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DEVELOPMENT OF A POST-TO-DECK CONNECTION FOR A TL-4 STEEL-TUBE BRIDGE RAIL

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16. Abstract

A new post-to-deck connection was designed for mounting an upcoming *Manual for Assessing Safety Hardware, Second Edition* (MASH 2016) Test Level 4 (TL-4) steel-tube bridge rail to the side of the bridge deck to limit the impact load transferred to the deck and to minimize the propensity for deck damage during impacts. With preference for the bridge rail system to be compatible for use on all deck configurations, the Illinois and Ohio Departments of Transportation desired the new system to attach to bridge decks comprised of concrete slabs or precast prestressed, concrete box-beam girders.

A new post-to-deck connection for a new MASH 2016 TL-4 steel-tube bridge rail was developed and tested. The connection featured a W6x15 steel post to be side-mounted to bridge decks. A post-to-deck connection concept was developed and optimized through dynamic component testing. Lateral impacts were conducted on the design concepts, side-mounted to a concrete box-beam girder. For the dynamic component testing, post-to-deck connection configurations varied on the type of welded post assembly connecting to longitudinal tubes, which in turn mounted to various anchors cast within the box-beam girder. The resulting damage from each test varied from plastic deformation of the post between two plate attachments on the post assembly or post bending above one plate attachment due to the development of a plastic hinge near the surface of the deck. The post-to-deck connection featuring a singular plate attachment was recommended for use based on the post bending near the surface of the deck, as intended.

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UNCERTAINTY OF MEASUREMENT STATEMENT

The Midwest Roadside Safety Facility (MwRSF) has determined the uncertainty of measurements for several parameters involved in standard full-scale crash testing and non-standard testing of roadside safety features. Information regarding the uncertainty of measurements for critical parameters is available upon request by the sponsor and the Federal Highway Administration. Test nos. ILOH4-1 through ILOH4-7 were non-certified component tests conducted for research and development purposes only and are outside the scope of the MwRSF's A2LA Accreditation.

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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 few decades, the Ohio Department of Transportation (ODOT) and the Illinois Department of Transportation (IDOT) have regularly installed steel-tube bridge railings as a protective barrier to treat the edges of their bridges. These bridge railings consist of multiple steel-tube rails mounted to the face of I-section steel posts, as shown in Figures 1 and 2 for the states of Ohio and Illinois, respectively. The systems 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.

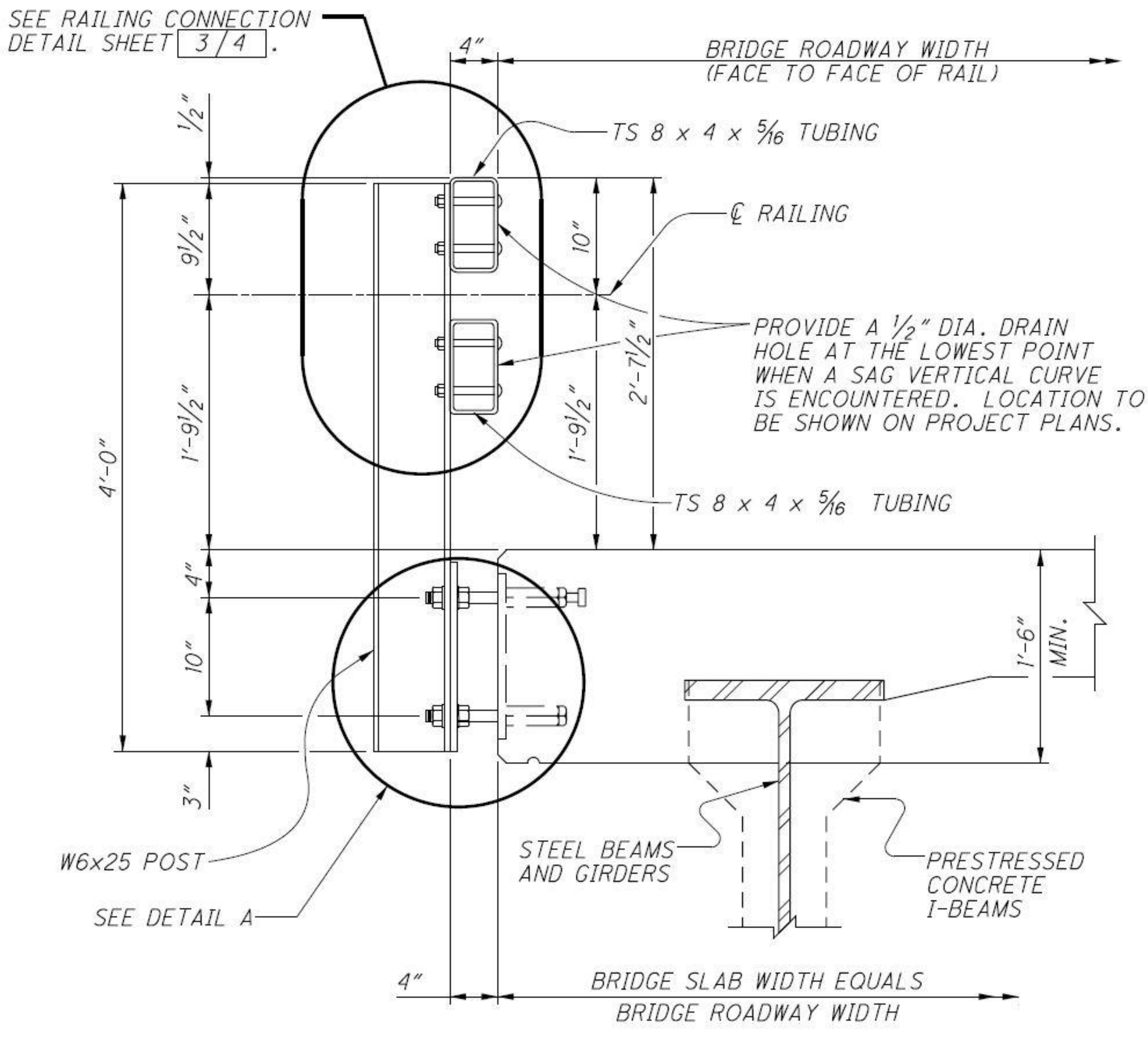


Figure 1. Existing ODOT Side-Mounted Steel Tube Bridge Railing [1]

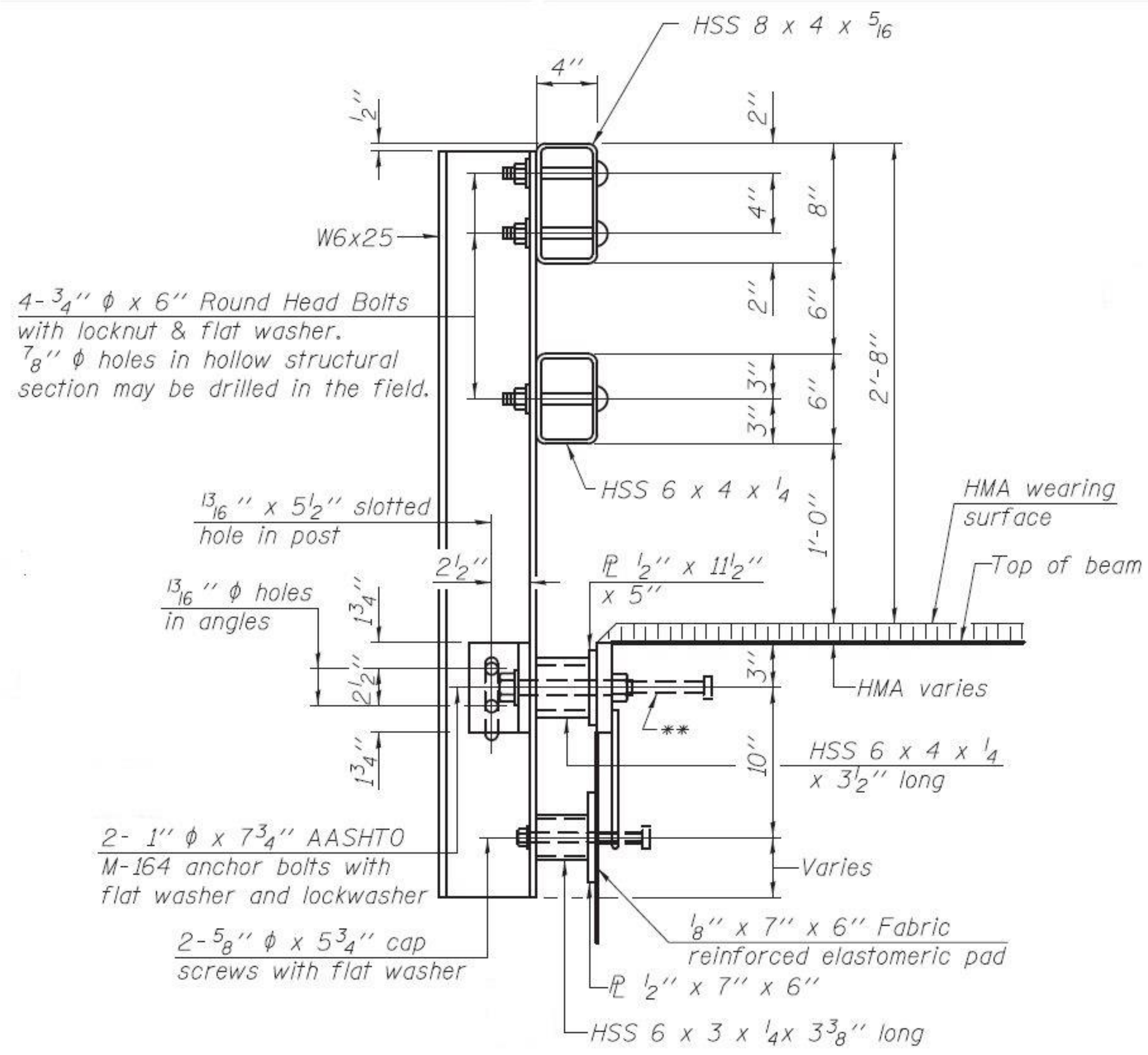


Figure 2. Existing IDOT Side-Mounted Steel Tube Bridge Railing [2]

The bridge railings shown in Figures 1 and 2 were originally developed and crash tested to satisfy the Test Level 4 (TL-4) safety criteria found in National Cooperative Highway Research Program (NCHRP) Report 350 [3]. NCHRP Report 350 TL-4 featured an 17,600-lb (8,000-kg) single-unit truck impacting the system at a speed of 50 mph (80 km/h) and at an angle of 15 degrees, and both an 1,800-lb (820-kg) small car and a 4,400-lb (2,000-kg) pickup truck impacting a longitudinal barrier at a speed of 62 mph (100 km/h) and but at an impact angle of 20 degrees for the small car and at an impact angle of 25 degrees for the pickup truck.

In 2009, the American Association of State Highway and Transportation Officials (AASHTO) implemented a new standard for the evaluation of roadside hardware, the *Manual for Assessing Safety Hardware* (MASH) [4]. Similar to NCHRP Report 350, MASH presented uniform guidelines for crash testing permanent and temporary highway safety features and recommends evaluation criteria to assess test results. The second edition of MASH was published in 2016 (MASH 2016) [5]. However, no changes were made to the impact conditions for bridge rails between the first and second editions. No side-mounted, steel tube bridge railings have been evaluated to the MASH 2016 TL-4 criteria as of the commencement of this project.

MASH 2016 TL-4 evaluation criteria for longitudinal barriers consists of three full-scale crash tests (test nos. 4-10, 4-11, and 4-12). Crash test nos. 4-10 and 4-11 involve a 2,420-lb (1,100-kg) small car and 5,000-lb (2,270-kg) pickup truck impacting a barrier system at a speed of 62 mph (100 km/h) and angle of 25 degrees, respectively. Test designation no. 4-12 involves a 22,000-lb (10,000-kg) single-unit truck (SUT) impacting the barrier system at a speed of 56 mph (90 km/h) and angle of 15 degrees.

With the implementation of MASH, significant changes were made to the TL-4 impact conditions, including the increase of the small car impact angle from 20 degrees to 25 degrees and an increase in speed for the single-unit truck from 50 mph (80 km/h) to 56 mph (90 km/h). Moreover, the vehicle mass of all test vehicles increased: the small car mass increased from 1,800 lb (820 kg) to 2,420 lb (1,100 kg); the pickup truck mass increased from 4,400 lb (2,000 kg) to 5,000 lb (2,268 kg); and SUT mass increased from 17,600 lb (8,000 kg) to 22,000 lb (10,000 kg). These changes have resulted in increased impact loads imparted to the barrier, so the required barrier capacity also increased. Additionally, the minimum barrier height required to prevent the TL-4 single-unit truck from overriding the barrier has increased from 32 in. (813 mm) to 36 in. (914 mm) [6]. Accordingly, significant changes may be required to update TL-4 barriers from NCHRP Report 350 to MASH 2016 safety performance standards. Therefore, a new side-mounted, steel-tube bridge railing was desired to satisfy MASH 2016 TL-4 safety criteria.

Further, the Federal Highway Administration (FHWA) and AASHTO established a MASH implementation policy which includes sunset dates for prior roadside hardware [7]. For contracts of bridge rails, transitions, and all other longitudinal barriers installed on the National Highway System (NHS) after December 31, 2019, only safety hardware evaluated using the 2016 edition of MASH will be allowed for new permanent installations and full replacements. The implementation policy also states all modifications to NCHRP Report 350-tested devices require testing under MASH 2016 in order to receive a federal-aid eligibility letter from the FHWA. Therefore, the development of a MASH 2016 TL-4, side-mounted, steel-tube bridge railing and an associated guardrail transition is required prior to 2020 to allow new installations of such railings in Ohio and Illinois.

Through initial discussions between ODOT, IDOT, and the Midwest Roadside Safety Facility (MwRSF), a preliminary steel-tube bridge railing design was developed, as shown in Figure 3. The preliminary design had a top height of 39 in. (991 mm) to account for up to a 3-in. (76-mm) thick future roadway overlay on the bridge while maintaining a minimum MASH 2016 TL-4 barrier height of 36 in. (914 mm). The railing consisted of three longitudinal steel tubes attached to side-mounted, W6x15 steel posts. The front face of the bridge rail was laterally offset 4 in. (102 mm) from the edge of the bridge deck to maximize the traversable deck width. For the top tension connection, the deck attachment hardware utilized a double angle connection bolted to the post web with tube spacers and plates embedded into the bridge deck. The lower compression anchorage connection featured two bolts connecting the post flange to tube spacers and plates embedded into to the side of the bridge deck.

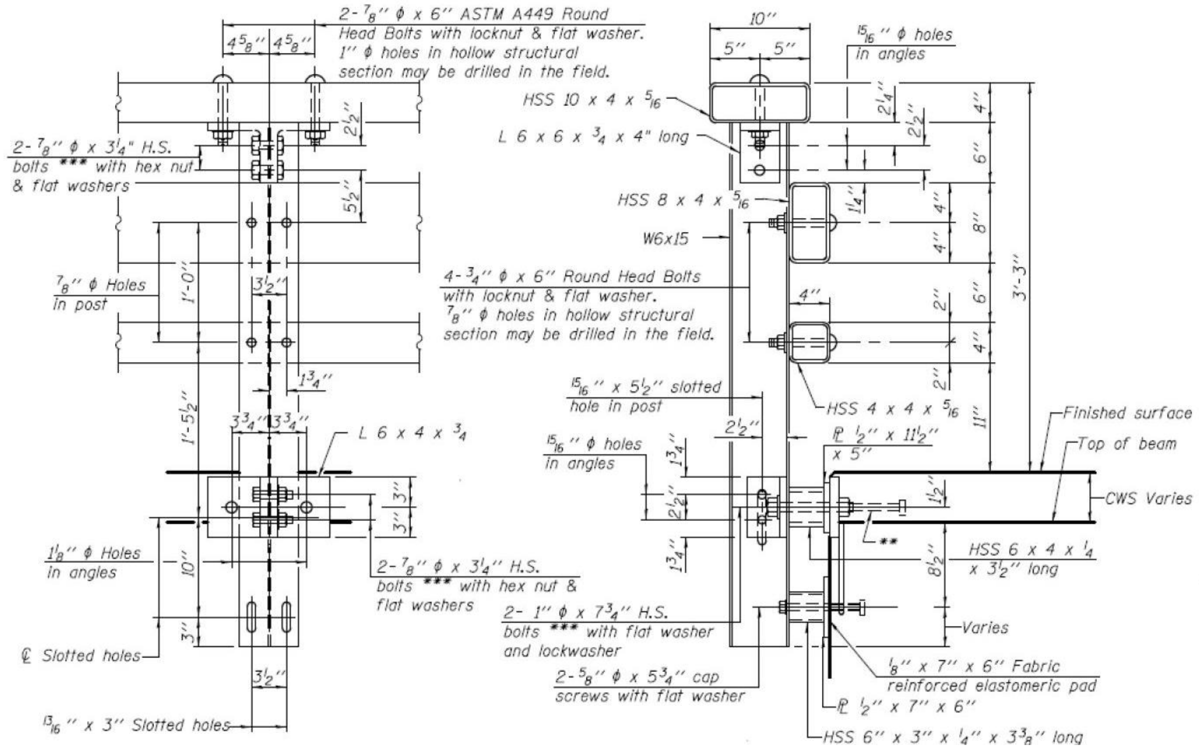


Figure 3. Preliminary Design for the MASH 2016 TL-4 Steel-Tube Bridge Railing

Both IDOT and ODOT desired to attach the new bridge rail 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, as shown in Figure 4. Specific deck configurations will be discussed in detail in Chapter 3. The bridge rail system was developed to be adaptable to all bridge superstructures utilized by IDOT and ODOT.

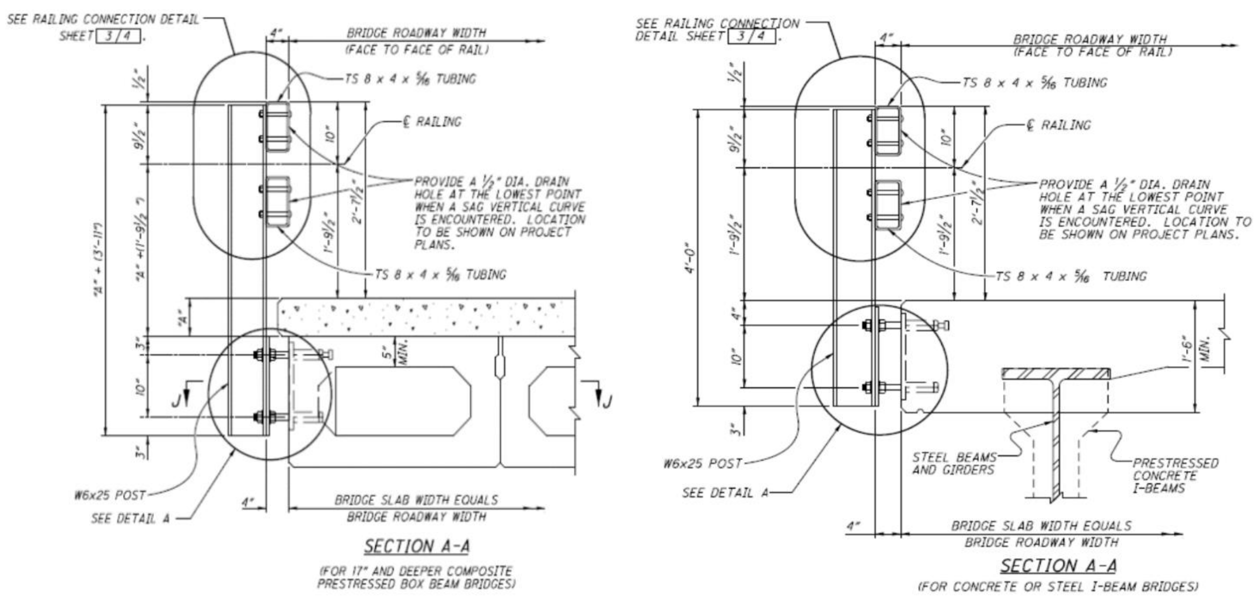


Figure 4. Bridge Rails Attached to Various Bridge Superstructures [1]

1.2 Research Objectives

The objective of this research study was to develop a MASH 2016 TL-4 steel-tube bridge rail. The bridge railing was to be mounted to the side of a bridge deck and not utilize a curb. The system was also required to limit impact load transferred to the deck, minimize the propensity for deck damage during impacts, and prevent vehicle snag and instabilities. ODOT and IDOT desired the new bridge rail to attach to bridge decks comprising of either a thick concrete slab or a pre-stressed concrete box-beam.

1.3 Research Scope

The development of the MASH 2016 TL-4 bridge rail and associated guardrail transition were 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, while Phase II consisted of the design and testing of an approach guardrail transition. This report describes the post-to-deck connection design, while the development and testing of the steel-tube bridge rail and approach guardrail transition connection are detailed in other reports [8-9]. The final implementation guidance and recommendations will provided in a final report [10]. Phase I of the research project began with a literature review of previously crash-tested side-mounted bridge rails and their components. Information garnered during the literature review was utilized to modify the preliminary railing design shown in Figure 3 and to develop a crashworthy MASH 2016 TL-4 bridge rail. The rail component sizes, locations, and orientations were optimized to limit installation costs while providing adequate strength. Additionally, the bridge rail components were designed to minimize the potential for vehicle snag on the posts and/or connection hardware.

Existing side-mounted post-to-deck connections for the various deck configurations were reviewed. A review of deck standards from both IDOT and ODOT were conducted to identify characteristics, such as deck thickness, overhang distance, reinforcement configurations, and material strengths, for both deck types within the two states. Finally, critical designs for each deck type were identified for use during the testing and evaluation of the bridge deck. Once the critical bridge deck configurations were selected, the post-to-deck attachment was designed and analyzed. Efforts were made to ensure that the attachment could withstand the full bending strength of the posts, thereby limiting the potential for deck damage during impact events. Concepts for the new post-to-deck attachment design were developed through a brainstorming process and were evaluated both analytically and through dynamic testing. A total of seven dynamic component tests were conducted on individual posts side-mounted to a pre-stressed, prefabricated concrete box-beam to evaluate the strength of the posts, attachment hardware, and the bridge deck, as well as to identify any damage that may be likely to occur during vehicle impacts. Finally, conclusions and recommendations were made pertaining to the post-to-deck connections design. The final implementation guidance and recommendations for the entire system will provided in a final report [10].

2 LITERATURE REVIEW

Phase I of the research project involved a literature search of previously crash-tested barriers that were considered relevant to the development of the steel-tube bridge rail. Prior research concerning steel-tube bridge rails, steel W-beam and thrie-beam bridge rails, and other side-mounted bridge rails were reviewed. The review focused on MASH TL-4 barrier rail systems that were side mounted. Few side-mounted rail systems have been tested to MASH TL-4 safety criteria. Therefore, the review was broadened to include any side-mounted systems evaluated to prior testing standards.

2.1 Safety Criteria

Over the years, a series of documents have been published to provide guidance on testing and evaluation of roadside safety features. In 1989, the American Associate of State Highway and Transportation Officials (AASHTO) adopted the *AASHTO Guide Specifications for Bridge Railings* that addressed bridge railing systems for three performance levels (PLs) [11]. These levels were defined by full-scale crash test conditions and performance evaluation criteria, and the guide further recommended procedures for determining which performance level was appropriate for a given facility and test condition. NCHRP Report 230 was also one of the first national standards used to provide guidance in regard to evaluating highway safety appurtenances across three multiple service levels (MSLs) [12]. NCHRP Report 350 replaced NCHRP Report 230 in 1993 and established six test levels (TLs) for longitudinal barriers to evaluate occupant risk, structural integrity of the barrier, and post-impact behavior of the vehicle for a variety of vehicles impacting at varying speeds and angles of impact [3].

Since its publication in 2009, MASH has been an update to and supersedes NCHRP Report 350 for the purpose of evaluating new safety hardware devices. Along with its 2016 edition, MASH implemented uniform guidelines for conducting full-scale crash tests for permanent and temporary highway safety features along with recommended evaluation criteria to assess test results. The guidelines and criteria, which have evolved over the past 40 years, incorporate current technology and the collective judgement and expertise of professionals in the field of roadside safety design.

2.2 Crash Testing Equivalencies

In a 1997 memorandum, the FHWA established crash test equivalencies amongst the NCHRP Report 350 and 230 test levels, and the *AASHTO Guide Specifications for Bridge Rails* performance levels [13]. No test level equivalencies have been determined for MASH test criteria. The equivalencies set forth by the FHWA are summarized in Table 1. Some test levels from NCHRP Report 230 and the *AASHTO Guide Specifications for Bridge Rails* do not pertain to the testing criteria set forth in NCHRP Report 350 and are therefore not listed in the table.

Table 1. FHWA Crash Test Equivalencies [13]

Bridge Railing Testing Criteria	Testing Level Equivalencies					
NCHRP Report 350 [3]	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
NCHRP Report 230 [12]	N/A	MSL-1 MSL-2	N/A	N/A	N/A	N/A
AASHTO Guide Spec. [11]	N/A	PL-1	N/A	PL-2	PL-3	N/A

N/A = No testing level equivalencies exist amongst standards

2.3 Impact Load and Height

Impact load studies for MASH TL-4 impacts were conducted and reported in NCHRP Project 22-20 *Design Guidelines for TL-3 through TL-5 Roadside Barrier Systems* to estimate the magnitude and distribution of the TL-4 impact load on barriers of different heights, as shown in Table 2, involving an SUT (10000S) vehicle weighing 22,036 lb (10,000 kg) impacting the barrier at a speed of 56 mph (90 km/h) at a 15-degree angle [6].

When an SUT impacts a barrier, there are two distinct impacts. The first impact occurs when the front cab of the vehicle contacts the barrier. The vehicle then begins to yaw or rotate away from the barrier. The second impact occurs when the rear axle and box contacts the barrier. This second impact is sometimes referred to as the "tail slap." Historically, the second impact generates the largest impact force. Due to changes in SUT vehicle properties and impact conditions incorporated into MASH, it was determined that 32-in. (813-mm) barrier height was no longer adequate for MASH TL-4.

The inadequate barrier height was demonstrated in a MASH TL-4 full-scale crash test of a 32-in. (813-mm) tall New Jersey Safety Shape bridge rail, in which the SUT vehicle rolled over the barrier and failed the structural adequacy criterion of MASH [14]. In a full-scale crash test of a 36-in. (914-mm) tall single slope traffic rail (SSTR), the 22,000-lb (9,982-kg) SUT was successfully contained and redirected after impacting the barrier at a speed of 57.2 mph (92 km/h) and an angle of 16.1 degrees. Therefore, a 36-in. (914-mm) barrier height is the minimum height that has successfully been crash tested and design impact loads at the minimum height were investigated.

From using simplified analysis techniques to explicit nonlinear Finite Element (FE) analysis, the variation and magnitude of the lateral, longitudinal, and vertical impact forces with barrier height were investigated [6]. A summary of the magnitude, distribution, and application of the resultant MASH TL-4 impact loads for the different barriers is presented in Table 2, with illustrations of the design forces shown in Figure 5. There are three forces involved: F_t is the transverse force, which is applied perpendicular to the barrier and is otherwise referred to as the impact force; F_L is the longitudinal force, which is applied by friction along the direction of the barrier; and F_v is the vertical force, which is applied downward on the top of the barrier. There are also three lengths associated with the results: the length L_L over which the lateral load F_t is distributed, though unevenly, in the longitudinal direction; the length L_V over which the lateral load F_t is distributed, though unevenly, in the vertical direction; and the height of the resultant of the peak force H_e from ground level. The design forces recommended by NCHRP, as shown in Figure 5, are applied to a beam and post railing, however, the forces, vertical locations, and horizontal distribution lengths shown apply to any type of railing.

Table 2. Magnitudes, Distributions, and Applications of the MASH TL-4 Impact Loads [6]

Design Forces And Designations	36-in. Tall Barrier	>36-in. Tall Barrier
F_t Transverse kip (kN)	70 (311)	80 (356)
F_L Longitudinal kip (kN)	22 (98)	27 (120)
F_v Vertical kip (kN)	38 (169)	33 (147)
L_L ft (m)	4 (1.2)	5 (1.5)
L_V ft (m)	18 (5.5)	18 (5.5)
H_e in. (mm)	25 (635)	30 (762)

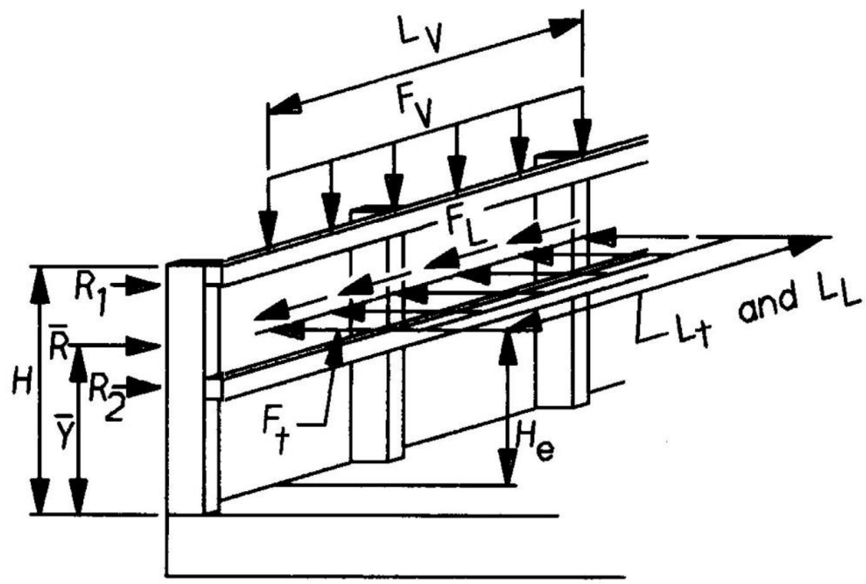


Figure 5. Metal Bridge Railing Design Forces and Designations [15]

2.4 Steel-Tube Bridge Rails

Various steel bridge rails incorporating tube-section rail elements have been developed and successfully tested. These bridge rail systems tend to be considered reasonably stiff, and feature steel posts side-mounted directly along the bridge deck or utilize post-to-deck attachment hardware that minimizes intrusion of the system onto the bridge deck.

2.4.1 California Type 15 Bridge Rail

The California Type 15 bridge rail is a steel-tube bridge rail featuring two HSS3½x3½x¼ rail elements mounted to W6x25 posts spaced 6 ft – 3 in. (1.91 m) apart, as shown in Figure 6 [16]. The Type 15 bridge rail met AASHTO PL-1 test criteria.

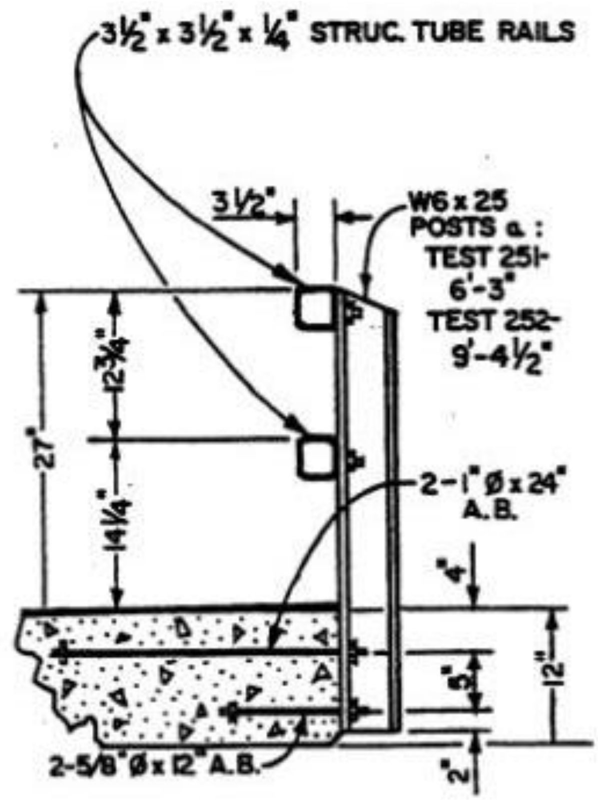


Figure 6. California Type 15 Bridge Barrier Rail [16]

The top rail height was 27 in. (686 mm), and the system was side-mounted to the bridge deck with two upper 1-in. (25-mm) by 24-in. (610-mm) long ASTM A108 Gr. 1144 threaded rods and two lower ⅝-in. (16-mm) by 12-in. (305-mm) long A325 high strength bolts cast into the concrete. The upper and lower anchorages were spaced 5 in. (127 mm) apart and the minimum slab deck thickness was 12 in. (305 mm). No post-to-deck lateral attachment hardware was utilized as the steel posts were placed flush to the bridge deck.

Successful crash tests were performed by the California Department of Transportation (Caltrans) using two passenger car vehicles. Two 4,500-lb (2,041-kg) passenger cars impacted the barrier rail at velocities of 64 mph (103 km/h) and 60 mph (97 km/h) and at impact angles of 12 and 15 degrees, respectively. These tests featured moderate damage, with only minor concrete spalling near the lower anchorages and on the underside of the bridge deck near impact locations. Impacted rail sections and posts were deformed, and replacement of the bridge rail would be necessary to sustain additional impacts. An 8-ft (2.4-m) post spacing was recommended to provide an overall smoother vehicle redirection.

2.4.2 California Type 18 Bridge Rail

Similar to the Type 15, the California Type 18 Bridge Rail consisted of W8x31 posts spaced at 8 ft (2.4 m) and supported an HSS4x4x$^{1}/_{4}$ upper rail and blockout, and an HSS12x3x$^{1}/_{4}$ lower rail mounted to a pipe section blockout designed to crush and absorb energy during impact, as shown in Figure 7 [17]. The bridge rail satisfied MSL-1 test criteria from NCHRP Report 230.

The top rail height was 36 in. (914 mm), and the posts were side-mounted to the bridge deck by two 1¼-in. (32-mm) diameter top bolts and two 1-in. (25-mm) diameter bottom bolts. All high strength bolts had a 24-in. (610-mm) embedment length. The top and bottom bolt layers were spaced at 4½ in. (114 mm) vertically. Five enclosing sets of No. 3 rebar reinforcement formed a cage around the bolts. A minimum deck thickness of 12 in. (305 mm) was required, and the top mounting height was 36 in. (914 mm) from the bridge deck surface. Posts featured ⅜-in. (10-mm) thick gusset plates placed between the post flanges at the deck surface level above the top anchors and in between the upper and lower bolts.

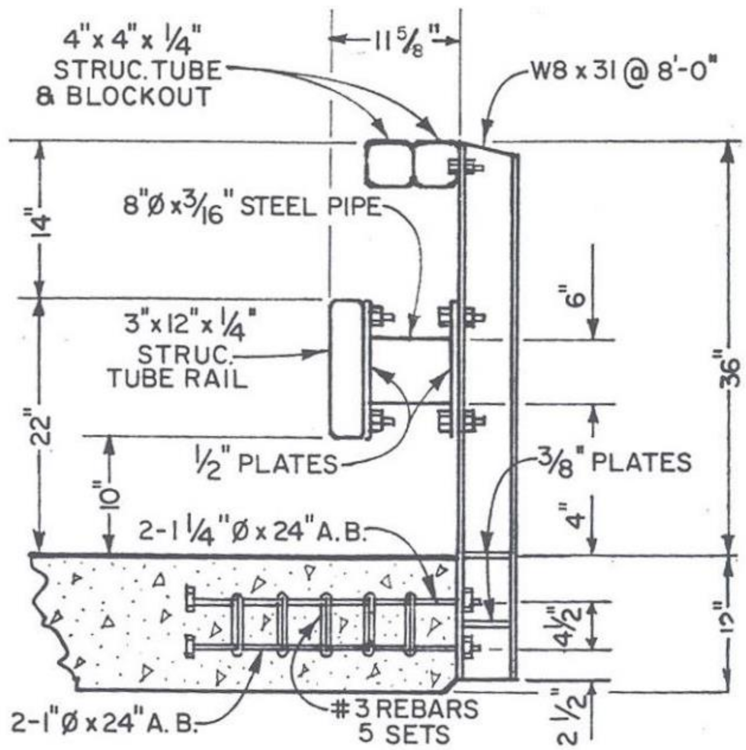


Figure 7. California Type 18 Bridge Rail [17]

Successful crash testing was performed on the California Type 18 Bridge Rail in a 1983 study [17]. The system smoothly redirected an 1,850-lb (839-kg) car impacting at 59.7 mph (96.1 km/h) and 12 degrees and a 4,530-lb (2,055-kg) car impacting at 60.7 mph (97.7 km/h) and 23 degrees. No distress was observed at the post-to-deck connections or at the cable end anchorages for the HSS12x3x¼ lower rail. The 1983 case study acknowledged the California Type 18 Bridge Rail needed to be better designed to prevent the wheels of small, lightweight cars from passing beneath the railings and from snagging on the posts when compared to the California Type 115 bridge rail.

2.4.3 California Type 115, 116, and 117 Bridge Rails

In the early 1990s, Caltrans developed and crash tested three similar side-mounted steel tube bridge rails for the state of California [18]. The California Type 115 featured two HSS4x4x¼ railings with W8x31 posts spaced at a minimum and maximum of 6 ft (1.83 m) and 8 ft (2.4 m), respectively, as shown in Figure 8. The system's top rail height was set at 30 in. (762 mm). The system failed to meet the intended AASHTO *Guide Specifications for Bridge Rails* test criteria at PL-2, but performed adequately at a PL-1 rating.

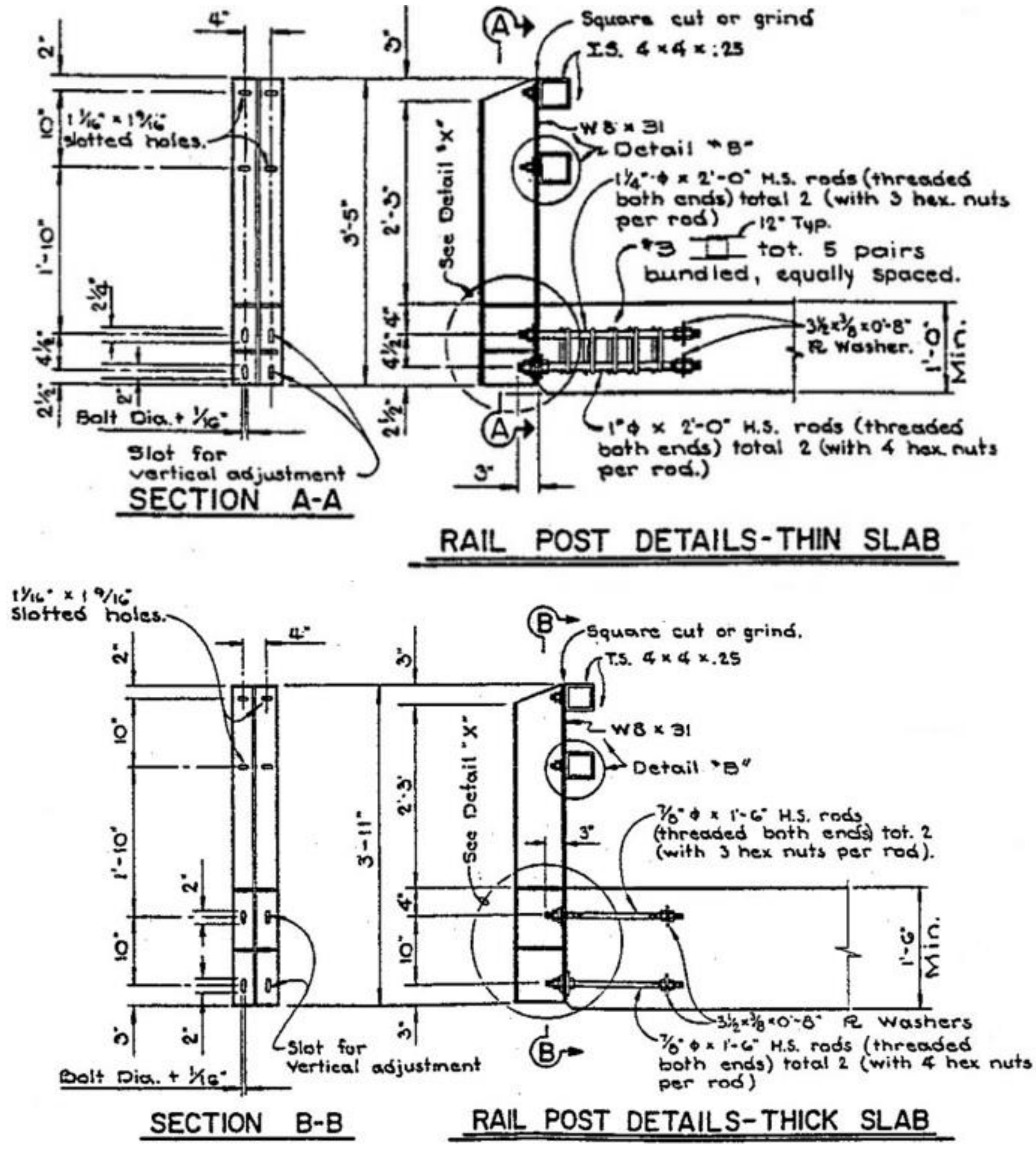


Figure 8. California Type 115 Bridge Rail Configurations [18]

The Type 115 was designed for bridge decks ranging from a minimum of 12 in. (305 mm) to 1 ft – 6 in. (457 mm). For the thin slab, posts were anchored to the side of the deck with two 1¼-in. (32-mm) diameter upper rods and two 1-in. (25-mm) diameter lower rods. Both upper and lower high strength threaded rods were 24 in. (610 mm) in length and were placed 4½ in. (114 mm) apart within the deck. The thin slab configuration was possible by the inclusion of the five sets of No. 3 loops encasing the upper and lower anchor rods. For the thick slab, the diameters of both the upper and lower anchor rods decreased to ⅞ in. (22 mm), with lengths of 18 in. (457 mm), and lateral anchor placements of 10 in. (254 mm) apart. The Type 116 and 117 Bridge Rails were similar to the Type 115 in that the Type 116 featured an additional, smaller upper rail section, whereas the Type 117 used two additional, smaller upper rail sections, as shown in Figure 9.

The California Type 115 was crashed tested in a 1993 study [18]. A 1,800-lb (816-kg) car impacted the barrier rail at 59 mph (94.8 km/h) and 19 degrees, and a 5,470-lb (2,450-kg) pickup truck impacted the rail at 64 mph (103 km/h) and 21 degrees. Wheel snagging and moderate pocketing by the small car impact disqualified the PL-2 test rating. The Type 115 bridge rail performed adequately for a PL-1 rating, which is considered equivalent to TL-2 safety criteria under NCHRP Report 350. The Type 116 and 117 bridge rails were also considered to be TL-2 barrier rail systems.

Figure 9. California Type 116 and 117 Bridge Rails [18]

2.4.4 California ST-70SM

The California ST-70SM is a MASH TL-4 steel-tube bridge rail developed and tested by the Caltrans to provide a side-mounted bridge rail that could be used in areas where the posted speed limit could be more than 45 mph (72 km/h) [19]. The ST-70SM is a four steel-tube side railing with built-up steel posts side-mounted to the edge of the bridge deck, as shown in Figure 10.

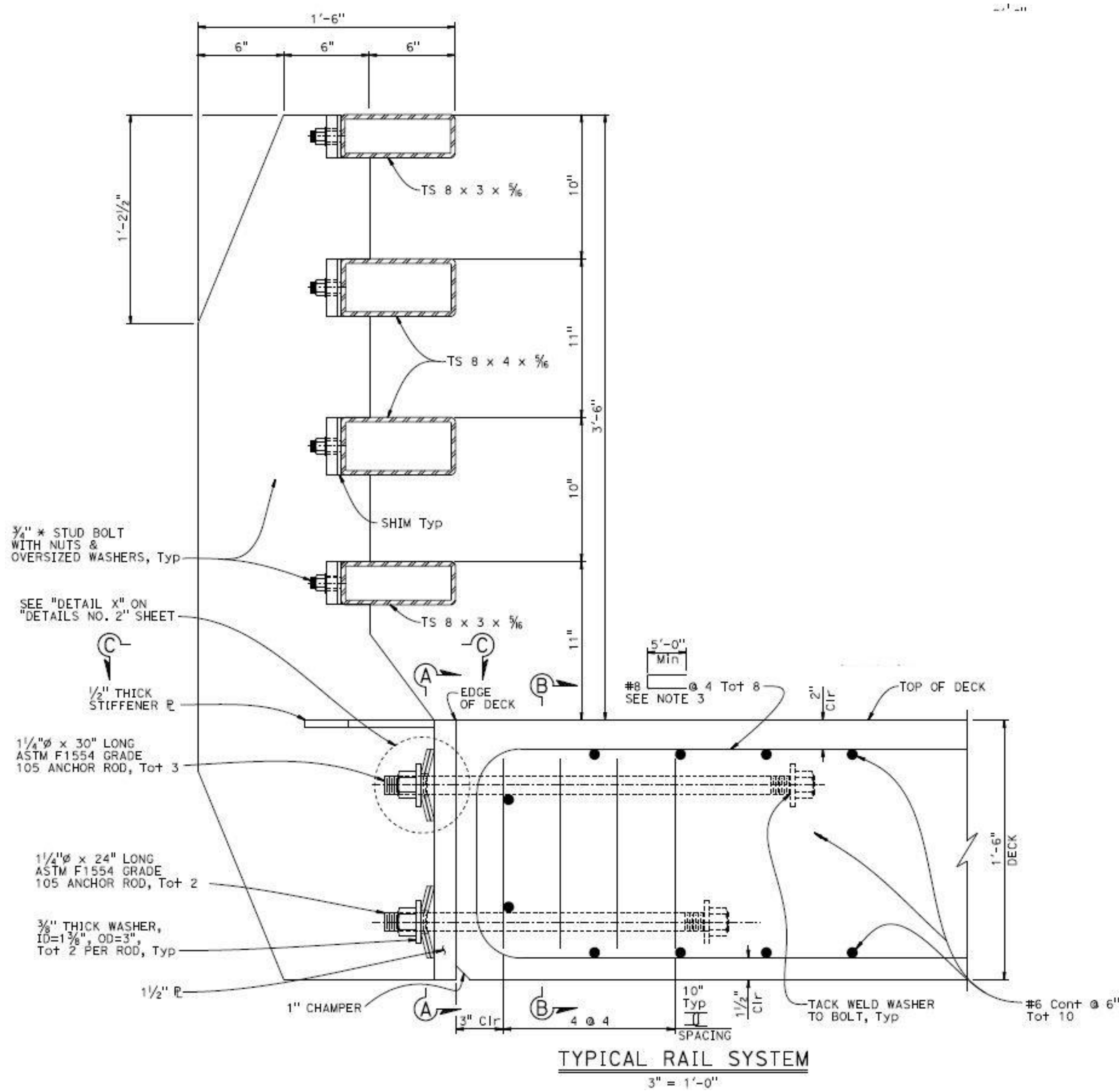


Figure 10. California ST-70SM Bridge Rail [19]

The top rail height was 42 in. (1,067 mm). The upper and lower longitudinal railings were HSS8x3x$^{5}/_{16}$, and the middle two rails were HSS8x4x$^{5}/_{16}$ with built-up posts spaced 10 ft (3.05 m) apart. Five anchor rods with disc springs attached each post to the edge of the deck. All anchorages used to anchor the posts to the bridge superstructure were 1¼-in. (32-mm) diameter ASTM F1554 Grade 105 rods, with the upper three rods having a length of 30 in. (762 mm) and the two lower anchor rods a length of 20 in. (508 mm). The steel bridge rail was designed for a maximum bridge deck thickness of 18 in. (457 mm). Disc springs and strain gages were located on posts within the expected impact location with string potentiometers instrumented on the anchor rods.

The California ST-70SM bridge rail met criteria set in MASH as a TL-4 longitudinal barrier after successfully being subjected to three full-scale crash tests [19]. Post-impact analysis determined that some of the high strength anchor rods may have entered plastic deformation during the SUT impact. However, the anchor rods were intact after the test and expected to have full capacity. Although the side-mounted bridge rail successfully redirected all test vehicles, it was recommended to inspect the disc springs and possibly replace them, if necessary, for impacts similar to the pickup truck and SUT.

2.4.5 Illinois Side-Mounted Bridge Rail

The Illinois Side-Mounted Bridge Rail is a side-mounted system consisting of wide-flange posts and tubular steel rail elements designed and tested to the former AASHTO crash standards at PL-2, equivalent to an NCHRP Report 350 TL-4 [20]. The bridge rail design consisted of W6x25 posts spaced at 6 ft – 3 in. (1.9 m) with a HSS8x4x$^{5}/_{16}$ top rail element and a HSS6x4x¼ bottom rail element, as shown in Figure 11.

The top height of the metal railing above the asphalt surface was 32 in. (813 mm). The steel posts were side-mounted to a prestressed-concrete deck with four AASHTO M164 anchor bolts. Post-to-deck attachment hardware featured an HSS member welded to the front face of the post, with two upper bolts anchoring the post into the deck through double angles that were bolted onto the post web. The lower bolts were anchored to the deck through the post flanges and an HSS member was also placed in between the bridge deck and the post. Anchors were spaced at 10 in. (254 mm) vertically on center for a 17-in. (432-mm) thick concrete deck.

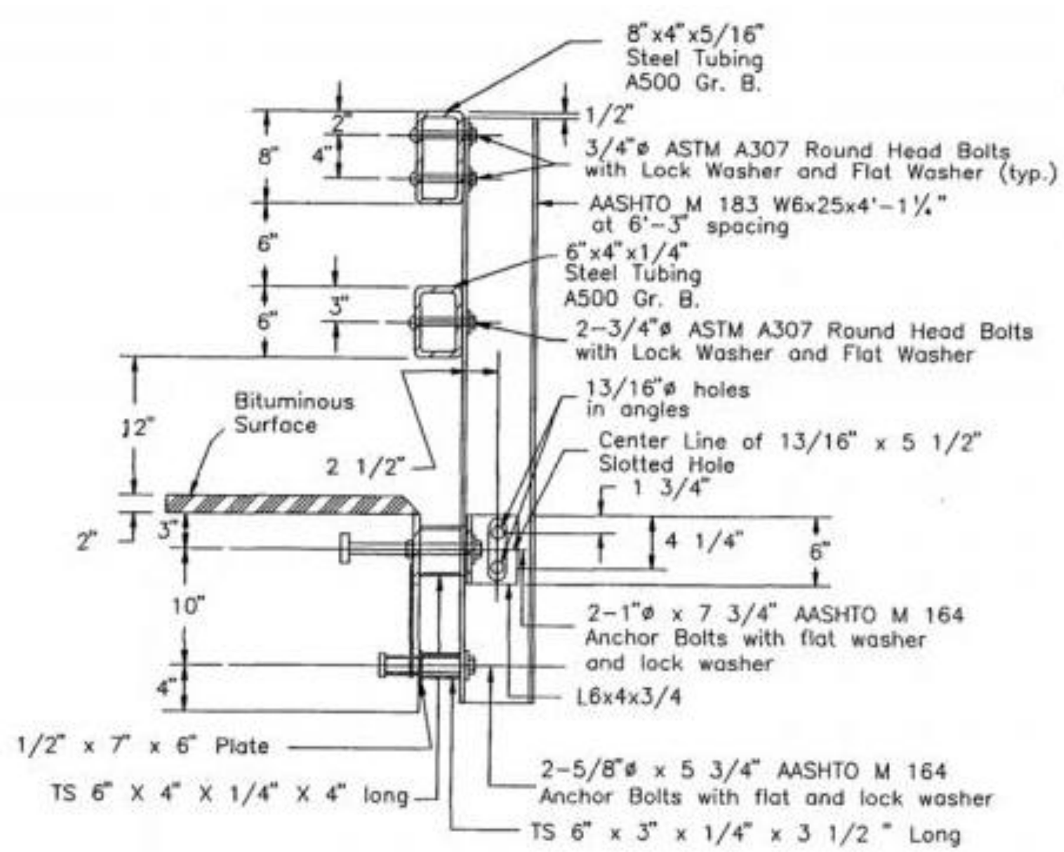


Figure 11. Illinois Side-Mounted Bridge Rail [20]

The Illinois Side-Mount Bridge Railing was tested to PL-2. Acceptable performance was demonstrated with 1,800-lb (817-kg) small car, 5,400-lb (2,452-kg) pickup truck, and 18,000-lb (8,200-kg) SUT crash tests with minimal to moderate damage observed in the post flanges at the post-to-deck connections. Some of the tube spacers between the deck and post flange were unfastened, and angles were deformed. The bridge rail met PL-2 safety criteria, and the barrier rail was considered equivalent to NCHRP Report 350 TL-4.

2.4.6 Oregon Two-Tube Bridge Rail

The Oregon Two-Tube Bridge Rail utilizes similar longitudinal rail elements, steel posts, post spacing, and post-to-deck connection attachments as the Illinois Side-Mounted Bridge Rail, as shown in Figure 12 [21].

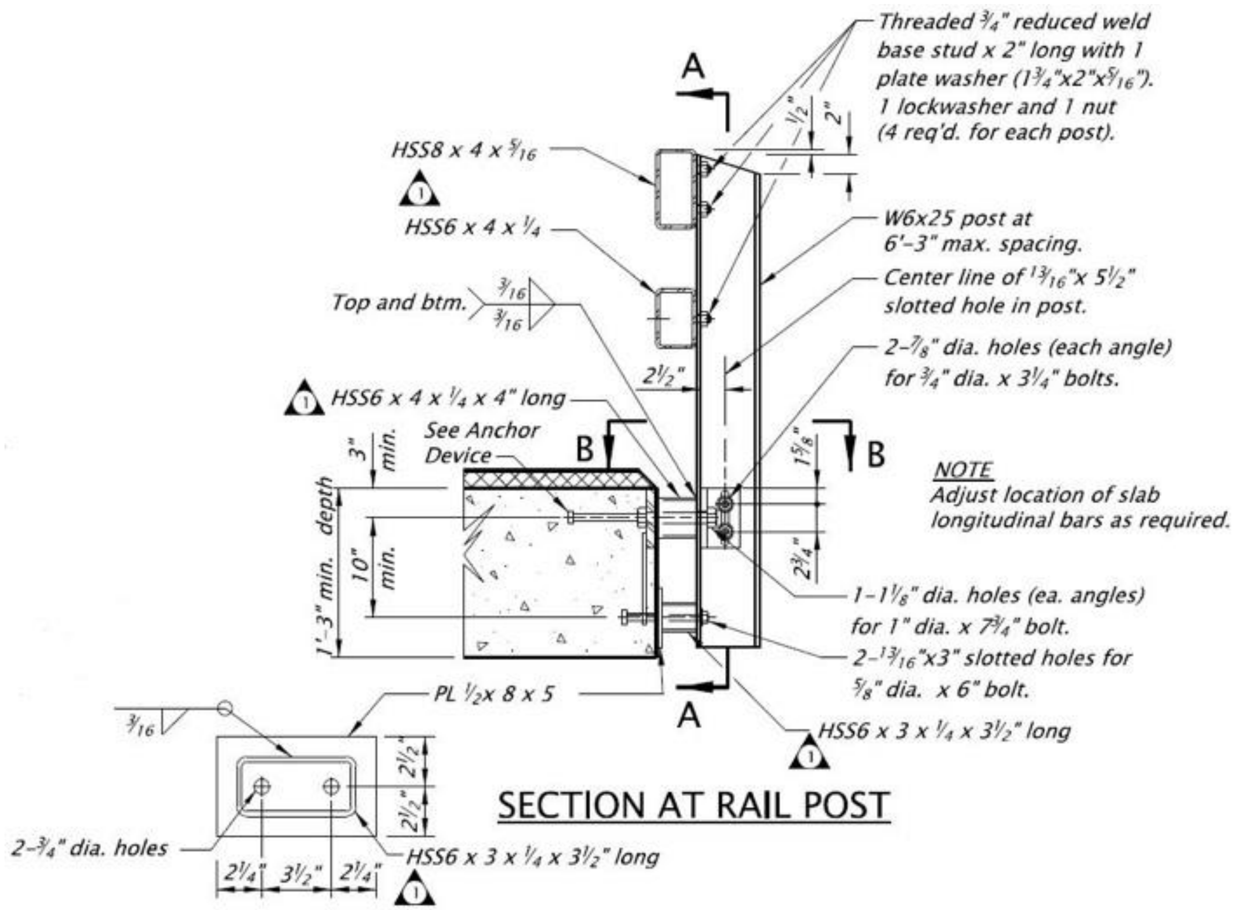


Figure 12. Oregon Two-Tube Bridge Rail [21]

Anchorages featured high strength ASTM A325 bolts spaced 10 in. (254 mm) vertically apart for a 15-in. (381-mm) minimum depth concrete slab. No actual crash test data and/or FHWA reports were found during the literature review of this system, but bridge rail plans of the system were obtained from Oregon DOT.

2.4.7 New York City Verrazano-Narrows Bridge Rail

The Verrazano-Narrows Bridge Rail is a steel-tube bridge railing designed specifically for use on the Verrazano-Narrows Bridge in New York City and was developed to satisfy MASH TL-5 impact safety criteria, as shown in Figure 13 [22].

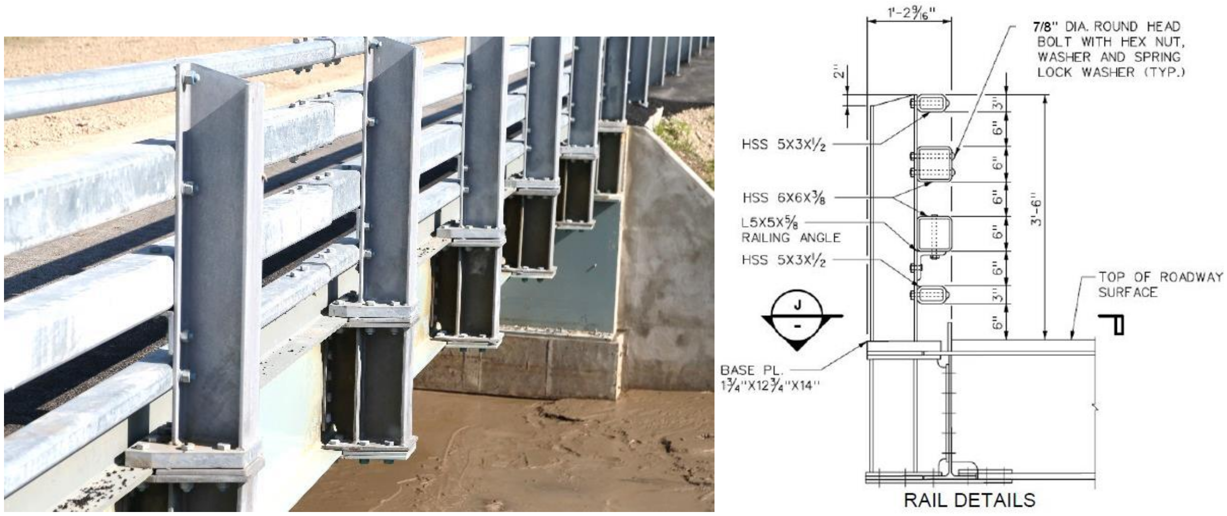


Figure 13. NYC Verrazano-Narrows Bridge Rail [22]

The top rail height was 42 in. (1,067 mm) and the system consisted of four longitudinal steel tubes mounted to side-mounted steel posts. The rail elements were two HSS5x3x½ upper and lower steel tubes and two HSS6x6x⅜ middle steel tubes. The lower middle rail was secured to the post with a 5-in. x 5-in. x ⅜-in. (127-mm x 127-mm x 9½-mm) railing shelf angle that was 6½-in. (165-mm) long. The bridge deck contained a 5-in. (127-mm) tall vertical steel plate curb and allowed the posts to be bolted to extensions off the side of the deck. The bolts were supported by and bolted to the bridge deck lateral sub-floor beams, longitudinal stringer extensions, and the railing connection extensions.

The system was subjected to, and successfully passed, all three full-scale crash tests required by MASH TL-5 [22]. In each of the tests, the vehicle did not penetrate, underride, or override the installation. The Texas A&M Transportation Institute (TTI) observed very small maximum dynamic and permanent deformations, which would not require repair after most impacts. The Verrazano-Narrows Bridge Rail performed acceptably according to MASH TL-5 evaluation criteria.

2.4.8 Ohio Steel Fascia Mounted Bridge Rail

The Ohio Steel Fascia Mounted Bridge Rail was a modification of the side-mounted Illinois two-tube bridge rail [23]. The original Illinois two-tube system was rated at NCHRP Report 350 TL-4, but the Ohio Steel Fascia bridge rail modified design was only considered for TL-3 applications. Modifications made to the bridge rail were limited to the post-mount design, as shown in Figure 14. No changes were made to any bridge rail components above the road surface.

The top rail height was 32 in. (813 mm). The original post-mount design was replaced with a modified basic fascia mount design concept featuring a structural tube spacer, either an HSS14x6x¼ or an HSS12x6x¼, between two 12-in. x 6-in. x ¾-in. (305-mm x 152-mm x 19-mm) thick plates. The new mount design concept also featured post-stiffeners utilizing 1-in. (25-mm) thick stiffening plates welded onto the post above the modified post-mount to compensate for the additional moment induced due to the increased length of the post required for the new mount design. Strength assessment of the new mount design was investigated via pendulum testing to verify equivalent stiffness response compared to the original mount design. The modified post-mount design was shown to provide equal or greater stiffness to the original post-mount and, therefore, shall result in equivalent or better crash performance for the system when installed on steel bridges with fascia beams of size W14x30 and larger. Through use of finite element analysis simulations, the new post-mount design satisfied NCHRP Report 350 TL-3 safety performance criteria.

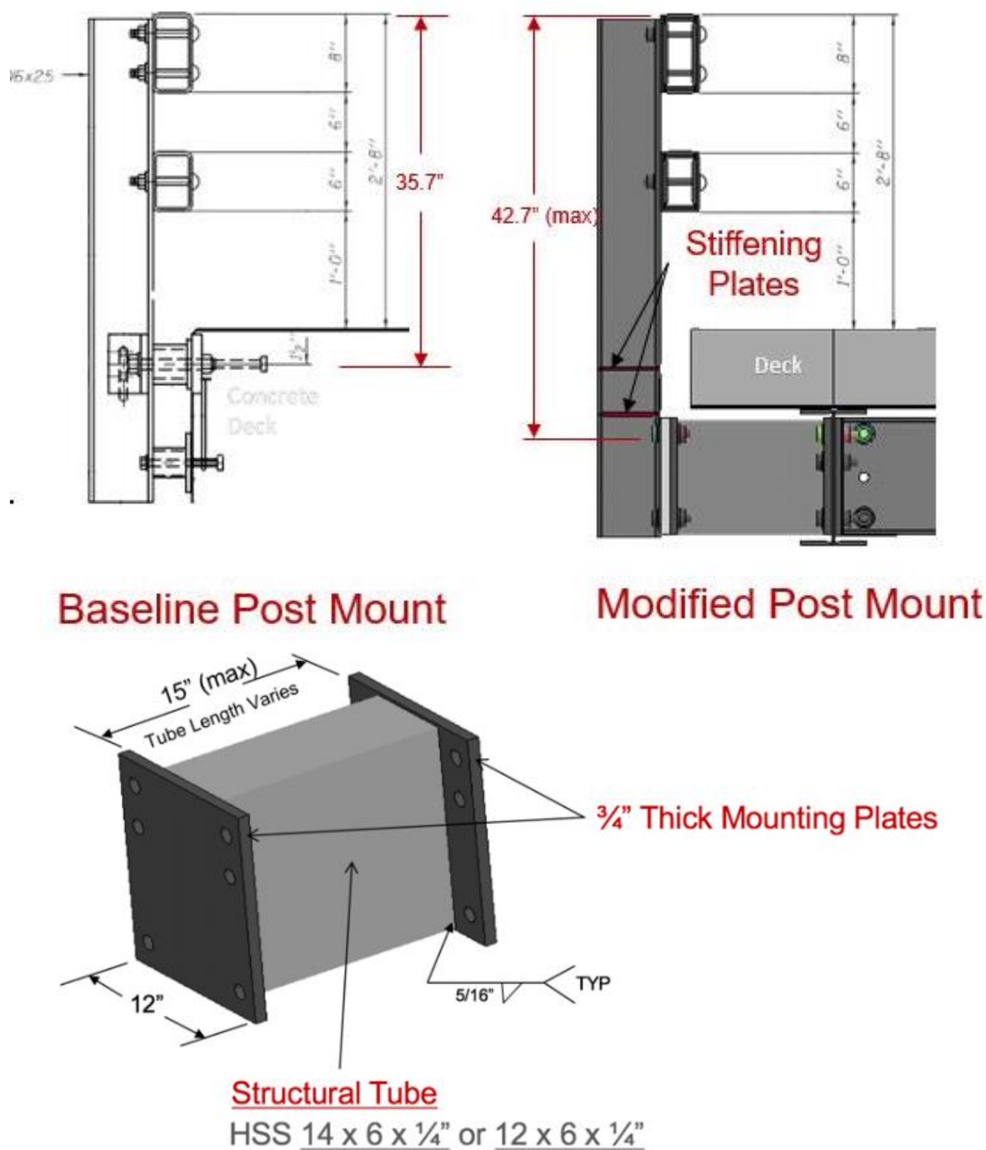


Figure 14. Ohio Steel Fascia Basic Mount Design Concept [23]

2.5 W-Beam and Thrie-Beam Bridge Rails and Guardrails

Several bridge rails utilizing W-beam rail sections with tube-section blockouts have specialized post-to-deck hardware attachments to minimize intrusion onto the bridge deck. A number of W-beam guardrails have been developed for MASH TL-2 and TL-3 performance criteria. Such systems tend to be much more forgiving than most bridge rail systems when impacted, and typically feature steel post-to-deck attachment hardware or feature steel posts anchored directly onto the bridge deck.

2.5.1 Ohio Deep Box-beam Rail

The Ohio Deep Box-beam Rail utilized a standard 12-gauge (2.5-mm) W-beam rail with an 8-in. x 4-in. x $^{3}/_{16}$–in. (203-mm x 102-mm x 5-mm) tubular backup beam, as shown in Figure 15 [23]. The Ohio Deep Box-beam Rail met all performance criteria for NCHRP Report 230 MSL-2, which is considered equivalent to NCHRP Report 350 TL-3.

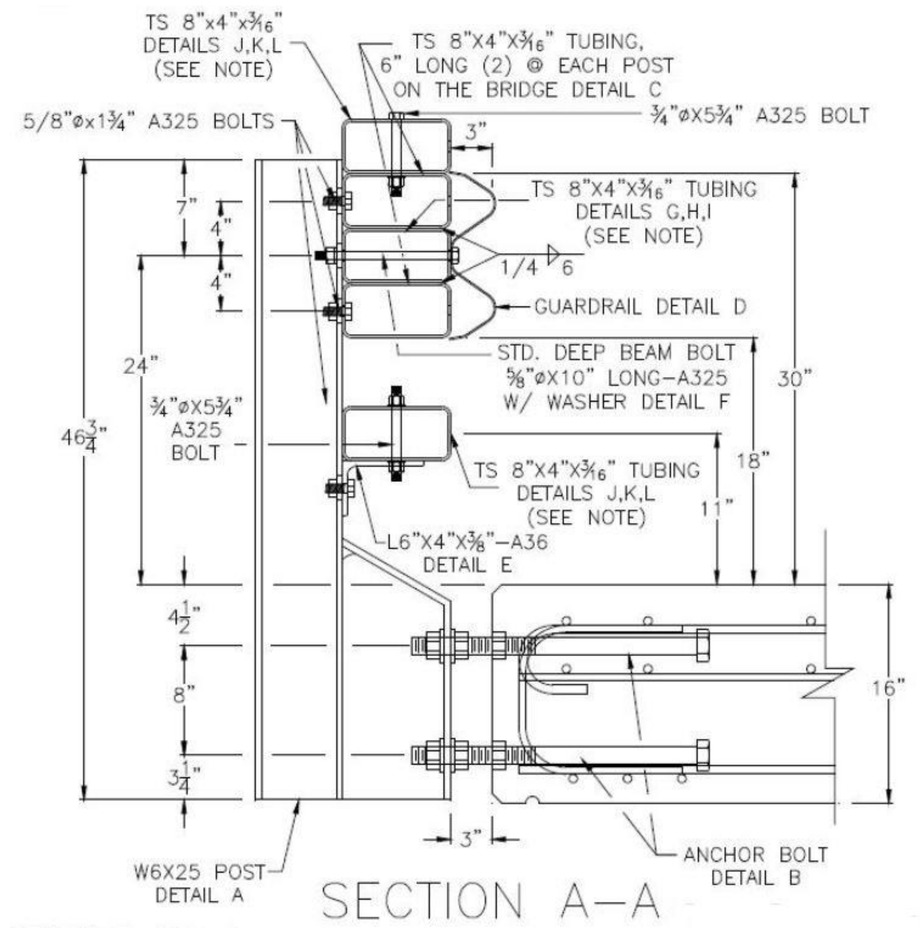


Figure 15. Ohio Deep Box-beam Rail [23]

Top W-beam rail height was 30 in. (762 mm) above the deck while the top box-beam rail height was 34 in. (864 mm), and the steel posts were W6x25 sections spaced at 6 ft – 3 in. (1.9 m) on center. Future modifications from the original box-beam rail featured additional 6-in. (152-mm) long box-beams attached above and below the backup rail at each post as blockouts. Steel posts were mounted with anchor assemblies featuring 1¼-in. (32-mm) diameter studs and bolts extending through the exterior edge of the bridge deck and passing through the front flanges of the posts.

The system was crash tested in 1987 under NCHRP Report 230 criteria as a MSL-2 system [23]. Two vehicles were used for testing, a 1,980-lb (898-kg) small car impacting the rail at 60.5 mph (97.4 km/h) and at an angle of 19.6 degrees, and a 4,790-lb (2,171-kg) pickup truck impacting the rail at 61 mph (98 km/h) and at an angle of 25 degrees. In both tests, the vehicles were smoothly directed, and the bridge rail and deck sustained only minor damage.

2.5.2 Michigan W-Beam Side-Mounted Rail

The Michigan Side-Mounted W-Beam bridge rail used W6x25 posts spaced at 6 ft – 3 in. (1.9 m) that supported 8-in. x 4-in. x $^{3}/_{16}$-in. (203-mm x 102-mm x 5-mm) box-beam and a standard 12-gauge (2.5-mm) W-beam, as shown in Figure 16 [24]. No research, crash testing reports, or FHWA approval letters were found during the literature review of the system; only bridge plans were obtained from Michigan DOT.

The top rail height of the W-beam was 27 in. (686 mm), and posts were attached directly to the side of the bridge slab using anchor bolts. Alternatively, the posts could be welded to spacer sections that were then bolted to the deck to help reduce rail encroachment onto the deck surface. Four 1¼-in. (32-mm) diameter anchor bolts were used, with upper anchors positioned 8 in. (203 mm) above the lower anchors. Additional box-beam blockouts were used above and below the box-beam rail at each post. The bridge rail can also be mounted to box girder bridge decks.

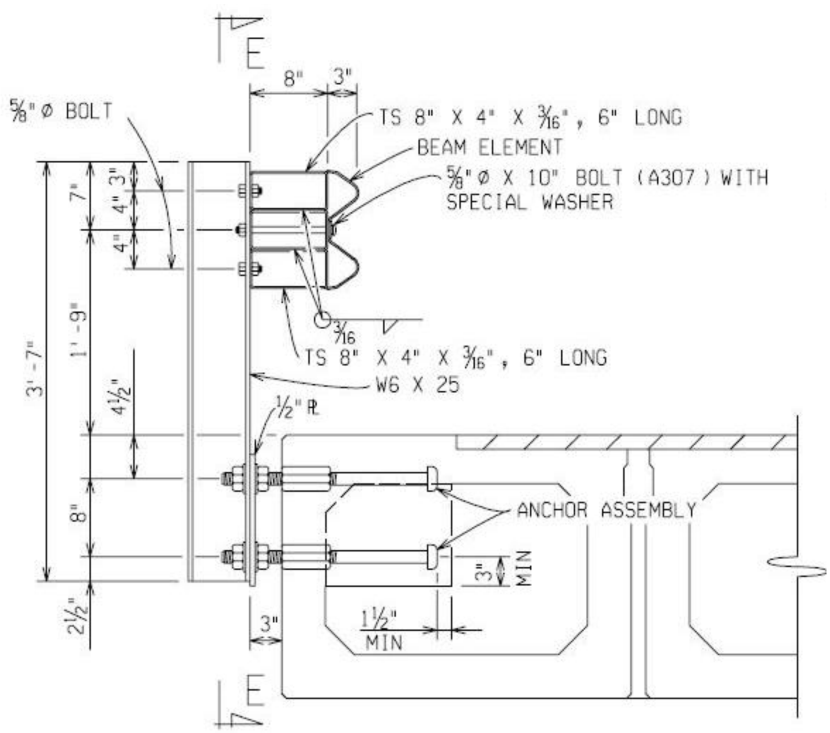


Figure 16. Michigan W-Beam Side-Mounted Rail [24]

2.5.3 California Thrie-Beam Bridge Rail

The California Thrie-Beam Bridge Rail utilized a 10-gauge (3-mm) thrie-beam rail on W6x15.5 posts and blockouts spaced at 6 ft – 3 in. (1.9 m) and side-mounted to the bridge deck, as shown in Figure 17 [17]. The bridge rail satisfied AASHTO PL-1 testing criteria, which was later deemed equivalent to NCHRP Report 350 TL-2.

Two 1¼-in. (32-mm) diameter top anchor rods and two ¾-in. (19-mm) diameter bottom anchor rods, with a length of 24 in. (610 mm), attached the posts to the side of the bridge superstructure. Posts were directly attached to the bridge deck with no lateral offset. The top and bottom anchors were vertically spaced 5 in. (127 mm) apart. The top rail height was 32 in. (813 mm) from the top of the bridge deck. Anchor rods were placed through the front flange at each post. Minimum deck thickness was 12 in. (305 mm), and an approach guardrail transition was required.

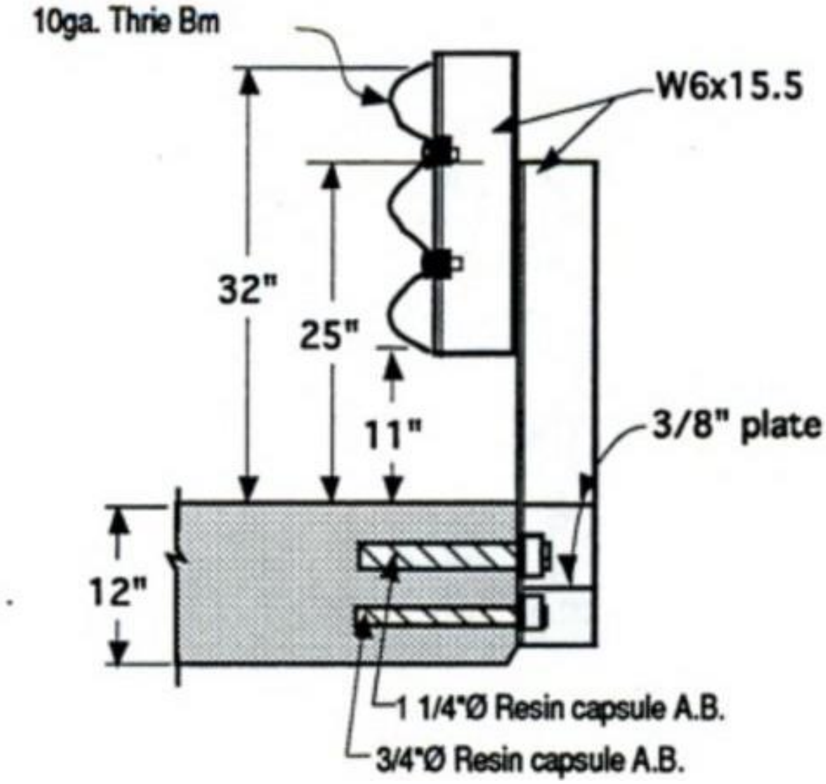


Figure 17. California Thrie-Beam Rail [17]

Crash testing for the California system was performed in a 1983 test study [17] by Caltrans under AASHTO test criteria at PL-1. The testing of the system was later deemed equivalent to NCHRP Report 350 criteria as a TL-2 system. A 5,400-lb (2,449-kg) pickup truck impacted the barrier at 44.9 mph (72.3 km/h) at an angle of 21 degrees and was successfully contained and redirected. Severity of impact was limited to the impact area with posts bent below the concrete deck level. The system also successfully redirected a 1,770-lb (803-kg) car impacting at 48.7 mph (78.4 km/h) at an angle of 18.3 degrees. Damage was limited to the impact area with minor scraping along the thrie-beam panel.

2.5.4 Oregon Side-Mounted Thrie-Beam Bridge Rail

The Oregon Side-Mounted Thrie-Beam Bridge Rail consisted of a 10-gauge (3-mm thick) thrie-beam rail mounted to W6x15 posts spaced at 6 ft – 3 in. (1.9 m) and met AASHTO PL-1 testing criteria, equivalent to NCHRP Report 350 TL-2, as shown in Figure 18 [20, 25].

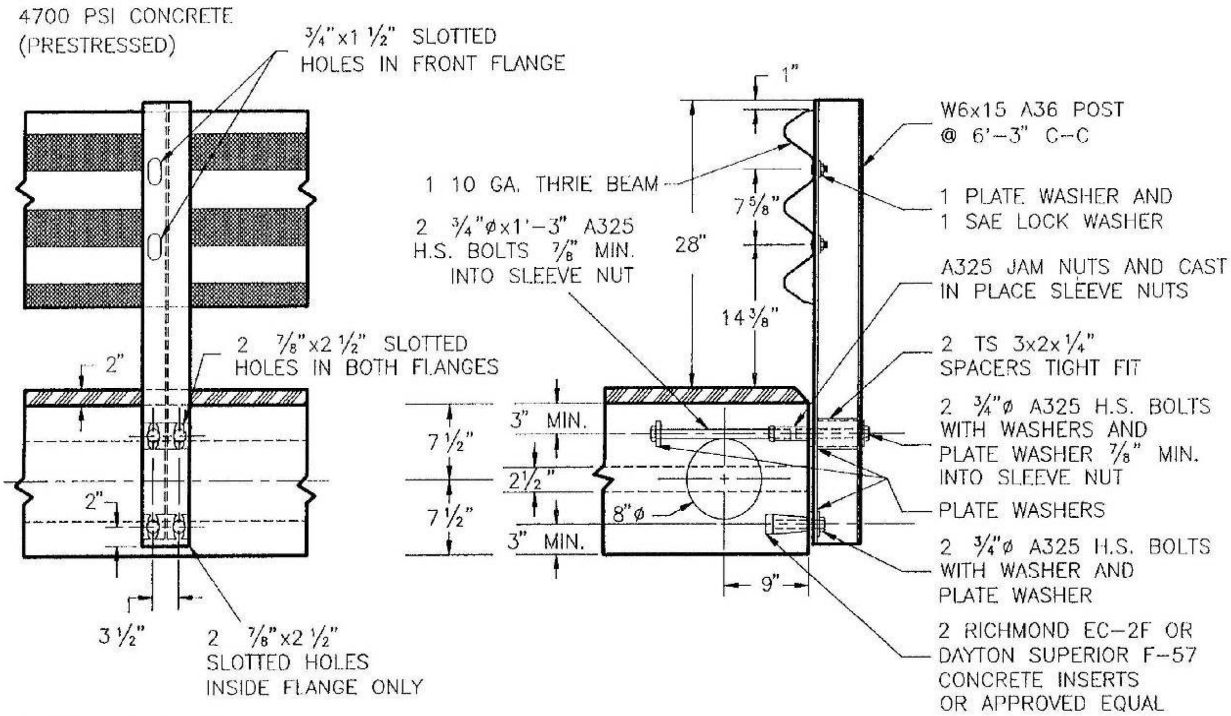


Figure 18. Oregon Side-Mounted Thrie-Beam Rail [25]

The top rail height of the system was 27 in. (686 mm) from the surface of the bridge deck. Steel posts were directly side-mounted to the bridge deck with no lateral offset. Side-mount anchors comprised two ¾-in. (19-mm) diameter by 1-ft 3-in. (381-mm) long top high strength A325 bolts and two ¾-in. (19-mm) bottom high strength A325 bolts placed in concrete inserts with an unknown embedded depth. The top two bolts were bolted through 3-in. x 2-in. x ¼-in. (76-mm x 51-mm x 6-mm) tube spacers placed between the post flanges. Minimum bridge deck thickness was 15 in. (381 mm), and an approach guardrail transition was required for the system.

The bridge rail system underwent two crash tests in a 1997 test study [20, 25]. The thrie-beam bridge rail system performed successfully for a 1,970-lb (894-kg) car impact at 52.2 mph (84 km/h) and at angle of 19.7 degrees and for a 5,738-lb (2,603-kg) pickup truck impact at 46.1 mph (74.2 km/h) and at an angle of 20.9 degrees.

2.5.5 TBC-8000 Bridge Rail

The Steel Thrie-Beam Rail with Upper Channel (TBC-8000) system is a steel thrie-beam bridge rail comprising a thrie-beam rail with an upper structural tube rail, a top mounted C-channel, and wide flange posts and blockouts [26], meeting AASHTO PL-2 testing criteria deemed equivalent to NCHRP Report 350 TL-4, as shown in Figure 19.

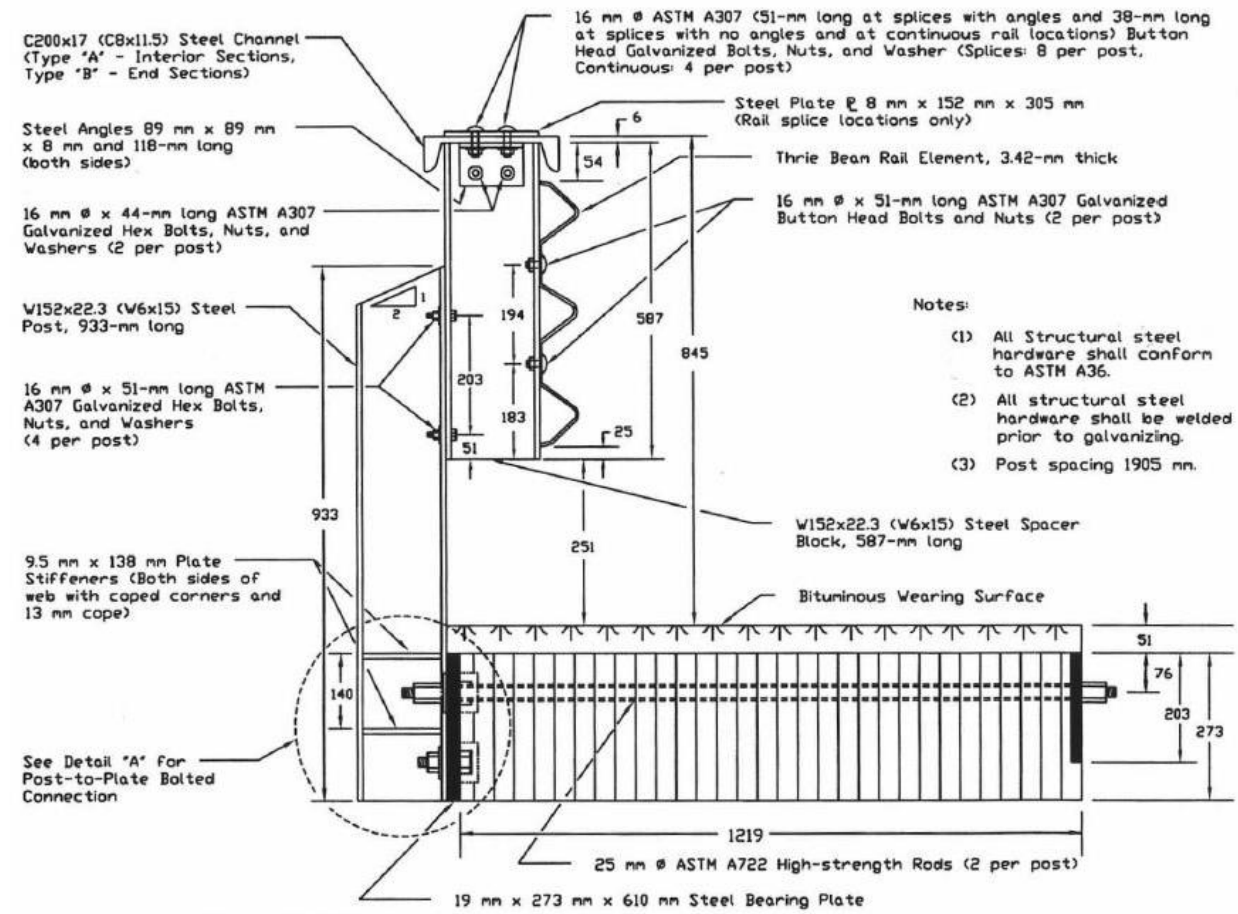


Figure 19. TBC-8000 Steel Thrie-Beam Rail [26]

The TBC-8000 system was designed for use on glulam longitudinal timber decks by MwRSF at the University of Nebraska-Lincoln. The system bridge rail consisted of W6x15 steel posts and blockouts spaced at 6 ft – 3 in. (1.91 m) supporting a 10-gauge (3-mm) thrie-beam rail and a C8x11.5 channel section. The top rail height of the system was an approximate 33 in. (838 mm) from the bridge deck surface. When a 2-in. (51-mm) wearing surface is utilized, the top rail height is 31 in. (787 mm). Posts were side-mounted to two exterior steel plates placed on the side of the bridge deck with two 1-in. (25-mm) diameter threaded anchors extending 4 ft (1.22 m) into the bridge deck and into an anchor plate.

The TBC-8000 bridge rail system was successfully tested to AASHTO PL-2 criteria. Successful crash testing involved an 18,000-lb (8,165-kg) SUT impacting the bridge rail at 47.4 mph (76.3 km/h) and at an impact angle of 16.1 degrees. The maximum permanent set was $8^{3}/_{16}$ in. (208 mm).

