may be reduced from the nominal 8 ft to as little as 4 ft in order to avoid installation obstructions, as described in Section 5.1.1. Shifting the bridge railing end post toward the middle of the bridge will result in the AGT posts shifting toward the bridge as well. However, the 9-ft spacing between the last AGT post and the first bridge railing post should be sufficient to extend over any ground obstructions (e.g., bridge abutments, wing walls, etc.) and allow proper placement of the AGT posts after a minor longitudinal shift.



(a) Box Beam Deck, Solid End Section

(b) Box Beam Interior Section

(c) CIP Slab Deck

Figure 32. Depictions of 15-in. Long Anchor Rods (Red) and their Corresponding Concrete Breakout surface (Orange) in Various Deck Configurations.

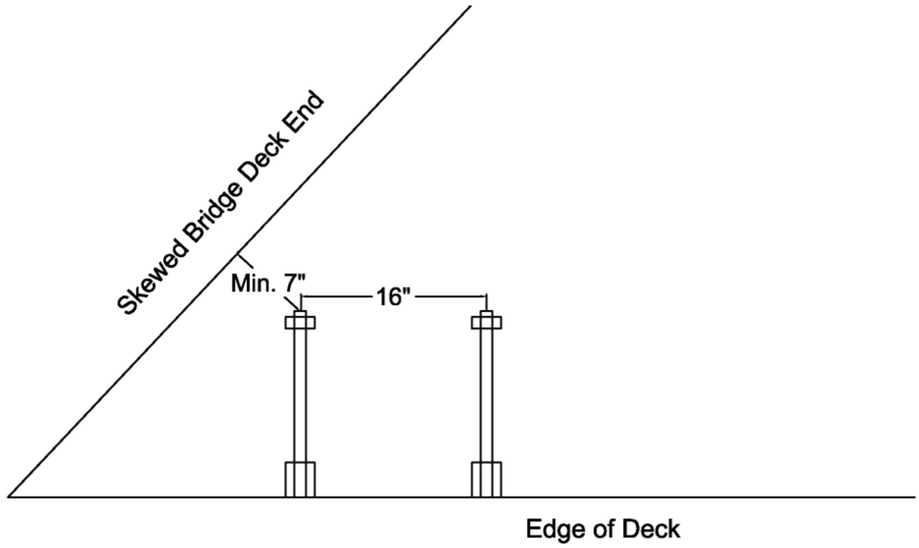


Figure 33. Minimum Distance for Anchor Placement near Skewed Deck Ends

5.2 Transition Implementation Guidance

5.2.1 Modifications After Overlays

The AGT connected to the new Steel Railing, Type IL-OH was designed to be compatible with future overlays up to 3 in. thick, just like the bridge railing. This AGT was originally developed and successfully MASH crash tested by the Nebraska DOT. The 34-in. mounting height is created by raising the thrie beam segments and associated blockouts 3 in. on the posts, as shown in Figure A-22. Note, the posts were not raised and still have their nominal embedment depths to maintain strength and stiffness in the AGT. A symmetric W-to-thrie transition rail segment is used to transition the rail to MGS with a nominal height of 31 in., as shown in Figure A-21.

After a 3-in. roadway overlay, the thrie beam portion of the AGT would be at its nominal height of 31 in. However, minor modifications are necessary to the upstream end of the transition to maintain MASH crashworthiness. First, the W-beam rail and blockouts in the MGS region of the installation should be raised 3 in. and reattached to the original posts. Previous research determined that raising guardrail in such a manner was acceptable for vertical shifts up to 4 in. [25-27], which is greater than the 3 in. recommended herein. This process allows the MGS rails to be raised to their nominal height without having to replace or reset the posts while also maintaining the nominal post embedment depth.

Second, the symmetric W-to-thrie transition segment would be replaced with an asymmetric rail segment, matching the original MGS stiffness transition design. Thus, by replacing only a single rail element and shifting the existing W-beam up 3 in., the entire transition system would be at its nominal 31-in. mounting height and would maintain its crashworthiness after a 3-in. roadway overlay. Drawings of the AGT both before and after an overlay are shown in Figures 34 through 36.

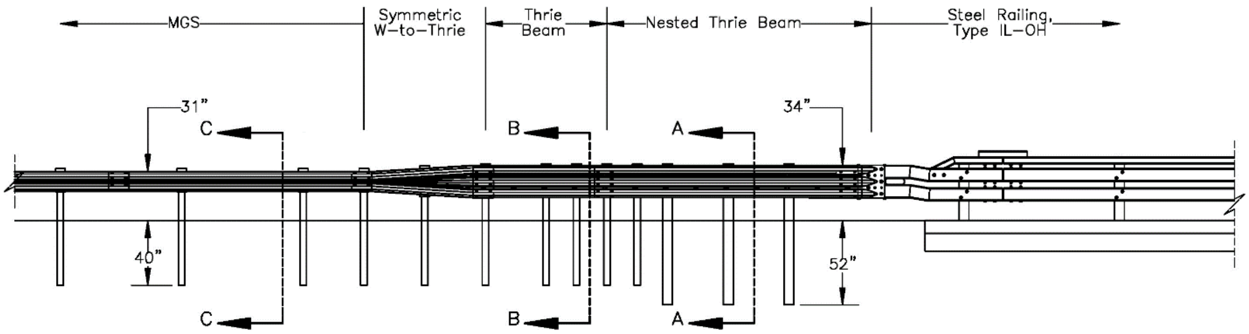


Figure 34. AGT Configuration at Time of Initial Installation, No Overlay

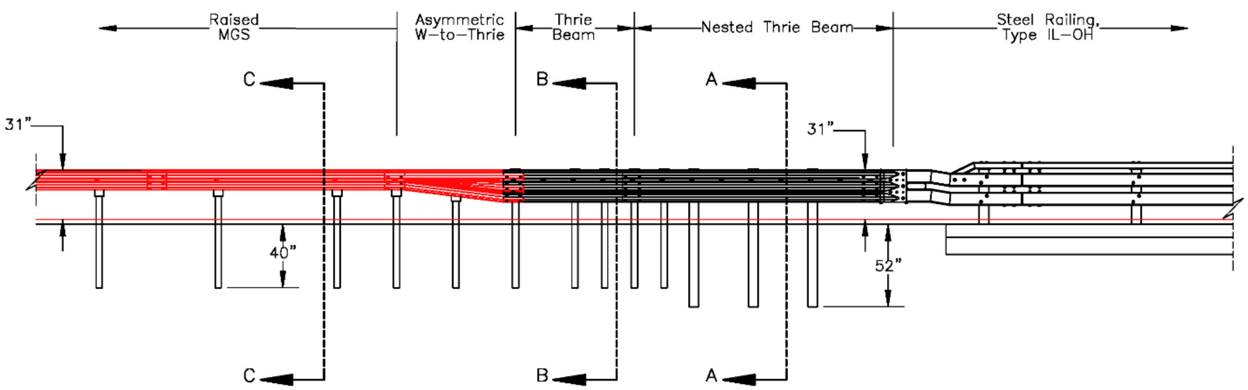


Figure 35. AGT Configuration After a 3-in. Roadway Overlay

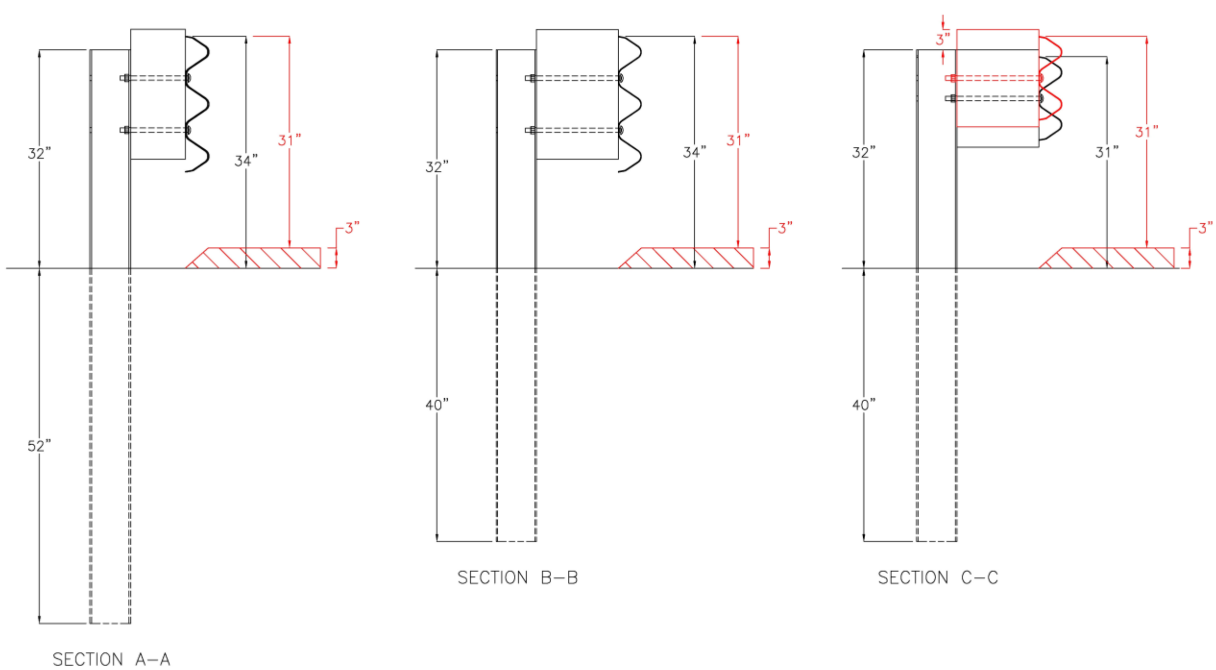


Figure 36. System Cross-Sections Before and After a 3-in. Overlay

It was assumed that any roadway overlays would be extended laterally at least to the face of the rail, but not farther than the face of the posts. Extending an overlay past the posts would increase the embedment depth and stiffen the soil resistance around the posts. Previous crash testing has shown this to alter the behavior of the posts, increase rail pocketing and stresses, and ultimately lead to rail rupture. As such, any applied roadway overlay should not be extended beyond the face of the posts unless leave-outs are placed around the posts.

5.2.2 AGT Component Modifications

As described in Section 5.1.1, the Steel Railing, Type IL-OH can be installed with a top height of 36 in. on bridges that will not be subjected to future overlays. On such installations, the attached AGT should be installed with a constant 31-in. guardrail mounting height. The blockouts and thrie beam rail segments should be attached 3 in. lower on the d4 and d1 posts shown in Figures A-21 and A-22. Also, an asymmetric W-to-thrie transition segment should be used for the 31-in. tall AGT instead of the symmetric transition rail. Note, no changes should be made to the post embedment depths. However, the reduced rail height and use of the asymmetric transition rail will shift the locations of the bolt holes in the transition posts, parts d1, d2, and d4.

It should be noted that the HSS transition rail assemblies shown in Figures A-16 through A-20 are not symmetric components. As such, the details would need to be flipped to address the AGT connection on the other side of the bridge (similar to the right- and left-side orientations of the asymmetric W-to-thrie transition rail).

The connection of the thrie beam terminal connector to the HSS transition rail assemblies was designed and tested utilizing the same ¾-in. diameter, e7 bolts that were used to connect the lower and middle HSS tube rails to the bridge railing post, as shown in Figure A-14. Guardrail terminal connectors are typically made with 1-in. diameter holes and connected with ⅞-in. diameter bolts. An analysis of the connection showed that the smaller diameter bolts still provided a higher shear resistance than the bearing failure of the terminal connector. Full-scale testing showed no issues with the use of these e7 bolts. However, if desired, ⅞-in. diameter round head bolts may be used in this connection to better match the holes in the terminal connector. Note, the holes in the HSS transition rail components, parts h6 and h9, were also detailed with 1-in. diameter holes, as shown in Figures A-17 and A-18.

The bolted connection between the lower and middle HSS transition tube rails, parts h6 and h9, and the angled cut tube assembly, part h1, that is sandwiched between them was designed with holes in the angled cut tube and slots in the transition tube rails, as shown in Figures A-15, A-17, and A-18. These slots were added to allow for some construction tolerances in the system during assembly of the components. The slots on the top surface of the middle transition rail may result in water getting into the tube rail. As such, the slots could be placed in the angled cut tube and holes could be used in the transition rails to prevent water damage. Note, the holes and slots would remain centered at their current locations and maintain their ⅞ in. diameter.

The AGT to Steel Railing, Type IL-OH incorporated 8-in. deep blockouts on the W6x15 posts within the downstream end of the transition and 12-in. deep blockouts on the W6x8.5 posts within the upstream MGS stiffness transition. Utilizing 12-in. deep blockouts throughout the AGT may help reduce vehicle snag on the larger transition posts, as the posts would need to be offset 4 in. farther away from the rail. Thus, incorporating 12-in. deep blockouts throughout the AGT should also be considered a crashworthy configuration. However, the upstream stiffness transition was developed and tested exclusively with 12-in. deep blockouts. Full-scale testing of the MGS stiffness transition did result in moderate vehicle snag on the guardrail posts when impacted with the small car [16-17, 28-29]. There are concerns that reducing the blockout depth in the MGS stiffness transition may result in increased vehicle snag. Consequently, blockouts less than 12 in. deep are not recommended for use within the upstream stiffness transition until further analysis is conducted.

5.2.3 Minimum Length of MGS Upstream from the AGT

Guardrail end terminals are designed, crash tested, and evaluated for use when directly attached to semi-rigid W-beam guardrail systems instead of the stiff approach guardrail transitions. The introduction of reduced post spacing and larger rail elements, which are required for AGTs, may lead to degraded performance of the crashworthy terminal. Additionally, the placement of the upstream end anchorage too close to the AGT may negatively affect system performance, thus potentially resulting in excessive barrier deflections, vehicle pocketing, wheel snagging on posts, vehicle-to-barrier override, or other vehicle instabilities. Thus, the following implementation guidelines should be considered when configuring the MGS upstream from the AGT:

1. A recommended minimum length of 12 ft – 6 in. of standard MGS should be installed between the upstream end of the W-to-thrie transition segment and the interior end of an acceptable TL-3 guardrail end terminal. In other words, the guardrail terminal’s stroke length should not intrude within 12.5 ft of the stroke length W-to-thrie transition segment.

2. A recommended minimum barrier length of 46 ft – 10½ in. is to be installed beyond the upstream end of the W-to-thrie transition segment, which includes standard MGS, a crashworthy guardrail end terminal, and an acceptable anchorage system. This distance is based on crash tested system installation lengths.

3. For flared guardrail applications, a minimum length of 25 ft is recommended between the upstream end of the W-to-thrie transition segment and the start of the flared section (i.e., bend between flare and tangent sections).

5.2.4 Wood Post AGT Alternative

The AGT tested in combination with the Steel Railing, Type IL-OH was a steel post system originally developed and tested by the Nebraska DOT. Previous studies have been conducted comparing the performances of various sizes of both steel and wood posts. In one study, a 6.5-ft long 8-in. x 10-in. wood post was found to provide equivalent strength and stiffness to the 7-ft long W6x15 post used in the AGT shown in Appendix A [30]. In the same study, 6-ft long 6-in. x 8-in. wood posts were found to be equivalent to 6-ft W6x8.5 steel posts. Although this previous study was conducted utilizing 31-in. tall guardrail, other studies have been conducted showing that guardrail can be raised up to 4 in. on both steel and wood posts without negatively affecting their performance [25-27]. Therefore, a crashworthy wood-post alternative for the AGT connected to the new Steel Railing, Type IL-OH would consist of the use of 6.5-ft long 8-in. x 10-in. posts and 6-ft long 6-in. x 8-in. wood posts, as shown in Figure 37.