2.5.6 TL-4 Thrie–Beam Bridge Rail for Glulam Timber Decks

An NCHRP Report 350 TL-4 thrie-beam bridge rail was developed for use on transverse glulam timber decks by MwRSF in 2002 [27], as shown in Figure 20. The system featured W6x15 steel posts side-mounted to the timber deck at an 8-ft (2.44-m) spacing with bolted connections to the upper and lower anchor plates. The anchor plates were attached to the top and bottom of the bridge deck with twelve ⅞-in. (22-mm) diameter bolts installed through the timber deck. Use of supplementary W6x15 steel sections were considered for blockage of the 10-gauge (3-mm) thrie-beam rail away from the posts. Steel tubes of 8 in. x 3 in. x $^{3}/_{16}$ in. (203-mm x 76-mm x 5-mm) sections were used as secondary railings placed above the thrie-beam.

Two crash tests were performed on the TL-4 steel bridge rail utilizing a pickup truck and a SUT to NCHRP Report 350 test criteria. The 4,396-kg (1,994-kg) pickup truck impacted the system at 58.2 mph (93.7 km/h) and at an angle of 25.5 degrees to the rail while the 17,785-lb (8,067-kg) SUT traveled at 47.5 mph (76.5 km/h) and at an angle of 14.6 degrees relative to the bridge rail. Both vehicles were smoothly redirected and contained maximum permanent deflections of 4⅝ in. (117 mm) and 5⅜ in. (137 mm), respectively.

Figure 20. TL-4 Thrie-Beam Bridge Rail for Timber Decks [27]

2.5.7 Weak-Post Midwest Guardrail System Bridge Railing

A low-cost bridge rail was designed to be compatible with the Midwest Guardrail System (MGS) with the intention to minimize bridge deck and rail costs without requiring a separate approach guardrail transition between the two barriers [28]. The system featured S3x5.7 steel posts equipped with ¼-in. (6-mm) thick standoff shim plates utilized within a 4-in. x 4-in. x ⅜-in. (102-mm x 102-mm x 9½-mm) steel tube designed as a post socket, with a ⅝-in. (16-mm) diameter bolt used to hold the post in the socket. The top rail height of the system was 31 in. (787 mm). With the weak-posts housed within the socket assemblies, the bridge rail was attached to the edge of an 8-in. (203-mm) thick bridge deck and anchored to the deck with one through-deck bolt, as shown in Figure 21. A W-beam section was used as the rail element and was attached to the weak-posts with a bolt designed to break during an impact event.

Figure 21. Weak-Post Midwest Guardrail System [28]

The weak-post, low-cost bridge rail was designed by MwRSF, and two full-scale crash tests were performed. The bridge rail successfully redirected a 2,425-lb (1,100-kg) passenger car impacting the system at a nominal speed and angle of 62 mph (100 km/h) and 25 degrees, respectively, and a 5,000-lb (2,268-kg) pickup truck impacting the system at a nominal speed and angle of 62 mph (100 km/h) and 25 degrees, respectively. Full-scale crash testing met all required safety criteria for a MASH TL-3 longitudinal barrier. The bridge rail dynamically deflected 28 in. (711 mm) during the passenger car impact and 48.9 in. (1,242 mm) during the pickup truck impact. Damage to the barrier was moderate, mainly consisting of deformed W-beam rail and bridge posts as well as splice extension due to membrane action to the rail. The bridge deck sustained minor damage in both tests, including deck cracking and spalling. In the passenger car crash test, punching shear cracks were observed on the outside edge of the deck at one post and lateral shear cracks were found at another post location. In the pickup truck test, severe cracking occurred at one post, however, the through-deck bolt and bolt sleeve were not displaced.

2.5.8 Weak-Post Midwest Guardrail System on Culvert Headwalls

A new weak-post, W-beam guardrail system for use on low-fill culverts was developed and evaluated by MwRSF [29]. The system was adapted from the MGS bridge railing for attachment to the outside face of culvert headwalls, utilizing the same weak, S3x5.7 posts spaced

3 ft – 1½ in. (953 mm) on center and positioned within HSS4x4x⅜ socket assemblies. The top rail height was 31 in. (787 mm). The HSS socket assemblies and the culvert attachment hardware had to be modified in order for the system to be mounted to the outside face of the culvert headwalls, as shown in Figure 22. A side-mounted design was recommended for use based on acceptable performance during dynamic component tests and ease of fabrication and installation.

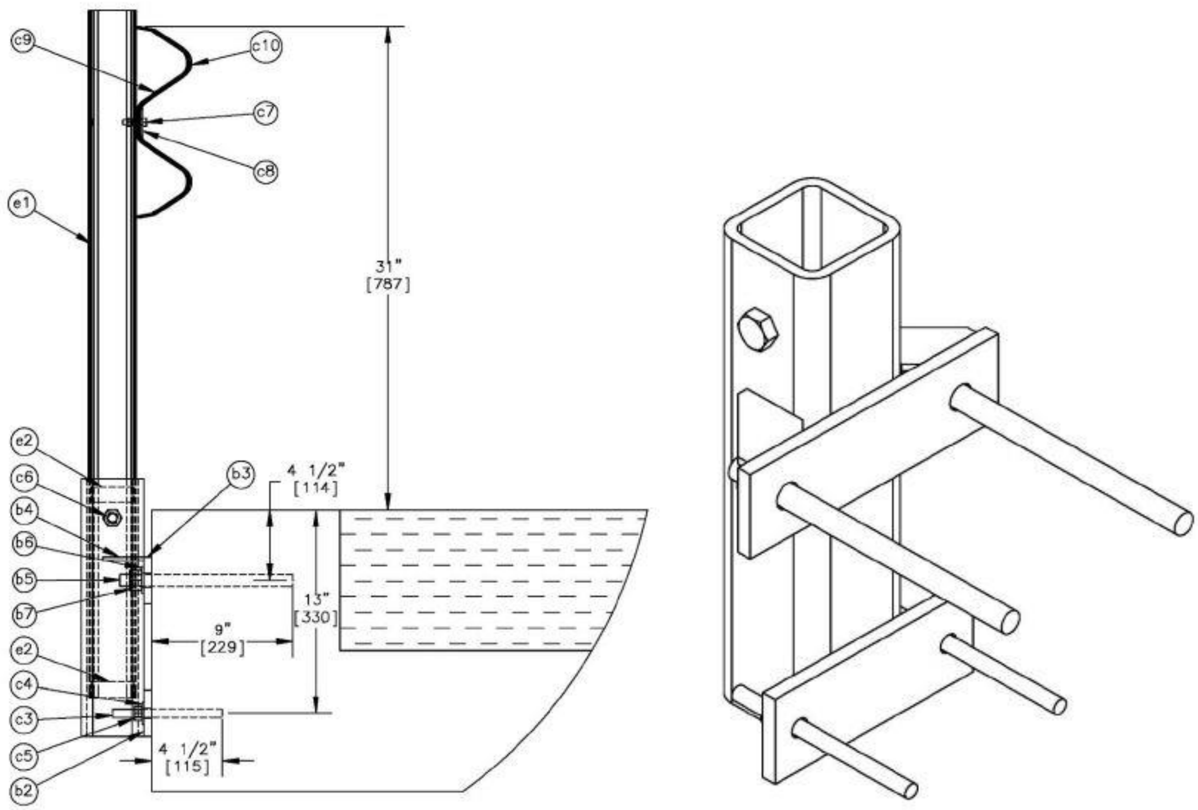


Figure 22. Weak-Post Guardrail Side-Mount Attachment

3 POST-TO-DECK ATTACHMENT DESIGN

Post-to-deck anchorage loads were investigated to minimize concrete deck damage. The weaker W6x15 steel post was selected over the stiffer W6x25 in order to reduce the impact load transferred to the post anchorage connection. The weaker W6x15 was designed to be fully developed to its plastic bending capacity under impact in order to reduce the magnitude of the load transferred to the deck and mitigate bridge deck damage. This assumption guided the selection of the weaker W6x15 over the existing W6x25 steel post in the IDOT and ODOT side-mount bridge rails.

3.1 Design Criteria for Steel-Tube Bridge Rail

Several design criteria were established for the new MASH 2016 TL-4 bridge rail. As previously mentioned, the bridge rail was to incorporate a 39-in. (991-mm) top height to account for future 3-in. (76-mm) thick roadway overlays on the bridge while maintaining a minimum MASH TL-4 barrier height of 36 in. (914 mm). The railing was to consist of three longitudinal steel tubes attached to side-mounted, W6x15 steel posts. The front face of the tube railings was to be flush with the outer edge of the bridge deck to maximize the traversable deck width. The post-to-deck attachment system was to be designed to fully develop the capacity of the W6x15 posts without causing bridge deck damage. The post attachment hardware was to be designed to sustain impact loads transferred to the deck while preventing deck damage. Both the post-to-deck connection and internal deck hardware needed to be compatible with IDOT and ODOT's existing state deck configurations.

3.2 Illinois and Ohio Existing Designs

3.2.1 Illinois Type Side-Mount Steel Bridge Rail

The existing Illinois steel bridge rail is a side-mounted system consisting of wide-flange posts and tubular steel rail elements previously designed and tested to the AASHTO PL-2 crash standards, now equivalent to the NCHRP Report 350 TL-4 standard [20]. The bridge rail design consists of a W6x25 steel post spaced at 6 ft – 3 in. (1.9 m) with an HSS8x4x$^{5}/_{16}$ top rail element and an HSS6x4x¼ bottom rail element, as shown in Figure 23.

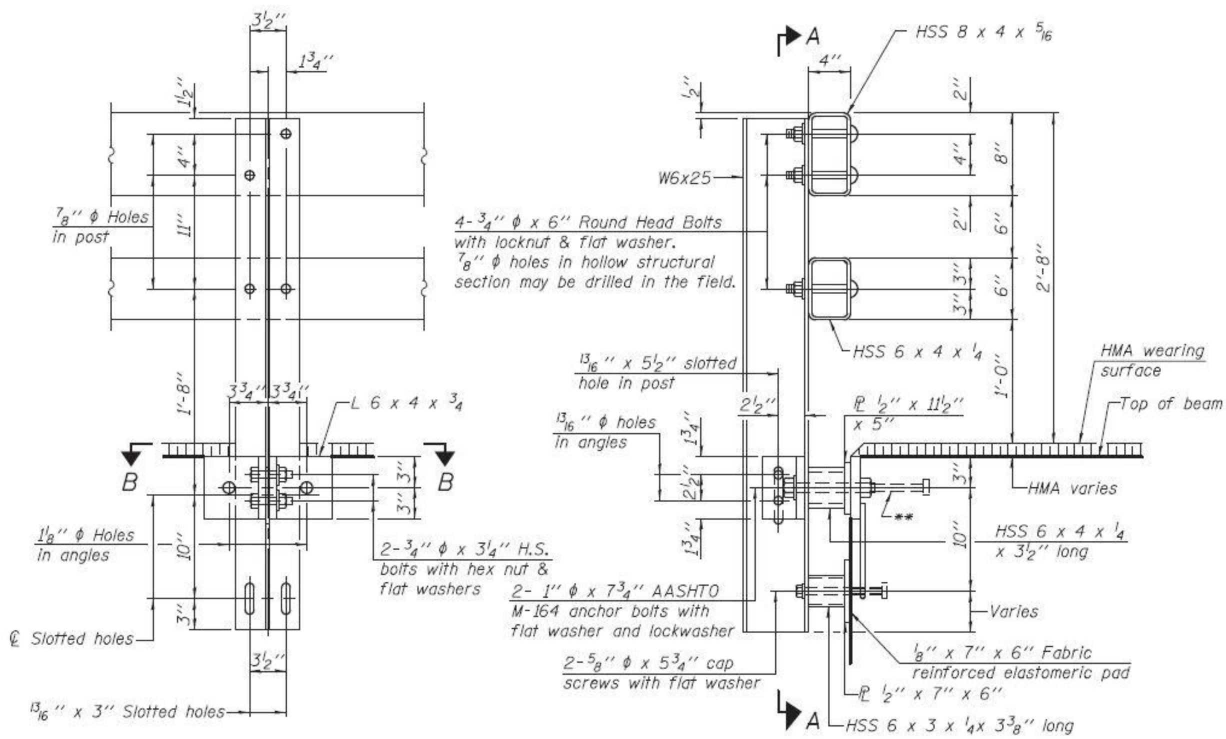


Figure 23. Existing Illinois Side-Mount Steel Bridge Rail [2]

Post-to-deck attachments consist of a top 6-in. x 4-in. x ¾-in. (152-mm x 102-mm x 19-mm) double angle connection at the post web that is bolted into the bridge deck with two 1-in. (25-mm) diameter AASHTO M164 bolts. Furthermore, the top connection features an HSS6x4x¼ structural tube that is welded to an 11½-in. x 5-in. x ½-in. (292-mm x 127-mm x 13-mm) plate mounted to the bridge deck. Similarly, the bottom connection utilizes an HSS6x3x¼ tube welded to a 7-in. x 6-in. x ½-in. (178-mm x 152-mm x 13-mm) mounting plate that is attached to a fabric-reinforced elastomeric pad on the bridge deck surface. Two ⅝-in. (16-mm) diameter cap screws are bolted through the post flanges into the bridge deck. The post is laterally offset 4 in. (102 mm) from the deck, the same depth as the tube rail elements. The deck hardware attachment configuration is shown in Figure 24.

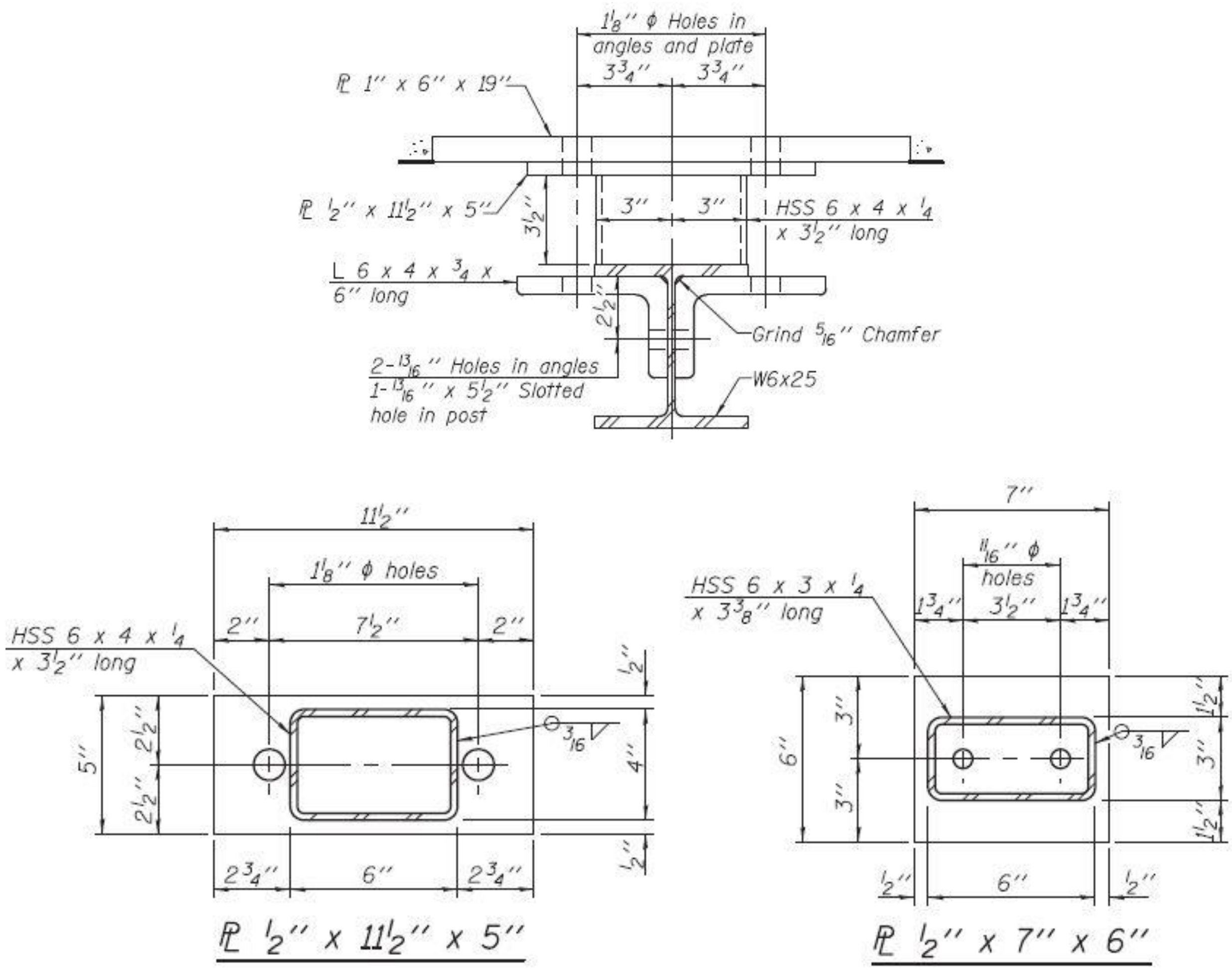


Figure 24. Illinois Deck Attachments [2]

The post-to-deck connection is anchored to the bridge slab or box-beam with a ¾-in. (19-mm) diameter by 6-in. (152-mm) long granular or solid flux filled headed studs bent at 30 degrees and welded to a 6-in. x 19-in. x 1-in. (152-mm x 483-mm x 25-mm) embedded plate, as shown in Figure 25. The top two AASHTO M164 anchor bolts are threaded into 1-in. (25-mm) diameter, high-strength, AASHTO M164 nuts that are welded to the embedded plate. Welded to the bottom of the embedded deck plate is a No. 3 rebar loop reinforcement that traverses below two 1-in. (25-mm) diameter round bar stocks that provide anchorage for the lower two ⅝-in. (16-mm) diameter cap screws that attach the lower post through the post flanges into the bridge deck. The round stocks and the No. 3 rebar loop reinforcement are welded to a 1½-in. x 6-in. x ½-in. (38-mm x 152-mm x 13-mm) embedded plate.

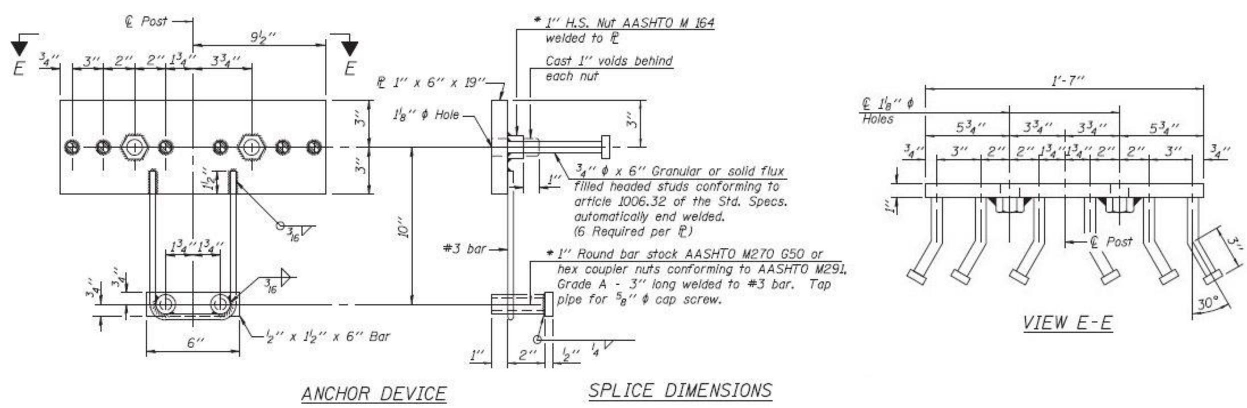


Figure 25. Illinois Post Anchorage Device [2]

3.2.2 Ohio Twin Steel Tube Bridge Railing

The existing Ohio side-mounted bridge rail is a steel tube system similar to the existing Illinois Type Side-Mount bridge rail. The Ohio bridge rail meets NCHRP Report 350 TL-4 safety criteria. The Ohio bridge rail design consists of W6x25 steel posts spaced at a maximum of 6 ft – 3 in. (1.9 m) with two HSS8x4x$^{5}/_{16}$ rail elements. The existing Ohio bridge railing system with typical connection hardware can be seen in Figure 26. Four 1-in. (25-mm) diameter anchor bolts extend from the deck to the flange with a 1-in. (25-mm) thick base plate on the post. Overall post lateral offset to the deck is 4 in. (102 mm), which is the 4-in. (102-mm) depth of the rail elements.

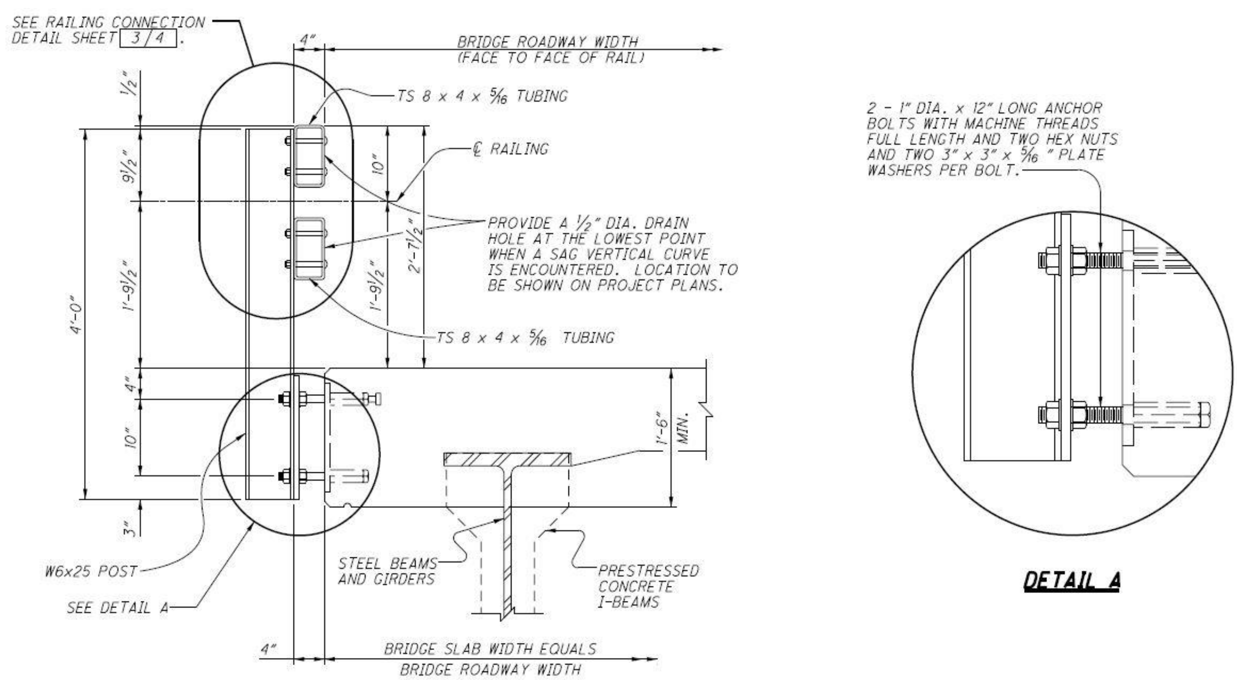


Figure 26. Existing Ohio Steel Tube Bridge Rail [1]

Post anchorage attachment into the concrete box-beam or the bridge slab consists of an embedded ¾-in. (19-mm) thick plate with ¾-in. (19-mm) diameter by 6-in. (152-mm) long welded studs. Attachment bolts connect to 5-in. (127-mm) long sleeve nuts welded to the anchorage plate, as shown in Figure 27.

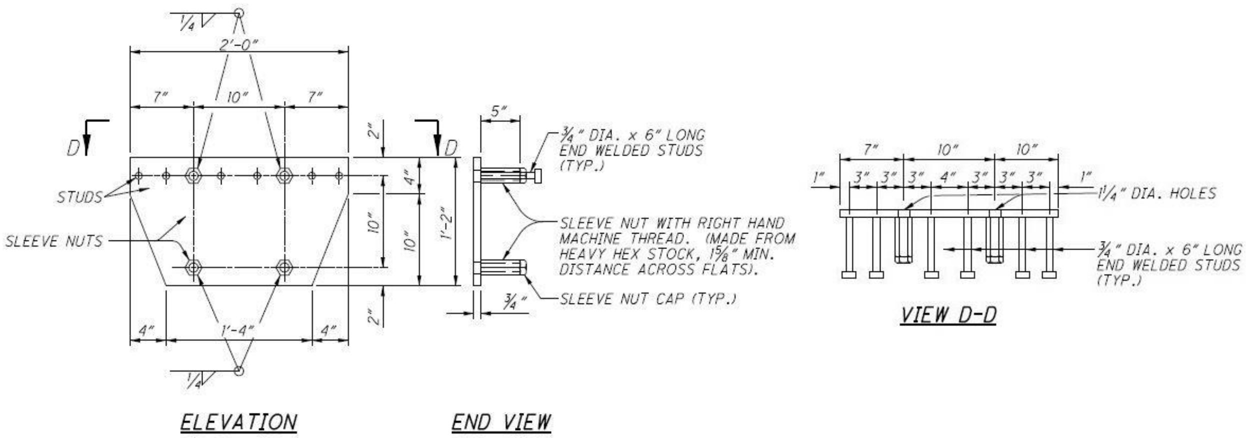


Figure 27. Ohio Post Anchorage Device [1]

3.3 Illinois-Ohio MASH TL-4 Steel-Tube Bridge Rail Prototype

Through initial discussions between IDOT, ODOT, and MwRSF, a preliminary steel-tube bridge railing design was developed. This steel bridge railing system would have a vertical face and may deflect under loading. The minimum height for a MASH 2016 TL-4 bridge rail was determined to be 36 in. (914 mm) [6]. The preliminary design had a top height of 39 in. (991 mm) to account for up to a 3-in. (76-mm) thick future roadway overlays on the bridge while still preventing SUTs from overriding the barrier. The railing consisted of three longitudinal steel tubes attached to side-mounted, W6x15 steel posts, as shown in Figure 28. The prototype rail design served as the basis for the new bridge rails, but several modifications were recommended throughout the design process.

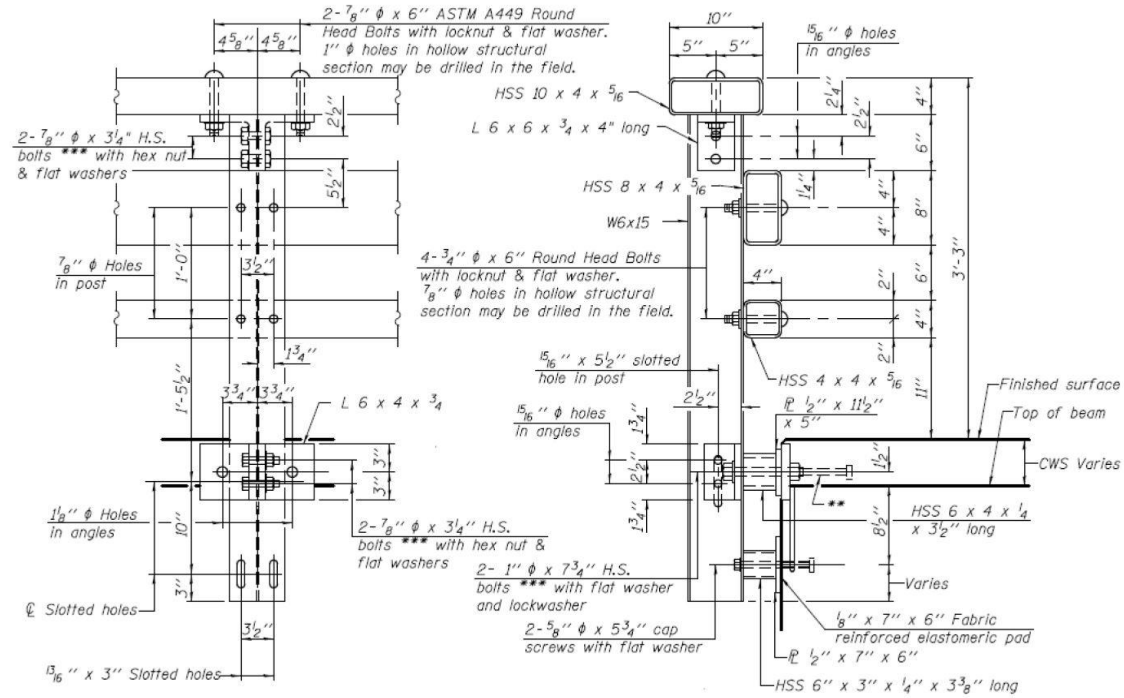


Figure 28. Preliminary Design for the MASH 2016 TL-4 Steel Tube Bridge Railing

3.4 Bridge Deck Configurations

Several bridge deck configurations were utilized by IDOT and ODOT for their side-mounted steel-tube bridge rails; the states have similar bridge decks but offer distinct deck anchorages for the bridge rail. Therefore, the IDOT and ODOT deck standards were reviewed to identify critical configurations for side-mounted bridge rails.

3.4.1 Illinois Bridge Deck Configuration

The Illinois 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 concrete box-beam girders were precast, pre-stressed box-beam girders of 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 29. Note that either option can feature an additional asphalt wearing surface.

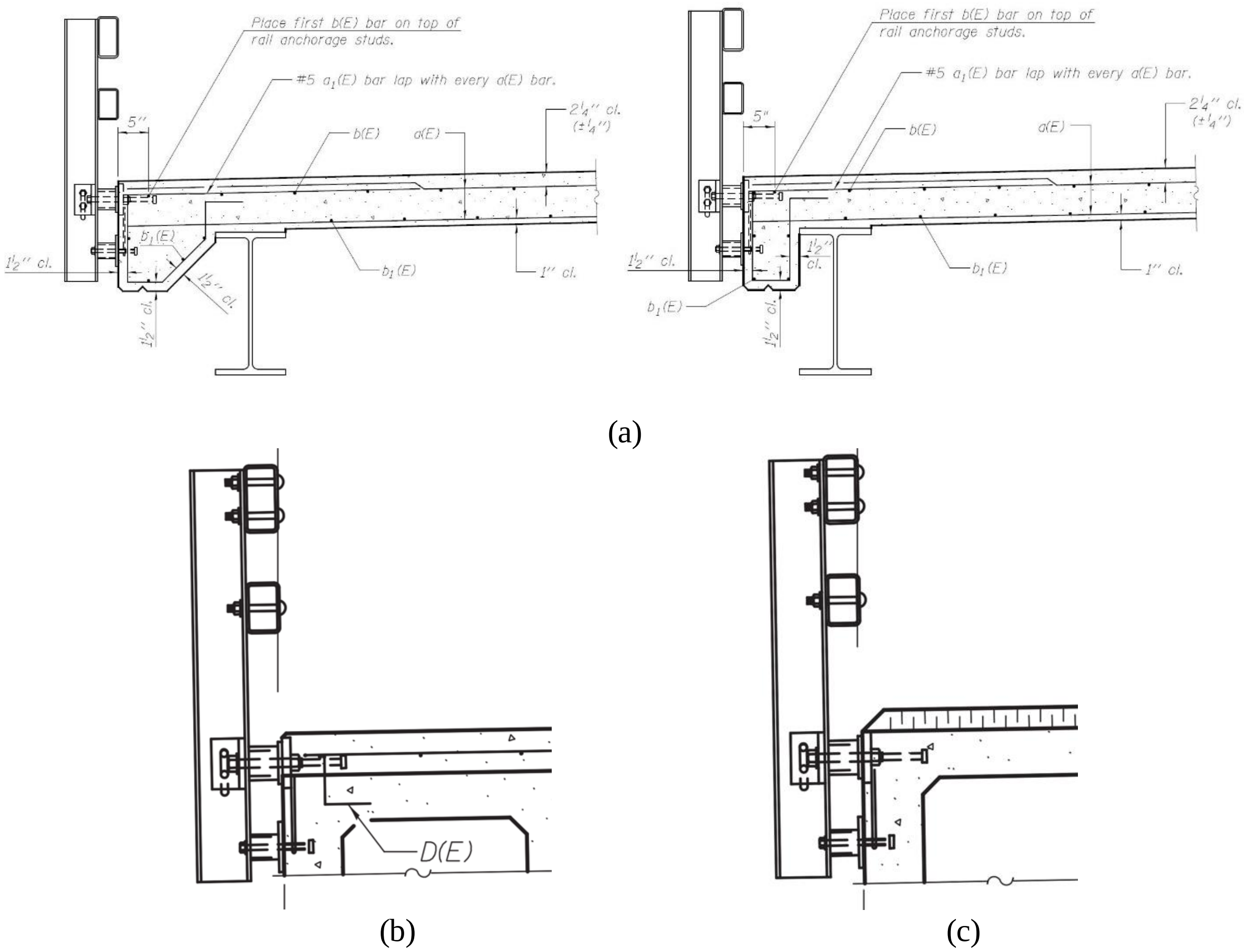


Figure 29. IDOT (a) Bridge Slab, (b) Box Girder with Concrete Wearing Surface, and (c) Box Girder with Asphalt Wearing Surface [2]

3.4.2 Ohio Bridge Deck Configuration

The Ohio bridge deck configurations were similar to the Illinois configurations, utilizing bridge slabs and pre-stressed box-beam girders. Ohio 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 in the box girders and not in the wearing surface. Anchorage types for bridge slabs and concrete box-beams for the state of Ohio are shown in Figure 30.

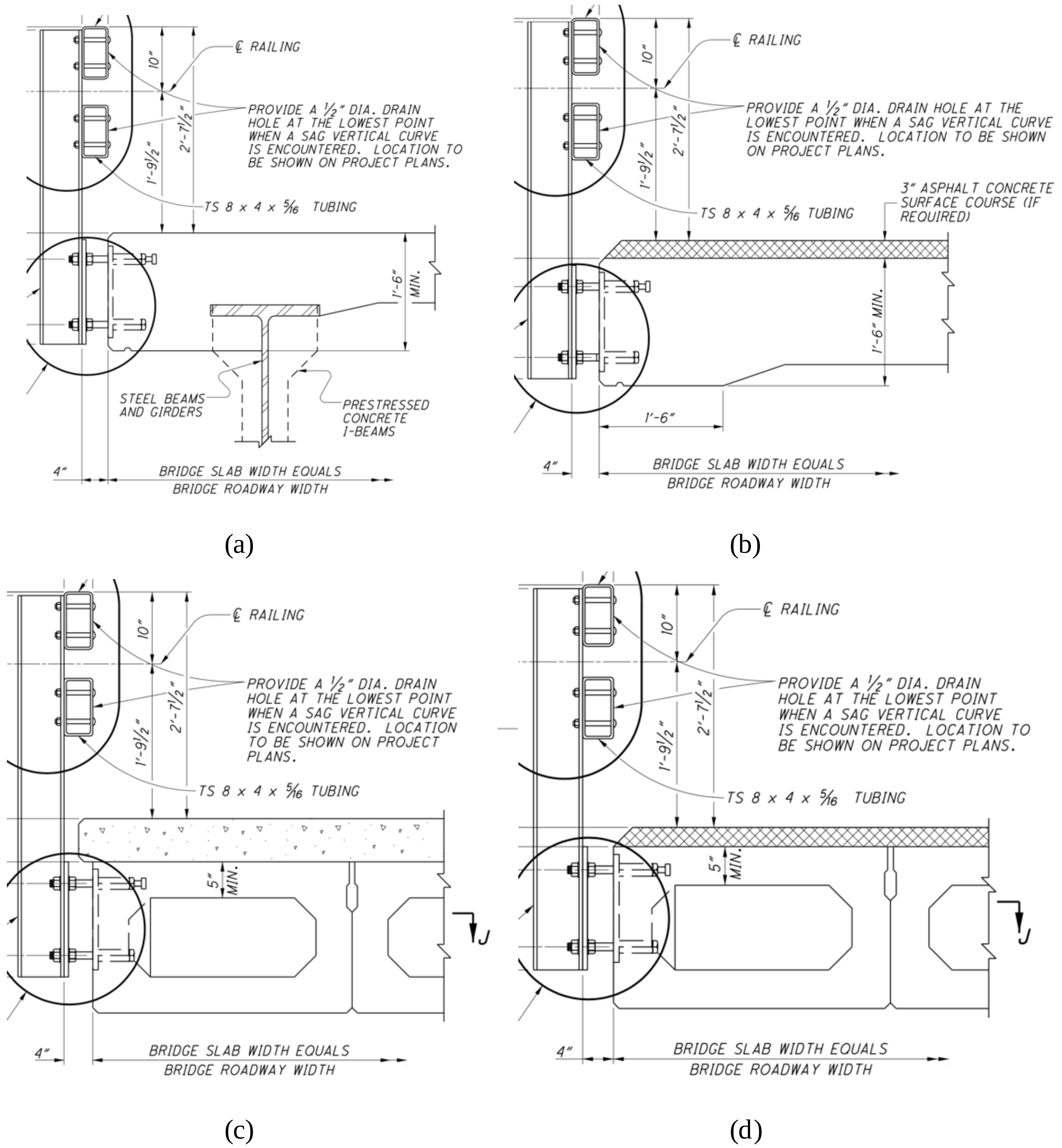


Figure 30. ODOT (a) Bridge Slab, (b) Bridge Slab with Asphalt Wearing Surface, (c) Box Girder with Concrete Wearing Surface, and (d) Box Girder with Asphalt Wearing Surface [1]

3.5 Preliminary Post Loads

An initial analysis was conducted on the capacity of the selected post shape during impact using the following assumptions: the W6x15 post would plastically deform during impact, and a Dynamic Magnification Factor (DMF) was applied for yield strengths that can be greater than the minimum specified static behavior of steel. A DMF is normally applied to the plastic section modulus of metal posts to estimate the dynamic yield force for a post, with a value of 1.5 typically assumed for W6x9 guardrail posts [5].

With impact loadings based on the plastic bending of the steel post, the plastic bending capacity of a steel post was determined by Equation 3.1.

$$M_u = \text{DMF} * F_y * Z_x \tag{3.1}$$

Where

M_u = Plastic bending capacity (kip − in.)
DMF = Dynamic magnification factor of 1.5
F_y = Yield stress of Steel Post, 50 ksi
Z_x = Post plastic section modulus (in.^{3}), 10.8 in.^{3}

The plastic bending capacity of the W6x15 steel post was determined to be 810 kip-in. (92 kN-m). Estimated anchor loads were then investigated on the basis of designing for the worst-case loading condition of all the deck configurations. An effective height of 30 in. (762 mm) above the deck surface was utilized for the applied impact load, as recommended in NCHRP Project 22-20 for a MASH TL-4 system [6].

The shortest distance from the impact height on the rail system to the tensile anchors would transmit the highest anchor loads into the bridge deck. Therefore, no wearing surfaces or overlays were considered for worst-case loading on the bridge deck. Based on IDOT and ODOT bridge deck standards, a 3-in. (76-mm) concrete cover for the tensile anchors and a 10-in. (254-mm) vertical anchor spacing were selected. Thus, with a post plastic bending capacity of 810 kip-in. (92 kN-m) and a distance of 33 in. (838 mm) from the top anchor to the impact loads, D1, an initial estimate of the impact force, F1, of 24.5 kips (109 kN) was expected to yield the post. An illustration of the anchor loads is shown in Figure 31.

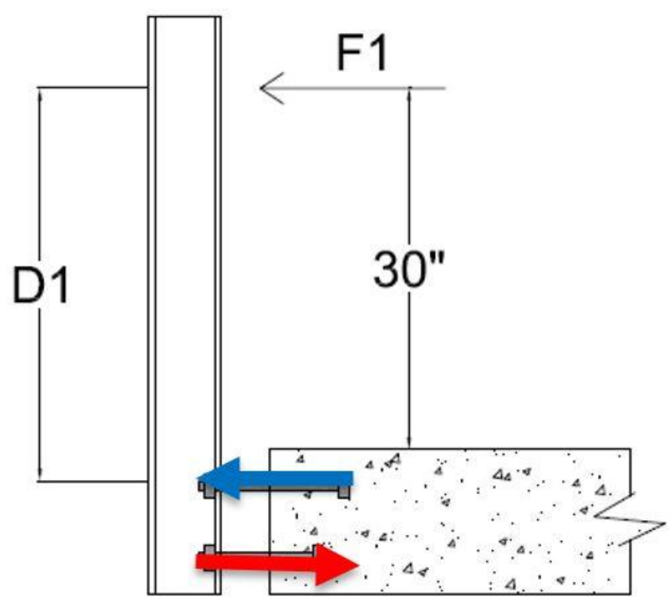


Figure 31. Deck Anchorage Loadings

3.6 Bridge Deck Anchorage Loadings

Preliminary anchor loads were investigated for all bridge deck configurations used by IDOT and ODOT in order to determine the worst-case loading. Initial estimates did not take into account actual concrete cover and reinforcement within the concrete slab surfacing, slab deck, or concrete box-beam girders. Four deck configurations were considered: (1) anchorage to a concrete deck slab, (2) anchorage in both a prestressed, concrete box-beam girder and the concrete wearing surface, (3) anchors only in the prestressed, concrete box-beam girder with a 5-in. (127-mm) to 6-in. (152-mm) concrete wearing surface, and (4) anchors only in the prestressed, concrete box-beam girder with a 2-in. (51-mm) to 3-in. (76-mm) asphalt wearing surface, as shown in Figure 32. Additionally, all four deck configurations could have a future 3-in. (76-mm) maximum overlay.

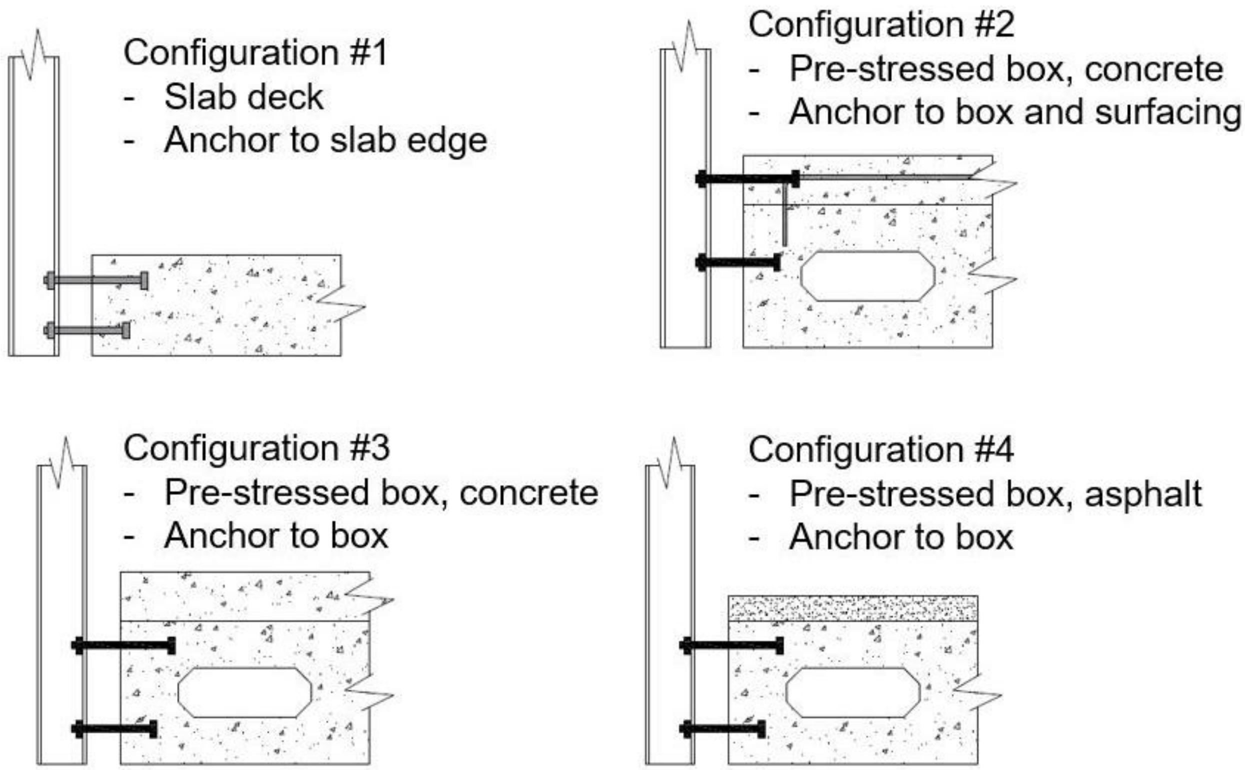


Figure 32. Summary of IDOT and ODOT Bridge Decks

It was anticipated that the anchorage loading strength would vary depending if the posts were attached to the slab deck, the box-beam structure, or the concrete wearing surface on top of the concrete box-beam girder. Also, different vertical anchorage locations would create different post lengths and possibly different redirective forces (a longer moment arm will likely result in lower forces necessary to bend the post). Thus, anchorage hardware had to be developed for the attachment to four different deck and wearing surface combinations.

For analysis of post strength and deck loads, a plastic hinge was assumed to form at the tensile anchor rods and the applied dynamic force was assumed to be located at a variable distance above the top anchor rods depending on the bridge deck configuration. In Configuration #1, shown in Figure 32, the 30-in. (762-mm) effective height above the top of the slab deck with an assumed

3-in. (76-mm) concrete cover positioned the applied dynamic force 33 in. (838-mm) above the tensile anchor rods. Therein, equilibrium equations determined the tension and compression forces transferred into the deck. A similar process was performed for the remaining deck configurations, with Configurations #2 and #3 utilizing a 6-in. (152-mm) concrete wearing surface and Configuration #4 featuring a 3-in. (76-mm) asphalt wearing surface.

The vertical anchor spacing was initially taken from similar side-mounted bridge rails investigated in the literature review. With the slab deck ranging from a 12-in. (305-mm) minimum deck thickness to a maximum 18-in. (457-mm) thickness, the anchor spacing was set at 5 in. (127 mm) to 10 in. (254 mm), respectively. For the remaining deck configurations, a 10-in. (254-mm) anchor spacing was utilized as this anchor spacing was used in bridge drawings by IDOT and ODOT [1-2]. The steel post was assumed to be a cantilever beam with the impact force applied 30 in. (762 mm) from the deck surface, with reactions at the location of the tensile and compression anchors. A typical free-body diagram used to determine preliminary anchor loads is shown in Figure 33 with preliminary anchor loads based on the deck configurations shown in Table 3.

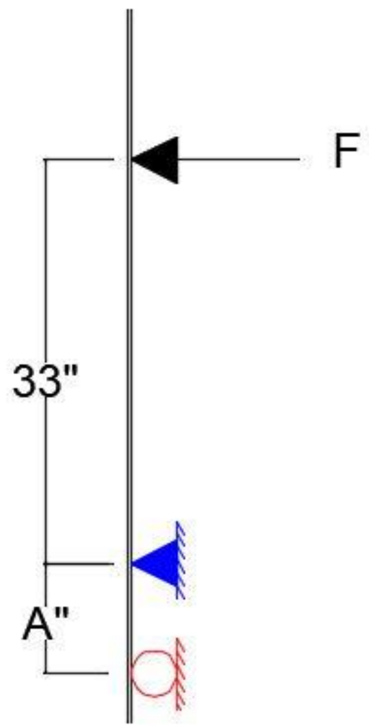


Figure 33. Free-Body Diagram for Determining Anchor Loads in Slab Decks

Table 3. Preliminary Anchor Loadings

Deck and Anchor Configuration	Moment Arm from Impact Load to Top Anchor, in. [mm]	Impact Load, Kips [kN]	Anchor Spacing, in. [mm]	Tension Loading (Top Anchors), Kips [kN]	Compression Loading (Bottom Anchors), Kips [kN]
12-in. Slab Deck	33 [838]	24.5 [109]	5 [127]	186.2 [828]	161.7 [719]
18-in. Slab Deck	33 [838]	24.5 [109]	10 [254]	105.4 [469]	80.9 [360]
Box-beam Girder & Concrete (#2) - Top Anchors in Concrete Surfacing	33 [838]	24.5 [109]	10 [254]	105.4 [469]	80.9 [360]
Box-beam Girder & Concrete (#3) - Anchors only in Box Girder	39 [991]	20.8 [93]	10 [254]	101.9 [453]	81.1 [361]
Box-beam Girder & Asphalt (#4) - Anchors only in Box Girder	36 [914]	22.5 [100]	10 [254]	103.5 [460]	81.0 [360]

3.6.1 Evaluation of Deck Configurations

The four deck configurations were further reviewed to determine if they were compatible with the embedded anchorages. In particular, there were concerns with Configuration #2, shown previously in Figure 32. This deck configuration featured a 5-in. (127-mm) or 6-in. (152-mm) concrete wearing surface on the concrete box-beam girder with a 2½-in. (64-mm) concrete clear cover to the No. 4 reinforcement placed both laterally and longitudinally. Assuming 1¼-in. (32-mm) diameter top anchor rods with coupling nuts were installed in the concrete wearing surface below the reinforcing steel mat, the 5-in. (127-mm) slab would have a maximum clear cover of ¼ in. (6.4 mm) to the bottom of the slab/top of box-beam girder. Similarly, the 6-in. (152-mm) slab would have an increased clear cover of 1¼ in. (32 mm).

For this deck configuration, the minimal bottom clear cover between the tension anchor and concrete wearing surface posed risks for reduced strength and an increased risk of anchor pullout. Options to remedy the concerns were to either increase the concrete wearing surface thickness or eliminate anchorage into the concrete wearing surface. The sponsors opted to eliminate Configuration #2 as an option for the new bridge rail. Therefore, only deck configurations #1, #3, and #4 were considered for post-to-deck attachment designs.

The preliminary anchor loads were further refined to estimate critical loads transferred into the deck by considering reinforcement patterns, anchor spacing, and concrete cover in all deck configurations, with 1-in. (25-mm) diameter anchor rods. Critical design loads were calculated for the minimum 12-in. (305-mm) thick slab deck, an 18-in. (457-mm) thick slab deck, and a 17-in. (432-mm) deep box-beam girder. A 33-in. (838-mm) deep box-beam girder was also considered to show critical design loads transferred into a box-beam girder of greater depth. However, the sponsors preferred to utilize a single anchorage design. Thus, the ability to utilize the greater girder depth to reduce the anchor loads was eliminated. It shall be noted that to ensure the top anchors were placed under the top lateral and longitudinal reinforcement within the bridge deck, a 4-in. (102-mm) concrete cover and a 3-in. (76-mm) concrete cover was used for the slab decks and concrete box-beam girders, respectively.

As preferred by both DOTs, the anchor rods were placed between the top and bottom lateral and longitudinal reinforcement in the slab decks and below the reinforcement placed in the top of the box-beam girder. To take advantage of the depth of the bridge deck and in order for the anchors to be placed between the steel reinforcement, the tensile and compression anchors were placed at a maximum spacing of 6 in. (152 mm) for the 12-in. (305-mm) thick slab deck. Similarly, the anchors were spaced 11 in. (280 mm) apart for both the 18-in. (457-mm) thick slab deck or 17-in. (432-mm) box-beam girder, and 27 in. (685 mm) for the 33-in. (838-mm) box-beam girder. A summary of the critical design loads for a 12-in. (305-mm) slab deck, an 18-in. (457-mm) slab deck, a 17-in. (432-mm) deep box-beam girder, and a 33-in. (838-mm) deep box-beam girder are shown in Table 4 and Figure 34.

Table 4. Critical Design Loadings for Anchorages

Critical Design Loads	**12-in. Slab**	**18-in. Slab**	**17-in. Box-beam Girder (Configuration#4)**	**33-in. Box-beam Girder (Configuration#4)**
Moment Arm from Impact Load to Top Anchor, in. [mm]	34 [864]	34 [864]	36 [914]	36 [914]
Impact Load, kips [kN]	23.8 [106]	23.8 [106]	22.5 [100]	22.5 [100]
Tension, kips [kN]	158.7 [706]	97.4 [433]	96.1 [427]	52.5 [234]
Compression, kips [kN]	134.9 [600]	73.6 [327]	73.6 [327]	30.0 [133]
Anchor Spacing, in. [mm]	6 [152]	11 [279]	11 [279]	27 [685]
Concrete Cover, in. [mm]	4 [102]	4 [102]	3 [76]	3 [76]

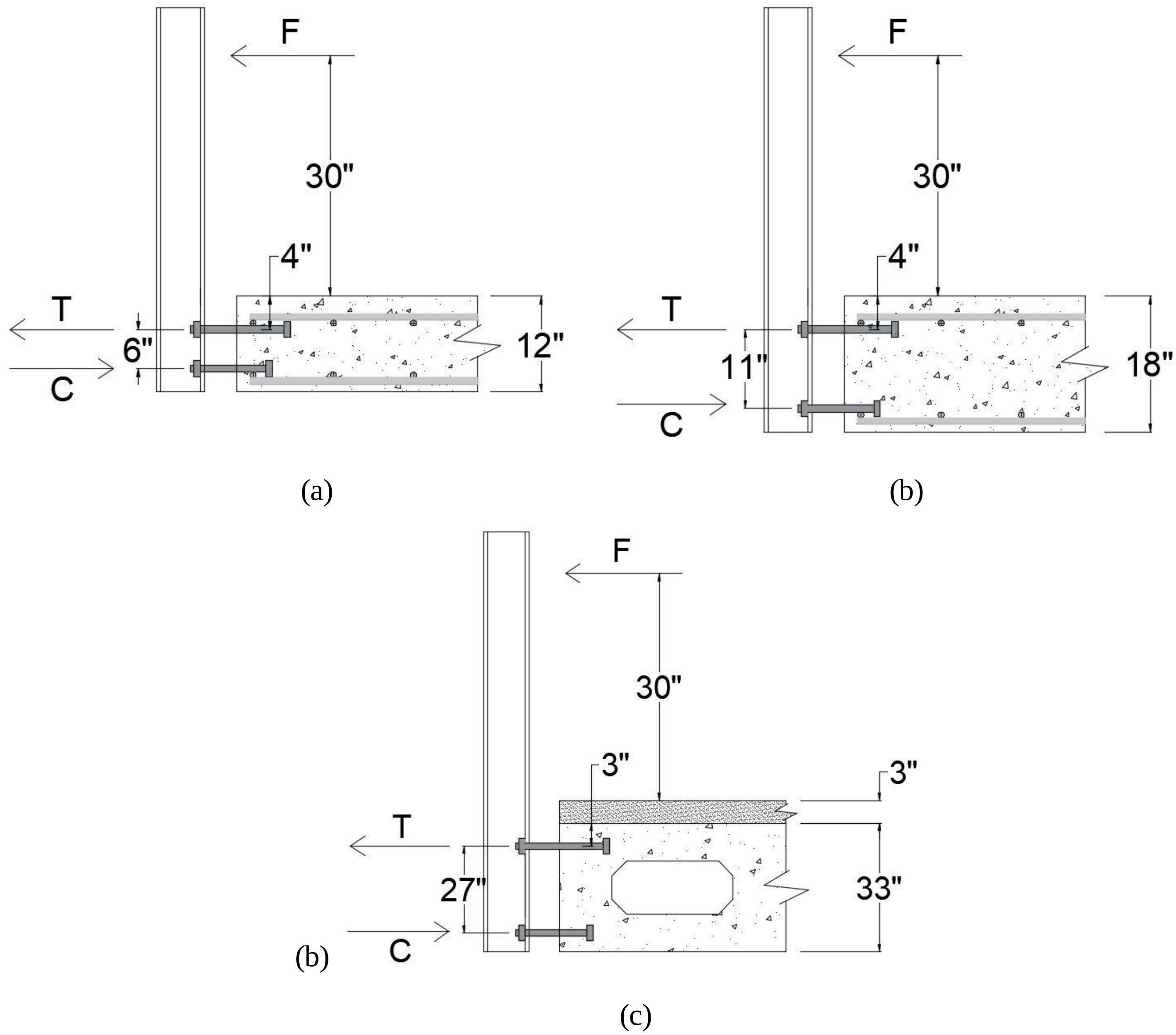


Figure 34. (a) 12-in. (305-mm) Slab Deck Design, (b) 18-in. (457-mm) Slab Deck Design, and (c) 33-in. (838-mm) Deep Concrete Box-Beam Girder Design

Concerns were expressed with the high anchor loads in the 12-in. (305-mm) slab design, such as requiring anchor diameters greater than 1 in. (25 mm). Although there are box-beam girders and slab decks 12 in. (305 mm) in depth, IDOT box-beam girders at a 12-in. (305-mm) depth are not adequate in depth to anchor a side-mount bridge railing, according to IDOT bridge drawings [2]. Thus, the side-mounted bridge rail requires a minimum deck depth of 18 in. (457 mm) and 17 in. (432 mm) to anchor to the slab deck and concrete box-beam girder, respectively. An advantage of the 18-in. (457-mm) slab deck/17-in. (432-mm) box-beam deck design is its ability to produce lower anchor loadings by benefiting from the greater bridge deck depth due to extending the anchor spacing to 11 in. (279 mm).

Since the 12-in. (305-mm) thick deck had much higher estimated anchor loads, that deck configuration was eliminated. Thus, the 18-in. (457-mm) slab deck/17-in. (432-mm) box-beam deck design was the minimum deck depth for the design of the post anchorage and the post-to-deck attachment hardware.

3.7 Deck Anchorage Concepts

Deck anchorage concepts were explored for anchoring the new side-mounted bridge rail to IDOT and ODOT bridge decks. Current deck anchorage features headed welded studs on an embedded plate with bolt sleeve inserts, as shown in Figures 25 and 27. The headed welded studs extended approximately 4¾ in. (121 mm) into the deck, which could result in concrete breakout during impact events due to shallow embedment and the use of butt-welded studs that are not ideal for tension anchoring. Improvements could be made to the current anchorage design, including lengthening the welded studs to a length greater than 10 in. (254 mm) and adding more studs to the embedded plate.

Other options were also investigated. One concept involved U-shaped rebar with flare bevel welds, as shown in Figure 35. This concept would provide greater bond capacity at a deeper development length and the flare bevel welds would be stronger in tension than butt welds.

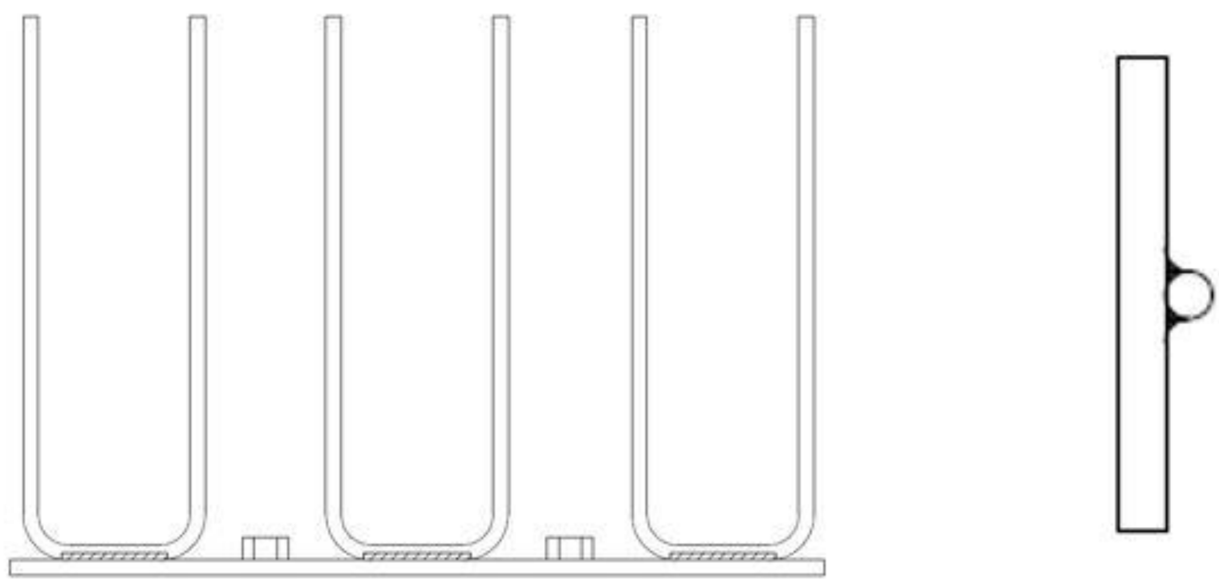


Figure 35. U-Shaped Rebar Anchorage

Structural shapes and built-up sections cast within the bridge deck were also considered as part of the anchorage device. The concepts proposed were an embedded T-section plates with gussets or a base plate with vertical inner plates, with both featuring rebar flare bevel welded onto the structural shape, as shown in Figure 36.

(a) T-Section with gussets

(b) Base plate with vertical inner plates

Figure 36. (a) T-Section Built-up Shape, or (b) Structural Base Plate Anchorage Devices

Anchorage devices utilizing threaded rods can be used in bridge deck anchorages. With the use of an embedded plate at the edge of the deck, coupling nuts, and threaded rods, as shown in Figure 37, this type of anchorage device is ideal to transfer tensile loads to the anchors. Based on the preference of the sponsors and the researchers' prior experience, the embedded plate with threaded rods and coupling nuts was selected for the deck anchorage.

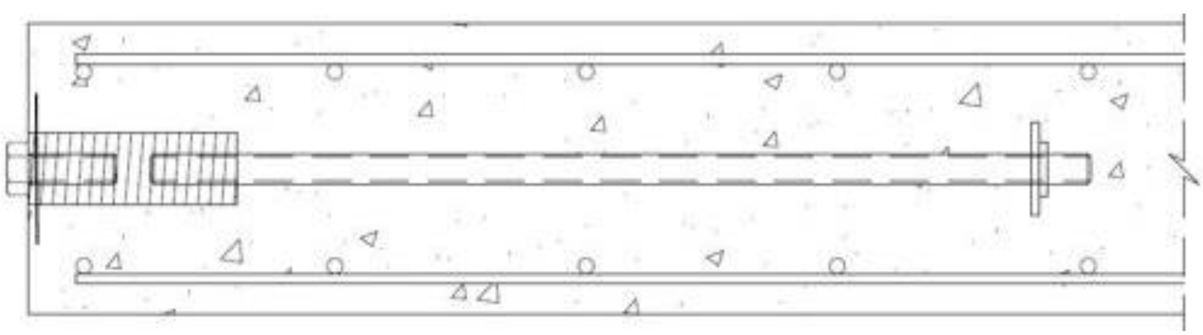


Figure 37. Threaded Anchor Rod Device

For the bottom compression anchors, an anchorage was desired that reduced the number of parts currently used in the anchorage devices by IDOT and ODOT, as shown in Figures 25 and 27, while fitting within the 5½-in. (140-mm) thick sidewalls of the concrete box-beam girders. Therefore, use of 3-in. (76-mm) long shear studs with heavy hex nuts welded to the inside of the embedded plate at the edge of the deck was considered for the bottom anchorage, as shown in Figure 38.

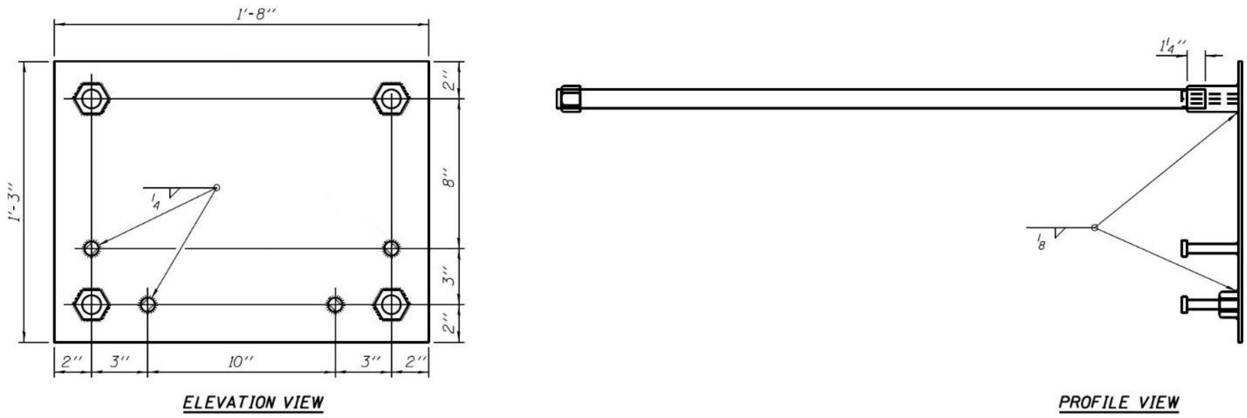
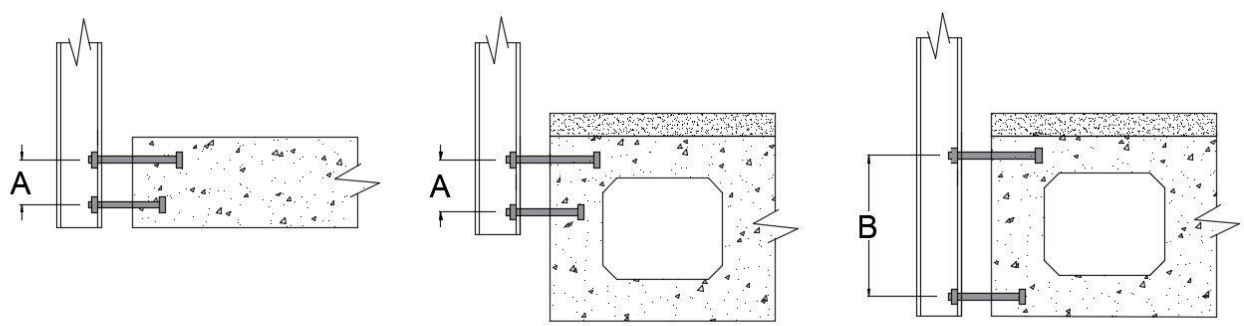


Figure 38. Bottom Anchorage

3.7.1 Vertical Anchor Spacing

A single anchorage design could be used for all bridge decks from 17 in. (432 mm) to 42 in. (1,067 mm) deep, as shown in Figure 39a, or a variable height anchorage could be used to lower anchor loads and anchor in the bottom layer of the concrete box-beam girder, as shown in Figure 39b. Dimensions A and B dictate either using a tighter vertical anchorage spacing for all bridge decks or using a wider spacing, respectively.



(a) Single Anchor Height (b) Variable Height

Figure 39. Singular Anchorage and a Variable Height Anchorage

A single design offered familiarity and consistency in design with all deck configurations, having the same anchor location on the bridge deck. A variable height provided the opportunity to benefit from the longer post and box-beam girder depth, thereby lowering the anchor loads with the greater distance between the anchors. Two anchorage layouts would exist with the variable height design: one design anchoring into slab decks and one design into the box-beam girders. The variable anchorage design layout for box-beam girders would allow the lower rods to anchor into the bottom layer of the box-beam girders, which would reduce the potential for punching shear failure by not anchoring into the 5½-in. (140-mm) thick sidewalls of the box-beam girders. However, prestressing strands may also be present at the bottom layers of the box-beam girders, which would interfere with the anchors.

Ultimately, a singular anchorage design was selected for all bridge deck configurations due to several factors: (1) design consistency, which would help mitigate construction errors with anchorage placement; (2) to keep the anchors farther from the prestressing strands; (3) a one-size-fits-all design would reduce the number of unique posts to stock in inventory or the varied concrete box-beam girder depths. Therefore, the vertical anchor spacing between the upper and lower anchorages was established at 11 in. (279 mm).

3.7.2 Longitudinal Anchor Spacing

The anchorage spacing in the bridge decks was configured to be 16 in. (406 mm) apart. This longitudinal distance was to provide the full development of the tensile forces required for the anchor rods embedded in the deck. Concrete breakout strengths are reduced with narrower spacing. A 16-in. (406-mm) spacing would distribute the anchor loads across more concrete and stirrups and provide a greater resistance to punching shear on the box-beam girder sidewalls. Therefore, all post-to-deck attachments utilized a 16-in. (406-mm) longitudinal spacing for the anchors in the bridge deck.

3.7.3 Anchor Rod Size and Embedment

Anchor rod diameters were dependent on the critical impact loads transferred to the deck for the minimum 18-in. (457-mm) thick slab deck and 17-in. (432-mm) deep concrete box-beam girder. For the minimum deck depth, the anchor rods were needed to resist total critical design loads of 97.4 kips (433 kN), as previously mentioned. It was noted in the literature review of post-to-deck connections that ASTM F1554 Grade 105 was a common material specification for bridge rail anchorages. Therefore, two ASTM F1554 Grade 105 anchor rods with a minimum 1-in. (25-mm) diameter were determined to be necessary to resist the tensile loads. IDOT and ODOT elected to proceed with the two 1-in. (25-mm) diameter anchor rods located 3-in. (76-mm) from the deck surface to provide adequate concrete clear cover when anchoring into the top 5½-in. (140-mm) layer of the concrete box-beam girders, as previously mentioned.

The anchor rod embedment was determined by assuming headed bars for the anchorage. The DOTs selected threaded rods with coupling nuts as the preferred anchorage, as shown in Figure 37. This type of anchorage would utilize a washer or bolt nut at the end of the rod which would increase the concrete breakout strength of the rods. An embedment length of 34½ in. (876 mm) was determined for two 1-in. (25-mm) diameter anchor rods to best utilize the width of the narrower 36-in. (914-mm) wide concrete box-beam girder to meet anchorage capacity and reduce the propensity for concrete breakout. Sample anchor rod sizing and embedment length calculations are shown in Appendix A.

3.8 Final Anchorage Design

After several brainstorming sessions, IDOT and ODOT elected for a singular anchorage design for all bridge decks; no protrusions from the deck side wall (i.e., the anchorage hardware should be flush with the deck edge), and anchors should be installed away from prestressing strands. The threaded anchor rods with coupling nuts were to be used in the final anchorage design, as shown in Figure 40.

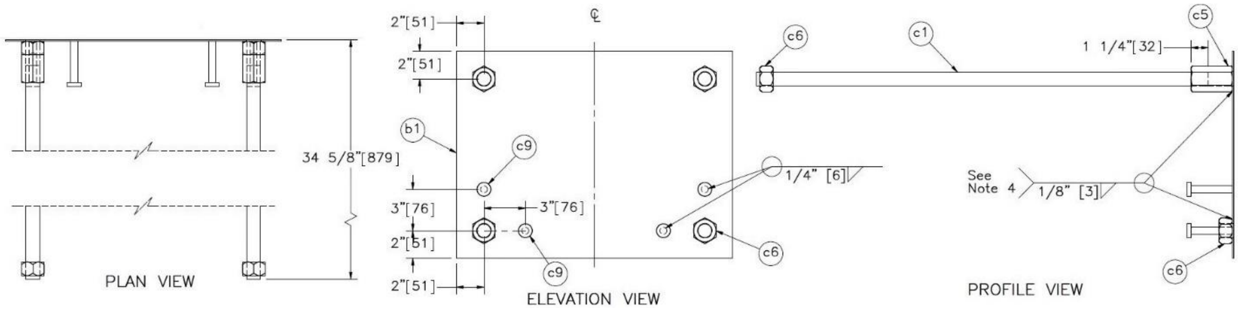


Figure 40. Final Deck Anchorage Design

The anchorage design utilized two upper F1554 Grade 105 anchor rods with coupling nuts fillet welded to the ⅛-in. (3.2-mm) embedded plate. Four ½-in. (13-mm) diameter by 3-in. (76-mm) long shear studs and two lower heavy hex nuts were also welded to the plate. The DOTs preferred typical anchorage to utilize the 36-in. (914-mm) width of the concrete box-beam girder. Therefore, the 32½-in. (836-mm) length was considered the nominal anchor rod length. The tensile anchor rods would be longitudinally spaced 16 in. (406 mm) at each post location and vertically spaced 11 in. (279 mm) to the lower two anchor bolts.

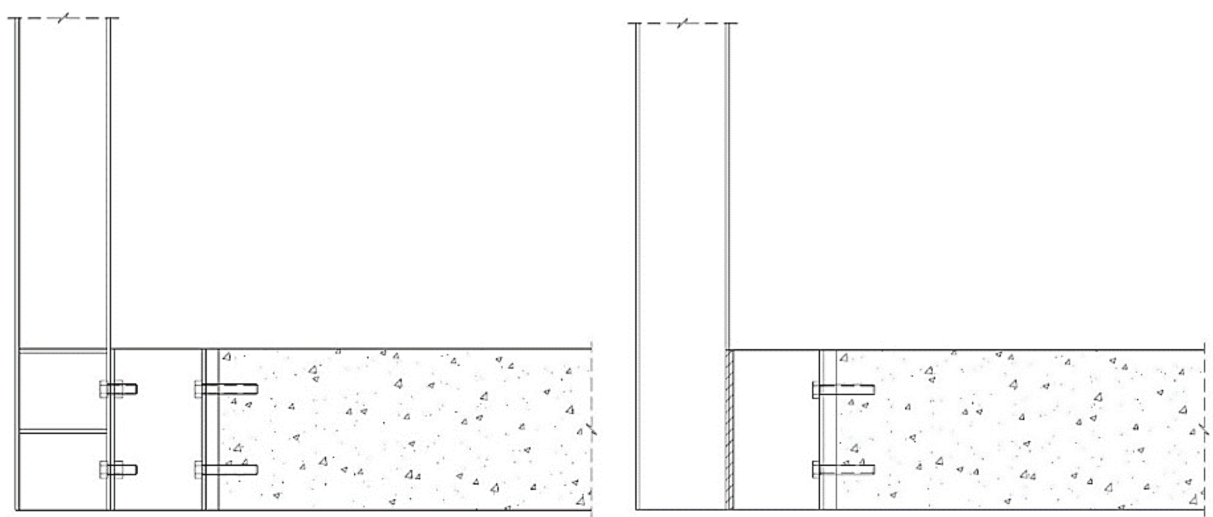
4 POST-TO-DECK ATTACHMENT HARDWARE DESIGN

Prior to this research study, both IDOT and ODOT utilized independent TL-4 bridge rail designs. Over the past decades, both DOTs have used side-mounted steel beam-and-post bridge rails without a curb to allow proper runoff from the bridge deck. However, IDOT and ODOT expressed interest in combining their existing designs for a new MASH 2016 TL-4 bridge rail. Existing post-to-deck connection designs were reviewed in detail. Estimations of the impact loads transferred into the various deck configurations were analyzed, and post-to-deck attachment and anchorage concepts were developed in brainstorming sessions.

4.1 Post-to-Deck Attachment Design

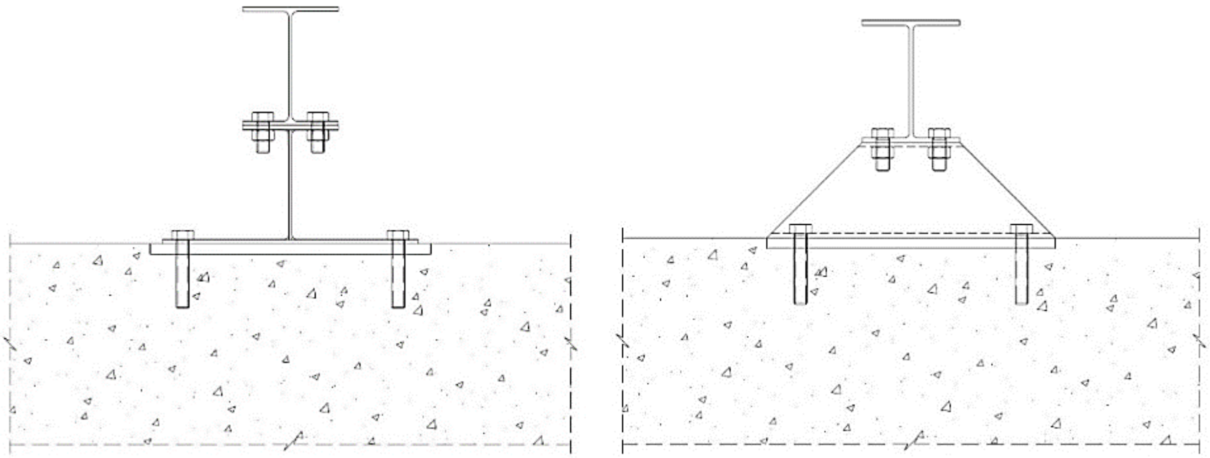
Post-to-deck attachment concepts were explored for side-mounting the W6x15 posts to the bridge deck. Existing hardware attachments feature the Illinois double angle with spacer tube tensile connection and the Ohio base plate with anchor bolts, as shown in Figures 24 and 26. Concerns with existing DOT attachment concepts included: (1) the Illinois attachment utilizing a the spacer tube in the tension connection is spot welded to the plate on the bridge deck, which would not transfer load for most impacts; and (2) both existing attachments have anchor bolts that span over a 4-in. (102-mm) offset from the front face of the post to the bridge deck, which could include bending in the bolts and lead to premature fracture.

Post assembly and spacer tube options were considered for the new post-to-deck attachment design. Note, the deck anchorage utilized would be as developed in Chapter 3, but are shown generically herein. A steel spacer could be welded to the post assembly or be composed of independent steel pieces, as shown in Figure 41. The spacer for either of the options could be a built-up I-section or a hollow steel section tube, as shown in Figure 42.



(a) (b)

Figure 41. Post Spacer Comprising: (a) Independent Pieces, or (b) Welded Post Assembly



(a) (b)

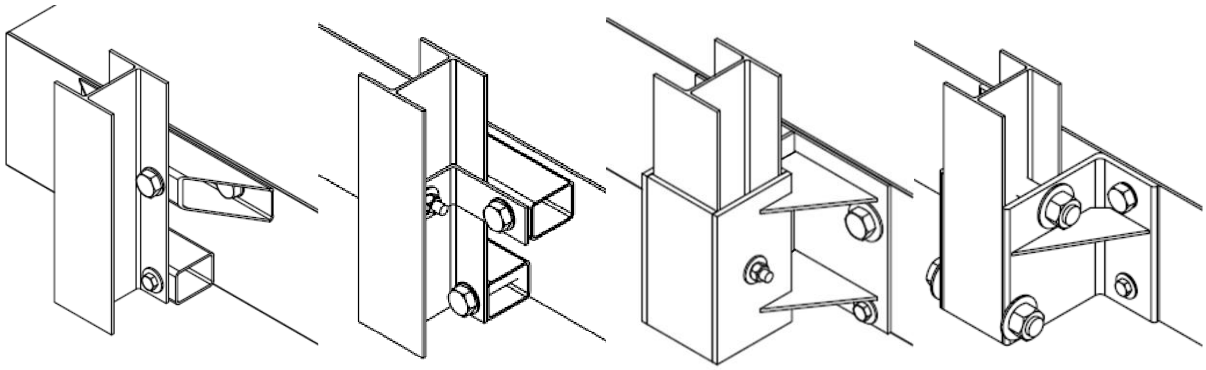
Figure 42. Structural Spacer as a: (a) Built-up I-section Spacer, or (b) Longitudinal Tube Spacer

4.1.1 Deck Attachment Concepts

Several post-to-deck attachment concepts were investigated that utilized either an independent spacer assembly or an integral post and spacer assembly. Therefore, the attachment design concepts featured two groups: (A) an independent spacer assembly, or bolted post attachments and (B) integral posts and spacers, or welded post assemblies. Group A attachments typically bolted through either the post flange or the web and the spacer. Group B attachments featured various welded post and spacer assemblies. All deck attachment concepts utilized the threaded tensile anchor rods with coupling nuts and shear studs welded to an embedded plate as the anchorage design in the bridge deck.

4.1.1.1 Group A – Bolted Post Attachment Concepts

Independent spacer options included longitudinal tubes, a socket assembly, or a double angle – shear bolt assembly, as shown in Figure 43. A design concept utilizing the existing Illinois double angle connection was provided with longitudinal tubes to help spread compression loading across the side of the bridge deck. Group A concepts had several disadvantages, including: the potential decrease in post strength from bolting through the flanges of the W6x15 post; potential higher loads the anchorage bolts due to the eccentric combined loading from the 4-in. (102-mm) offset between the post and the deck; having a large, heavy socket assembly; and, possible web bearing failure in the angle and shear bolt concept.



(a) (b) (c) (d)

Figure 43. Group A Concepts: (a) Bolted Flange, (b) Double Angle Connection, (c) Socket Assembly, (d) Angle and Shear Bolt Concepts

4.1.1.2 Group B – Welded Post Assembly Attachment Concepts

Group B deck spacers utilized longitudinal tubes, a welded plate and spacer block, or welded plates with shear bolts as the spacer attachment. The Welded Post-Tube Assembly concept was similar to its Group A counterpart in that it was bolted to the bridge deck, but was welded onto the post front flange, providing the option to either have the longitudinal tube spacers as a bolted or welded assembly. The impact loads are distributed along the bridge deck by using the longitudinal tube as a structural spacer and the post and spacer are one piece, as shown in Figure 44. Disadvantages of the assembly are it may be a heavy post assembly and the bolts span over the 4-in. (102-mm) tube spacer width, where the bolts may be susceptible to premature failure due to additional bending loads.

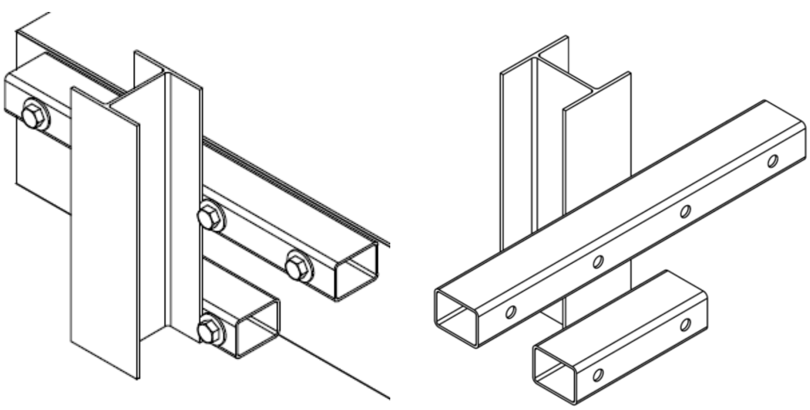


Figure 44. Welded Post-Tube Assembly Concept

Another variation of the Group B welded post assembly concepts was the Welded Plate and Spacer Block, which comprised a plate welded to the post's front flange and a plate and tube spacer block, as shown in Figure 45. This concept was considered to be a strong, stiff attachment. Disadvantages of the concept were having two fabricated assemblies in the welded plate, the post and spacer, and having multiple fasteners.

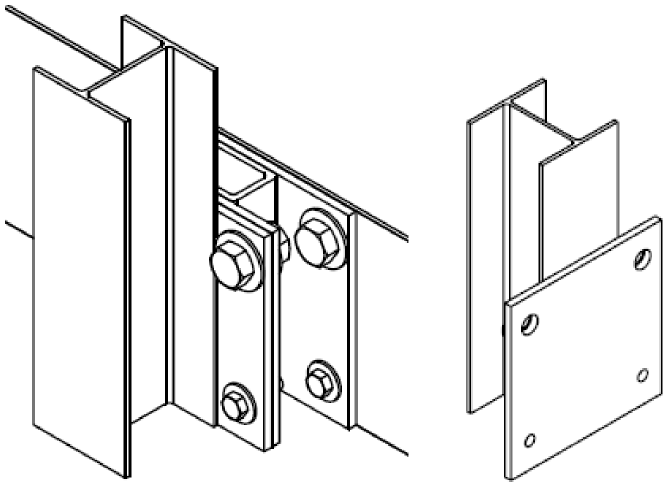
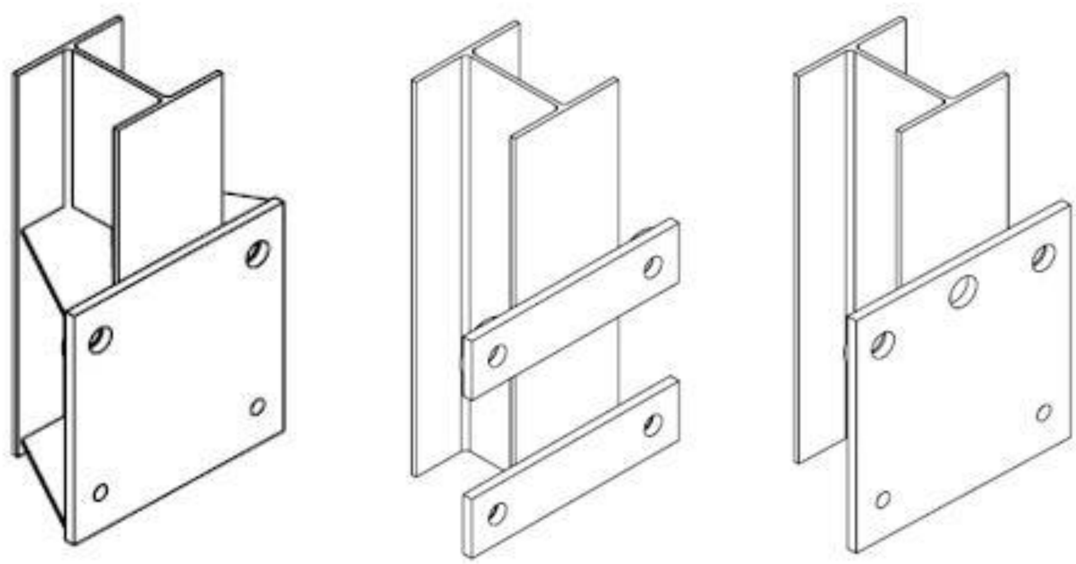


Figure 45. Welded Plate and Spacer Block Concept

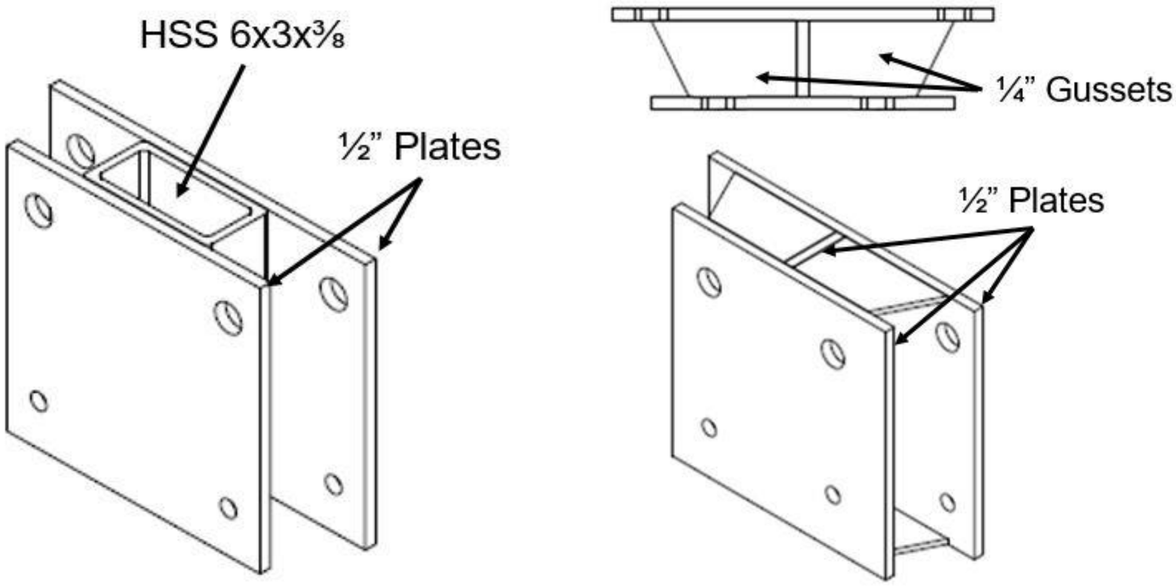
Multiple post assembly variations were possible for the Welded Plate and Spacer Block Concept, as shown in Figure 46. Considerations were made to strengthen the single welded plate in the event that the weld strength along the entire height of the plate and the front flange of the post could not meet capacity. Post assembly options consisted of a gusset to transfer load across the post-to-deck attachment and to better stiffen the post web to prevent the web from buckling during impact. A second option allowed the welded plate to be replaced by two smaller sized mounting plates to reduce material, fabrication costs, and overall weight. The mounting plates could also be gusseted to the post web and flanges. Finally, a third option provided a plug weld in the event that additional tensile strength was necessary in the top tensile spacer connection during impact.