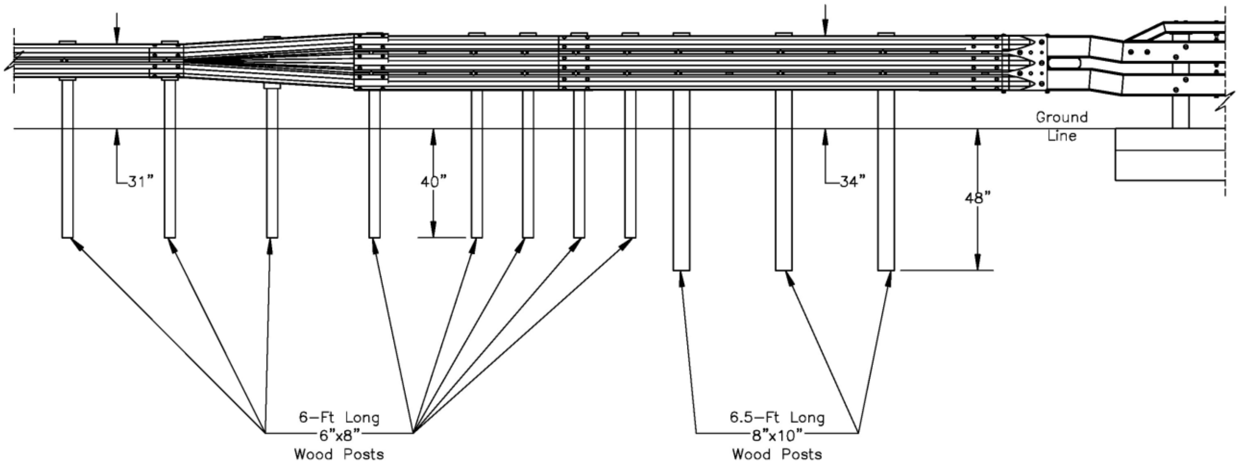


Figure 37. Wood Post Alternative Approach Guardrail Transition

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7 APPENDICES

Appendix A. Drawing Details for Steel Railing, Type IL-OH

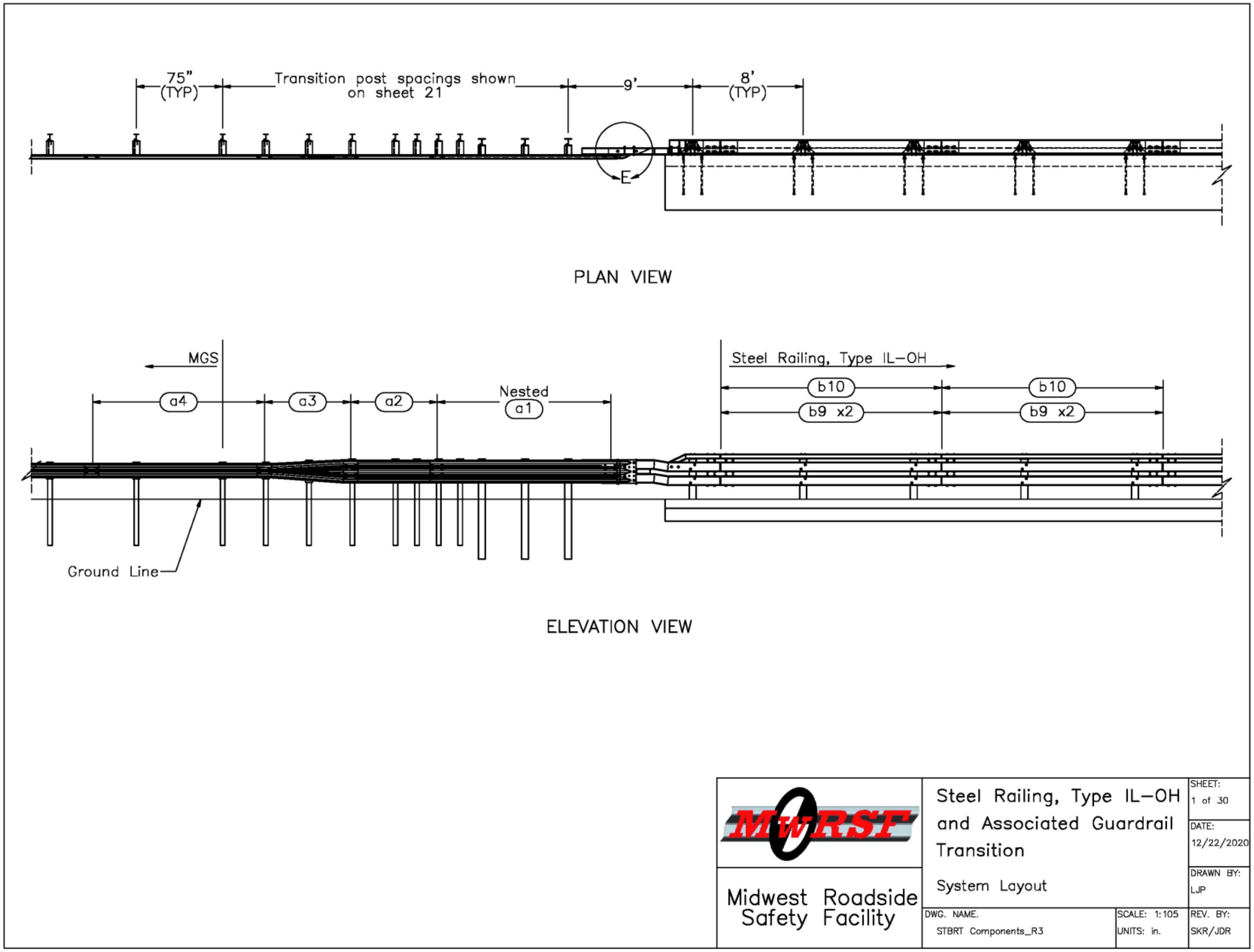


Figure A-1. Steel Railing, Type IL-OH and Associated Guardrail Transition

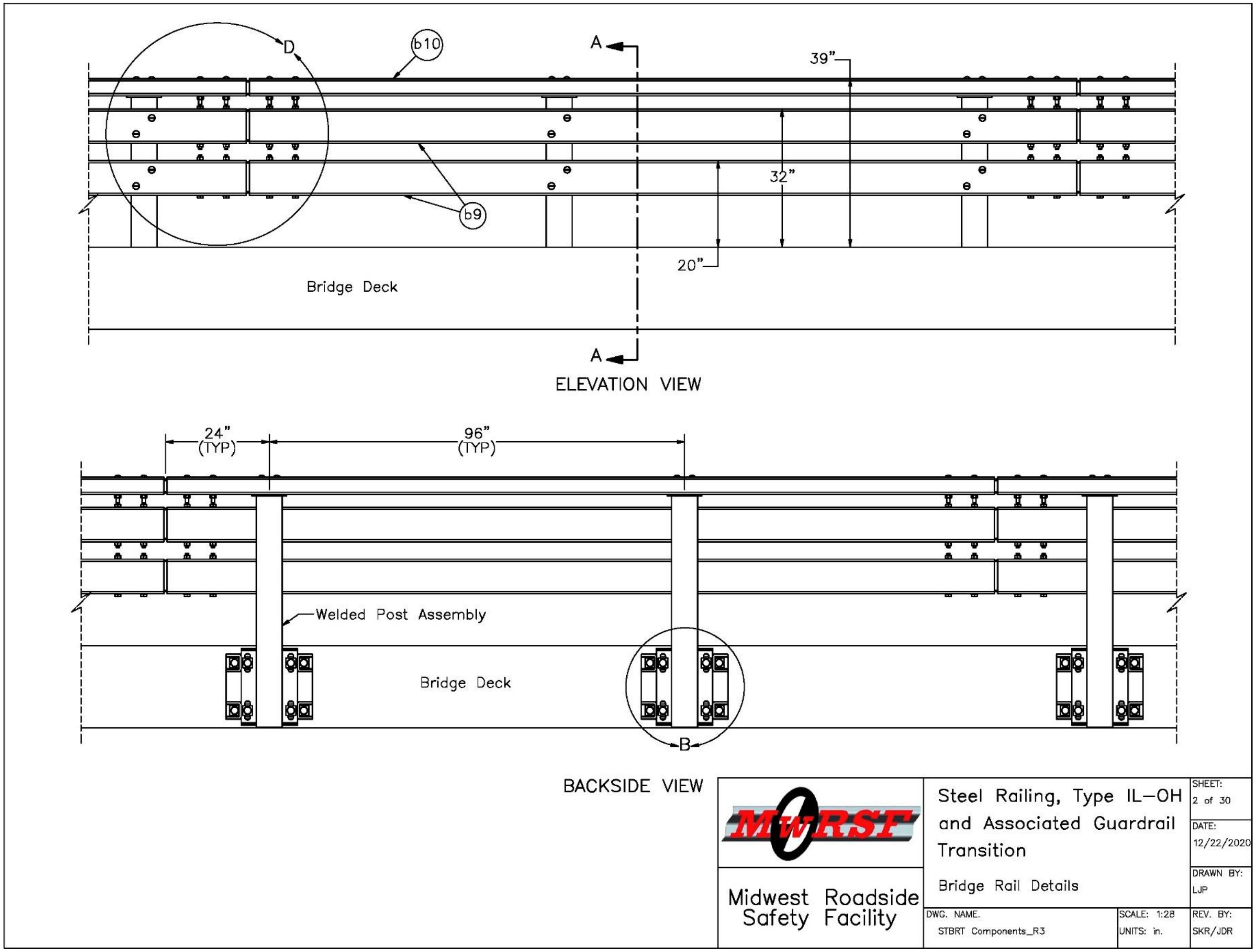


Figure A-2. Steel Railing, Type IL-OH, Elevation View

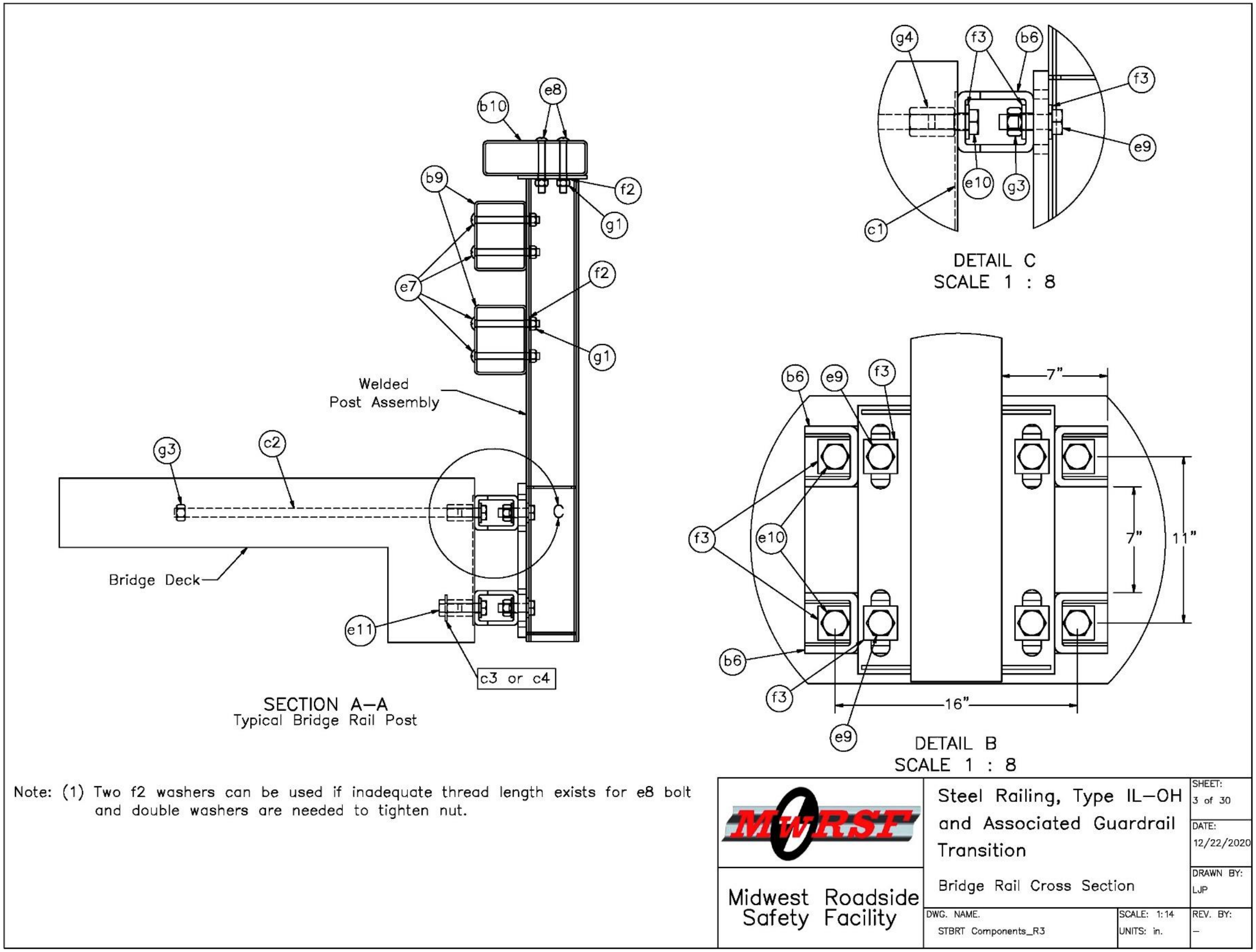
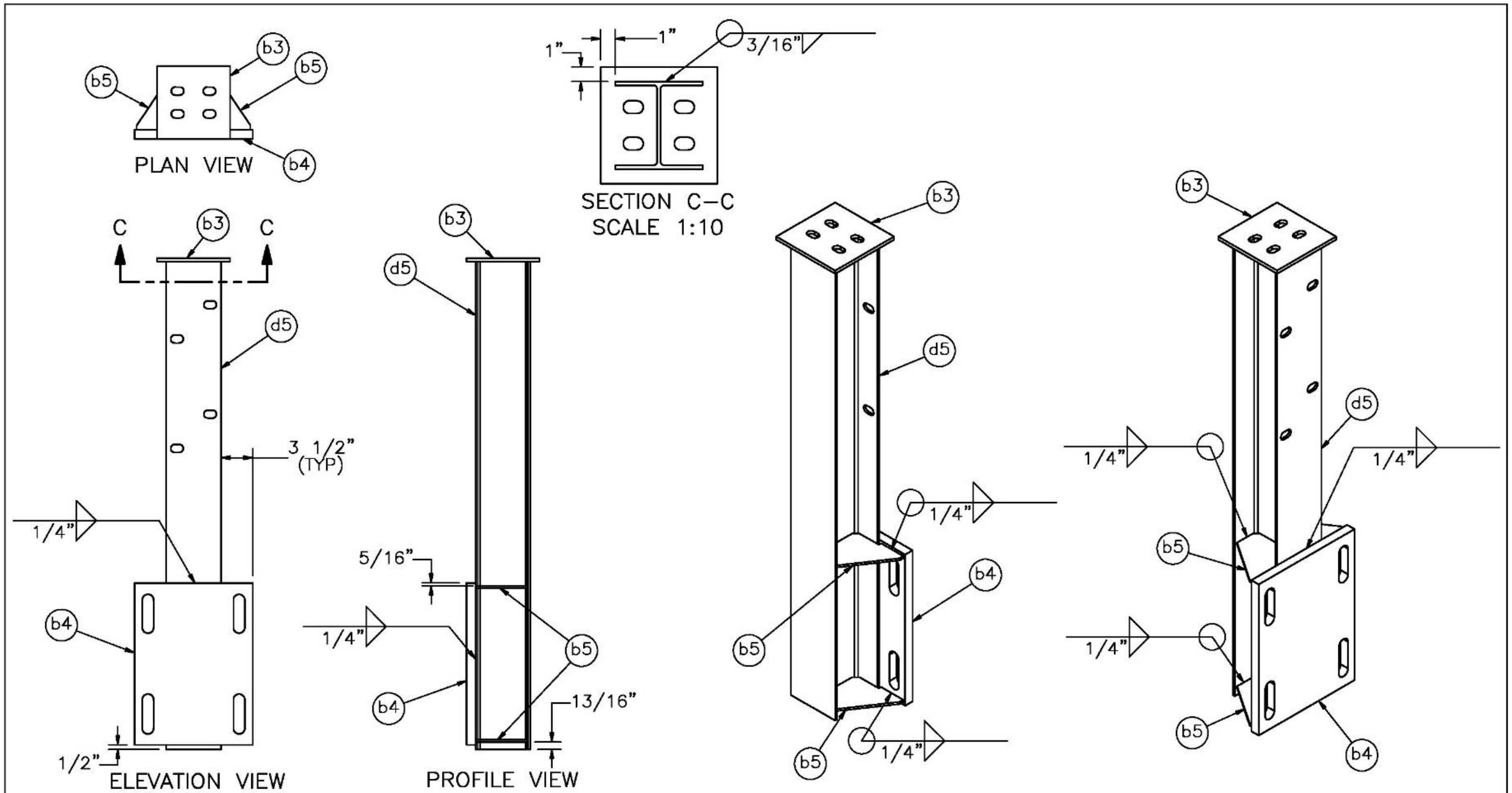


Figure A-3. Steel Railing, Type IL-OH, Cross Section and Attachment Details



Notes: (1) Gussets on both sides of post.
(2) To prevent welds to end at edges of gussets, leave 5/16" of base material beyond weld to edge of welded part.
(3) Leave coped corners of gussets open for drainage. Do not fill with welds.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	–	Welded Post Assembly	–	ASTM A123
b3	1	8"x8"x3/8" Plate	ASTM A572 Gr. 50	See Assembly
b4	1	17 3/4"x13"x1" Post Plate	ASTM A572 Gr. 50	See Assembly
b5	4	6 1/8"x5 11/16"x1/4" Gusset Plate	ASTM A572 Gr. 50	See Assembly
d5	1	W6x15 Post, Length Varies	ASTM A992	See Assembly

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Steel Railing, Type IL–OH and Associated Guardrail Transition

Welded Post Assembly

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DATE: 12/22/2020

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SCALE: 1:16 UNITS: in.