(a) (b) (c)

Figure 46. Welded Plate and Spacer Block Post Assembly Concepts: (a) Plate Attachment with Gussets, (b) Two Mounting Plates, and (c) Singular Plate Attachment

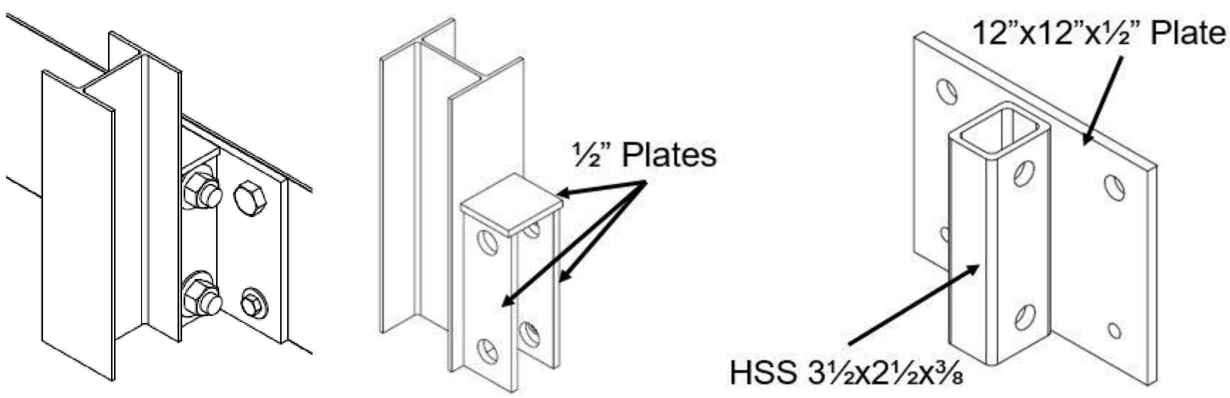
Spacer options for the Welded Plate and Spacer Block attachment concept showcased plates and a tube attachment or a fabricated I-section comprising plates with gussets. Both structural spacers were to be welded spacer assemblies, as shown in Figure 47.



(a) (b)

Figure 47. Welded Plate and Spacer Block Spacer Options: (a) Plates and Tube, and (b) Fabricated I-section with Gussets

The Welded Plates and Shear Bolts concept featured plates welded to the front flange of the post as a post assembly and a tube welded to a plate as the spacer assembly, as shown in Figure 48. The attachment concept would have a post assembly, spacer assembly, and only two bolts to connect the assemblies. Disadvantages were having two fabricated assemblies (post and spacer), the required shear bolts may be large in size, and the two assemblies would be relatively loose to have installation tolerances.



(a) (b)

Figure 48. Welded Plates and Shear Bolts Concepts: (a) Post Assembly and (b) Spacer Assembly

4.1.2 Deck Attachment Preference

IDOT and ODOT elected to proceed with variations of the Welded Plate and Spacer Block and the existing Illinois double angle connection deck attachment concepts. With the selection of the deck attachments and the previously mentioned 17-in. (432-mm) deck design, preference was also made for a maximum of 1-in. (25-mm) diameter anchor rods and for using two tensile anchor rods in the anchorage design. Other preferences for the deck attachment hardware were: (1) using the two-mounting plate post assembly as shown in Figure 46, but with no gussets on the post; (2) the ability to provide vertical adjustment on either the post or deck side of the deck attachment; and (3) using HSS longitudinal tubes as the structural spacer.

The Welded Plate and Spacer Block concept and the existing Illinois double angle attachment were considered due to their flexibility and familiarity in design. In both designs, the spacer can be bolted to the deck followed by the corresponding post assembly or post and double angles connecting into the spacer itself. Both deck attachments also offer the ability to provide a 4- to 6-in. (102- to 152-mm) lateral post offset from the edge of the deck, which would equal the depth of the selected tube railings so that the front face of the tube rails could be flush with the edge of the bridge deck. In the system's bridge railing design, the DOTs selected an HSS12x4x¼ for the top railing and HSS8x6x¼ tube railings for the middle and lower tubes, therefore providing a 6-in. (152-mm) lateral offset of the post to the deck. Thus, all deck attachment hardware was designed to provide the 6-in. (152-mm) post-to-deck offset.

For the Welded Plate and Spacer Block, a strength analysis was performed on the post assembly. The two-plate welded post assembly was considered be a more critical design due to having no additional strength to the post web or plates with gussets. Therefore, the two-plate welded post assembly was designed to meet the tensile loads of 96 kips (427 kN) expected to be transferred to the deck. A structural analysis of the HSS longitudinal tube spacer was also performed. The tube webs were analyzed to resist the tension and compression loads of 97.4 kips (433 kN) and 73.6 kips (327 kN), respectively. Furthermore, the bending capacity of the tube to resist eccentric vertical load induced from the SUT weight transferred through the post to the deck attachment was also investigated. This vertical load was taken as the applied vertical design load of 33 kips (148 kN) for MASH TL-4 rail systems applied over 18 ft (5.5 m) with the assumption that the vertical load was distributed evenly over four posts [6]. Therefore, a vertical load of 8.25 kips (2.5 kN) was assumed to be transferred down each post, causing eccentric loading of the longitudinal tube spacers. Preliminary calculations of the post assembly and longitudinal tubes are shown in Appendix A.

The post-to-plate assembly required a ¼-in. (6.4-mm) fillet weld across the 6-in. (152-mm) post flange along the top and bottom of each plate and vertically along the flange edges to develop required weld capacity. To meet bending capacity of the plate, the required plate thickness was 1¼ in. (32 mm) without gussets. For the double angle connection, the bending capacity of the angles required a 1-in. (25-mm) thick angle. IDOT and ODOT elected to use a 6-in. x 4-in. x ⅞-in. (152-mm x 102-mm x 22-mm) angle, which was the thickest standard angle shape. Thus, the angles may plastically deform during a severe impact event. The double angles would require two ¾-in. (19-mm) bolts to connect the angles to the post web.

It was determined that a minimum ⅜-in. (9.5-mm) thickness was required for utilizing a longitudinal tube as a structural spacer, thus an HSS6x4x⅜ tube spacer was selected in order to meet design loads and provide the 6-in. (152-mm) post-to-deck lateral offset. Finally, the tube was designed to have extended ends along the deck side for better load transfer along the spacer tube and to help prevent bowing of the HSS sidewalls, as shown in Figure 49.

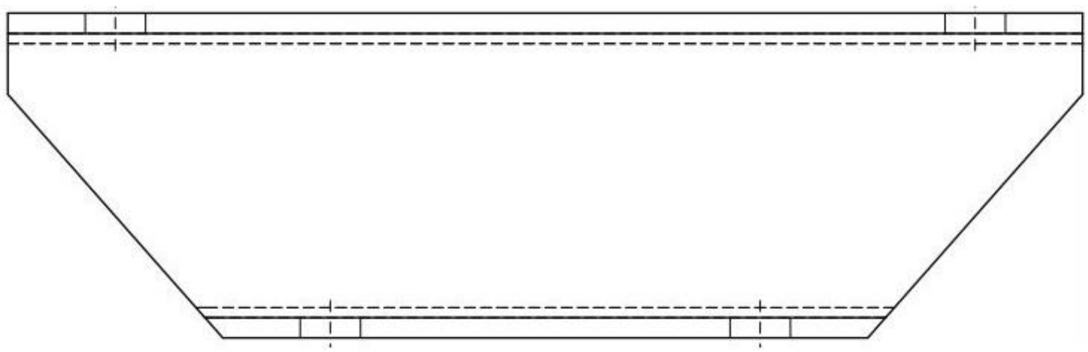
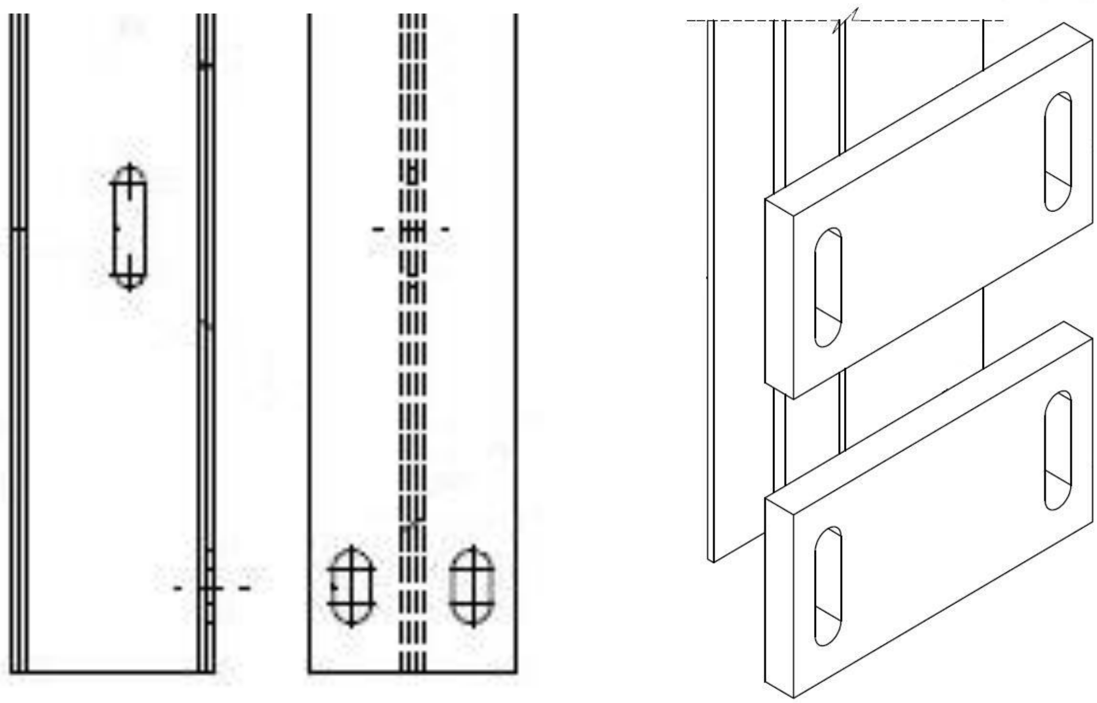


Figure 49. Longitudinal Tube Spacer with Extended Ends

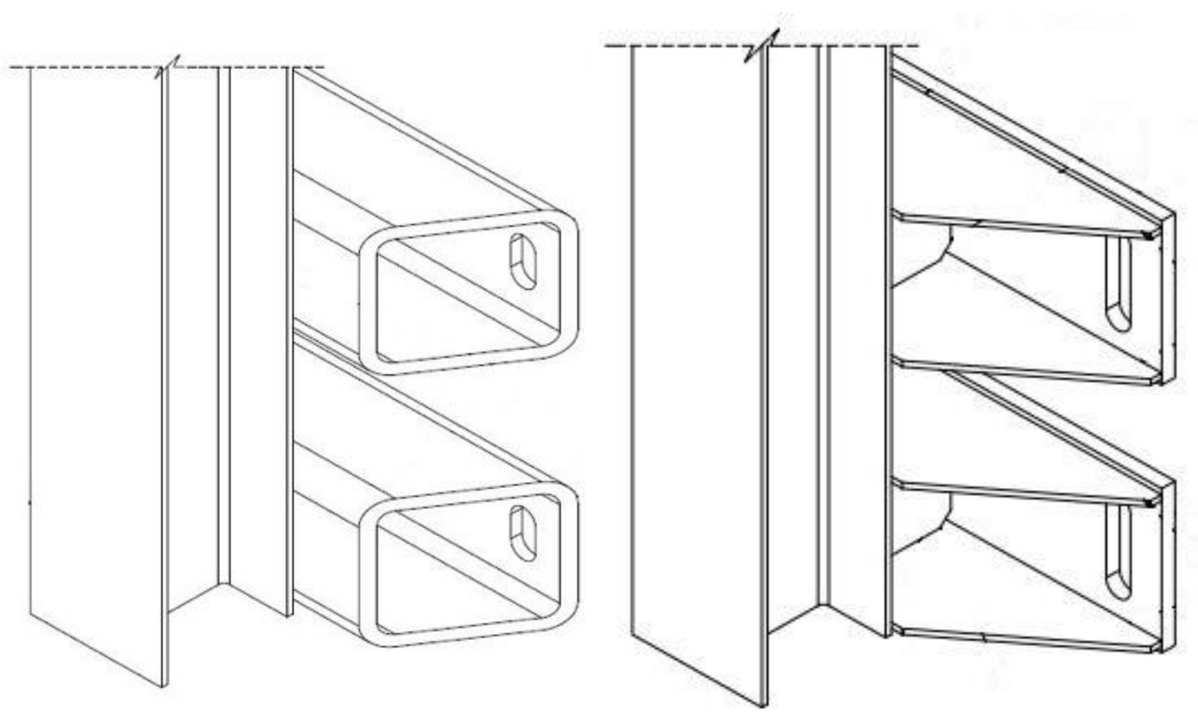
4.1.2.1 Vertical Tolerance

A vertical tolerance height was requested by both DOTs for camber and vertical grade adjustments. Therefore, post-to-deck attachments were designed to provide such vertical tolerance at either the post or deck of the attachment. Current tolerances allowed a 2⅛-in. (54-mm) movement in the post web and flange for the Illinois Double Angle connection, while the Ohio Twin Tube bridge railing offered a combination of 1-in. (25-mm) adjustment within the tube railings and a 1½-in. (38-mm) adjustment at the deck connection. A vertical tolerance of 3⅛ in. (79 mm) was provided in the post-to-deck attachment for the new bridge rail. This required tolerance could be provided on the post or deck side of the post-to-deck connection: if on the post side, the vertical adjustment was provided within the post flanges and web for the double angle connection, as shown in Figure 50a, or within the mounting plates in the two-plate welded post assembly of the Welded Plate and Spacer Block concept, as shown in Figure 50b. When vertical adjustment was provided at the bridge deck side, the adjustment was configured within the structural spacer, as shown in Figure 51.



(a) (b)

Figure 50. Post Side Vertical Adjustment in (a) Post Web and Flanges and (b) Mounting Plates



(a) (b)

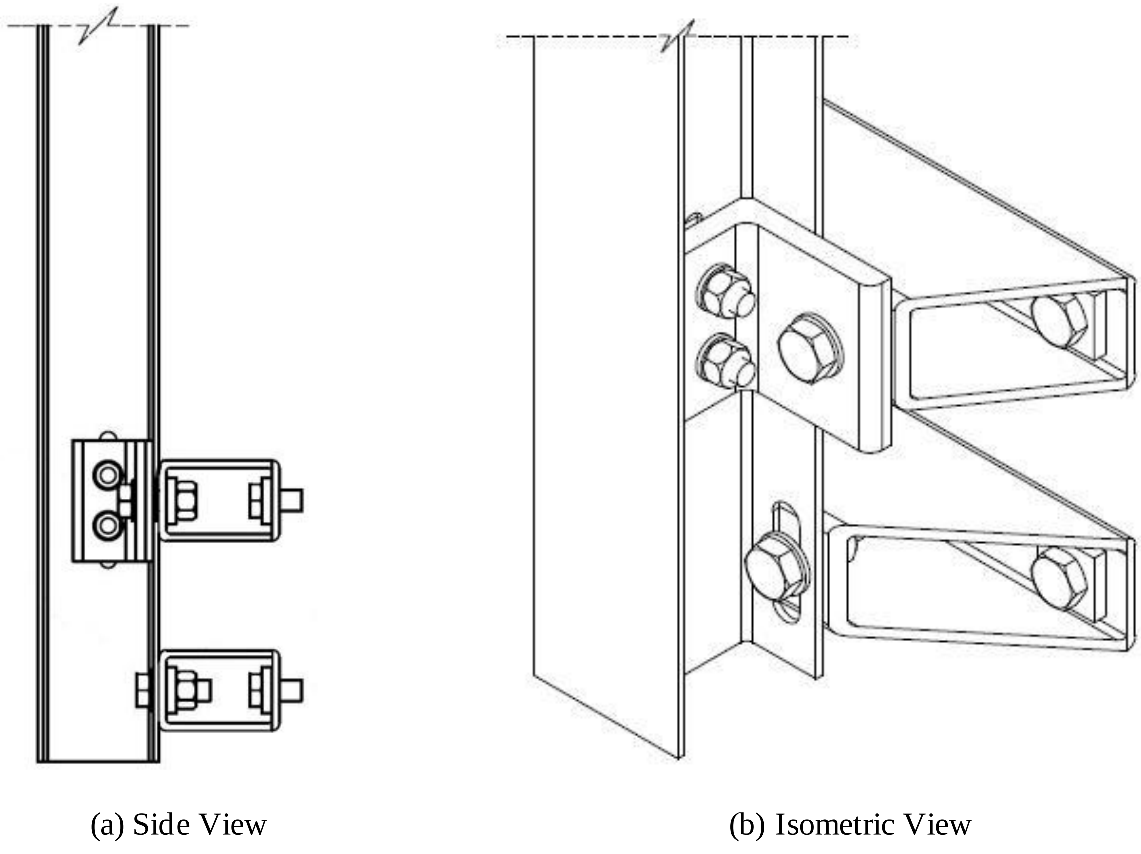
Figure 51. Deck Side Vertical Adjustment in (a) Spacer Tubes or (b) Built-up Spacers

4.1.3 Deck Attachment Designs

Through brainstorming sessions, six post-to-deck attachment designs were developed and proposed to IDOT and ODOT, with the intention of selecting one design for dynamic testing and evaluation. The designs were: (1) a double angle connection with longitudinal tubes; (2) a two-plate welded post with longitudinal tubes; (3) a 1¼-in. (32-mm) thick two-plate attachment with longitudinal tubes; (4) an HSS welded assembly; (5) a 1¼-in. (32-mm) thick two-plate attachment with fabricated spacer; and (6) a welded plate assembly. Each design had a unique method of transferring impact loads to the side of the bridge deck with the intention of minimizing attachment and deck damage. Where possible, all designs used square washers at slot locations to reduce the propensity for bolt pullout during an impact event. The designs are described in the following sections.

4.1.3.1 Double Angle Connection with Longitudinal Tubes

A double angle connection with longitudinal tubes was developed to be similar to the original double angle attachment currently utilized by IDOT, as shown in Figure 52. The design featured 6-in. x 4-in. x ⅞-in. (152-mm x 102-mm x 22-mm) double angles and HSS6x4x⅜ structural spacers, as shown in Figure 52. Impact loads would be transferred into the bridge deck as a tensile force through the double angles and top longitudinal tube, and a compression force through the bottom post bearing against the lower tube and the side of the deck. Vertical tolerances of 3⅛ in. (79 mm) were incorporated in the post web and front post flanges.

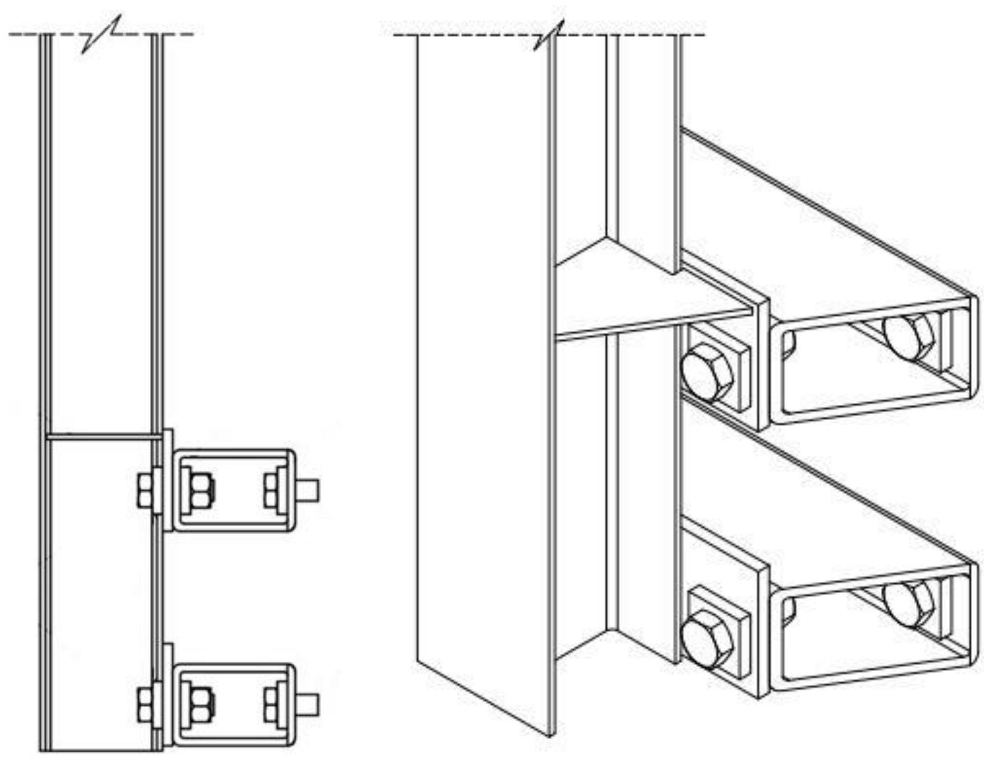


(a) Side View (b) Isometric View

Figure 52. Double Angle Connection with Longitudinal Tubes

4.1.3.2 Two-Plate Welded Post with Longitudinal Tubes

A two-plate welded post with longitudinal tubes featured two plate attachments with post gussets welded to the top plate attachment with HSS6x4x⅜ longitudinal tubes as the structural spacers, as shown in Figure 53. The top plate attachment was strengthened with a post gusset to distribute impact load evenly across the lateral connection while compression forces were transferred from the post to the deck by having bearing against the lower longitudinal tube. The vertical tolerances were established in the two-plate attachments as slotted holes. Along with the slotted holes on the two plate attachments, square washers were utilized inside the HSS spacers to help distribute the load along the tube sidewalls to prevent buckling.

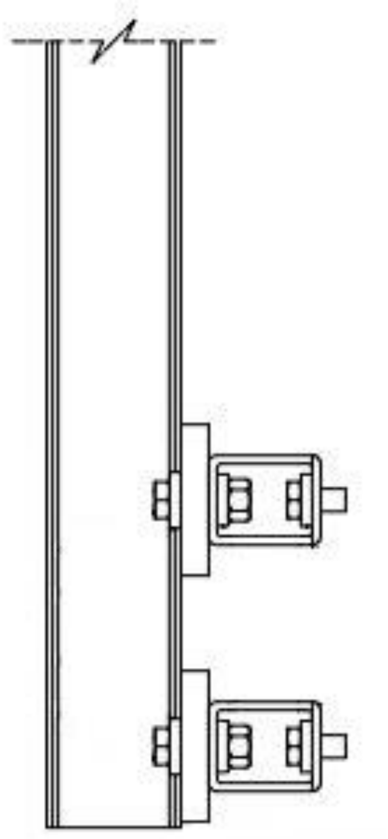


(a) Side View (b) Isometric View

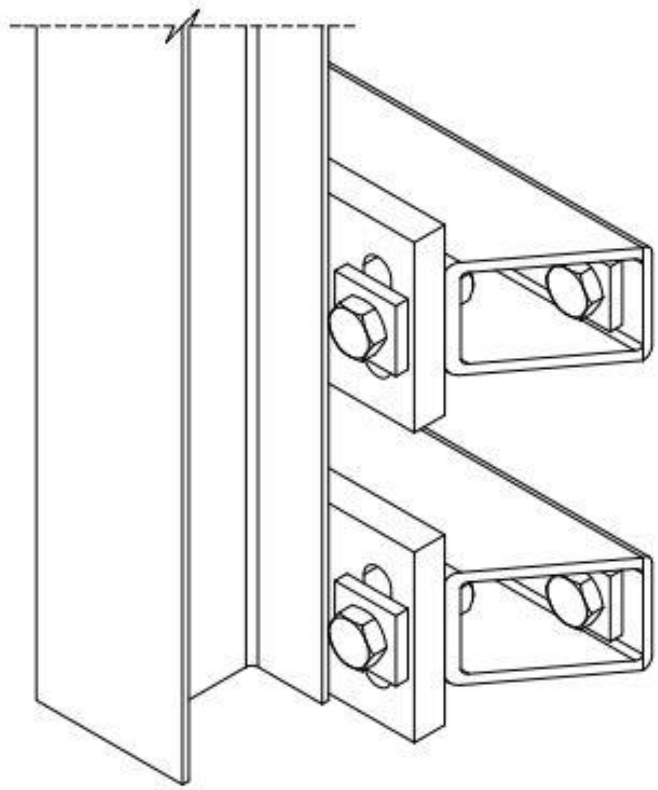
Figure 53. Two-Plate Welded Post with Longitudinal Tubes

4.1.3.3 1¼-in. (32-mm) Two-Plate Attachment with Longitudinal Tubes

The 1¼-in. (32-mm) thick two-plate attachment with longitudinal tubes featured a welded post assembly of two plate attachments welded to the front post flange, bolting to HSS5x4x⅜ longitudinal tubes, as shown in Figure 54. IDOT and ODOT expressed a preference for having a welded-plate post attachment consisting of a two-plate post with HSS tube spacers, but without use of web stiffeners in the steel post, as featured in the previous two-plate welded post with longitudinal tubes. Plates 1¼ in. (32 mm) thick were required to mitigate plate bending, without gussets, during an impact event. The plate attachments allowed vertical tolerances on the post side of up to 3⅛ in. (79 mm) as requested by the states. Similar to previous designs, the longitudinal tubes were bolted to the two-plate attachments and to the deck side.



(a) Side View

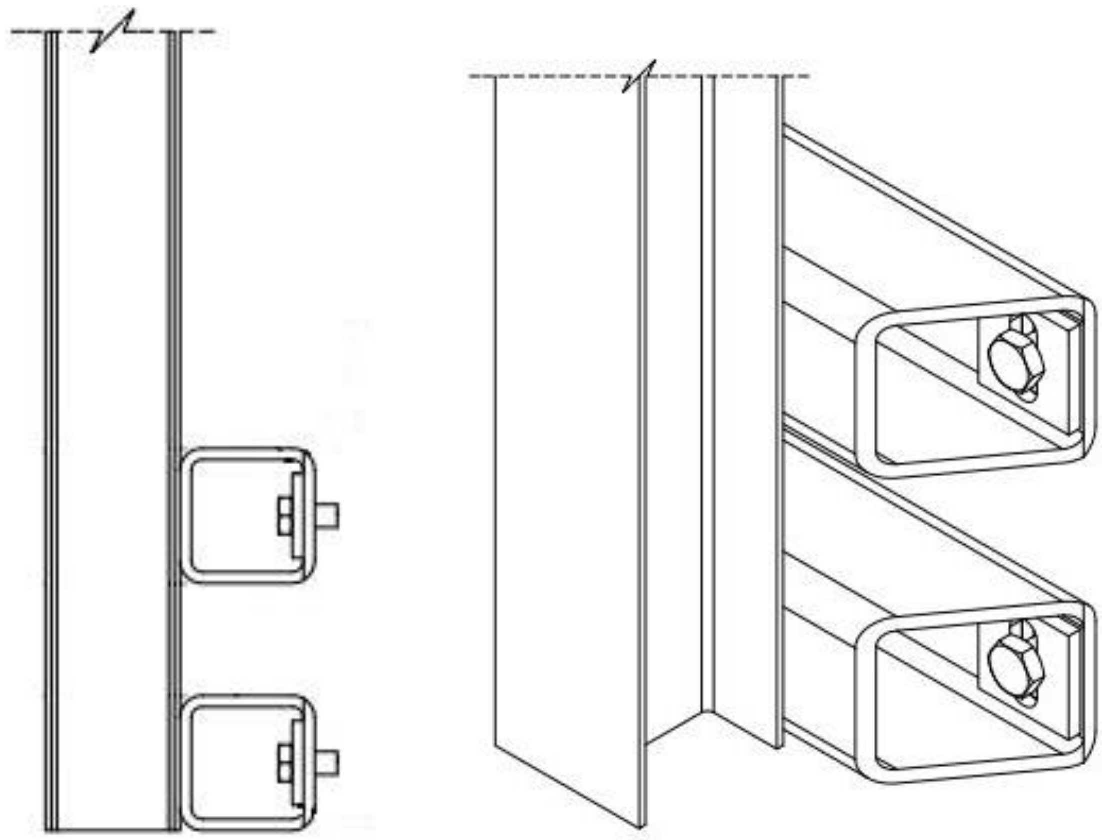


(b) Isometric View

Figure 54. 1¼-in. (32-mm) Two-Plate Attachment with Longitudinal Tubes

4.1.3.4 HSS Welded Assembly

The HSS welded assembly featured two HSS6x6x½ structural tubes welded directly to the front flange of the post, as shown in Figure 55. A design featuring a welded connection between the post and the deck spacer without gussets was desired by the states. An HSS welded assembly offered the removal of any bolted connections on the post side of the spacer. Without the addition of the two-plate attachments that were in previous concepts, material cost and weight would be reduced. A disadvantage of utilizing the welded HSS tubes as spacers was that the 6-in. (152-mm) depth of the tube would limit the vertical tolerance to a 2-in. (51-mm) maximum on the deck side due to workable gauge length in the tube sidewall. Increasing the spacer depth would require increasing the tube thickness to reduce the propensity for sidewall bowing. However, material cost and weight would increase as well.

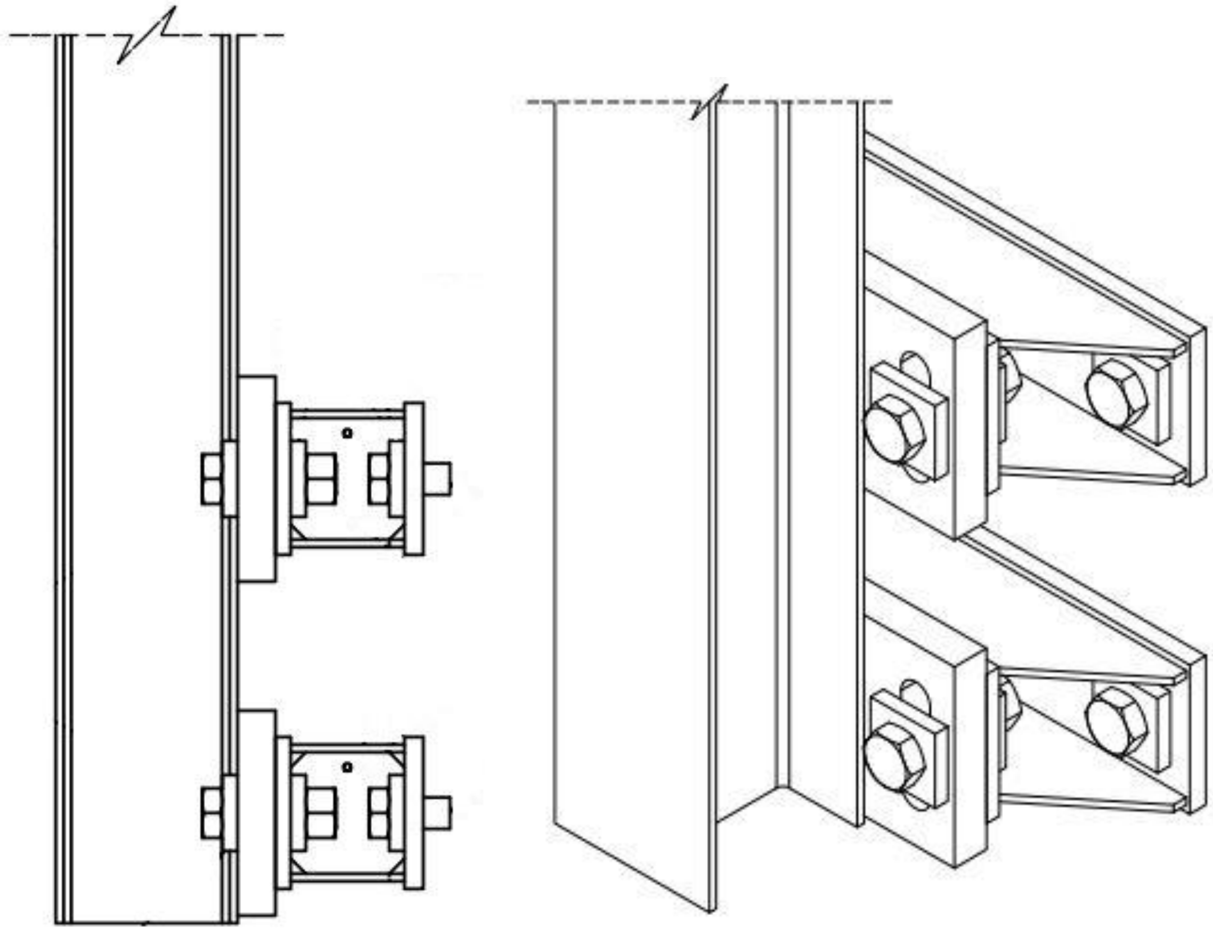


(a) Side View (b) Isometric View

Figure 55. HSS Welded Assembly

4.1.3.5 1¼-in. (32-mm) Plates with Fabricated Spacer

The 1¼-in. (32-mm) thick plates with a fabricated spacer concept had thicker plate attachments and no gussets, as shown in Figure 56. A fabricated spacer consisting of horizontal gussets welded to two plates was bolted to the 1¼-in. (32-mm) thick mounting plates and to the bridge deck. The design concept allowed the 3⅛-in. (79-mm) vertical tolerance on the post side.

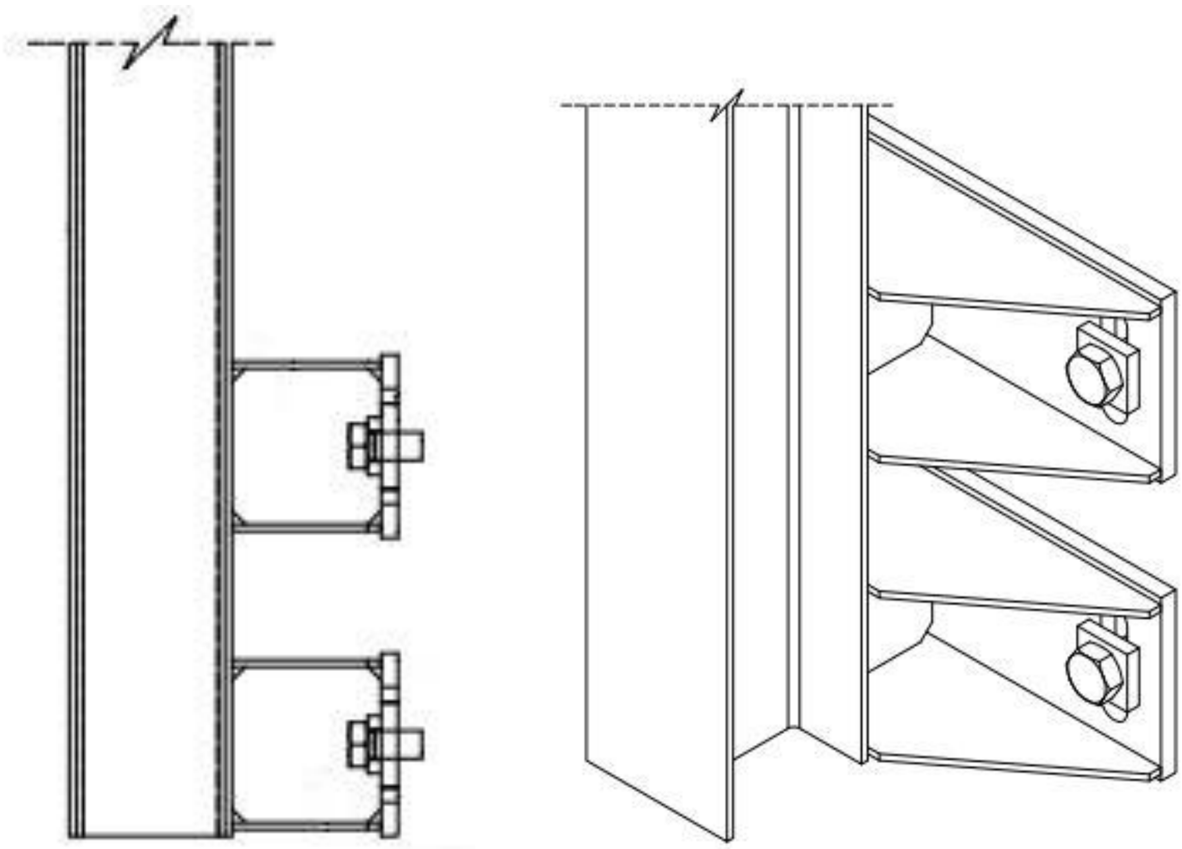


(a) Side View (b) Isometric View

Figure 56. 1¼-in. (32-mm) Two-Plate Attachment with Fabricated Spacer

4.1.3.6 Welded Spacer Assembly

The welded spacer assembly had a fabricated spacer block welded directly to the front post flange with no post gussets, as shown in Figure 57. On the deck side, vertical tolerance of $3^1/_8$ in. (79 mm) was allowed in slotted holes on the back-side mounting plate attached to the deck. The welded spacer assembly was comprised of two ¼-in. (6.4-mm) top and bottom plates welded to a vertical ½-in. (13-mm) plate and a ⅜-in. (9.5-mm) backside plate that is anchored to the deck.



(a) Side View (b) Isometric View

Figure 57. Welded Spacer Assembly

IDOT and ODOT proceeded to select the 1¼-in. (32-mm) thick two-plate attachment concept, as shown in Figure 54, with HSS5x4x⅜ longitudinal tube spacers for component testing of the concrete box-beam girder. The final design of the post-to-deck attachment was optimized and refined through the component tests.

5 BOX-BEAM GIRDER DESIGN

In order to design a bridge rail attachment that would be applicable to the wide range of bridge decks utilized by IDOT and ODOT, a critical box-beam girder configuration needed to be identified. The most critical box-beam girder design was selected for component testing of the deck attachment and evaluating the structural integrity of the beam girder. Design details such as top and bottom layer thickness of the box-beam girder, sidewall thickness, and steel reinforcement configurations were obtained from current IDOT and ODOT box-beam girder standards. IDOT and ODOT girders had 2-ft 6-in. (762-mm) long, reinforced capped ends with a hollow middle core. The middle core had a thin wall structure in which the post-to-deck attachment hardware would be anchored.

During an impact event, the impact load would transfer from the bridge rail post to the deck, and the post-to-deck attachment would bear against the box-beam girder walls with risk of wall failure due to punching shear. Of the box-beam girders provided by the states, the girder with the weakest sidewall was considered to be a critical design, in terms of having the least amount of steel reinforcement with the thinnest walls.

5.1 Illinois Box-Beam Girder

The existing Illinois precast, prestressed concrete box-beam girder designs had 36-in. (914-mm) widths with depths of 17 in. (432 mm), 21 in. (533 mm), 27 in. (686 mm), 33 in. (838 mm), and 42 in. (1,067 mm), as shown in Figure 58. The top and bottom layers of the box-beam girders are 5½-in. (140-mm) thick and have 7-in. (178-mm) thick walls. The top layer features No. 4 lateral reinforcement straight bars at a 36-in. (914-mm) spacing, No. 4 U-bars at an 18-in. (457-mm) spacing, and No. 5 longitudinal reinforcement straight bars placed symmetrically across the girder's width. The bottom layer features prestressing strands placed symmetrically about the centerline of the girder with a No. 4 U-bar at a 9-in. (229-mm) spacing around the strands. Top, bottom, and edge concrete covers are 1½ in. (38 mm), 1 in. (25 mm), and 2½ in. (63.5 mm), respectively. It is important to note that the Illinois box-beam girders may have prestressing strands located within its side walls that progress upward along the wall as the girder depth increases.

Steel reinforcement consisted of 60,000 psi (414 MPa) minimum yield strength, epoxy coated rebar. Prestressing steel consisted of uncoated high strength, low relaxation 7-wire strands, Grade 270 ksi, with a nominal diameter of ½ in. (13 mm) with a total nominal cross-sectional area of 0.153 in.^{2} (99 mm^2). A 28-day compressive strength of prestressed concrete was 6,000 psi (41.4 MPa), and the compressive strength of prestressed concrete at release was 5,000 psi (34.5 MPa).

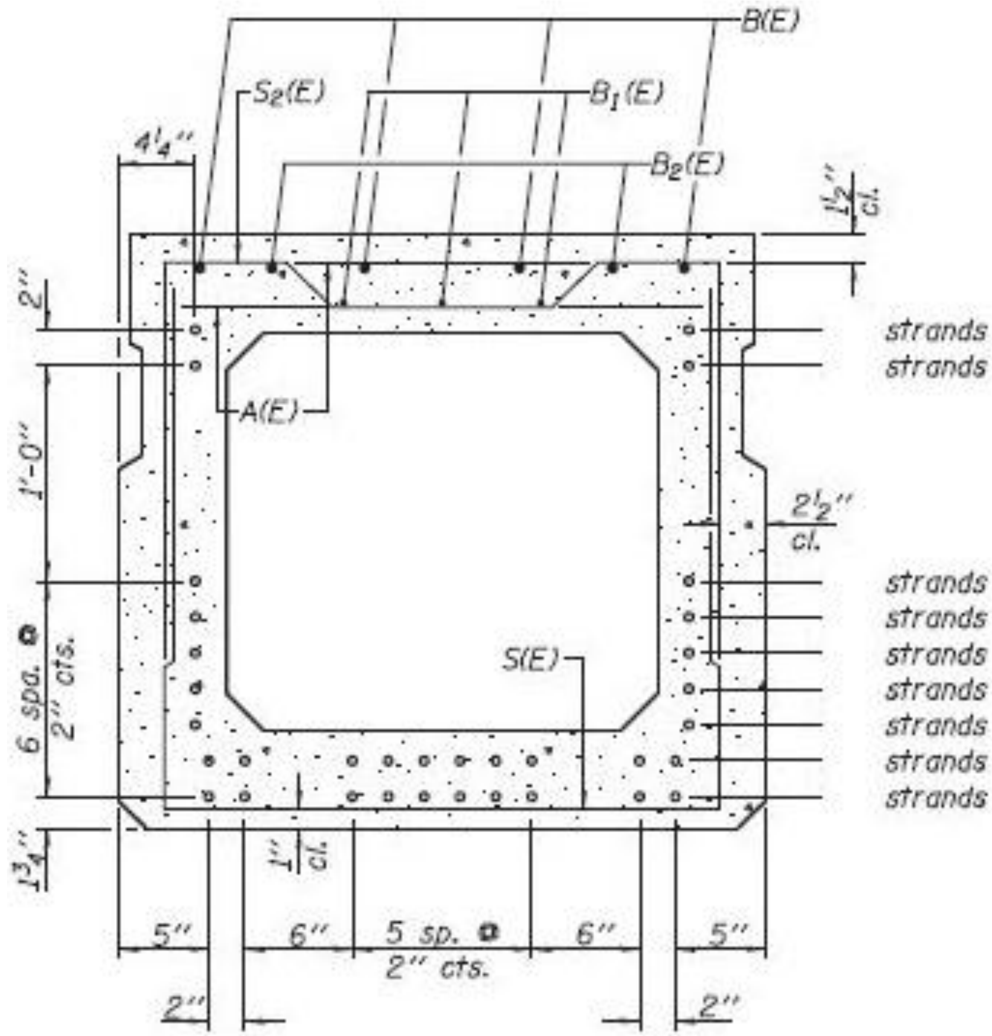


Figure 58. Typical Illinois Precast, Prestressed Box-Beam Girder Details [2]

5.2 Ohio Box-Beam Girder

The Ohio precast, prestressed concrete box-beam girders may have 36-in. (914-mm) or 48-in. (1,219-mm) widths with depths of 12 in. (305 mm), 17 in. (432 mm), 21 in. (533 mm), 33 in. (838 mm), and 42 in. (1067 mm). The top layer, bottom layer, and the sidewalls are 5½ in. (140 mm) thick, as shown in Figure 59. The top layer features lateral reinforcement of two No. 4 U-bars spaced at 18 in. (457 mm) and No. 5 longitudinal reinforcement straight bars placed symmetrically across the girder width. The bottom layer has prestressing strands placed symmetrically about the vertical centerline of the girder and are distributed over the girder width, with a No. 4 U-bar placed under the strands. Two No. 5 longitudinal straight bars are also placed in the lower row of the prestressing strands. Unlike the Illinois design, the Ohio box-beam girders do not have prestressing strands within their sidewalls and only have splicing of the U-bar reinforcement.

As stated for the Illinois girder reinforcement details, the Ohio precast, prestressed box-beam girders utilize the same strand pattern and details. Steel reinforcement consisted of 60,000 psi (414 MPa) minimum yield strength epoxy coated rebar. Prestressing steel consisted of uncoated high strength, low relaxation 7-wire strands, Grade 270 ksi (1862 MPa), with a nominal diameter of ½ in. (13 mm) with a total nominal cross-sectional area of 0.153 in.^{2} (99 mm^2) or 0.167 in.^{2} (108 mm^2). A 28-day compressive strength of prestressed concrete was between 5,500 psi (37.9 MPa) and 7,000 psi (48.3 MPa), and the compressive strength of prestressed concrete at release was between 4,000 psi (27.6 MPa) and 5,000 psi (34.5 MPa).

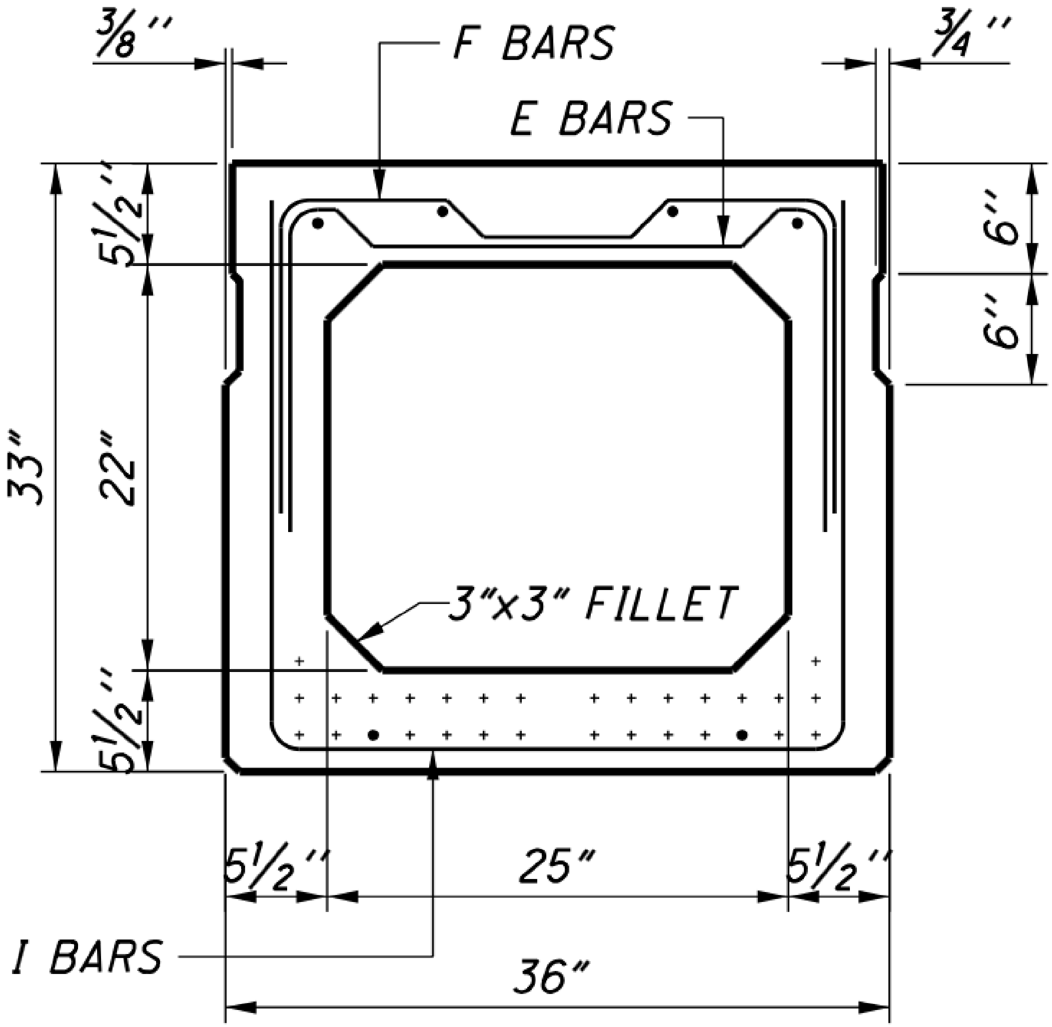


Figure 59. Typical Ohio Precast, Prestressed Box-Beam Girder Details [1]

5.3 Critical Box-Beam Girder

As previously mentioned, the most critical box-beam girder design was selected for component testing of the post-to-deck attachment and evaluating the structural integrity of the beam girder. The 36-in. wide x 42-in. (914-mm x 1,067-mm) deep box-beam girder used by ODOT was considered the most critical and weakest deck girder since the 5½-in. (140-mm) thick wall was the thinnest, had the least reinforcement, and had the longest unsupported wall span height. The dimensions and strand patterns used in the ODOT girder were therefore used to construct the box-beam girder selected for component testing, as shown in Figure 60.

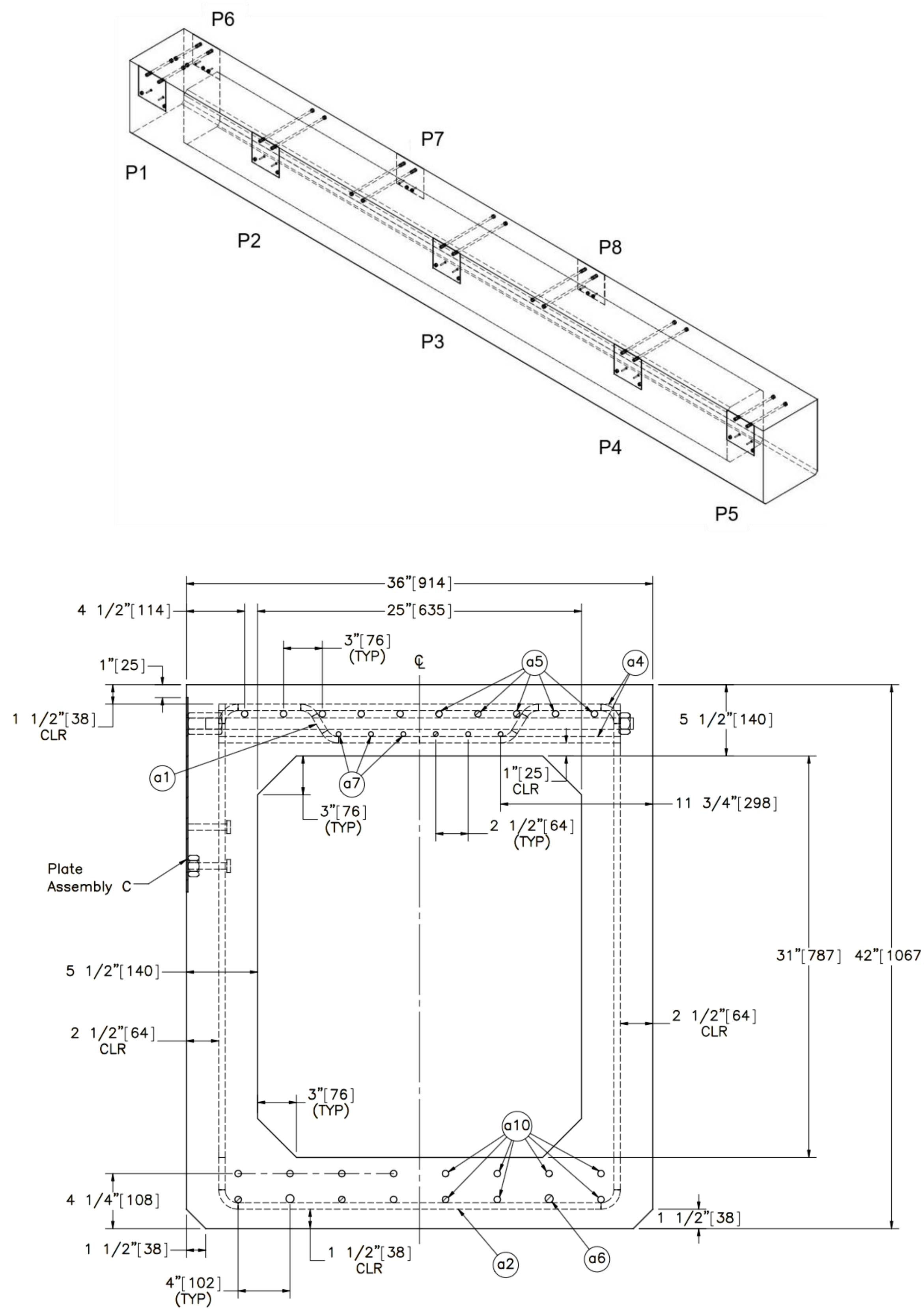


Figure 60. Illinois-Ohio (IL-OH) Box-Beam Girder Selected for Component Testing

The critical precast, prestressed concrete box-beam girder is 36 in. (914 mm) wide and 42 in. (1,067 mm) deep with a 5½-in. (140-mm) thick top layer, bottom layer, and wall. The top layer features No. 4 lateral reinforcement straight bars spaced at 18 in. (457 mm) and No. 5 longitudinal reinforcement straight bars placed symmetrically across the girder width. The bottom layer has prestressing strands placed symmetrically across the vertical centerline of the girder and are distributed over the girder's width, with a No. 4 U-bar placed under the strands. Similar to the Ohio box-beam girders, no prestressing strands were placed within the sidewalls, only splicing of the U-bar reinforcement.

Steel reinforcement consisted of 60,000 psi (414 MPa) minimum yield strength epoxy coated rebar. Prestressing steel consisted of uncoated high strength, low relaxation 7-wire strands, Grade 270 ksi, with a nominal diameter of ½ in. (13 mm) and a total nominal cross-sectional area of 0.153 $in.^2$ (99 mm^2). A 28-day compressive strength of prestressed concrete was 6,000 psi (41.4 MPa), and a compressive strength of prestressed concrete at release was 5,000 psi (34.5 MPa).

Eight post locations were implemented in the box-beam girder as possible testing locations to optimize the attachment hardware in relation to anchor diameter and embedment, plate and tube thickness in the deck attachment hardware, and stirrup spacing, as shown in Figure 60. Except for post location P3, all post locations featured 1-in. (25-mm) diameter ASTM F1554 Grade 105 anchor rods. Post location P3 utilized ⅞-in. (22-mm) diameter anchors in the event that the anchor diameter could be minimized. All anchor rods were situated between the No. 4 lateral reinforcement straight bars in the top layer of the box-beam girder. It shall be noted that fully threaded anchor rods were to be utilized, however, round bars with threaded ends were cast during fabrication of the girder. The ⅛-in. (3.2-mm) embedded plates with ½-in. (13-mm) diameter, 3-in. (76-mm) long shear studs and heavy hex nuts were also embedded within the girder during fabrication.

Within the hollow core section, the box-beam girder utilized two stirrup spacings to evaluate and minimize deck damage due to punching shear. Post locations P2 and P7 utilized the current state girder standard of 9-in. (229-mm) stirrup spacing while P3, P8, and P4 utilized a narrower 4½-in. (114-mm) spacing. The anchorage embedment lengths at post locations P2, P3, P4, P7, and P8 were 34½ in. (876 mm). Finally, post locations P1, P6, and P5 were located in the 30-in. (762-mm) long solid ends of the box-beam girder for testing of the anchorage embedment. P1 and P6 utilized embedment lengths of 16½ in. (419 mm) while P5 utilized lengths of 25½ in. (648 mm). Anchorage embedment varied to investigate the minimum required embedment length. In turn, shortened embedment would benefit anchorage in skewed bridges located at the ends of the bridge deck. A typical view of the reinforcement with the post anchorage is shown in Figure 61 and full set of drawings of the critical concrete box-beam girder are shown in Figures 62 through 75. Manufacturer drawings are shown in Appendix B.

Figure 61. Reinforcement and Post Anchorage Placement Prior to Casting

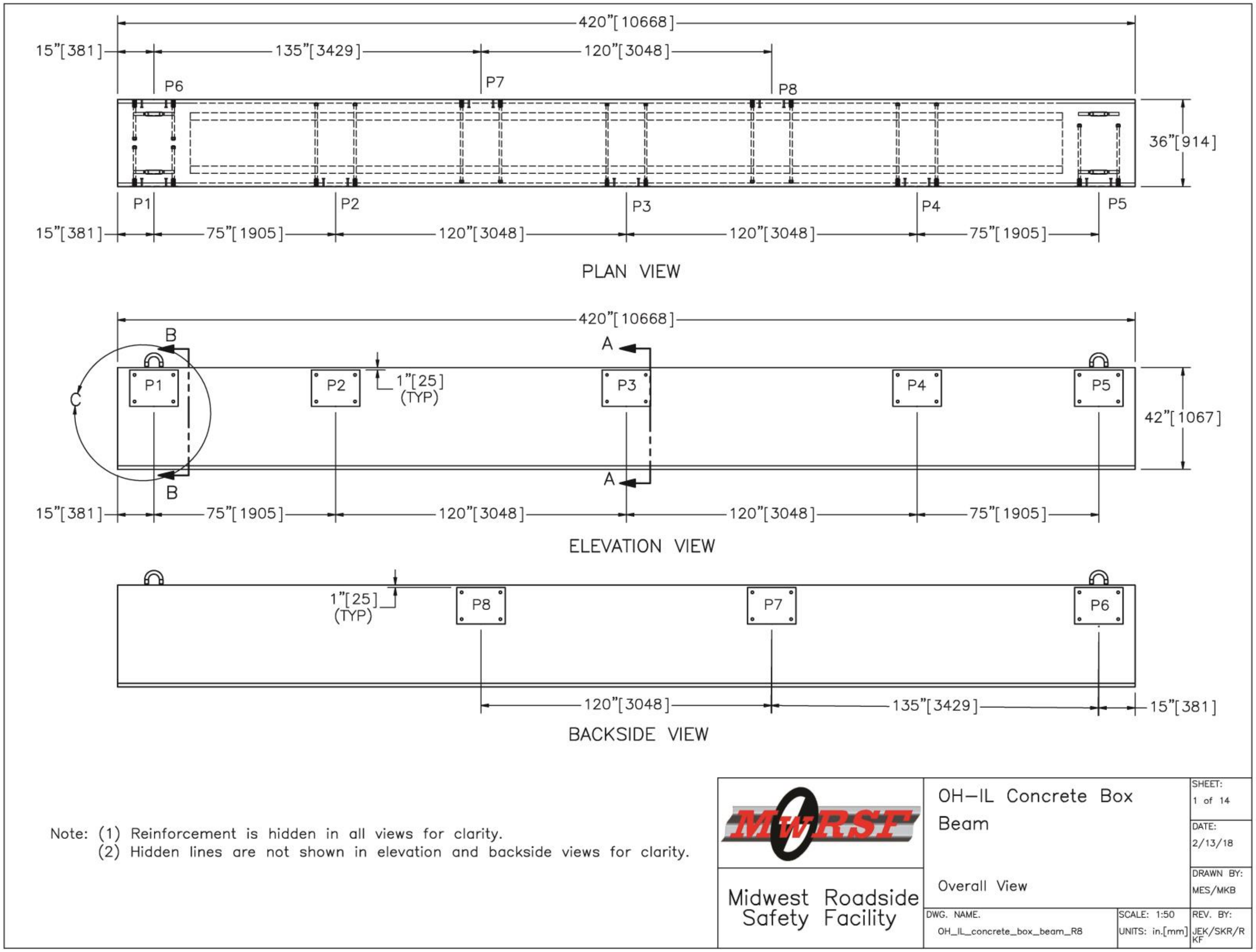


Figure 62. Critical Concrete Box-Beam Girder Details

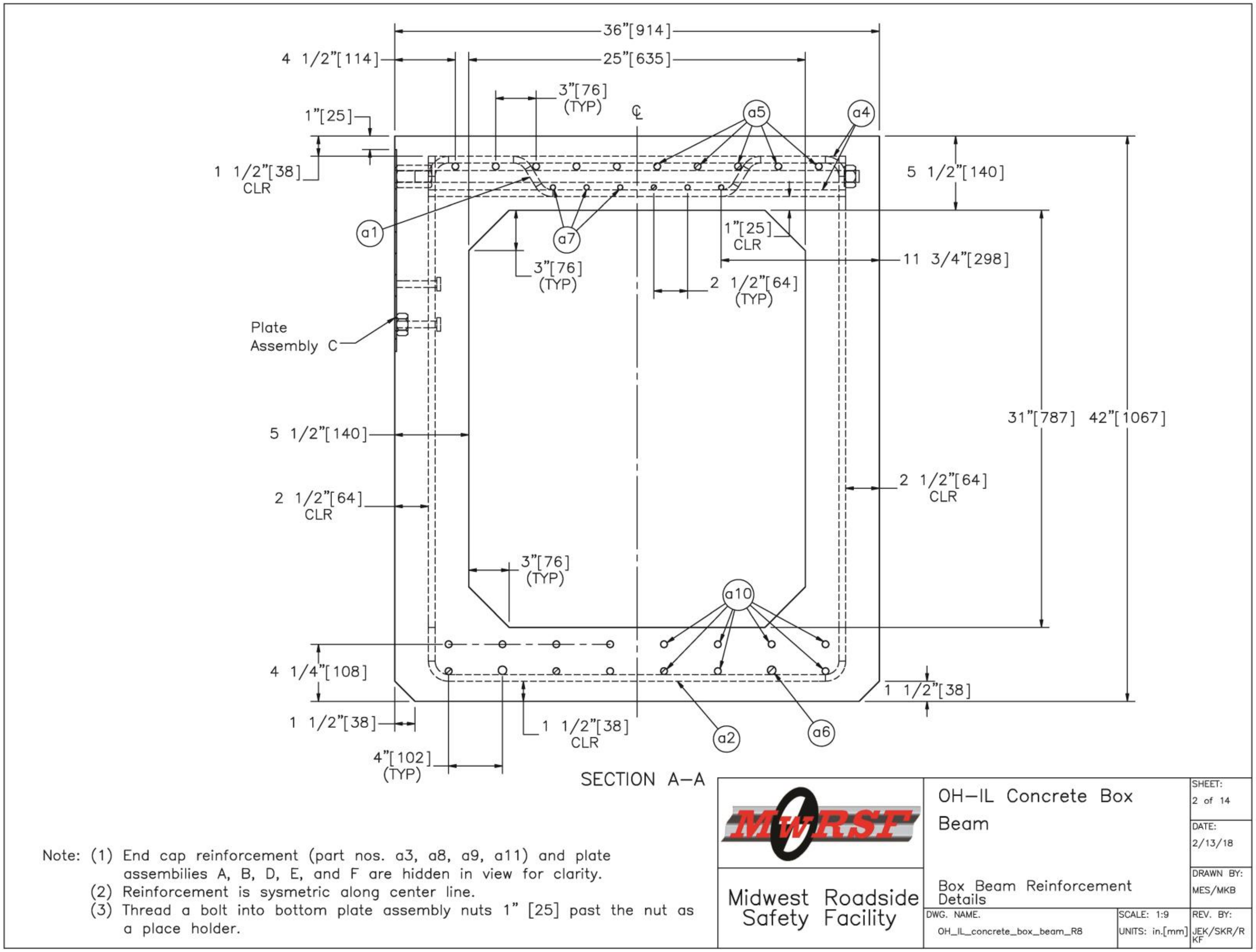


Figure 63. Critical Concrete Box-Beam Girder Reinforcement Details for Hollow Core Section

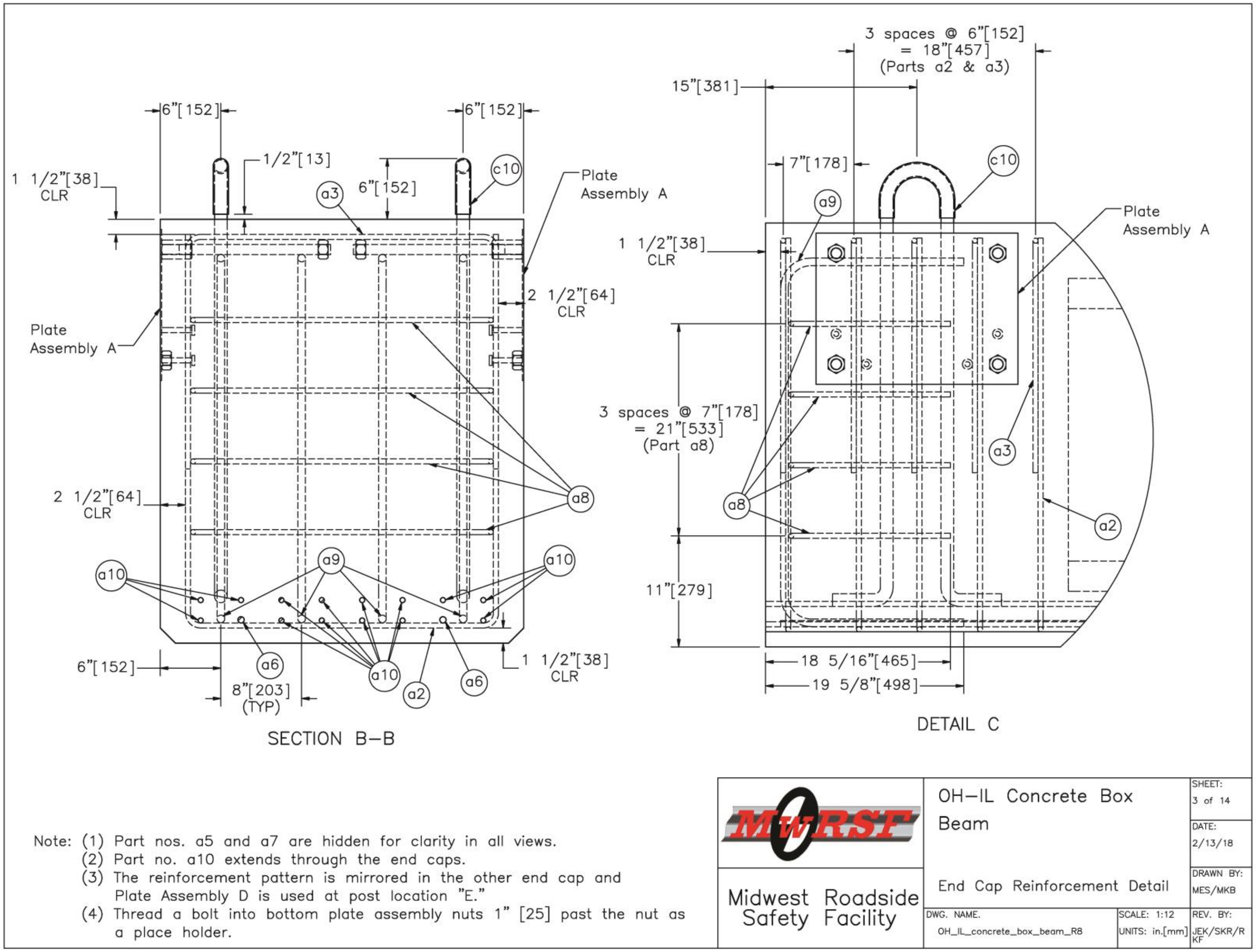


Figure 64. Critical Box-Beam Girder Reinforcement Details for End Cap Sections

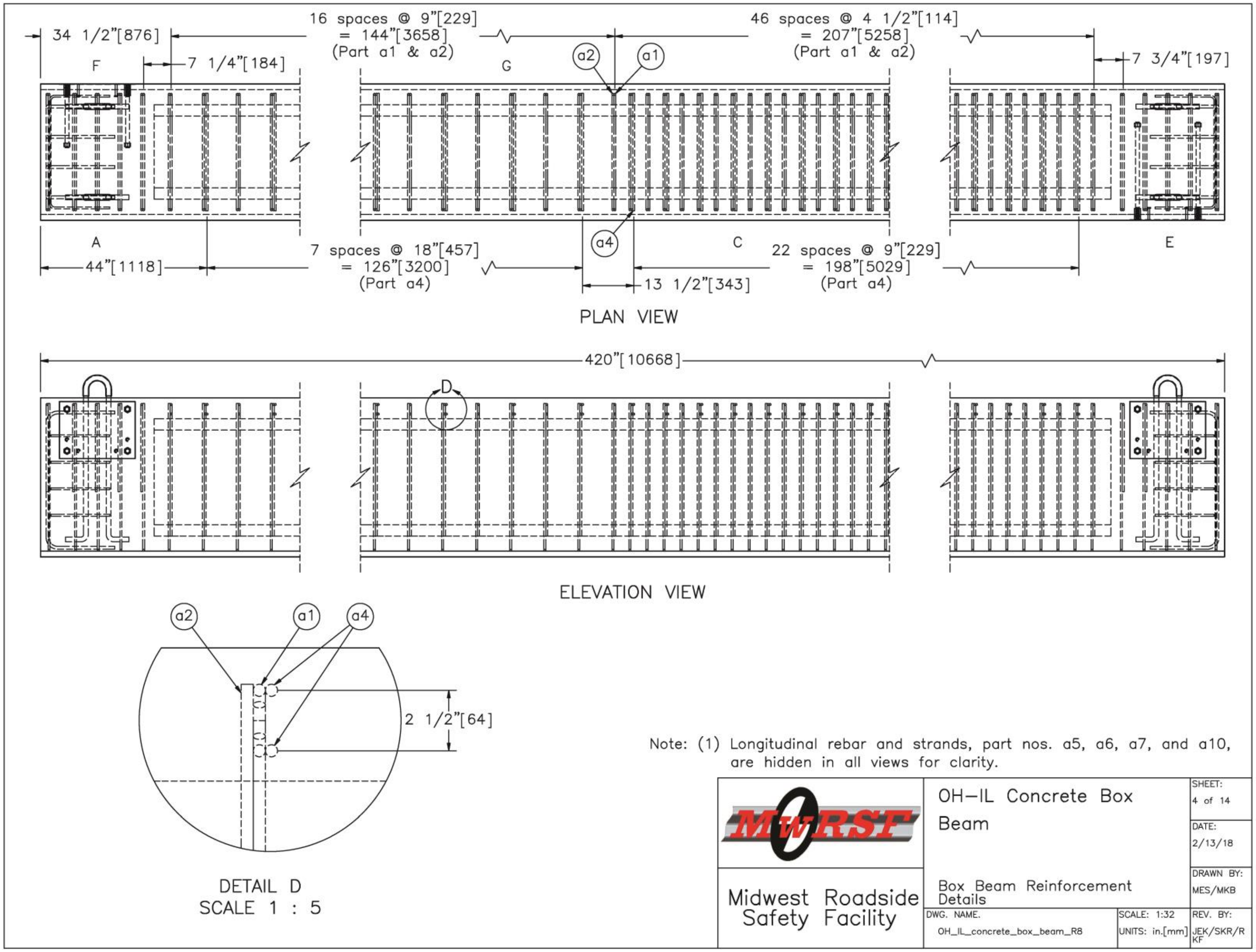


Figure 65. Overall Reinforcement Details

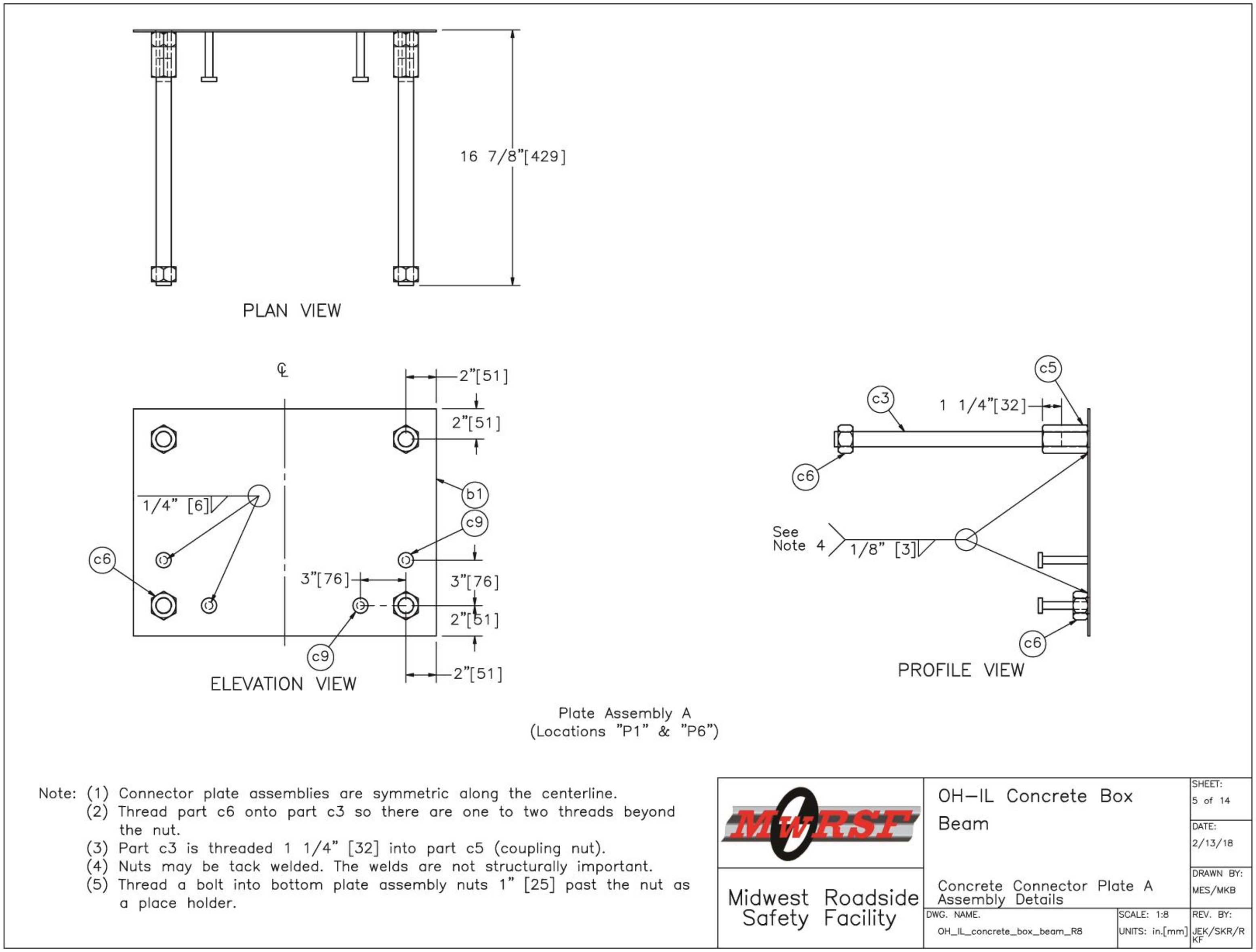


Figure 66. Plate Assembly A Details

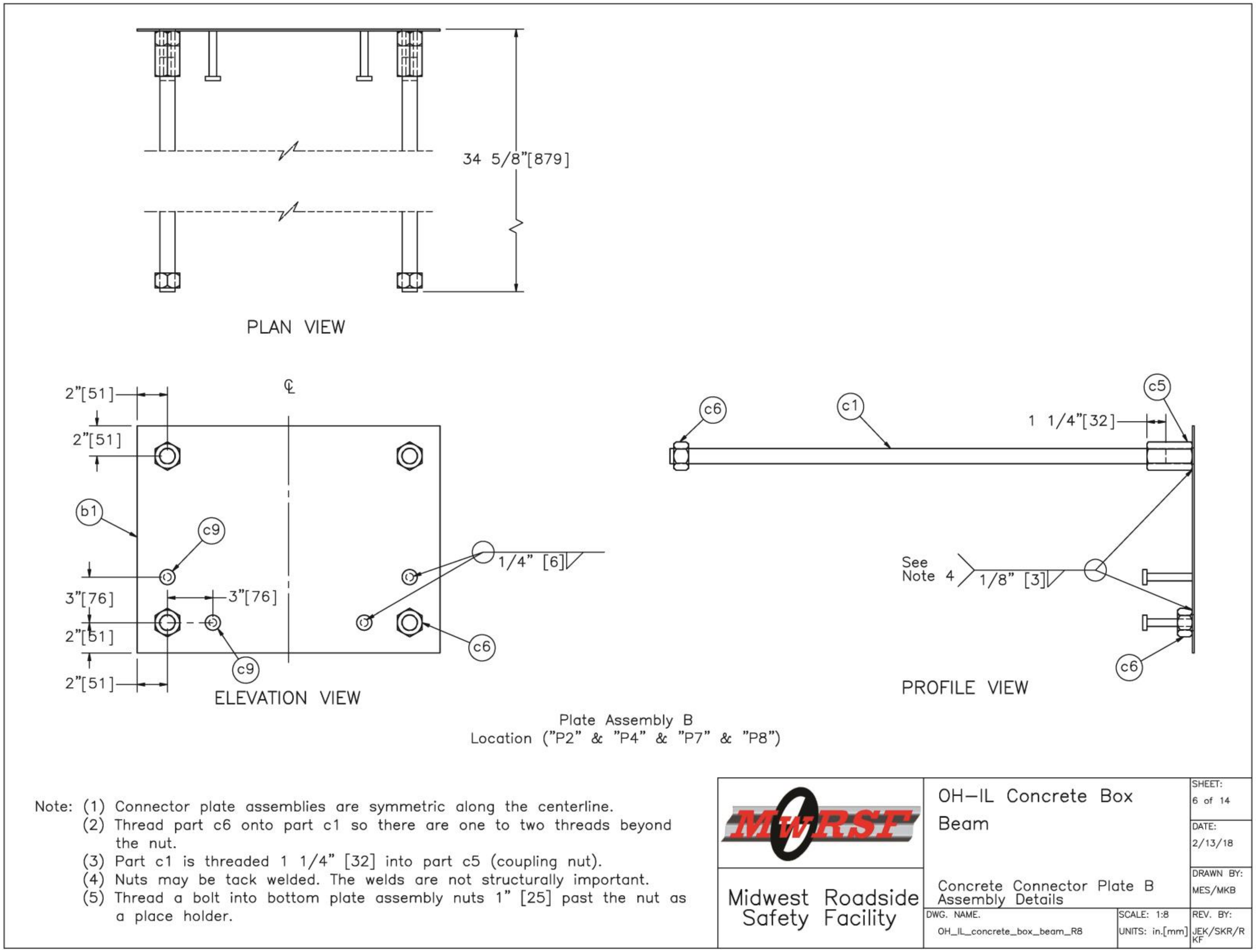


Figure 67. Plate Assembly B Details

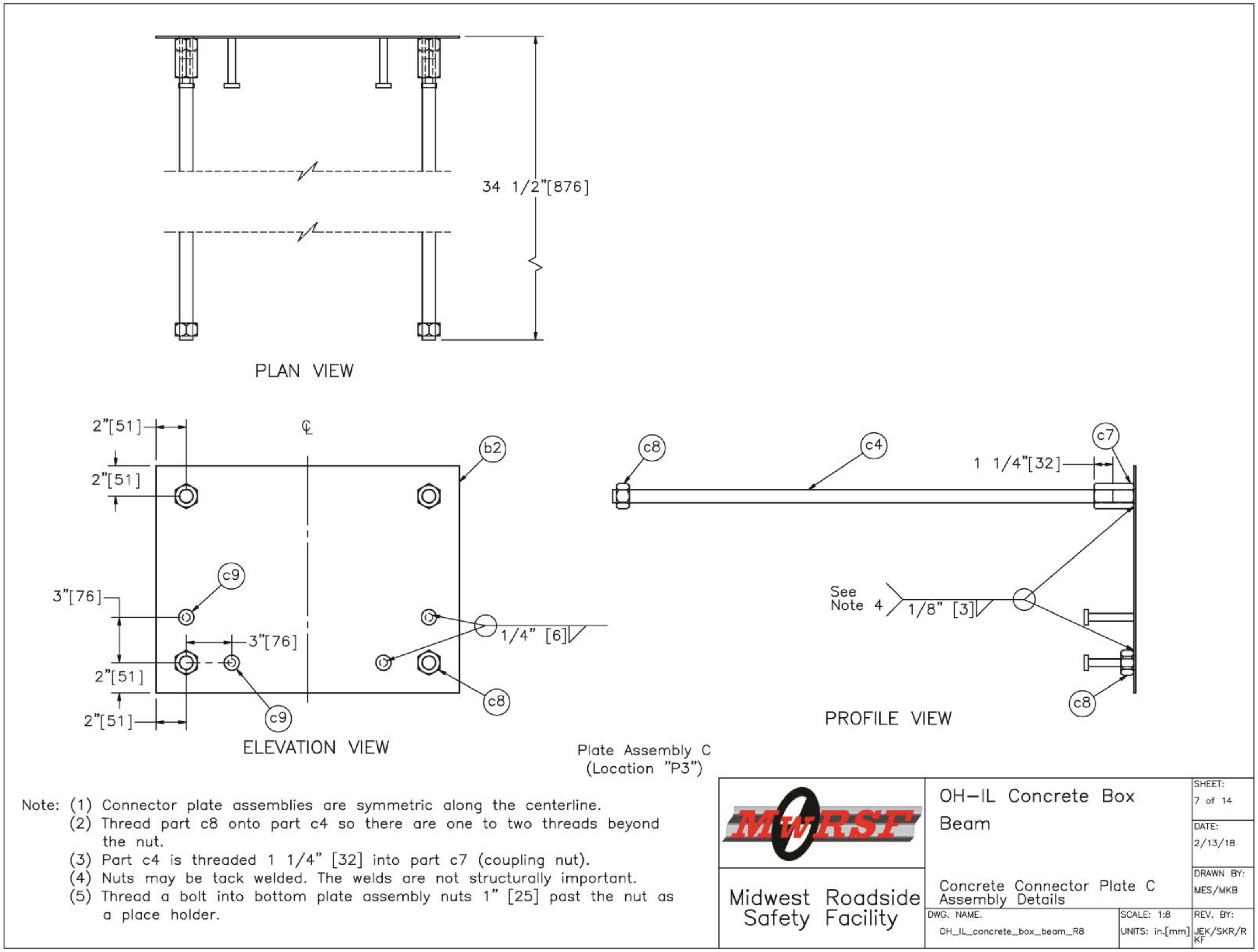


Figure 68. Plate Assembly C Details

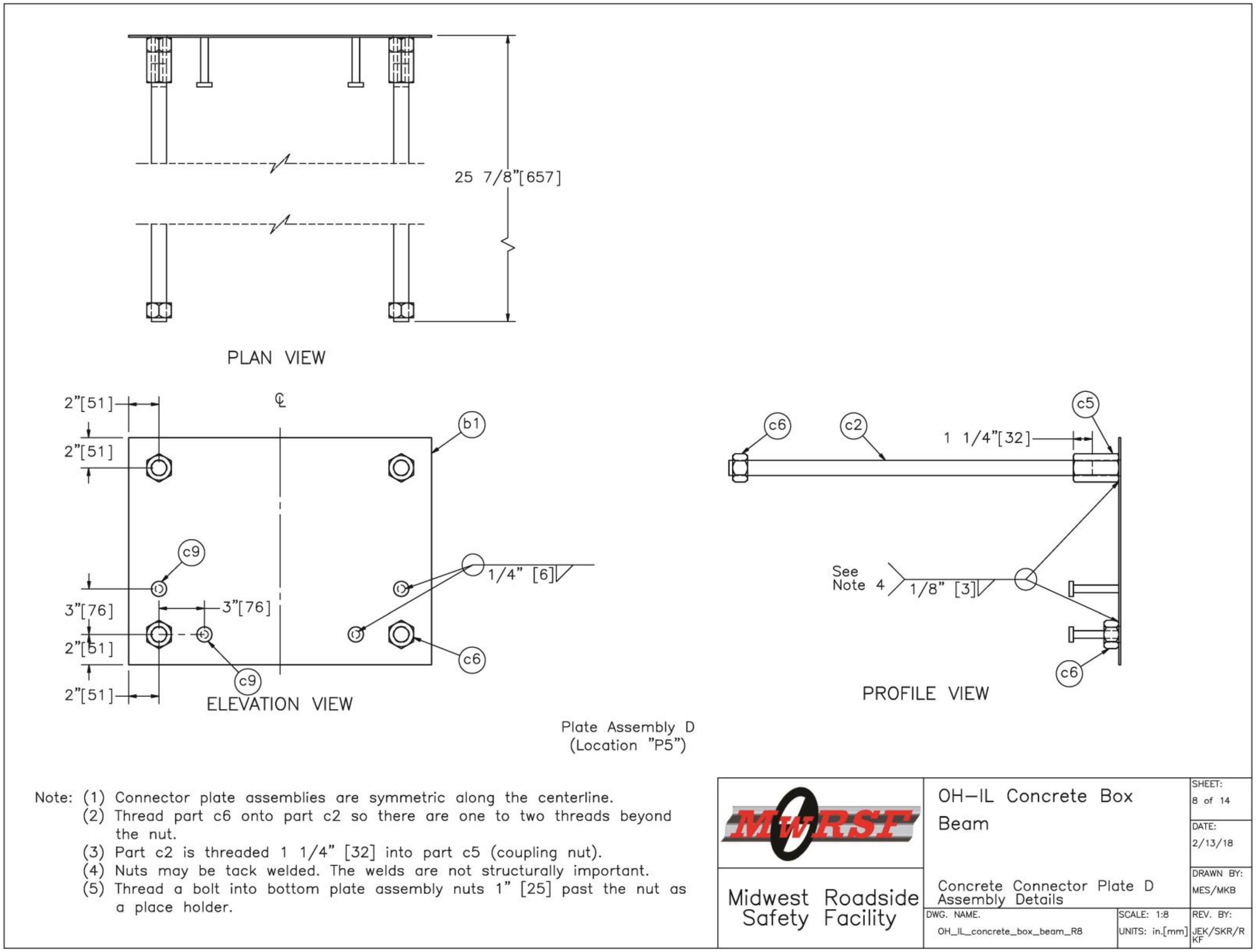


Figure 69. Plate Assembly D Details

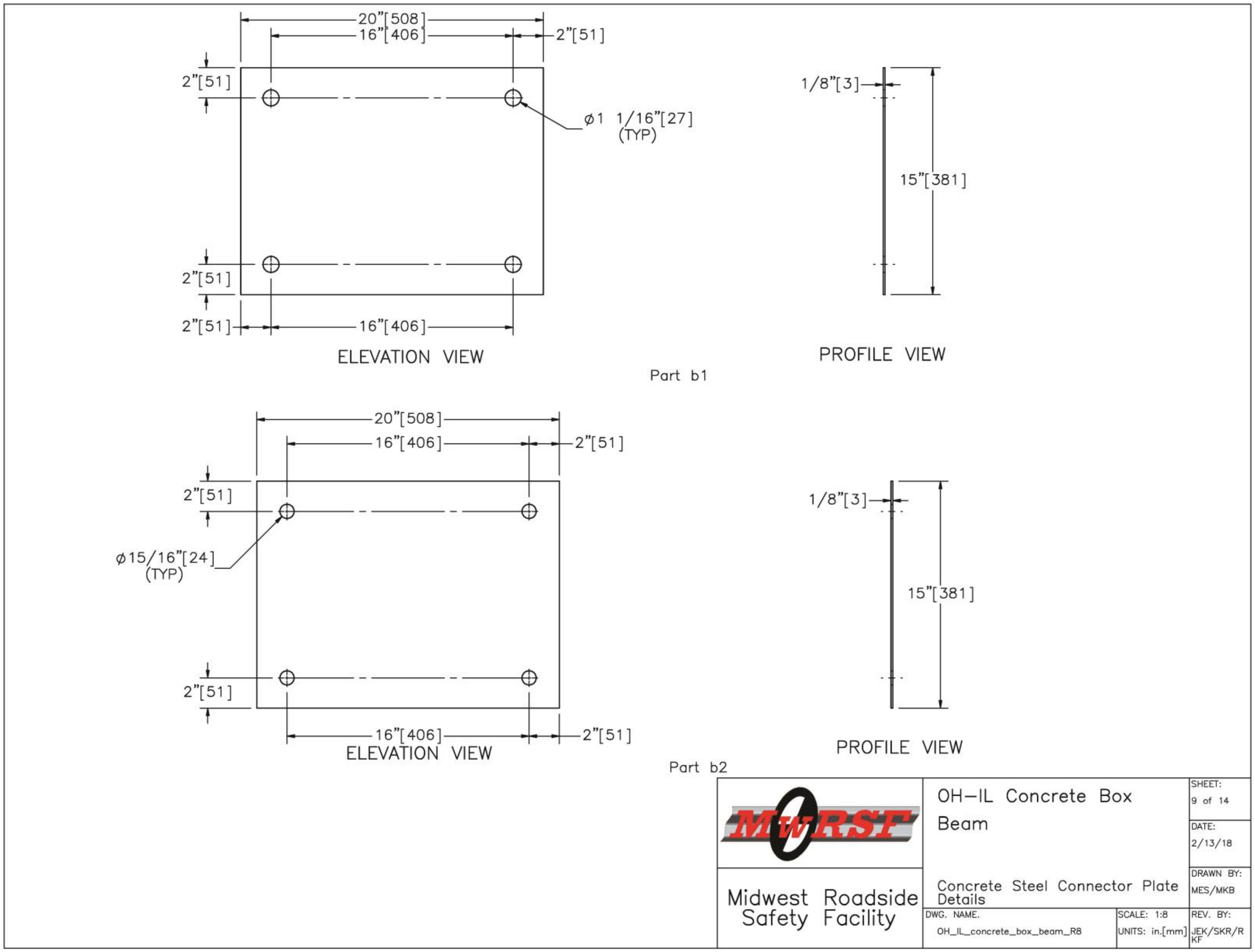
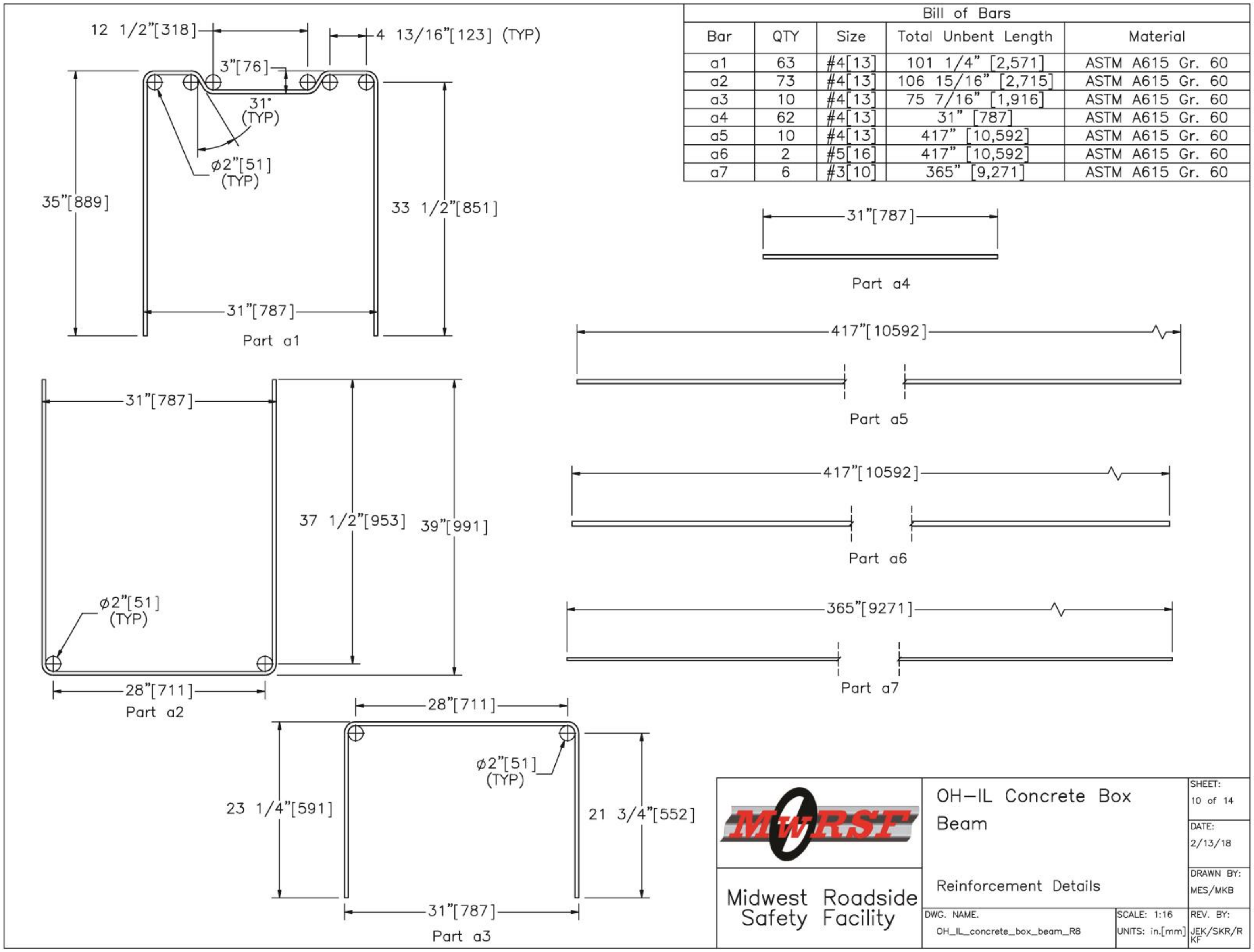


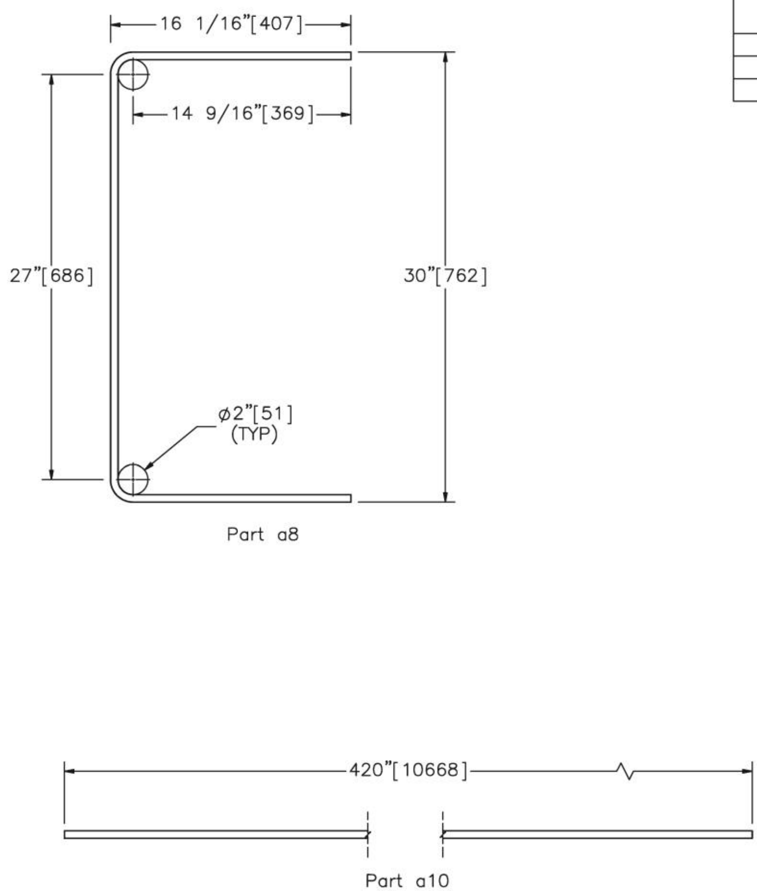
Figure 70. Connector Plate Details

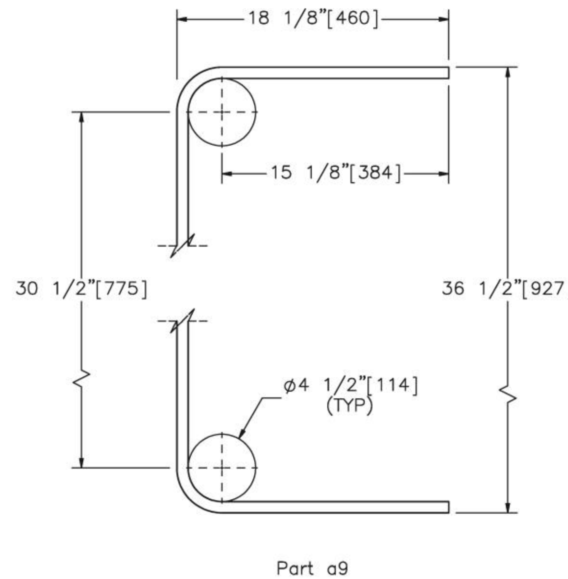


Bill of Bars				
Bar	QTY	Size	Total Unbent Length	Material
a1	63	#4[13]	101 1/4" [2,571]	ASTM A615 Gr. 60
a2	73	#4[13]	106 15/16" [2,715]	ASTM A615 Gr. 60
a3	10	#4[13]	75 7/16" [1,916]	ASTM A615 Gr. 60
a4	62	#4[13]	31" [787]	ASTM A615 Gr. 60
a5	10	#4[13]	417" [10,592]	ASTM A615 Gr. 60
a6	2	#5[16]	417" [10,592]	ASTM A615 Gr. 60
a7	6	#3[10]	365" [9,271]	ASTM A615 Gr. 60

Figure 71. Reinforcement Details, Sheet 1 of 2

Bill of Bars				
Bar	QTY	Size	Total Unbent Length	Material
a8	8	#4[13]	60" [1,524]	ASTM A615 Gr. 60
a9	8	#6[19]	69" [1,753]	ASTM A615 Gr. 60
a10	12	1/2" [13]	420" [10,668]	ASTM A416 Gr. 270





MwRSF Midwest Roadside Safety Facility	OH–IL Concrete Box Beam Reinforcement Details	SHEET: 11 of 14 DATE: 2/13/18 DRAWN BY: MES/MKB
	DWG. NAME. OH_IL_concrete_box_beam_R8 / SCALE: 1:10 UNITS: in.[mm]	REV. BY: JEK/SKR/RKF

Figure 72. Reinforcement Details, Sheet 2 of 2

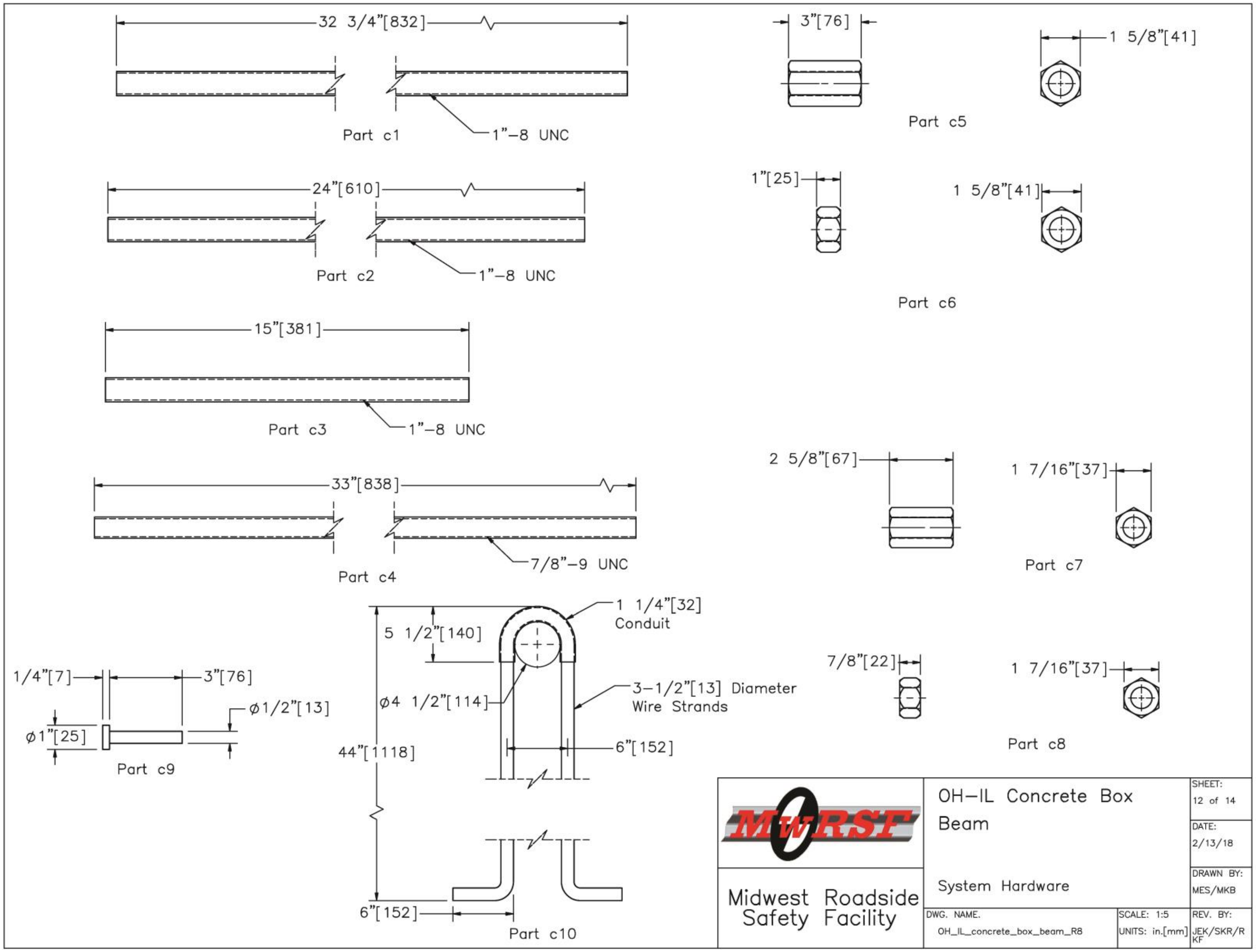


Figure 73. System Hardware Details

Item No.	QTY.	Description	Material Specification	Galvanization Specification
a1	63	#4 [13] Bent Rebar, Upper Stirrup, 101 1/4" [2,571] Total Unbent Length	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a2	73	#4 [13] Bent Rebar, Bottom Stirrup, 106 15/16" [2,715] Total Unbent Length	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a3	10	#4 [13] Bent Rebar, Bottom Stirrup, 75 7/16" [1,916] Total Unbent Length	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a4	62	#4 [13] Rebar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a5	10	#4 [13] Rebar, 417" [10,592] Long	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a6	2	#5 [16] Rebar, 417" [10,592] Long	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a7	6	#3 [10] Rebar, 365" [9,271] Long	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a8	8	#4 [13] Bent Rebar, U-Bar, 60" [1,524] Total Unbent Length	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a9	8	#6 [19] Bent Rebar, U-Bar, 69" [1,753] Total Unbent Length	ASTM A615 Gr. 60	Epoxy Coated (ASTM A775 or A934)
a10	14	1/2" [13] Dia., 7-Wire Prestressing Strand 420" [10,668] Long	ASTM A416 Gr. 270	–
b1	7	20"x15"x1/8" [508x381x3] Steel Plate	ASTM A572 Gr. 50	ASTM A123
b2	1	20"x15"x1/8" [508x381x3] Steel Plate	ASTM A572 Gr. 50	ASTM A123
c1	8	1" [25] Dia., 32 3/4" [832] Long Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
c2	2	1" [25] Dia., 24" [610] Long Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
c3	4	1" [25] Dia., 15" [381] Long Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
c4	2	7/8" [22] Dia., 33" [838] Long Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
c5	14	1" [25] Dia. Heavy Hex Coupling Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
c6	28	1" [25] Dia. Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
c7	2	7/8" [22] Dia. Heavy Hex Coupling Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
c8	4	7/8" [22] Dia. Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
c9	32	1/2" [13] Dia. Shear Stud, 3" [76] Long	ASTM A108	–

MwRSF

Midwest Roadside Safety Facility

OH-IL Concrete Box Beam		SHEET: 13 of 14
		DATE: 2/13/18
Bill of Materials		DRAWN BY: MES/MKB
DWG. NAME. OH_IL_concrete_box_beam_R8	SCALE: None UNITS: in.[mm]	REV. BY: JEK/SKR/RKF

Figure 74. Bill of Materials, Sheet 1 of 2

Item No.	QTY.	Description	Material Specification	Galvanization Specification
c10	4	3–1/2" [13] Dia., 7–wire prestressed strands, 98 3/8" [2,499] Long, Lifting Loops and Conduit	Strands – ASTM A416 Gr. 270 Conduit – As supplied	–
d1	1	420"x42"x36" [10,668x1,067x914] Concrete Box Beam	Min. f'c = 6,000 psi [41.4 MPa] 5,000 psi [34.5 MPa] @ Release	–

Midwest Roadside Safety Facility

OH–IL Concrete Box Beam

Bill of Materials

SHEET: 14 of 14

DATE: 2/13/18

DRAWN BY: MES/MKB

DWG. NAME. OH_IL_concrete_box_beam_R8

SCALE: None
UNITS: in.[mm]

REV. BY: JEK/SKR/RKF

Figure 75. Bill of Materials, Sheet 2 of 2

6 COMPONENT TESTING CONDITIONS

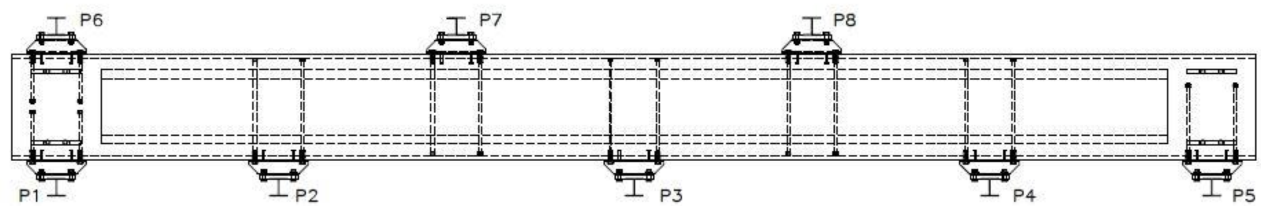
6.1 Purpose

Following the revision of the initial concepts, dynamic component tests were conducted to evaluate the performance of the selected post-to-deck connection featuring a 1¼-in. (32-mm) two-plate attachment with HSS5x4x⅜ longitudinal tube spacers and to evaluate deck damage. Posts and post-to-deck attachments were dynamically tested to verify if the preliminary estimated resistive forces of 26 kips (116 kN) were developed and if damage occurred to the concrete box-beam girder. Based on the results of the tests, the design concept was further refined. All dynamic tests were conducted at the MwRSF's Outdoor Test Site in Lincoln, Nebraska.

6.2 Scope

Seven dynamic bogie tests were conducted to explore the behavior of the W6x15 bridge rail posts and several post-to-deck attachment designs. The target impact conditions were a speed of 20 mph (32 km/h) and an angle of 90 degrees, creating a lateral impact and strong axis bending in the post. The posts were impacted 28 in. (711 mm) above the ground line perpendicular to the front face of the post to simulate impact height to the middle bridge railing. The bogie test matrix is shown in Figure 76, and component test full set of drawings are shown in Figures 77 through 98. Bogie impact height, velocity, and mass determination calculations are shown in Appendix C. Component test results for all transducers are provided in Appendix D. Material specifications, mill certifications, and certificates of conformity for the (test component description, e.g. post) are shown in Appendix E.

Test Name	Post Placement (on Box Beam)	Bogie No.	Bogie Weight lb [kg]	Impact Height in. [mm]	Impact Speed mph [km/h]	Impact Orientation	Welded Post Assembly	Tube Spacers	Exterior Washers	Exterior Bolts	Interior Washers	Interior Bolts
ILOH4-1	P2	3	2,000 [907]	28 [686]	20 [32]	Strong Axis	A	a6	b5	b1	b5	b2
ILOH4-2	P4	3	2,000 [907]	28 [686]	20 [32]	Strong Axis	D	a6	b5	b1	b5	b2
ILOH4-3	P8	3	2,500 [1134]	28 [686]	20 [32]	Strong Axis	G	a7	b5	b1	b5	b9
ILOH4-4	P8	3	2,500 [1134]	28 [686]	20 [32]	Strong Axis	G	a7	b5	b1	b5	b9
ILOH4-5	P7	3	2,500 [1134]	28 [686]	20 [32]	Strong Axis	G	a7	b5	b1	b5	b9
ILOH4-6	P5	3	2,500 [1134]	28 [686]	20 [32]	Strong Axis	G	a7	b5	b1	b5	b9
ILOH4-7	P1	3	2,500 [1134]	28 [686]	20 [32]	Strong Axis	E	a7	b5	b1	b5	b9



PLAN VIEW

Notes: (1) Reinforcement is hidden for clarity.
(2) Refer to OH_IL_concrete_box_beam for concrete box beam details.

Midwest Roadside Safety Facility

IL-OH Post Attachment
Bogie Testing
Test Nos. ILOH4-1-7

Test Layout and Test Matrix

SHEET: 1 of 21
DATE: 8/7/2018
DRAWN BY: JWB/JDJ/GRL/MBD/AEL
DWG. NAME. ILOH4-1-7_R14
SCALE: 1:50
UNITS: in.[mm]
REV. BY: JDS/RKF/PM/JEK/SKR

Figure 76. Bogie Testing Matrix and Test Layout

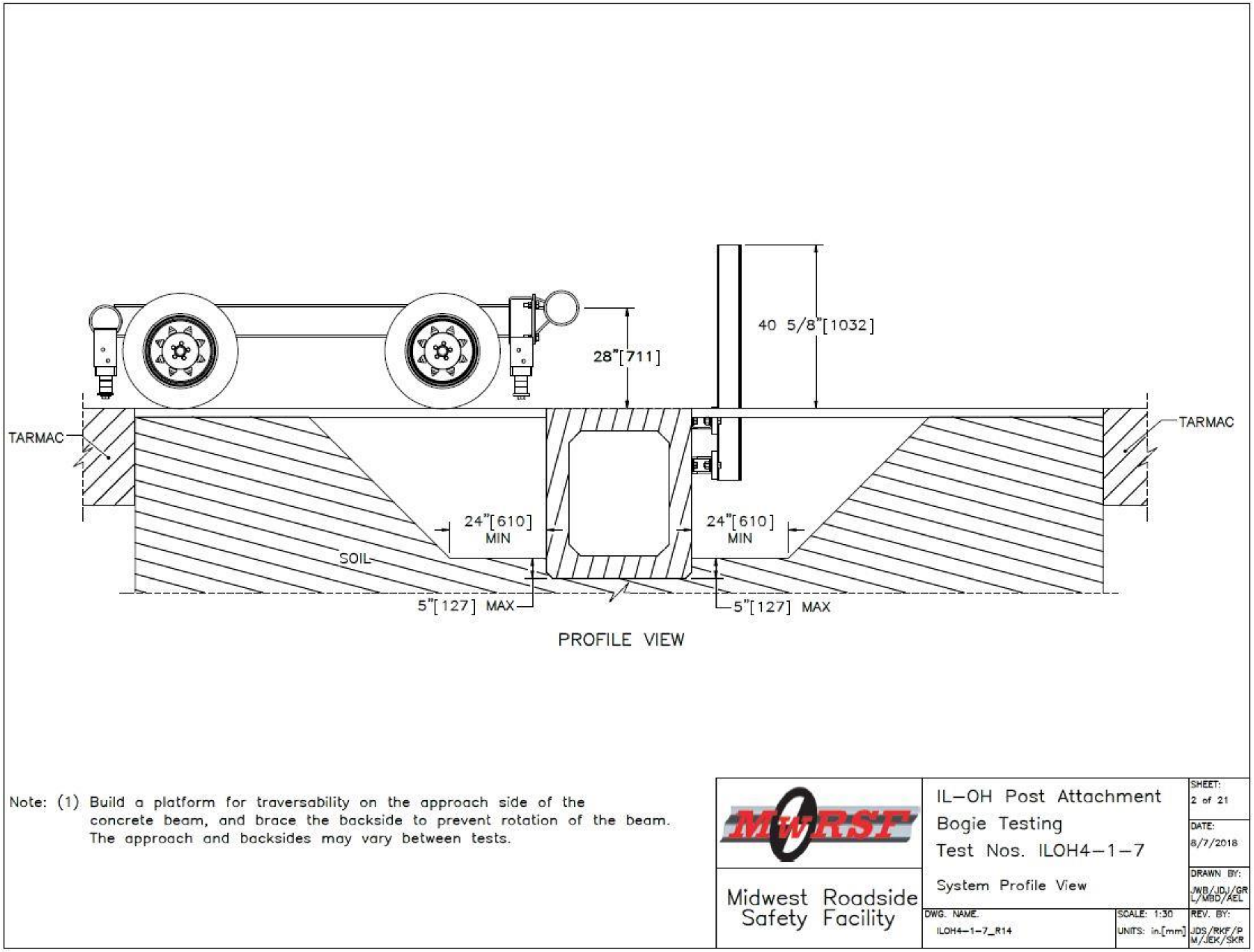


Figure 77. Bogie Testing Setup

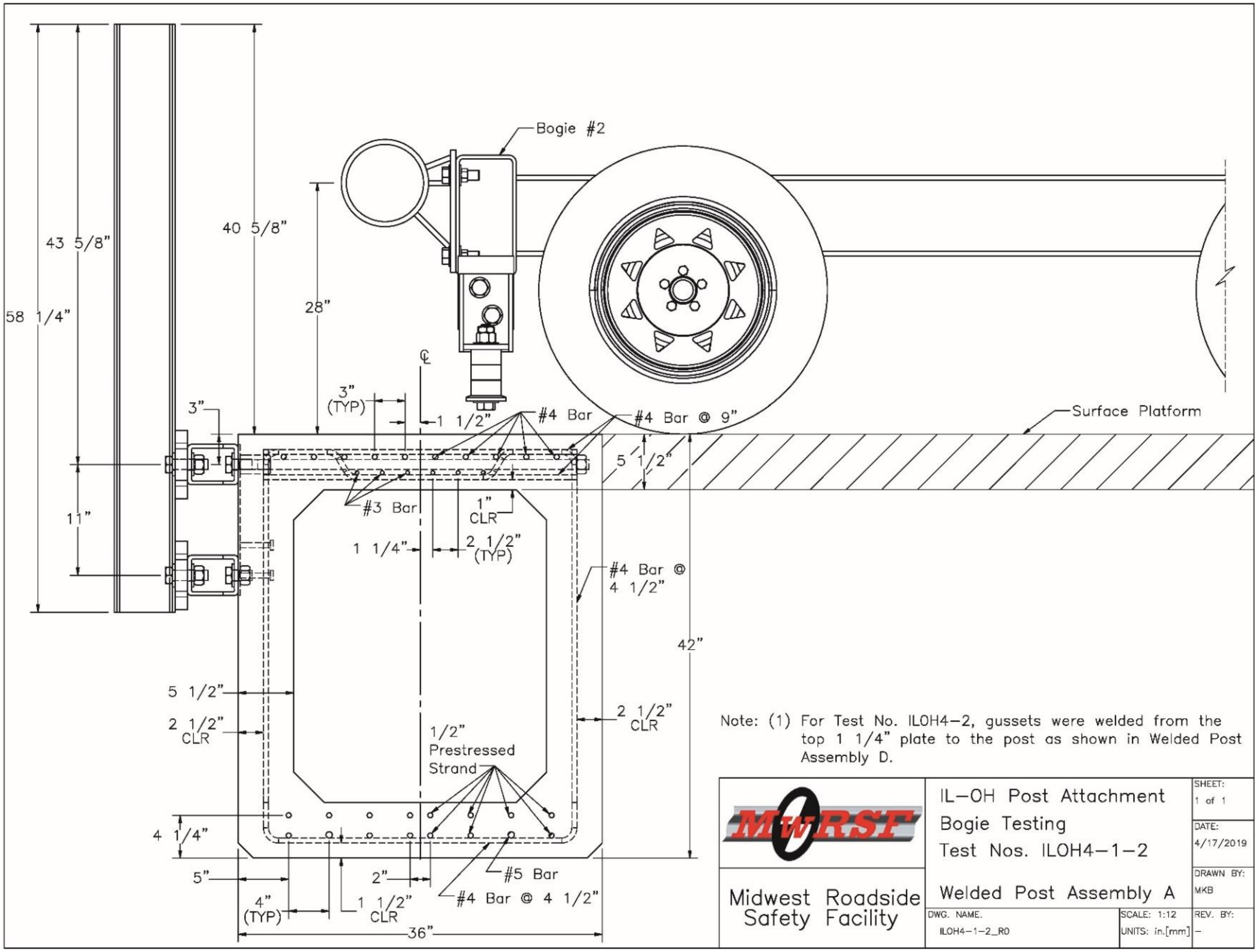


Figure 78. Test Configuration for Test Nos. ILOH4-1 and ILOH4-2

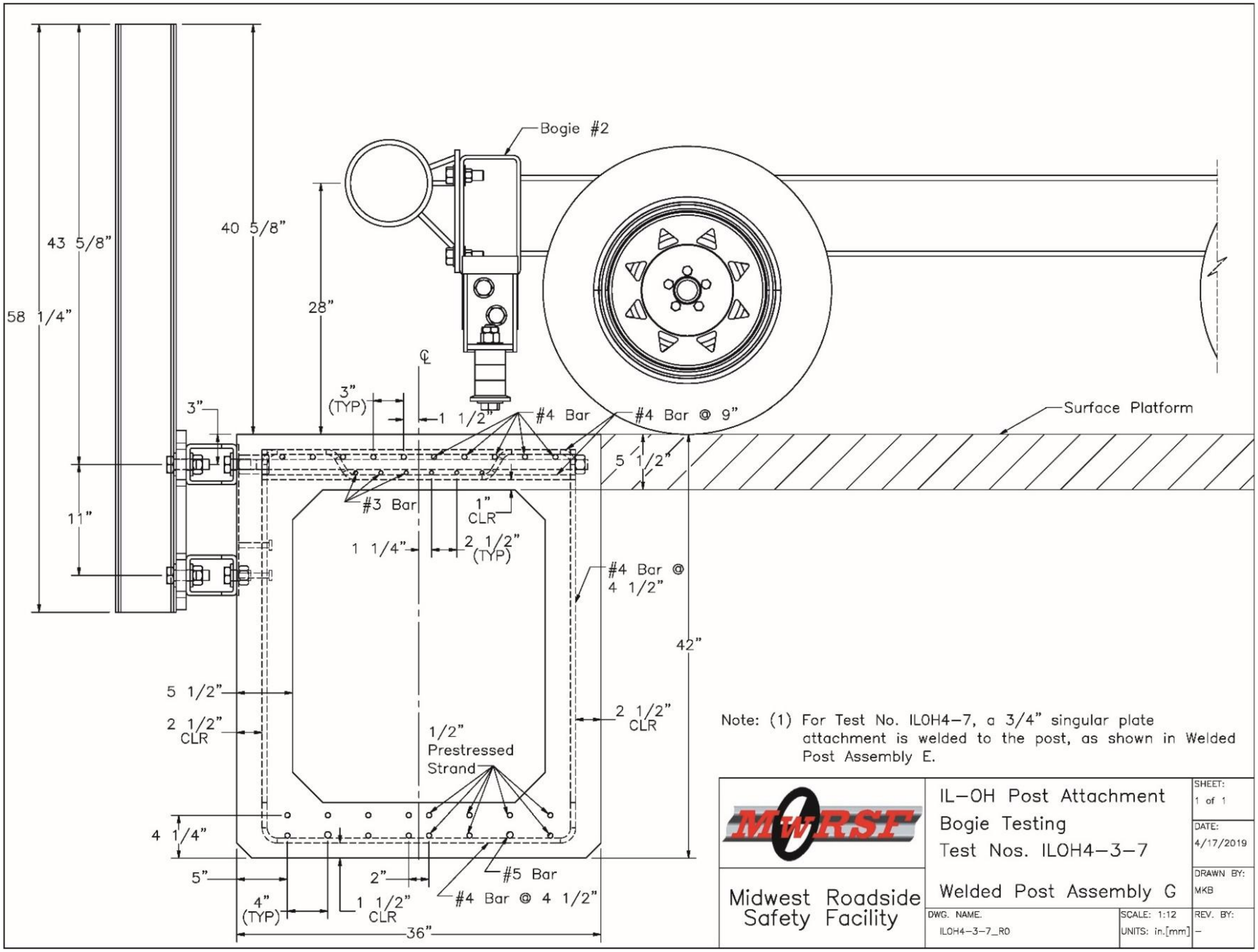


Figure 79. Test Configuration for Test Nos. ILOH4-3 through ILOH4-7

Test Name	Welded Post Assembly	Tube Spacers	Exterior Washers	Exterior Bolts	Interior Washers	Interior Bolts
ILOH4-1	A	a6	b5	b1	b5	b2
ILOH4-2	D	a6	b5	b1	b5	b2
ILOH4-3	G	a7	b5	b1	b5	b9
ILOH4-4	G	a7	b5	b1	b5	b9
ILOH4-5	G	a7	b5	b1	b5	b9
ILOH4-6	G	a7	b5	b1	b5	b9
ILOH4-7	E	a7	b5	b1	b5	b9

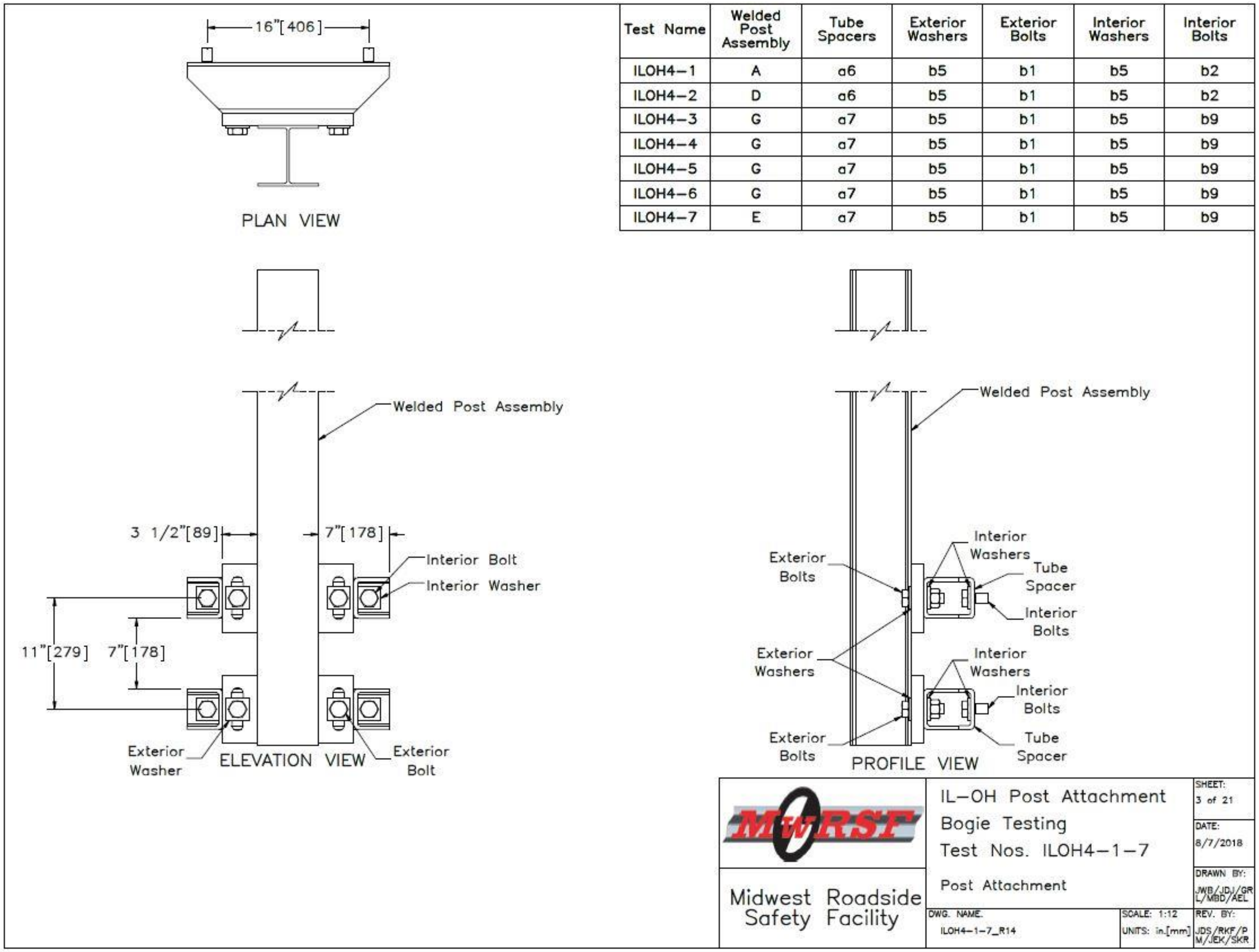
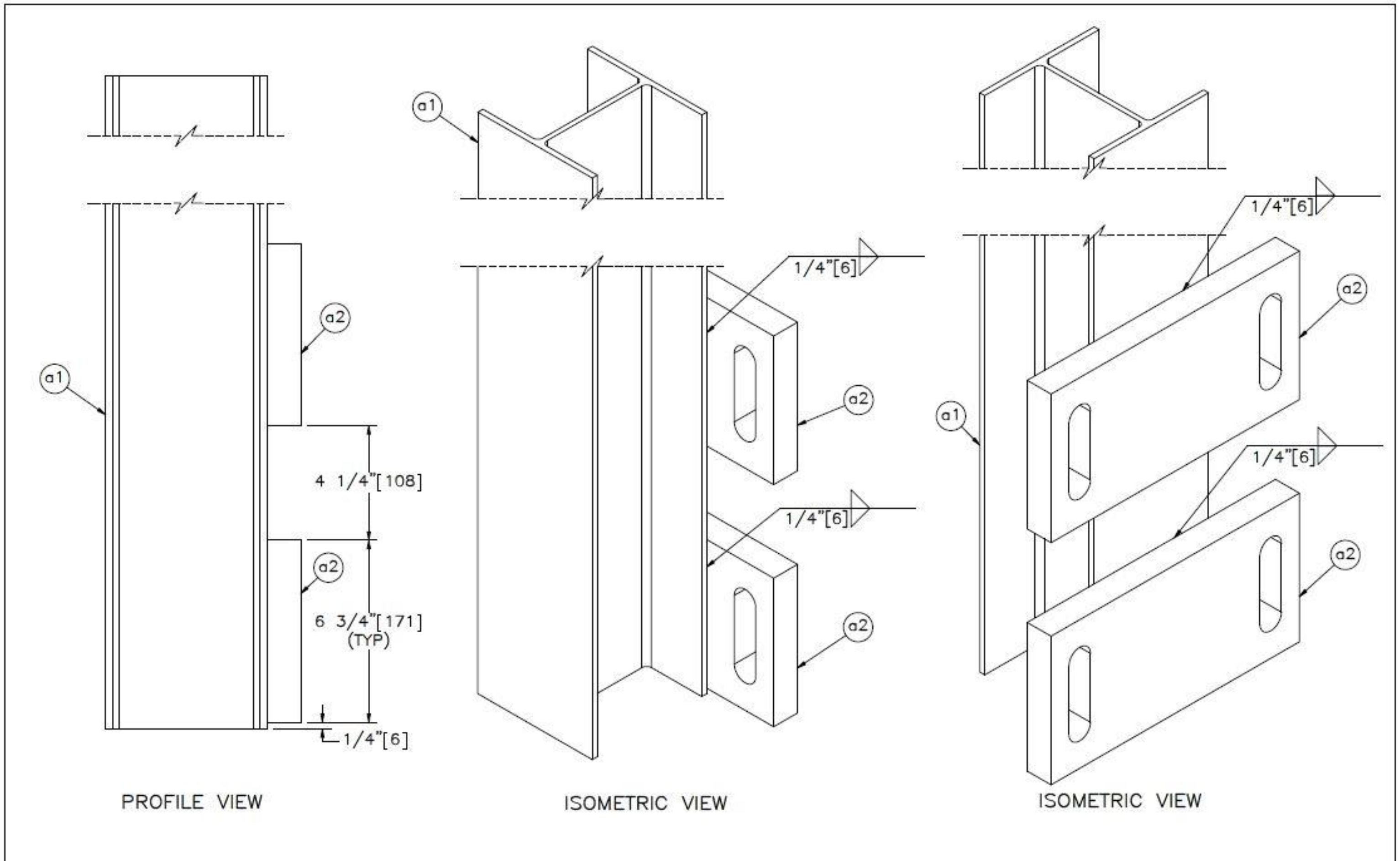


Figure 80. Post Attachment Testing Device



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Welded Post Assembly A	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
a2	2	13"x6 3/4"x1 1/4" [330x171x32] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–

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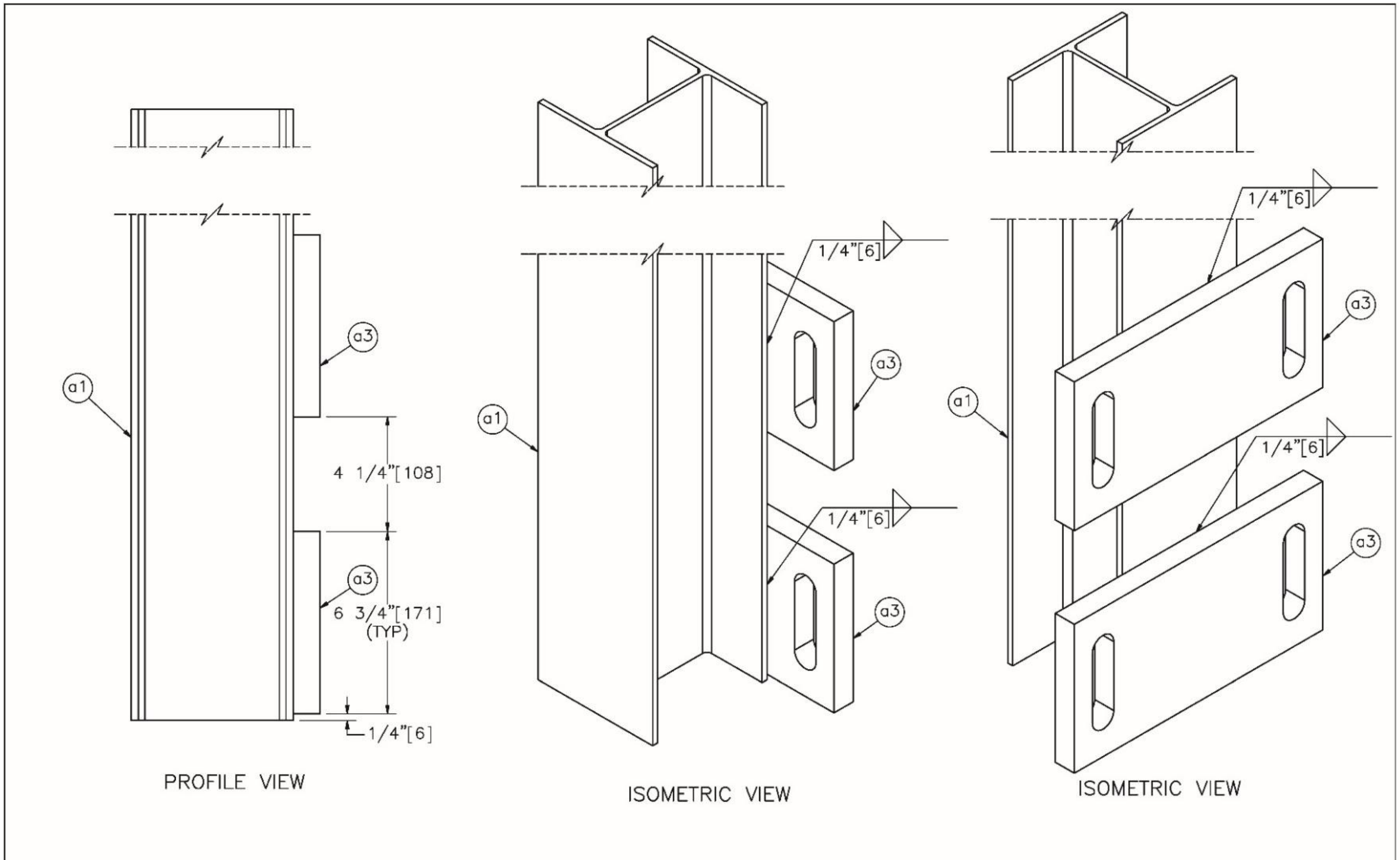
Midwest Roadside Safety Facility

IL–OH Post Attachment
Bogie Testing
Test Nos. ILOH4–1–7
Welded Post Assembly A Details

SHEET: 4 of 21
DATE: 8/7/2018
DRAWN BY: JWB/JDJ/GRL/MBD/AEL
DWG. NAME. ILOH4–1–7_R14
SCALE: 1:5
UNITS: in.[mm]
REV. BY: JDS/RKF/PM/JEK/SKR

Figure 81. Welded Post Assembly A Details, Test No. ILOH4-1



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	0	Welded Post Assembly B	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
a3	2	13"x6 3/4"x1" [330x171x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–

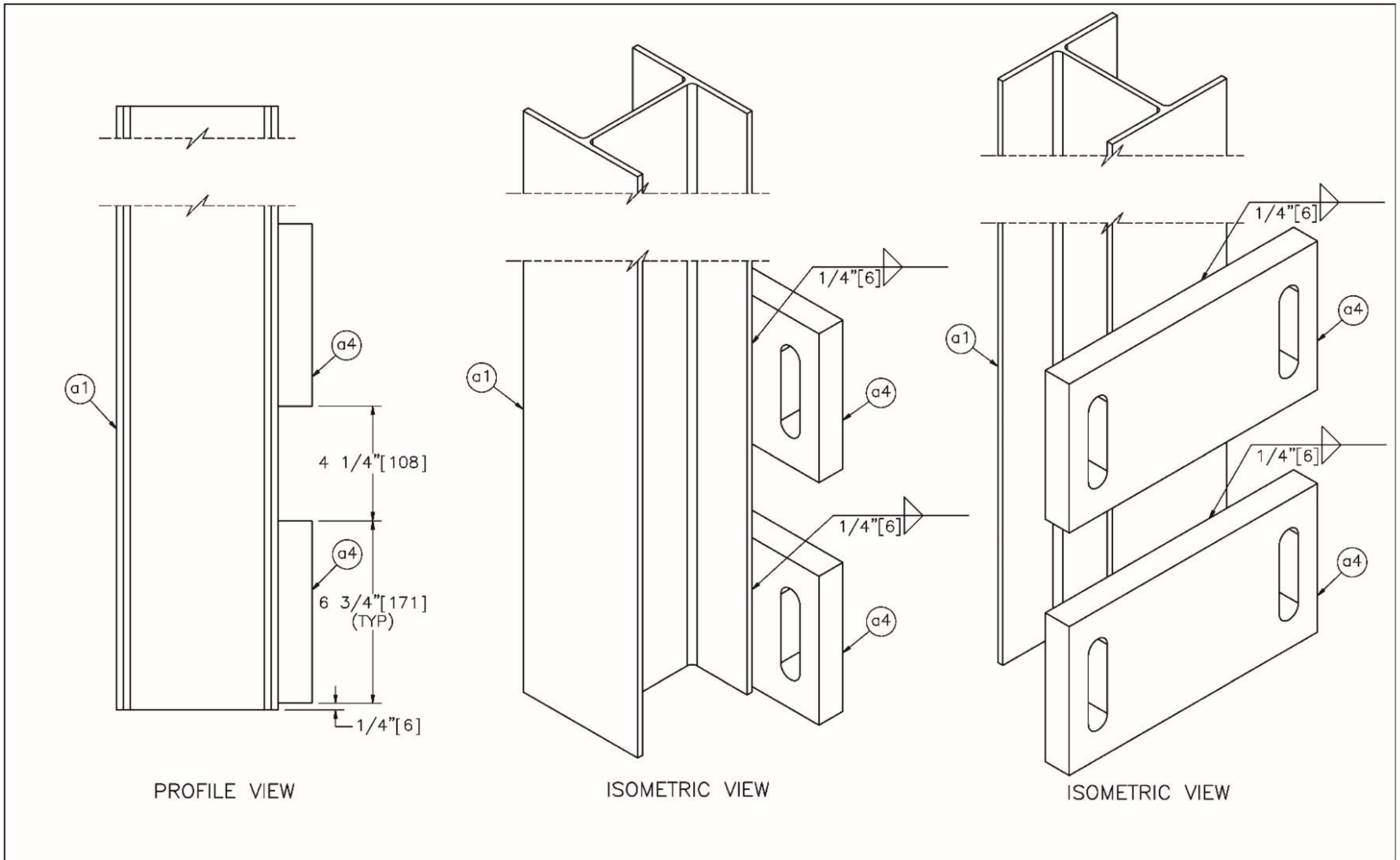
Midwest Roadside Safety Facility

IL-OH Post Attachment Bogie Testing Test Nos. ILOH4-1-7

Welded Post Assembly B Details

SHEET: 5 of 21
DATE: 8/7/2018
DRAWN BY: JWB/JDJ/GRL/MBD/AEL
DWG. NAME. ILOH4-1-7_R14
SCALE: 1:5
UNITS: in.[mm]
REV. BY: JDS/RKF/PM/JEK/SKR

Figure 82. Welded Post Assembly B Details



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	0	Welded Post Assembly C	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
a4	2	13"x6 3/4"x1 1/4" [330x171x32] Post Plate with Slots for 7/8" [22] Bolts	ASTM A572 Gr. 50	–

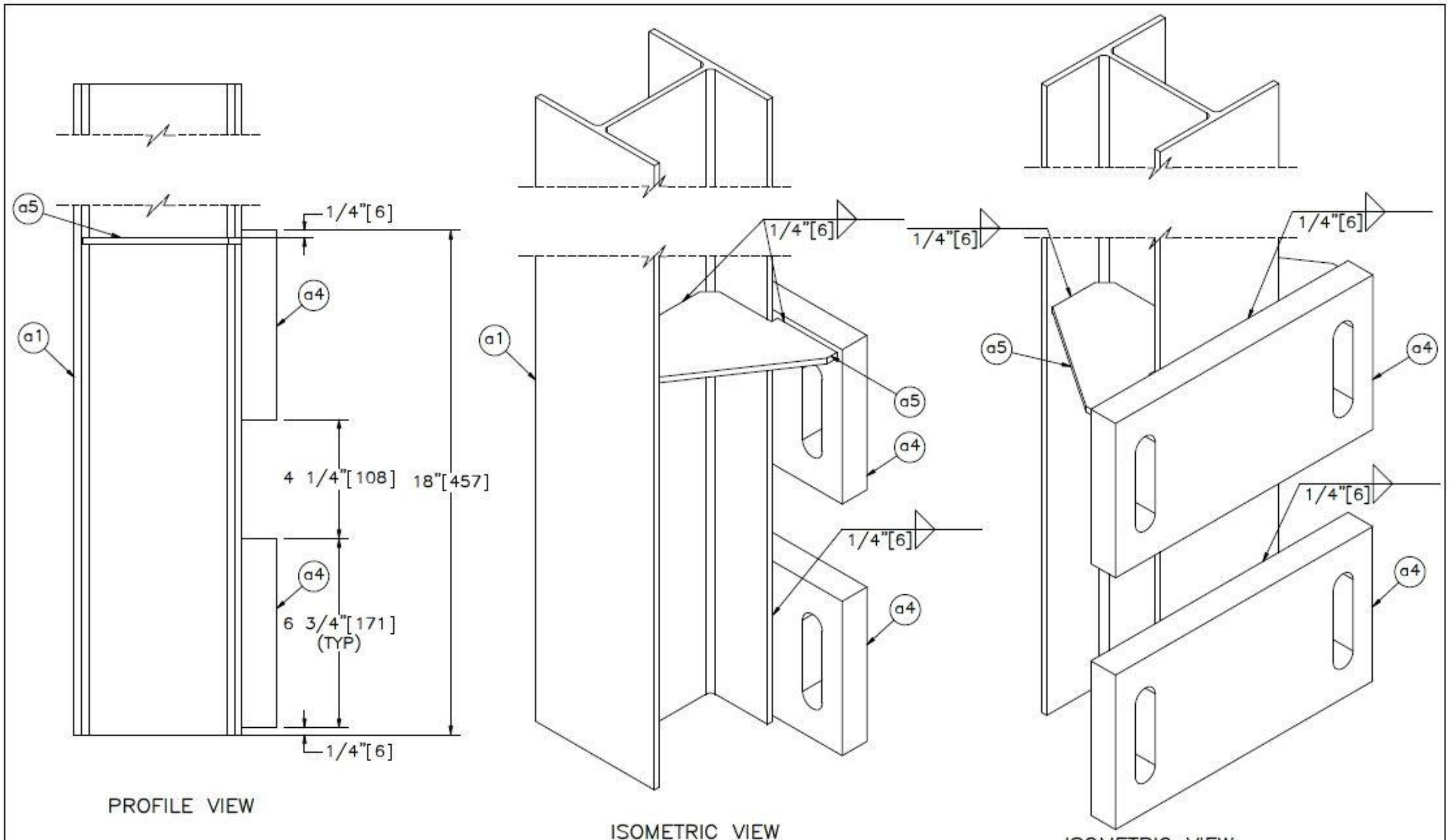
Midwest Roadside Safety Facility

IL–OH Post Attachment
Bogie Testing
Test Nos. ILOH4–1–7

Welded Post Assembly C Details

SHEET: 6 of 21
DATE: 8/7/2018
DRAWN BY: JWB/JDJ/GRL/MBD/AEL
DWG. NAME. ILOH4–1–7_R14
SCALE: 1:5
UNITS: in.[mm]
REV. BY: JDS/RKF/PM/JEK/SKR

Figure 83. Welded Post Assembly C Details

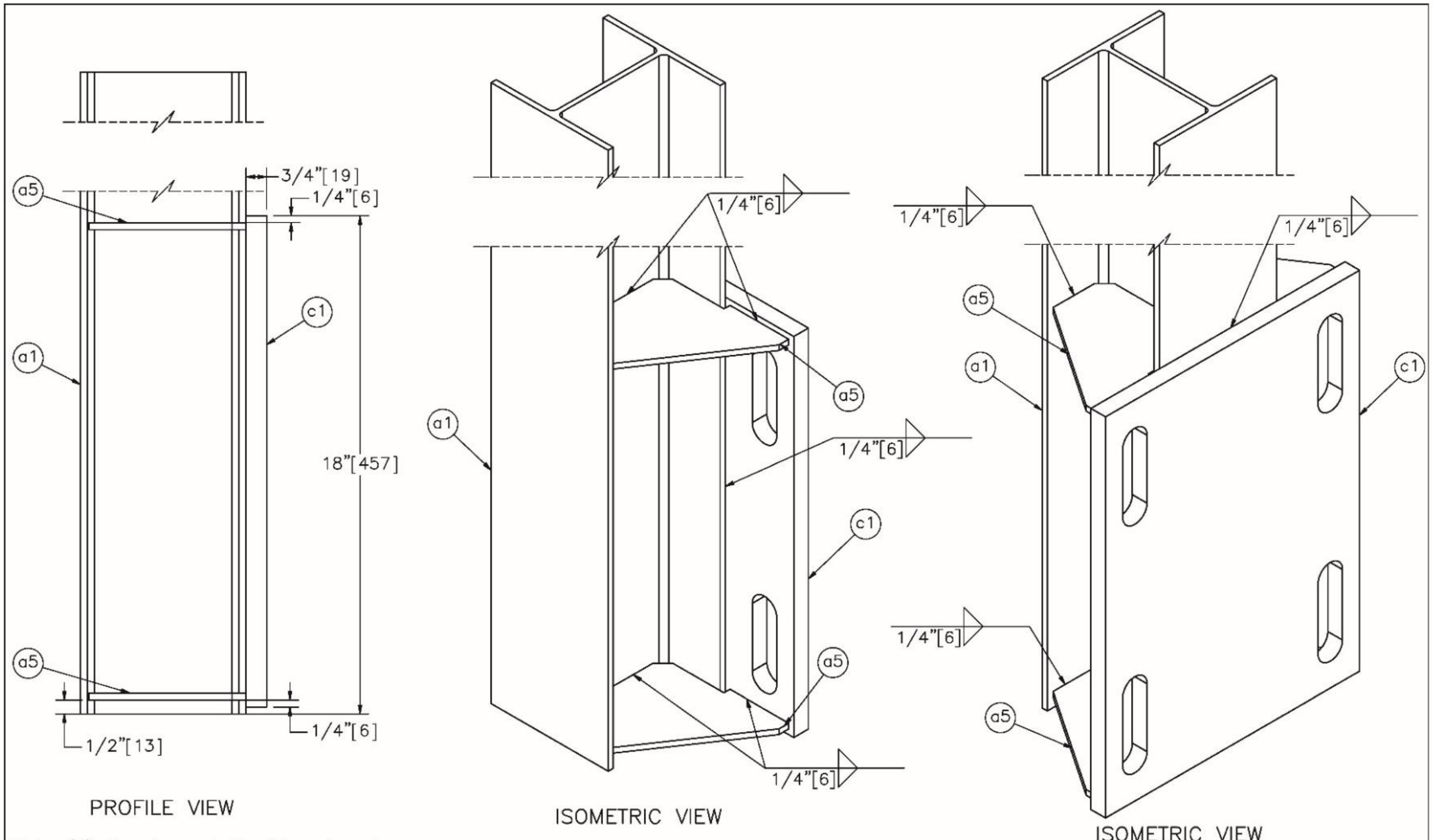


Note: (1) Gusset on both sides of post.
(2) Gussets not welded to front flange of post.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Welded Post Assembly D	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
a4	2	13"x6 3/4"x1 1/4" [330x171x32] Post Plate with Slots for 7/8" [22] Bolts	ASTM A572 Gr. 50	–
a5	2	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	–

Midwest Roadside Safety Facility	IL–OH Post Attachment Bogie Testing Test Nos. ILOH4–1–7	SHEET: 7 of 21
		DATE: 8/7/2018
	Welded Post Assembly D Details	DRAWN BY: JWB/JDJ/GRL/MBD/AEL
DWG. NAME. ILOH4–1–7_R14	SCALE: 1:5 UNITS: in.[mm]	REV. BY: JDS/RKF/PM/JEK/SKR

Figure 84. Welded Post Assembly D Details, Test No. ILOH4-2



Note: (1) Gussets on both sides of post.
(2) Gussets not welded to front flange of post.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Welded Post Assembly E	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
c1	1	13"x17 3/4"x3/4" [330x451x19] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–
a5	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	–

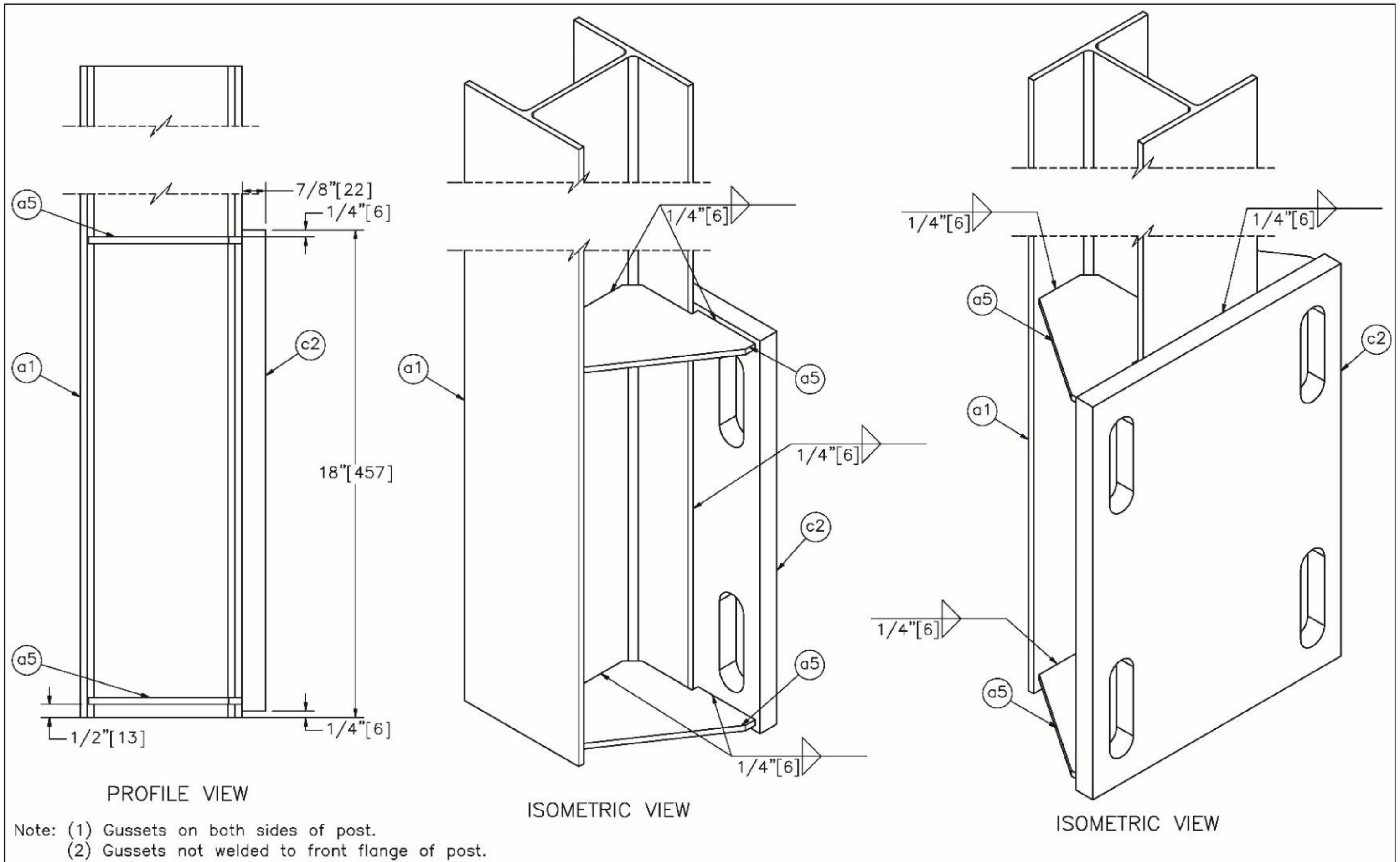
Midwest Roadside Safety Facility

IL–OH Post Attachment Bogie Testing Test Nos. ILOH4–1–7

Welded Post Assembly E Details

SHEET:	8 of 21
DATE:	8/7/2018
DRAWN BY:	JWB/JDJ/GRL/MBD/AEL
DWG. NAME.	ILOH4–1–7_R14
SCALE:	1:5
UNITS:	in.[mm]
REV. BY:	JDS/RKF/PM/JEK/SKR

Figure 85. Welded Post Assembly E Details, Test No. ILOH4-7



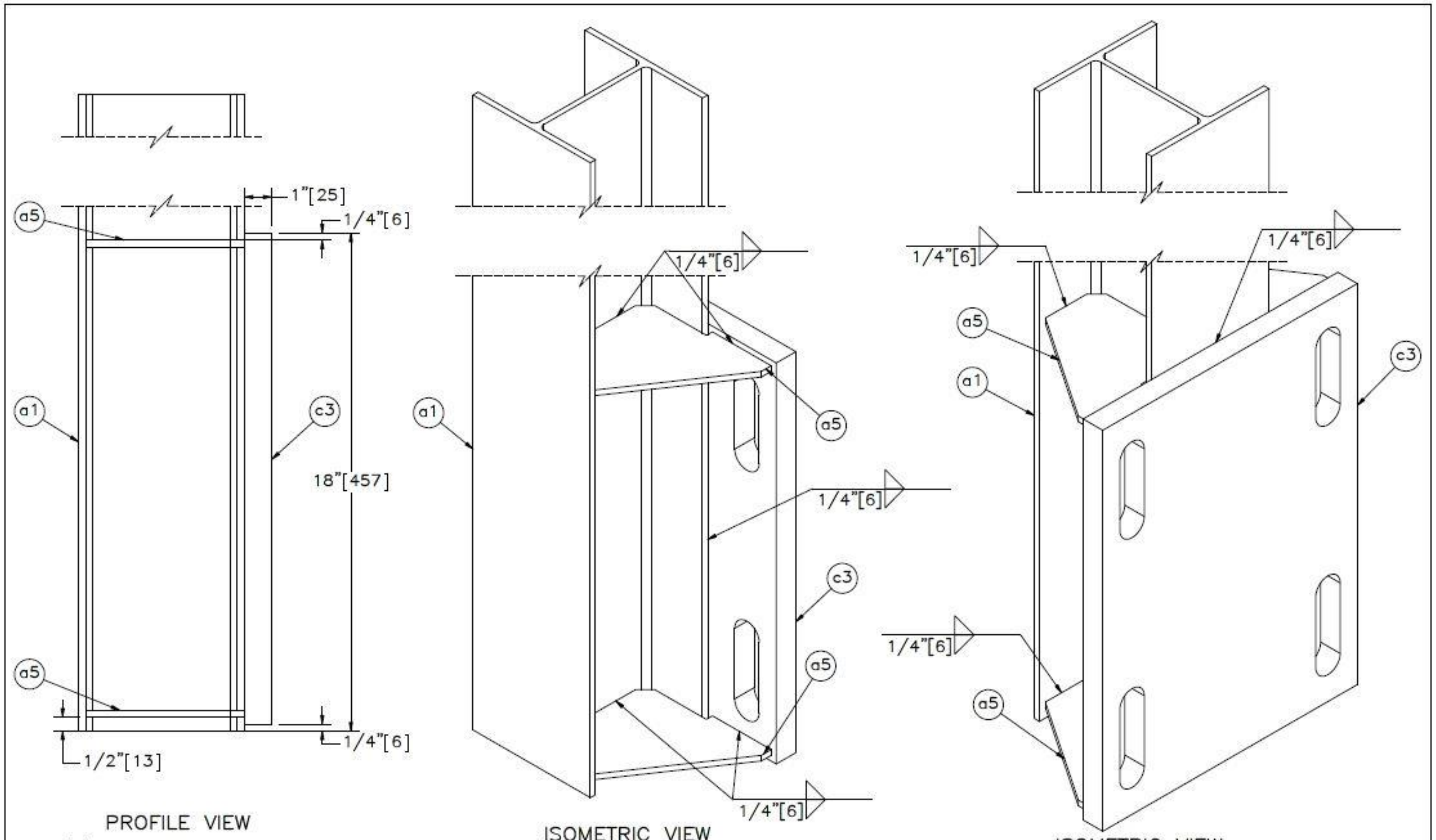
Item No.	QTY.	Description	Material Specification	Treatment Specification
–	0	Welded Post Assembly F	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
c2	1	13"x17 3/4"x7/8" [330x451x22] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–
a5	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	–

Midwest Roadside Safety Facility

IL–OH Post Attachment
Bogie Testing
Test Nos. ILOH4–1–7
Welded Post Assembly F Details

SHEET: 9 of 21
DATE: 8/7/2018
DRAWN BY: JWB/JDJ/GRL/MBD/AEL
REV. BY: JDS/RKF/PM/JEK/SKR
DWG. NAME. ILOH4–1–7_R14
SCALE: 1:5
UNITS: in.[mm]

Figure 86. Welded Post Assembly F Details



Note: (1) Gussets on both sides of post.
(2) Gussets not welded to front flange of post.

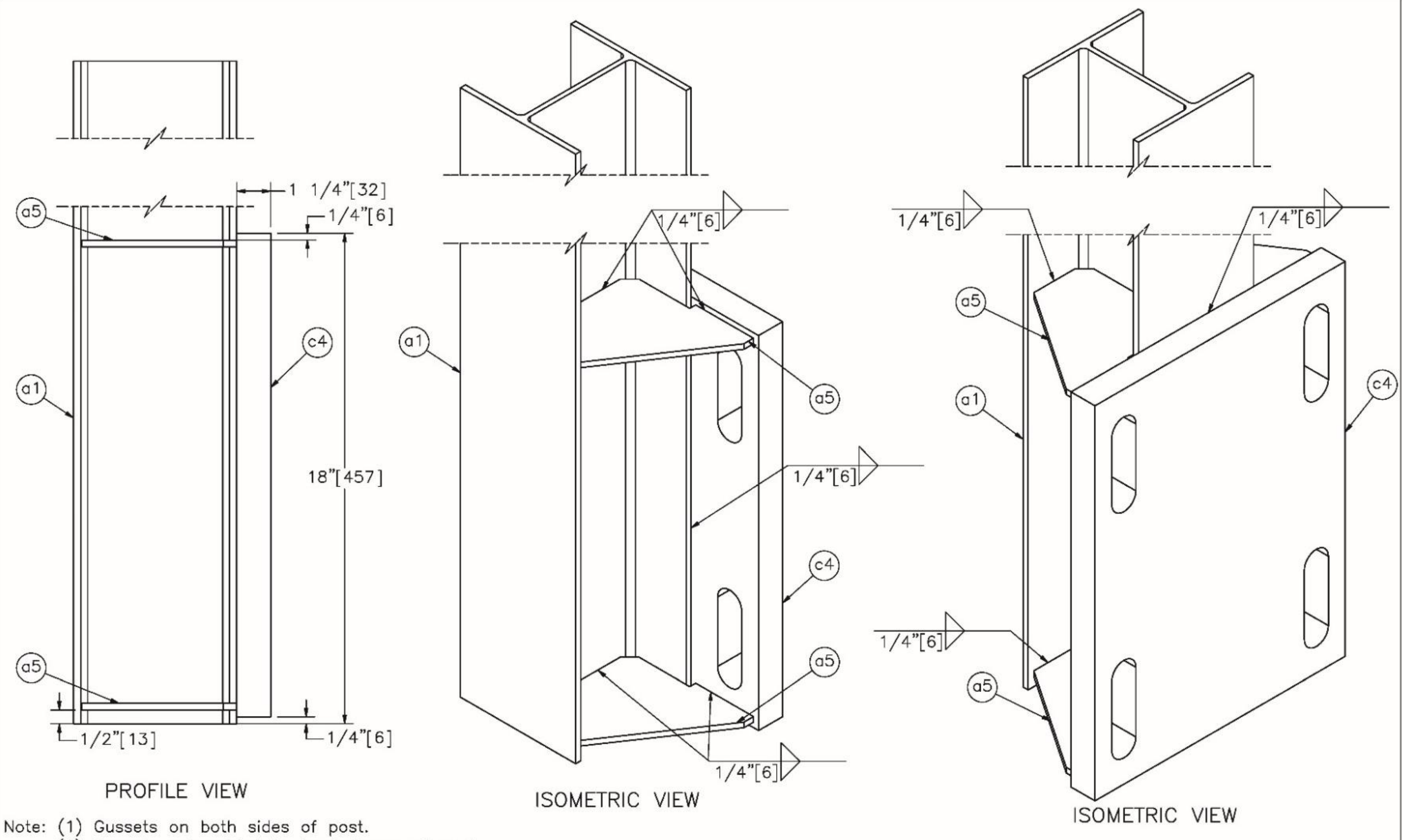
Item No.	QTY.	Description	Material Specification	Treatment Specification
–	4	Welded Post Assembly G	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
c3	1	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–
a5	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	–

Midwest Roadside Safety Facility

IL–OH Post Attachment
Bogie Testing
Test Nos. ILOH4–1–7
Welded Post Assembly G Details

SHEET: 10 of 21
DATE: 8/7/2018
DRAWN BY: JWB/JDJ/GRL/MBD/AEL
DWG. NAME. ILOH4–1–7_R14
SCALE: 1:5
UNITS: in.[mm]
REV. BY: JDS/RKF/PM/JEK/SKR

Figure 87. Welded Post Assembly G Details, Test Nos. ILOH4-3 through ILOH4-6



Note: (1) Gussets on both sides of post.
(2) Gussets not welded to front flange of post.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	0	Welded Post Assembly H	–	–
a1	1	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–
c4	1	13"x17 3/4"x1 1/4" [330x451x32] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–
a5	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	–

Midwest Roadside Safety Facility

IL-OH Post Attachment Bogie Testing
Test Nos. ILOH4-1-7

Welded Post Assembly H Details

SHEET:	11 of 21
DATE:	8/7/2018
DRAWN BY:	JWB/JDJ/GRL/MBD/AEL
DWG. NAME.	ILOH4-1-7_R14
SCALE:	1:5
UNITS:	in.[mm]
REV. BY:	JDS/RKF/PM/JEK/SKR