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Figure A-4. Steel Railing, Type IL-OH, Welded Post Assembly

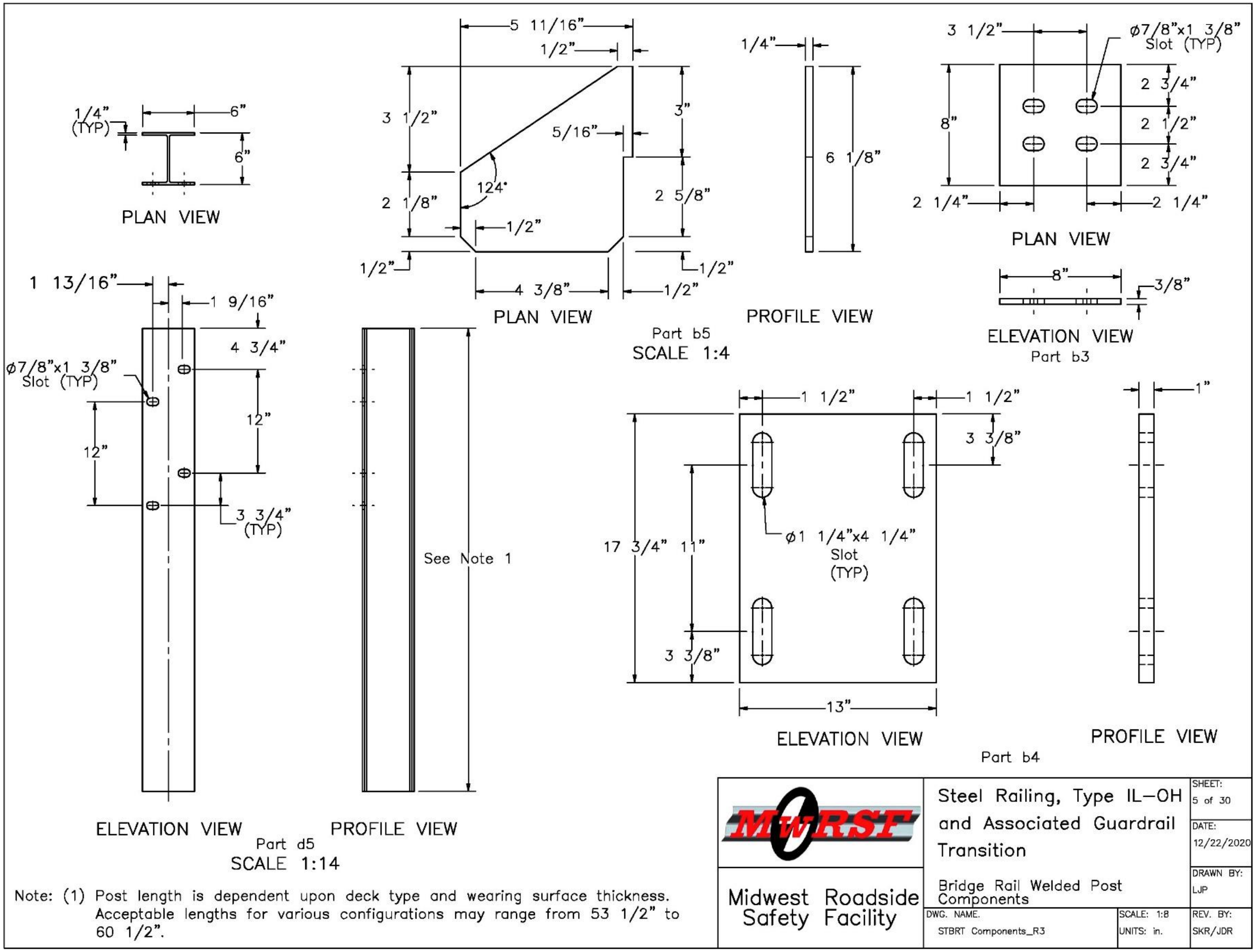


Figure A-5. Steel Railing, Type IL-OH, Post Assembly Components

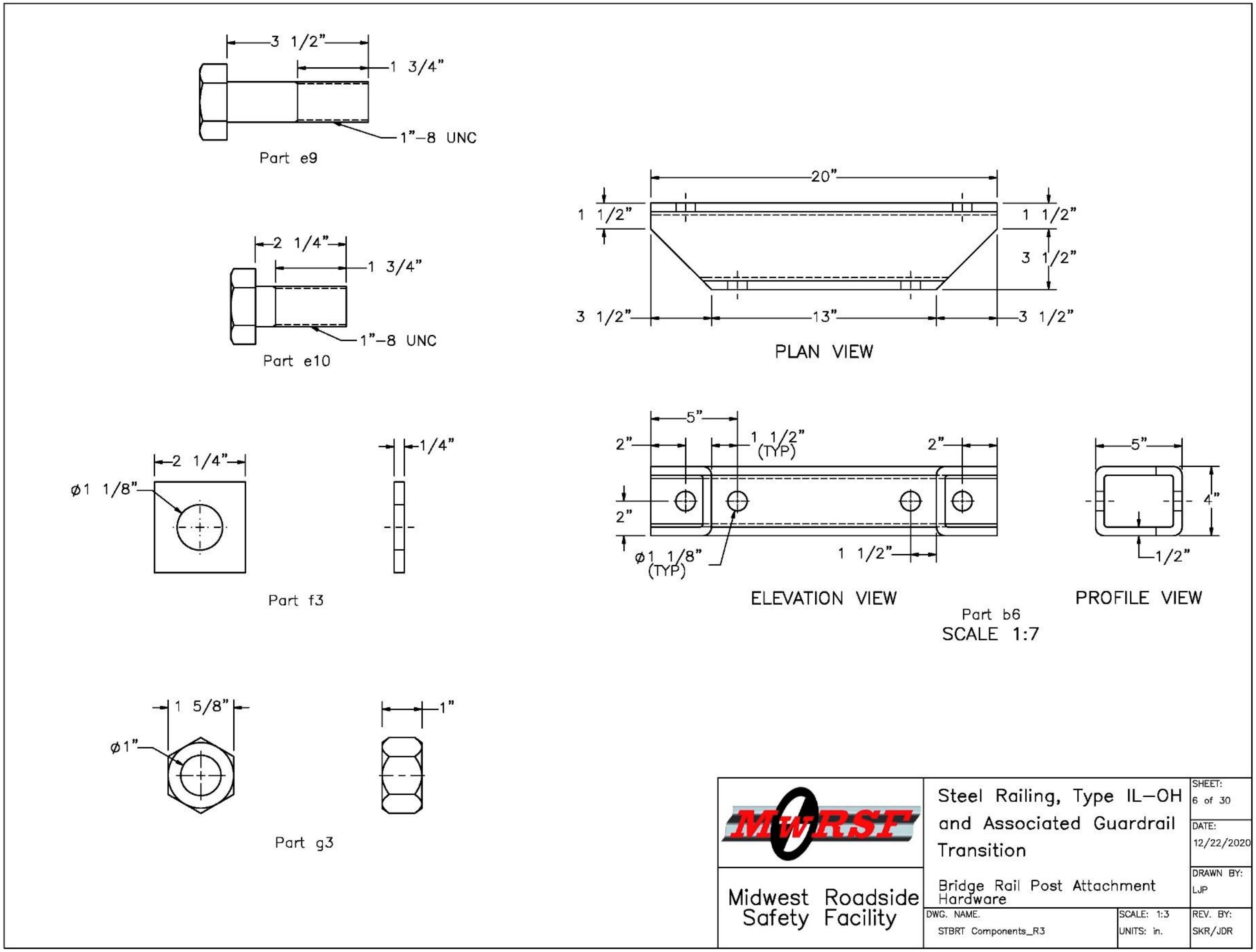


Figure A-6. Steel Railing, Type IL-OH, Post Attachment Hardware

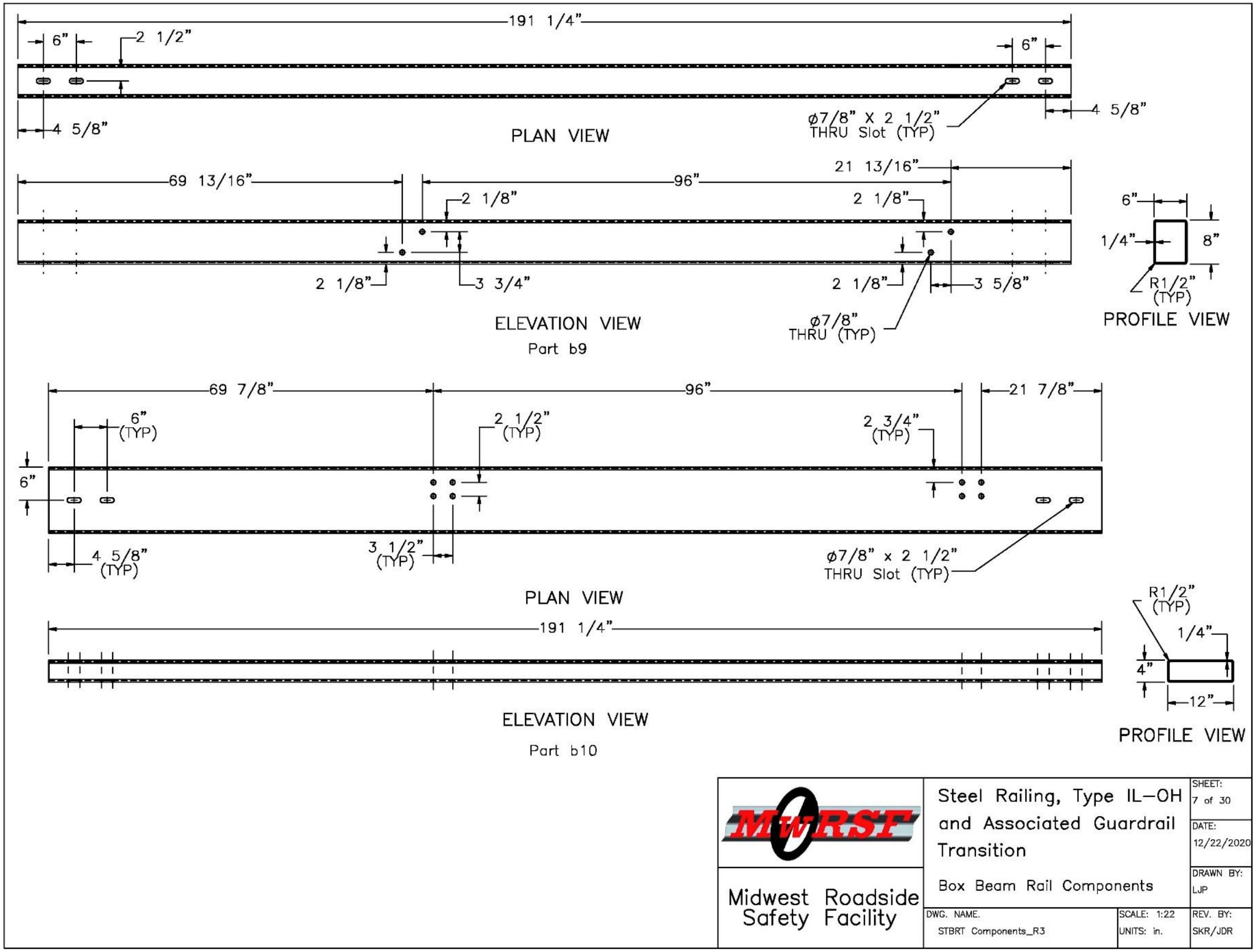


Figure A-7. Steel Railing, Type IL-OH, Rail Components

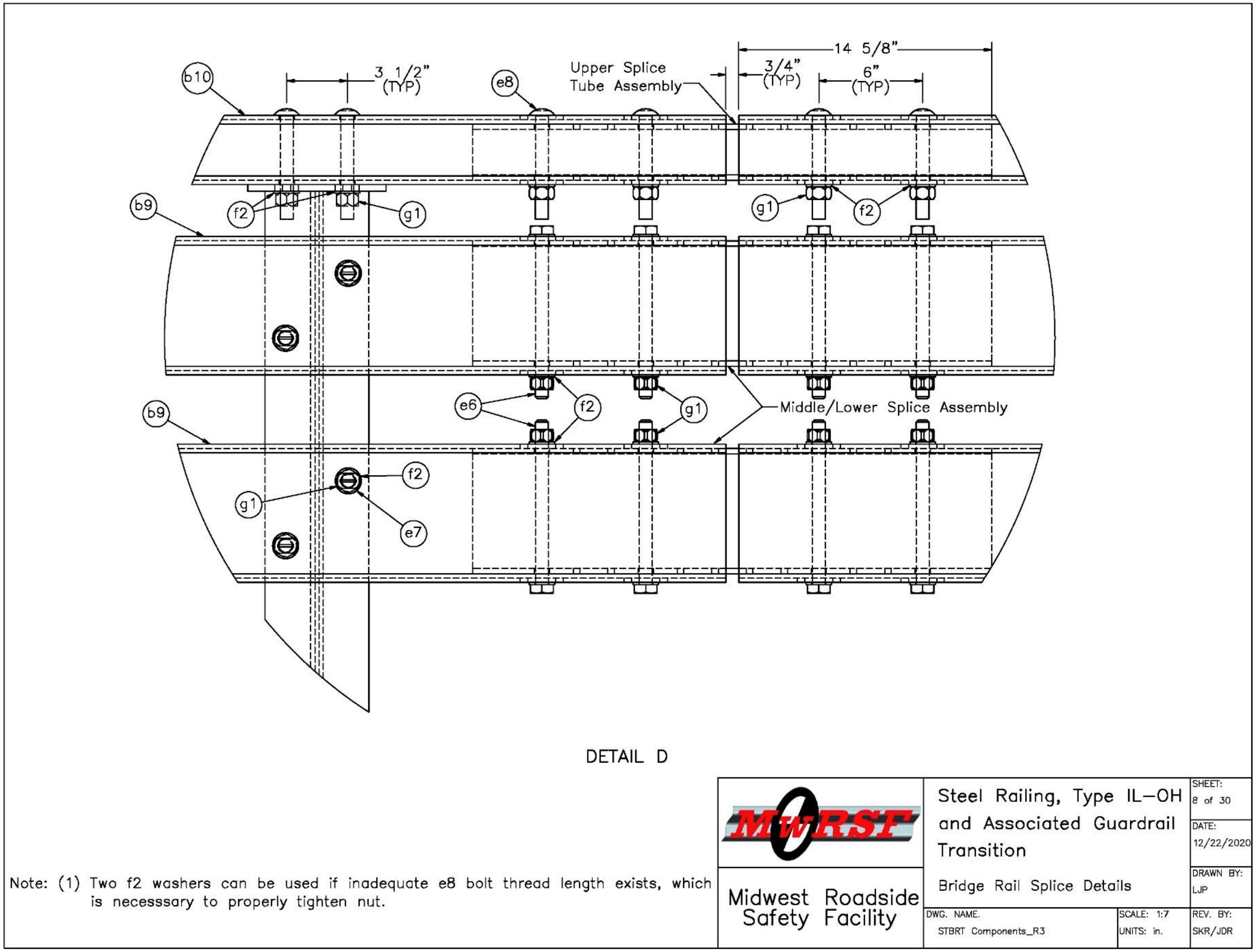
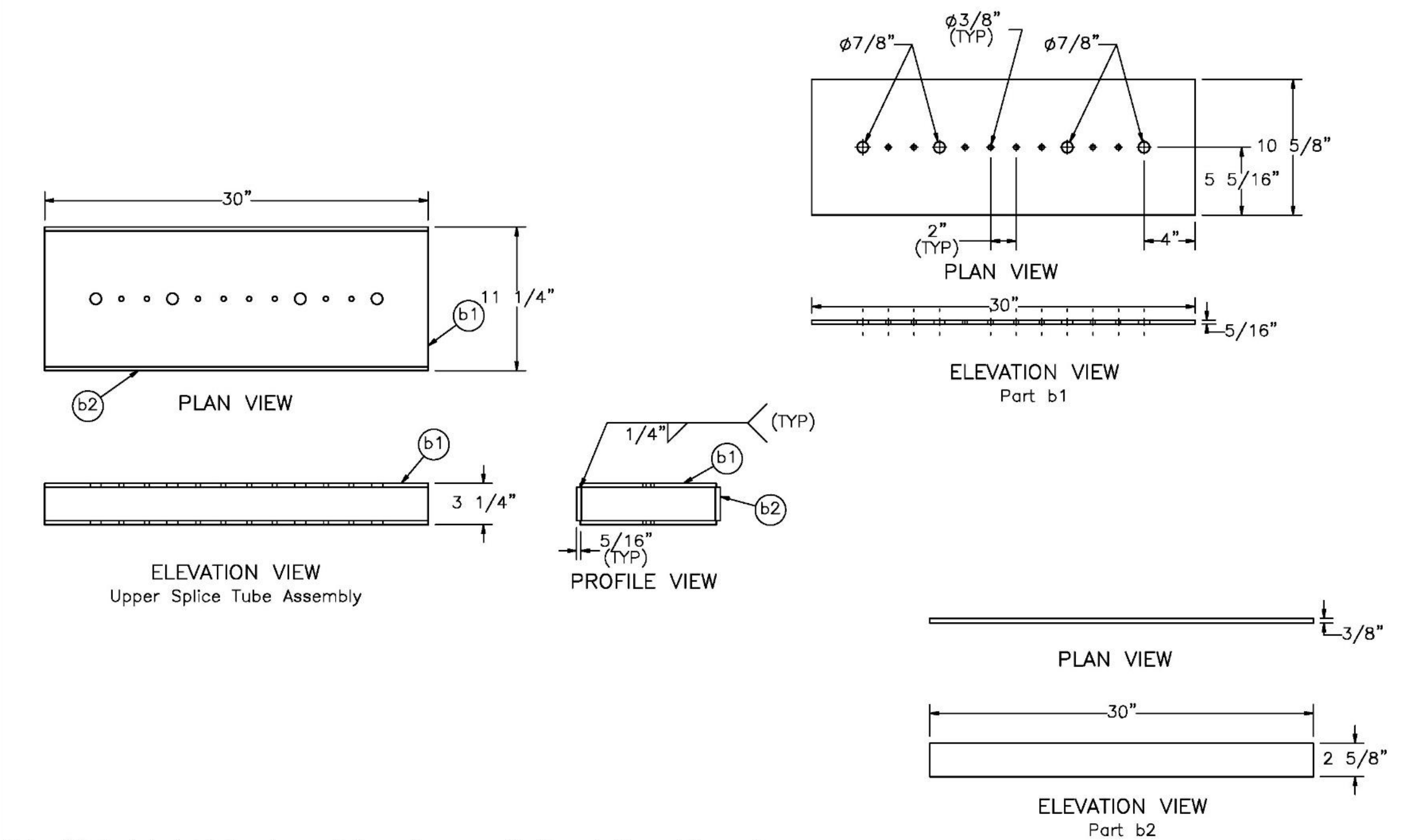


Figure A-8. Steel Railing, Type IL-OH, Railing Splice Details



Note: (1) To help hold the shape of the splice assembly through the welding and galvanizing processes, it is recommended to use gussets or shim blocks.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	–	Upper Splice Tube Assembly – Option 1	–	ASTM A123
b1	2	30"x10 5/8"x5/16" Plate	ASTM A572 Gr. 50	See Assembly
b2	2	30"x2 5/8"x3/8" Plate	ASTM A572 Gr. 50	See Assembly

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Steel Railing, Type IL–OH and Associated Guardrail Transition

Upper Splice Tube Assembly

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DWG. NAME. STBRT Components_R3
SCALE: 1:11
UNITS: in.
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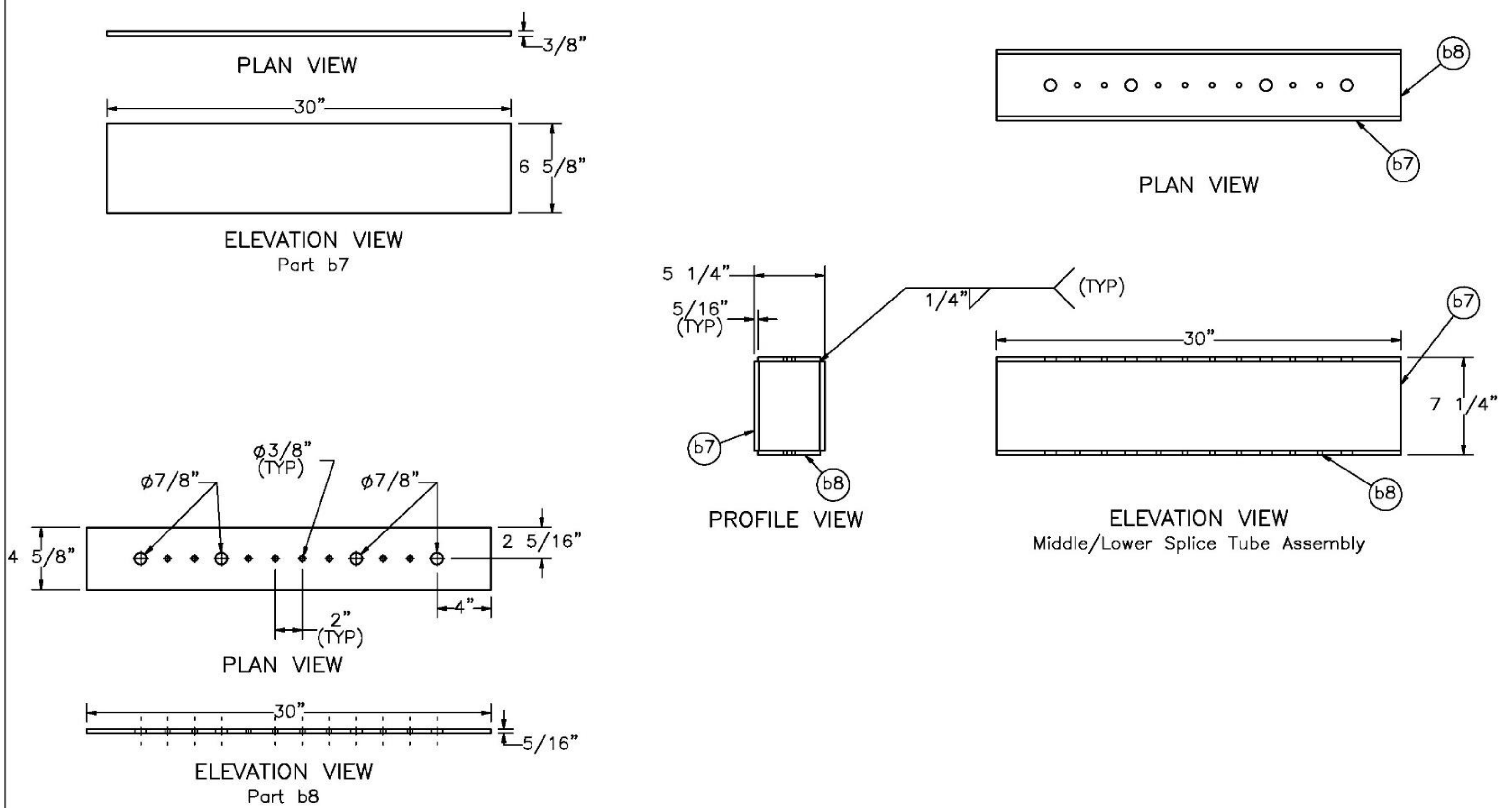
Figure A-9. Steel Railing, Type IL-OH, Upper Splice Tube Assembly



Note: (1) To help hold the shape of the splice assembly through the welding and galvanizing processes, it is recommended to use gussets or shim blocks.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	–	Middle/Lower Splice Tube Assembly – Option 1	–	ASTM A123
b7	2	30"x6 5/8"x3/8" Plate	ASTM A572 Gr. 50	See Assembly
b8	2	30"x4 5/8"x5/16" Plate	ASTM A572 Gr. 50	See Assembly

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Steel Railing, Type IL–OH and Associated Guardrail Transition

Middle/Lower Splice Tube Assembly

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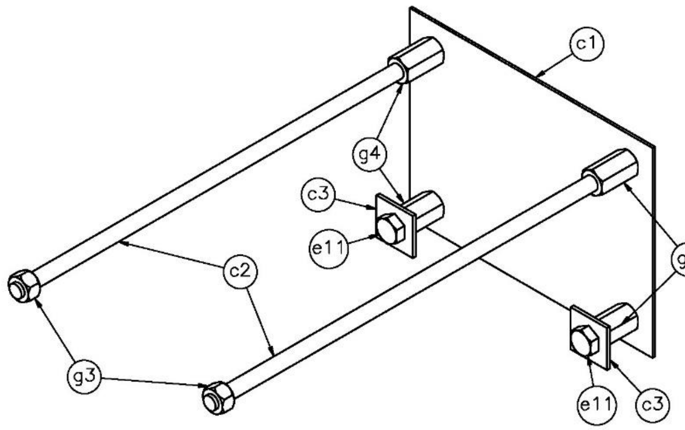
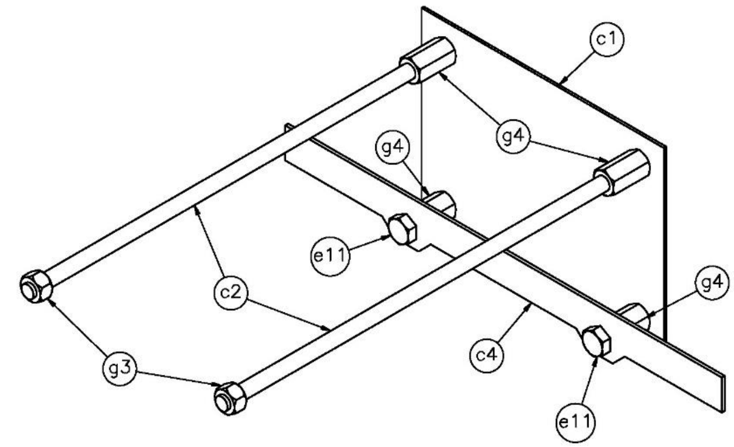
DWG. NAME. STBRT Components_R3	SCALE: 1:11 UNITS: in.	REV. BY: SKR/JDR

Figure A-10. Steel Railing, Type IL-OH, Middle/Lower Splice Tube Assembly



ISOMETRIC VIEW
Bridge Rail Welded Post Assembly Attachment
Option 1



ISOMETRIC VIEW
Bridge Rail Welded Post Assembly Attachment
Option 2

Notes: (1) Connector plate assembly is symmetric along the centerline.
(2) During casting, coupling nuts (part g4) may be held in place by extending bolts through the formwork into the nuts, or by tack welding nuts to inside face of the plate (part c1). The welds are not structurally important.
(3) Part c2 is threaded 1 1/4" into the coupling nut (part g4).
(4) Option 2 is to be used in all 17" deep box girders and 18" thick bridge decks. May be used for all decks if desired.
(5) Part c4 may be welded to coupling nut (part g4) to avoid bolt interference with prestressing strands if necessary.

MwRSF	Steel Railing, Type IL–OH and Associated Guardrail Transition	SHEET: 11 of 30
		DATE: 12/22/2020
Midwest Roadside Safety Facility	Bridge Rail Welded Post Attachment Options	DRAWN BY: LJP
	DWG. NAME. STBRT Components_R3 / SCALE: 1:9 UNITS: in.	REV. BY: SKR/JDR