Figure 88. Welded Post Assembly H Details

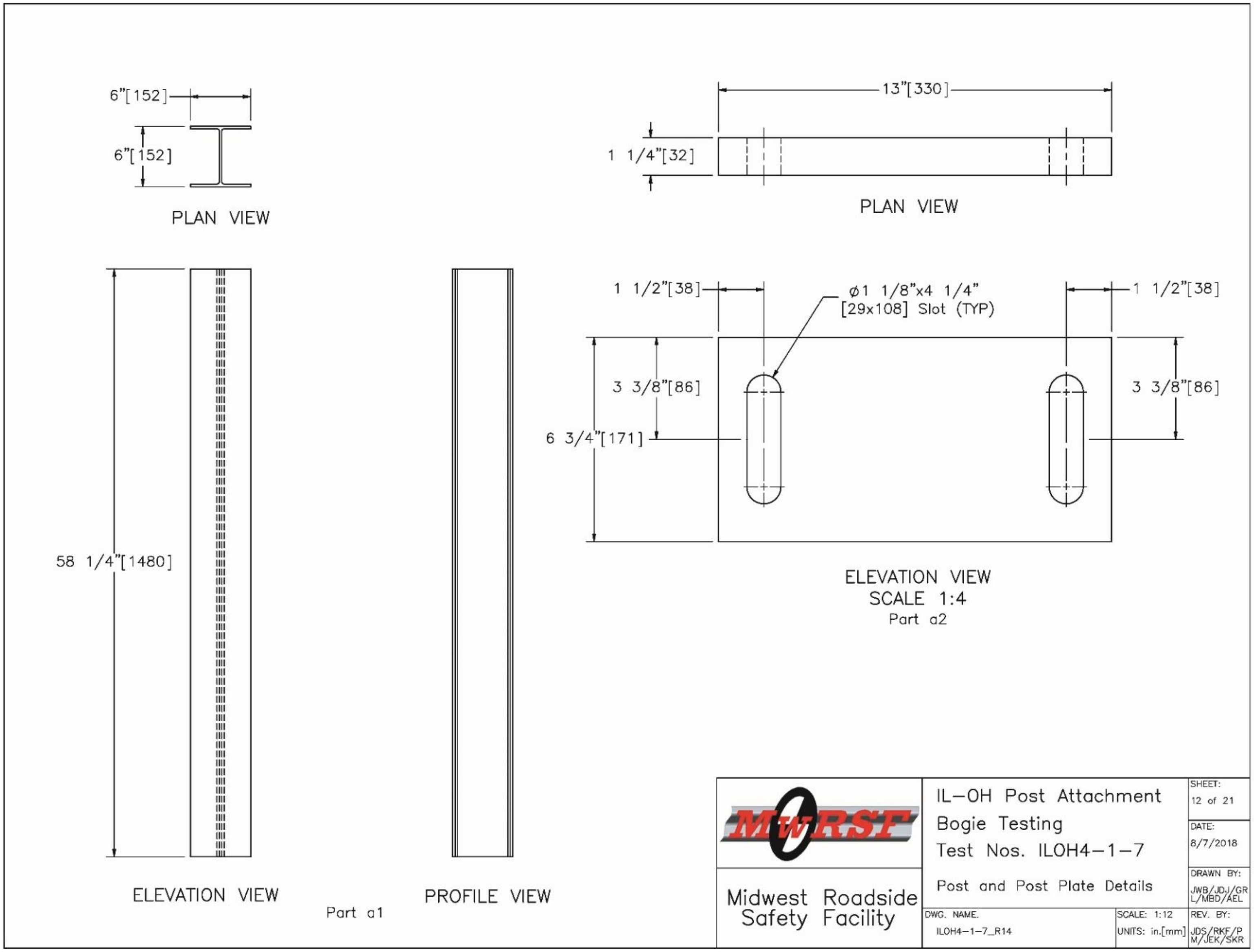


Figure 89. Post and 1¼-in. (32-mm) Post Plate Details

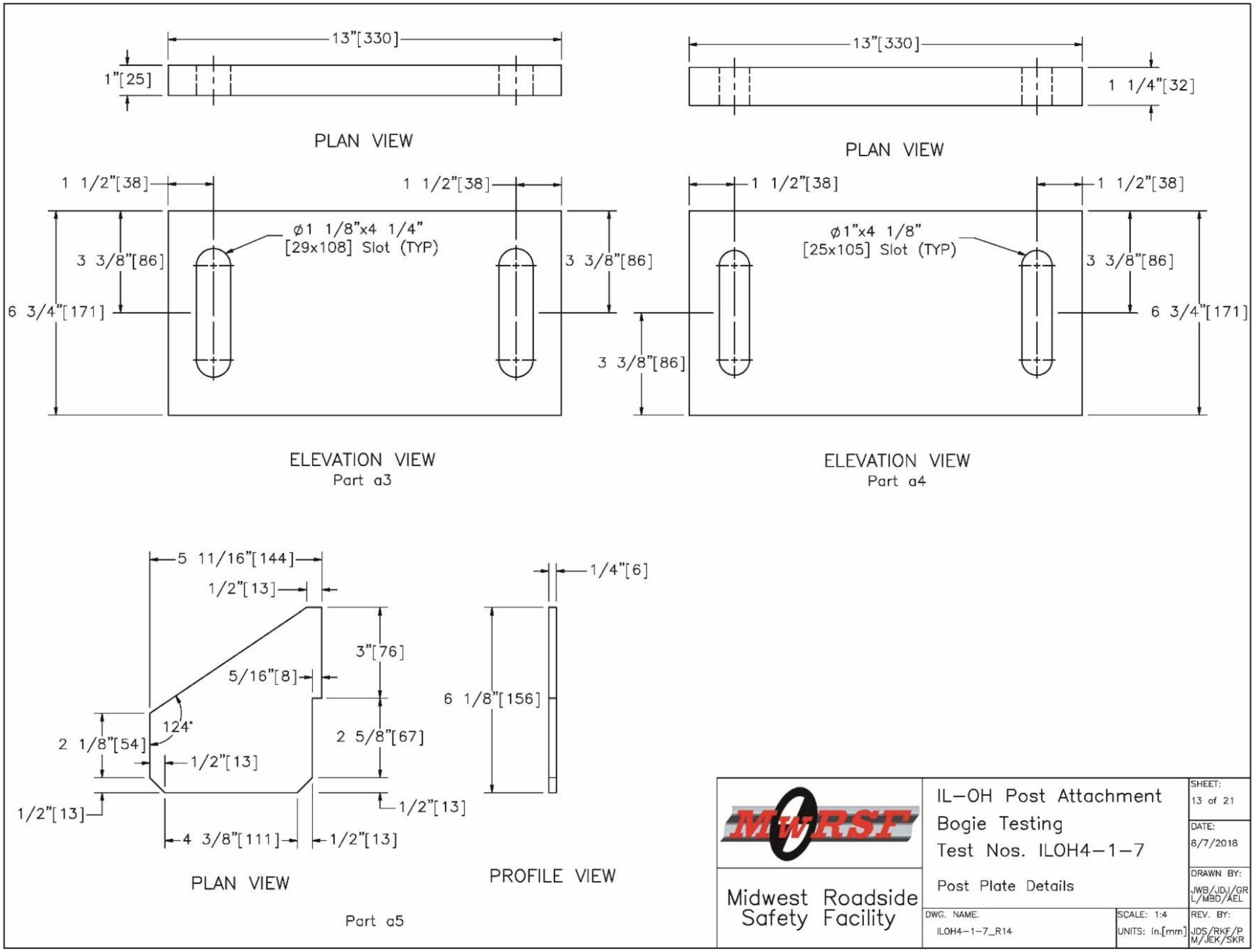


Figure 90. 1-in. (25-mm) and 1¼-in. (32-mm) Post Plate and Gusset Details

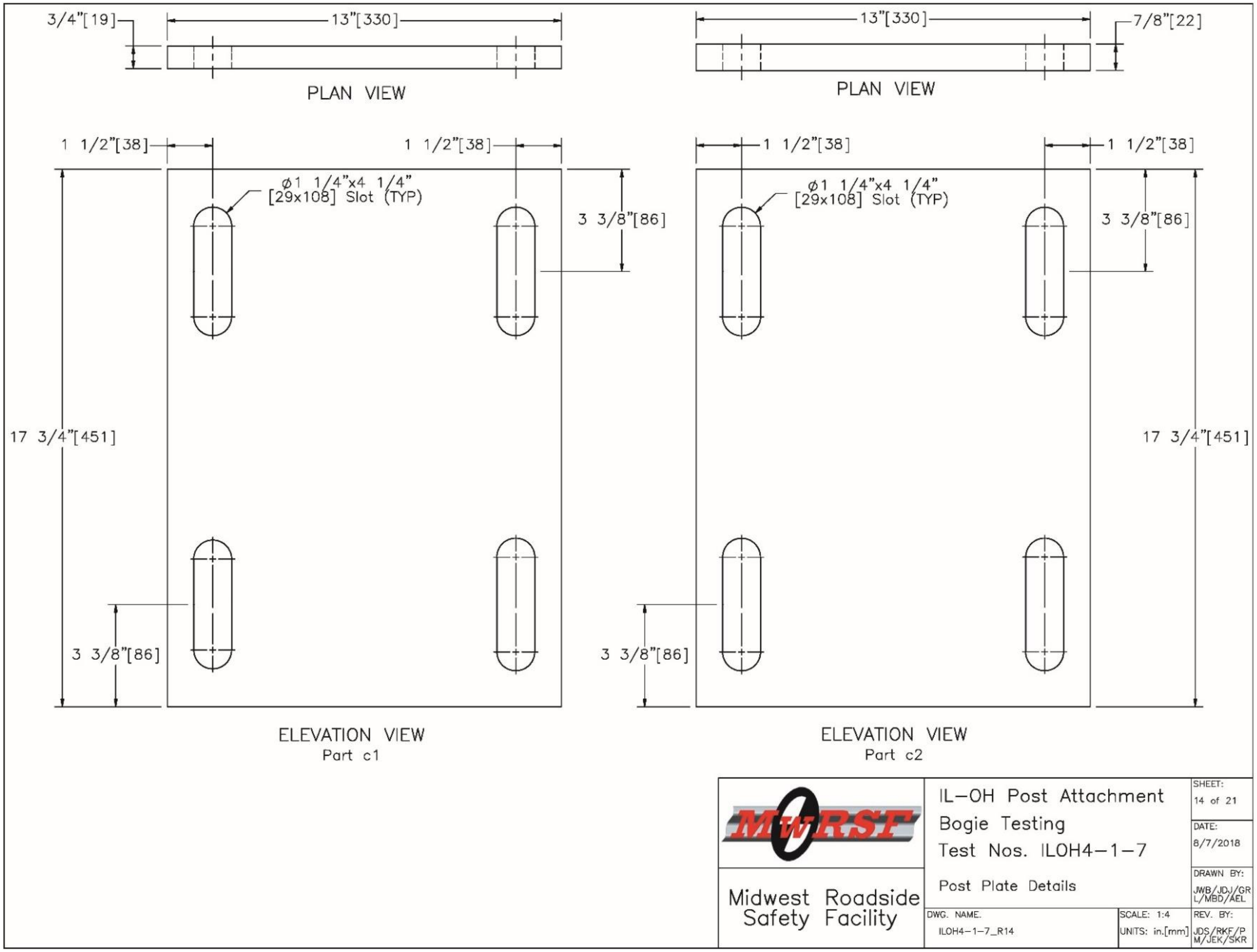


Figure 91. ¾-in. (19-mm) and ⅞-in. (22-mm) Singular Plate Attachment Details

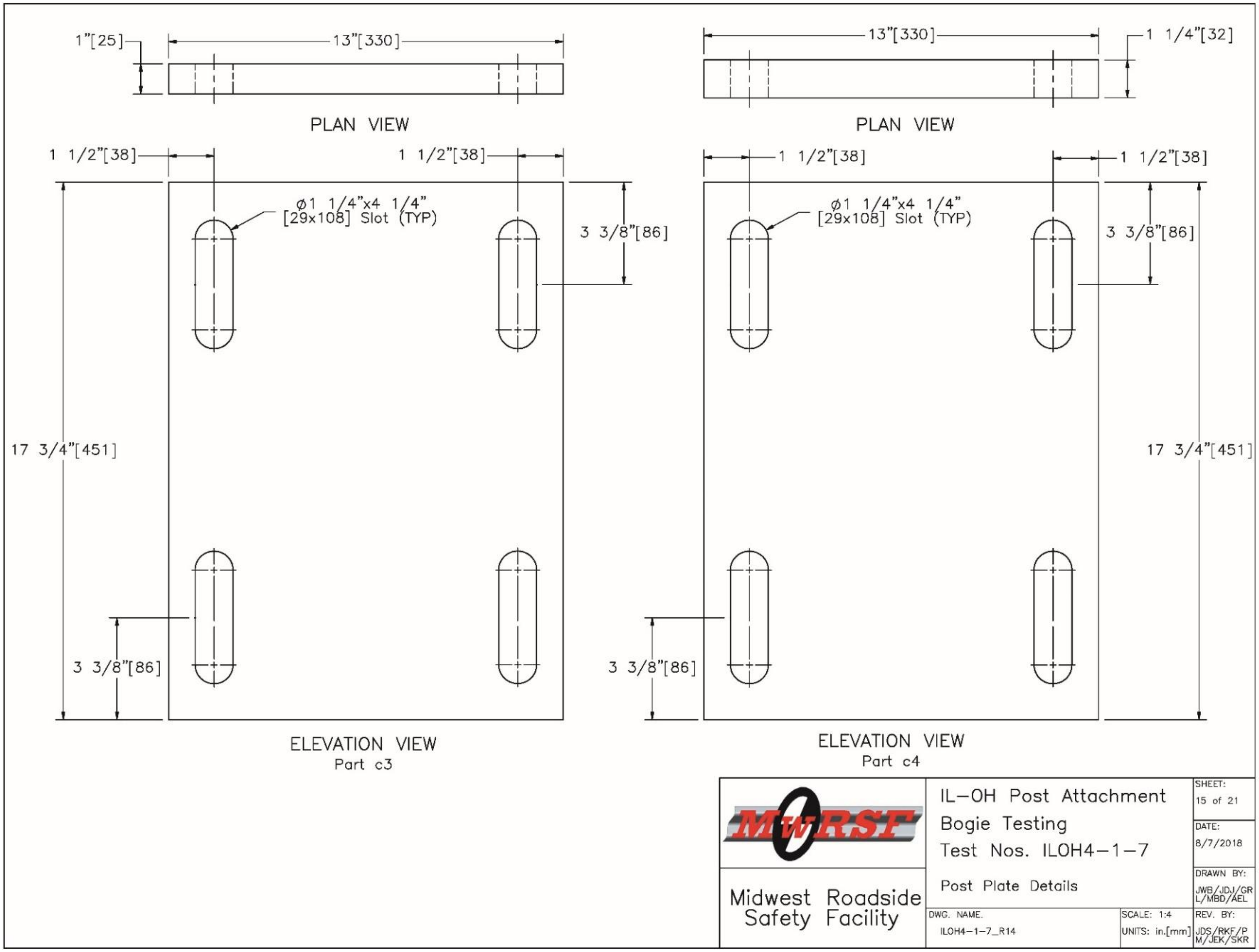


Figure 92. 1-in. (25-mm) and 1¼-in. (32-mm) Singular Plate Attachment Details

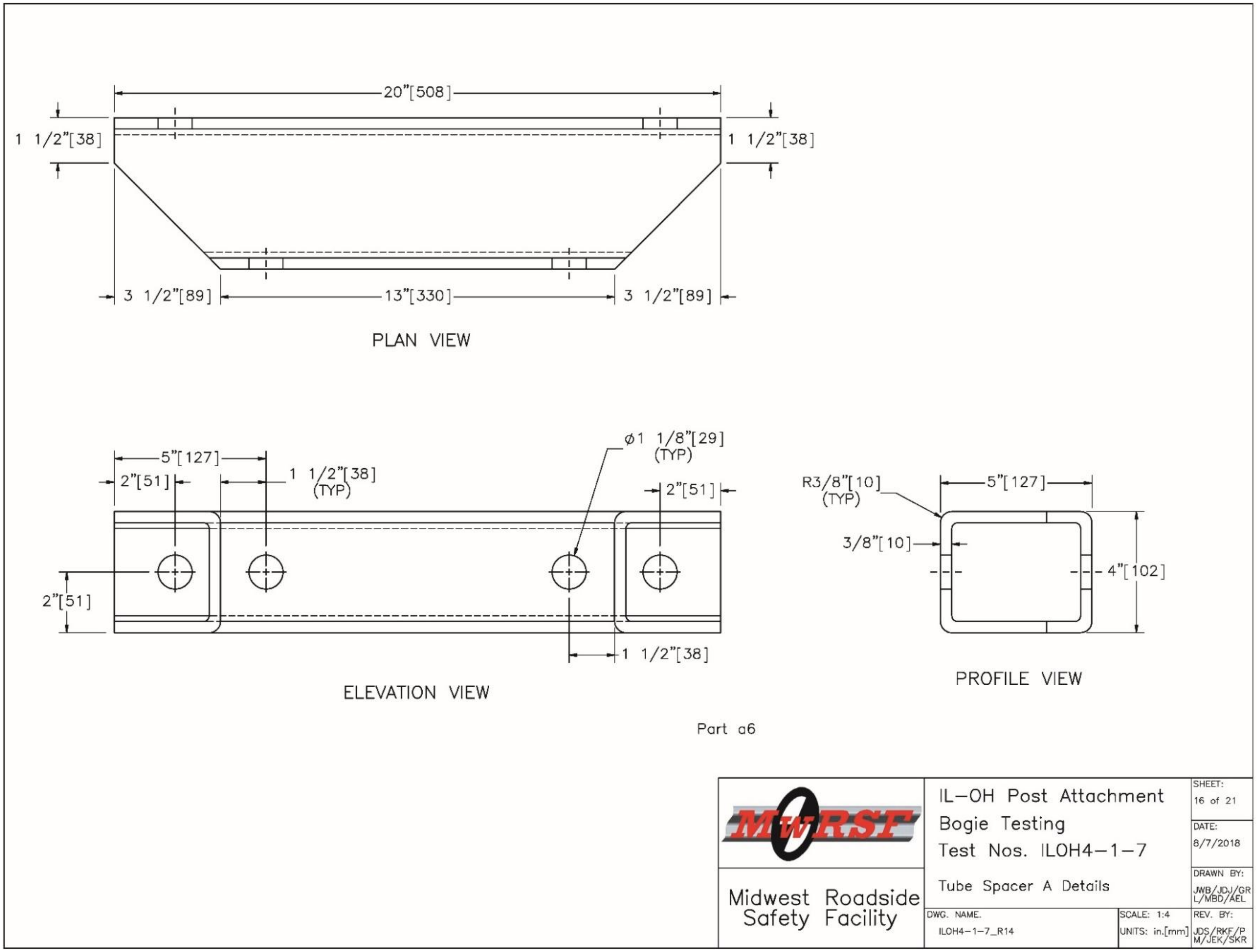


Figure 93. Tube Spacer A Details, HSS5x4x⅜

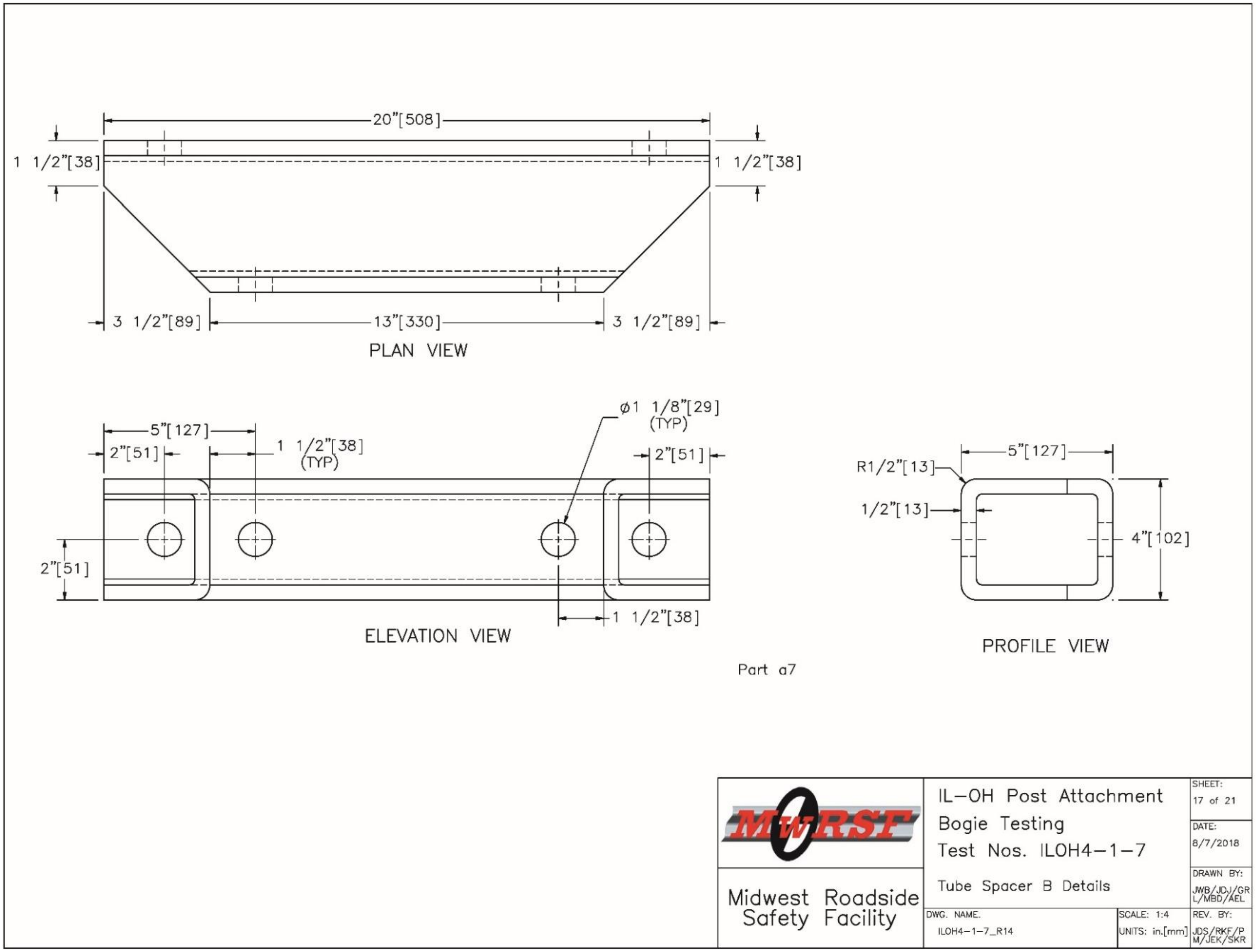


Figure 94. Tube Spacer B Details, HSS5x4x½

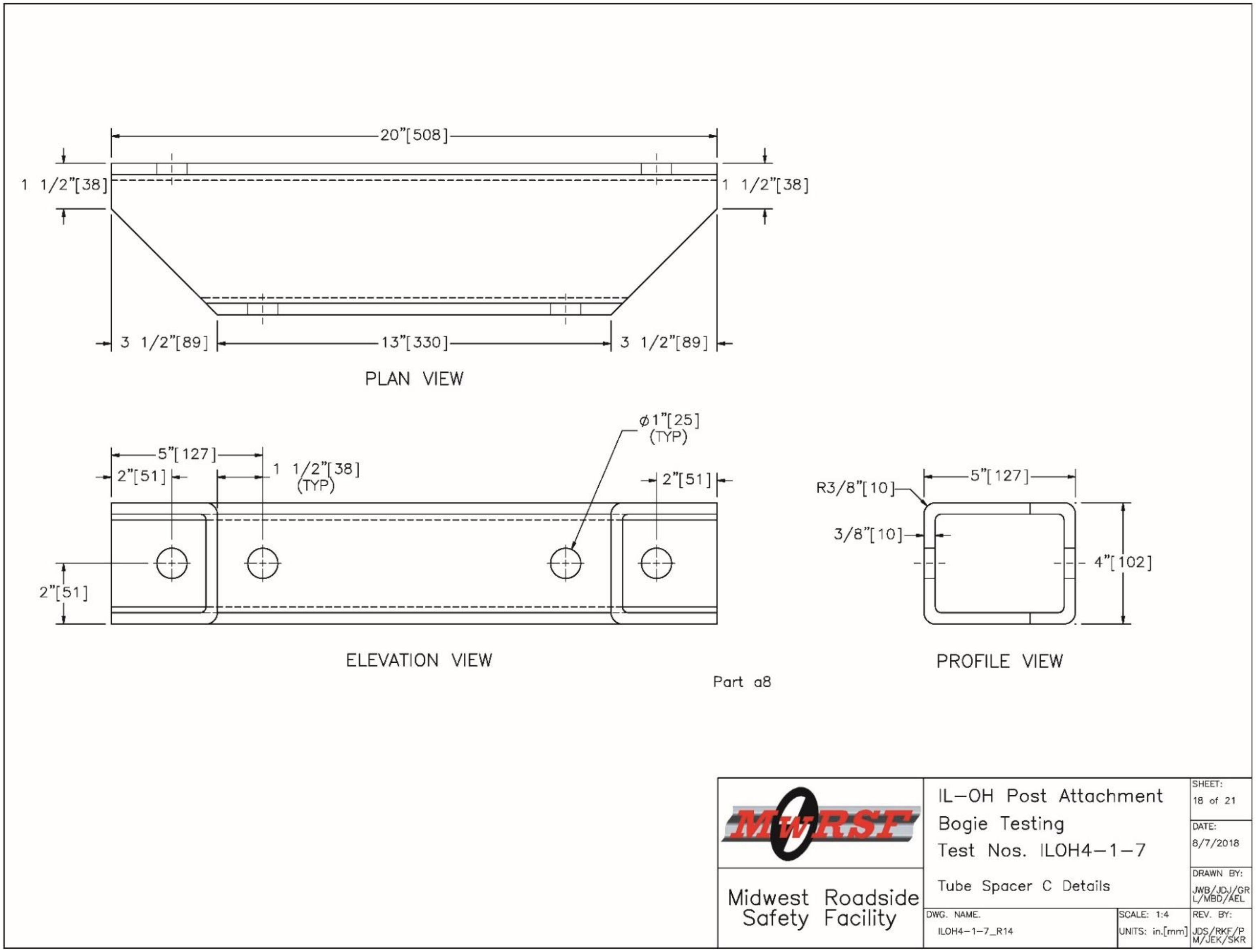


Figure 95. Tube Spacer C Details, HSS5x4x⅜ with 1-in. (25-mm) Diameter Bolt Holes

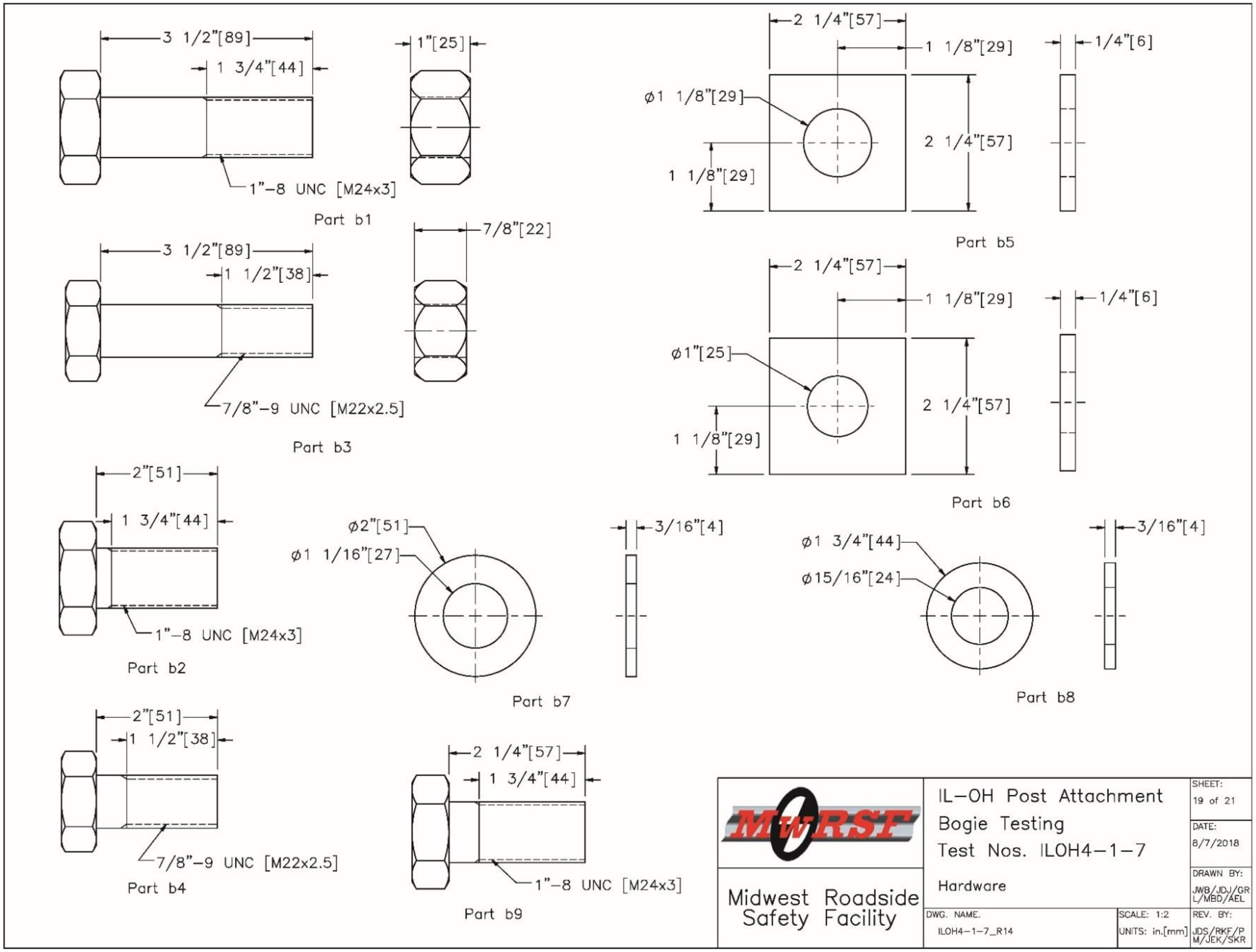


Figure 96. Bolt and Washer Details

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
a1	7	W6x15 [W152x22], 58 1/4" [1,480] Long	ASTM A992	–	–
a2	2	13"x6 3/4"x1 1/4" [330x171x32] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–	–
a3	0	13"x6 3/4"x1" [330x171x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–	–
a4	2	13"x6 3/4"x1 1/4" [330x171x32] Post Plate with Slots for 7/8" [22] Bolts	ASTM A572 Gr. 50	–	–
a5	22	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	–	–
a6	4	HSS 5"x4"x3/8" [127x102x10], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	ASTM A123*	–
a7	10	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	ASTM A123*	–
a8	0	HSS 5"x4"x3/8" [127x102x10], 20" [508] Long with 1" [25] Holes	ASTM A500 Gr. C	ASTM A123*	–
b1	28	1"–8 UNC [M24x3], 3 1/2" [89] Long Heavy Hex Head Bolt and Nut	Bolt – ASTM F3125 Gr. A325 Type 1 Nut – ASTM A563DH	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329 or F2833 Gr. 1	FBX24b
b2	8	1"–8 UNC [M24x3], 2" [51] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329 or F2833 Gr. 1	FBX24b
b3	0	7/8"–9 UNC [M22x2.5], 3 1/2" [89] Long Heavy Hex Head Bolt and Nut	Bolt – ASTM F3125 Gr. A325 Type 1 Nut – ASTM A563DH	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329 or F2833 Gr. 1	FBX22b
b4	0	7/8"–9 UNC [M22x2.5], 2" [51] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329 or F2833 Gr. 1	FBX22b
b5	84	1" [25] Square Washer	ASTM A36	ASTM A123*	–
b6	0	7/8" [22] Square Washer	ASTM A36	ASTM A123*	–
b7	0	1" [25] Dia. Hardened Round Washer	ASTM F436	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329	FWC24b
b8	0	7/8" [22] Dia. Hardened Round Washer	ASTM F436	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329	FWC22b
b9	20	1"–8 UNC [M24x3], 2 1/4" [57] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329 or F2833 Gr. 1	FBX24b

* If time to galvanize will significantly hinder time to receive parts, then the parts can just be primed.

Midwest Roadside Safety Facility

IL–OH Post Attachment Bogie Testing
Test Nos. ILOH4–1–7

Bill of Materials

SHEET: 20 of 21
DATE: 8/7/2018
DRAWN BY: JWB/JDJ/GRL/MBD/AEL
DWG. NAME. ILOH4–1–7_R14
SCALE: None
UNITS: in.[mm]
REV. BY: JDS/RKF/PM/JEK/SKR

Figure 97. Bill of Materials, Sheet 1 of 2

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
c1	1	13"x17 3/4"x3/4" [330x451x19] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–	–
c2	0	13"x17 3/4"x7/8" [330x451x22] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–	–
c3	4	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–	–
c4	0	13"x17 3/4"x1 1/4" [330x451x32] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	–	–

Midwest Roadside Safety Facility

IL–OH Post Attachment
Bogie Testing
Test Nos. ILOH4–1–7

Bill of Materials

SHEET: 21 of 21	DATE: 8/7/2018	DRAWN BY: JWB/JDJ/GRL/MBD/AEL	REV. BY: JDS/RKF/PM/JEK/SKR
DWG. NAME. ILOH4–1–7_R14	SCALE: 1:96	UNITS: in.[mm]	

Figure 98. Bill of Materials, Sheet 2 of 2

6.3 Component Test Summary

Several post-to-deck connection designs were tested throughout the bogie testing program in an effort to optimize the design. A 12-in. (305-mm) post deflection was of interest due to the upcoming steel-tube bridge rail anticipating no more than 12 in. (305 mm) of deflection to prevent rollover of the SUT in the full-scale crash test. Furthermore, the 12-in. (305-mm) deflection was anticipated based on the literature review of previous crashworthy post-and-rail bridge rail deflections. Details regarding the 12-in. (305-mm) deflection threshold are stated in the conclusion and summary section of this chapter.

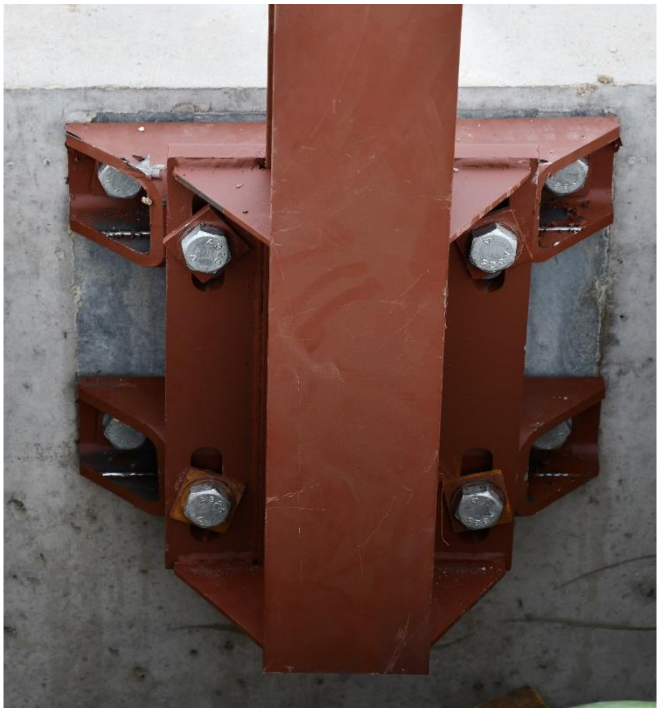
In the first two tests, the selected 1¼-in. (32-mm) two-plate attachment with HSS5x4x⅜ longitudinal tube spacers was evaluated, as shown in Figure 99. The initial design without gussets was tested as IDOT and ODOT preferred to have thick plate attachments welded to the post without stiffeners. After tensile weld failure of the top plate attachment, the post assembly was altered to include gussets in the top plate attachment to reinforce the welds to the post. In the second test, the reinforced plate held but the post deformed creating a plastic hinge between the upper and lower plate attachments with post web buckling at the bottom of the post and the top tube spacer bowing outward during impact. These deformities were a concern for causing additional post deflection since the plastic hinge on the post was not near the surface of the deck but further below the post, and large deflections are critical for SUT stability during impact.

In the remaining five tests, the post-to-deck connection was optimized to feature a singular plate attachment in place of the two-plate attachment for the welded post assembly, with increased tube spacer thickness to prevent bowing of its sidewalls, as shown in Figure 100. With the updated post assembly, the post-to-deck connection developed a plastic hinge in the post near the surface of the deck, and no anchorage or significant concrete damage was observed. A final post-to-deck connection design was selected to be used in the full-scale crash testing of the new steel-tube bridge rail.

(a) Test No. ILOH4-1

(b) Test No. ILOH4-2

Figure 99. Pre-test Assembly for (a) Test No. ILOH4-1 and (b) Test No. ILOH4-2



(a) Test Nos. ILOH4-3 thorough ILOH4-6

(b) Test No. ILOH4-7

Figure 100. Pre-test Assembly for (a) Test Nos. ILOH4-3 through ILOH4-6 and (b) Test No. ILOH4-7

Several installation issues emerged during the bogie testing program. In the pre-assembly stages for some designs, concrete spalling was observed around the embedded plate as the connection was attached to the deck. In post-test stages, spalling was more pronounced around the bottom half of the embedded plate. In some tests when attaching the tube spacers to the deck, two washers were utilized at each bolt connection due to bolts threading less than their anticipated thread length into the coupling nuts and heavy hex nuts cast in the concrete box-beam girder. It was determined that the anchor rods were threaded further than desired into the coupling nuts, preventing the bolts from providing a snug fit for the deck attachments. Finally, the anchor rods were designed to be fully threaded, however, the concrete box-beam girder was cast with solid bars with threaded ends during fabrication.

Along with the optimization of the plate attachments and tube spacers, the anchor rod embedment lengths were also evaluated. The smaller ⅞-in. (22-mm) diameter anchor rods were never tested. Investigations of the dynamically tested post-to-deck connection designs are described in detail in Chapter 7.

Prior to delivery of the box-beam girder, a ground pit was excavated for placement of the simulated concrete box-beam girder. The box-beam girder was situated in the middle of the pit and bracing, in the manner of wooden planks bolted to the girder's sidewalls and to the tarmac, was used to brace the girder and prevent unnecessary rotation during testing, as shown in Figure 77. The back side of the box-beam girder was braced adjacent to the testing location. Platforms were made and placed along the approach side for traversability of the bogie vehicle. Bracing of the concrete box-beam girder and use of platforms can be seen in Figure 101.

Figure 101. Bogie Test Setup

6.4 Test Facility

Physical testing of the side-mounted deck attachment to the box-beam girder was conducted at the MwRSF Outdoor Test Site, which is located at the Lincoln Air Park on the northwest side of the Lincoln Municipal Airport. The facility is approximately 5 miles (8 km) northwest from the University of Nebraska-Lincoln's City Campus

6.5 Equipment and Instrumentation

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

6.5.1 Bogie Vehicle

Two rigid-frame bogies were used to impact the posts. A variable height, detachable impact head was used in the testing. On each test vehicle, the bogie head was constructed of 8-in. (203-mm) diameter, ½-in. (13-mm) thick standard steel pipe, with ¾-in. (19-mm) neoprene belting wrapped around the pipe to prevent local damage to the post from the impact. The impact head was bolted to the bogie vehicles, creating a rigid frame with an impact height of 28 in. (711 mm).

Initially, a smaller bogie vehicle with a target weight of 2,000 lb (907 kg) was intended to be used for all tests, however, only the first two component tests were completed with the lighter bogie vehicle. Observation of the first two component tests found that the impact head on the lighter bogie vehicle slid upward along the post as the bogie overrode the post, and it was determined that the bogie’s mass was not sufficient for the post to meet capacity. Thus, the bogie’s weight was increased to 2,500 lb (1,133 kg). The heavier bogie vehicle was used for the remaining five dynamic tests. The weights of the bogie vehicles, including the mountable impact head and accelerometers, are listed in Table 5. The bogies are shown in Figure 102.

Table 5. Actual Bogie Vehicle Weights

Bogie Weight	Test No. ILOH4-1	Test No. ILOH4-2	Test No. ILOH4-3	Test No. ILOH4-4	Test No. ILOH4-5	Test No. ILOH4-6	Test No. ILOH4-7
lb (kg)	1,786 (810)	1,786 (810)	2,522 (1,145)	2,522 (1,145)	2,522 (1,145)	2,522 (1,145)	2,522 (1,145)

Figure 102. Rigid-Frame Bogie Vehicles

The tests were conducted using a steel corrugated beam guardrail to guide the tire of the bogie vehicle. A pickup truck was used to push the bogie vehicle to the required impact velocity. After reaching the target velocity, the push vehicle braked, allowing the bogie to be free rolling as it came off the track. A remote braking system was installed on the bogie allowing it to be brought safely to rest after the test.

6.5.1 Accelerometers

Two accelerometer systems were mounted on the bogie vehicle near its center of gravity (c.g.) to measure the acceleration in the longitudinal, lateral, and vertical directions. However, only the longitudinal acceleration was processed and reported. The two systems, the SLICE-1 and SLICE-2 units, were modular data acquisition systems manufactured by Diversified Technical Systems, Inc. (DTS) of Seal Beach, California. The acceleration sensors were mounted inside the bodies of custom-built, SLICE 6DX event data recorders and recorded data at 10,000 Hz to the onboard microprocessor. Each SLICE 6DX was configured with 7 GB of non-volatile flash memory, a range of ±500 g's, a sample rate of 10,000 Hz, and a 1,650 Hz (CFC 1000) anti-aliasing filter. The "SLICEWare" computer software program and a customized Microsoft Excel worksheet were used to analyze and plot the accelerometer data.

6.5.2 Retroreflective Optic Speed Trap

The retroreflective optic speed trap was used to determine the speed of the bogie vehicle before impact. Three retroreflective targets, spaced at approximately 18-in. (457-mm) intervals, were applied to the side of the smaller 2,000-lb (907-kg) bogie vehicle and four retroreflective targets were applied to the side of the heavier 2,500-lb (1,133-kg) bogie vehicle, as shown in Figure 102. When the emitted beam of light was reflected by the targets and returned to the Emitter/Receiver, a signal was sent to the data acquisition computer, recording at 10,000 Hz, as well as the external LED box activating the LED flashes. The speed was then calculated using the spacing between the retroreflective targets and the time between the signals. LED lights and high-speed digital video analysis are only used as a backup in the event that vehicle speeds cannot be determined from the electronic data.

6.5.3 Digital Photography

AOS high-speed digital video cameras, GoPro digital video cameras, and JVC digital cameras were used to document each test. The AOS high-speed cameras had a frame rate of 500 frames per second, the GoPro video cameras had a frame rate of 120 frames per second, and the JVC digital video cameras had a frame rate of 29.97 frames per second. The cameras were placed laterally from the post, with a view perpendicular to the bogie's direction of travel, in-line and upstream from the bogie's path, and positioned below the test apparatus and zoomed-in on the tension and compression connection areas. A digital still camera was also used to document pre- and post-test conditions for all tests.

6.6 End of Test Determination

When the impact head initially contacts the test article, the force exerted by the surrogate test vehicle is directly perpendicular to the post face and aligned with the longitudinal axis of the bogie vehicle. However, as the post rotates, the surrogate test vehicle's orientation and path moves further from perpendicular. This introduces two sources of error: (1) the contact force between the impact head and the post has a vertical component and (2) the impact head slides upward along the test article. Therefore, only the initial portion of the accelerometer trace should be used since variations in the data become significant as the system rotates and the surrogate test vehicle overrides the system.

6.7 Data Processing

The electronic accelerometer data obtained in dynamic testing was filtered using the SAE Class 60 Butterworth filter conforming to the SAE J211/1 specifications [30]. The pertinent acceleration signal was extracted from the bulk of the data signals. The processed acceleration data was then multiplied by the mass of the bogie to get the impact force using Newton's Second Law. Next, the acceleration trace was integrated to find the change in velocity versus time. Initial velocity of the bogie, calculated from the pressure tape switch data, was then used to determine the bogie velocity, and the calculated velocity trace was integrated to find the bogie's displacement. This displacement is also the displacement of the post. Combining the previous results, a force vs. deflection curve was plotted for each test. Finally, integration of the force vs. deflection curve provided the energy vs. deflection curve for each test.

7 COMPONENT TESTING RESULTS AND DISCUSSION

7.1 Results

The accelerometer data for each test was processed in order to obtain acceleration, velocity, and deflection curves, as well as force vs. deflection and energy vs. deflection curves. The values described herein were calculated from the SLICE-1 data in order to provide common basis for comparing results from multiple tests, though both transducers provided similar results. Test results for all transducers are provided in 0. A summary of all bogie testing results is shown in Table 6.

It should be noted that although the acceleration data was applied to the impact location, the data came from the center of gravity (c.g.) of the bogie. This added some error to the data, since the bogie was not perfectly rigid and vibrations in the bogie were recorded. The bogie also rotated during impact, causing differences in accelerations between the bogie's center of mass and the bogie impact head. To address these concerns, filtering procedures were applied to the data to smooth out vibrations, and rotations of the bogie were tracked but deemed to be minor. Significant pitch angles did develop late in some tests as the bogie overrode the post, but the analysis was terminated prior to these times.

For all component tests, the post-to-deck attachments were side-mounted to the concrete box-beam girder utilizing the 1-in. (25-mm) diameter anchor rods as the anchorage system. Although the box-beam girder also featured the ⅞-in. (22-mm) diameter anchor rods, the smaller diameter anchor rods were never tested. In the fabrication of the concrete box-beam girder, smooth bars with threaded ends were used instead of fully threaded anchor rods. Tests also varied on anchor rod embedment depth and stirrup spacing, dependent on location along the box-beam girder. Current IDOT and ODOT designs implement a 9-in. (229-mm) stirrup spacing. The simulated girder was tested with both a stirrup spacing of 9 in. (229 mm) and 4½ in. (114 mm) to determine if the anchorage required tighter reinforcement patterns to lessen deck damage of the concrete box-beam girder.

Table 6. Dynamic Testing Results

Test No.	Post Assembly[1]	Peak Force, kips (kN)	Average Force, kips (kN)			Maximum Deflection, in. (mm)	Total Energy, k-in. (kJ)	Failure Type	Post-Test Modification
			@5"	@10"	@15"				
ILOH4-1	Two - 1¼-in. Plates	30.1 (134)	17.8 (79)	13.7 (61)	9.9 (44)	38.9 (988)	158.6 (18)	Tensile Weld Failure	Add Tensile Gussets
ILOH4-2	Two - 1¼-in. Plates w/ Tensile Gussets	27.8 (124)	17.4 (77)	18.1 (81)	17.0 (76)	21.7 (551)	328.8 (37)	Post Yield between Plate Attachments	Singular Plate Attachment, HSS5x4x½
ILOH4-3	One 1-in. Plate Attachment	36.9 (164)	13.8 (61)	N/A[2]	N/A[2]	5.9 (150)	76.8 (9)	Manufacturer Weld Failure	Remanufacture Post Assemblies
ILOH4-4	One 1-in. Plate Attachment	39.6 (176)	19.6 (87)	21.4 (95)	20.4 (91)	27.7 (704)	367.1 (41)	Plastic Hinge / Post Tear	N/A[3]
ILOH4-5	One 1-in. Plate Attachment	37.6 (167)	21.3 (95)	21.8 (97)	19.7 (88)	25.5 (648)	377.9 (43)	Plastic Hinge	N/A[3]
ILOH4-6	One 1-in. Plate Attachment	33.9 (151)	20.1 (89)	20.7 (92)	18.5 (82)	26.6 (676)	356.1 (40)	Plastic Hinge	N/A[3]
ILOH4-7	One ¾-in. Plate Attachment	29.2 (130)	17.9 (80)	19.8 (88)	18.6 (83)	22.1 (561)	347.2 (39)	Plastic Hinge / Bent Plate Attachment	N/A[3]

[1]Only Test Nos. ILOH4-1 and ILOH4-2 utilized HSS5x4x⅜ deck spacers.
[2]Forces not obtained due to premature failure.
[3]No modifications recommended.
N/A = Not applicable

7.1.1 Test No. ILOH4-1, Welded Post Assembly A

The first bogie test, test no. ILOH4-1, was performed on the 1¼-in. (32-mm) two-plate attachment with HSS5x4x⅜ longitudinal tube spacers, as originally chosen for the post-to-deck attachment design by IDOT and ODOT. The testing was conducted at location P2 with a stirrup spacing of 9 in. (229 mm) and an anchor embedment length of 34½ in. (876 mm). A pre-test assembly is shown in Figure 99a.

The bogie impacted the W6x15 steel post traveling at a speed of 22.5 mph (36.3 km/h) perpendicular to the face of the post. Upon impact of the bogie, the W6x15 post briefly rotated backward until weld failure occurred between the top plate attachment and the post, and the post detached and rotated backward as the bogie overrode the post during impact, as shown in Figure 103.

Inspection of the post assembly and deck attachment after the test revealed that the post bent minimally prior to the tensile weld rupture of the top plate attachment. The plate attachment remained bolted to the HSS longitudinal tube side-mounted to the box-beam girder. At the lower connection, the post and lower mounting plate remained intact with no visible deformation and remained bolted to the HSS longitudinal tube. Throughout the impact event, the entire lower connection area comprising the post assembly and HSS spacer rotated backward and caused bulging out of the embedded plate in the box-beam girder. The bolts sustained no visible damage, and the upper HSS tube showed minor deformations as the side walls began to bow outward due to the tensile loading.

IMPACT

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(a) Sequential Images

(b) Damage at End of Test

(c) Damage after Removal

Figure 103. Time-Sequential and Post-Impact Photographs, Test No. ILOH4

A reoccurring issue in several component tests was concrete spalling along the box-beam girder's sidewall during pre-test assembly and post-test inspection. In pre-test, spalling occurred when the post-to-deck attachment was assembled and side-mounted to the concrete box-beam girder. In post-test, spalling developed around the bottom of the embedded plate, but this damage was likely the result of high bending loads impacted in the lower connection area after the tensile weld failure occurred. These spalling issues are further discussed in detail in the discussion section of this chapter. Pre- and post-test spalling damage is shown in Figures 104 and 105 respectively.

Force-deflection and energy-deflection curves were created from the accelerometer data, and are shown in Figure 106. Initially, inertial effects resulted in a high peak force of 26 kips (116 kN) over the first 2½ in. (64 mm) of deflection. The post sustained average forces of 18 kips (80 kN) and 14 kips (62 kN) at 5 in. (127 mm) and 10 in. (254 mm) of displacement, respectively, with average loads of 11 kips (49 kN) at a 12-in. (305-mm) deflection.

Figure 104. Pre-Test Concrete Spalling for Test No. ILOH4-1

Figure 105. Post-Test Concrete Spalling for Test No. ILOH4-1

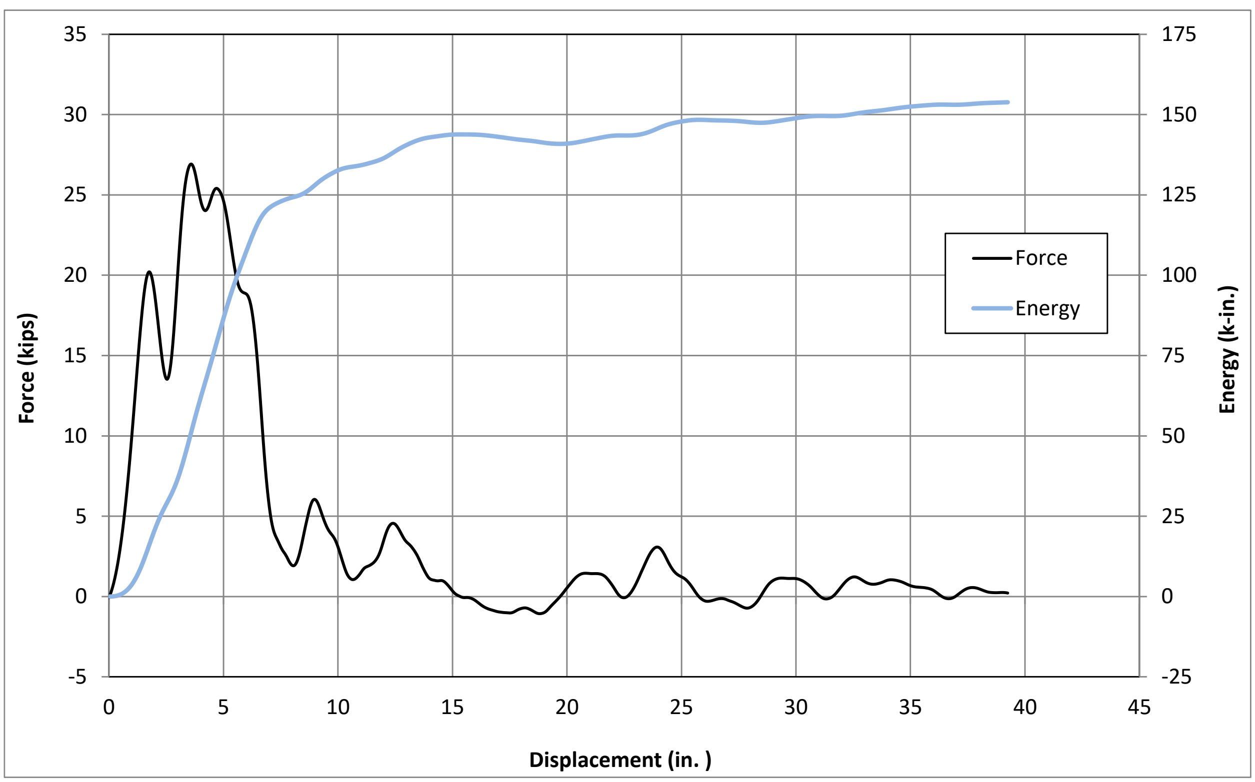


Figure 106. Force vs. Deflection and Energy vs. Deflection, Test No. ILOH4-1

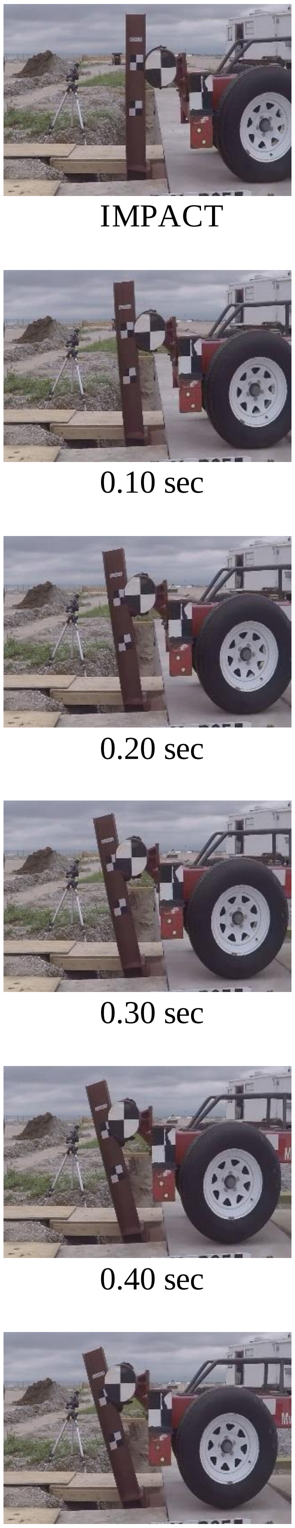
7.1.2 Test No. ILOH4-2, Welded Post Assembly D

Following the weld failures in the upper plate attachment from the first test, the upper 1¼-in. (32-mm) plate was strengthened by the addition of gussets to the top plate attachment to reinforce the weld strength. The same longitudinal HSS5x4x⅜ tubes were used as spacers. The test was conducted at location P4 with anchorage using a 34½-in. (876-mm) embedment depth at the 4½-in. (114-mm) stirrup spacing. During test no. ILOH4-2, the bogie impacted the W6x15 post at a speed of 20.8 mph (33.5 km/h) causing strong-axis bending in the post. The post rotated backward approximately 15 in. (381 mm) before the bogie overrode the top of the post. The post-to-deck attachment is shown in Figure 107.


Figure 107. Added Tensile Gussets for Test No. ILOH4-2

Time-sequential photographs and post-test damage of the post assembly and deck spacers is shown in Figure 108. Deformations to the post assembly were located between the top and bottom mounting plates as opposed to a plastic hinge forming near the surface of the deck (above the tensile bolts and gusset) as intended. The web at the bottom of the post buckled under the impact load and a plastic hinge formed between the upper and lower plate attachments. Also, the upper bolts connecting the upper plate and longitudinal tube slid downward in the slotted holes in the plate attachment as the post deformed and rotated back. The upper spacer bowed outward from the tensile loads but the lower spacer did not deform. No other damage occurred to the post assembly or anchorage. No concrete spalling occurred before or during testing.

Force-deflection and energy-deflection curves were created from the accelerometer data and are shown in Figure 109. Peak impact loads were similar to test no. ILOH4-1 at approximately 27 kips (120 kN). However, the post sustained average forces of 18 kips (79 kN) and 14 kips (61 kN) at 5 in. (127 mm) and 10 in. (254 mm) of displacement, respectively, with average loads of over 17 kips (76 kN) at a deflection of 12 in. (305 mm).



(a) Sequential Images

(b) Damage at End of Test

(c) Damage after Removal

Figure 108. Time-Sequential and Post-Impact Photographs, Test No. ILOH4-2

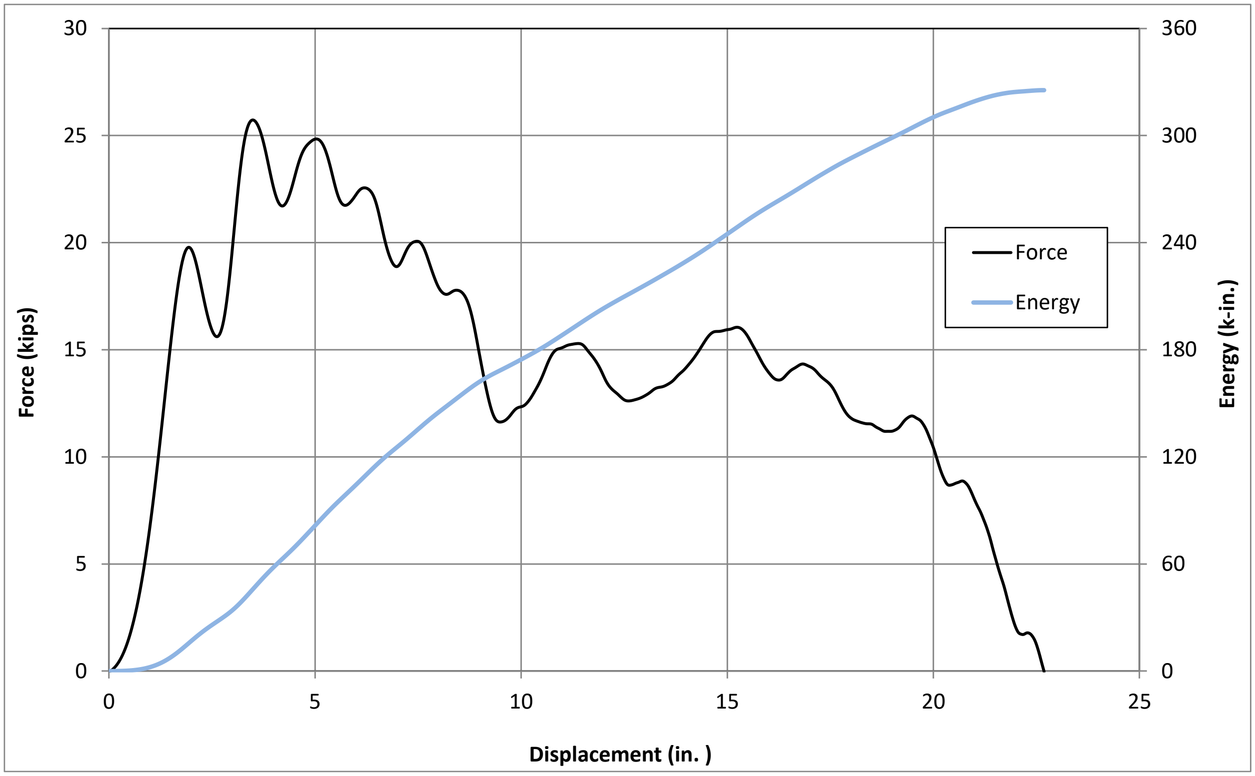


Figure 109. Force vs. Deflection and Energy vs. Deflection, Test No. ILOH4-2

7.1.3 Test No. ILOH4-3, Welded Post Assembly G

The third bogie test featured design changes in the form of a singular attachment plate, the addition of gussets at the lower connection, and thicker longitudinal tube spacers, as shown in Figure 110. The singular attachment plate replaced the two plate attachments and was welded to the front face of the post to provide a continuous front flange support and prevent localized deformations. Since gussets between the post and plate attachment were utilized, the plate thickness was reduced from 1¼ in. (32 mm) to 1 in. (25 mm). Gussets were also included at the bottom of the singular plate attachment to mitigate localized web buckling in the compression region of the post. Finally, the thickness of the HSS longitudinal tubes were increased to ½ in. (13 mm) to prevent the tubes from bowing outward. The test was conducted at location P8 using a 34½-in. (876-mm) anchor rod embedment depth at 4½-in. (114-mm) stirrup spacing.

Figure 110. Singular Plate Attachment, Thicker Deck Spacers, and Gussets, Test No. ILOH4-3

Time-sequential photographs and post-test damage are shown in Figure 111. During test no. ILOH4-3, the bogie laterally impacted the W6x15 steel post at a speed of 21.8 mph (35.1 km/h). Upon impact, the welds along the top and bottom gussets, as well as the welds along the attachment plate connected to the front flange of the post, sheared off and the post rotated backward and rested on the tarmac. The post did not bend or deform as the welds completely failed and the post detached and rotated backward as the bogie overrode the post. After careful investigation of the post assembly, it was determined that poor burn-in of the welds was the cause of the complete weld failure. All post assemblies were returned to the manufacturer for complete rework of the fillet welds to the base materials. No concrete spalling was observed during pre- and post-test.

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(a) Sequential Images

(b) Damage at End of Test

(c) Damage after Removal

Figure 111. Time-Sequential and Post-Impact Photographs, Test No. ILOH4-3

Force-deflection and energy-deflection curves were created from the accelerometer data and are shown in Figure 112. As expected, peak impact loads were higher with increased bogie mass and velocity at 38 kips (169 kN). The post resisted average loads of 14 kips (62.3 kN) at a 5-in. (127-mm) deflection before early weld failure of the post assembly.

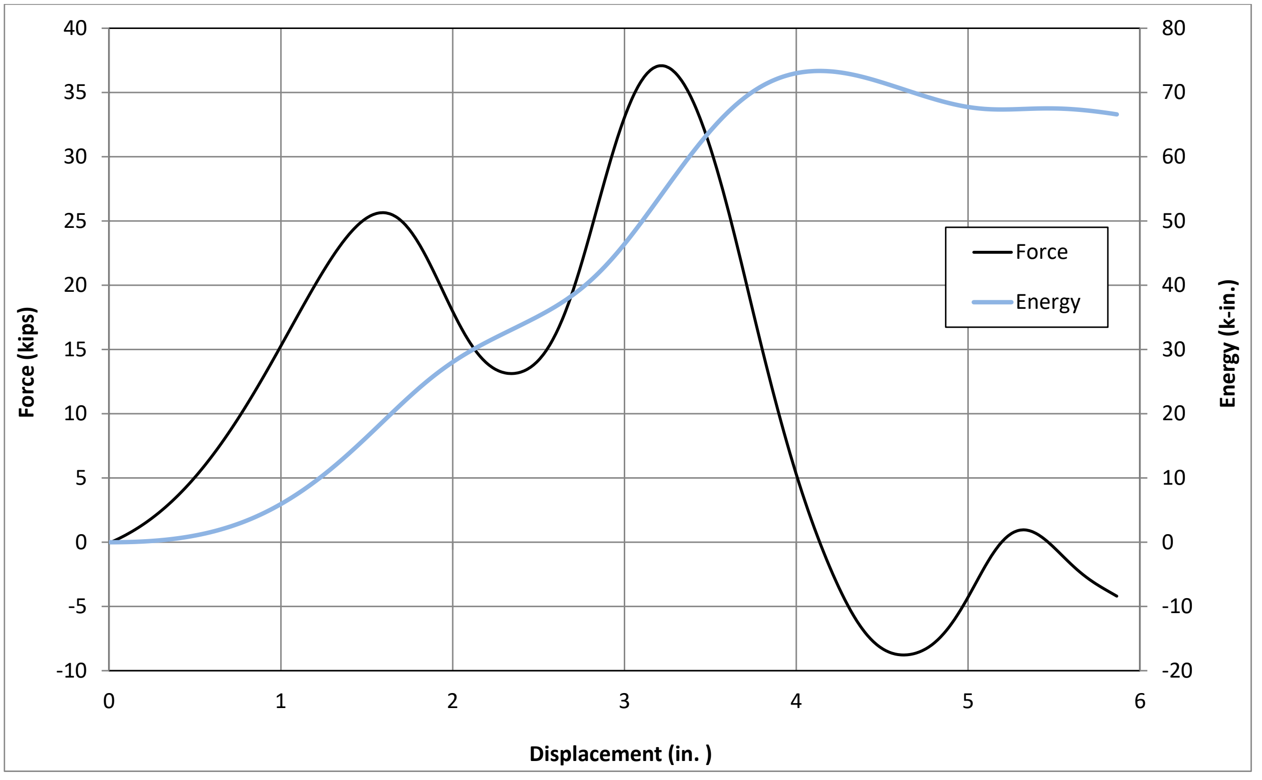

Figure 112. Force vs. Deflection and Energy vs. Deflection, Test No. ILOH4-3

7.1.4 Test No. ILOH4-4, Welded Post Assembly G

After manufacturing new post assemblies similar to the one that experienced weld failure and verifying proper welds, a repeat of test no. ILOH4-3 was performed at the same location, P8, with 34½-in. (876-mm) rod embedment depth at 4½-in. (114-mm) stirrup spacing. During test no. ILOH4-4, the bogie impacted the W6x15 post at a speed of 21.4 mph (34.4 km/h) causing strong-axis bending in the post. The post rotated and tore above the tensile gussets at a displacement of 17 in. (432 mm). Time-sequential photographs and post-test damage are shown in Figure 113.

No deformations were observed within the longitudinal spacer tubes, the plate attachment, and the post section between the tension and compression areas. The post tore above the 6-in. (152-mm) weld at the front flange of the post and the top of the plate attachment, and tore diagonally upward along the post web until reaching the back flange. Buckling of the back flanges was seen above the tensile gussets. It is assumed that the post tore from impact due to experiencing peak loading.

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(a) Sequential Images

(b) Damage at End of Test

(c) Damage after Removal

Figure 113. Time-Sequential and Post-Impact Photographs, Test No. ILOH4-4

Force-deflection and energy-deflection curves were created from the accelerometer data and are shown in Figure 114. Peak loading of 39.6 kips (176.1 kN) was measured with a post-sustained average loading of 20 kips (89 kN) and 21 kips (95 kN) at 5 in. (127 mm) and 10 in. (254 mm) of displacement, respectively. Post rupture occurred at approximately 17 in. (432 mm) of deflection at approximately 20 kips (89 kN) of sustained average load. Tearing of the post flange was deemed not critical due to failure occurring at a deflection that was approximately 50 percent greater than the 12-in. (305-mm) deflection anticipated in the full-scale crash tests. The anticipated 12-in. (305-mm) deflection is based on the literature review of previous crashworthy post-and-rail bridge rails commonly observed to deflect at this amount. Details regarding the 12-in. (305-mm) deflection threshold are stated in the Discussion section of this chapter. Therefore, the post rupture at 17 in. (432 mm) of deflection was deemed non-critical.

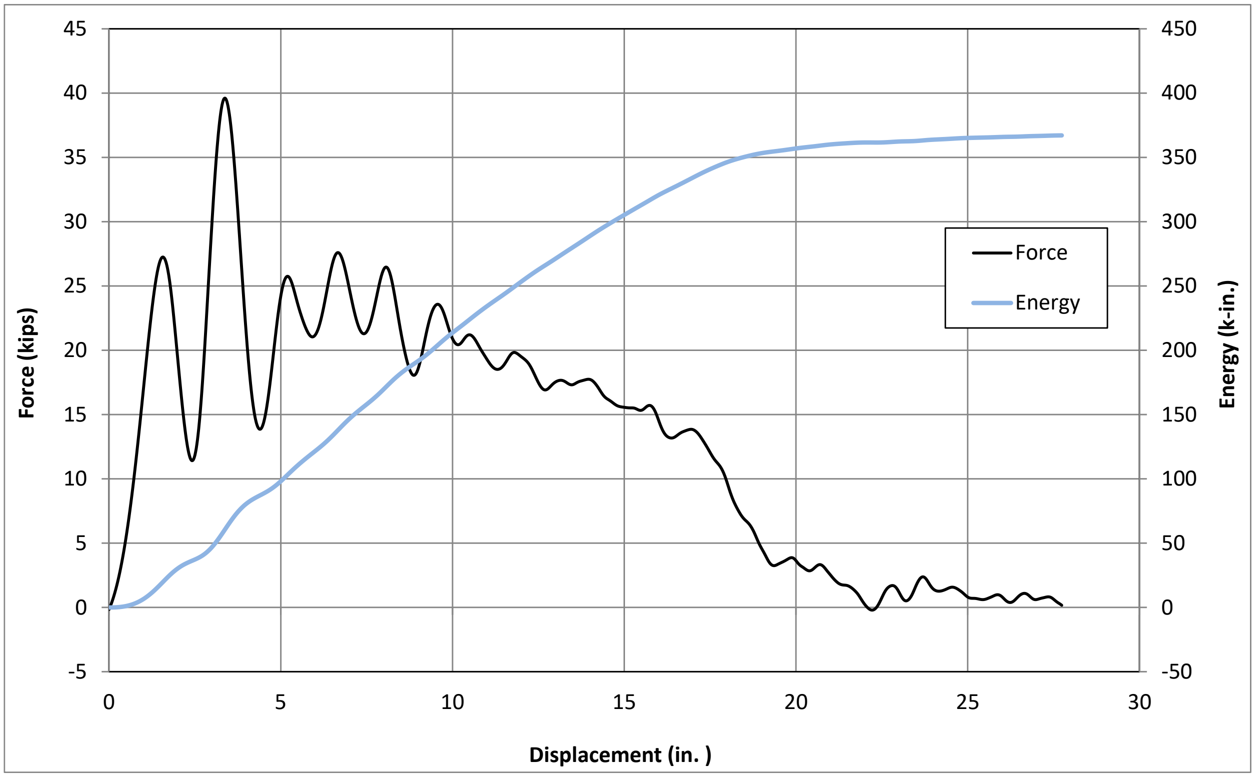


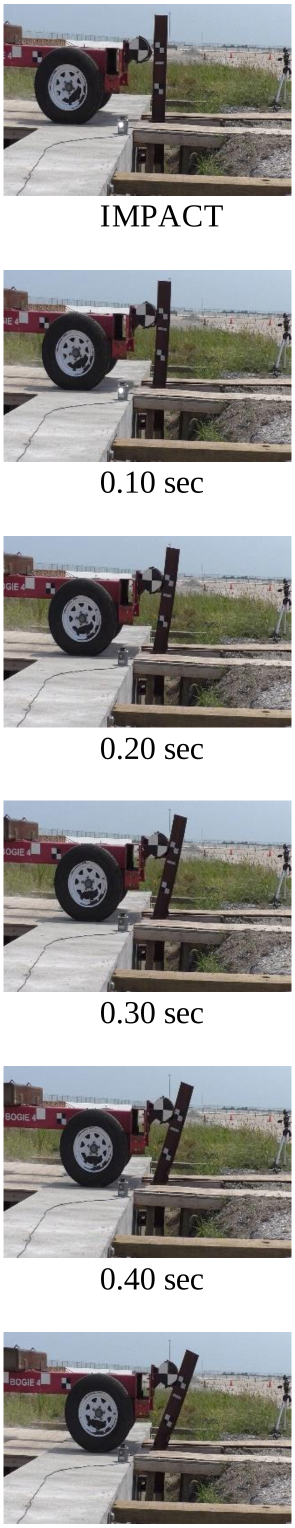
Figure 114. Force vs. Deflection and Energy vs. Deflection, Test No. ILOH4-4

During deck attachment assembly prior to the component test, concrete spalling was observed around the bottom corners of the embedded plate in the concrete box-beam. Pre- and post-test concrete spalling of the deck attachment assembly is shown in Figure 115. The sidewalls began spalling as the lower longitudinal tube was bolted to the embedded plate. Furthermore, measurements of the available threaded length within the coupling nuts and heavy hex nuts were taken at every embedded plate location in the concrete box-beam and several were determined to have lengths less than the threaded length of the bolt. It was assumed that the less available threaded length in the coupling nuts resulted from the anchor rods threaded further into the coupling nuts than desired before casting of the concrete. The lower bolts were believed to be contacting the concrete beyond the heavy hex nuts and pushing the nut and embedded plate outward. During casting of the concrete box-beam girder, it is unclear if any methods were utilized to prevent concrete from entering the heavy hex nuts. For the duration of the component testing, the lower compression bolts were only tightened "hand tight." For full-scale crash testing, design modifications were recommended to replace the lower two heavy hex nuts with coupling nuts and increasing the thickness of the embedded plate to prevent concrete spalling during assembly. The concrete spalling was more evident after the test ran.

Figure 115. Pre- and Post-Test Sidewall Spalling, Test No. ILOH4-4

7.1.5 Test No. ILOH4-5, Welded Post Assembly G

Test no. ILOH4-5 used the same post-to-deck design attachment hardware, but testing was conducted at location P7 using the wider stirrup spacing of 9 in. (229 mm) with the 34½-in. (876-mm) rod embedment depth. During assembly of the deck attachment, no pre-test concrete spalling was observed along the sidewall of the girder. During test no. ILOH4-5, the bogie impacted the W6x15 post at a speed of 20.6 mph (33.2 km/h) causing strong-axis bending and a plastic hinge in the post right above the tensile gussets. The post rotated backward 15 in. (381 mm). Time-sequential photographs and post-test damage are shown in Figure 116.



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(a) Sequential Images

(b) Damage at End of Test

(c) Damage after Removal

Figure 116. Time-Sequential and Post-Impact Photographs, Test No. ILOH4-5

A plastic hinge developed near the top surface of the concrete box-beam girder, which was expected to occur in the full-scale crash tests. No other evident deformations of the post nor of the deck attachments were seen. Concrete spalling was observed near the compression connection area on the girder's sidewall, as shown in Figure 117.

Figure 117. Post-test Concrete Spalling, Test No. ILOH4-5

Force-deflection and energy-deflection curves were created from the accelerometer data and are shown in Figure 118. Peak loading of 37.6 kips (167.3 kN) was measured with the post sustaining an average loading of 21 kips (93 kN) over a 5-in. (127-mm) to 10-in. (254-mm) deflection. The post also sustained average loading of 21 kips (93 kN) through 12 in. (305 mm) of post deflection.

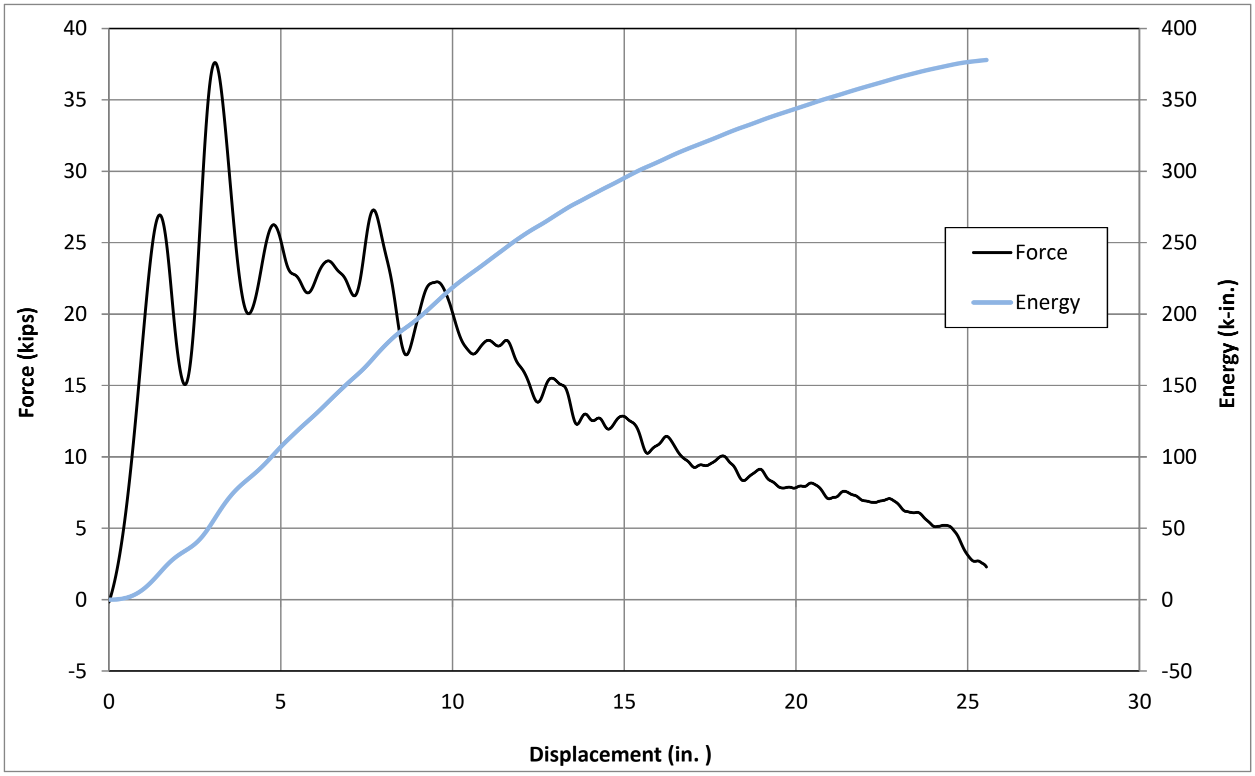

Figure 118. Force vs. Deflection and Energy vs. Deflection, Test No. ILOH4-5

7.1.6 Test No. ILOH4-6, Welded Post Assembly G

The following two component tests, test nos. ILOH4-6 and ILOH4-7, focused on testing of the shortened anchor rod embedment length within the concrete box-beam. Previous tests had embedment lengths of 34½ in. (876 mm). Located in the capped ends of the girder, the rods with the shortened embedment length of 25½ in. (648 mm) were utilized for test no. ILOH4-6. The sixth test was conducted at location P5 using the 1-in. (25-mm) singular plate attachment post assembly with HSS5x4x½ longitudinal tube spacers and 1-in. (25-mm) diameter anchor rods, as previously tested. No pre-test concrete spalling was observed during installation of the deck attachment. The bogie vehicle impacted the W6x15 post at a speed of 20.5 mph (33 km/h) and the post developed a plastic hinge near the surface of the girder similar to the previous test. A maximum deflection of 26.6 in. (675.6 mm) was observed as the bogie overrode the post. Time-sequential photographs and post-test photographs are shown in Figure 119.

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(a) Sequential Images

(b) Damage at End of Test

(c) Damage after Removal

Figure 119. Time-Sequential and Post-Impact Photographs, Test No. ILOH4-6

Sidewall hairline cracks occurred near the right compression area connection, near the edge of the concrete box-beam girder, as shown in Figure 120. No concrete spalling was observed along the sidewall. No other concrete failure was observed and there was no evidence of anchorage failure. Post deformation was only localized to the plastic hinge that developed near the surface of the deck; no other deformations were seen in the deck attachment assembly.

Figure 120. Post-Test Concrete Cracking from Test No. ILOH4-6

Force-deflection and energy-deflection curves were created from the accelerometer data and are shown in Figure 121. Peak loading of 34 kips (151 kN) was observed, and the post sustained average loading of 20 kips (89 kN) over a 5-in. (127-mm), 10-in. (254-mm), and 12-in. (305-mm) post deflection.

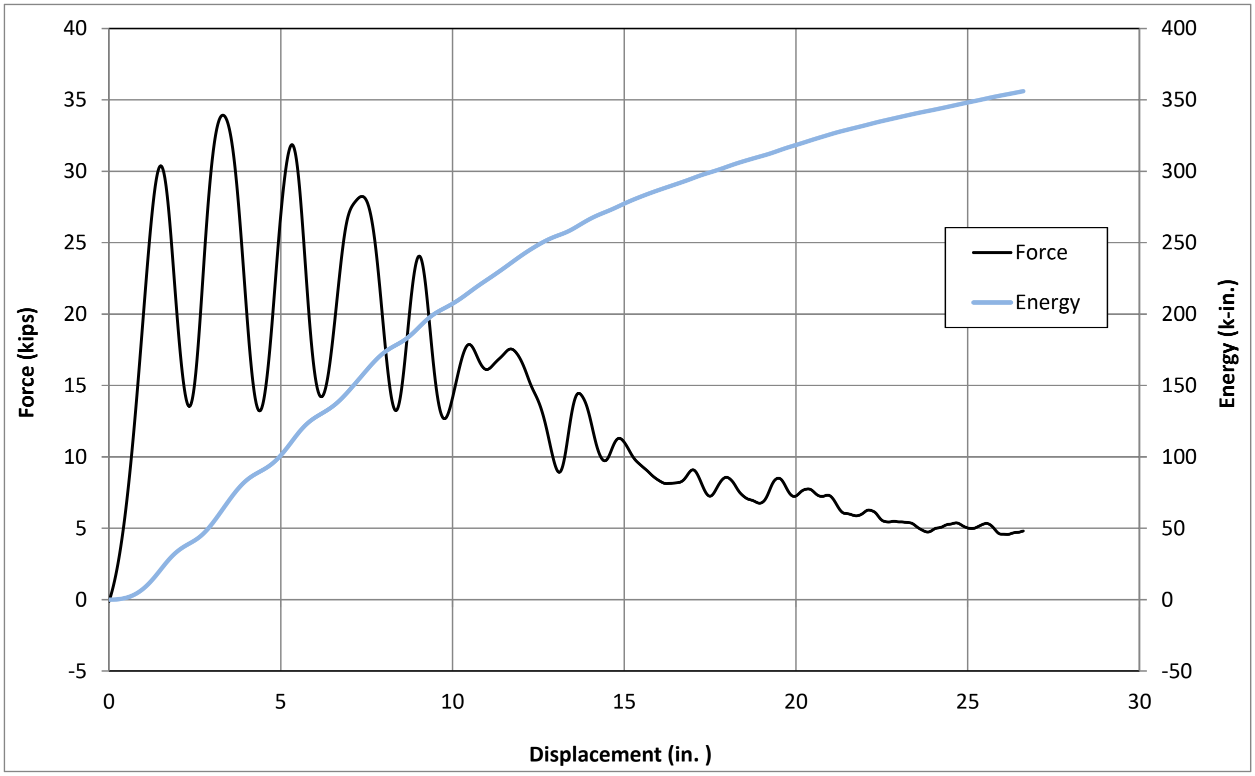

Figure 121. Force vs. Deflection and Energy vs. Deflection, Test No. ILOH4-6

7.1.7 Test No. ILOH4-7, Welded Post Assembly E

The final component test involved testing the shortest anchor embedment of 16½ in. (419 mm) with the thickness of the singular plate attachment reduced to ¾ in. (19 mm). The seventh test still utilized the 1-in. (25-mm) diameter anchor rods and testing was conducted at location P1, also located in the capped ends of the concrete box-beam girder. No concrete spalling was seen during pre-test assembly. The bogie vehicle impacted the W6x15 post at a speed of 19.9 mph (32 km/h) and the post developed a plastic hinge near the top surface of the concrete box-beam girder. A maximum deflection of 22.1 in. (561 mm) was observed as the bogie overrode the post. Time-sequential photographs and post-test photographs are shown in Figure 122.

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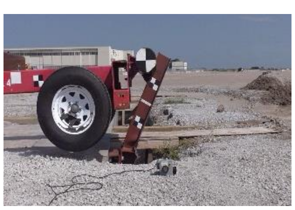

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(a) Sequential Images

(b) Damage at End of Test

(c) Damage after Removal

Figure 122. Time-Sequential and Post-Impact Photographs, Test No. ILOH4-7

No cracks or concrete spalling were observed along the sidewall or on top of the girder. The post developed a plastic hinge near the surface of the deck. The ¾-in. (19-mm) plate attachment was bent at the top bolt connections. No other deformations were observed in the deck attachment. Force-deflection and energy-deflection curves were created from the accelerometer data and are shown in Figure 123. Peak loading of 29.2 kips (129.9 kN) were observed and the post sustained average loading of approximately 18 kips (80 kN) and 20 kips (88 kN) over a 5-in. (127-mm) and 10-in. (254-mm) deflection. The post sustained 19 kips (85 kN) over a 12-in. (305-mm) deflection.

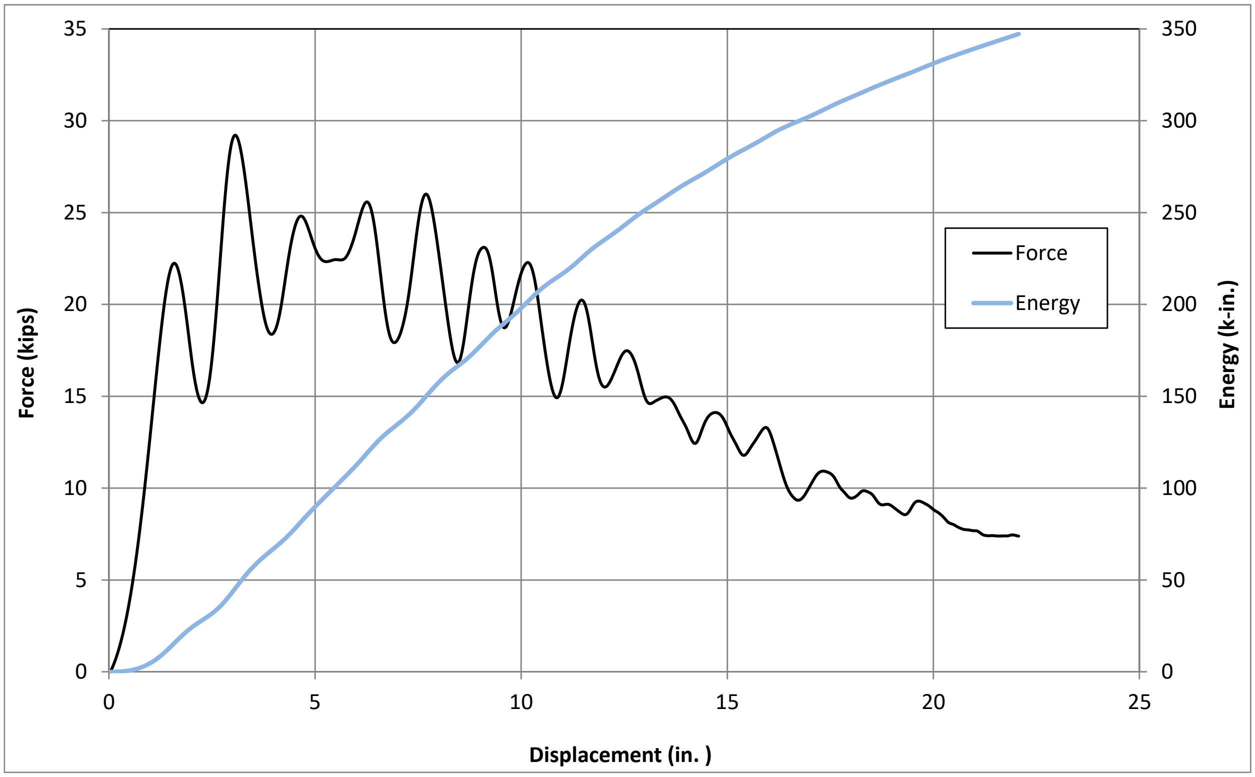


Figure 123. Force vs. Deflection and Energy vs. Deflection, Test No. ILOH4-7

7.2 Discussion

Component testing was performed on post-to-deck attachments side-mounted to a concrete box-beam girder utilizing 1-in. (25-mm) diameter ASTM F1554 Grade 105 anchor rods with varying stirrup spacing dependent on the post location along the box-beam girder. Although the box-beam girder also featured ⅞-in. (22-mm) diameter anchor rods, the smaller diameter rods were never tested. Bogie testing was utilized to optimize the post attachments and the stirrup spacing.

In all component tests, no anchorage failure was observed with the anchor rods. In the hollow section featuring the 5½-in. (140-mm) thick sidewalls and 5½-in. (140-mm) thick top and bottom layers of the concrete box-beam girder, the 34½-in. (876-mm) anchor embedment lengths had sufficient capacity and showed no slippage or concrete breakout from the top surface layer of the box-beam girder. Similarly, the same results were evident in the testing of the shortened anchor embedment lengths, within the solid end caps, with 25½ in. (648 mm) and 16½ in. (419 mm) embedment depths. A hairline crack did form along a section of the sidewall closest to the edge of the concrete box-beam girder, but was considered non-critical.

Surface spalling on the concrete box-beam girder's sidewalls resulted throughout the test series. In some tests, the spalling was observed either during pre-test assembly or post-test, with spalling only occurring within the hollow section of the box-beam girder. It should be noted that all spalling was observed to be localized only to the surface of the sidewall and was no greater than ⅛ in. (3.2 mm) in depth. With minimal depth in the concrete spalling occurring in the sidewalls, it was evident that the embedded anchorage plate cast within the sidewall of the concrete box-beam girder during manufacturing was flawed and the plate design needed to be optimized to prevent such spalling issues in the future full-scale crash tests and for actual bridge design applications. Table 7 shows when the spalling was observed and for which test setups and locations along the box-beam girder. It should be noted that concrete spalling observed during pre- and post-test occurred regardless of whether the deck attachment anchorage utilized IDOT and ODOT's 9-in. (229-mm) stirrup spacing or the narrower 4½-in. (114-mm) spacing.

Table 7. Sidewall Concrete Damage

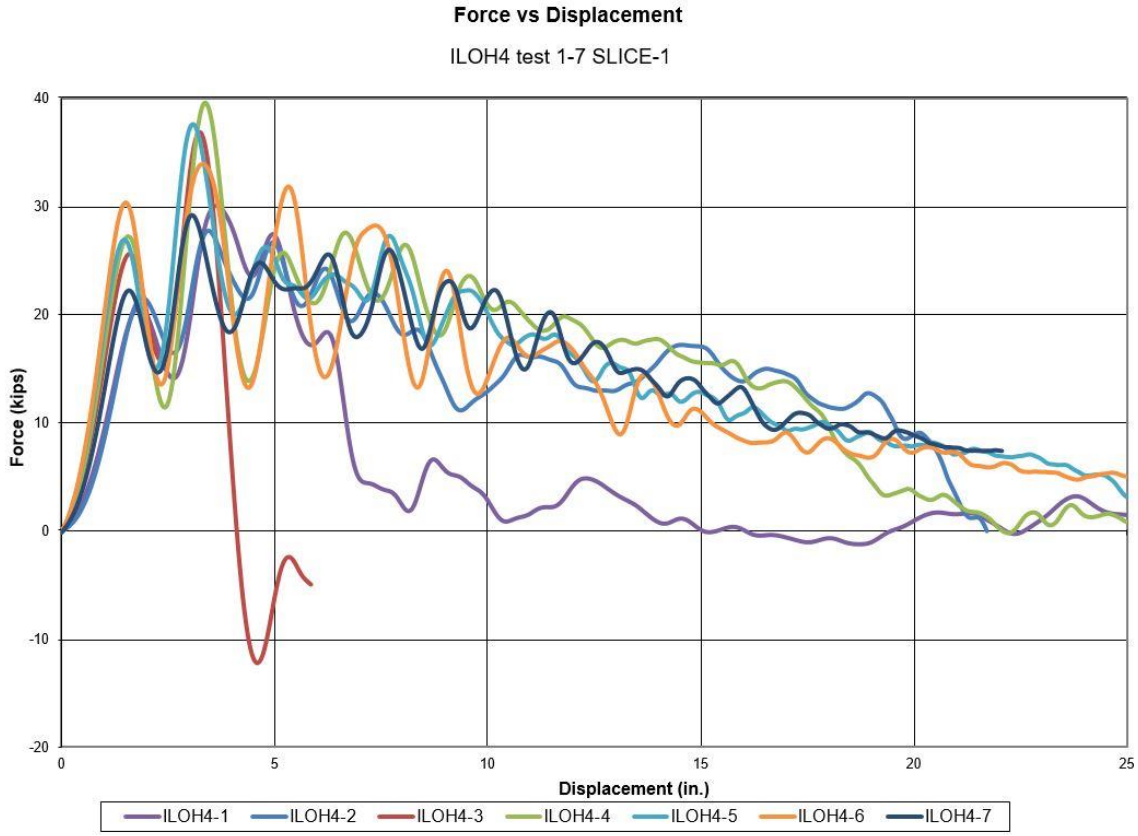
Test No.	Welded Post Assembly	No. of Mounting Plates	Mounting Plate Thickness, in. (mm)	Gussets	Stirrup Spacing, in. (mm)	Sidewall Damage
ILOH4-1*	A	2	1¼ (31)	None	9 (229)	Pre- & Post-Test Spalling
ILOH4-2*	D	2	1¼ (32)	2 on Top Plate	4½ (114)	None
ILOH4-3	G	1	1 (25)	4	4½ (114)	None
ILOH4-4	G	1	1 (25)	4	4½ (114)	Pre- & Post-Test Spalling
ILOH4-5	G	1	1 (25)	4	9 (229)	Post-Test Spalling
ILOH4-6	G	1	1 (25)	4	Solid End Cap	Hairline Cracks
ILOH4-7	E	1	¾ (19)	4	Solid End Cap	None

*Test Nos. ILOH4-1 and ILOH4-2 featured HSS5x4x⅜ deck spacers.

It should be noted that all steel posts were loaded beyond yielding during the tests as each post was deformed and bent backward. In several cases, either the post attachment hardware and/or the box-beam girder sustained visible damage. The post assemblies either plastically deformed between the upper and lower plate attachments, as shown in test nos. ILOH4-1 and ILOH4-2, or bent at the top bolt locations with the thinner ¾-in. (19-mm) singular plate attachment, as observed in test no. ILOH4-7. However, design optimizations for the post attachment hardware led to the discovery that the 1-in. (25-mm) singular mounting plate attachment, from test nos. ILOH4-4 through ILOH4-6, did result in the post plastically hinging near the surface of the deck, as intended, while achieving at or above 19 kips (85 kN) of post resistance.

Test nos. ILOH4-4 through ILOH4-7 all performed similarly in developing the ideal post plastic hinge near the surface of the deck. Recall the differences between the four tests: test nos. ILOH4-4 and ILOH4-5 were localized in the hollow section of the concrete box-beam girder and utilized 4½-in. (114-mm) and 9-in. (229-mm) stirrup spacing, respectively, whereas test nos. ILOH4-6 and ILOH4-7 were confined in the solid end caps for testing of the anchor rod embedment lengths at 25½ in. (648 mm) and 16½ in. (419 mm), respectively. Furthermore, test no. ILOH4-7 also tested a thinner ¾-in. (19-mm) plate attachment that resulted in plate bending localized at the top bolts on the post assembly. Nonetheless, even with the differences in test setups, the last four tests were successful in that they developed the intended plastic hinge in the post near the surface of the deck and no critical damage was imparted onto the sidewall of the critical box-beam girder. This plastic deformation of the post assembly is expected in the full-scale crash tests of the steel-tube bridge rail, with minimal, if not negligible, concrete spalling along the bridge deck.

An analysis of the force-deflection plots from the four successful tests illustrates similar results in forces as the post was displaced through each test and a plastic hinge was developed. Inertial effects from the post assemblies at the beginning of each impact were observed during all seven bogie tests. As illustrated in Figure 124, the recorded data from each test showed large force spikes over approximately the first 2 in. (51 mm) of deflection. These force spikes had a magnitude ranging from 21.6 kips (96.1 kN) to 31 kips (138 kN). The inertia of the post assemblies as they began to deflect and rotate backward caused these force spikes, and since all post assemblies were nearly identical, the difference in inertia effects was attributed to the impact velocity.