Figure A-11. Steel Railing, Type IL-OH, Embedded Attachment Hardware

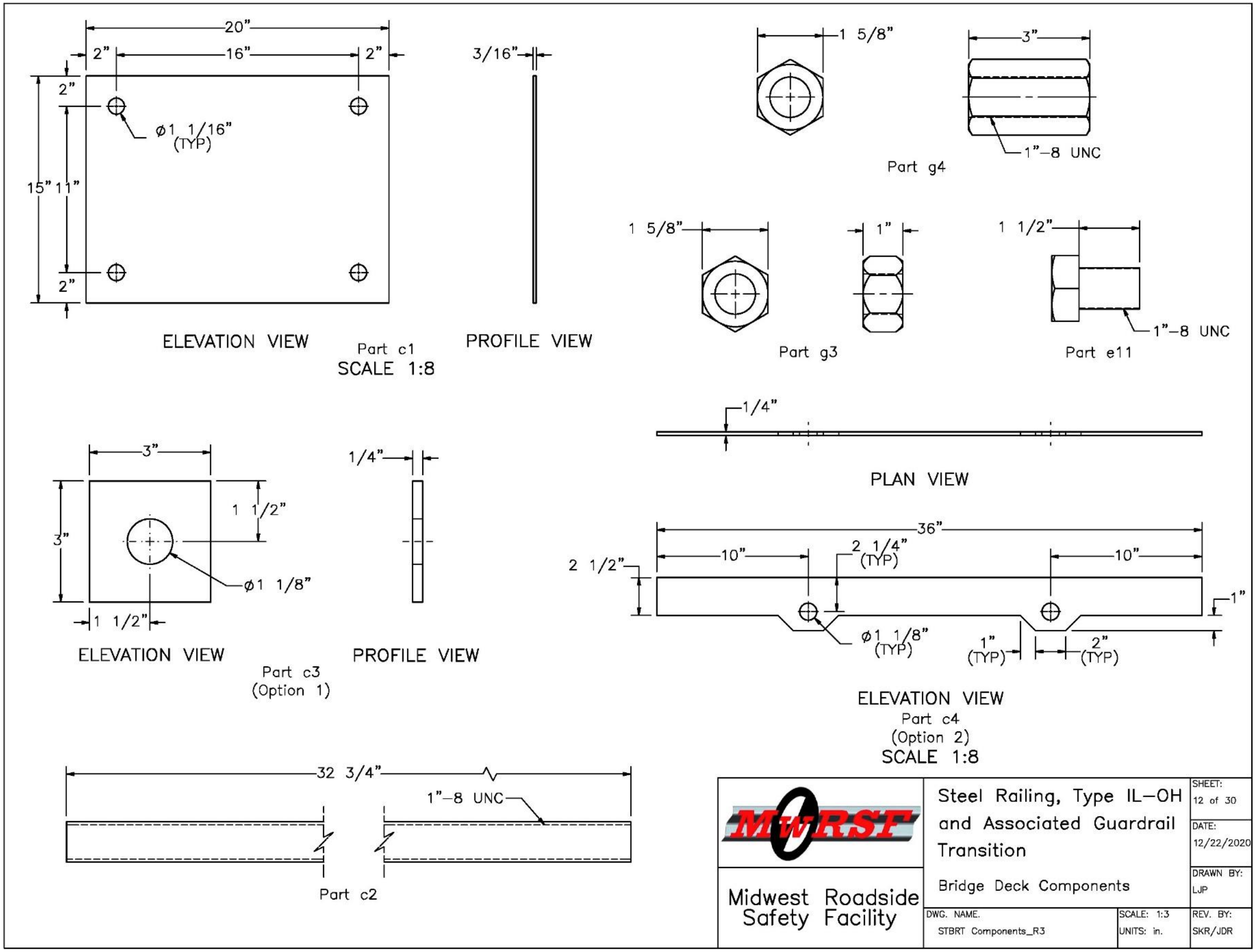


Figure A-12. Steel Railing, Type IL-OH, Embedded Attachment Hardware

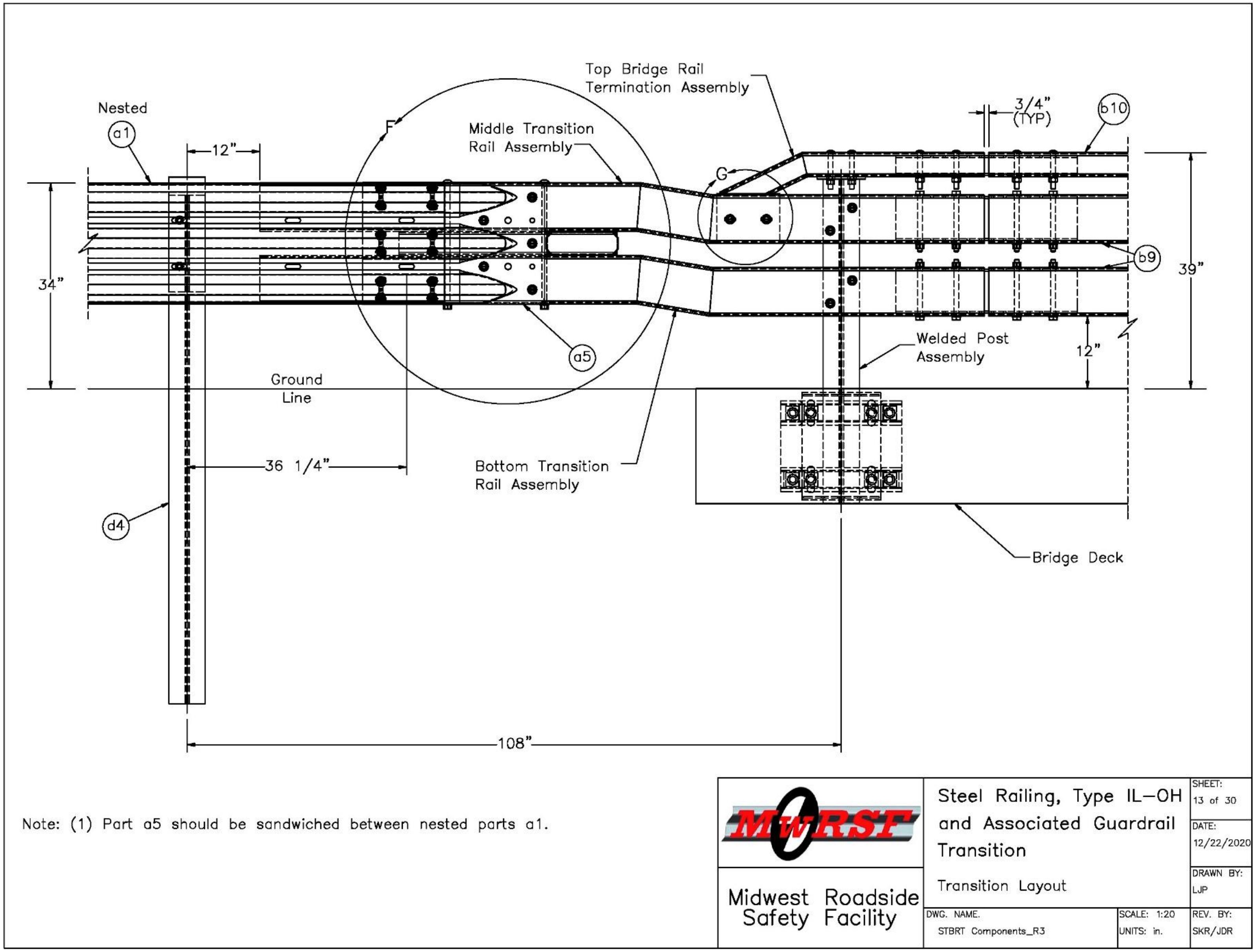
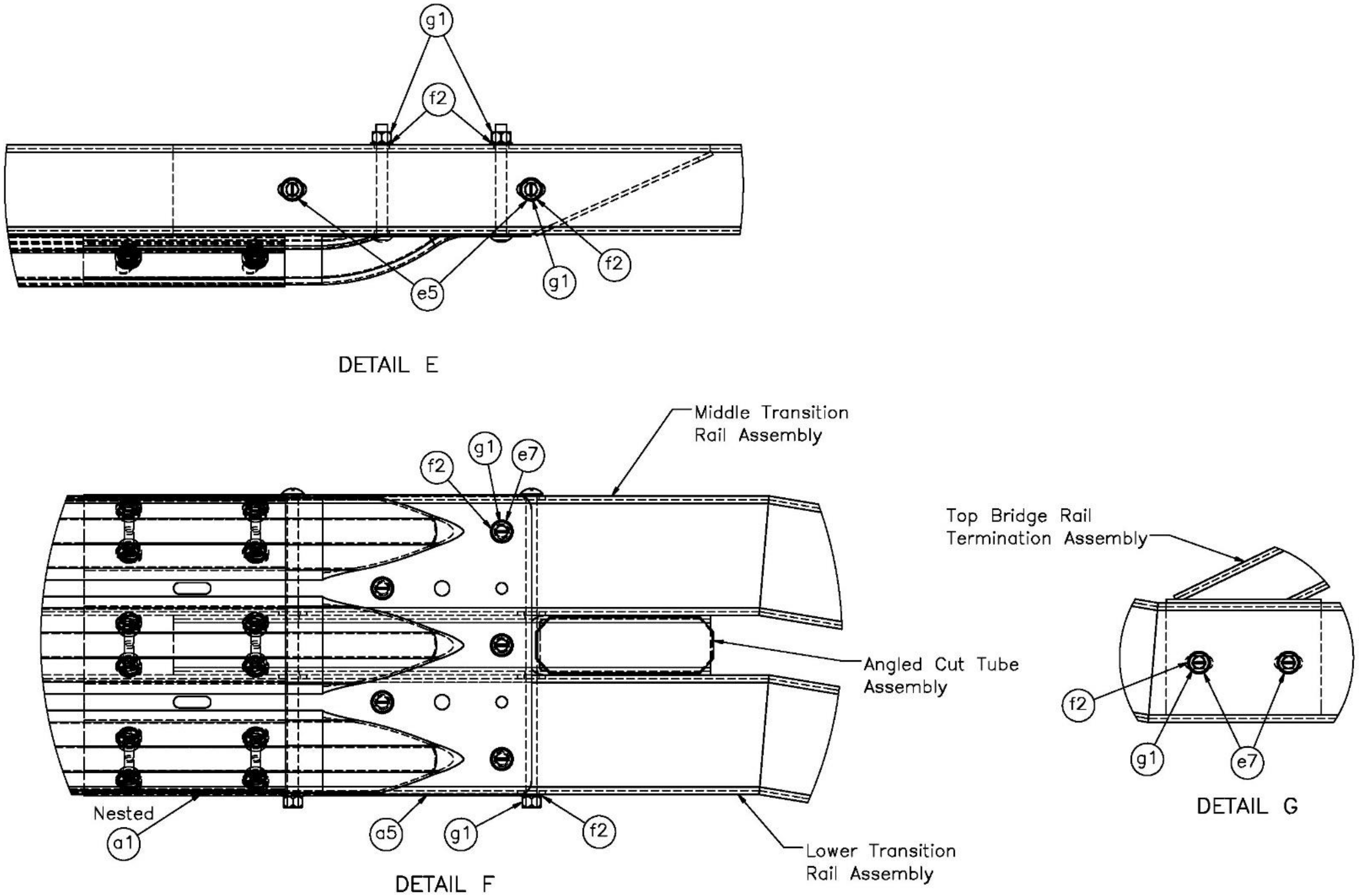


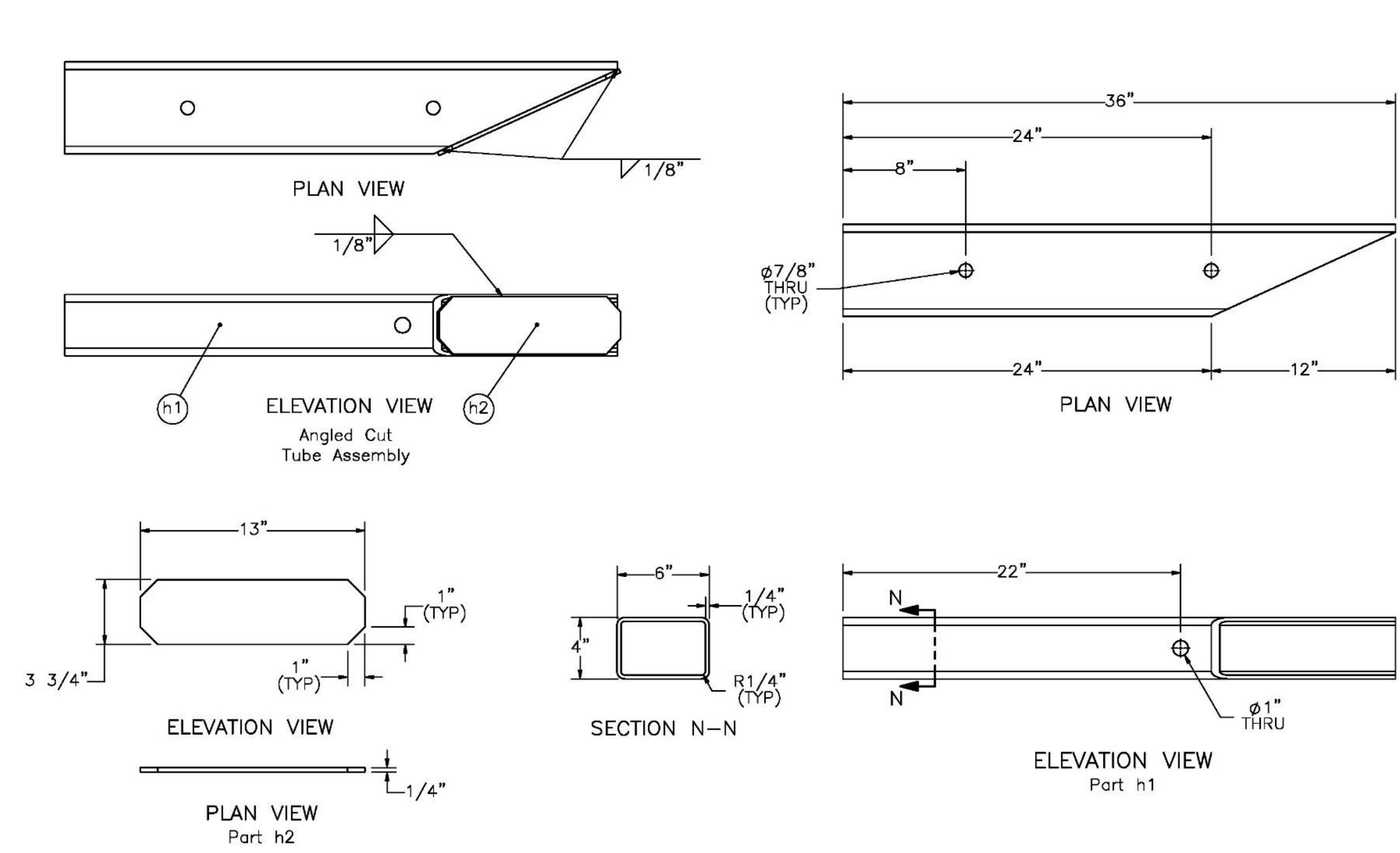
Figure A-13. Steel Railing, Type IL-OH, Connection to Guardrail Transition



Notes: (1) Part a5 should be sandwiched between nested parts a1.
(2) f1 washers may be used with e3 bolts on backside of the splice between nested a1 rails and a5 terminal connector.

MwRSF Midwest Roadside Safety Facility	Steel Railing, Type IL–OH and Associated Guardrail Transition Transition Details		SHEET: 14 of 30 DATE: 12/22/2020 DRAWN BY: LJP
	DWG. NAME. STBRT Components_R3	SCALE: 1:10 UNITS: in.	REV. BY: SKR/JDR

Figure A-14. Steel Railing, Type IL-OH, Connection Details



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Angled Cut Tube Assembly	–	ASTM 123
h1	1	HSS 6"x4"x1/4", 36" Long Angled Cut Tube	ASTM A500 Gr. C	See Assembly
h2	1	13"x3 3/4"x1/4" Plate	ASTM A572 Gr. 50	See Assembly

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Steel Railing, Type IL–OH and Associated Guardrail Transition

Angled Cut Tube Assembly

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DWG. NAME. STBRT Components_R3

SCALE: 1:9 UNITS: in.

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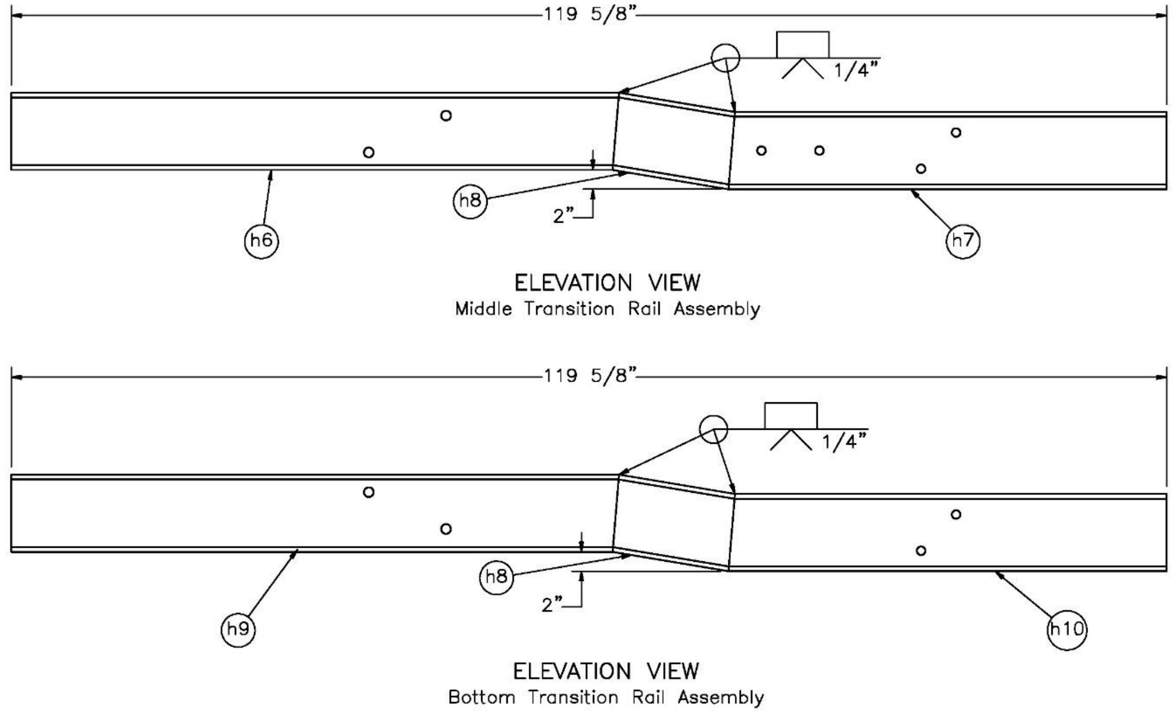
Figure A-15. Steel Railing, Type IL-OH, Angled Cut Tube Assembly



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Middle Transition Rail Assembly	ASTM A500 Gr. C	ASTM A123
h6	1	HSS 8"x6"x1/4", 62 15/16" Long Middle Transition Rail	ASTM A500 Gr. C	See Assembly
h7	1	HSS 8"x6"x1/4", 45 3/8" Long Middle Transition Rail	ASTM A500 Gr. C	See Assembly
h8	1	HSS 8"x6"x1/4", 12 7/8" Long Transition Rail	ASTM A500 Gr. C	See Assembly

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Bottom Transition Rail Assembly	ASTM A500 Gr. C	ASTM A123
h8	1	HSS 8"x6"x1/4", 12 7/8" Long Transition Rail	ASTM A500 Gr. C	See Assembly
h9	1	HSS 8"x6"x1/4", 62 15/16" Long Bottom Transition Rail	ASTM A500 Gr. C	See Assembly
h10	1	HSS 8"x6"x1/4", 45 3/8" Long Bottom Transition Rail	ASTM A500 Gr. C	See Assembly

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Steel Railing, Type IL–OH and Associated Guardrail Transition

Middle/Bottom Transition Rail Assemblies

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16 of 30	12/22/2020	LJP	STBRT Components_R3	1:16	in.	SKR/JDR