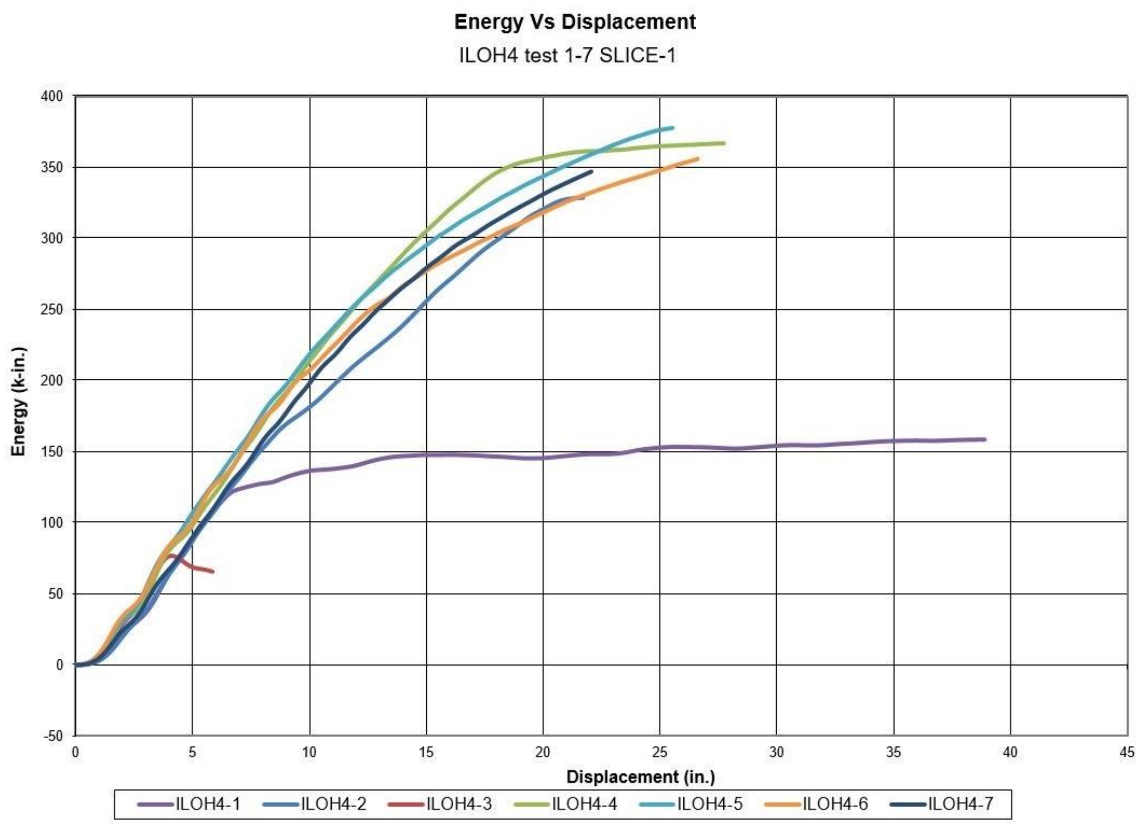


Figure 124. Force-Deflection and Energy-Displacement Plots from Component Testing

A significant result from the component testing was that the true post strength was lower than the preliminary estimated post resistance. Compared to the preliminary strength of 26 kips (116 kN), the observed post strength of approximately 19 kips (85 kN) was developed to plastically deform the W6x15 post while creating the ideal hinge above the tensile gussets, near the surface of the deck. Review of the force-deflection plots determined that the post strengths, during the four successful tests where the posts plastically deformed near the surface of the deck, diminished after deflecting 12 in. (305 mm) from the impact event. Thus, at the 12-in. (305-mm) deflection the post was determined to develop a resistance of 19 kips (85 kN). It is crucial to note the 12-in. (305-mm) deflection is commonly seen in full-scale crash tests of steel-tube bridge rails utilizing similar steel post sections, bridge rail post spacings, tube rail sections, and bridge rail heights. The in-development MASH 2016 TL-4 steel-tube bridge rail is similarly anticipated to deflect approximately 12 in. (305 mm).

The forces observed were similar for tests where a plastic hinge developed on the post above the tensile gussets near the surface of the deck. No matter the test setup, such as the test location utilizing the 4½-in. (114-mm) or 9-in. (229-mm) stirrup spacing, such similarity in forces and energies was expected as the same post bending occurred during each test. Test nos. ILOH4-4 through ILOH4-6 used Welded Post Assembly G featuring the 1-in. (25-mm) singular mounting plate and gussets, with HSS5x4x½ deck spacers, while test no. ILOH4-7 used Welded Post Assembly E featuring a ¾-in. (19-mm) mounting plate with plate bending at the top bolts. A few exceptions in the force curves were for test no. ILOH4-1, which had tensile weld failure at the upper 1¼-in. (32-mm) mounting plate, and for test no. ILH4-3, which saw the premature weld failure at the mounting plate attachment to the post. Although test no. ILOH4-2 developed plastic deformations and post bending between the upper and lower 1¼-in. (32-mm) mounting plates used in Welded Post Assembly D, the second test resulted in similar forces as the other four successful component tests. Force-deflection and energy-deflection plots for the lateral component tests are shown in Figure 124.

After completion of the component testing, it was clear that the post-to-deck attachment design featuring the Welded Post Assembly G with the 1-in. (25-mm) singular mounting plate with gussets and HSS5x4x½ spacer tubes would not generate enough load to cause critical damage to the sidewall of the concrete box-beam girder. Therefore, the post-to-deck attachment hardware and the concrete box-beam girder was adequate for use in the new steel-tube bridge rail. Design optimizations from the component testing showed that the utilization of the thicker HSS5x4x½ longitudinal spacer tubes did not deform, but rather transferred the impact load uniformly to the deck. The thinner HSS5x4x⅜ deck spacers, originally evaluated in the first two tests, had their sidewalls bow outward from the tensile loading induced by the post rotation. The singular mounting plate attachment provided a robust, continuous support to the front flange of the post and prevented localized deformations between the tensile and compression areas of the attachment. Specifically, the 1-in. (25-mm) mounting plate did not deform while the thinner ¾-in. (19-mm) mounting plate bent at the top tensile bolts. This bend is critical in that it can allow greater bridge rail deflection than anticipated. In addition to the single mounting plate attachment, tension and compression gussets also prevented localized deformations at the deck attachment. Use of ¼-in. (6-mm) thick square washers were beneficial in preventing bolt pullout at slotted holes during impact events.

The 1-in. (25-mm) diameter anchor rods and the 9-in. (229-mm) stirrup spacing resulted in minimal deck damage, which met an original design criteria. Use of coupling nuts with a 3-in. (76-mm) square washer plate and bolt in the bottom, compression section of the anchorage was recommended to prevent the reverse-bending effect that caused concrete spalling in the compression area of the embedded plate. Finally, the thickness of the embedded plate was increased to $^{3}/_{16}$ in. (4.8 mm) to increase its bending strength. All three anchor embedment lengths were successfully tested, however, only the 34½-in. (876-mm) anchor length was tested within the thin 5½-in. (140-mm) upper thin slab of the concrete box-beam girder. The shortened 25½-in. (648-mm) and 16½-in. (419-mm) embedment lengths were considered appropriate for use in skewed bridges where post anchorage would be localized in the solid end caps of the concrete box-beam girder, similar to the bogie test conditions for the two shortened lengths. Therefore, use of the longer anchor embedment length shall be considered for use within the hollow core section of the girder. The final post-to-deck attachment design and post anchorages are shown in Figures 125 and 126, respectively.

Figure 125. Post-to-Deck Connection Final Design

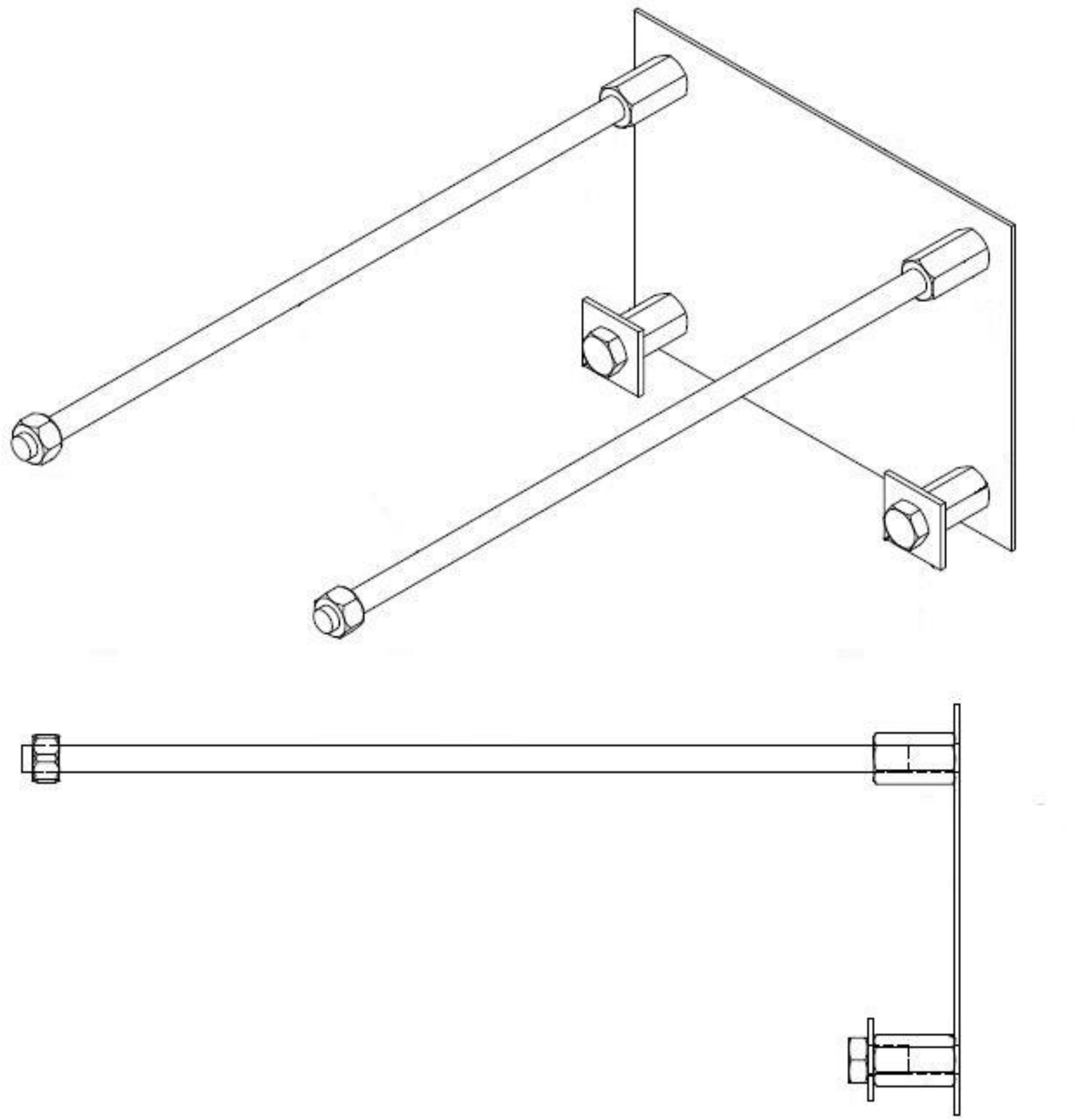


Figure 126. Post Anchorage Final Design

8 SIMULATION

Computer simulation using the 3D, non-linear finite element analysis program LS-DYNA was performed to compare to the results of the dynamic bogie tests. A model of the W6x15 steel post with the post-to-deck connection that was used in test nos. ILOH4-4 through ILOH4-6 was created and validated against the component tests. The post and deck model will be used in the Phase II of the project to develop a thrie-beam transition to the steel-tube bridge rail.

8.1 Post-to-Deck Connection Model Details

The simulation model of the post-to-deck connection was developed and validated against the strong-axis bogie testing, test nos. ILOH4-4 through ILOH4-6. Bogie vehicle velocity and mass, impact height, and the post-to-deck connection configuration were taken from test no. ILOH4-5.

8.1.1 Part Details

The W6x15 post flanges and web, gussets, longitudinal HSS tube spacers, and embedded deck plate were meshed as shell elements with an approximately 0.24-in. (6-mm) length. The mounting plate, bolts, square washers, coupling nuts, and heavy hex nuts were modeled as solid elements due to their increased thickness. The post-to-deck connection consisted of several parts, as shown in Figure 127 and listed in Table 8. The parts had the element types and material properties shown in Table 8.

A piecewise linear plasticity material model (MAT_024) was used for all parts, but the stress-strain data differed for the various steels. Material data previously developed from tensile tests for ASTM A992, ASTM A500 Gr. B, and ASTM A36 were used for the steel post and mounting plate, HSS spacers, and square washers, respectively. Similarly, a material model was taken from previous studies for ASTM A325 to be utilized for the bolts and heavy hex nuts. The ASTM A992 material model was used for the mounting plate in spite of the material designation of ASTM A572 Grade 50 for fabrication. Such an alternative for modeling the mounting plate material was considered appropriate due to similarities in yield strengths and stress-strain curve data.

All heavy hex nuts and coupling nuts were modeled as rigid components due to no observed deformations in the component testing. As mentioned in Chapter 7, although permanent deformation was only observed in the W6x15 post, the post plate attachment, gussets, and HSS deck spacers were modeled to be steel deformable parts as forces would have transferred through these parts. The embedded plate was constrained in all directions since the plate was cast-in-place during fabrication of the box-beam girder. Finally, neither the concrete nor the actual box-beam girder were explicitly modeled; the embedded plate was modeled as rigid and constrained to represent the contribution of the box-beam girder.

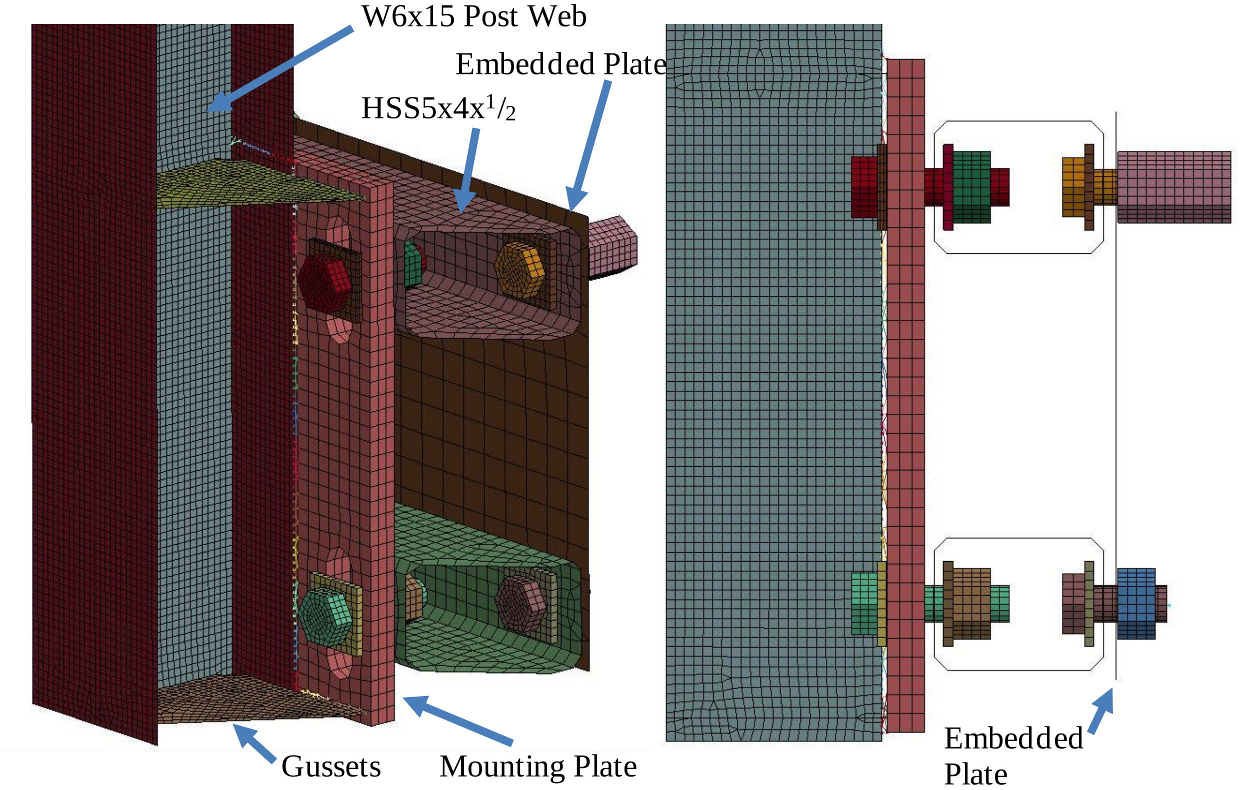


Figure 127. Meshed Post-to-Deck Connection Design

Table 8. Model Parts, Element Types, and Materials

Parts	Part Nos.	Element* Type	Actual Material	Model Material
W6x15 Post Flanges	100001	Shell	ASTM A992	MAT_024
W6x15 Post Web	100002	Shell	ASTM A992	MAT_024
Gussets	100004 - 100007	Shell	ASTM A572	MAT_024
Mounting Plate	100008	Solid	ASTM A992	MAT_024
Post Bolts	100020 - 100023	Solid	ASTM A325	MAT_024
Square Washers	100024 – 100031, 100040 - 100043	Solid	ASTM A36	MAT_024
Heavy Hex Nuts	100032 – 100035, 100046 - 100047	Solid	ASTM A325	MAT_020
HSS5x4x½ Spacers	100009 - 100010	Shell	ASTM A500 Grade B	MAT_024
Deck Bolts	100036 - 100039	Solid	ASTM A325	MAT_020
Coupling Nuts	100044 - 100045	Solid	ASTM A325	MAT_020
Embedded Plate	100011	Shell	ASTM A992	MAT_024

*All element types were formulated as Type 2 integration.

8.1.2 Connection Details

Various techniques were used to connect the model parts. For the W6x15 post flange and web, the nodes were merged. For each bolt, the nodes between the bolt head and shaft were merged. Constrained nodal rigid bodies were used to model the ¼-in. (6.35-mm) fillet weld attaching the 1-in. (25.4-mm) mounting plate to the front face of the W6x15 post and to the gussets. Nodes between the gussets and the post web and flanges were also merged. Of the welded post assembly, the gussets and the post were meshed together, however, the mounting plate was meshed separately.

The bolted connections were explicitly modeled. The welded post assembly was connected to the HSS deck spacers with the bolts, washers, and nuts that were preloaded. For bolt preload, bolt stress was specified using the *INITIAL_STRESS_SECTION command with bolt preloaded defined by a plane through the bolt. The prestress was applied normal to the section plane. The intent of the bolt preload was to model stresses induced by a torqued bolt. Longitudinal springs were utilized for bolts connecting the deck spacers to the coupling nuts and heavy hex nuts at the deck side. The springs were defined as discrete elements, with the spring attached to the bolt head and to the heavy hex nut or coupling nut. Spring forces were determined by considering a linear stiffness of the spring per displacement of the bolt during impact event.

8.2 Bogie Model Details

A previously developed bogie model was added to the post-to-deck connection model, as shown in Figure 128. The bogie vehicle was a rigid-frame bogie with a bogie head consisting of a standard steel pipe with a neoprene belting, as utilized in the actual bogie tests. The bogie mass was 2,522 lb (1,144 kg), the velocity was 20 mph (32.2 km/h), and the impact height was 28 in. (711 mm), similar to test nos. ILOH4-4 through ILOH4-6. A *CONTACT_AUTOMATIC_SINGLE_SURFACE was used for the contact between the bogie head, neoprene pad, and the W6x15 post.

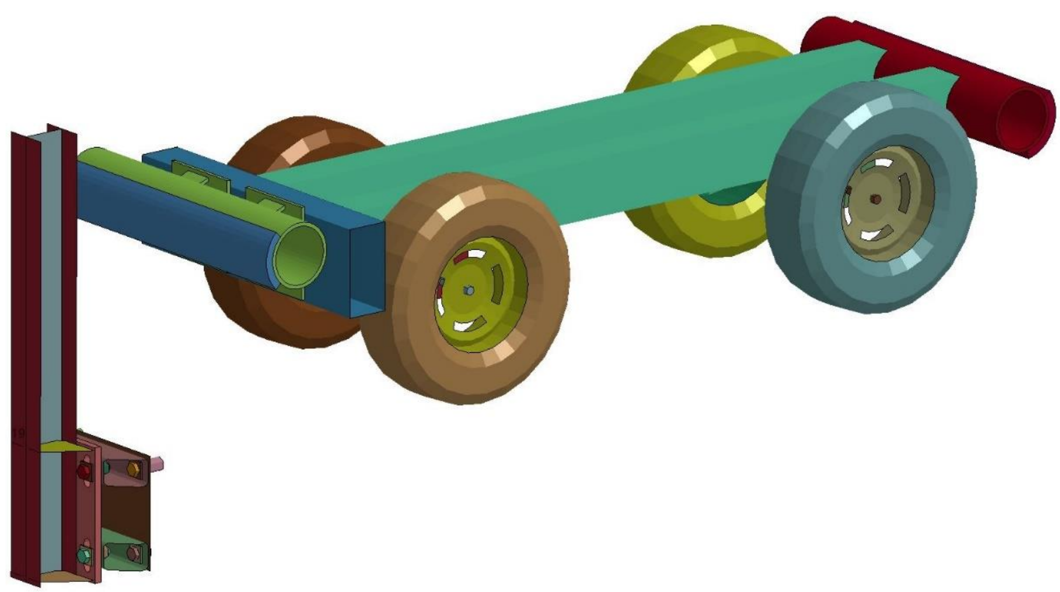


Figure 128. Post-to-Deck Connection Model with the Bogie Vehicle Model

8.3 Simulation Results

The simulation was performed in order to match the simulation results with those obtained from component test nos. ILOH4-4 through ILOH4-6. The goal for the simulation was to be able to match the physical behavior observed in the bogie testing. The W6x15 post was to develop a plastic hinge near the top of the surface of the deck. Additionally, the force and energy vs. deflection curves were compared, as well as the acceleration, velocity, and the displacement of the post with respect to time.

As previously outlined in Chapter 7, for test nos. ILOH4-4 through ILOH4-6, the bogie head impacted the steel post and caused strong-axis bending. During impact in test nos. ILOH4-5 and ILOH4-6, the post rotated backward and developed a plastic hinge near the surface of the deck, above the tensile gussets. Recall that in test no. ILOH4-4 a post flange and web tear occurred at the 6-in. (152-mm) weld at the front flange of the post to the top of the 1-in. (25-mm) mounting plate. Nonetheless, the steel post bent at a similar location above the tensile gussets. Beside the post deformation, no yielding occurred within the mounting plate, HSS longitudinal spacer tubes, embedded plate, or bolted connections in all three bogie tests.

The simulation results were very similar to the general post behavior observed during the bogie tests. As shown in Figure 129, the general simulated behavior followed the behavior in the actual component tests. More deformation occurred in the flanges of the upper W6x15 post region at the point of impact in the simulation, which was not observed in the actual bogie tests, as shown in Figure 130. The post similarly deformed as seen in the bogie tests with the plastic hinge localized above the tensile gussets. Recall the bogie model was a rigid-frame bogie with a bogie head consisting of a standard steel pipe with a neoprene belting, as utilized in the actual bogie tests. In the simulation, the bogie overrode the post as the post deflected back after initial impact, similar to what was observed in the bogie tests.

The force vs. deflection and energy vs. deflection curves for the simulation and the actual bogie tests are shown in Figures 131 and 132, respectively. The simulation tended to follow the same general trend in forces levels and energy levels observed in the bogie tests. A comparison between the bogie test results and the simulation results is shown in Table 9. The main difference was that the post experienced less peak forces yet the simulation results had maximum deflection and total energy similar to the bogie tests. Lastly, recall in Chapter 7, an inelastic analysis of the steel-tube bridge rail established a maximum deflection limit of 12 in. (305 mm) due to potential roll over of the SUT vehicle if the deflection threshold was exceeded. As seen in Table 9, the average forces at a 12-in. (305-mm) deflection were very similar between the bogie tests and the simulation.

Analogous to the force and energy deflection curves, the acceleration, velocity, and deflection curves for the simulation and the actual bogie tests were very similar as the simulation closely followed such parameters observed in the bogie tests. Slight differences lie in the acceleration plot as the simulation results are noticeably less noisy than the bogie tests. The simulation may have produced less data points than the bogie tests, thus filtering the simulation data could have yielded a smoother curve. The acceleration, velocity, and deflection curves are shown in Figure 132.

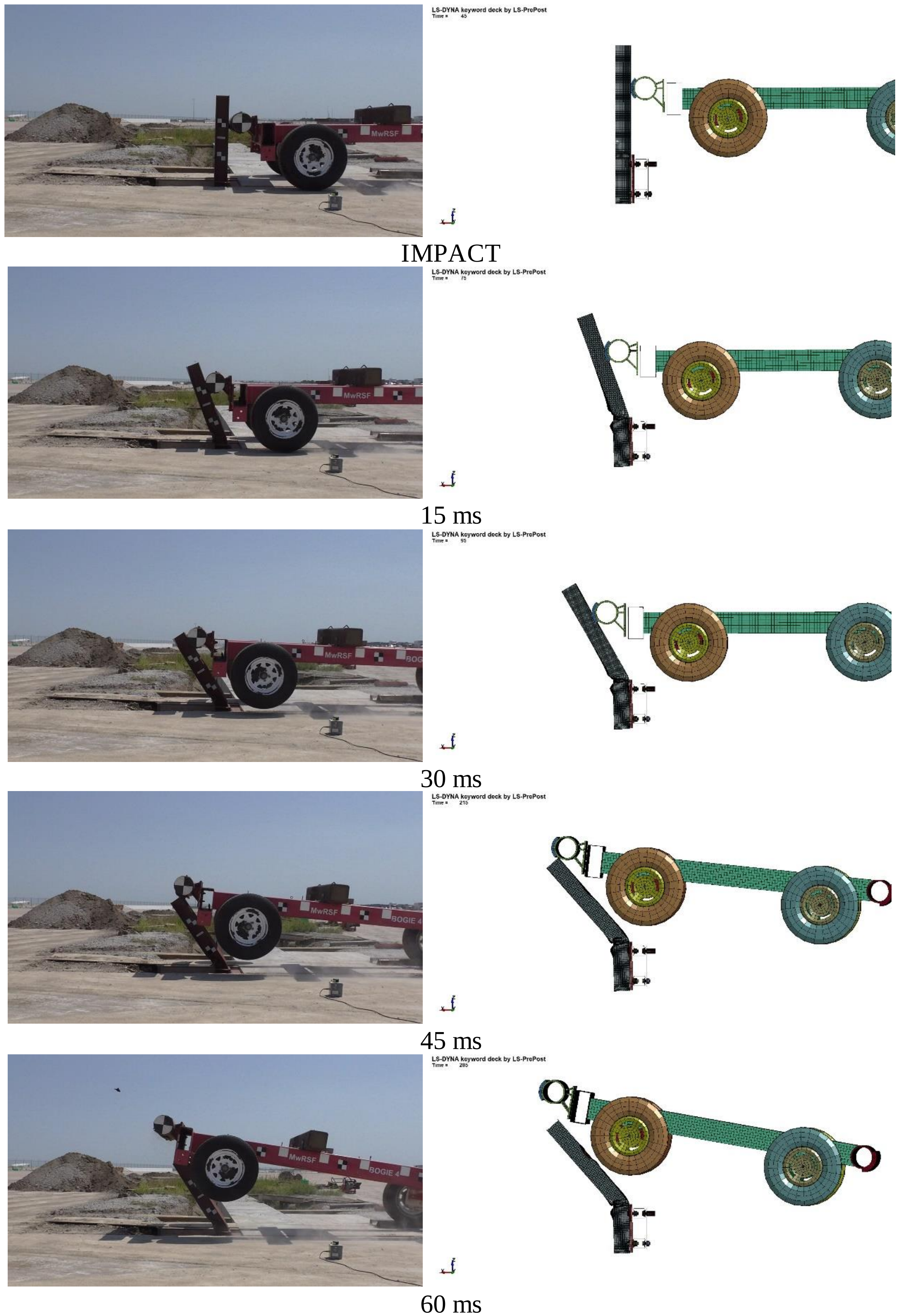


Figure 129. Time Sequential Photographs of Typical Bogie Test and Simulation

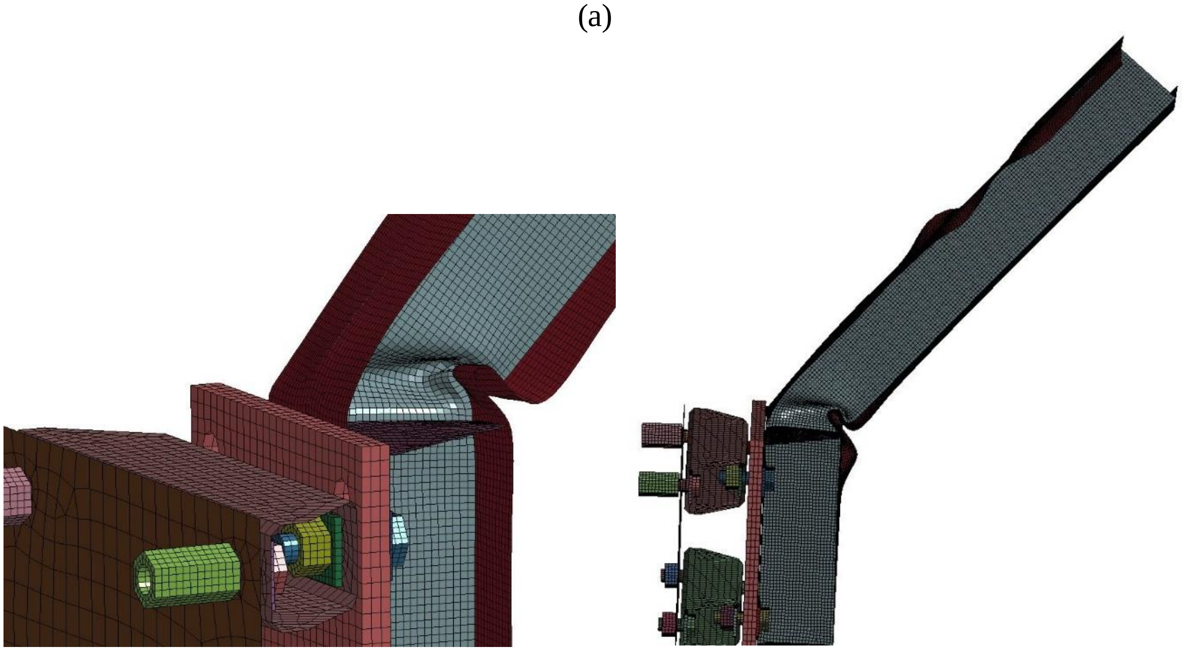
Test No. ILOH4-4

Test No. ILOH4-5

Test No. ILOH4-6

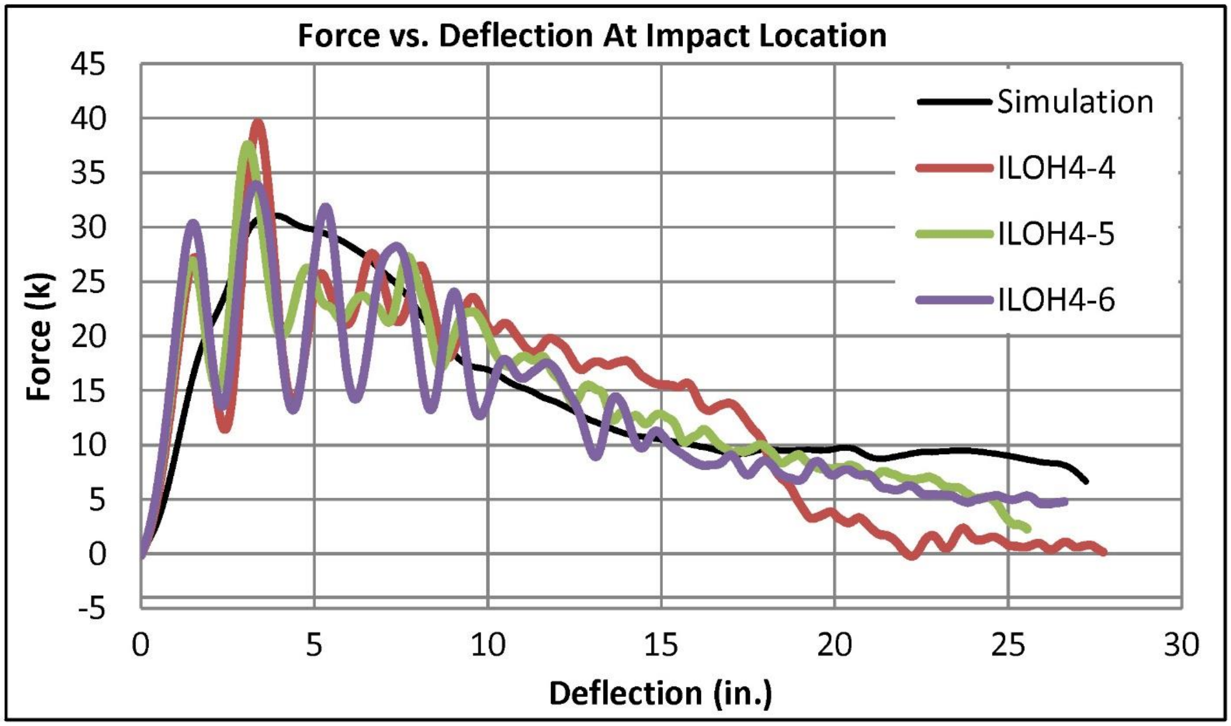
Bogie Test Post Deformation

(a)



(b)

Figure 130. Plastic Hinge and Post Deformation in (a) Test Nos. ILOH4-4 through ILOH4-6 and (b) Simulation



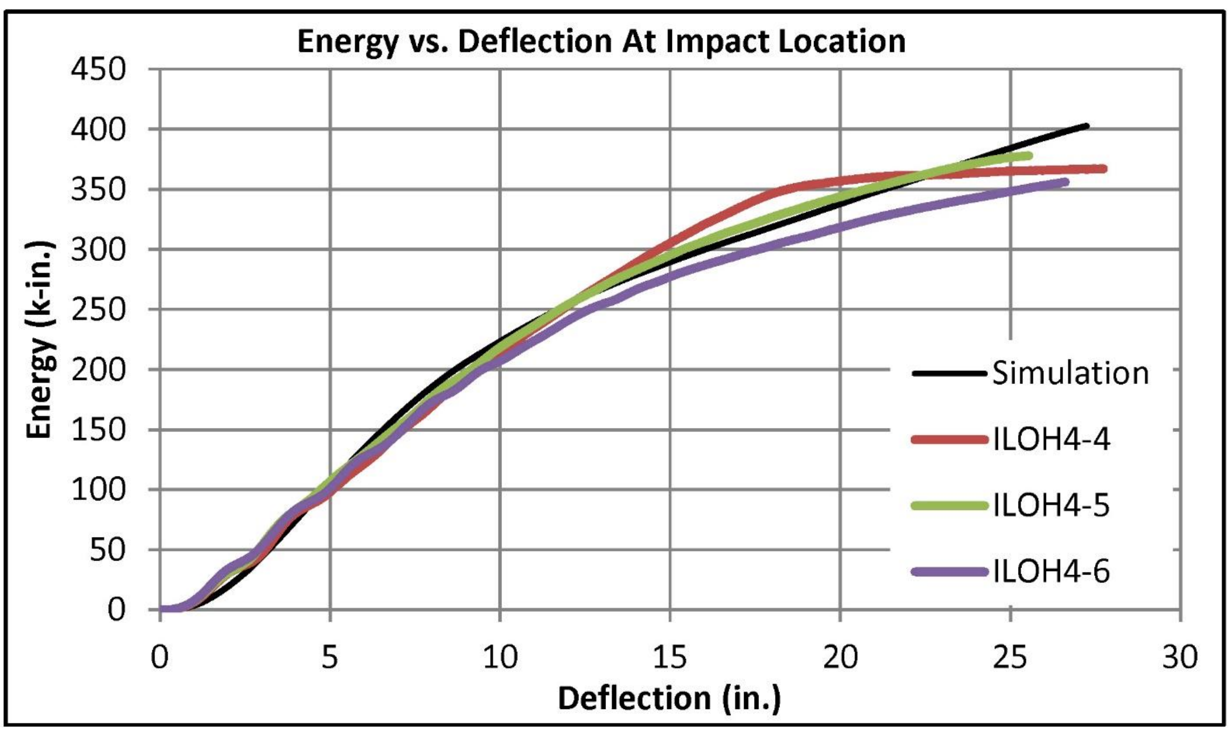


Figure 131. Force and Energy vs. Deflection Curves, Simulation and ILOH4-4 through ILOH4-6

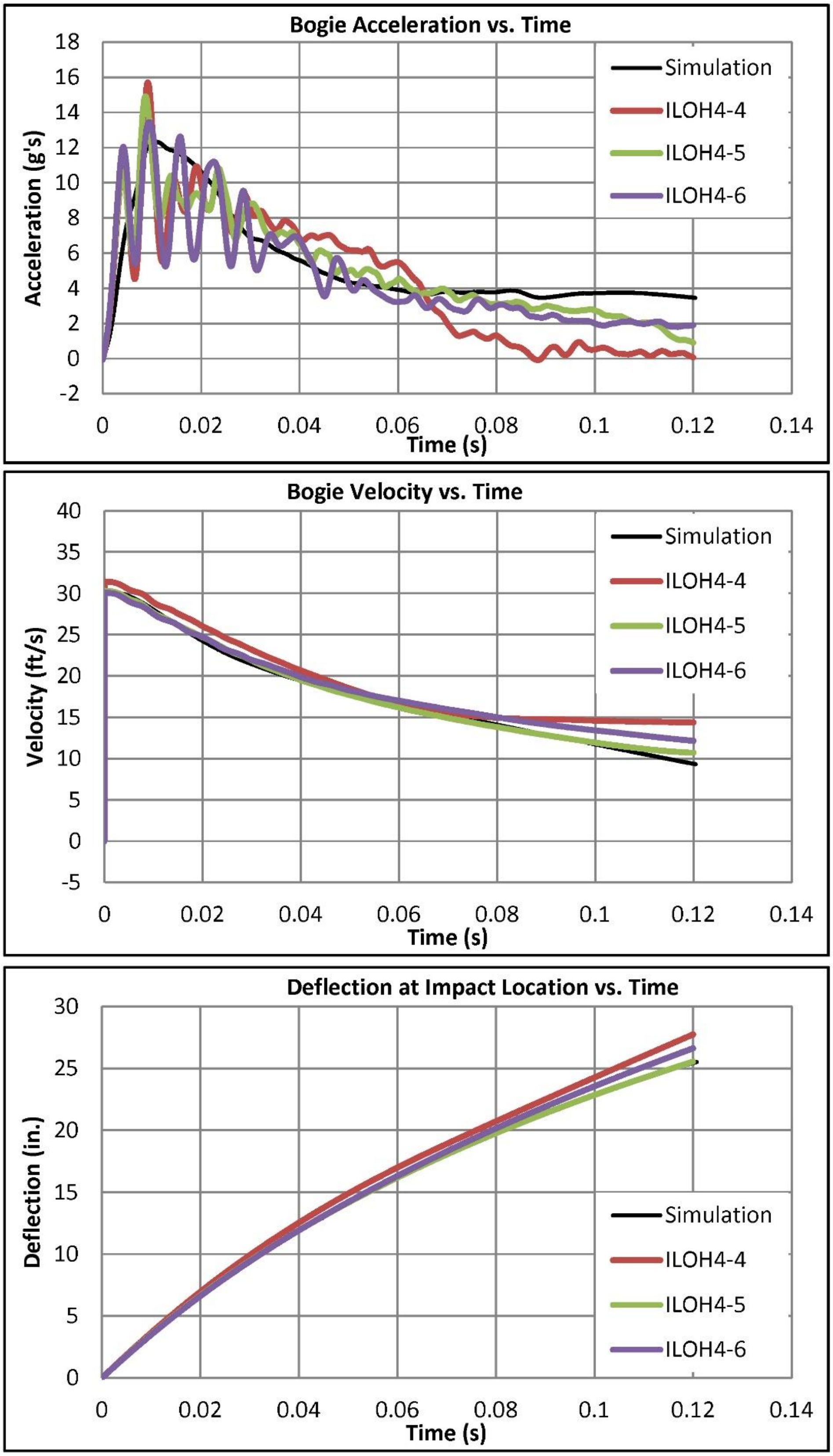


Figure 132. Acceleration, Velocity, and Deflection vs. Time Curves, Simulation and ILOH4-4 through ILOH4-6

Table 9. Comparison of Dynamic Test and Simulation Results

Test No.	Peak Force, kips (kN)	Average Force, kips (kN)				Maximum Deflection, in. (mm)	Total Energy, k-in. (kJ)
		@5”	@10”	@12”	@15”		
ILOH4-4	40 (176)	20 (89)	21 (93)	21 (93)	20 (89)	28 (711)	367 (41)
ILOH4-5	38 (169)	21 (93)	22 (98)	20 (89)	20 (89)	26 (660)	378 (43)
ILOH4-6	34 (151)	20 (89)	21 (93)	19 (85)	19 (85)	27 (685)	356 (40)
Bogie Test Averages	37 (166.1)	20.3 (90.4)	21.3 (94.8)	20 (89)	19.6 (87.5)	27 (685)	367 (41)
Simulation	31 (138)	21 (93)	22 (98)	21 (93)	19 (85)	27 (685)	402 (45)

8.3.1 Prestress and Spring Bolts

Efforts were made to analyze the bolt preload force for the bolts connecting the welded post assembly to the HSS spacer tubes and the bolt spring force in the bolts attaching the HSS spacer tubes to the embedded coupling nuts. It shall be noted that in the actual bogie tests no torque value was specified for the bolts and the preload in the bolts is unknown; all bolts were tightened to be “snug tight.” Therefore, bolt models with preload were utilized to comprehend typical bolt loading during an impact event. Preload forces of prestress and spring bolt models are shown in Figure 133.

For bolts on the post side, the prestress loading was initially equalized at 1 kip (4.5 kN) prior to impact with a peak load of approximately 52 kips (231.3 kN) and 17 kips (75.6 kN) during impact for the top and bottom bolts, respectively. Similarly, the bolts at the deck side had preload forces of 1 kip (4.5 kN) before impact.

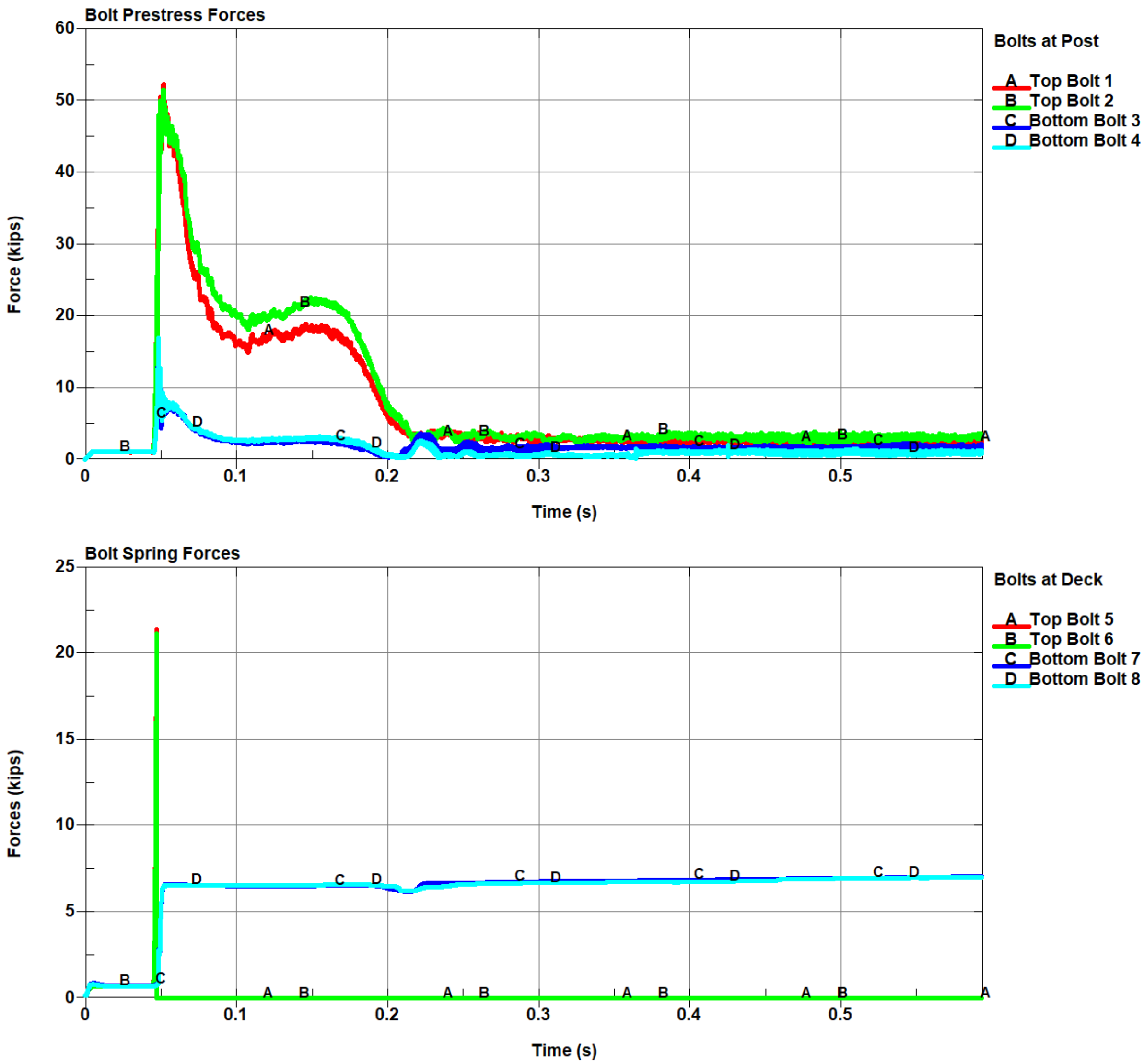


Figure 133. Bolt Prestress and Spring Forces

8.4 Conclusions

LS-DYNA computer simulations were performed with a bogie model impacting a W6x15 post with the selected post-to-deck connection. The steel post deformed by developing a plastic hinge above the tensile gussets as observed in test nos. ILOH4-5 and ILOH4-6. Furthermore, the post front flange deformed slightly more than that observed in the bogie tests. The post flange and web tearing that developed in test no. ILOH4-4, did not occur. Either the plastic hinge or tearing could have occurred. The simulation results closely represented the force, energy, acceleration, velocity, and deflection curves of the actual bogie tests. The average forces and energy through 12-in. (305-mm) of deflection were very similar between the simulation and the bogie tests. Thus, the post-to-deck connection model was considered accurate and could be used in computer simulation of full-scale vehicle crash tests of the MASH 2016 TL-4 steel-tube bridge rail connecting with a MASH 2016 TL-3 approach guardrail transition.

The differences between the simulation and the physical test results may have originated from multiple sources, such as actual versus model material. A difference may have been how the embedded plate in the deck was modeled versus how it performed in the bogie tests. The concrete box-beam girder was not explicitly modeled. Instead, the embedded plate was modeled to be rigid and constrained in all degrees of freedom to simulate the concrete box-beam girder. However, concrete spalling was observed in test nos. ILOH4-4 and ILOH4-5 and hairline cracks were found at the edge of the girder in test no. ILOH4-6, as discussed in Chapter 7. Thus, in the bogie tests, the embedded plate experienced enough movement to cause concrete spalling and cracks along the girder sidewall. The embedded plate in the simulation may have been overly constrained. Nonetheless, the post resisted average forces very similar to what was observed in the actual bogie tests, as shown in Table 9.

9 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The objective of this project was to develop a new MASH 2016 TL-4 steel-tube bridge rail for IDOT and ODOT. The new system was to be side-mounted to the bridge superstructure and utilize a post offset to maximize the traversable width of the deck. Furthermore, the bridge railing system was designed to limit impact loads transferred to the deck, minimize the propensity for deck damage during impacts, and prevent vehicle snag and instabilities during impact events. In this phase of the project, a new post-to-deck connection design was developed to mount the new bridge rail to the bridge decks utilized by IDOT and ODOT. The post-to-deck attachment design consisted of a W6x15 post welded to a steel mounting plate, which was then side-mounted to the bridge deck with two longitudinal HSS spacer tubes.

Several design criteria were established for the new MASH 2016 TL-4 bridge rail. The bridge rail was to incorporate a 39-in. (991-mm) top height to account for a future 3-in. (76-mm) thick roadway overlay on the bridge while maintaining a minimum MASH TL-4 barrier height of 36 in. (914 mm). The railing consisted of three longitudinal steel tubes attached to side-mounted, W6x15 steel posts. The front faces of the middle and lower tube railings were to be flush with the outer edge of the bridge deck to maximize the traversable deck width. The post-to-deck attachment system was to be designed to fully develop the capacity of the W6x15 posts, and the post anchorage hardware was to be designed to sustain impact loads transferred to the deck while preventing deck damage. Both the post-to-deck connection and the anchorage hardware were to be compatible with IDOT and ODOT's existing state deck configurations.

Anchorage loads were investigated to minimize concrete deck damage. The W6x15 steel post was selected over the existing W6x25 post in order to reduce the impact load transferred to the deck and to prevent anchorage breakout in the bridge deck. The W6x15 post was designed to develop its full plastic bending capacity under impact loads. Thus, limiting the loads transferred to this anchor compared to larger posts. This assumption guided the selection of the W6x15 post over the existing W6x25 steel post in IDOT and ODOT's side-mounted bridge rails.

The plastic bending capacity of the W6x15 steel post was determined to be 810 kip-in. (92 kN-m) using a dynamic magnification factor of 1.5. Estimated anchor loads were then investigated on the basis of designing for the worst-case loading condition of all the deck configurations. An effective height of 30 in. (762 mm) was utilized as recommended in NCHRP Project 22-20 for a MASH TL-4 system [6]. Four deck configurations were considered: (1) anchorage to a thick concrete deck slab, (2) anchorage in both a prestressed, concrete box-beam girder and the concrete wearing surface, (3) anchorage only in a prestressed, concrete box-beam girder with a concrete wearing surface, and (4) anchorage in a prestressed, concrete box-beam girder with an asphalt wearing surface.

There were some concerns with deck Configuration #2, shown previously in Figure 32, due to minimal bottom clear cover that posed risks for reduced anchor strength and an increased risk of anchor pullout for the top anchor rods embedded in the concrete wearing surface. The sponsors opted to eliminate deck Configuration #2 as an option for the new bridge rail. Therefore, only deck configurations #1, #3, and #4 were considered for post-to-deck attachment designs and anchorage hardware.

The preliminary anchor loads were further refined in all deck configurations to estimate critical loads transferred into the deck by considering reinforcement patterns, vertical anchor spacing, and concrete cover, with 1-in. (25-mm) diameter anchor rods. Concerns were expressed with the high anchor loads in the 12-in. (305-mm) thick slab design, such as requiring anchor diameters greater than 1 in. (25 mm). In the end, the 12-in. (305-mm) thick deck was eliminated due to much higher estimated anchor loads and 12-in. (305-mm) depth concrete box-beam girders were not adequate in depth for anchoring the side-mount bridge railing, according to IDOT bridge drawings [2].

Thus, an 18-in. (457-mm) minimum depth for the slab deck and a 17-in. (432-mm) minimum depth for the concrete box-beam girder were established for the design of the post anchorage and the post-to-deck attachment hardware. Utilization of the 18-in. (457-mm) slab deck and 17-in. (432-mm) box-beam girder would reduce component sizes of the post-to-deck attachment design, such as bolt diameter size and anchorage development length. Critical loads at the minimum deck thickness design were estimated to transfer 97.4 kips (433 kN) of tension and 73.6 kips (327 kN) of compression loads.

Deck anchorage concepts were explored for the new side-mounted bridge rail. Concerns were noted of the existing deck anchorage methods due to shallow embedment and the use of butt-welded studs that are not ideal for tension anchoring. Options to modify the current anchorage were proposed along with new anchorage concepts. Ultimately, a new anchorage design was selected for all bridge deck configurations with no deck extrusions. Post anchorage hardware was selected featuring fully threaded 1-in. (25-mm) diameter ASTM F1554 Grade 105 anchor rods with coupling nuts welded to an embedded plate cast into the edge of the deck for the tensile connection, as shown in Figure 40. The anchor rods at the top were tensile connections embedded 32½ in. (826 mm) into the deck. Shear welded studs 3 in. (76 mm) long and ½ in. (13 mm) in diameter with heavy hex nuts were utilized in the compression connection. The tensile rods and the compression connection were spaced 11 in. (279 mm) vertically and 16 in. (406 mm) longitudinally to fully develop the tensile forces required for the anchor rods.

Post-to-deck attachment concepts were explored for side-mounting the W6x15 posts to the bridge deck. Existing post-to-deck connection designs feature the Illinois double angle with spacer tube tensile connection and the Ohio base plate with anchor bolts, as shown in Figures 24 and 26. Both states featured a 4-in. (102-mm) post offset from the edge of the deck due to their bridge railings having a 4-in. (102-mm) depth. Concerns with existing DOT attachment concepts included: (1) the Illinois attachment utilizing a spacer tube in the tension connection that is spot welded to the plate on the bridge deck, which would not transfer load for most impacts; and (2) both existing attachments have anchor bolts that span over a 4-in. (102-mm) offset from the front face of the post to the bridge deck, which could include bending in the bolts and lead to premature fracture.

In the upcoming steel-tube bridge railing design, the DOTs selected an HSS12x4x¼ for the top railing and HSS8x6x¼ tube railings for the middle and lower tubes, therein, requiring a 6-in. (152-mm) lateral offset of the post to the deck. Thus, all post-to-deck connection hardware was designed to provide the 6-in. (152-mm) post-to-deck offset to the edge of the deck. A vertical tolerance height was requested by both DOTs for camber and vertical grade adjustments. A vertical tolerance of 3⅛ in. (79 mm) was provided in the post-to-deck attachment for the new bridge rail. This required tolerance could be provided on the post or deck side of the post-to-deck connection.

IDOT and ODOT expressed a preference for having a welded-plate post attachment consisting of a two-plate post with HSS tube spacers, but without use of web stiffeners in the steel post. Plates 1¼ in. (32 mm) thick were required to mitigate plate bending, without gussets, during an impact event. The plate attachments allowed vertical tolerances on the post side of up to 3⅛ in. (79 mm) as requested by the states. The longitudinal tubes were bolted to the two-plate attachments and to the deck side. Thus, IDOT and ODOT selected the 1¼-in. (32-mm) thick two-plate attachment concept, as shown in Figure 54, with HSS5x4x⅜ longitudinal tube spacers for component testing of the concrete box-beam girder. The final design of the post-to-deck attachment was optimized and refined through the component tests.

In order to design a bridge rail attachment that would be applicable to the wide range of bridge decks utilized by Illinois and Ohio, a critical box-beam girder configuration needed to be identified. The most critical box-beam girder design was selected for component testing of the deck attachment and for evaluating the structural integrity of the beam girder. A 36-in. wide x 42-in. (914-mm x 1,067-mm) deep box-beam girder used by ODOT was considered the most critical and weakest deck girder since the 5½-in. (140-mm) thick wall was the thinnest, had the least reinforcement, and had the longest unsupported wall span height. Out of six attachment design concepts, one concept was explored and optimized through seven dynamic bogie tests: a 1¼-in. (32-mm) two-plate attachment with longitudinal tubes. Initially, it was believed that the attachment design concept would be sufficient to withstand the tensile loading transmitted from the post assembly, through the longitudinal tube spacers, to the sidewall of the box-beam girder without gussets supporting the post web and flange at the upper plate attachment. Over the course of seven component tests, the design concept was subjected to a lateral impact (causing strong-axis bending in the post).

Results from the first bogie test, test no. ILOH4-1, showed the 1¼-in. (32-mm) two-plate attachment was insufficient to fully develop the tensile capacity of the welds attaching the upper mounting plate to the front face of the steel post. During the impact event, the upper mounting plate detached completely before the post plastically deformed and rotated backward as the bogie overrode the post. Therefore, no plastic hinge formed near the surface of the deck, as intended. For the second bogie test, test no. ILOH4-2 saw changes to the post assembly involving strengthening the upper 1¼-in. (32-mm) plate with gussets to reinforce the weld strength, which resulted in localized post deformations between the two plate attachments. In this section of the post, the post plastically deformed and the web at the bottom of the post buckled. Along with the plastic hinge forming between the mounting plates as opposed to near the surface of the deck, the upper longitudinal tube bowed outward from the tensile loads. It is believed that due to the plastic deformations forming between the plate attachments, the post was not able to reach its estimated impact loading capacity.

The third bogie test, test no. ILOH4-3, featured design changes in the form of a singular 1-in. (25-mm) mounting plate, the addition of ¼-in. (6.4-mm) gussets at the top and bottom of the mounting plate, and increasing the thickness of the longitudinal tube spacers to ½ in. (13 mm) to help prevent post web buckling and plastic bending between the upper and lower connections. This post-to-deck design was used for test nos. ILOH4-3 through ILOH4-6, although the last test featured a thinner ¾-in. (19-mm) mounting plate. Although test no. ILOH4-3 resulted in a manufacture weld failure, the proceeding component tests for test nos. ILOH4-4 through ILOH4-7 had very similar results as all posts plastically deformed above the tensile gussets and the ideal plastic hinge formed near the surface of the deck. All four successful tests had similar force vs. deflection plots, and tube spacers and the anchor rods were undamaged. The seventh test, test no. ILOH4-7, did show that use of a thinner ¾-in. (19-mm) singular mounting plate would bend at the upper bolt connection, however, no post-test damage was observed to the box-beam girder's sidewall and the post still developed a plastic hinge near the deck surface.

After completion of the component testing, it was clear that the post-to-deck attachment design featuring the 1-in. (25-mm) singular mounting plate with HSS5x4x½ longitudinal spacer tubes would not generate enough load to cause significant damage to the sidewall of the concrete box-beam girder or any of the post-to-deck connections. In several component tests, while installing the post attachment design concept and from post-test impacts, concrete spalling was evident on the sidewalls of the box-beam girder. This spalling was very shallow and near the surface of the sidewall as it was never observed to be deeper than ⅛ in. (3 mm). However, alterations of the post anchorage were considered in order to prevent further sidewall spalling in the subsequent full-scale crash testing of the post-to-deck attachment design with the new bridge rail system. Therefore, full-scale crash testing was recommended with the post and connection attachment utilized in test no. ILOH4-5, with updates to the anchorage design.

Computer simulation utilizing the finite element analysis program LS-DYNA was performed to compare the results of the dynamic component tests of the selected post-to-deck connection. A model of the W6x15 steel post with the post-to-deck connection that was used in test nos. ILOH4-4 through ILOH4-6 was created and validated against the component tests. The intent of the simulation was to create and validate a model to be used in the Phase II development of the thrie-beam transition connection to the steel-tube bridge rail. Bogie vehicle velocity and mass, and impact height, and the post-to-deck connection configuration were taken from test no. ILOH4-5.

The simulation results were similar to the general post behavior observed in the bogie tests. In the simulation, the W6x15 post developed its plastic bending moment capacity by developing a plastic hinge near the top surface of the deck, as seen in test nos. ILOH4-5 and ILOH4-6. The post flange and web tear that developed in test no. ILOH4-4 did not occur, as tearing failure did not occur in the model. Instead, the post deformed with the plastic hinge localized above the tensile gussets. Additionally, the force and energy vs. deflection curves were compared, as well as the acceleration, velocity, and the displacement of the post with respect to time. In comparisons of the bogie tests results with the simulation results, the simulated post experienced less peak force and very similar average force, maximum deflection, and total energy.

A few refinements could be made to the post-to-deck connection model in the forms of more accurately modeling the concrete box-beam girder and the damage observed during the bogie tests. The embedded plate was modeled to be rigid and constrained in all degrees of freedom to simulate the concrete box-beam girder. If concrete damage modeling is desired, the deck and embedded plate model would need to be modified. The model may be able to be further simplified since many of the components did not deform significantly. Overall, the post-to-deck connection model was considered to be accurate and may be used in computer simulation of the full-scale vehicle crash tests of the MASH 2016 TL-4 steel-tube bridge rail connecting with a MASH TL-3 approach guardrail transition.

Since the HSS5x4x½ longitudinal tube spacers and the sidewall of the concrete box-beam girder remained undamaged during the bogie impacts tests, repair to the damage deck attachment system would consist of removing the damaged W6x15 post assemblies, attaching new replacement post assemblies to the undamaged longitudinal tube spacers, and bolting on new tube railing segments.

Preliminary recommendations on the deck reinforcement designs are set henceforth. Final recommendations will be provided after observing deck behavior in the full-scale crash tests. The post anchorage hardware for all bridge decks shall utilize two 1-in. (25-mm) diameter ASTM F1554 Grade 105, fully threaded anchor rods with coupling nuts welded to a $^{3}/_{16}$-in. (4.8-mm) embedded plate. Anchorage embedment length shall be 34½ in. (876 mm). The bottom anchorage shall utilize coupling nuts bolted with 3-in. (76-mm) square washer plates. The vertical spacing between the upper and lower anchorages is established at 11 in. (279 mm) and the longitudinal spacing shall be 16 in. (406 mm). The top anchor rods shall be placed 4 in. (102 mm) below the top surface of the slab deck and 3 in. (76 mm) below the top surface of the concrete box-beam girder, in order to ensure the top anchors are placed below the top steel reinforcement located in the slab deck or located within the top layer of the concrete box-beam girder.

For bridge decks utilizing a slab deck, the minimum thickness shall be 18 in. (457-mm) with the longitudinal, transverse, and vertical reinforcement as established in the bridge drawings by IDOT and ODOT [1-2]. The 28-day compressive strength of concrete shall be 4,000 psi (27.6 MPa). No. 5 U-bar stirrups placed in the slab deck overhang shall be spaced 6 in. (152 mm) longitudinally along the bridge deck. Typical bridge slab deck reinforcement design with post anchorage are shown in Figures 134 and 135. The minimum depth for concrete box-beam girders was set at 17 in. (432 mm) to anchor the side-mounted bridge rail with the longitudinal, transverse, vertical reinforcement, and prestressing strands as established in the bridge drawings by IDOT and ODOT [1-2]. The 28-day compressive strength of the concrete wearing surface shall be 4,000 psi (27.6 MPa). A 28-day compressive strength of the box-beam girder prestressed concrete shall be 6,000 psi (41.4 MPa), and a compressive strength of prestressed concrete at release shall be 5,000 psi (34.5 MPa). A typical concrete wearing surface is 5 in. (127 mm) to 6 in. (152 mm) thick. A typical asphalt wearing surface is 2 in. (51 mm) to 3 in. (76 mm) thick. The post anchorage developed for the new bridge rail could be adapted in both the 36-in. (914-mm) wide and 48-in. (1,219-mm) wide box-beam girders utilized by IDOT and ODOT. Within the hollow core section, No. 4 U-bar stirrups placed under the strands shall be spaced 9 in. (229 mm) longitudinally along the box-beam girder. Typical concrete box-beam girder and deck reinforcement design with post anchorage are shown in Figures 136 through 143. Complete implementation details and recommendations for the bridge rail will be provided in a guidance and implementation report after the completion of the transition testing [10].

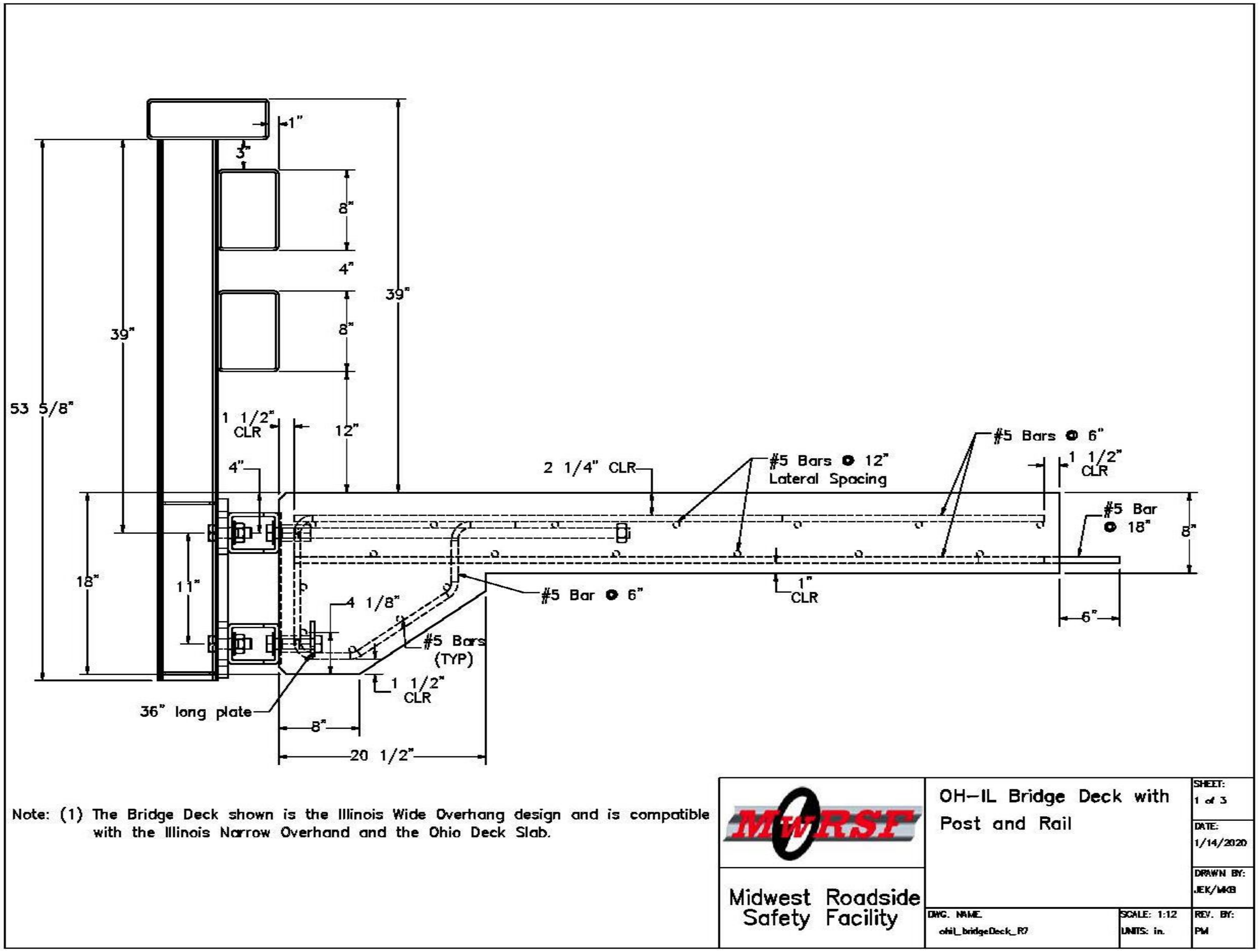


Figure 134. Typical Slab Deck Reinforcement Configuration without Future Overlay

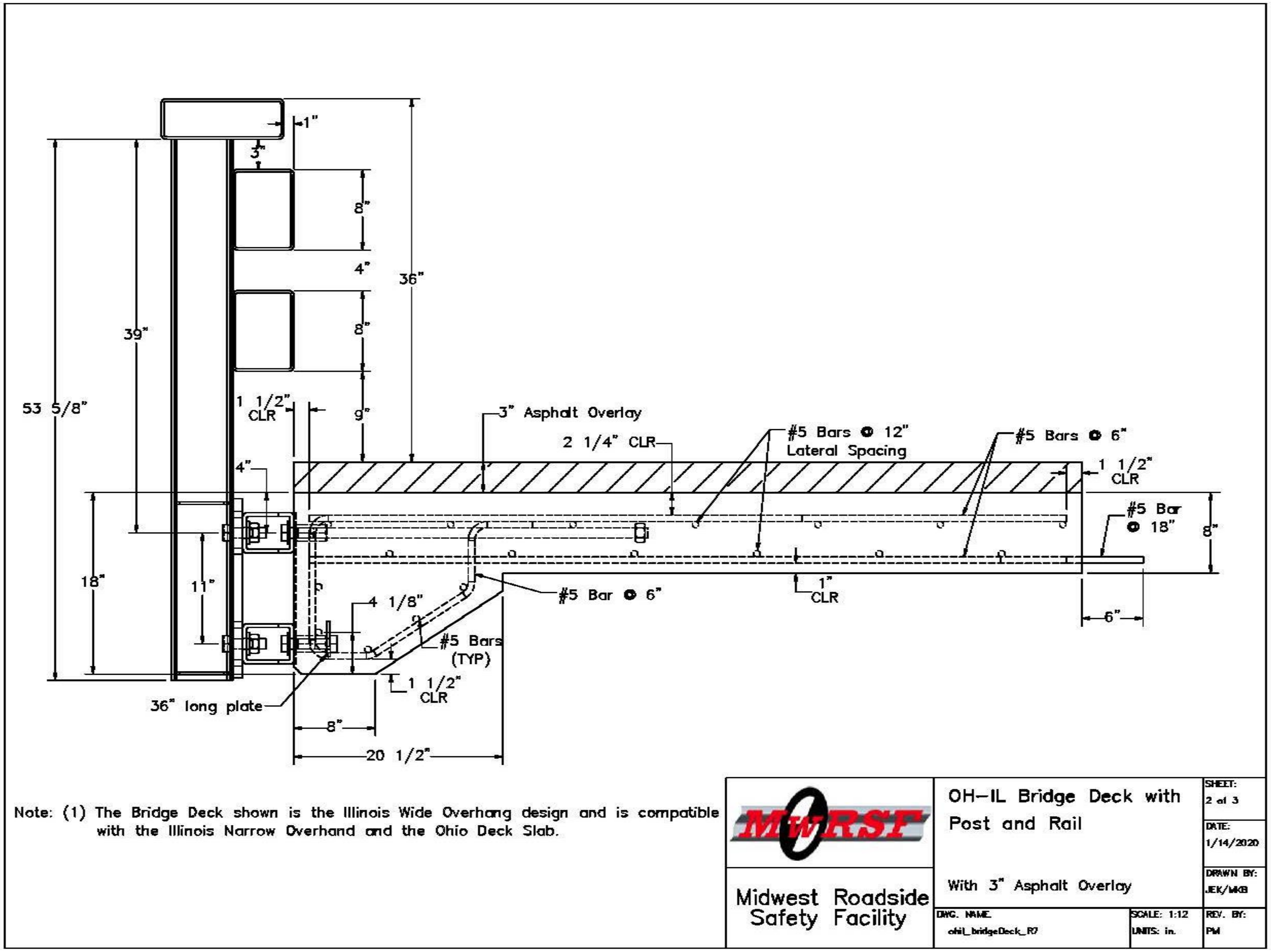


Figure 135. Typical Slab Deck Reinforcement Configuration with Future Overlay

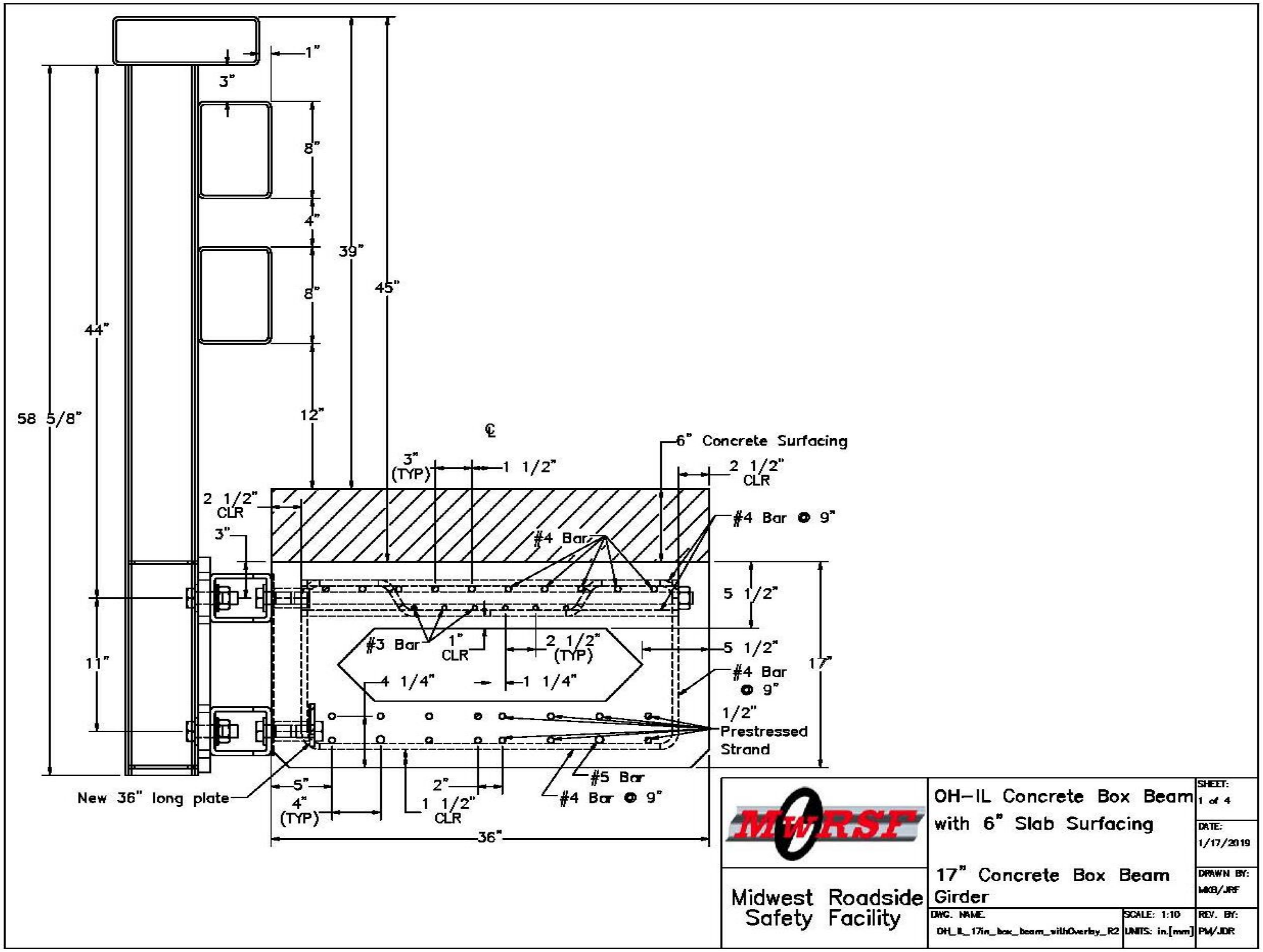


Figure 136. Typical 17-in. (432-mm) Deep Concrete Box-Beam Girder Configuration with Concrete Wearing Surface

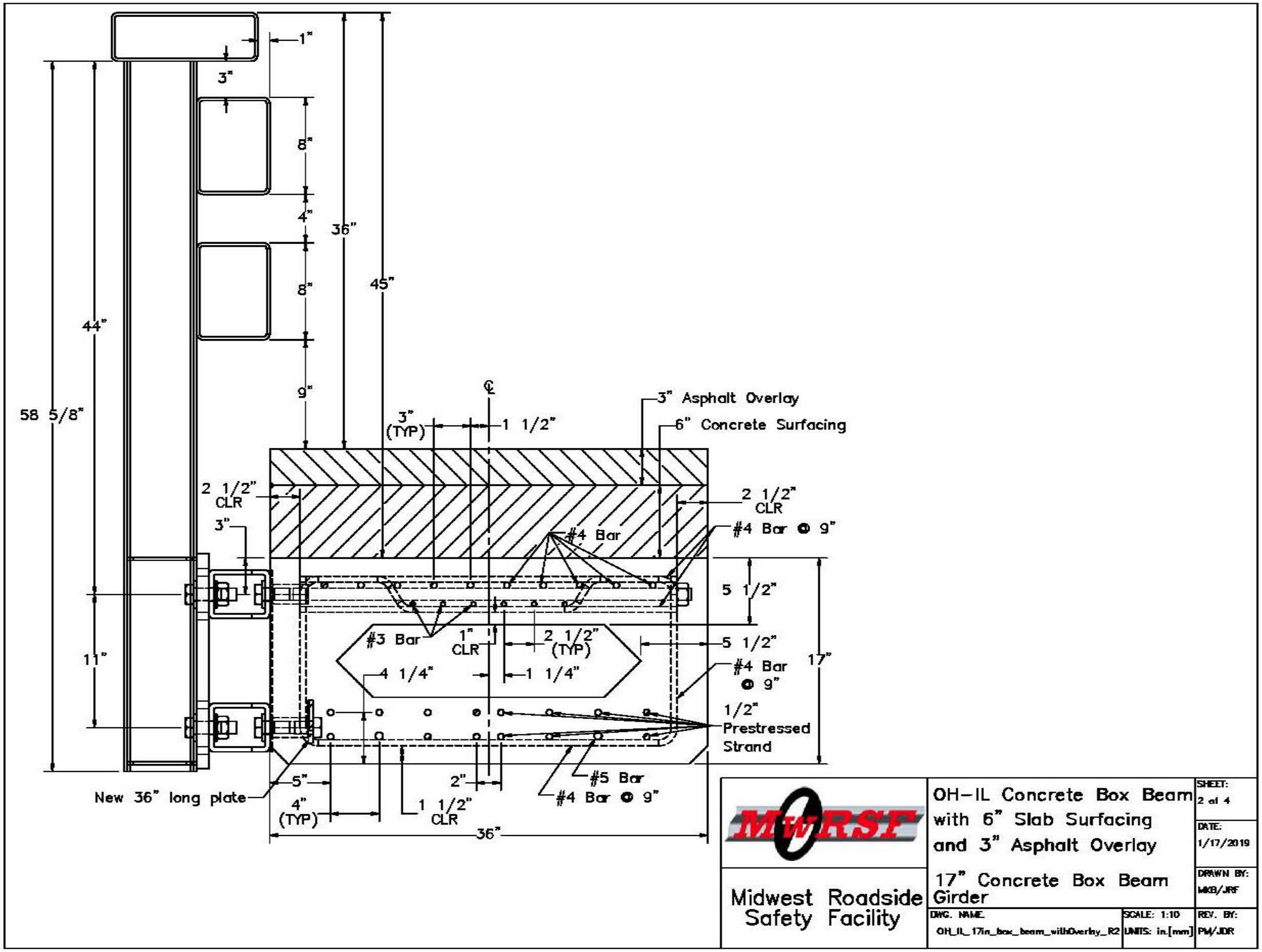


Figure 137. Typical 17-in. (432-mm) Deep Concrete Box-Beam Girder with Concrete Wearing Surface and Future Overlay

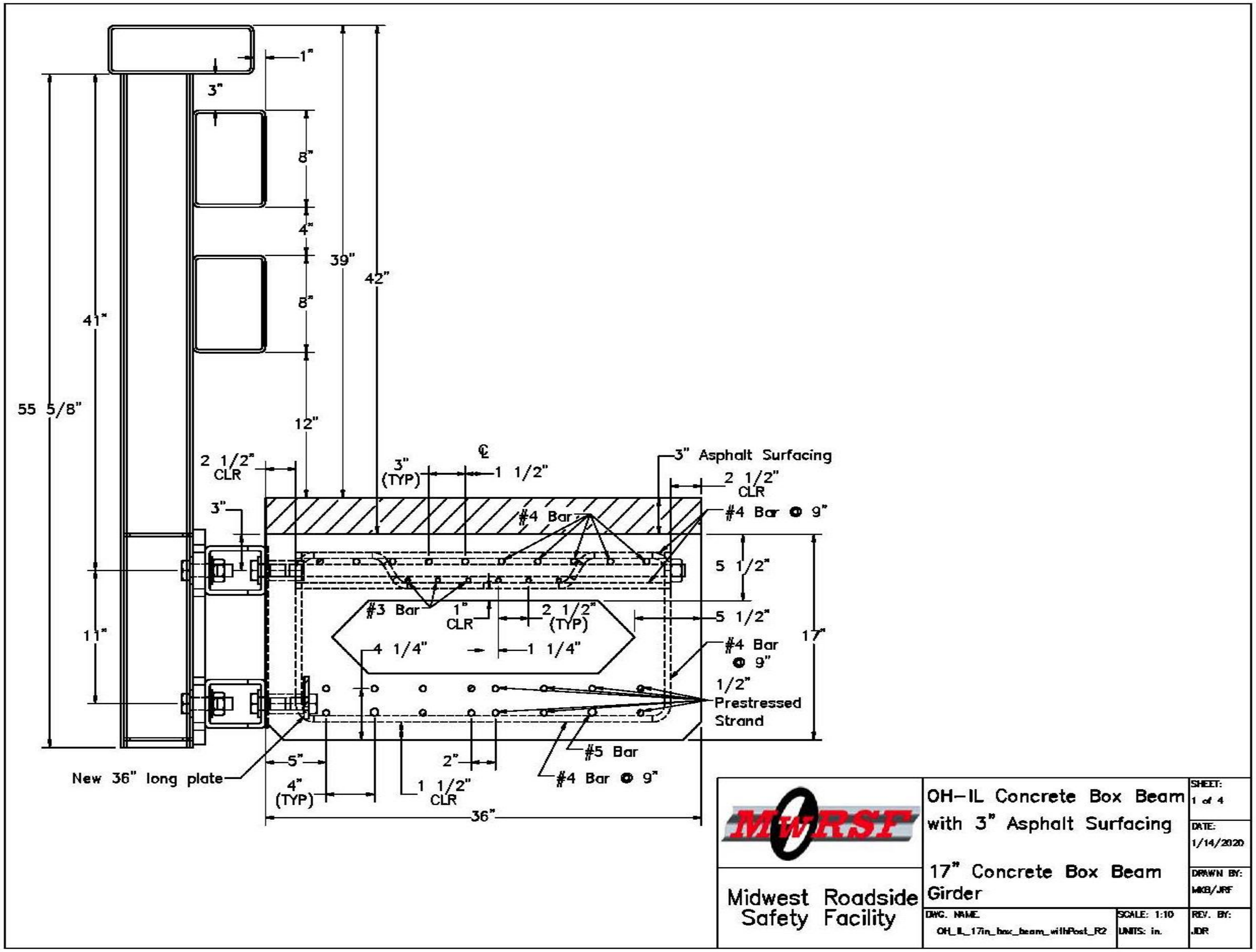


Figure 138. Typical 17-in. (432-mm) Deep Concrete Box-Beam Girder with Asphalt Wearing Surface

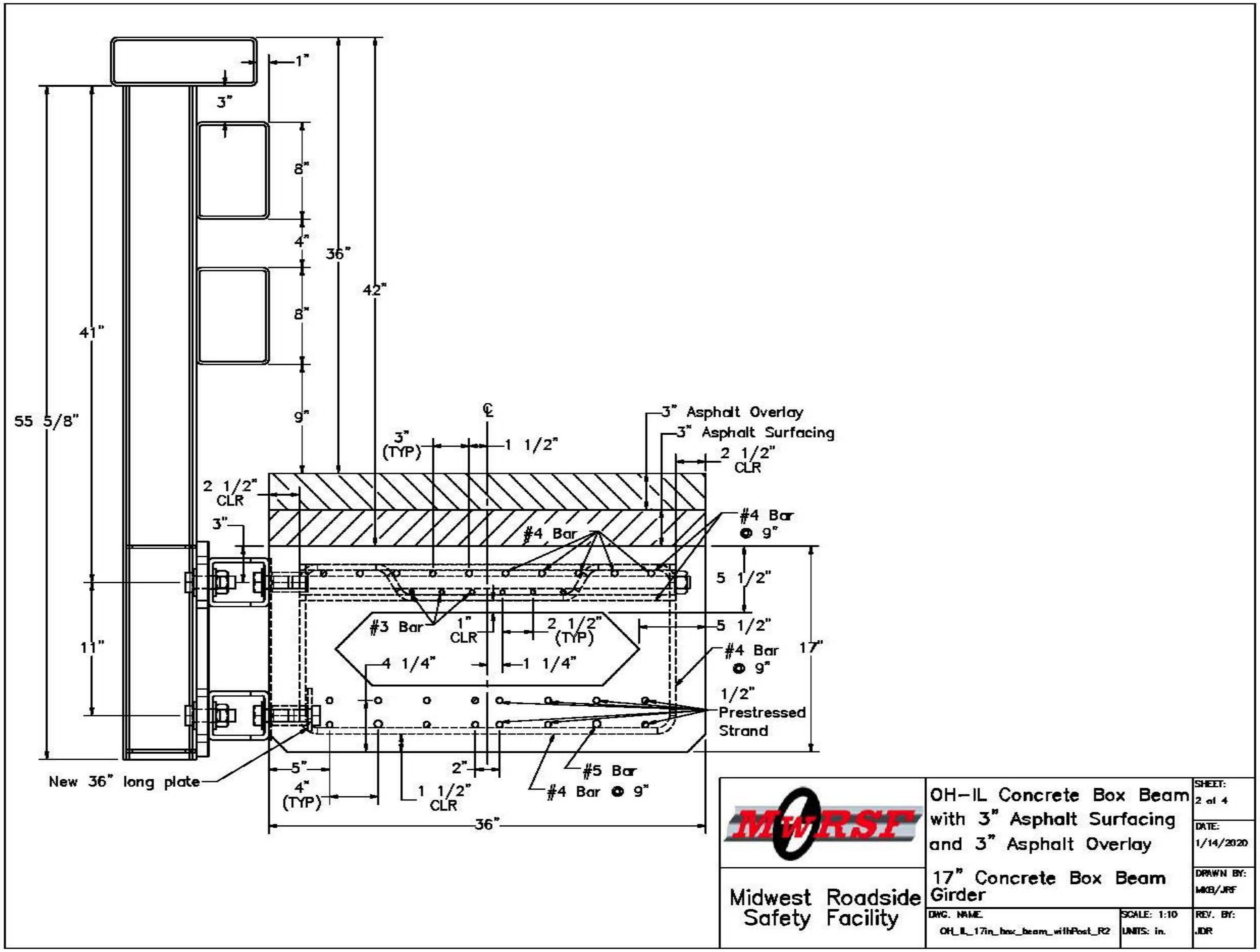


Figure 139. Typical 17-in. (432-mm) Deep Concrete Box-Beam Girder with Asphalt Wearing Surface and Future Overlay