Figure A-16. Steel Railing, Type IL-OH, Middle and Bottom Transition Tube Rails

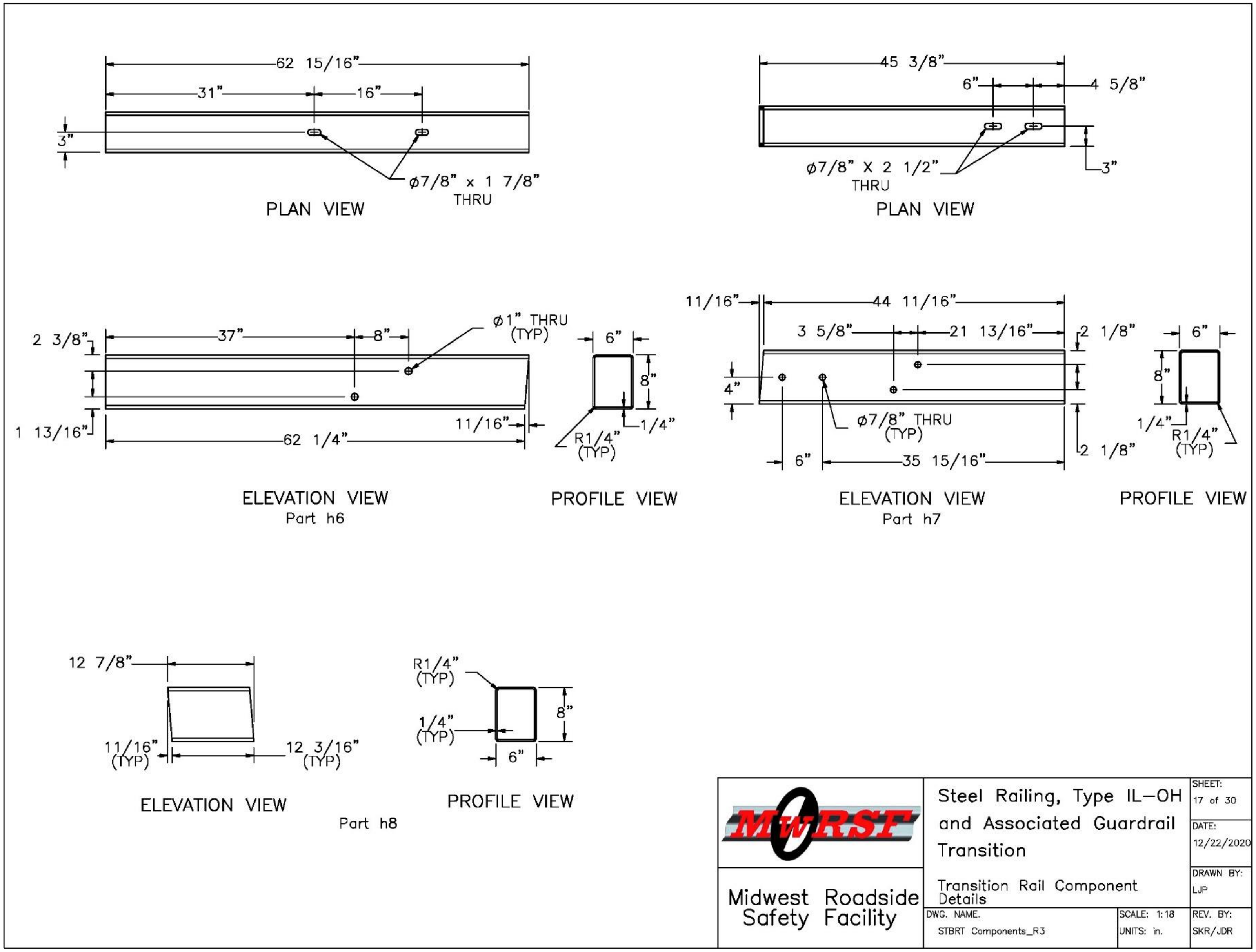


Figure A-17. Steel Railing, Type IL-OH, Transition Tube Rail Components

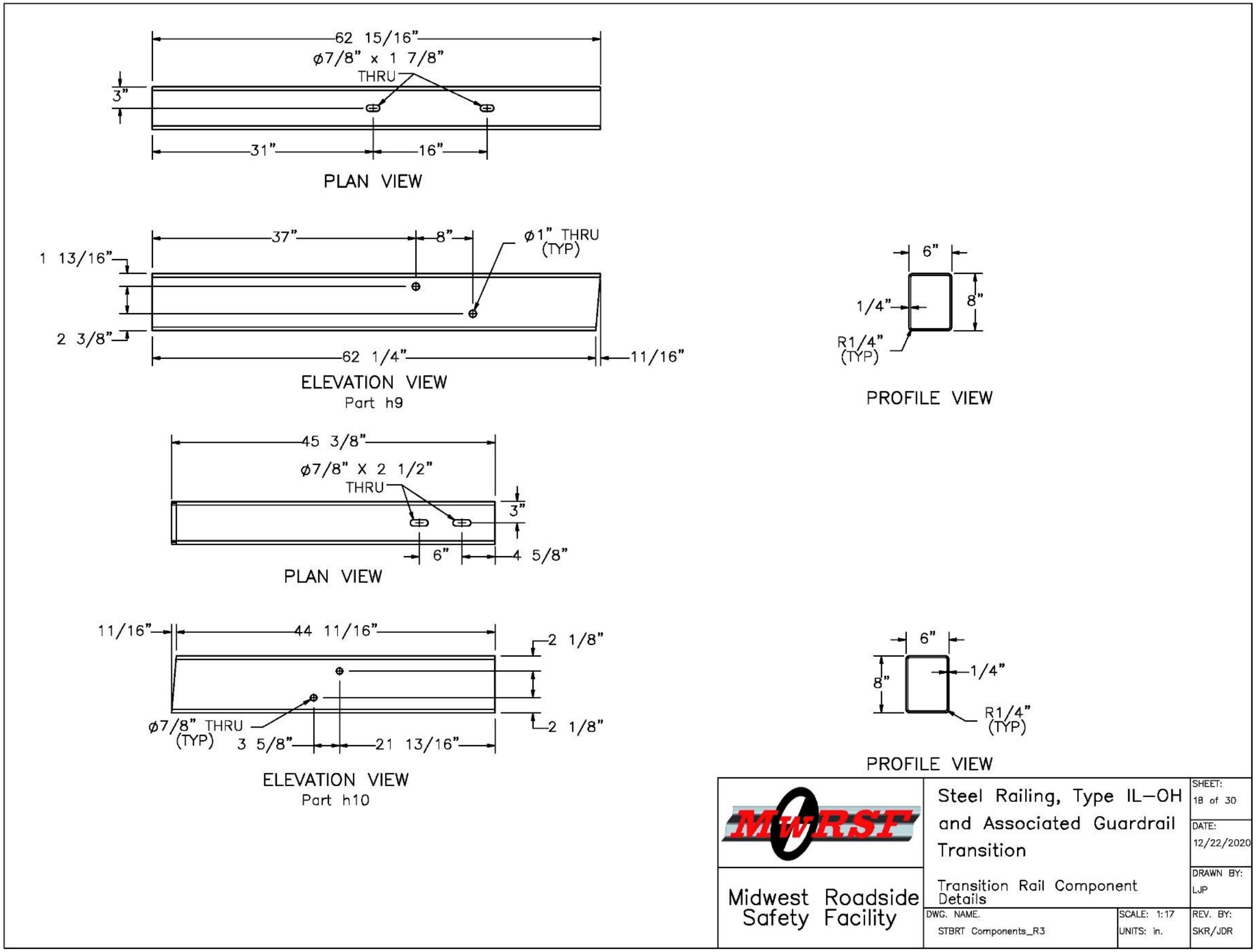
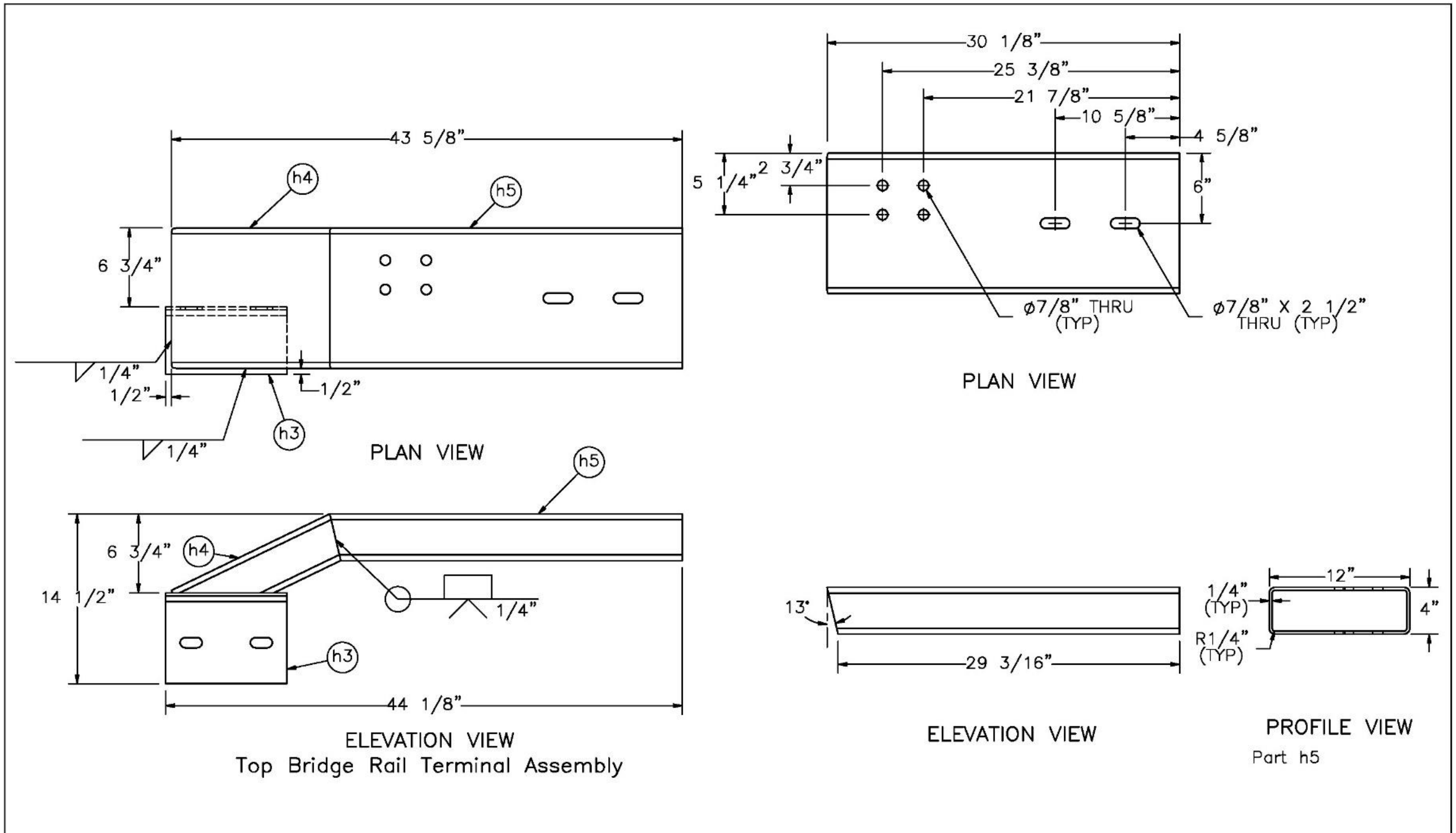


Figure A-18. Steel Railing, Type IL-OH, Transition Tube Rail Components



Note: Top bridge rail terminal assembly must be mirrored to use on other end of bridge.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Top Bridge Rail Termination Assembly	–	ASTM A123
h3	1	13"x10 3/8"x1/4" Bent Plate	ASTM A572 Gr. 50	See Assembly
h4	1	12"x4"x1/4", 15" Long Sloped Transition Rail	ASTM A500 Gr. C	See Assembly
h5	1	12"x4"x1/4", 30 1/8" Long Transition Rail	ASTM A500 Gr. C	See Assembly

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Steel Railing, Type IL–OH and Associated Guardrail Transition

Top Bridge Rail Termination Assembly

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DWG. NAME. STBRT Components_R3

SCALE: 1:12

UNITS: in.