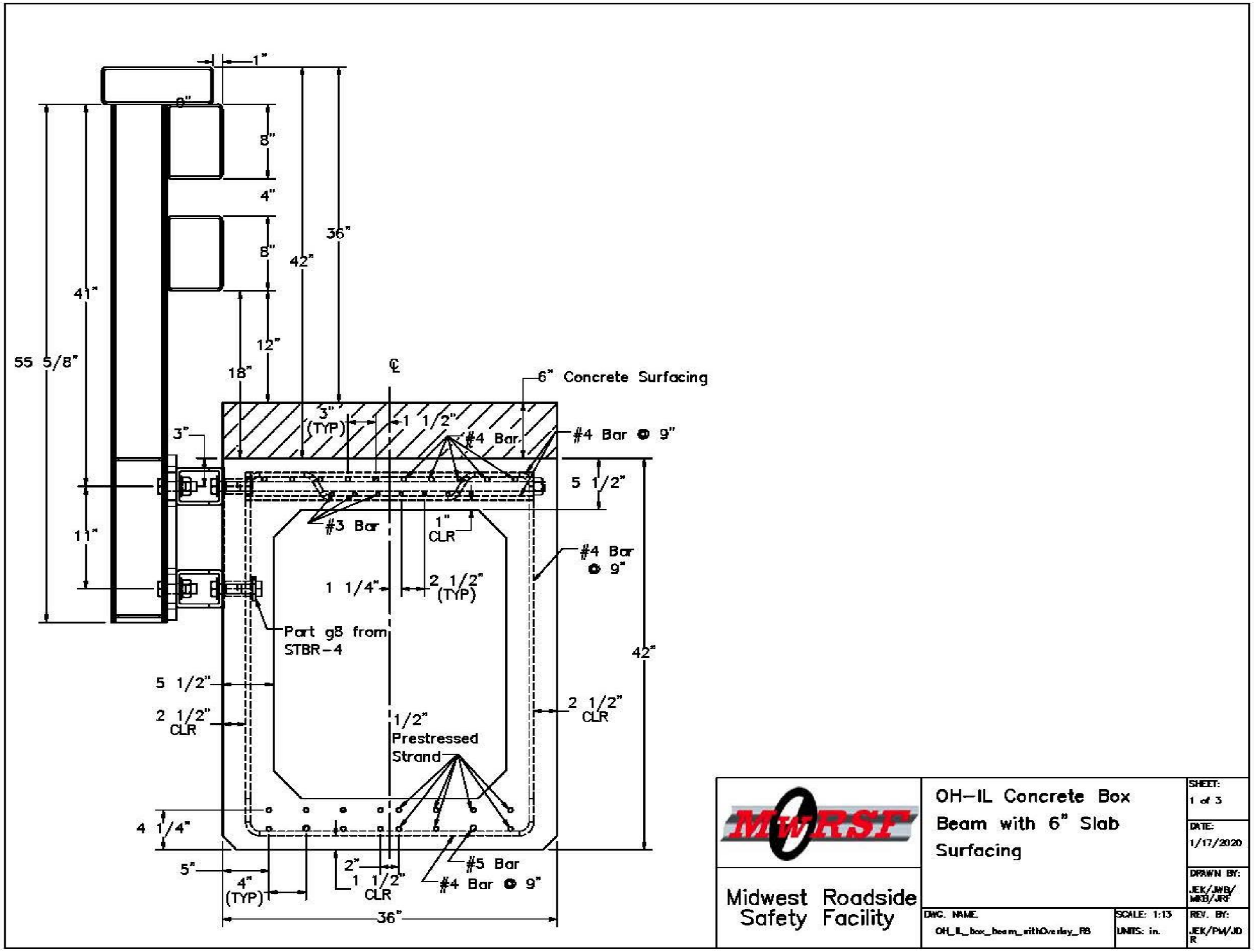


Figure 140. Typical 42-in. (1067-mm) Deep Concrete Box-Beam Girder Configuration with Concrete Wearing Surface

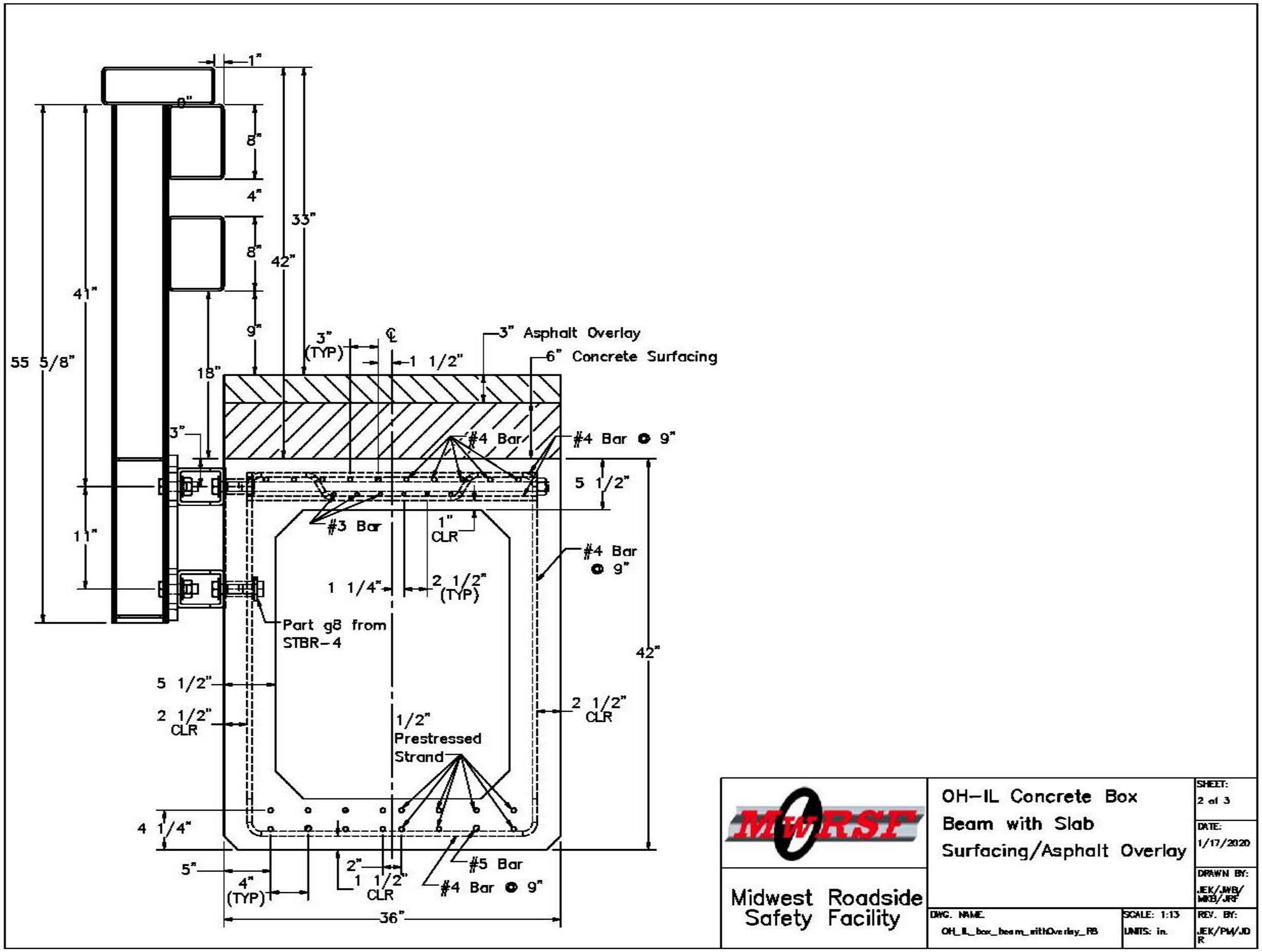


Figure 141. Typical 42-in. (1067-mm) Deep Concrete Box-Beam Girder with Concrete Wearing Surface and Future Overlay

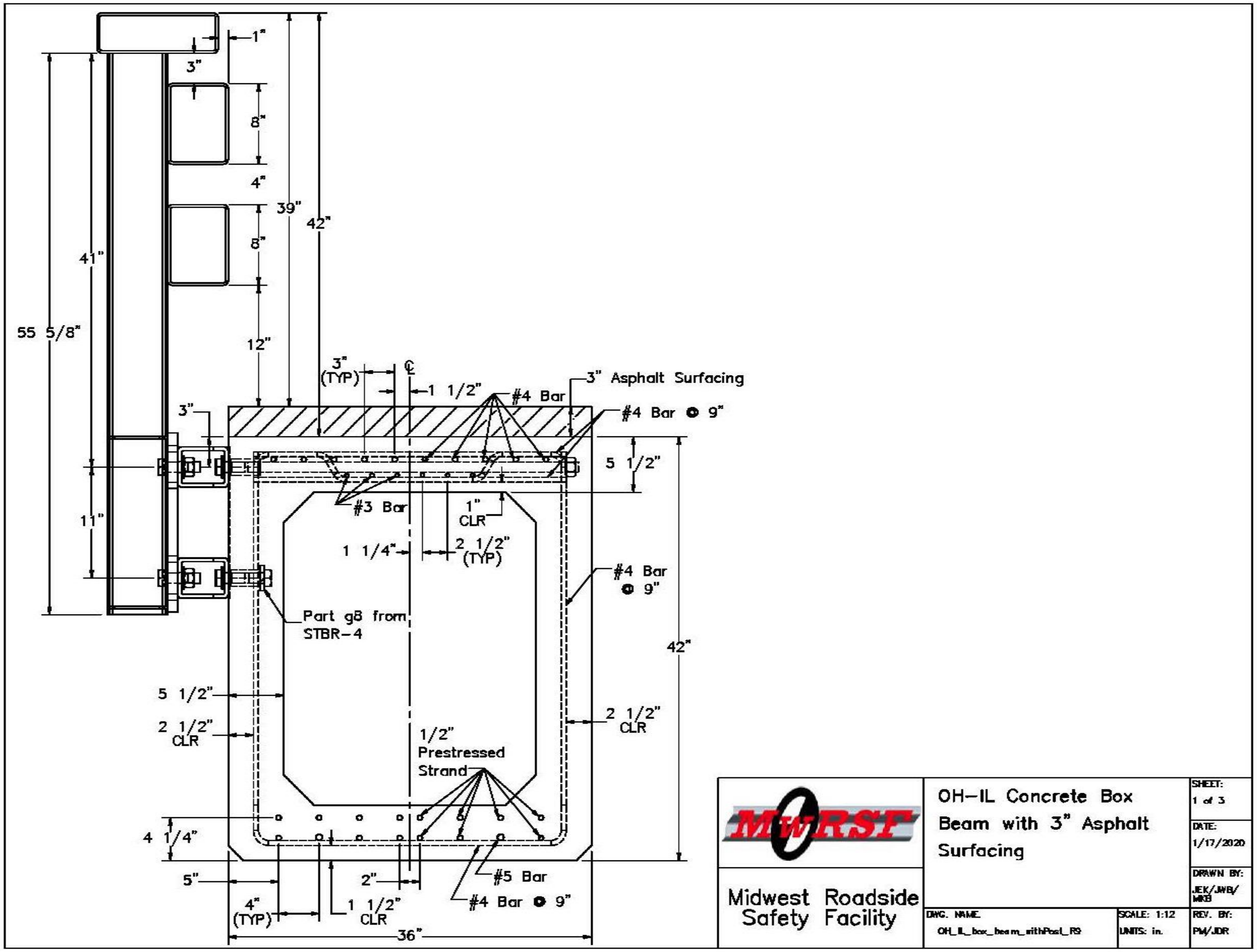


Figure 142. Typical 42-in. (1067-mm) Deep Concrete Box-Beam Girder with Asphalt Wearing Surface

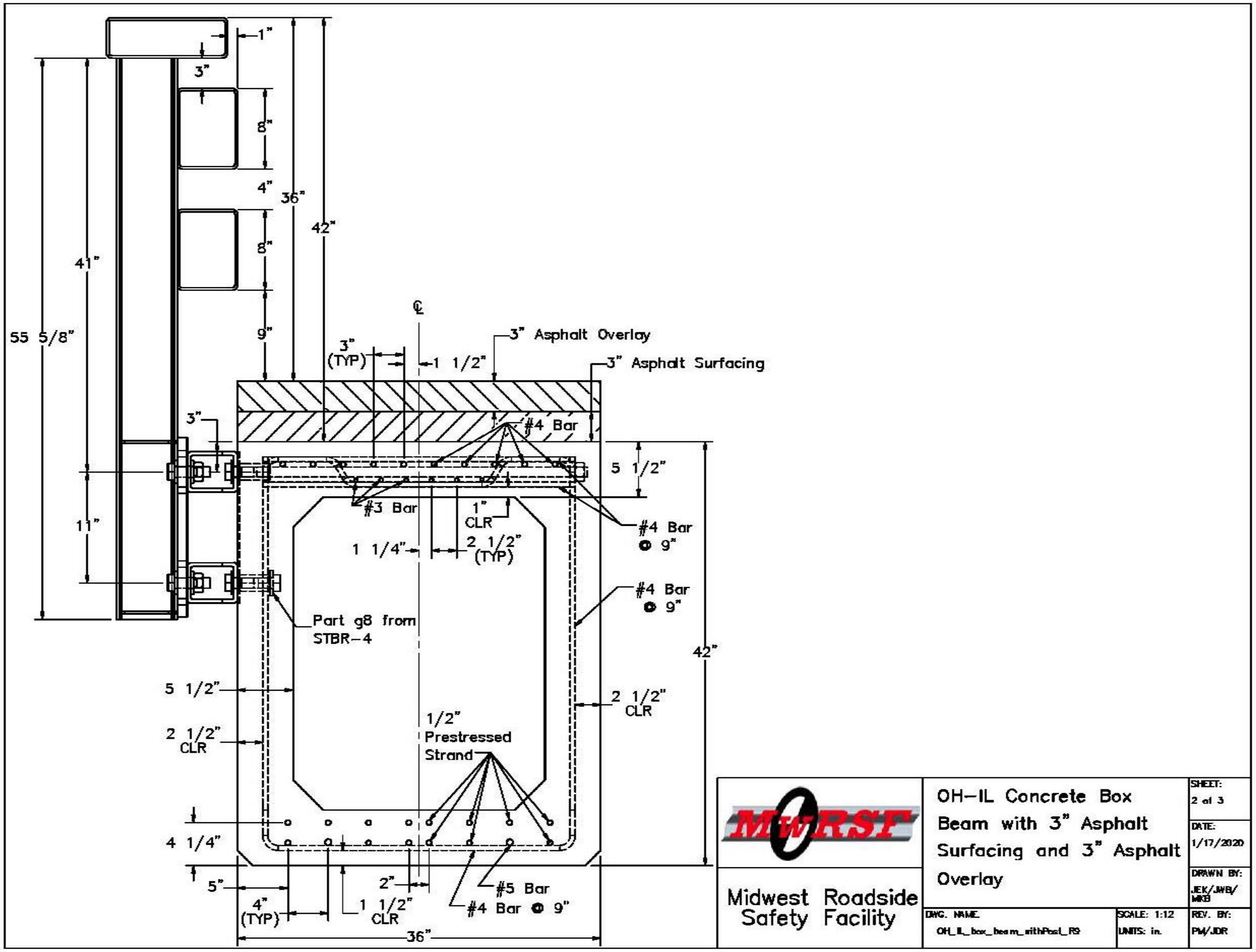


Figure 143. Typical 42-in. (1067-mm) Deep Concrete Box-Beam Girder with Asphalt Wearing Surface and Future Overlay

10 REFERENCES

1. *Twin Steel Tube Bridge Railing,* State of Ohio Department of Transportation, Office of Structural Engineering, Bridge Rail Drawing No. TST-1-99, Columbus, Ohio, July 2016.

2. *Type Side-Mounted Steel Bridge Rail,* State of Illinois Department of Transportation, Bureau of Bridges and Structures, Bridge Rail Drawing No. R-34HMAWS, Springfield, Illinois, November 2016.

3. Ross, H.E., Sicking, D.L., Zimmer, R.A., and Michie, J.D., *Recommended Procedures for the Safety Performance Evaluation of Highway Features*, National Cooperative Highway Research Program (NCHRP) Report 350, Transportation Research Board, Washington, D.C., 1993.

4. *Manual for Assessing Safety Hardware (MASH),* American Association of State and Highway Transportation Officials (AASHTO), Washington, D.C., 2009.

5. *Manual for Assessing Safety Hardware (MASH), Second Edition*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 2016.

6. Bligh, R.P., Brand, J.L., Kim, K.M., and Abu-Odeh, A.Y., *NCHRP Project 22-20(02) Design Guidelines for TL-3 Through TL-5 Roadside Barrier Systems Placed on Mechanically Stabilized Earth (MSE) Retaining Walls,* NCHRP Report No. 663, Texas Transportation Institute, College Station, Texas, August 2010.

7. *MEMO, AASHTO/FHWA Joint Implementation Agreement for Manual for Assessing Safety Hardware (MASH),* U.S. Department of Transportation Federal Highway Administration, Washington, D.C., January 2016.

8. Peña, O., Faller, R.K., Schmidt J.D., Steelman, J.S., Rosenbaugh, S.K., Bielenberg, R.W., and Mauricio, P., *Development of a MASH Test Level 4 Steel, Side-Mounted, Beam-and-Post, Bridge Rail,* Draft, Research Report No. 03-410-20, Midwest Roadside Safety Facility, University of Nebraska – Lincoln, Lincoln, Nebraska, 2020.

9. Mauricio, P., Faller, R.K., Schmidt J.D., Rosenbaugh, S.K., Bielenberg, R.W., Steelman, J.S., Stolle, C., and Peña, P., *Development of a Connection between an Approach Guardrail Transition and a MASH TL-4 Steel-Tube Bridge Rail*, Draft, Research Report No. 03-411-20, Midwest Roadside Safety Facility, University of Nebraska – Lincoln, Lincoln, Nebraska, 2020.

10. *Guidance and Implementation of Steel Railing, Type IL-OH*, DRAFT Research Report No. 03-438-20, Midwest Roadside Safety Facility, University of Nebraska – Lincoln, Draft 2020.

11. *Guide Specifications for Bridge Railings*, American Association of State Highway and Transportation Officials (AASHTO), Washington, DC, 1989.

12. Michie, J.D. *Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances,* National Cooperative Highway Research Program (NCHRP) Report 230, Transportation Research Board, Washington, D.C., 1979.

13. Steinke, D.P., *Federal-Aid Reimbursement Eligibility Process for Safety Hardware Devices,* Memorandum, Federal Highway Administration, July 1997.

14. Bullard, D.L., Bligh, R., and Menges, W., *MASH08 TL-4 Testing and Evaluation of the New Jersey Safety Shape Bridge Rail,* Prepared for National Cooperative Highway Research Program Transportation Research Board National Research Council, Texas Transportation Institute, College Station, Texas, November 2008.

15. *AASHTO LRFD Bridge Design Specifications, 7th Edition,* American Association of State Highway and Transportation Officials, Washington, D.C., 2014.

16. Stoker, J.R., Hackett, R.P., and Doty, R.N., *Dynamic Tests of the California Type 15 Bridge Barrier Rail Series XXV,* State of California Transportation Agency, Department of Public Works, Division of Highways, Sacramento, California, July 1971.

17. Stoughton, R.L., Stoker, J.R., Hale, P., and Nagai, I.M., *Vehicle Impact Tests of a See-Through, Collapsing Ring, Structural Steel Tube, Bridge Barrier Railing,* California Department of Transportation, Sacramento, California, June 1983.

18. Jewell, J., Stoughton, R.L., and Glauz, D., *Vehicle Crash Tests of Type 115 Barrier Rail Systems for Use on Secondary Highways,* Paper No. 1419, Transportation Research Board, Washington, D.C., 1993.

19. Her, V., *Compliance Crash Testing of a Side Mounted Bridge Rail,* Caltrans Division of Research and Innovation, California Department of Transportation, Sacramento, California, March 2013.

20. Buth, C.E., Hirsch, T.J., and Menges, W.L., *Testing of New Bridge Rail and Transition Designs Illinois Side Mount Bridge Railing,* Report No. FHWA-RD-93-066, Texas Transportation Institute, College Station, Texas, September 1993.

21. *Oregon Two-Tube Side Mount Rail,* Oregon Standard Drawings, Oregon Department of Transportation, January 2017.

22. Sheikh, N.M., Menges, W.L., and Kuhn, D.L., *MASH TL-5 Testing and Evaluation of the TBTA Bridge Rail,* Report No. 603911-1-3, Texas Transportation Institute, College Station, Texas, August 2016.

23. Abu-Odeh, A.Y., Williams, W.F., Buth, C.E., and Kim, K.M., *Development of a TL-3 Deep Beam Tubular Backup Bridge Rail,* Report No. FHWA-OH-2010-18, Texas Transportation Institute, College Station, Texas, October 2010.

24. *Michigan Bridge Railing, Guardrail Type, and Guardrail Anchorage-Bridge, Nested Type,* Bureau of Highways Standard Plans, Michigan Department of Transportation, September 1990.

25. Buth, C.E., Hirsch, T.J., and Menges, W.L., *Testing of New Bridge Rail and Transition Designs Oregon Side Mounted Bridge Railing*, Report No. FHWA-RD-93-059, Texas Transportation Institute, College Station, Texas, September 1997.

26. Polivka, K.A., Faller, R.K., Ritter, M.A., and Rosson, B.T., *Development of the TBC-8000 Bridge Railing*, MwRSF Research Report No. TRP-03-30-93, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, September 5, 2000.

27. Polivka, K.A. Faller, R.K., Ritter, M.A., Rosson, B.T., Fowler, M.D., and Keller, E.A., *Two Test Level 4 Bridge Railing and Transition Systems for Transverse Glue-Laminated Timber Decks*, Report No. TRP-03-71-01, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, January 30, 2002.

28. Thiele, J.C., Sicking, D.L., Faller, R.K., Bielenberg, R.W., Lechtenberg, K.A., Reid, J.D., and Rosenbaugh, S.K., *Development of a Low-Cost, Energy-Absorbing Bridge Rail*, Report No. TRP-03-226-10, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, August 11, 2010.

29. Schneider, A.J., Rosenbaugh, S.K., Faller, R.K., Sicking, D.L., Lechtenberg, K.A., and Reid, J.D., *Safety Performance Evaluation of Weak-Post, W-Beam Guardrail Attached to Culvert*, Report No. TRP-03-227-14, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, February 12, 2014.

30. Society of Automotive Engineers (SAE), *Instrumentation for Impact Test – Part 1 – Electronic Instrumentation*, SAE J211/1 MAR95, New York City, NY, July 2007.

11 APPENDICES

Appendix A. Post-to-Deck Connection Design – Sample Calculations

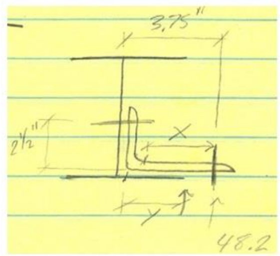
Design of Double Angle Connection

Tension	96.37 kips	17" bridge deck design loadings
Compression	72.81 kips	

Double Angles bear against inside front flanges of and along post web.

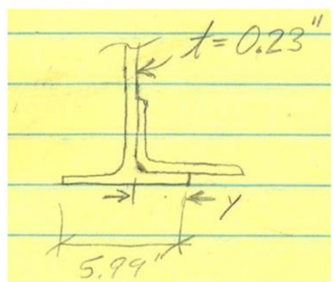
Moment arm distance from centerline of tension bolt to end of flat distance of angle, x

$x = 3.75" - t_w/2 - k =$ 2.36 in

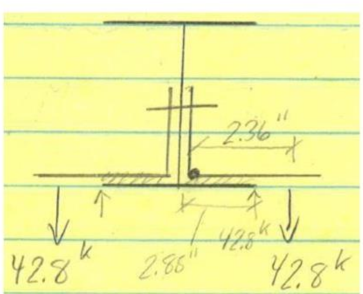


Moment arm distance from edge of post flange to end of angle, y

$y = b_f/2 - t_w/2 =$ 2.88 in



FBD



$\Sigma M = 42.8\,(2.88) - 42.8\,(2.36)$
= 22.26 in-kips

Plastic Bending Moment
$\varnothing M_p = \varnothing F_y Z_x$
$22.26 = 0.9\,(50)\,b*h^2/4$
h = 0.903 in. Select a 6" x 4" x 1" angle

Figure A-1. Double Angle Connection Design

Design for Bolts in Post Web for Double Angle Connection

Tension	96.37 kips	17" bridge deck design loadings
Compression	72.81 kips	

Bolts experience tension loading transferred into deck + vertical (shear) loading

Vertical loading, F_v = 8.25 kips

(from applied vertical loading of F_v = 33 kips from NCHRP 22-20 for MASH TL-4 rail systems)

---> The Fv = 33 kips is applied over 18 ft
---> Assume Fv is distributed evenly over 4 post
(may actually have higher concentration over middle 2 posts)

=> $$Fv = \frac{33\text{ kips}}{4\text{ posts}} = 8.25 \text{ kips per post}$$

Determine Shear Loading over 2 Bolts per Post

$$\frac{8.25\text{ kips}}{2\text{ bolts per post}} = 4.13 \text{ kips per bolt per post}$$

Find resultant vector load between downward shear force and horizontal tension loading for design of bolts in the post web

$$\sqrt{48.2^2 + 4.13^2} = 48.4\ kips$$

ØRn = ØFnAb

$48.4 \leq 0.75\,(68)\,\pi d^2/2$

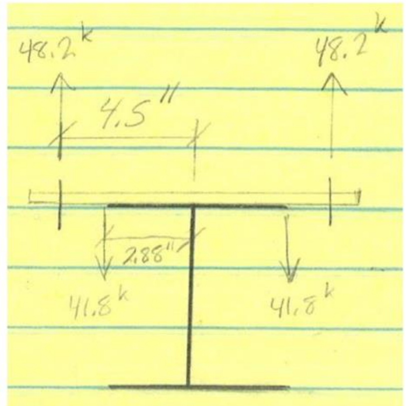
d = 1.10 in. $\boxed{\text{Use 2 - } 1^1/_8 \text{ dia. A325 bolts per post}}$

Figure A-2. Design for Bolts in Post Web for Double Angle Connection

Design of Post Plate Attachment

17" bridge deck design loadings

Tension	96.37	kips
Compression	72.81	kips



Plastic Bending Moment
$\text{Ø}M_p = \text{Ø}F_yZ_x$
Plastic Bending Moment
$\text{Ø}M_p = \text{Ø}F_yZ_x$

Loadings:
Tension anchor loading is 96.4 kips. Assume each bolt resists 48.2 kips.
The 12"x15" on the front flange of the post is fillet welded to post (1/4" weld)

Welding capacity of a 6" weld across top of plate to post and 4" along sides of plate to flange

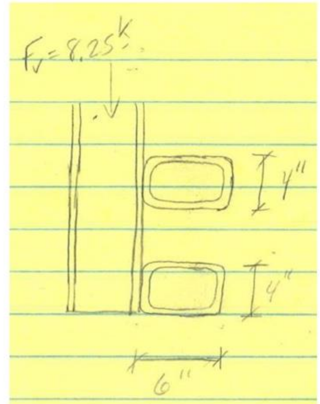
$\text{Ø}R_n = \text{Ø}F_nA_{we}$
= 116.92 kips (See Post Assembly Weld Connection Calcs)

From FBD: $\Sigma M = 48.2\ kips\ (24.5") - 58.6\ kips\ (2.88\ ")$
= 48.53 in-kips

$\text{Ø}M_p = \text{Ø}F_yZ_x$
$48.53 = 0.9\ (50)\ b{*}h^2/4$ Let b = 5"
1.236 in. Need a minium 1.25" thickness for plate attachment without gussets

Figure A-3. Post Plate Attachment Design

Design of HSS Spacers



Tension	96.37 kips	17" bridge deck design loadings
Compression	72.81 kips	

Treat the HSS spacer as a fixed-fixed beam with a point load at the end

Vertical loading, F_v = 8.25 kips

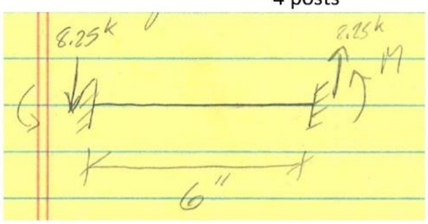
(from applied vertical loading of F_v = 33 kips from NCHRP 22-20 for MASH TL-4 rail systems)

---> The Fv = 33 kips is applied over 18 ft

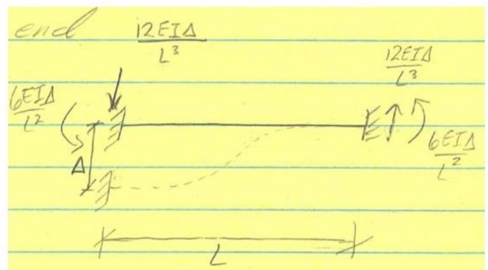
---> Assume Fv is distributed evenly over 4 post

(may actually have higher concentration over middle 2 posts)

=> $Fv = \frac{33 \text{ kips}}{4 \text{ posts}}$ = 8.25 kips per post



FBD



FEM

Let Fv = 8.25 kips be the vertical force at each fixed end over 4 HSS sidewalls (2 HSS)

$$F = \frac{12EI\Delta}{L^3} = \frac{8.25 \text{ kips per post}}{4 \text{ HSS sidewalls}}$$

$$I = \frac{(2.06 \text{ kips})(6 \text{ in})^3}{12(29{,}000 \text{ ksi})(0.25 \text{ in})}$$

$$\frac{(6 \text{ in})\, h^3}{12} = 0.00514 \text{ in}^4$$

$$h = 0.217 \text{ in}$$

Consider a 0.25 in. deflection for △

L =	6	in
E =	29,000	ksi
△ =	0.25	in

$\boxed{\text{Select an HSS6x4x}^3/_8}$

Figure A-4. HSS Spacer Design

IL/OH TL-4 Steel Tube Bridge Rail

Lateral Load Resistance
Post-to-Deck Connections

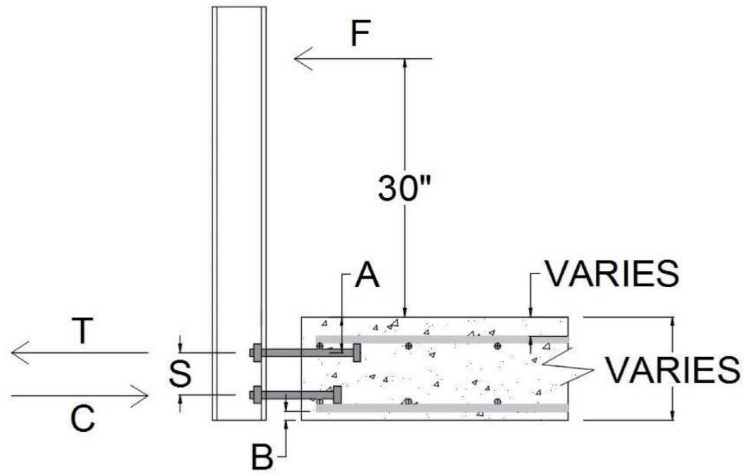
Calculations Overview

Design Description:

These lateral load resistances are based on the bridge deck configurations provided by the Illinois and Ohio Department of Transportations. They are used to provide preliminary anchor loadings for the side-mount post-to-deck connections of the MASH TL-4 steel tube bridge rail.

Plastic Moment Capacity in W6x15 Post: $M = 1.5F_yZ_x =$ 810 in*k

Concrete Slab on Deck Configuration



Givens:

Barrier Rail Load Effective Height (H_e)	
30	in

Slab Reinforcement		
Lateral:		#6 @ 9"
Longitudinal:		#6 @ 18"
Clear Cover (in)	Top:	2.25

Depth of Slab Deck, t_s (in):
12 - 18

Selecting Anchor Ø (in)
Varies

Inputs:

Select Anchor Ø: 1 in
Select Slab Depth: 12 in

Select Top Clear Cover: 1 in
Select Compression Block Cover: 1.5 in

Resulting Anchor Forces:

Tension Anchor Loading (T):	**159.55**	kips

Compression Anchor Loading (C):	**135.00**	kips

Moment Arm: **33** in
Applied Load (F): **24.55** kips

From Deck to Centerline of T anchors
A: **3** in

Between T and C anchors
S: **6** in

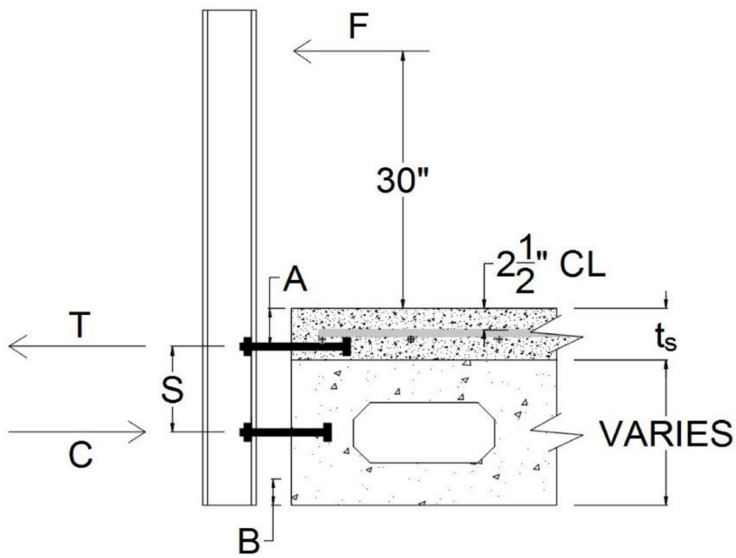
Designed by: Pascual Mauricio

Figure A-5. Concrete Slab on Deck Configuration, Preliminary Anchor Loadings

IL/OH TL-4 Steel Tube Bridge Rail

Lateral Load Resistance
Post-to-Deck Connections

Slab & Deck Box Beam Configuration



Givens:

Barrier Rail Load Effective Height (H_e)	
30	in

Slab Reinforcement		
Lateral:		#4
Longitudinal:		#4
Clear Cover (in)	Top:	2.50
	Bot.:	1.5

Depth of Slab Deck, t_s (in):
Varies

Selecting Anchor Ø (in):
Varies

Box Beam Reinforcement	
Lateral:	#4
Longitudinal:	#4
Clear Cover (in)	1.50

Sizes of Deck Box Beams (in)	
Depth:	17, 21, 27, 33, 42
Widths:	36 or 48

Anchor Spacing, S (in)
Varies

Inputs:

Select Box Beam Depth:	17	in
Select Slab Thickness, t_s:	6	in
Select Anchor Ø:	1	in
Select Anchor Spacing, S:	10	in
Select Compression Block Cover, B:	1.5	in

Resulting Anchor Forces:

Tension Anchor Loading (T):	**104.82**	kips
Compression Anchor Loading (C):	**81.00**	kips
Moment Arm:	**34**	in
Applied Load (F):	**23.82**	kips

From Deck to Centerline of T anchors
A: **4** in

Between T & C anchors
S: **17.50** in

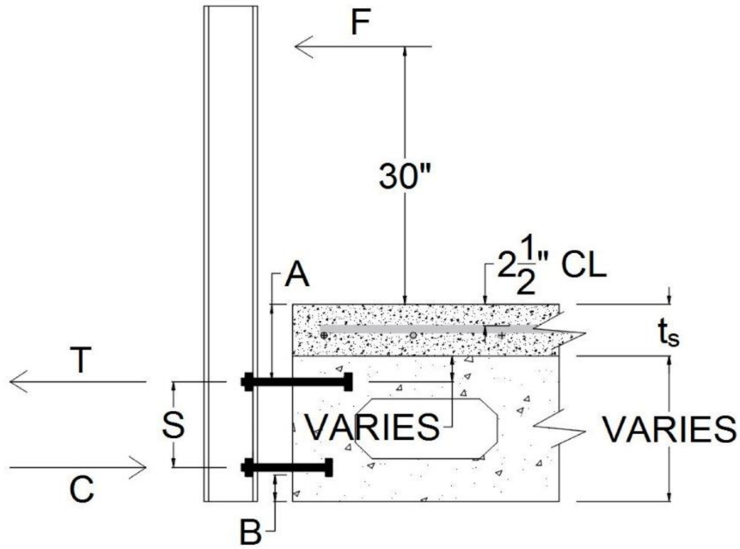
Designed by: Pascual Mauricio

Figure A-6. Slab and Box-Beam Girder Configuration, Preliminary Anchor Loadings

IL/OH TL-4 Steel Tube Bridge Rail

Lateral Load Resistance
Post-to-Deck Connections

Deck Box Beam Configuration - Concrete Slab Topping



Givens:

Barrier Rail Load Effective Height (H_e)	
30	in

Slab Reinforcement		
Lateral:		#4 (@ 9")
Longitudinal:		#4 (@ 18")
Clear Cover (in)	Top:	2.50
	Bot.:	1.5

Depth of Slab Deck, t_s (in):
Varies

Selecting Anchor Ø (in):
Varies

Box Beam Reinforcement	
Lateral:	#4
Longitudinal:	#4
Clear Cover (in)	1.50

Sizes of Deck Box Beams (in)	
Depth:	17, 21, 27, 33, 42
Widths:	36 or 48

Anchor Spacing, S (in)
Varies

Inputs:

Select Box Beam Depth:	17	in
Select Slab Thickness, t_s:	0	in
Select Clear Cover of Tension Anchor:	3	in
Select Anchor Ø:	1	in
Select Anchor Spacing, S:	11	in
Select Compression Block Cover, B:	1.5	in

Resulting Anchor Forces:

Tension Anchor Loading (T):	**94.14**	kips
Compression Anchor Loading (C):	**72.64**	kips
Moment Arm:	**33**	in
Applied Load (F):	**24.55**	kips

From Deck to Centerline of T anchors
A: **3** in

Between T and C anchors
S: **11** in

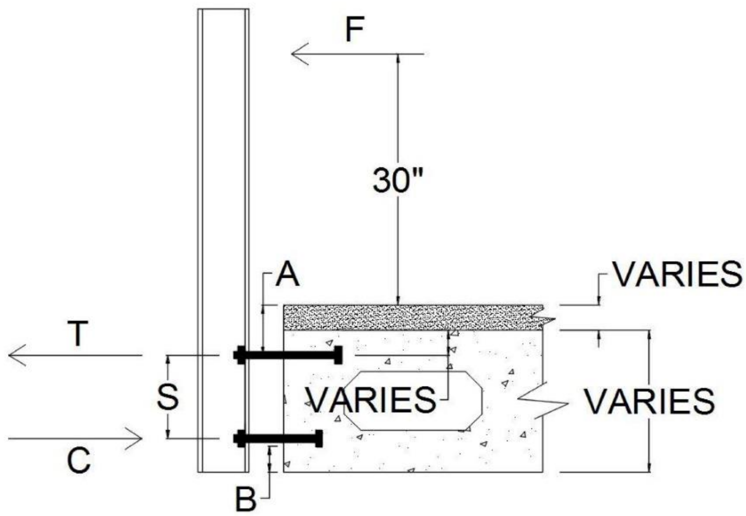
Designed by: Pascual Mauricio

Figure A-7. Concrete Box-Beam Girder with Concrete Top, Preliminary Anchor Loadings

IL/OH TL-4 Steel Tube Bridge Rail

Lateral Load Resistance
Post-to-Deck Connections

Deck Box Beam Configuration - Asphalt Topping



Givens:

Barrier Rail Load Effective Height (H_e)	
30	in

Depth of Slab Deck (in):
Varies

Selecting Anchor Ø (in):
Varies

Box Beam Reinforcement	
Lateral:	#4
Longitudinal:	#4
Clear Cover (in)	1.50

Sizes of Deck Box Beams (in)	
Depth:	17, 21, 27, 33, 42
Widths:	36 or 48

Anchor Spacing, S (in)
Varies

Inputs:

Select Box Beam Depth:	33	in
Select Slab Thickness, t_s:	0	in
Select Clear Cover of Tension Anchor:	3	in
Select Anchor Ø:	1	in
Select Anchor Spacing, S:	11	in
Select Compression Block Cover, B:	1.5	in

Resulting Anchor Forces:

Tension Anchor Loading (T):	**53.11**	kips
Compression Anchor Loading (C):	**28.93**	kips

Moment Arm:	**33.5**	in
Applied Load (F):	**24.18**	kips

From Deck to Centerline of T anchors
A: **3.5** in

Between T and C anchors
S: **28** in

Designed by: Pascual Mauricio

Figure A-8. Concrete Box-Beam Girder with Asphalt Top, Preliminary Anchor Loadings

IL/OH TL-4 Steel Tube Bridge Rail

Anchor Design
Post-to-Deck Connections

Calculations Overview

Design Description:

These anchor design calculations are based on the bridge deck configurations provided by the Illinois and Ohio Department of Transportations. They are used to provide preliminary anchorage designs for the side-mount post-to-deck connections of the MASH TL-4 steel tube bridge rail.

Selecting Criteria

	Select ASTM Desig.	Select Anchor Ø	
For	F1554 Gr. 105	1	"Ø

Analyzing for:

Slab Depth	12	in
Tension Anchor Loading	159.55	kips
Compression Anchor Loading	135.00	kips

Req.D # Anchors

$\O r_n =$ **55.2** kips/rod — Tensile Fastener Strength

Req. # of Tension Fasteners
=>>> #Anchors = 1.84902162

Use 2 - 1 "Ø F1554 Gr. 105 anchors

Req. # of Compression Fasteners
=>>> #Anchors = 1.5655675

Use 2 - 1 "Ø F1554 Gr. 105 anchors

w/o Ø $r_n =$ 73.6 kips/rod — #T-Anchors = 2.16682 — Need 3
(without LRFD phi factor) — #C-Anchors = 1.83346 — Need 2

Development Length

1 for Headed Deformed Bars in Tension — ACI 318-14 25.4.4.2 — 32.37 in

l_{dt}

$$\left(\frac{0.016F_y\varphi_e}{\sqrt{f_c'}}\right)d_b = \mathbf{32.37}\ \text{in}$$

$$8d_b = 8\ \text{in}$$

$$6" = 6\ \text{in}$$

with consideration for Slab Lateral Reinf

1	27.53 in
2	66.56 in
3	99.84 in

2 for Deformed Bars in Tension — ACI 318-14 25.4.2.2 — *78.26 in*

l_d

*****(#6 & smaller) --- (≤ 3/4"Ø)

$$\left(\frac{F_y\varphi_t\varphi_e}{25\sqrt{f_c'}}\right)d_b = \mathbf{62.61}\ \text{in}$$

*****(#7 & larger) --- (≥ 7/8"Ø)

$$\left(\frac{F_y\varphi_t\varphi_e}{20\sqrt{f_c'}}\right)d_b = \mathbf{78.26}\ \text{in}$$

Designed by: Pascual Mauricio

Figure A-9. Anchorage and Embedment Design, Sheet 1 of 2

IL/OH TL-4 Steel Tube Bridge Rail

Anchor Design
Post-to-Deck Connections

12"	=	12	in	12"	=	12	in

Note: The above two equations assume min. clear cover of db along with either min. clear spacing of 2db, or a combination of min. clear spacing of db and min. ties or stirrups.

$\boxed{3}$ for Deformed Bars in Tension (General Development Length Eq.) ACI 318-14 25.4.2.3a

l_d

$$\left(\frac{3}{40}\frac{F_y}{\lambda\sqrt{f_c'}}\frac{\varphi_t\varphi_e\varphi_s}{\frac{c_b+K_{tr}}{d_b}}\right)d_b = \mathbf{117.39}\ \textbf{in}$$

where C_b = 1
Min. spacing = 2

12" = 12 in

Tension Development Length using AASHTO AASHTO LRFD 5.11.2.1.1

C_{db}

$$\left(\frac{1.25A_b f_y}{\sqrt{f_c'}}\right) = \mathbf{48.59}\ \textbf{in}$$

without reinforcement magnifier (λ = 1.5)

Designed by: Pascual Mauricio

Figure A-10. Anchorage and Embedment Design, Sheet 2 of 2

IL/OH TL-4 Steel Tube Bridge Rail

Weld Connections
Post-to-Deck Connections

Calculations Overview

Design Description:

These weld connection designs are based on the bridge deck configurations provided by the Illinois and Ohio Department of Transportations. They are used to provide preliminary weld connections for the side-mount post-to-deck connections of the MASH TL-4 steel tube bridge rail.

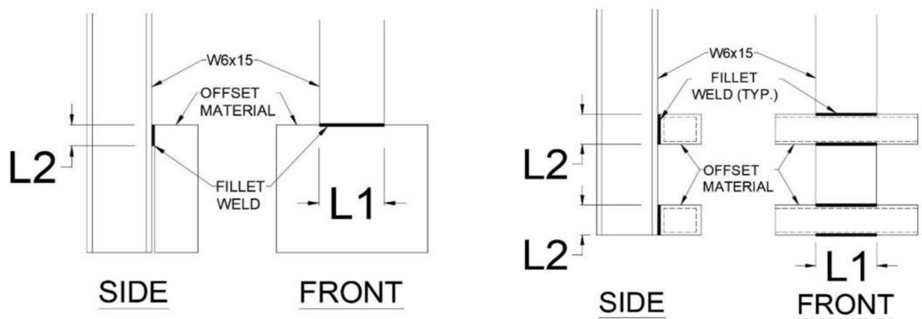
Design Criteria

Weld Max Size:	1/4	in		Slab Depth	17	in
Effective throat:	0.177	in	0.707*weld size	Tension Anchor Loading	96.37	kips
Use E70 ksi filler metal strength	Fexx =	70	ksi			

Analyzing for:

Post Assembly Weld Connection

Weld capacities shown here are for welding a W6x15 post to a plate attachment or HSS deck spacers.



from AISC Steel Manual, 14th ed. (J2-4)

where Ø = 0.75

θ (°) = 90

$F_{nw} = 0.60F_{exx}(1 + 0.5\sin^{1.5}(\theta))$ = 63 ksi

#Welds = 1 | 2

Weld Length, (in) L1: 6 L2: 4

Awe, (in^2) = Eff. throat * weld length

Weld Strength: ØRn = ØFnwAwe

≥

Top W Strength	ØRn =	50.11	kips		Rn =	66.81
Side W Strength	ØRn =	66.81	kips		Rn =	89.08
Total W Strength	ØRn =	**116.92**	**kips**	≥	96.37	kips
	Rn =	155.89				

Welding Post to Offset is Sufficient

Designed by: Pascual Mauricio

Figure A-11. Weld Connection Design, Sheet 1 of 2

IL/OH TL-4 Steel Tube Bridge Rail

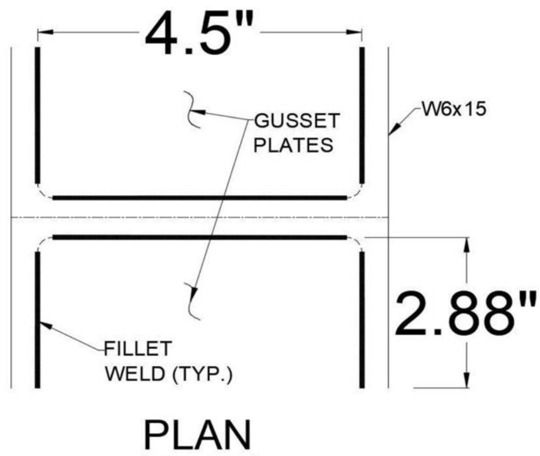
Weld Connections
Post-to-Deck Connections

Welding Gusset Plates to Post

Weld capacities shown here are for welding gusset plates to a W6x15 to meet post plastic bending capacity. Gusset plates are welded to the interior post flanges and the post web. Dimensions shown are approximate inner dimensions of the post. Assume loading is applied perpendicularly to the post flange.

from AISC Steel Manual, 14th ed. (J2-4)

where	Ø =	0.75		
	θ (°) =	0	web	
		90	flanges	
	#Welds =	1		2
Weld Length	Flange:	2.88		
(in)	Web:	4.5		



Weld Strength: ØRn = ØFnwAwe

W Str on	Fnw =	63			
Flanges	ØRn =	48.10	kips		
				Rn =	64.14
W Str on	Fnw =	42.00			
Web	ØRn =	50.11	kips		
				Rn =	66.81
Total W	ØRn =	**98.21**	**kips**		
Strength				Rn =	130.95

Total Weld Strength of Post Assembly + Gusset Plates

	ØRn =	116.92	+	98.21			
Total W	ØRn =	**215.13**	**kips**	≥	**96.37**	kips	Welding Post to Offset is Sufficient
Strength	Rn =	286.84					

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Figure A-12. Weld Connection Design, Sheet 2 of 2

Appendix B. Illini Concrete Box-Beam Girder Drawings

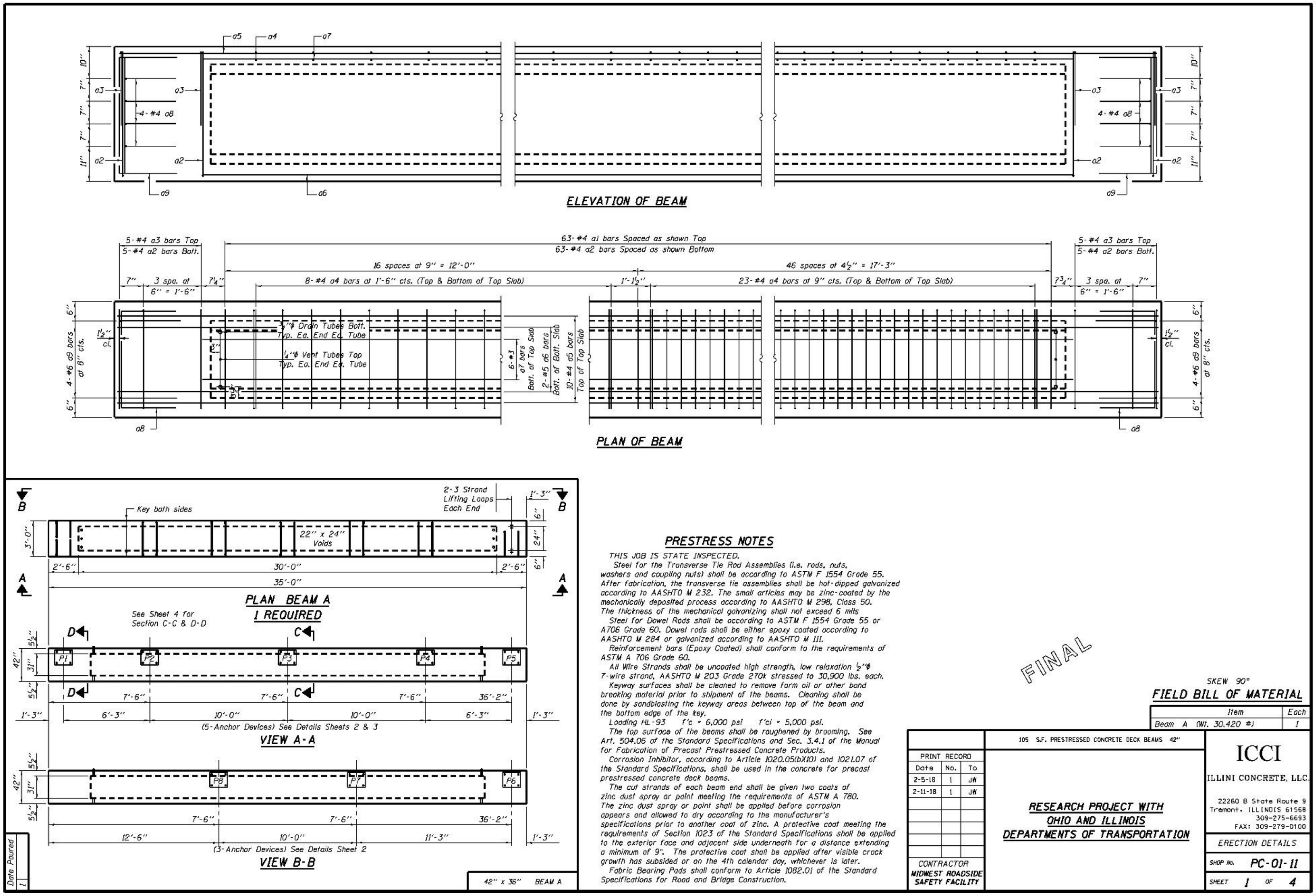


Figure B-1. Illini Concrete Box-Beam Girder Details, Sheet 1 of 4

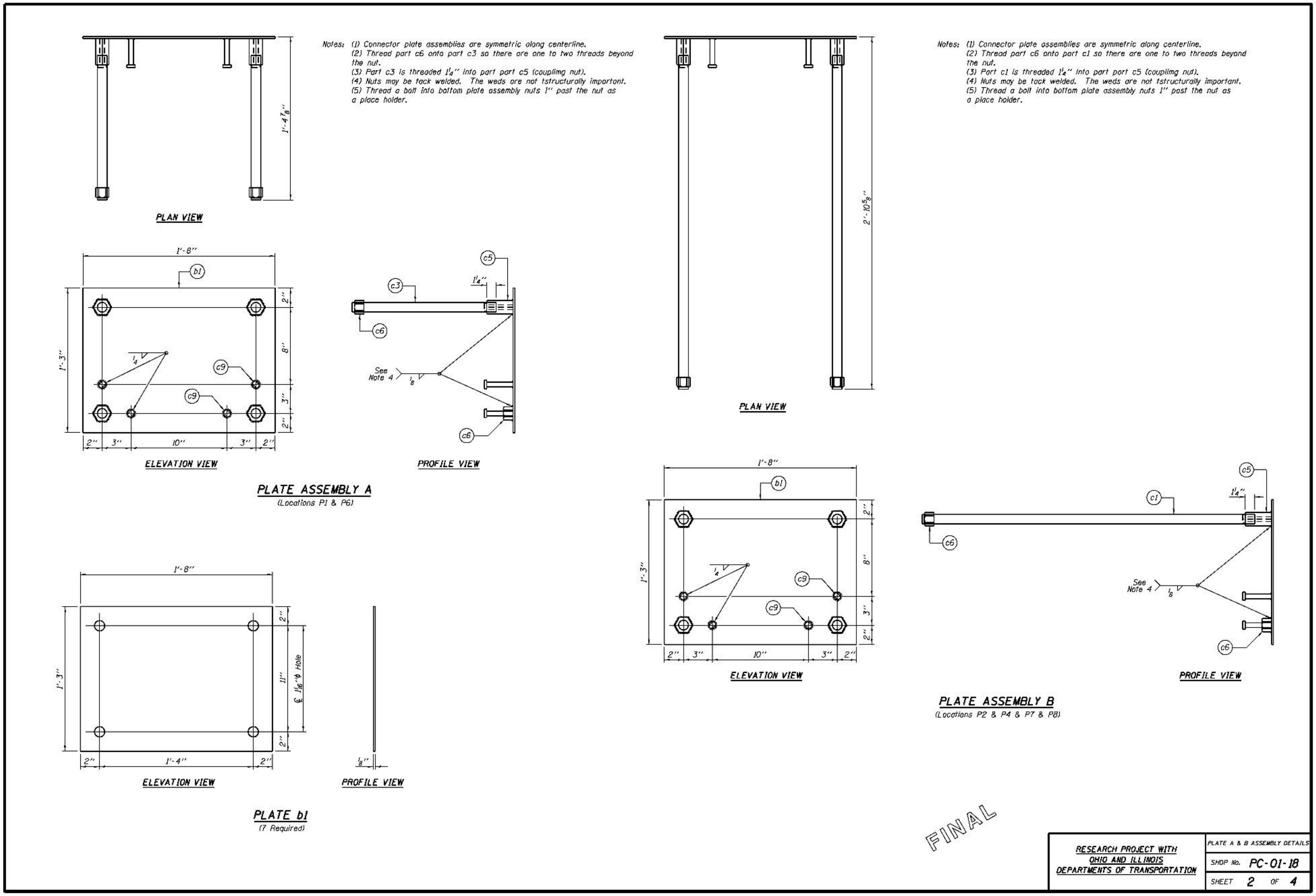


Figure B-2. Illini Concrete Box-Beam Girder Details, Sheet 2 of 4

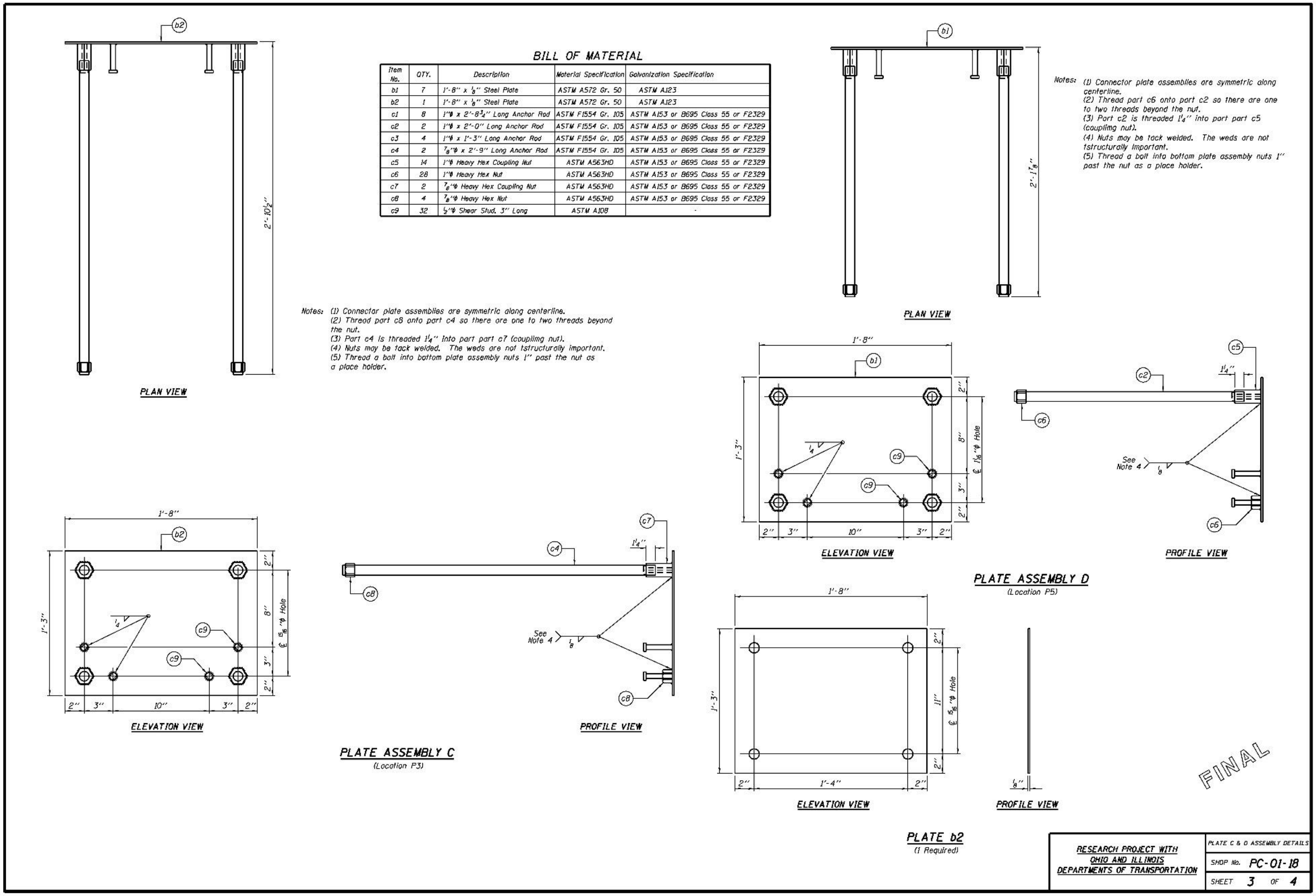


Figure B-3. Illini Concrete Box-Beam Girder Details, Sheet 3 of 4

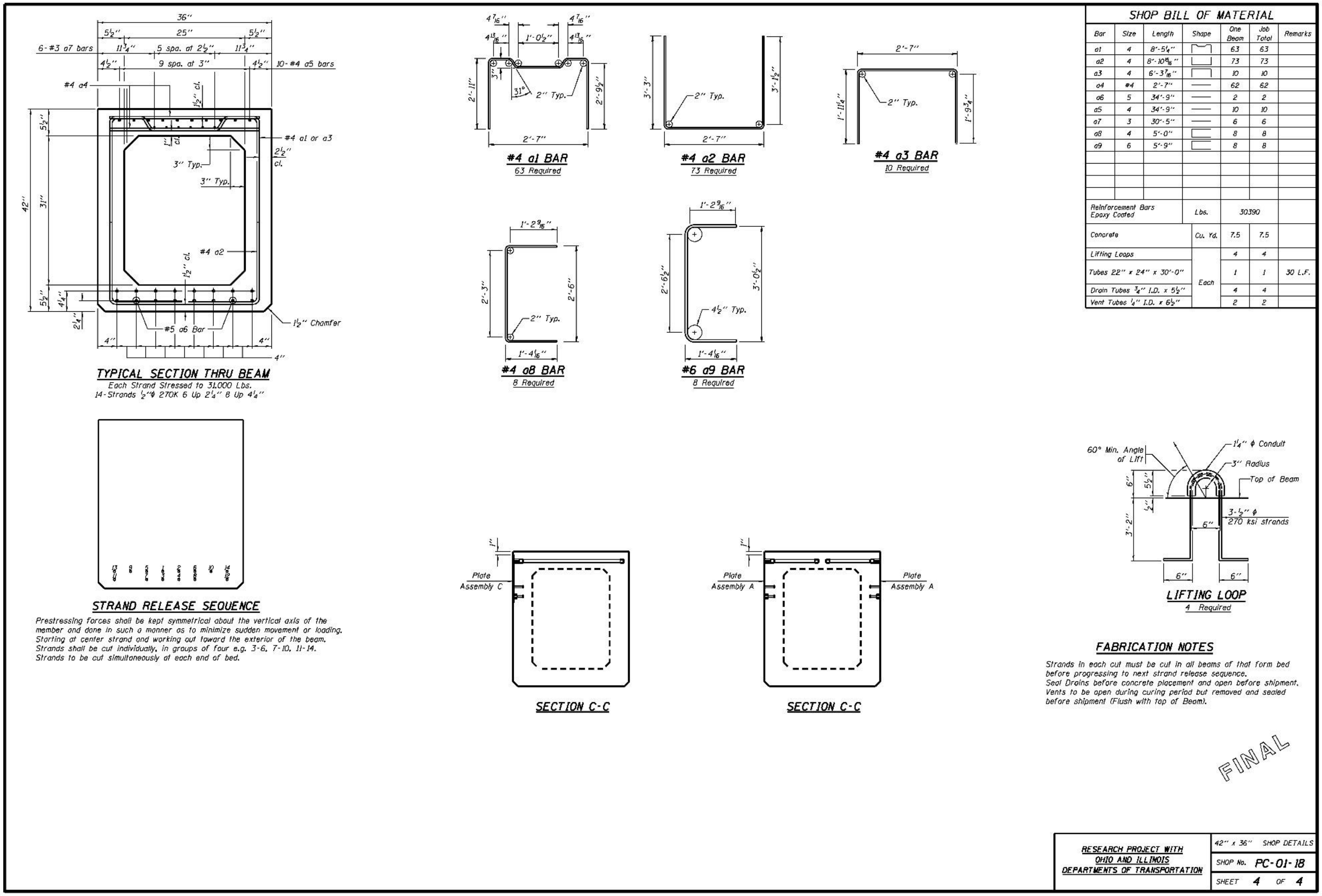


Figure B-4. Illini Concrete Box-Beam Girder Details, Sheet 4 of 4

Appendix C. Bogie Calculations

Preliminary anchor loads were calculated with a design force load impacting a MASH 2016 TL-4 system at an effective height of 30 in. (762 mm) [6]. However, the effective height of the bogie vehicle was altered for several reasons. In the proposed MASH 2016 TL-4 steel-tube bridge rail, the tube rail configurations have the center of the bridge railing system being at 28 in. (711 mm) from the top of the bridge deck, which is the center of the middle rail. It was assumed that impact loads will be distributed to all three tube rails, therefore a 28-in. (711-mm) height would impact the center of the post in relation to the vertical positioning of the tube railings in the bridge rail system. Concurrently, the center of gravity (CG) of the 2270 pickup truck is also located 28 in. (711 mm) from the ground line. At this corresponding height, it was believed that the W6x15 post will fully develop its plastic bending moment capacity, and the anchor loadings will experience maximum loading transferred into the bridge deck. The bridge railing configuration with the 28-in. (711-mm) impact height is shown in Figure C-1.

Furthermore, the bogie head height was based on 3-in. (76-mm) vertical intervals with a nominal starting height of 25 in. (635 mm). Such vertical height movements constrained the ability to impact the steel post at a higher impact height. It shall be noted that a lower 28-in. (711-mm) impact height would transfer higher loading into the bridge deck as opposed to the 30-in. (762-mm) effective height, as shown in Table C-1. Therefore, with the center of the bridge railing system being the center of the middle rail at 28 in. (711 mm), and also, subsequently, the CG of the pickup truck established at 28 in. (711 mm), which was very close in height to the 30-in. (762-mm) effective height established from standard guidance, the impact height of the bogie vehicle was selected to be at 28 in. (711 mm).

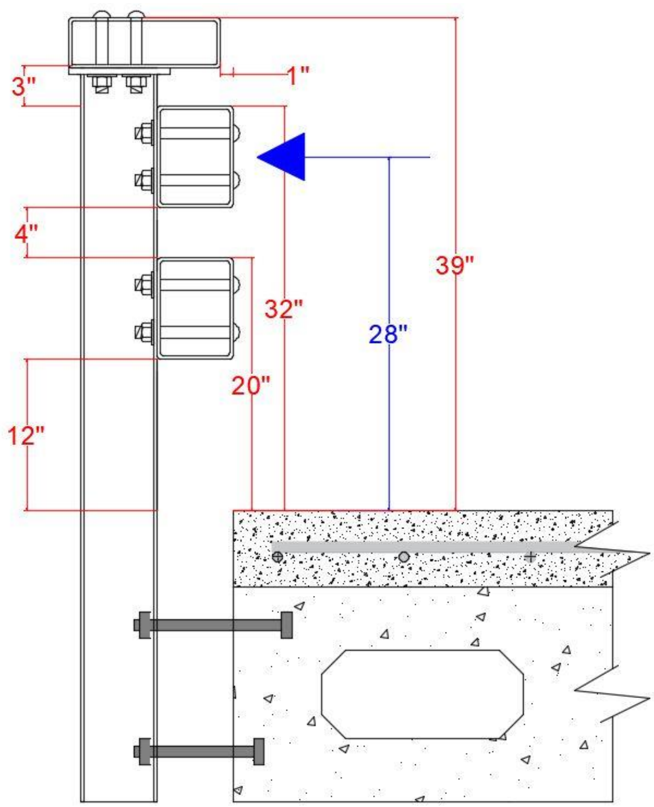


Figure C-1. Bridge Rail Configuration with 28-in. (711-mm) Impact Height

Table C-1. Force Comparison due to Impact Height

Impact Configuration	Impact Height, in. (mm)	Post Plastic Bending Capacity, kip-in.	Concrete Cover, in. (mm)	Force, Kip (kN)
Effective Height	30 (762)	810	3 (76)	24.5 (109)
Center of Middle Rail / C.G. of Pickup Truck	28 (711)			26.1 (116)

Initial impact forces were estimated on the W6x15 post as the bogie vehicle would impact the post's strong-axis at an impact height of 28 in. (711 mm) from the top of the concrete box-beam girder with a 3-in. (76-mm) anchor rod concrete cover. The total 31-in. (787-mm) height from the point of impact of the bogie head impacting the post to the location of the tensile anchor rods was initially expected to encompass the entire moment arm induced on the post, and a plastic hinge was expected to develop at or near the anchor rods, as designed, as shown in Figure C-2.

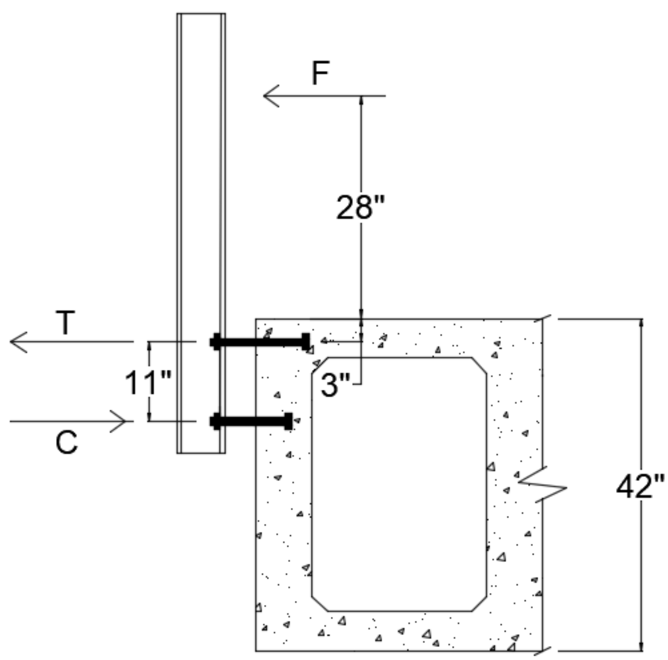


Figure C-2. Impact Height for Bogie Tests

The plastic bending capacity of the W6x15 post was calculated to determine the force the post could resist before plastically deforming. A reduction factor was not used in order to determine the post strength to its truest capacity. With impact loadings based on the plastic bending of the steel post, the plastic bending capacity was determined by Equation C.2.

$$M_u = \mathrm{DMF} * F_y * Z_x \tag{C.2}$$

Where

$M_u =$ Plastic bending capacity (kip − in.)
$DMF =$ Dynamic magnification factor of 1.5
$F_y =$ Yield stress of Steel Post, 50 ksi
$Z_x =$ Post plastic section modulus(in.^{3}), 10.8 in^3

The plastic bending capacity of the W6x15 post was 810 kip-in. With a moment induced at the location of the anchor rods by the impact force applied at the total 31-in. (787-mm) impact height, the side-mounted posts were initially estimated to resist a force of approximately 26.13 kips (116.23 kN) as determined by Equation C.3.

$$\mathrm{F} = {}^{M_u}\!/_{d} \tag{C.3}$$

Where

$F =$ Post designed resistive force, 26.13 kips (116.23 kN)
$M_u =$ Plastic bending capacity, 810 kip − in. (91.5 kN − m)
$d =$ Total impact height, 31 in. (787 mm)

To determine the bogie mass and velocity, preliminary estimates were obtained from determining the resistive force the W6x15 post can sustain as the post is displaced during impact. Previous bogie tests done by MwRSF under similar test conditions were analyzed, and it was initially assumed that the W6x15 post would resists 26 kips (116 kN) over a 15-in. (381-mm) deflection. The bogie mass was assumed to determine the velocity of the bogie required to fully develop the post near the surface of the deck. The bogie velocity was determined by Equation C.4.

$$\frac{1}{2}mv^2 = \mathrm{E} \tag{C.4}$$

Where

$m =$ Bogie mass, 2000 lbs (907 kg)
$v =$ Bogie velocity, mph $(\frac{\mathrm{km}}{\mathrm{h}})$
$E =$ Energy required to fully develop post, 392 in − k (45 kN − mm)

A bogie velocity of 20 mph (32 km/h) was determined necessary to fully develop the post and create a plastic hinge near the surface of the deck with the 28-in. (711-mm) impact height and 3-in. (76-mm) anchor rod concrete cover. After observing in the test nos. ILOH4-1 and ILOH4-2 the bogie head traveling up the post after impact, the bogie mass was increased with the additional weight placed near the bogie head at the front of the vehicle. This was done to prevent early bogie head override of the post which would increase the 31-in. (797-mm) moment arm and transfer less critical forces into the deck.

True post resistive forces were calculated for posts that developed a plastic hinge, as designed. From test nos. ILOH4-4 through ILOH4-7, the posts plastically deformed right above the top tensile gussets at an impact height of approximately 27⅜ in. (695 mm), as shown in Figure C-3, rather than the 31-in. (787-mm) impact height to the location of the anchor rods. Furthermore, interest was placed on the post's lateral deflection. A 12-in. (305-mm) lateral deflection was determined to be acceptable for the bridge rail due to two circumstances: a drop in post resistance was seen in the force-deflection plots from the bogie tests at 12 in. (305 mm), and a literature review of previously tested post-and-tube bridge rails often observed 12 in. (305 mm) of deflection during full-scale crash tests. Therefore, a maximum deflection of 12 in. (305 mm) was determined for the W6x15 posts.

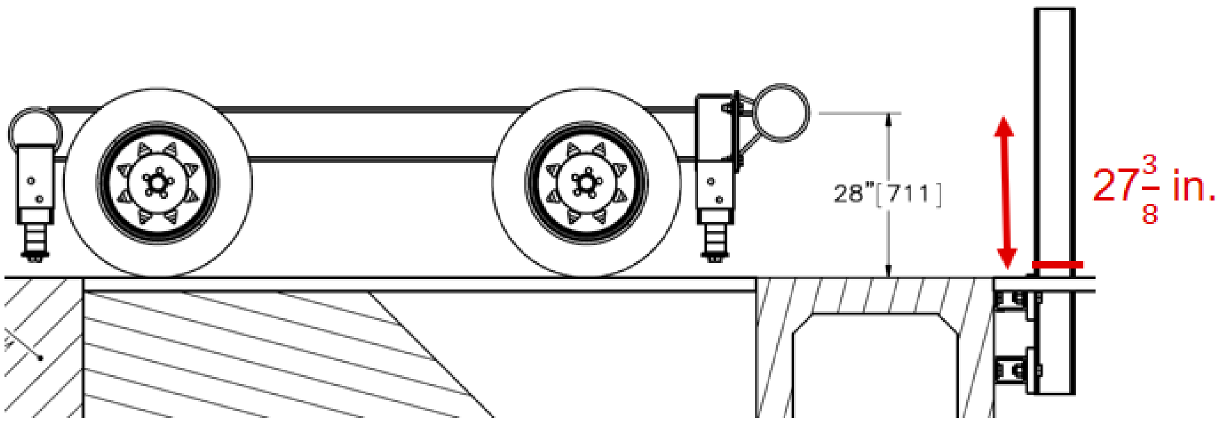


Figure C-3. Location of Plastic Hinge from Component Tests

Post lateral resistive forces were then calculated for the bogie tests that showed posts developing a plastic hinge above the tensile gussets, as shown in Table C-2.