REV. BY: SKR/JDR

Figure A-19. Steel Railing, Type IL-OH, Top Transition Tube Rail Assembly

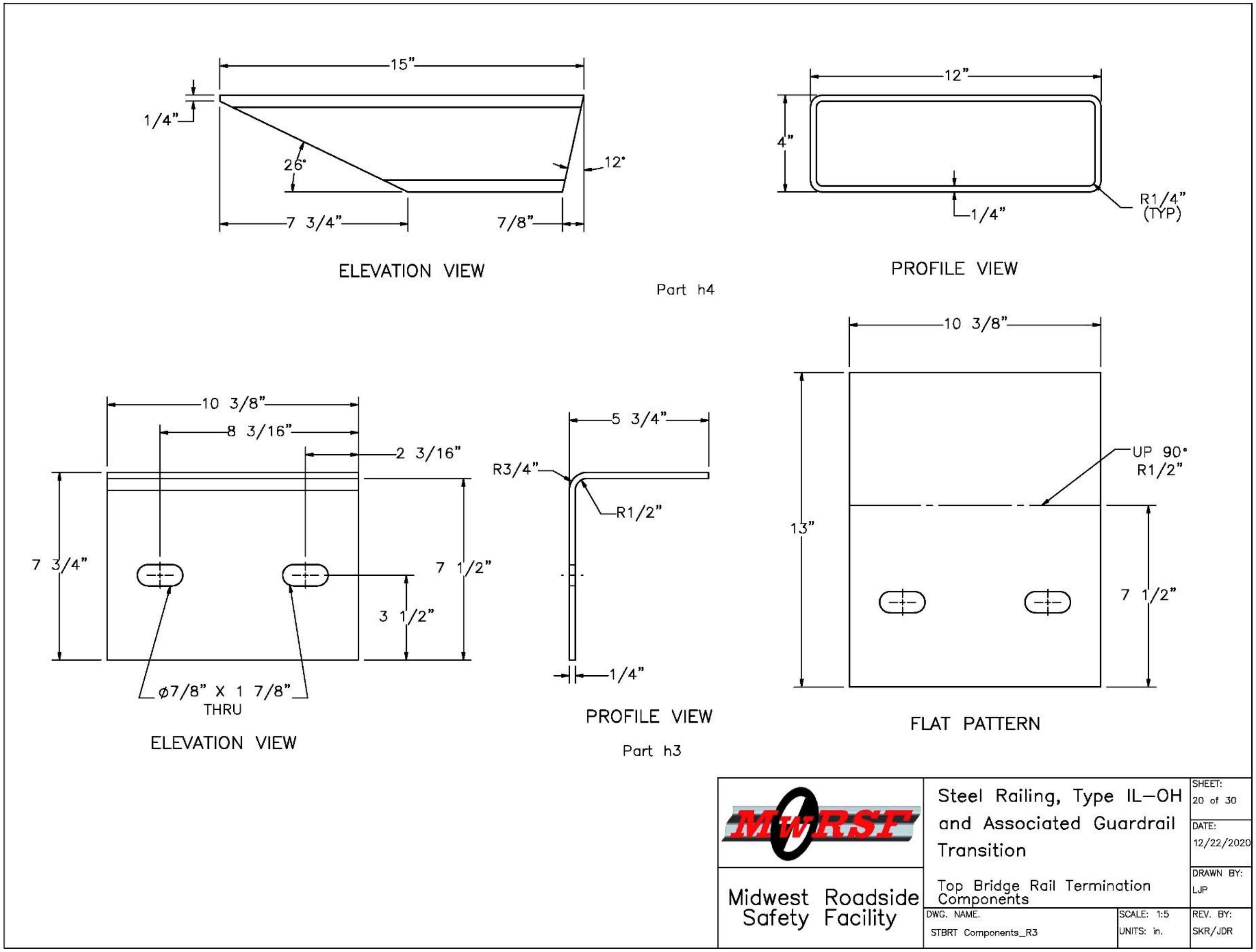


Figure A-20. Steel Railing, Type IL-OH, Top Transition Tube Rail Components

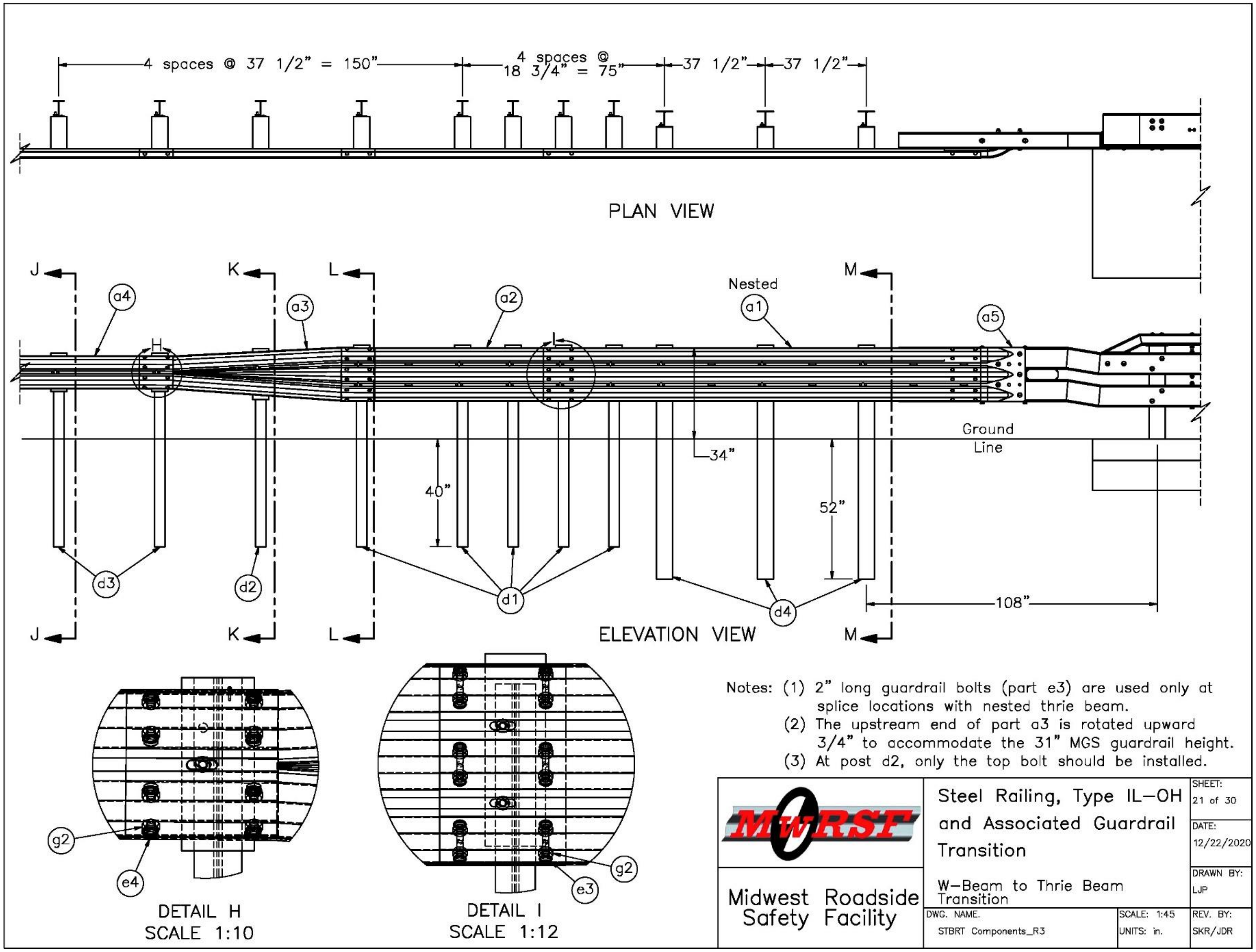
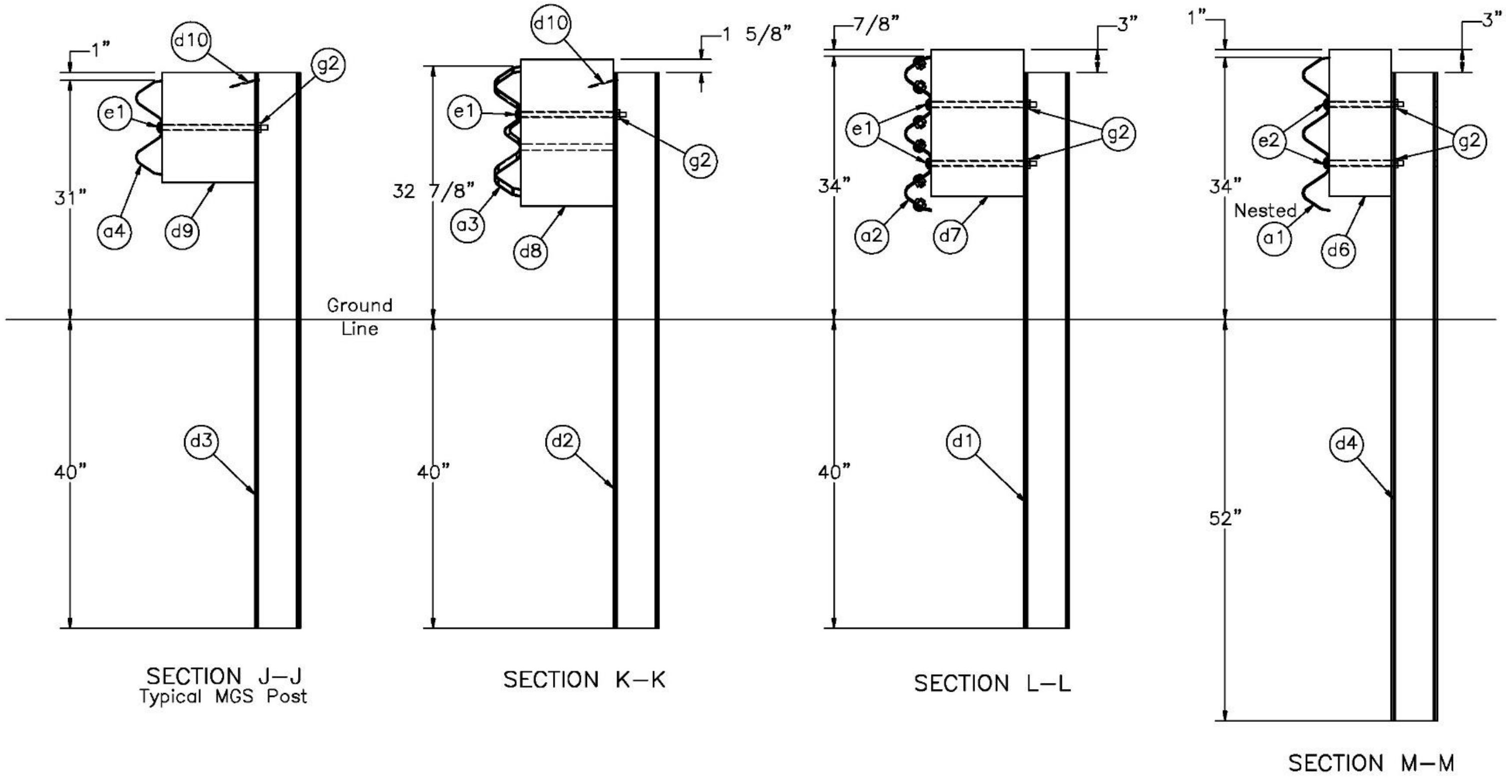


Figure A-21. Steel Railing, Type IL-OH, Approach Guardrail Transition Layout



Notes: (1) AGT system may have routed blockouts or may use nails (part d10) to prevent blockout rotation.
(2) Only the top bolt is installed at post d2.

MwRSF	Steel Railing, Type IL–OH and Associated Guardrail Transition	SHEET: 22 of 30
		DATE: 12/22/2020
Midwest Roadside Safety Facility	Post Sections	DRAWN BY: LJP
	DWG. NAME. STBRT Components_R3 — SCALE: 1:20 UNITS: in.	REV. BY: SKR/JDR

Figure A-22. Steel Railing, Type IL-OH, AGT Cross-Sections

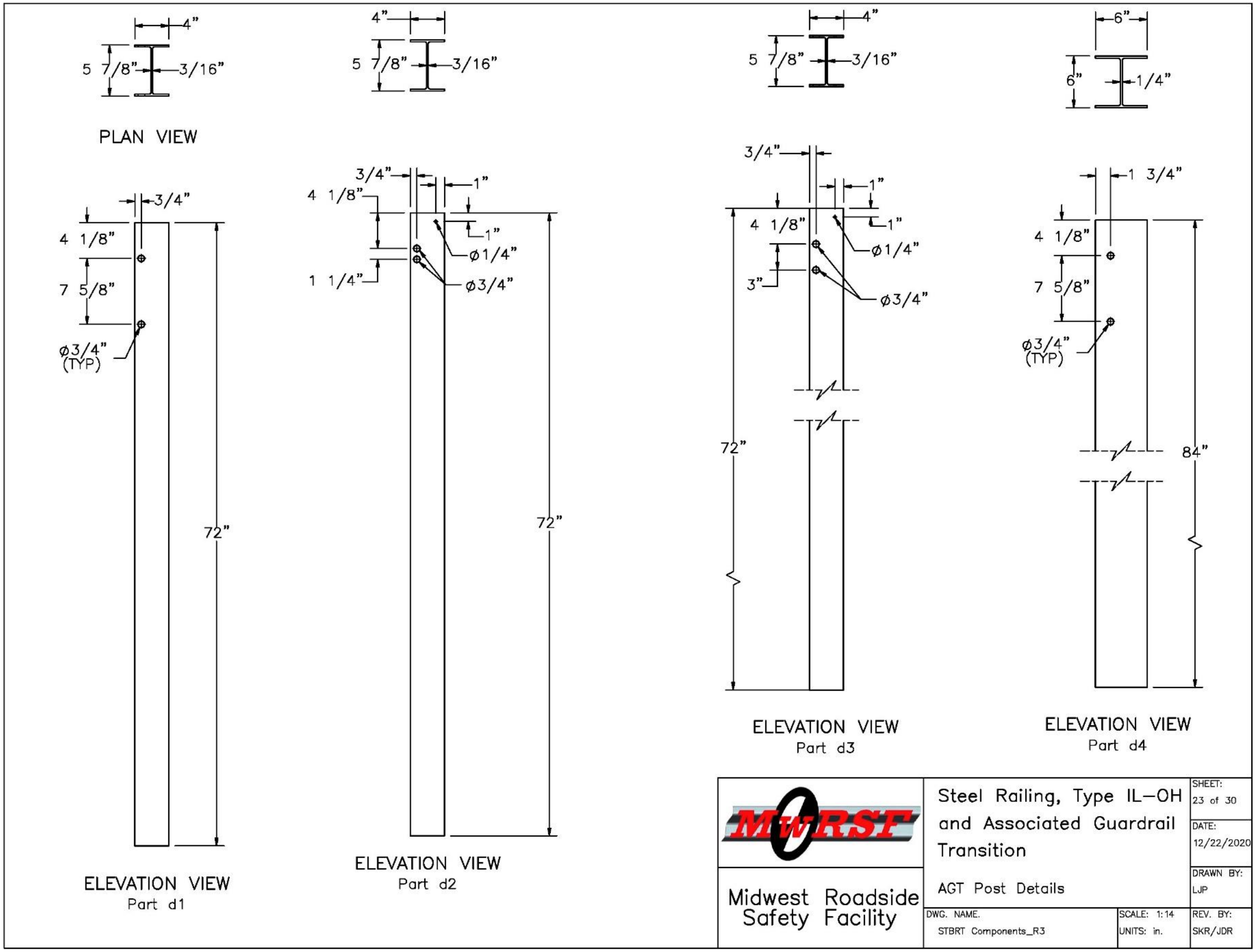


Figure A-23. Steel Railing, Type IL-OH, AGT Post Details

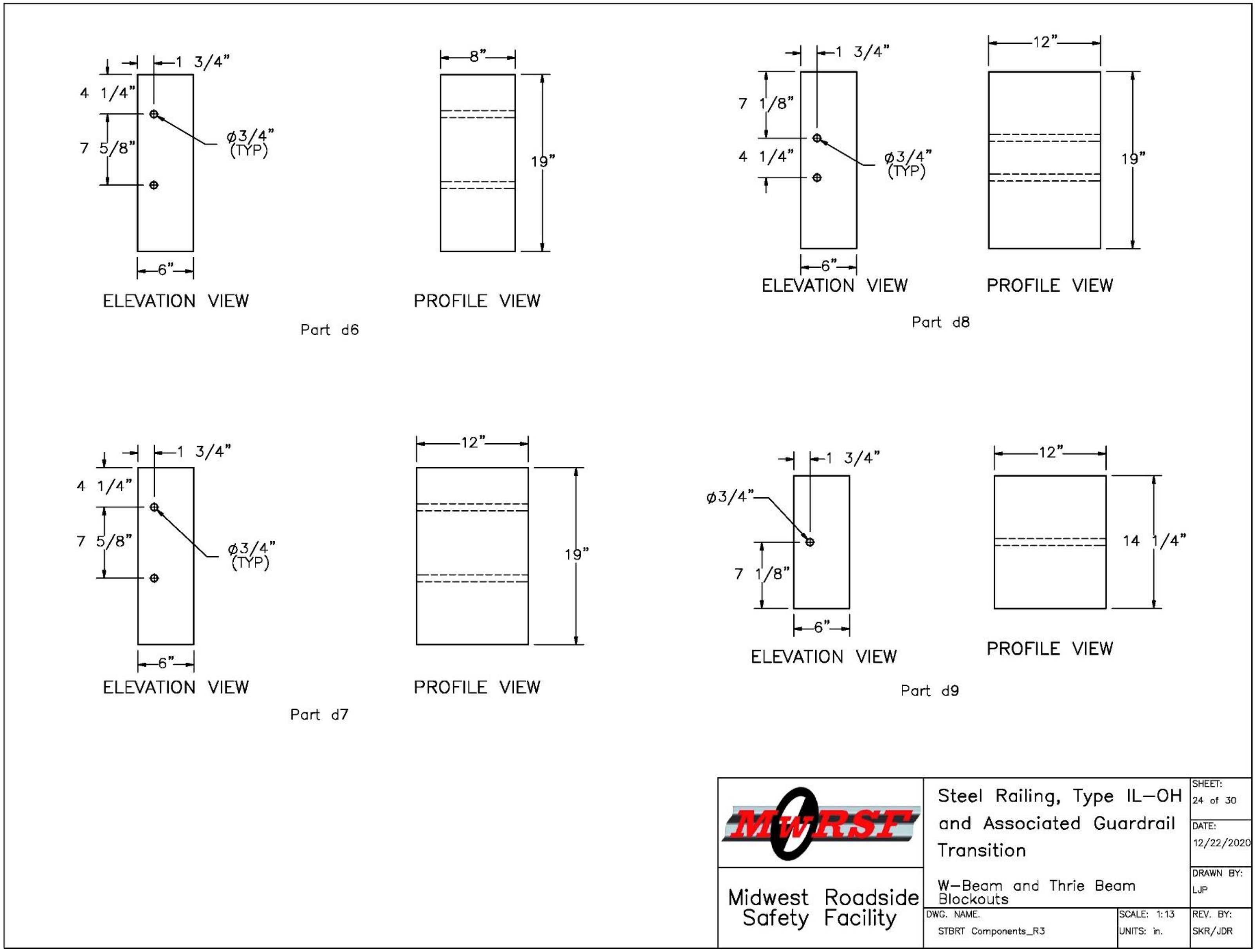


Figure A-24. Steel Railing, Type IL-OH, AGT Blockouts

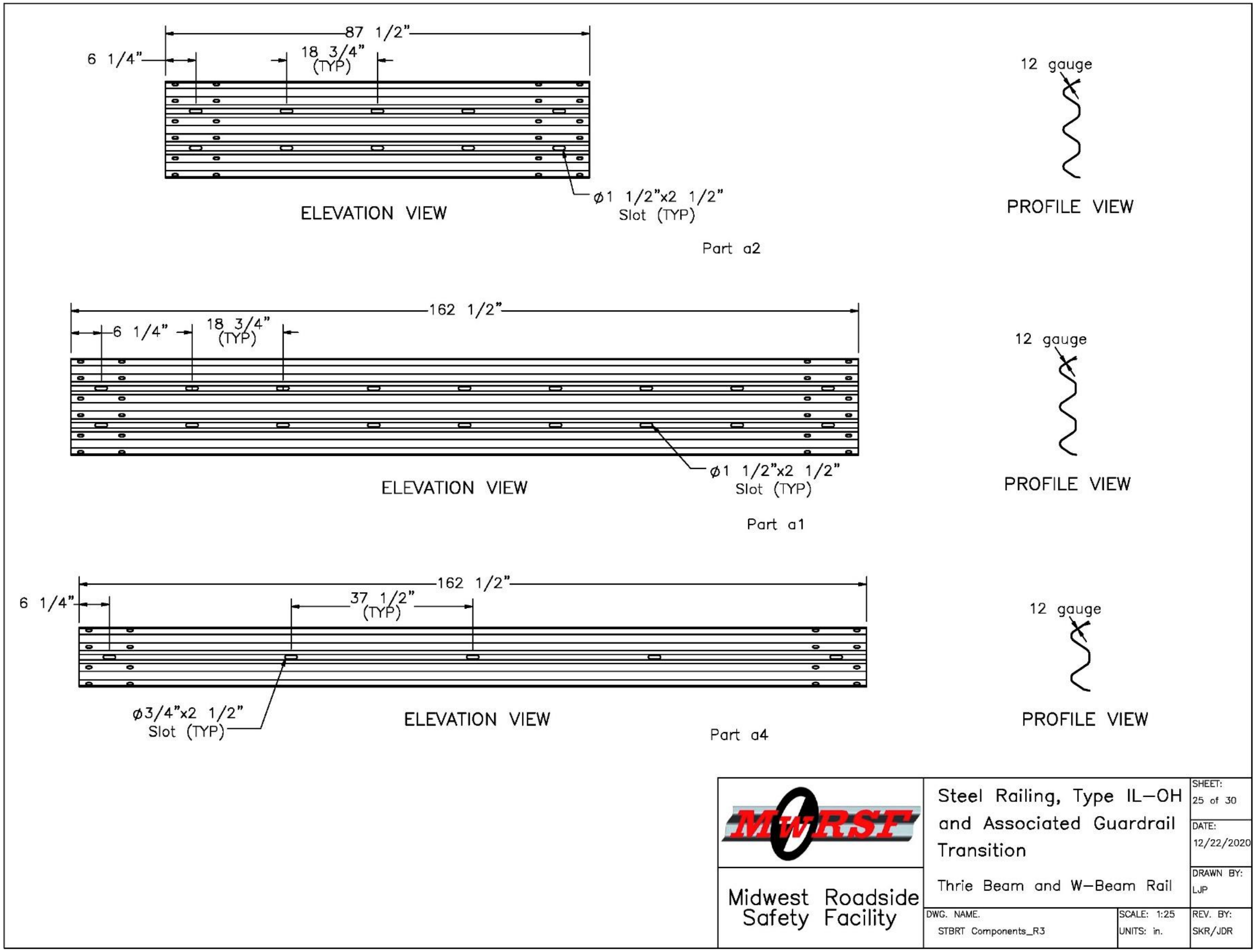


Figure A-25. Steel Railing, Type IL-OH, AGT Rail Segments

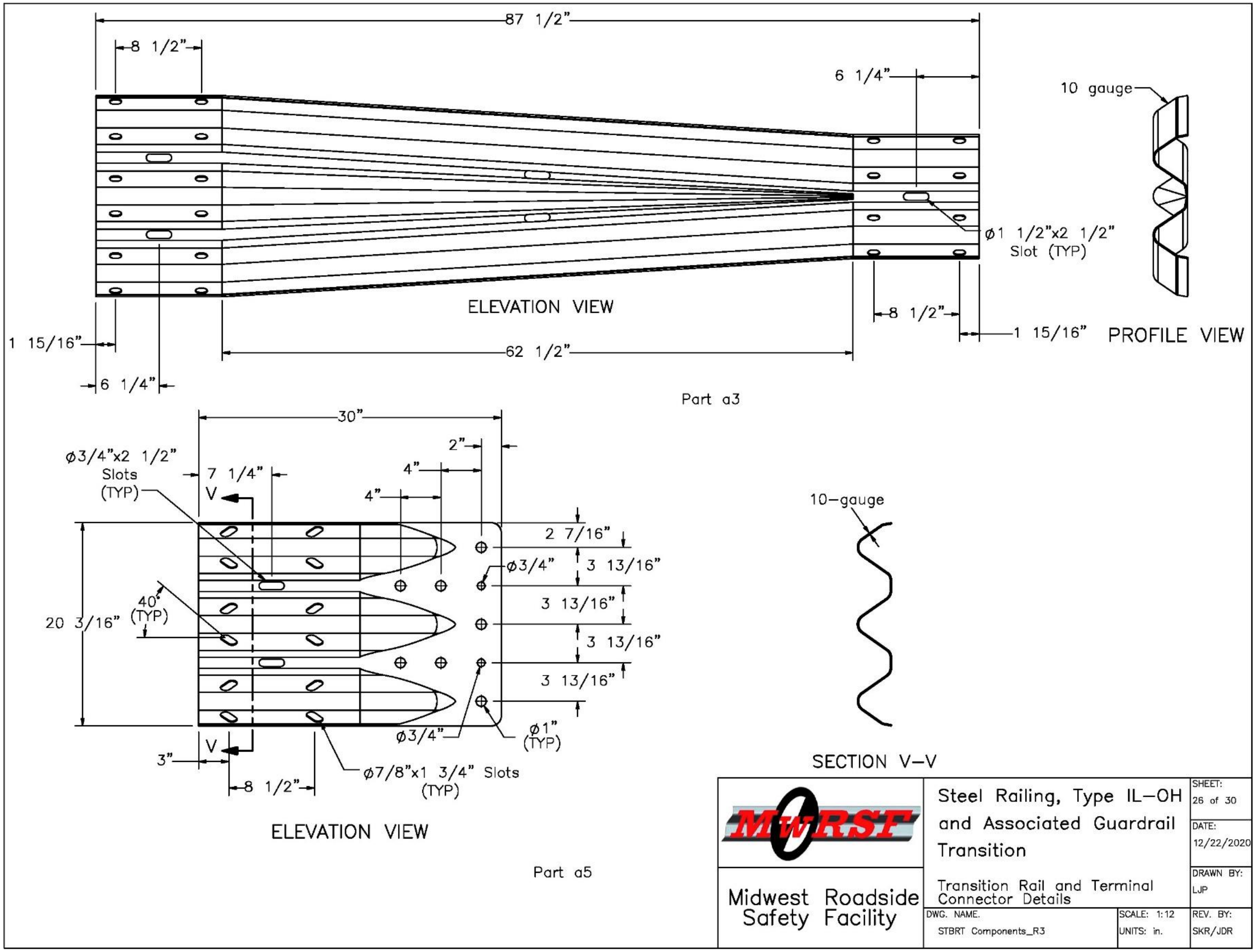


Figure A-26. Steel Railing, Type IL-OH, Transition Rail and Terminal Connector

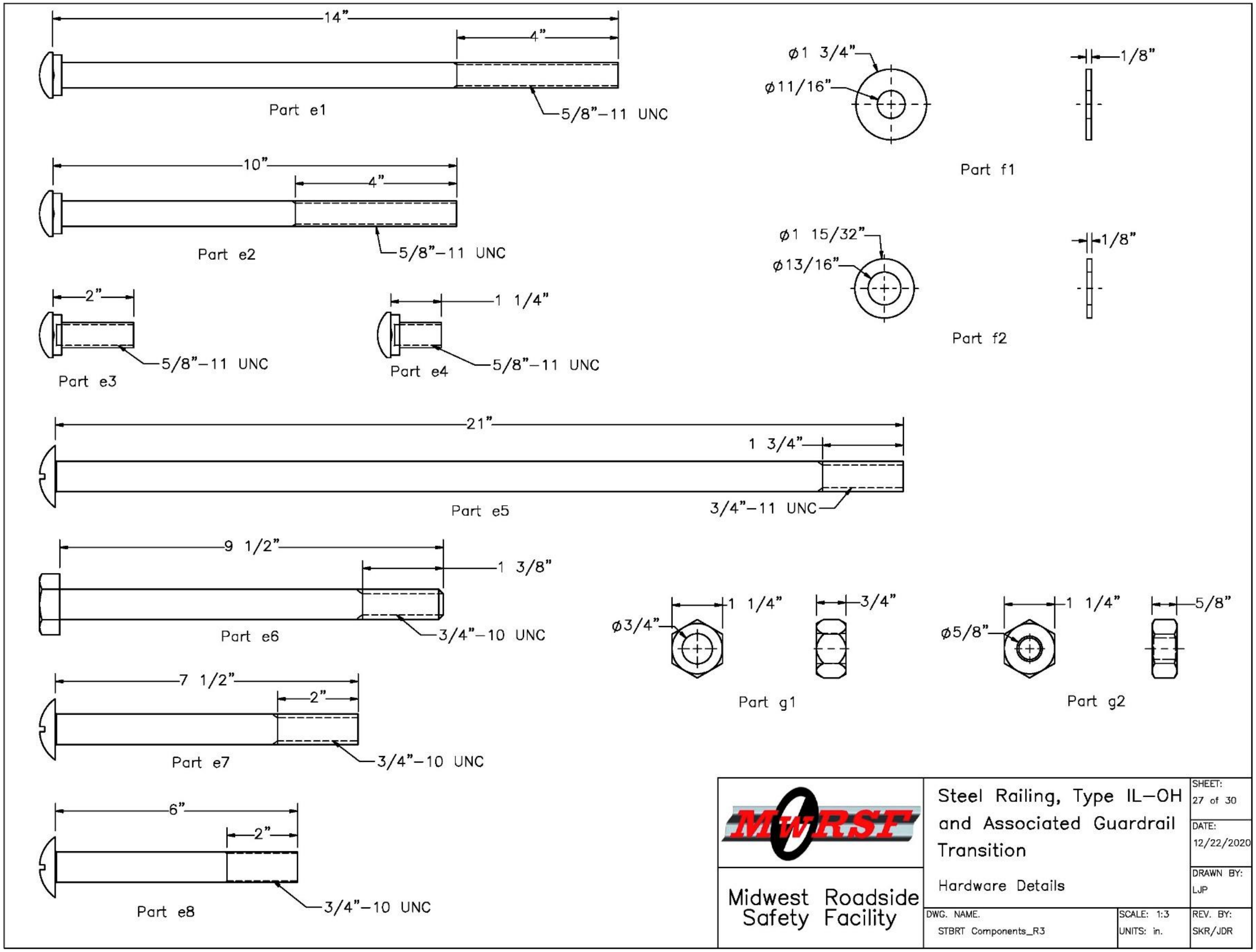


Figure A-27. Steel Railing, Type IL-OH, Hardware Details

Item No.	Description	Material Specification	Treatment Specification	Hardware Guide
a1	12'–6" 12–gauge Thrie Beam Section	AASHTO M180	A123 or ASTM A653	RTM08a
a2	6'–3" 12–gauge Thrie Beam Section	AASHTO M180	A123 or ASTM A653	RTM19a
a3	10–gauge Symmetrical W–beam to Thrie Beam Transition	AASHTO M180	A123 or ASTM A653	RWT01b
a4	12'–6" 12–gauge W–Beam Section	AASHTO M180	A123 or ASTM A653	RWM04a
a5	10–gauge Thrie Beam Terminal Connector	AASHTO M180 (Min yield strength = 50 ksi Min. ultimate strength = 70 ksi)	A123 or ASTM A653	RTE01b
b1	30"x10 5/8"x5/16" Plate	ASTM A572 Gr. 50	See Assembly	–
b2	30"x2 5/8"x3/8" Plate	ASTM A572 Gr. 50	See Assembly	–
b3	8"x8"x3/8" Plate	ASTM A572 Gr. 50	See Assembly	–
b4	17 3/4"x13"x1" Post Plate	ASTM A572 Gr. 50	See Assembly	–
b5	6 1/8"x5 11/16"x1/4" Gusset Plate	ASTM A572 Gr. 50	See Assembly	–
b6	HSS 5"x4"x1/2", 20" Long	ASTM A500 Gr. C	ASTM A123	–
b7	30"x6 5/8"x3/8" Plate	ASTM A572 Gr. 50	See Assembly	–
b8	30"x4 5/8"x5/16" Plate	ASTM A572 Gr. 50	See Assembly	–
b9	HSS 8"x6"x1/4", 191 1/4" Long	ASTM A500 Gr. C	ASTM A123	–
b10	HSS 12"x4"x1/4", 191 1/4" Long	ASTM A500 Gr. C	ASTM A123	–
c1	20"x15"x3/16" Steel Plate	ASTM A572 Gr. 50	See Assembly	–
c2	1"–8 UNC, 32 3/4" Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329	FRR24b
c3	3"x3"x1/4" Plate	ASTM A36	ASTM A123	–
c4	36"x3 1/2"x1/4" Plate	ASTM A36	ASTM A123	–
d1	W6x9 or W6x8.5, 72" Long Steel Post	ASTM A992	ASTM A123	–
d2	W6x9 or W6x8.5, 72" Long Steel Post	ASTM A992	ASTM A123	–
d3	W6x9 or W6x8.5, 72" Long Steel Post	ASTM A992	ASTM A123	–
d4	W6x15, 84" Long Steel Post	ASTM A992	ASTM A123	PWE12
d5	W6x15 Post, Length Varies	ASTM A992	See Assembly	–
d6	6"x8"x19" Timber Blockout	SYP Grade No.1 or better	–	PDB17
d7	6"x12"x19" Timber Blockout	SYP Grade No.1 or better	–	–
d8	6"x12"x19" Timber Blockout	SYP Grade No.1 or better	–	PDB18
d9	6"x12"x14 1/4" Timber Blockout	SYP Grade No.1 or better	–	PDB10a
d10	Bent 16D Double Head Nail	–	–	–

Midwest Roadside Safety Facility

Steel Railing, Type IL–OH and Associated Guardrail Transition

Bill of Materials

SHEET: 28 of 30

DATE: 12/22/2020

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DWG. NAME. STBRT Components_R3

SCALE: None UNITS: in.

REV. BY: SKR/JDR

Figure A-28. Steel Railing, Type IL-OH, Bill of Materials

Item No.	Description	Material Specification	Treatment Specification	Hardware Guide
e1	5/8" Dia. UNC, 14" Long Guardrail Bolt	ASTM A307 Gr. A	ASTM A153 or B695 Class 55 or F1941 or F2329	FBB06
e2	5/8" Dia. UNC, 10" Long Guardrail Bolt	ASTM A307 Gr. A	ASTM A153 or B695 Class 55 or F1941 or F2329	FBB03
e3	5/8" Dia. UNC, 2" Long Guardrail Bolt	ASTM A307 Gr. A	ASTM A153 or B695 Class 55 or F1941 or F2329	FBB02
e4	5/8" Dia. UNC, 1 1/4" Long Guardrail Bolt	ASTM A307 Gr. A	ASTM A153 or B695 Class 55 or F1941 or F2329	FBB01
e5	3/4"-11 UNC, 21" Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F1941 or F2329	FBB08
e6	3/4"-10 UNC, 9 1/2" Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F1941 or F2329 or F2833 Gr. 1	FBX20b
e7	3/4"-10 UNC, 7 1/2" Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F1941 or F2329	FBB08
e8	3/4"-10 UNC, 6" Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F1941 or F2329	FBB08
e9	1"-8 UNC, 3 1/2" Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F1941 or F2329 or F2833 Gr. 1	FBX24b
e10	1"-8 UNC, 2 1/4" Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F1941 or F2329 or F2833 Gr. 1	FBX24b
e11	1"-8 UNC, 1 1/2" Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX24b
f1	5/8" Dia. SAE Plain Round Washer	ASTM F844	ASTM A123 or A153 or F2329	FWC16a
f2	3/4" Dia. SAE Hardened Flat Washer	ASTM F436	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329	FWC20b
f3	2 1/4"x2 1/4"x1/4" Square Washer	ASTM A36	ASTM A123	–
g1	3/4 -10 UNC Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX20b
g2	5/8" Dia. Guardrail Nut	ASTM A563A	ASTM A153 or B695 Class 55 or F2329	–
g3	1"-8 UNC Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX24b
g4	1"-8 UNC Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–

Midwest Roadside Safety Facility

Steel Railing, Type IL-OH and Associated Guardrail Transition

Bill of Materials

SHEET: 29 of 30
DATE: 12/22/2020
DRAWN BY: LJP
DWG. NAME. STBRT Components_R3
SCALE: None
UNITS: in.
REV. BY: SKR/JDR

Figure A-29. Steel Railing, Type IL-OH, Bill of Materials

Item No.	Description	Material Specification	Treatment Specification	Hardware Guide
h1	HSS 6"x4"x1/4", 36" Long Angled Cut Tube	ASTM A500 Gr. C	See Assembly	–
h2	13"x3 3/4"x1/4" Plate	ASTM A572 Gr. 50	See Assembly	–
h3	13"x10 3/8"x1/4" Bent Plate	ASTM A572 Gr. 50	See Assembly	–
h4	12"x4"x1/4", 15" Long Sloped Transition Rail	ASTM A500 Gr. C	See Assembly	–
h5	12"x4"x1/4", 30 1/8" Long Transition Rail	ASTM A500 Gr. C	See Assembly	–
h6	HSS 8"x6"x1/4", 62 15/16" Long Middle Transition Rail	ASTM A500 Gr. C	See Assembly	–
h7	HSS 8"x6"x1/4", 45 3/8" Long Middle Transition Rail	ASTM A500 Gr. C	See Assembly	–
h8	HSS 8"x6"x1/4", 12 7/8" Long Transition Rail	ASTM A500 Gr. C	See Assembly	–
h9	HSS 8"x6"x1/4", 62 15/16" Long Bottom Transition Rail	ASTM A500 Gr. C	See Assembly	–
h10	HSS 8"x6"x1/4", 45 3/8" Long Bottom Transition Rail	ASTM A500 Gr. C	See Assembly	–

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Steel Railing, Type IL–OH and Associated Guardrail Transition

Bill of Materials

SHEET: 30 of 30
DATE: 12/22/2020
DRAWN BY: LJP
DWG. NAME. STBRT Components_R3
SCALE: None
UNITS: in.
REV. BY: SKR/JDR

Figure A-30. Steel Railing, Type IL-OH, Bill of Materials

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