Table C-2. Post Lateral Resistive Forces from Bogie Tests

Test No.	**Failure**	**Average Force, kips (kN)**		
		@ 5”	**@ 10”**	**@ 12”**
ILOH4-4	Flange & Web Tear	20 (89)	21 (93)	21 (93)
ILOH4-5	Post Hinge	21 (93)	22 (98)	20 (89)
ILOH4-6	Post Hinge	20 (89)	19 (85)	19 (85)
ILOH4-7	Post Hinge	18 (80)	20 (89)	19 (85)

Appendix D. Bogie Test Results

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

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachement to concrete box beam
Test Number:	ILOH4-1
Test Date:	6/7/2018
Failure Type:	Plate and weld tension failure

Post Properties

Post Type:	Steel post with welded plate
Post Size:	W6x15
Post Length:	58-1/4"
Embedment Depth:	NA
Orientation:	Strong

Soil Properties

Gradation:	NA
Moisture Content:	NA
Compaction Method:	NA

Bogie Properties

Impact Velocity:	22.53 mph	(33.04 ft/s)
Impact Height:	28"	
Bogie Mass:	1786 lb	

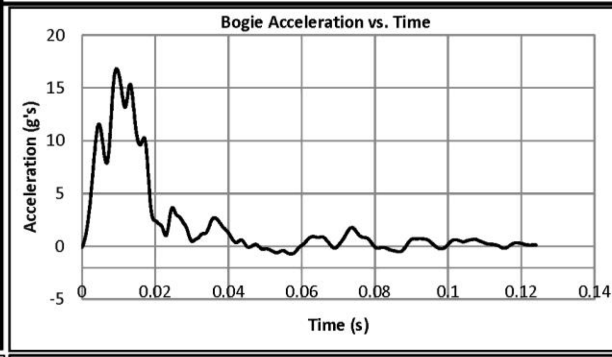
Data Acquired

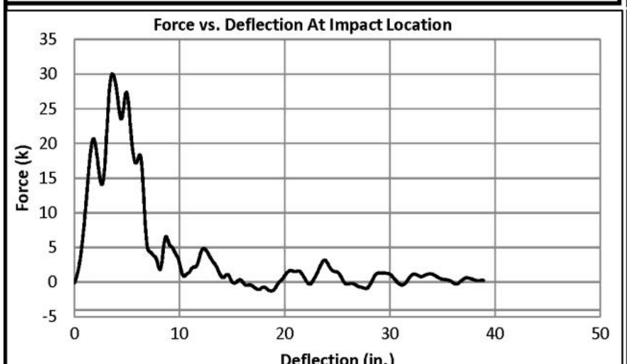
Accelerometer:	SLICE1
Camera Data:	aos8, gp14, gp15, gp16

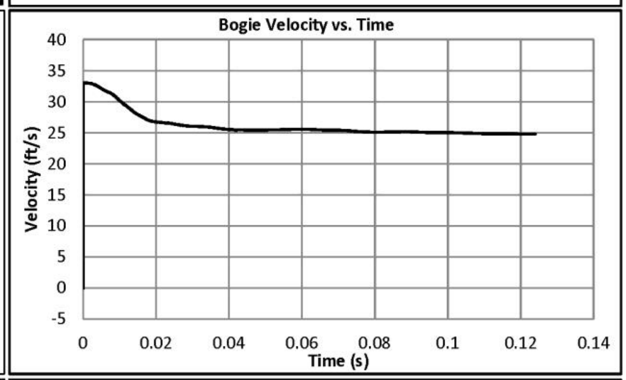
Test Results Summary

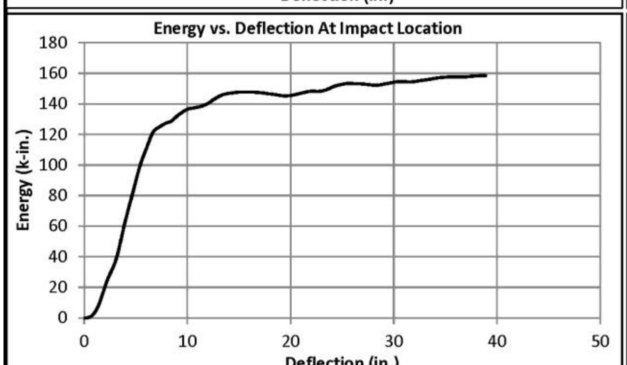
Event Duration:	0.1240	sec
Max. Deflection:	38.9	in.
Peak Force:	30.1	k
Initial Linear Stiffness:	8.3	k/in.
Total Energy:	158.6	k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	17.77	13.65	9.85	7.28
Energy (k-in.)	88.9	136.5	147.7	145.5









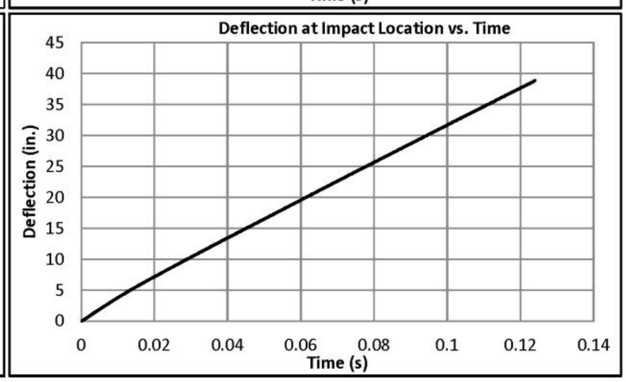


Figure D-1. Test No. ILOH4-1 Results (SLICE-1)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachement to concrete box beam
Test Number:	ILOH4-1
Test Date:	6/7/2018
Failure Type:	Plate and weld tension failure

Post Properties

Post Type:	Steel post with welded plate
Post Size:	W6x15
Post Length:	58-1/4"
Embedment Depth:	NA
Orientation:	Strong

Soil Properties

Gradation:	NA
Moisture Content:	NA
Compaction Method:	NA

Bogie Properties

Impact Velocity:	22.53 mph	(33.04 ft/s)
Impact Height:	28"	
Bogie Mass:	1786 lb	

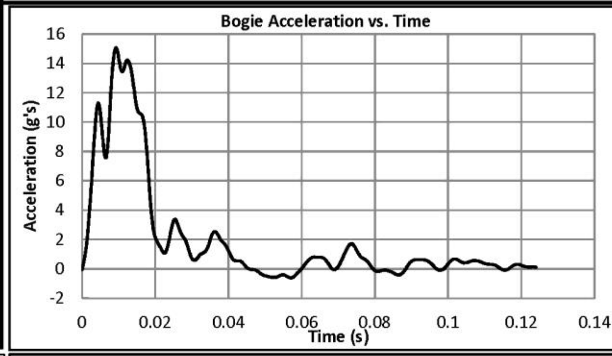
Data Acquired

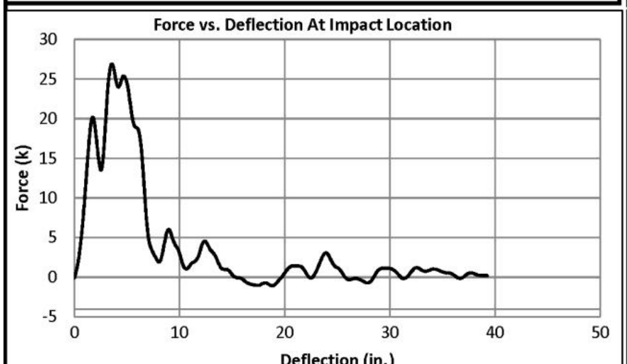
Accelerometer:	SLICE2
Camera Data:	aos8, gp14, gp15, gp16

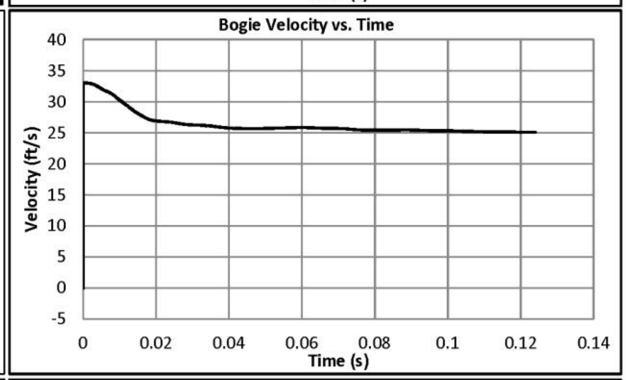
Test Results Summary

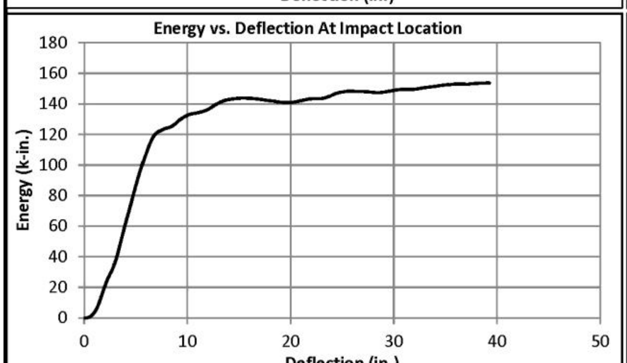
Event Duration:	0.1240	sec
Max. Deflection:	39.2	in.
Peak Force:	26.9	k
Initial Linear Stiffness:	7.5	k/in.
Total Energy:	153.8	k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	17.21	13.26	9.58	7.05
Energy (k-in.)	86.1	132.6	143.8	141.0









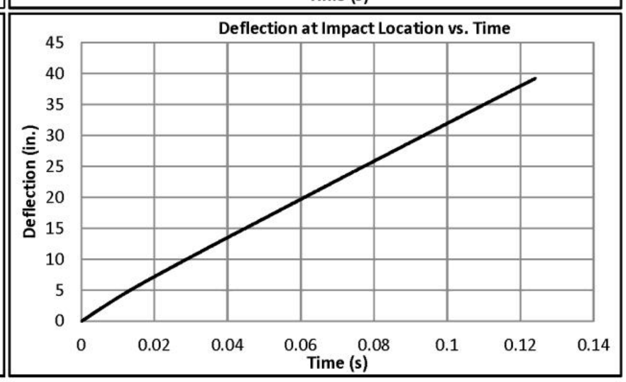


Figure D-2. Test No. ILOH4-1 Results (SLICE-2)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH-2
Test Date:	6/22/2018
Failure Type:	Post yeilding between plates

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	21.83 mph	(32.02 ft/s)
Impact Height:	28"	
Bogie Mass:	1786 lb	

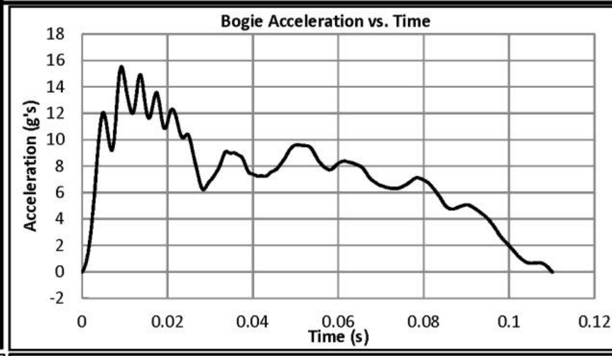
Data Acquired

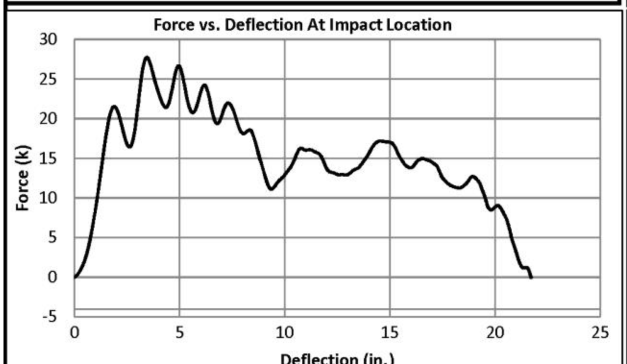
Accelerometer:	SLICE1
Camera Data:	aos9, gp13, gp15, gp16

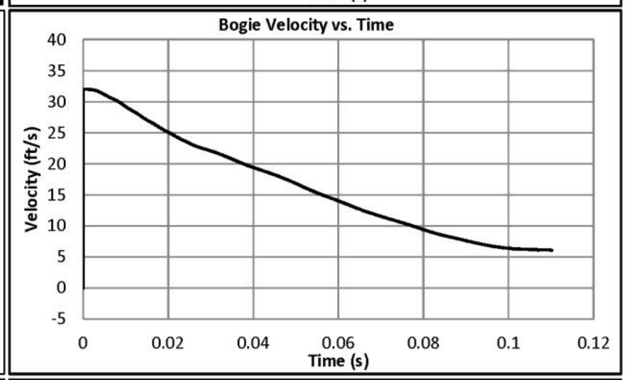
Test Results Summary

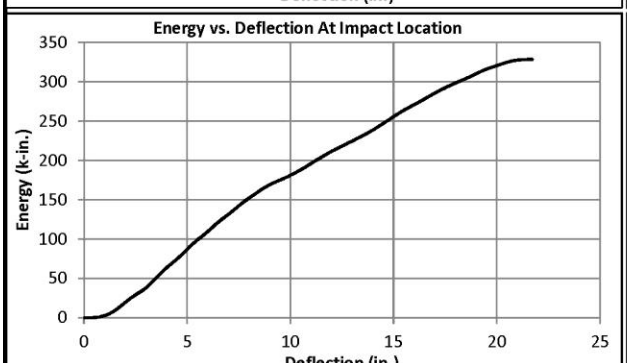
Event Duration:	0.1101 sec
Max. Deflection:	21.7 in.
Peak Force:	27.8 k
Initial Linear Stiffness:	8.1 k/in.
Total Energy:	328.8 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	17.35	18.10	17.03	16.03
Energy (k-in.)	86.7	181.0	255.5	320.7









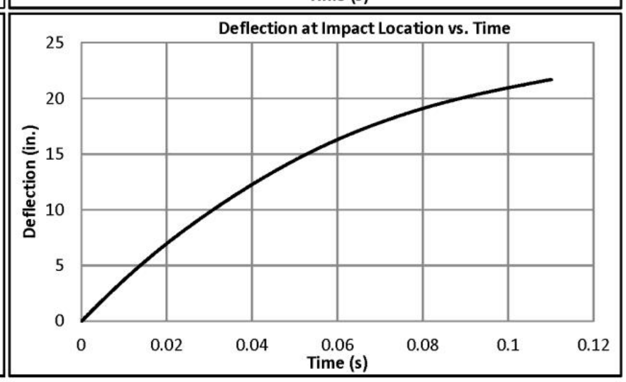


Figure D-3. Test No. ILOH4-2 Results (SLICE-1)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH-2
Test Date:	6/22/2018
Failure Type:	Post yielding between plates

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	21.83 mph	(32.02 ft/s)
Impact Height:	28"	
Bogie Mass:	1786 lb	

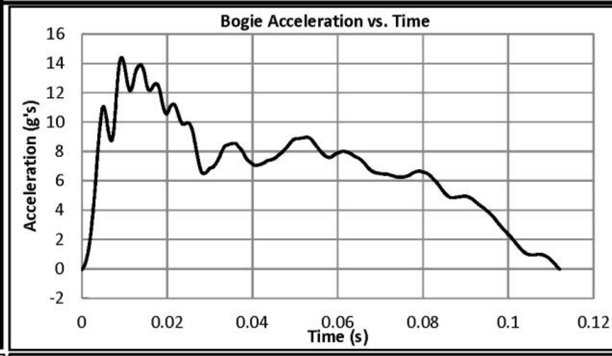
Data Acquired

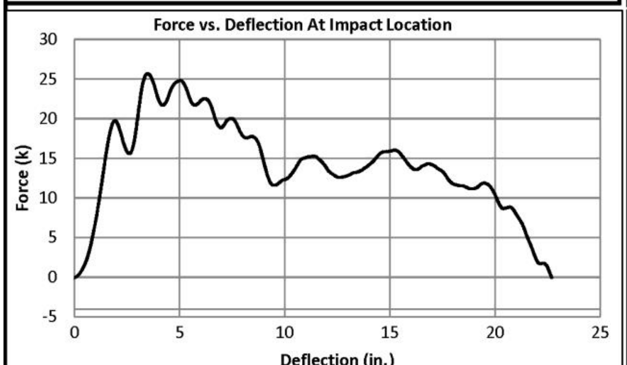
Accelerometer:	SLICE2
Camera Data:	aos9, gp13, gp15, gp16

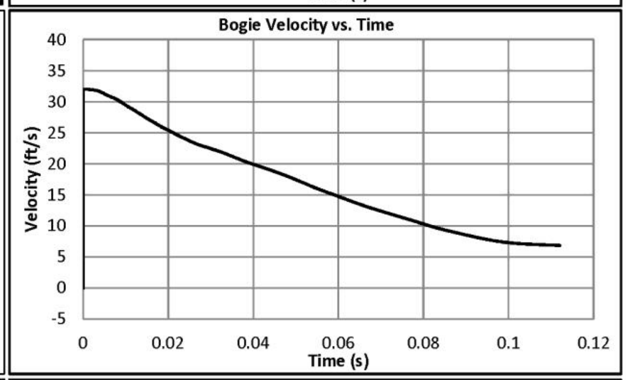
Test Results Summary

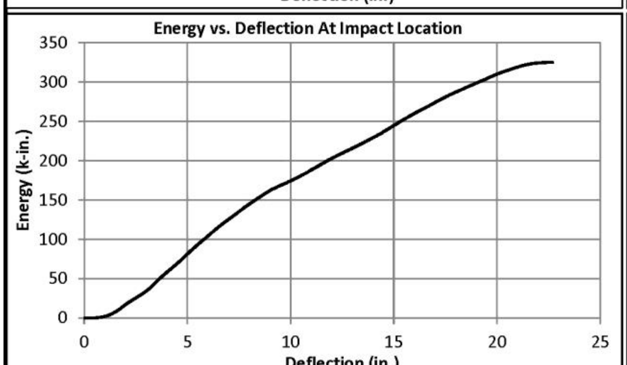
Event Duration:	0.1120 sec
Max. Deflection:	22.7 in.
Peak Force:	25.7 k
Initial Linear Stiffness:	7.4 k/in.
Total Energy:	325.4 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	16.19	17.44	16.31	15.51
Energy (k-in.)	81.0	174.4	244.7	310.2









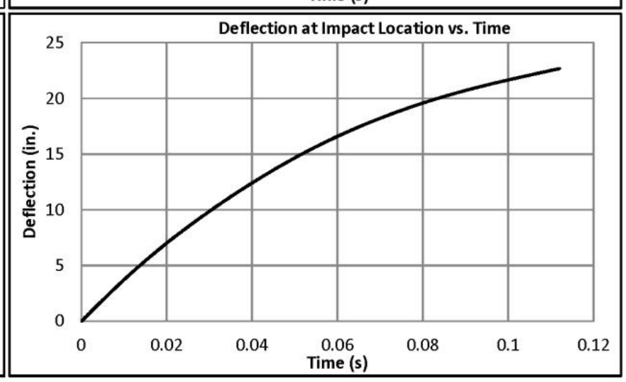


Figure D-4. Test No. ILOH4-2 Results (SLICE-2)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH4-3
Test Date:	7/20/2018
Failure Type:	Weld failure

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	21.83 mph	(32.02 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

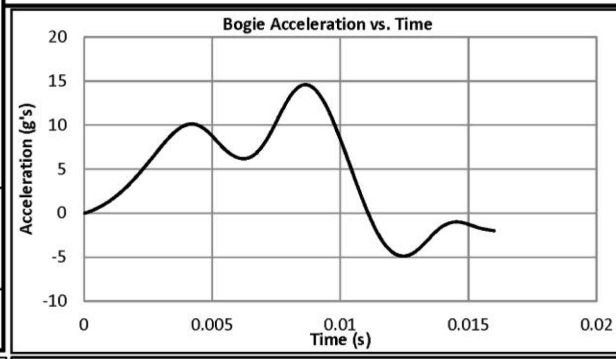
Data Acquired

Accelerometer:	SLICE1
Camera Data:	aos8, gp13, gp14, gp17

Test Results Summary

Event Duration:	0.0160 sec
Max. Deflection:	5.9 in.
Peak Force:	36.9 k
Initial Linear Stiffness:	11.4 k/in.
Total Energy:	76.8 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	13.80	NA	NA	NA
Energy (k-in.)	69.0	NA	NA	NA



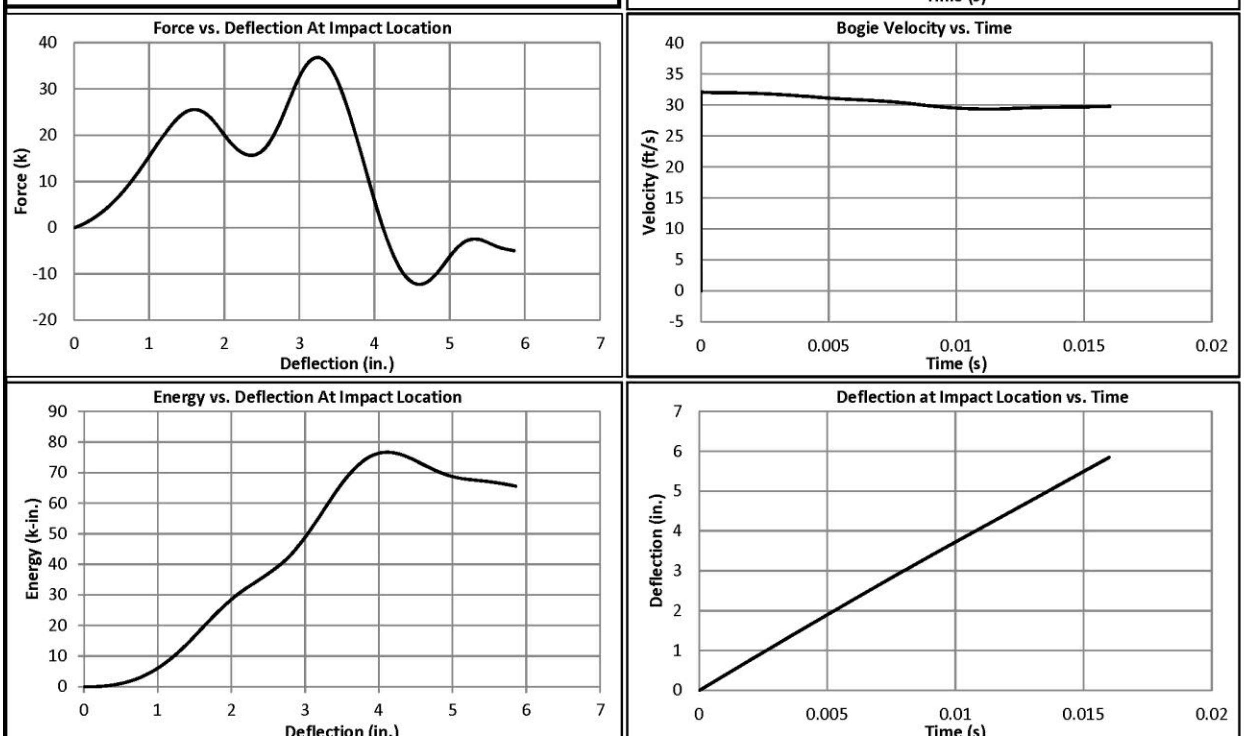


Figure D-5. Test No. ILOH4-3 Results (SLICE-1)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH4-3
Test Date:	7/20/2018
Failure Type:	Weld failure

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	21.83 mph	(32.02 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

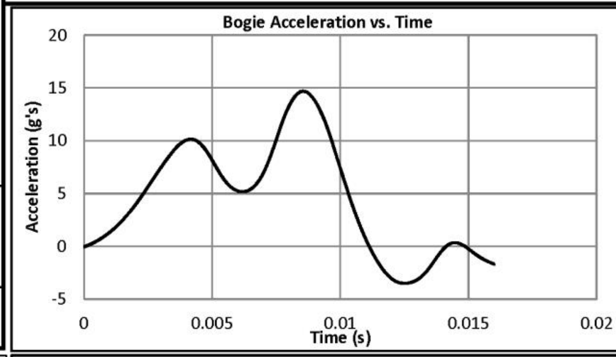
Data Acquired

Accelerometer:	SLICE2
Camera Data:	aos8, gp13, gp14, gp17

Test Results Summary

Event Duration:	0.0160 sec
Max. Deflection:	5.9 in.
Peak Force:	37.1 k
Initial Linear Stiffness:	11.5 k/in.
Total Energy:	73.3 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	13.57	NA	NA	NA
Energy (k-in.)	67.8	NA	NA	NA



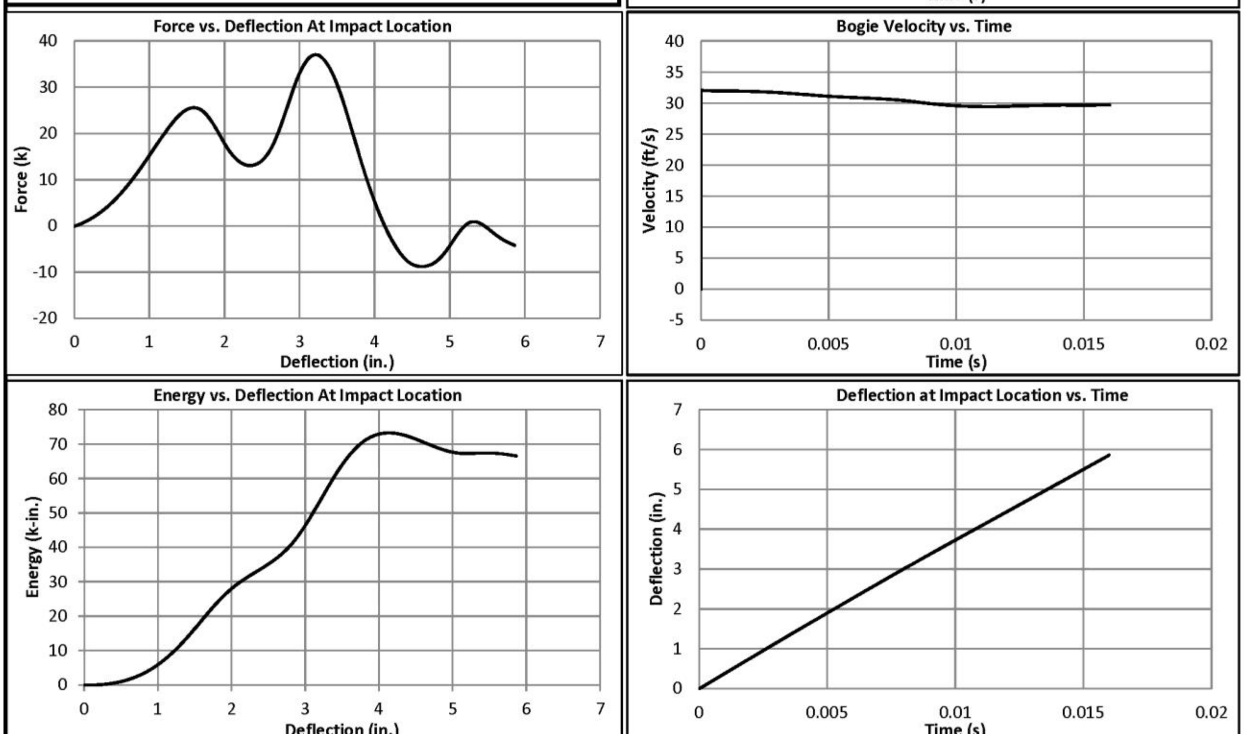


Figure D-6. Test No. ILOH4-3 Results (SLICE-2)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	iloh4-4
Test Date:	7/30/2018
Failure Type:	

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	21.43 mph	(31.42 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

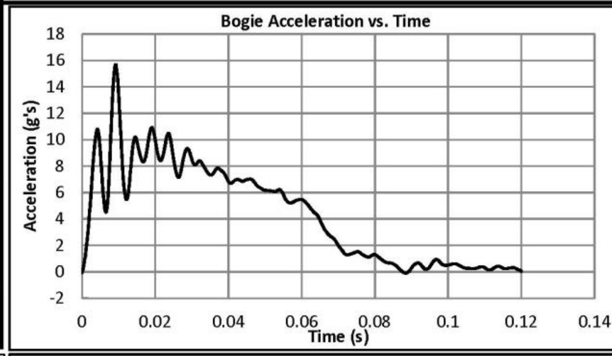
Data Acquired

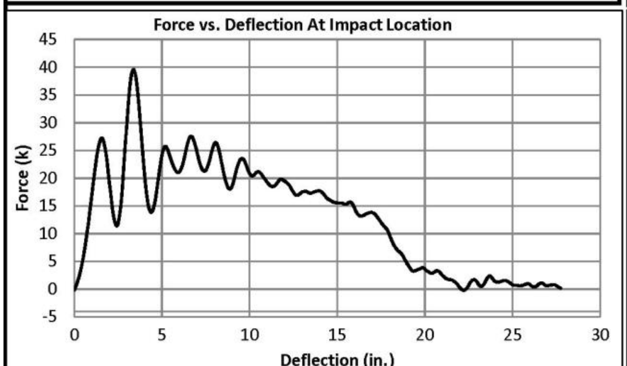
Accelerometer:	SLICE1
Camera Data:	AOS9, gp9, gp10, gp17, soloshot

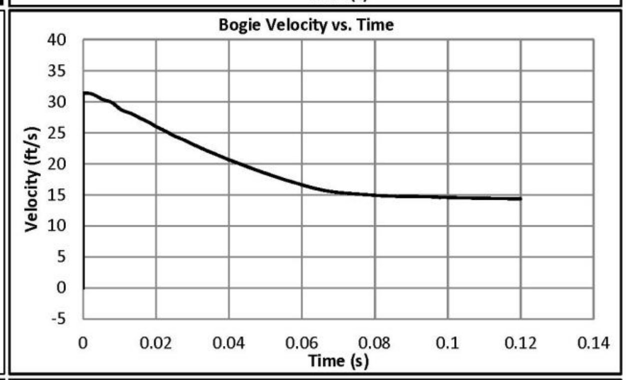
Test Results Summary

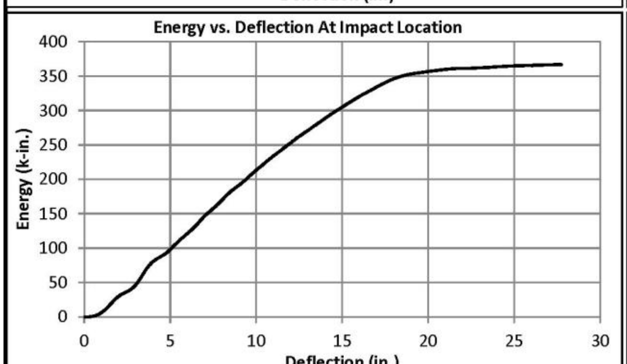
Event Duration:	0.1200 sec
Max. Deflection:	27.7 in.
Peak Force:	39.6 k
Initial Linear Stiffness:	11.7 k/in.
Total Energy:	367.1 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	19.56	21.35	20.35	17.84
Energy (k-in.)	97.8	213.5	305.2	356.9









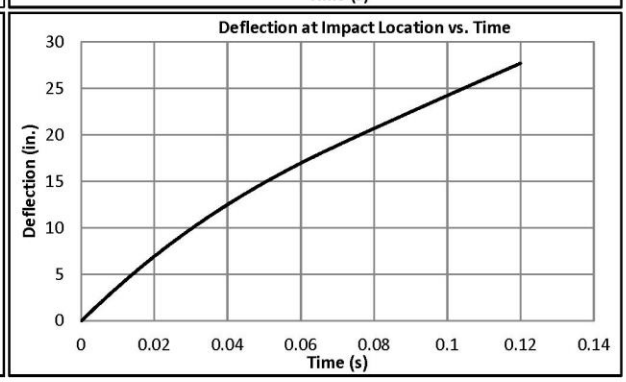


Figure D-7. Test No. ILOH4-4 Results (SLICE-1)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	iloh4-4
Test Date:	7/30/2018
Failure Type:	

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	21.43 mph	(31.42 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

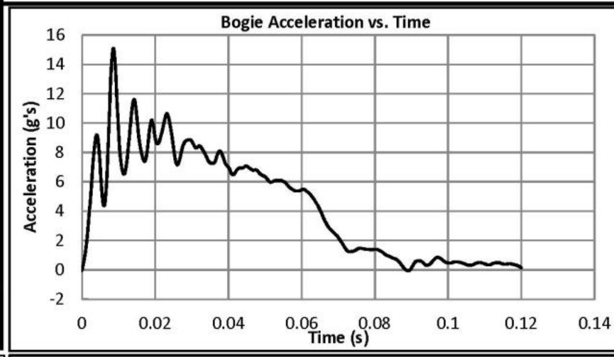
Data Acquired

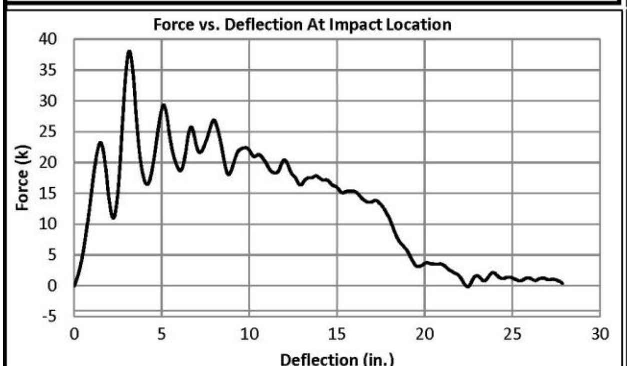
Accelerometer:	SLICE2
Camera Data:	AOS9, gp9, gp10, gp17, soloshot

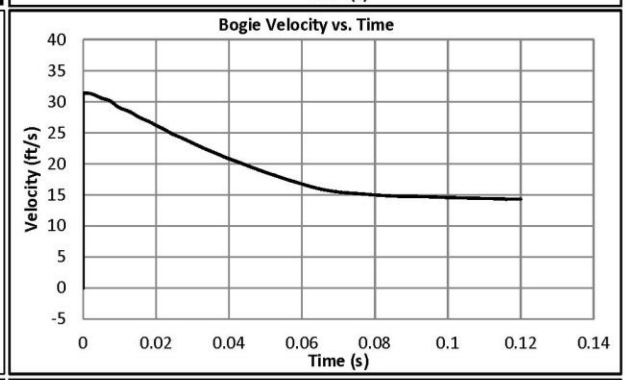
Test Results Summary

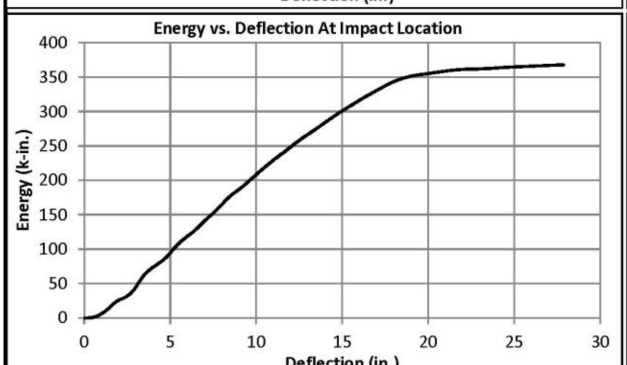
Event Duration:	0.1200	sec
Max. Deflection:	27.9	in.
Peak Force:	38.1	k
Initial Linear Stiffness:	12.0	k/in.
Total Energy:	368.0	k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	18.78	20.83	20.07	17.77
Energy (k-in.)	93.9	208.3	301.1	355.3









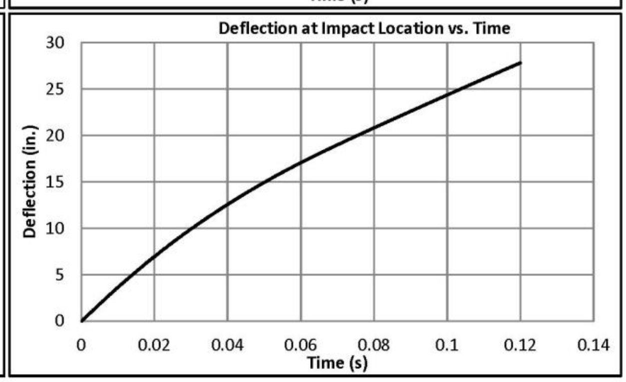


Figure D-8. Test No. ILOH4-4 Results (SLICE-2)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH4-5
Test Date:	8/1/2018
Failure Type:	Post yielding

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	20.66 mph	(30.31 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

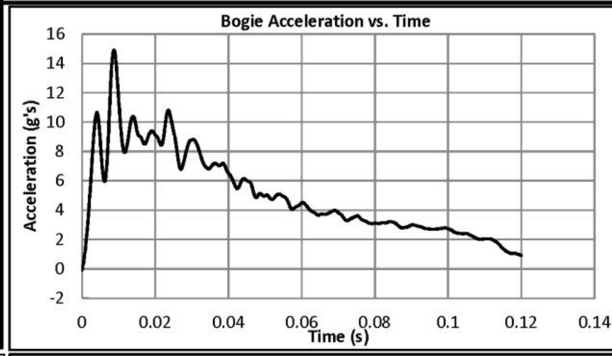
Data Acquired

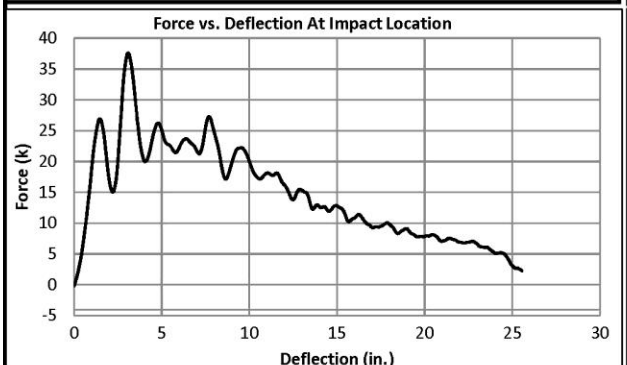
Accelerometer:	SLICE1
Camera Data:	aos9, pan1, pan2

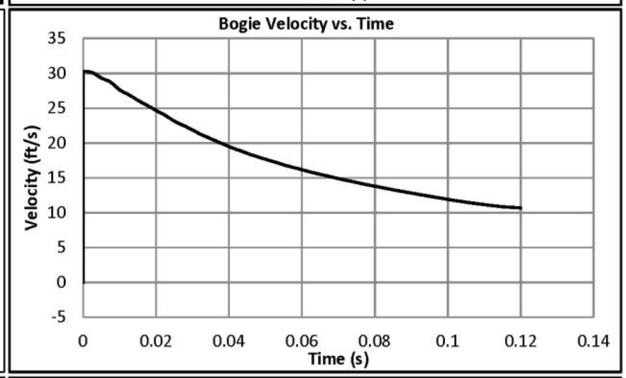
Test Results Summary

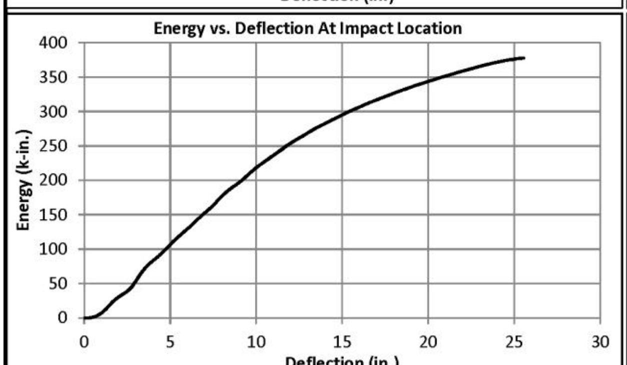
Event Duration:	0.1200 sec
Max. Deflection:	25.5 in.
Peak Force:	37.6 k
Initial Linear Stiffness:	12.2 k/in.
Total Energy:	377.9 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	21.27	21.84	19.67	17.19
Energy (k-in.)	106.3	218.4	295.1	343.8









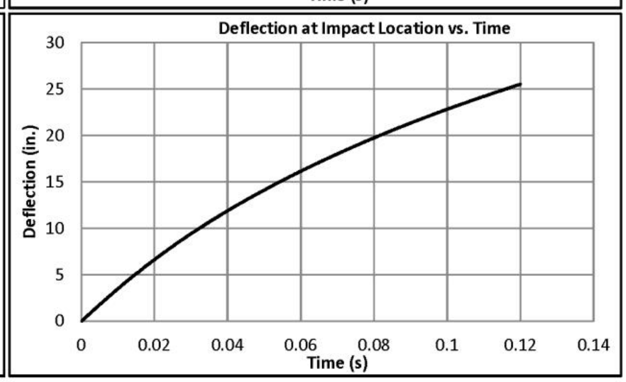


Figure D-9. Test No. ILOH4-5 Results (SLICE-1)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH4-5
Test Date:	8/1/2018
Failure Type:	Post yield

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	20.66 mph	(30.31 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

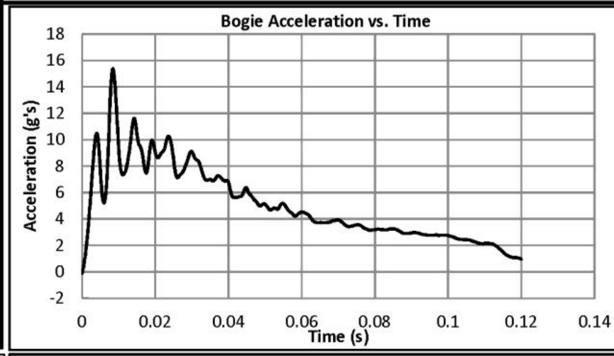
Data Acquired

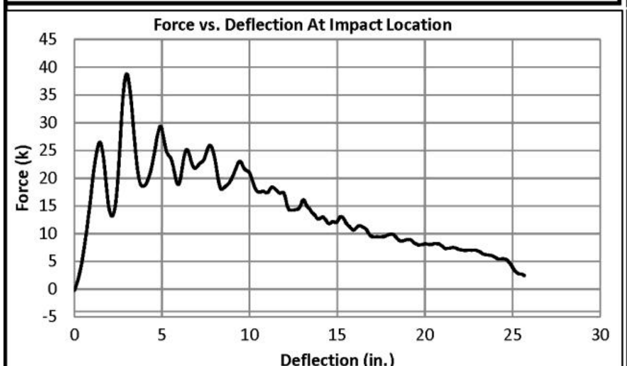
Accelerometer:	SLICE2
Camera Data:	aos9, pan1, pan2

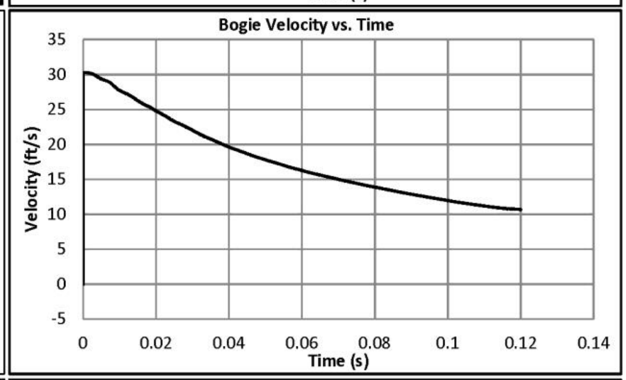
Test Results Summary

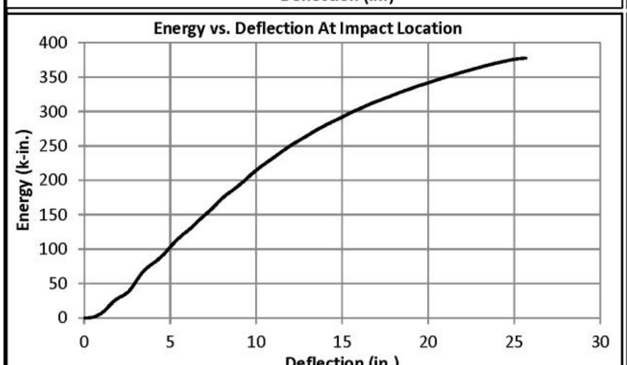
Event Duration:	0.1200 sec
Max. Deflection:	25.7 in.
Peak Force:	38.8 k
Initial Linear Stiffness:	13.0 k/in.
Total Energy:	378.0 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	20.59	21.44	19.46	17.09
Energy (k-in.)	103.0	214.4	291.9	341.9









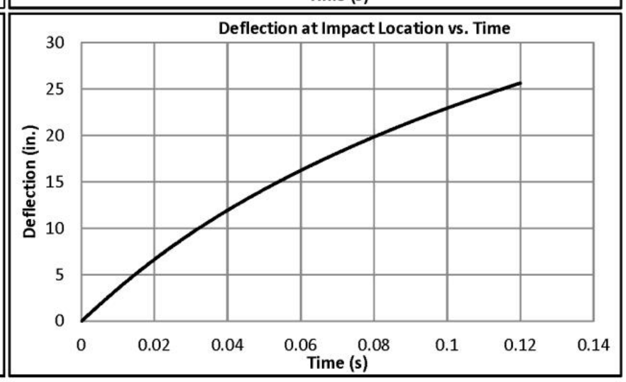


Figure D-10. Test No. ILOH4-5 Results (SLICE-2)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH4-6
Test Date:	8/2/2018
Failure Type:	Post yielding

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	20.51 mph	(30.08 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

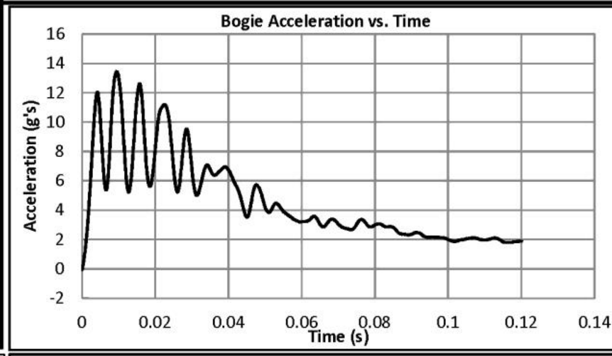
Data Acquired

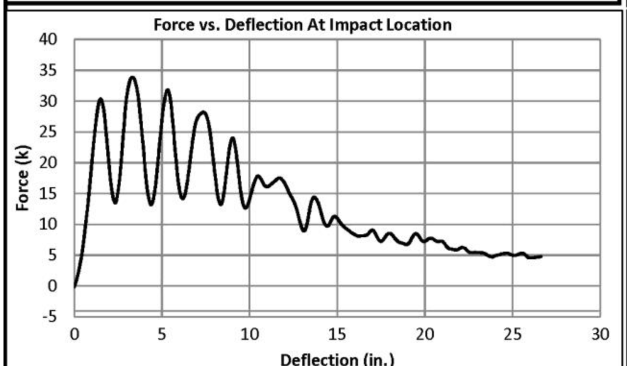
Accelerometer:	SLICE1
Camera Data:	aos9, pan2

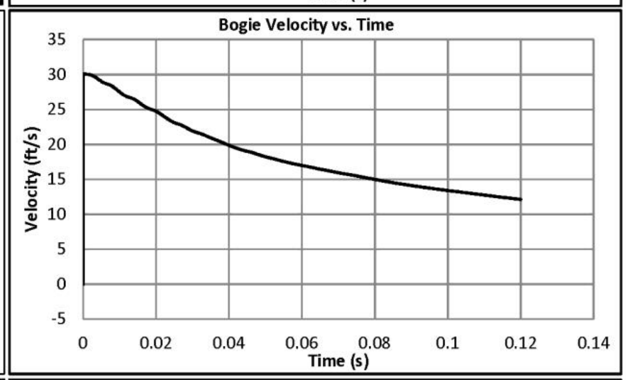
Test Results Summary

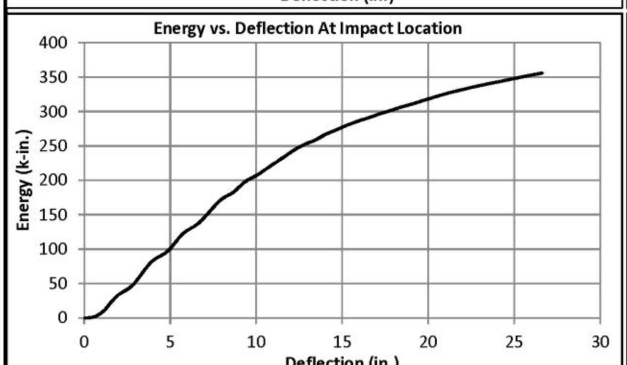
Event Duration:	0.1200 sec
Max. Deflection:	26.6 in.
Peak Force:	33.9 k
Initial Linear Stiffness:	10.2 k/in.
Total Energy:	356.1 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	20.05	20.70	18.48	15.92
Energy (k-in.)	100.2	207.0	277.1	318.3









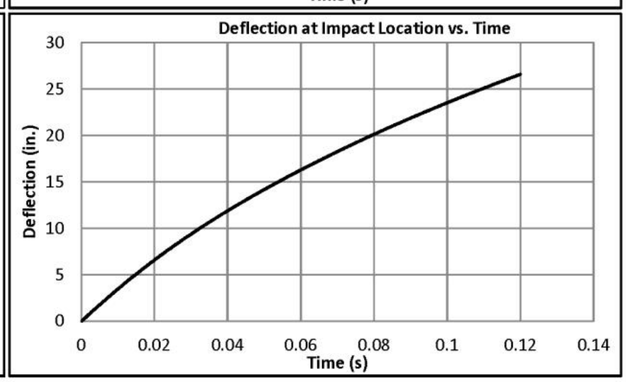


Figure D-11. Test No. ILOH4-6 Results (SLICE-1)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachment to concrete box beam
Test Number:	ILOH4-6
Test Date:	8/2/2018
Failure Type:	Post yielding

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	20.51 mph	(30.08 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

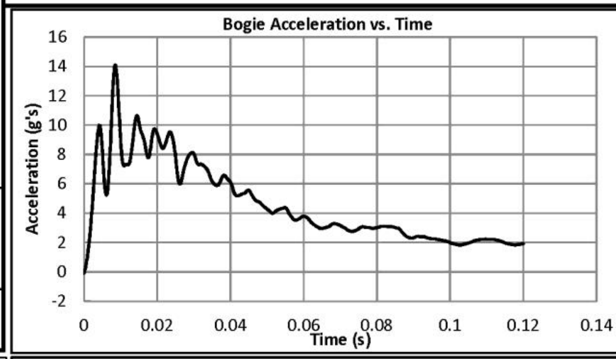
Data Acquired

Accelerometer:	SLICE2
Camera Data:	aos9, pan2

Test Results Summary

Event Duration:	0.1200 sec
Max. Deflection:	26.9 in.
Peak Force:	35.6 k
Initial Linear Stiffness:	11.9 k/in.
Total Energy:	355.1 k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	19.06	20.17	18.13	15.72
Energy (k-in.)	95.3	201.7	271.9	314.3



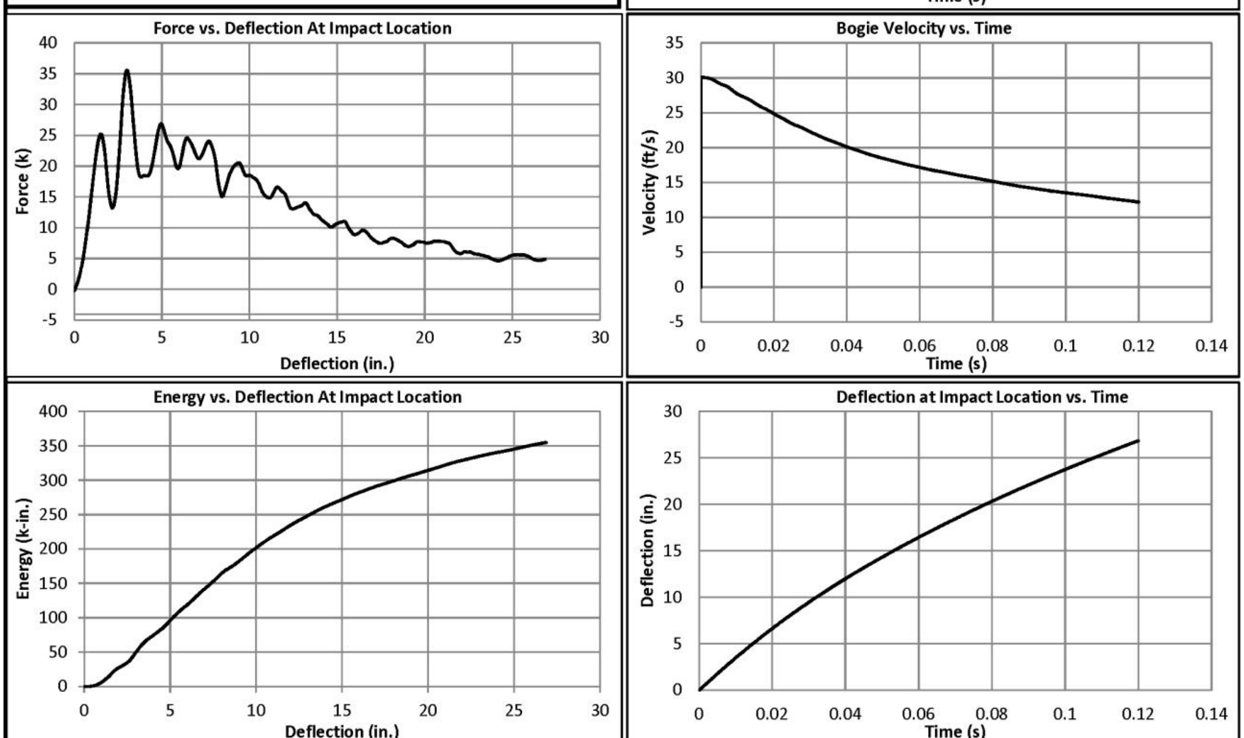


Figure D-12. Test No. ILOH4-6 Results (SLICE-2)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachement to concrete box beam
Test Number:	iloh4-7
Test Date:	8/8/2018
Failure Type:	Post yielding

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	19.95 mph	(29.26 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

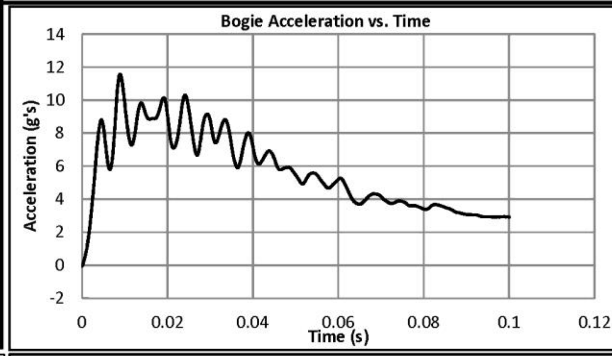
Data Acquired

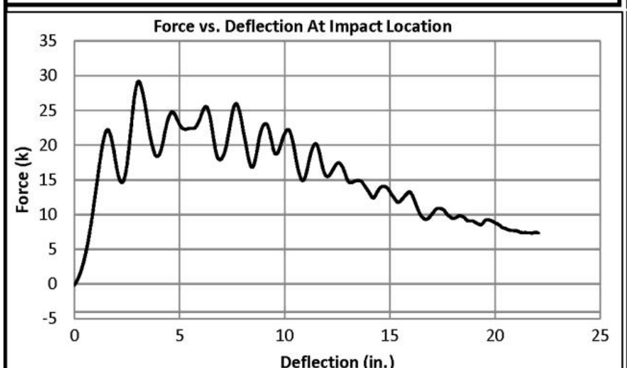
Accelerometer:	SLICE1
Camera Data:	aos9, aos8, pan1, pan2, solo1

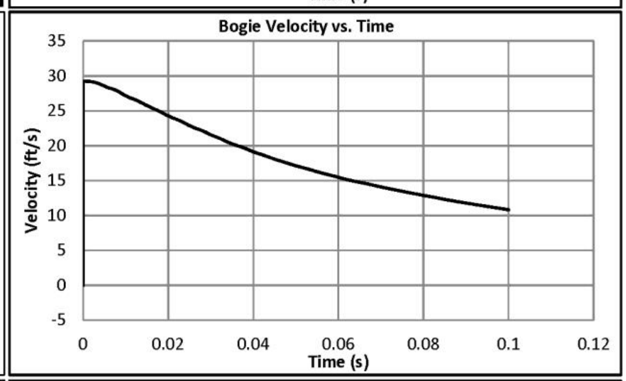
Test Results Summary

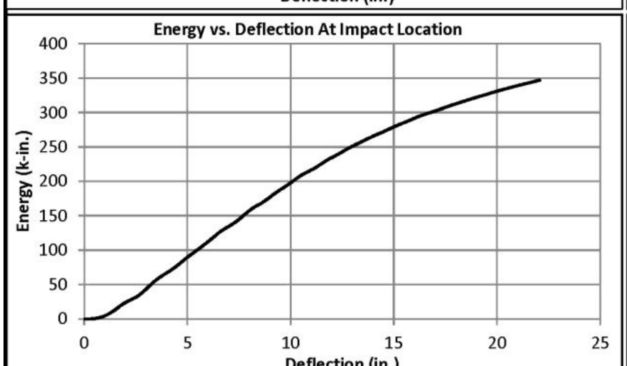
Event Duration:	0.1000	sec
Max. Deflection:	22.1	in.
Peak Force:	29.2	k
Initial Linear Stiffness:	9.5	k/in.
Total Energy:	347.2	k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	17.87	19.76	18.60	16.55
Energy (k-in.)	89.4	197.6	279.1	331.1









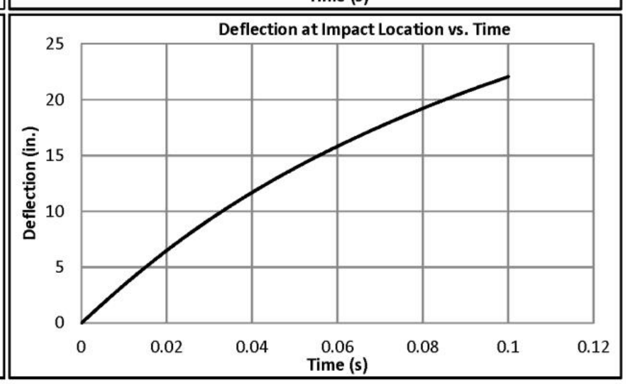


Figure D-13. Test No. ILOH4-7 Results (SLICE-1)

MIDWEST ROADSIDE SAFETY FACILITY

Bogie Test Summary

Test Information

Test Description:	Post attachement to concrete box beam
Test Number:	iloh4-7
Test Date:	8/8/2018
Failure Type:	Post yielding

Post Properties

Post Type:	Steel
Post Size:	W6x15 ASTM A992
Post Length:	58-1/4"
Embedment Depth:	15-5/8"
Orientation:	Strong

Bogie Properties

Impact Velocity:	19.95 mph	(29.26 ft/s)
Impact Height:	28"	
Bogie Mass:	2522 lb	

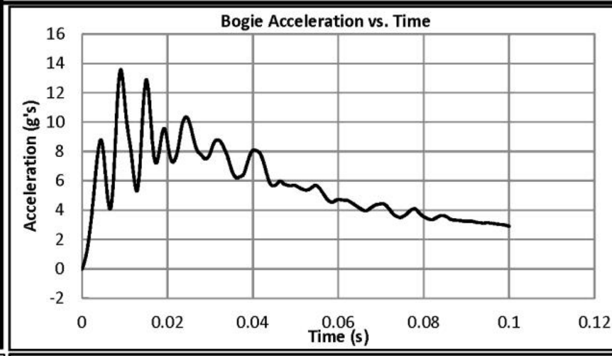
Data Acquired

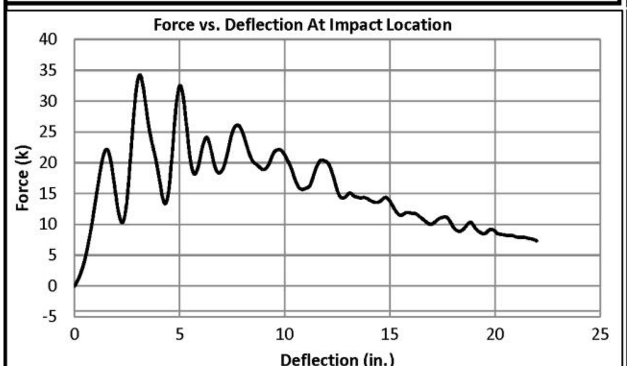
Accelerometer:	SLICE2
Camera Data:	aos9, aos8, pan1, pan2, solo1

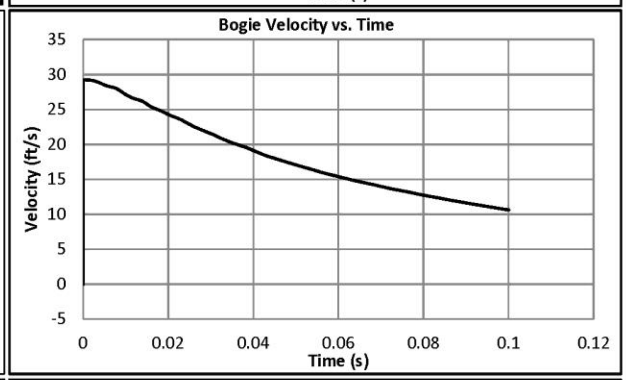
Test Results Summary

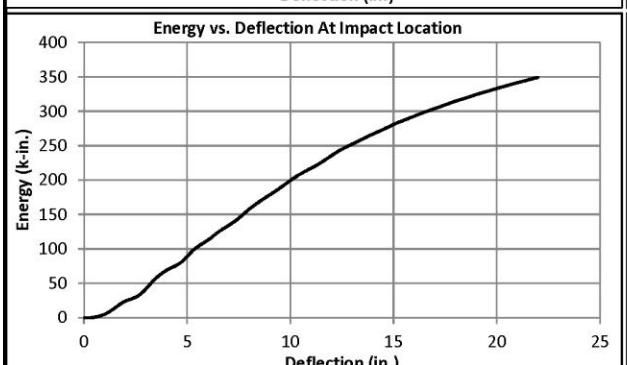
Event Duration:	0.1000	sec
Max. Deflection:	22.0	in.
Peak Force:	34.3	k
Initial Linear Stiffness:	11.0	k/in.
Total Energy:	349.3	k-in.

	@ 5"	@ 10"	@ 15"	@20"
Average Force (k)	17.72	19.96	18.71	16.66
Energy (k-in.)	88.6	199.6	280.7	333.2









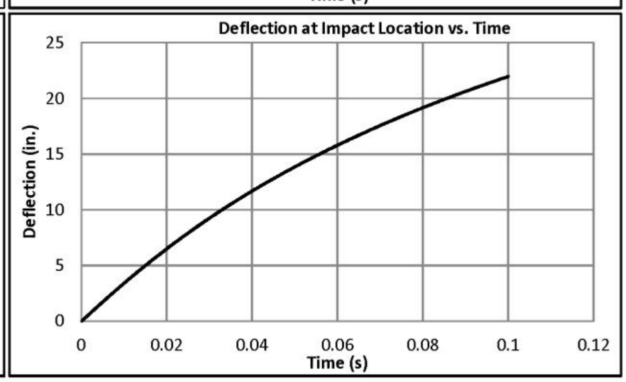


Figure D-14. Test No. ILOH4-7 Results (SLICE-2)

Appendix E. Material Specifications

Table E-1. Material Certification List, Simulated Box-beam Girder

Part Description	Material Specifications	Material Reference
#4 Bent Rebar, Upper Stirrup, 101¼ in. (2,572 mm) Total Unbent Length	ASTM A615 Gr. 60	Cert#: IL8280
#4 Bent Rebar, Bottom Stirrup, $106\frac{15}{16}$ in. (2,716 mm) Total Unbent Length	ASTM A615 Gr. 60	Cert#: IL8280
#4 Bent Rebar, Bottom Stirrup, $75\frac{7}{16}$ in. (1,916 mm) Total Unbent Length	ASTM A615 Gr. 60	Cert#: IL8280
#4 Rebar, 31 in. (787 mm) Long	ASTM A615 Gr. 60	Cert#: IL8280
#4 Rebar, 417 in. (10,592 mm) Long	ASTM A615 Gr. 60	Cert#: IL8280
#5 Rebar, 417 in. (10,592 mm) Long	ASTM A615 Gr. 60	Cert#: IL8280
#3 Rebar, 365 in. (9,271 mm) Long	ASTM A615 Gr. 60	Heat#: KN17103434 Cert#: IL8280 Batch#: 3777133B
#4 Bent Rebar U-Bar, 60 in. (1,524 mm) Total Unbent Length	ASTM A615 Gr. 60	Cert#: IL8280
#6 Bent Rebar, U-Bar, 69 in. (1,753 mm) Total Unbent Length	ASTM A615 Gr. 60	Heat#: KN17104670 KN1710585802 Cert#: IL8280 Batch#: 3813929
½ in. (13 mm) Dia., 7-Wire Prestressing Strand, 420 in. (10,668 mm) Long	ASTM A416 Gr. 270	-----
20 in. x 15 in. x ⅛ in. (508 mm x 381 mm x 3 mm) Steel Plate	ASTM A572 Gr. 50	-----
1 in. (25 mm) Dia., 32¾ in. (832 mm) Long Anchor Rod	ASTM F1554 Gr. 105	-----
1 in. (25 mm) Dia., 24 in. (610 mm) Long Anchor Rod	ASTM F1554 Gr. 105	-----
1 in. (25 mm) Dia., 15 in.(381 mm) Long Anchor Rod	ASTM F1554 Gr. 105	-----
⅞ in. (22 mm) Dia., 33 in.(838 mm) Long Anchor Rod	ASTM F1554 Gr. 105	-----
1 in. (25 mm) Dia., Heavy Hex Nut and Coupling Nut	ASTM A563DH	-----
⅞ in. (22 mm) Dia., Heavy Hex Nut and Coupling Nut	ASTM A563DH	-----
½ in. (13 mm) Dia. Shear Stud, 3 in. (76 mm) Long	ASTM A108	-----
3½ in. (13 mm) Dia., 7-Wire Prestressed Strands, 98⅜ in. (2,499 mm) Long, Lifting Loops and Conduit	Strands – ASTM A416 Gr. 270 Conduit – As supplied	-----
420 in. x 42 in. x 36 in.(10668 mm x 1067 mm x 914 mm) Concrete Box-beam	Min. f'c = 6,000 psi [41.4 MPA] 5,000 psi [34.5 MPa] @ Release	Batch# PC-01-18

Table E-2. Material Certification List, Welded Post Assembly A and D, Test Nos. ILOH4-1 and ILOH4-2

Part Description	Material Specifications	Material Reference
W6x15, 58¼ in. (1,480 mm) Long Steel Post	ASTM A992	Heat#: 59077011 Heat#: B145356
13 in. x 6¾ in. x 1¼ in. (330 mm x 171 mm x 32 mm) Post Plate with Slots for 1 in. (25 mm) Dia. Bolts	ASTM A572 Gr. 50	Heat#: A8B242
HSS5 in. x 4 in.x ⅜ in. (127 mm x 102 mm x 10 mm), 20 in. (508 mm) Long with 1⅛ in. (29 mm) Holes	ASTM A500 Gr. C	Heat#: 831559
1 in.-8 UNC (M24x3), 3½ in. (89 mm) Long Heavy Hex Head Bolt and Nut	Bolt-ASTM F3125 Gr. A325 Type 1 Nut-ASTM A563DH	Bolt Heat#:A28910 Nut Heat#:C114375 Part#: 19377 Cert#: 120297131
1 in.-8 UNC (M24x3), 2 in. (51 mm) Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	Bolt Heat#: 10440690 Nut Heat#: C114375 Part#: 19371 Cert#: 120297131
1 in. (25 mm) Square Washer	ASTM A36	Heat#: A8C270

Table E-3. Material Certification List, Welded Post Assembly G and E, Test Nos. ILOH4-3 to ILOH4-7

Part Description	Material Specifications	Material Reference
W6x15, 58¼ in. (1,480 mm) Long	ASTM A992	Heat#: 59077011 Heat#: B145356
13 in. x17¾ in. x 1 in. (330 mm x 451 mm x 25 mm) Post Plate with Slots for 1 in. (25 mm) Bolts	ASTM A572 Gr. 50	Heat#: A8D186
13 in. x 17¾ in. x ¾ in. (330 mm x 451 mm x 19 mm) Post Plate with Slots for 1 in. (25 mm) Bolts	ASTM A572 Gr. 50	Heat#: A7K866
HSS5 in.x4 in.x½ in. (127 mm x 102 mm x 13 mm), 20 in. (508 mm) Long with 1⅛ in. (29 mm) Holes	ASTM A500 Gr. C	Heat#: D42472
1 in.-8 UNC (M24x3), 3½ in. (89 mm) Long Heavy Hex Head Bolt and Nut	Bolt-ASTM F3125 Gr. A325 Type 1 Nut-ASTM A563DH	Bolt Heat#:A28910 Nut Heat#:C114375 Part#: 19377 Cert#: 120297131
1 in.-8 UNC (M24x3), 2¼ in. (57 mm) Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	Bolt Heat#: 10440690 Nut Heat#: C114375 Part#: 19371 Cert#: 120297131
1 in. (25 mm) Square Washer	ASTM A36	Heat#: A8C270

SOLD TO: ABC COATING CO INC
PO BOX 9693
TULSA, OK 74157-

NUCOR
NUCOR STEEL KANKAKEE, INC.

CERTIFIED MILL TEST REPORT Page: 1

SHIP TO: ABC COATING CO - IL
1160 BOUDREAU RD
MANTENO, IL 60950-

Ship from:
MTR #: 0000201264
Nucor Steel Kankakee, Inc.
One Nucor Way
Bourbonnais, IL 60914
815-937-3131

Date: 10-Nov-2017
B.L. Number: 548281
Load Number: 290308

Material Safety Data Sheets are available at www.nucorbar.com or by contacting your inside sales representative.

NBMG-08 January 1, 2012

LOT # HEAT #	DESCRIPTION	PHYSICAL TESTS YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% DEF	CHEMICAL TESTS C Ni	Mn Cr	P Mo	S V	Si Cb	Cu Sn	C.E.
PO# =>	092217-IL												
KN1710343401 KN17103434	Nucor Steel - Kankakee Inc 10/#3 Rebar 40' A706 GR60 WELDABLE ASTM A706/A706M-16 GR60 TEN/YD = 1.39 Melted 07/01/17 Rolled 07/11/17	71,955 496MPa	100,050 690MPa	17.1%	OK	-1.6% .030	.27 .19	1.08 .13	.015 .063	.042 .031	.20 .001	.36	.48

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.
1.) Weld repair was not performed on this material.
2.) Melted and Manufactured in the United States.
3.) Mercury, Radium, or Alpha source materials in any form have not been used in the production of this material.

QUALITY ASSURANCE: Caitlin Widdicombe

Caitlin Widdicombe

Figure E-1. Concrete Box-beam Reinforcement, No. 3 Bars

SOLD TO: ABC COATING CO INC
PO BOX 9693
TULSA, OK 74157-

NUCOR
NUCOR STEEL KANKAKEE, INC.

CERTIFIED MILL TEST REPORT Page: 1

SHIP TO: ABC COATING CO - IL
1160 BOUDREAU RD
MANTENO, IL 60950-

Ship from:
MTR #: 0000225247
Nucor Steel Kankakee, Inc.
One Nucor Way
Bourbonnais, IL 60914
815-937-3131

Date: 24-Mar-2018
B.L. Number: 555982
Load Number: 297078

Material Safety Data Sheets are available at www.nucorbar.com or by contacting your inside sales representative.

NBMG-08 January 1, 2012

LOT # / HEAT #	DESCRIPTION	PHYSICAL TESTS: YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% / DEF	CHEMICAL TESTS: C / Ni	Mn / Cr	P / Mo	S / V	Si / Cb	Cu / Sn	C.E.
PO# => KN1710585802 KN17105858	031418-IL Nucor Steel - Kankakee Inc 19/#6 Rebar 64' A615M GR420 (Gr60) 42013 - ASTM A615/A615M-16 GR 60 AASHTO M31-15 Melted 11/09/17 Rolled 11/12/17	69,957 482MPa	108,853 751MPa	12.4%	OK	-3.3% .048	.39 .19	.96 .16	.016 .061	.054 .009	.17 .001	.41 .016	
PO# => KN1810153501 KN18101535	031418-IL Nucor Steel - Kankakee Inc 13/#4 Rebar 40' A706 GR60 WELDABLE ASTM A706/A706M-16 GR60 TEN/YD = 1.44 Melted 03/04/18 Rolled 03/10/18	64,861 447MPa	93,417 644MPa	17.5%	OK	-4.2% .036	.25 .19	1.00 .14	.014 .059	.041 .032	.19 .001	.30	.44
PO# => KN1810153601 KN18101536	031418-IL Nucor Steel - Kankakee Inc 13/#4 Rebar 40' A706 GR60 WELDABLE ASTM A706/A706M-16 GR60 TEN/YD = 1.43 Melted 03/04/18 Rolled 03/10/18	66,501 459MPa	95,336 657MPa	15.8%	OK	-3.6% .036	.25 .19	1.01 .16	.013 .066	.030 .034	.21 .001	.32	.45

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.
1.) Weld repair was not performed on this material.
2.) Melted and Manufactured in the United States.
3.) Mercury, Radium, or Alpha source materials in any form have not been used in the production of this material.

Caitlin Widdicombe

QUALITY ASSURANCE: Caitlin Widdicombe

Figure E-2. Concrete Box-beam Reinforcement, No. 4 and No. 6 Bars

ABC Coating Company of IL
1160 N. Boudreau Road
Manteno, IL 60950-
Phone: (708)258-9633

IL8280

JOB NUMBER	RELEASE NUMBER	REQ. DELIVERY DATE	PAGE
8280	13E		1 of 1

JOB NAME

CC: UGS

CUSTOMER: ICCI

BY: FR

MATERIAL TYPE	REFERENCE	DRAWING ID	DESCRIPTION
Rebar, Grade A706, Epoxy	PO#3-20-18	IL-8280	STRAIGHT EC STOCK

Itm	Qty	Size	Length	Mark	Shape	Lbs	A	B	C	D	E	F/R	G	H	J	K	O	BC
1	400	4	40-00			10688				[illegible]								0
	400.					10688.												
2	12	3	21-00			95												0
	12.					95.												

Total Weight: 10,783 Lbs

Longest Length: 40-00

WEIGHT SUMMARY

	TOTAL			STRAIGHT			LIGHT BE[NDING]		[BEND]ING
SIZE	ITEMS	PIECES	LBS	ITEMS	PIECES	LBS	ITEMS	PIECE[S]	LBS
				Rebar, Grade A706, Epoxy					
3	1	12	95	1	12	95	0		0
4	1	400	10,688	1	400	10,688	0		0
	2	412	10,783	2	412	10,783	0		0

A706
3 (21-0) 95
4 (40-0) 10,688
10,783#

Total Weight: 10,783 Lbs

Longest Length: 40-00

R:13

40-0

Brown | pink.

v16.01.018 (T) (ABC)

Figure E-3. Epoxy Coating Reinforcement Details, No. 3 and No. 4 Bars

ABC Coating Company of IL
1160 N. Boudreau Road
Manteno, IL 60950-
Phone: (708)258-9633

IL8280

JOB NUMBER	RELEASE NUMBER	REQ. DELIVERY DATE	PAGE
8280	1E-REV-1		1 of 1

JOB NAME:

CC: QWH

CUSTOMER: ICCI

BY: FR

MATERIAL TYPE	REFERENCE	DRAWING ID	DESCRIPTION
Rebar, Grade A706, Epoxy	PO#3-7-18	IL-8280	PC-01-18 42" X 36" RESEARCH OH IL

Itm	Qty	Size	Length	Mark	Shape	Lbs	A	B	C	D	E	F/R	G	H	J	K	O	BC
1	8	6	5-02	6A9	17	62		1-04	2-062	1-04								
	8.					62.												H08
2	73	4	9-01	4A2	17	443		3-03	2-07	3-03								
3	63	4	8-07	4A1	4	361	2-11	0-043	0-052	1-002	0-052	0-043	2-11	0-03		0-042	2-07	H05
4	10	4	6-05	4A3	17	43		1-111	2-07	1-111								H06
5	8	4	4-11	4A8	17	26		1-04	2-03	1-04								H05
	154.					873.												H05

Total Weight: 935 Lbs

Longest Length: 9-01

WEIGHT SUMMARY

A706
4 - 873
6 - 62
935#

SIZE	TOTAL ITEMS	TOTAL PIECES	TOTAL LBS	STRAIGHT ITEMS	STRAIGHT PIECES	STRAIGHT LBS	LIGHT BE ITEMS	PIECI	ING LBS
				Rebar, Grade A706, Epoxy					
4	4	154	873	0	0	0	1		512
6	1	8	62	0	0	0	0		62
	5	162	935	0	0	0	1		574

R: 1-REV-1

Total Weight: 935 Lbs

Longest Length: 9-01

9-1

v16.01.018 (T) (ABC)

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Figure E-4. Epoxy Coating Reinforcement Details, No. 4 and No. 6 Bars

HDSUPPLY
CONSTRUCTION & INDUSTRIAL
WHITE CAP

PC-01-18 Pour date = 4/13/18

1. Concrete temp = 68°
Slump = 8 1/2"
Air = 7.2%
Cylinder break 4/14/18 = 6826 psi

2. Concrete temp = 70°
Slump = 8 1/2"
Air = 6.3%
Cylinder break 4/14/18 = 7560 psi

Bottom = 7:49 – 7:58
Top = 9:01 – 9:44
Tarp = 9:55

Figure E-5. Concrete Box-beam Girder, Strength Tes

GERDAU

US-ML-MIDLOTHIAN
300 WARD ROAD
MIDLOTHIAN, TX 76065
USA

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID:
STEEL & PIPE SUPPLY CO INC 401 NEW CENTURY PKWY NEW CENTURY,KS 66031-1127 USA	STEEL & PIPE SUPPLY CO INC MANHATTAN,KS 66505-1688 USA	A992/A572-50	Wide Flange Beam / 6 X 15# / 150 X 22.5	0000171713

LENGTH	WEIGHT	HEAT / BATCH
40'00"	7,200 LB	59077011/02

SALES ORDER	CUSTOMER MATERIAL Nº
5852732/000020	000000000376150040

SPECIFICATION / DATE or REVISION
ASTM A6-14
ASTM A709-15
ASTM A992-11 (2015), A572-15
CSA G40.21-13 345WM

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
G450024810	1327-0000259022	12/04/2017

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	Nb %	Al %
0.10	0.86	0.013	0.037	0.19	0.25	0.12	0.19	0.014	0.006	0.002	0.012	0.003

CHEMICAL COMPOSITION

CEqyA6 %
0.31

MECHANICAL PROPERTIES

YS 0.2% PSI	UTS PSI	YS MPa	UTS MPa	Y/T ratio %	G/L Inch
54153	75828	373	523	0.720	8.000
53731	74011	370	510	0.730	8.000

MECHANICAL PROPERTIES

G/L mm	Elong. %
200.0	25.00
200.0	25.00

COMMENTS / NOTES

The above figures are certified chemical and physical test records as contained in the permanent records of company. We certify that these data are correct and in compliance with specified requirements. This material, including the billets, was melted and manufactured in the USA. CMTR complies with EN 10204 3.1.

Bhaskar
BHASKAR YALAMANCHILI
QUALITY DIRECTOR
Phone: (409) 769-1014 Email: Bhaskar.Yalamanchili@gerdau.com

Wade A. Lumpkins
WADE LUMPKINS
QUALITY ASSURANCE MGR.
Phone: 972-779-3118 Email: Wade.Lumpkins@gerdau.com

Figure E-6. W6x15 Steel Post, Test Nos. ILOH4-1 to ILOH4-7

Steel Dynamics, Inc.
Long Products Group

(260) 625-8100 (260) 625-8950 FAX
Quality Steel 100% EAF Melted and Manufactured in the USA
Recycled content: PC = 75.0%, PI = 22.0%
ISO 9001:2008 and ABS Certified

CERTIFIED MILL TEST REPORT

Printed: **03 / 13 / 2018**
Produced: **02 / 13 / 2018**

Customer # 000058

Ship to:
Steel & Pipe Supply
401 New Century Parkway
New Century KS, 66031 US
Attn: Receiving

Bill to:
Steel & Pipe Supply - Kansas
555 Poyntz Avenue
PO Box 1688
Manhattan KS, 66505 US
Attn: Kaycia VanSickle

GENERAL INFORMATION

Product Wide Flange Beam
Size W6X15
W150X22.5
Heat Number B145356
Condition(s) As-Rolled
Fine Grained
Fully Killed
No Weld Repair

SPECIFICATIONS

Standards	Grades*
ASTM A6/A6M - 17	
» ASTM A992/A992M - 11	A992 / A992M
ASTM A572/A572M - 15	A572 gr50/gr345
ASTM A709/A709M - 17	A709 g50S/g345S
AASHTO M270M/M270 - 12	M270 gr345/gr50
CSA G40.21-13	50WM/345WM
ASTM A36/A36M - 14	A36 / A36M

*SDI-MULTI meets the requirements of ASTM A992, A572-50, A529-50, A709-50, M270-50, A36, A709-36, M270-36, CSA300W, CSA345WM, CSA350W.

SHIPMENT DETAILS

BOL # 0000490517 - 18000.00 lbs

Bundle / ASN #	Length	pcs	Cust PO | Recv PO | Job
060880618	25' 0"	12	4500303152
060880619	25' 0"	12	4500303152
060880620	25' 0"	12	4500303152
060880617	25' 0"	12	4500303152

CHEMICAL ANALYSIS (weight percent)

C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Sn	V	Nb/Cb	Al	N	B	*C1	*C2	*C3	*PC	*I	Analysis Type
.05	1.01	.009	.035	.21	.25	.11	.09	.04	.013	.031	<.001	.001	.0060	.0002	.27	.310	.24	.14	5.22	Heat

MECHANICAL TESTING

Test	Yield (fy) Strength ksi / MPa	Tensile (fu) Strength ksi / MPa	fy / fu ratio	% Elong. (8" gage)
1	56 / 386	69 / 476	.80	28
2	56 / 386	70 / 483	.80	30
3				
4				

CHARPY IMPACT TESTS (available only when specified at time of order)

Test	Temp F / C	Absorbed Energy ft-lbf / J Specimen 1	Specimen 2	Specimen 3	Average	Minimum
1						
2						
3						
4						
5						
6						
7						

Notes: *Calculated Chemistry Values Carbon Equivalents (C1 C2 C3 PC). Corrosion Index (I) (ASTM G101)= 26.01(Cu)+3.88(Ni)+1.20(Cr)+1.49(Si)+17.29(P)-7.29(Cu)(Ni)-9.10(Ni)(P)-33.39(Cu²) Pcm(AWS) = C+Si/30+Mn/20+Cu/20+Ni/60+Cr/20+mo/15+V/10+5B
CE1 (IIW)=C+Mn/6+(Cr+Mo+V)/5+(Ni+Cu)/15 CE2 (AWS)=C+(Mn+Si)/6+(Cr+Mo+V)/5+(Ni+Cu)/15 CE3 (CET) = C + (Mn/6) + (Si/24) + (Cr/5) + (Ni/40) +(Mo/4) + (V/14)

I hereby certify that the material described herein has been made to the applicable specification by the electric arc furnace/continuous cast process and tested in accordance with the requirements of American Bureau of Shipping Rules with satisfactory results.

Signed:

ABS CERTIFICATION

I hereby certify that the content of this report are accurate and correct. All tests and operations performed by this material manufacturer are in compliance with the requirements of the material specifications and applicable purchaser designated requirements.

Signed: **Todd Bashford** Todd A Bashford
Quality Manager

State of Indiana, County of Whitley Sworn to and subscribed before me

this ________ day of ________________

Signed: ____________________ My commission expires: __________
Notary Public

Form F-5120-002-054

ASTM A6 - 14.6. A signature is not required on the test report however the document shall clearly identify the organization submitting the report. Notwithstanding the absence of a signature the organization submitting the report is responsible for the content of the report.

Page 3 of 3

Figure E-7. W6x15 Steel Post, Test Nos. ILOH4-1 to ILOH4-7

SSAB

Preliminary Test Certificate

Form TC1: Revision 3: Date 7 Feb 2018

1770 Bill Sharp Boulevard, Muscatine, IA 52761-9412, US **Official copy to follow**

Customer: STEEL & PIPE SUPPLY P.O. BOX 1688 MANHATTAN KS 66502	Customer P.O. No.: 4500301093	Mill Order No.: 41-528365-01	Shipping Manifest : MT340200
	Product Description: ASTM A572-50/M345(15)/A709-50/M345(17)	Ship Date: 07 Mar 18 Cert Date: 07 Mar 18	Cert No: 061695426 (Page 1 of 1)
	Size: 1.250 X 96.00 X 240.0 (IN)		

Tested Pieces				Tensiles						Hardness	Charpy Impact Tests					BDWTT
Heat Id	Piece Id	Tested Thickness	Tst Loc	YS (KSI)	UTS (KSI)	%RA	Elong % 2in	Elong % 8in	Tst Dir	Hardness	Abs. Energy(FTLB) 1 2 3 Avg	% Shear 1 2 3 Avg	Tst Tmp	Tst Dir	Tst Siz (mm)	Tmp %Shr
A8B242	D03	1.248 (DISCRT)	L	55	79			29	T							
B7K740	B08	1.248 (DISCRT)	L	65	76			25	T							
B8A862	B49	1.248 (DISCRT)	L	60	81			24	T							
B8A862	B50	1.582 (DISCRT)	L	57	79			25	T							

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti	ORGN
A8B242	.17	1.12	.017	.001	.24	.030	.28	.16	.19	.03	.002	.051	.009	USA
B7K740	.05	1.30	.017	.004	.19	.028	.32	.16	.17	.03	.028	.039	.002	USA
B8A862	.17	1.23	.010	.003	.20	.033	.32	.20	.11	.03	.002	.040	.008	USA

KILLED STEEL
MERCURY IS NOT A METALLURGICAL COMPONENT OF THE STEEL AND NO MERCURY WAS INTENTIONALLY ADDED DURING THE MANUFACTURE OF THIS PRODUCT.
MTR EN 10204:2004 INSPECTION CERTIFICATE 3.1 COMPLIANT
100% MELTED AND MANUFACTURED IN THE USA.
PRODUCTS SHIPPED:

A8B242	D03	PCES: 3, LBS:	24504	B8A862	B47	PCES: 1, LBS:	8168
B7K740	B08	PCES: 1, LBS:	8168				

(H) Cust Part # : 7210896240A2

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

SENIOR METALLURGIST - PRODUCT

Figure E-8. 13 in. x 6¾ in. x 1¼ in. (330 mm x 172 mm x 32 mm) Post Plate with Slotteds for 1 in. (25 mm) Dia. Bolts, Test Nos. ILOH4-1 & ILOH4-2

SSAB

Preliminary Test Certificate

Form TC1: Revision 3: Date 7 Feb 2018

1770 Bill Sharp Boulevard, Muscatine, IA 52761-9412, US **Official copy to follow**

Customer: STEEL & PIPE SUPPLY P.O. BOX 1688 MANHATTAN KS 66502	Customer P.O. No.: 4500305027	Mill Order No.: 41-535857-01	Shipping Manifest : MT345299
	Product Description: ASTM A572-50/M345(15)/A709-50/M345(17)	Ship Date: 30 Apr 18 Cert Date: 30 Apr 18	Cert No: 061706586 (Page 1 of 1)
	Size: 1.000 X 96.00 X 240.0 (IN)		

Tested Pieces				Tensiles						Charpy Impact Tests						
Heat Id	Piece Id	Tested Thickness	Tst Loc	YS (KSI)	UTS (KSI)	%RA	Elong % 2in 8in	Tst Dir	Hardness	Abs. Energy(FTLB) 1 2 3 Avg	% Shear 1 2 3 Avg	Tst Tmp	Tst Dir	Tst Siz (mm)	BDWTT Tmp %Shr
A8D186	C79	1.123 (DISCRT)	L	56	76		27	T							
A8D186	C80	0.370 (DISCRT)	L	62	80		26	T							

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti	ORGN
A8D186	.16	1.11	.010	.001	.04	.032	.25	.14	.12	.04	.001	.034	.007	USA

KILLED STEEL
MERCURY IS NOT A METALLURGICAL COMPONENT OF THE STEEL AND NO MERCURY WAS INTENTIONALLY ADDED DURING THE MANUFACTURE OF THIS PRODUCT.
MTR EN 10204:2004 INSPECTION CERTIFICATE 3.1 COMPLIANT
100% MELTED AND MANUFACTURED IN THE USA.
PRODUCTS SHIPPED:
A8D186 C78 PCES: 3, LBS: 19602

Cust Part # : 7210096240A2

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

SENIOR METALLURGIST - PRODUCT

Figure E-9. 13 in. x 17¾ in. x 1 in. (330 mm x 451 mm x 25 mm) Post Plate with Slotteds for 1 in. (25 mm) Dia. Bolts, Test Nos. ILOH4-3 to ILOH4-6

SSAB

Preliminary Test Certificate

1770 Bill Sharp Boulevard, Muscatine, IA 52761-9412, US **Official copy to follow**

Form TC1: Revision 2: Date 23 Apr 2014

Customer: STEEL & PIPE SUPPLY P.O. BOX 1688 MANHATTAN KS 66502	Customer P.O. No.: 4500297759	Mill Order No.: 41-521655-06	Shipping Manifest : MT333724	
	Product Description: ASTM A572-50/M345(15)/A709-50/M345(17)		Ship Date: 20 Dec 17 Cert Date: 20 Dec 17	Cert No: 061682485 (Page 1 of 1)
	Size: 0.750 X 96.00 X 240.0 (IN)			

Tested Pieces			Tensiles								Charpy Impact Tests					
Heat Id	Piece Id	Tested Thickness	Tst Loc	YS (KSI)	UTS (KSI)	%RA	Elong % 2in	Elong % 8in	Tst Dir	Hardness	Abs. Energy(FTLB) 1 2 3 Avg	% Shear 1 2 3 Avg	Tst Tmp	Tst Dir	Tst Siz (mm)	BDWTT Tmp %Shr
A7K866	D58	0.750 (DISCRT)	L	58	79		26		T							
A7L809	B12	0.748 (DISCRT)	L	56	70		35		T							

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti	ORGN
A7K866	.16	1.00	.014	.004	.17	.028	.36	.17	.16	.06	.001	.026	.004	USA
A7L809	.05	1.34	.016	.006	.16	.030	.32	.12	.12	.02	.026	.024	.010	USA

KILLED STEEL
MERCURY IS NOT A METALLURGICAL COMPONENT OF THE STEEL AND NO MERCURY WAS INTENTIONALLY ADDED DURING THE MANUFACTURE OF THIS PRODUCT.
MTR EN 10204:2004 INSPECTION CERTIFICATE 3.1 COMPLIANT
100% MELTED AND MANUFACTURED IN THE USA.
PRODUCTS SHIPPED:
A7L809 B12 PCES: 5, LBS: 24505 A7K866 D58 PCES: 2, LBS: 9802

(II) Cust Part # : 722496240A2

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

SENIOR METALLURGIST - PRODUCT

Figure E-10. 13 in. x 17 ¾ in. x ¾ in. Post Plate with Slotteds for 1 in. (25 mm) Dia. Bolts, Test No. ILOH4-

Atlas Tube Canada ULC
200 Clark Street
Harrow, Ontario, Canada
N0R 1G0
Tel: (519) 738-5000
Fax: (519) 738-3537

Atlas Tube
A DIVISION OF ZEKELMAN INDUSTRIES

Ref.B/L: 80814177
Date: 03.29.2018
Customer: 193

MATERIAL TEST REPORT

Sold to

Tubular Steel
1031 Executive Parkway
ST. LOUIS MO 63141
USA

Shipped to

Tubular Steel
7220 Polson Lane
HAZELWOOD MO 63042
USA

Material: 10.0x3.0x375x48'0"0(1x3)PB **Material No:** 100030375 **Made in:** Canada **Melted in:** USA

Sales order: 1272550 **Purchase Order:** PO-067967 **Cust Material #:** 013282

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
L67389	0.200	0.840	0.010	0.008	0.019	0.054	0.030	0.004	0.002	0.010	0.060	0.000	0.000	0.000	0.002

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M101765353	3	060072 Psi	072623 Psi	35 %	ASTM A500-13 GRADE B&C	0.36

Material Note:
Sales Or.Note:

Material: 5.0x4.0x375x40'0"0(3x3). **Material No:** 500403754000 **Made in:** Canada **Melted in:** Canada

Sales order: 1271023 **Purchase Order:** PO-067883 **Cust Material #:** 012320

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
831559	0.200	0.800	0.007	0.006	0.017	0.049	0.026	0.005	0.002	0.012	0.027	0.002	0.002	0.000	0.005

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M101761465	9	068472 Psi	075119 Psi	28.5 %	ASTM A500-13 GRADE B&C	0.34

Material Note:
Sales Or.Note:

Material: 5.0x4.0x375x40'0"0(3x3). **Material No:** 500403754000 **Made in:** Canada **Melted in:** Canada

Sales order: 1271023 **Purchase Order:** PO-067883 **Cust Material #:** 012320

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
831559	0.200	0.800	0.007	0.006	0.017	0.049	0.026	0.005	0.002	0.012	0.027	0.002	0.002	0.000	0.005

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M101761464	9	068472 Psi	075119 Psi	28.5 %	ASTM A500-13 GRADE B&C	0.34

Material Note:
Sales Or.Note:

Authorized by Quality Assurance: Jason Richard

The results reported on this report represent the actual attributes of the material furnished and indicate full compliance with all applicable specification and contract requirements.
D1.1 method.

Steel Tube Institute OF NORTH AMERICA

Page : 2 Of 3

Metals Service Center Institute

Figure E-11. HSS5 in. x 4 in. x $^3/_8$ in., 20 in. (508 mm) Long with $1^1/_8$ in. (29 mm) Holes, Test Nos. ILOH4-1 & ILOH4-2

Atlas Tube Corp (Chicago)
1855 East 122nd Street
Chicago, Illinois, USA
60633
Tel: 773-646-4500
Fax: 773-646-6128

Atlas Tube
A DIVISION OF ZEKELMAN INDUSTRIES

Ref.B/L: 80782893
Date: 09.22.2017
Customer: 193

MATERIAL TEST REPORT

Sold to

Tubular Steel
1031 Executive Parkway
ST. LOUIS MO 63141
USA

Shipped to

Tubular Steel
7220 Polson Lane
HAZELWOOD MO 63042
USA

Material: 5.0x4.0x500x40'0"0(5x1)PB **Material No:** 50040500 **Made in:** USA **Melted in:** USA

Sales order: 1215474 **Purchase Order:** PO-064102 **Cust Material #:** 012321

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
D42472	0.180	0.740	0.014	0.008	0.015	0.039	0.030	0.006	0.002	0.010	0.040	0.001	0.001	0.000	0.004

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M800731244	5	060035 Psi	075435 Psi	35 %	ASTM A500-13 GRADE B&C	0.32

Material Note:
Sales Or.Note:

Material: 8.0x8.0x375x36'0"0(1x1)REC **Material No:** 80080375 **Made in:** USA **Melted in:** USA

Sales order: 1215474 **Purchase Order:** PO-064102 **Cust Material #:** 012965

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
17068721	0.200	0.720	0.007	0.003	0.020	0.031	0.100	0.001	0.010	0.030	0.040	0.003	0.000	0.000	0.007

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M900943835	1	061062 Psi	076619 Psi	34 %	ASTM A500-13 GRADE B&C	0.34

Material Note:
Sales Or.Note:

Material: 8.0x8.0x375x32'0"0(1x1)REC **Material No:** 80080375 **Made in:** USA **Melted in:** USA

Sales order: 1215474 **Purchase Order:** PO-064102 **Cust Material #:** 012965

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
D42170	0.210	0.810	0.015	0.014	0.010	0.052	0.040	0.004	0.008	0.020	0.040	0.002	0.001	0.000	0.004

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M900943803	1	067323 Psi	080361 Psi	32 %	ASTM A500-13 GRADE B&C	0.36

Material Note:
Sales Or.Note:

Jason Richard
Jason Richard

Authorized by Quality Assurance:
The results reported on this report represent the actual attributes of the material furnished and indicate full compliance with all applicable specification and contract requirements.
CE calculated using the AWS D1.1 method.

Steel Tube Institute OF NORTH AMERICA

Page : 1 Of 4

Metals Service Center Institute

PO 50508
MAY, 18
54411

Figure E-12. HSS5 in. x 4 in. x ½ in., 20 in. (508 mm) Long with $1^1/_8$ in. (29 mm) Holes, Test Nos. ILOH4-3 to ILOH4-7

INFASCO

MANUFACTURER'S LOT REFERENCE

This is to confirm that according to Infasco's database, Ifastgroupe 2004 L.P. and/or its divisions or subsidiaries have supplied the following lot number(s) to:

FASTENAL CO INC (WINONA MN)

201726016

in connection with purchase order **120297131** and part number **19377**

Figure E-13. 1 in.-8 UNC (M24x3), 3½ in. (89 mm) Long Heavy Hex Head Bolt and Nut, Test Nos. ILOH4-1 to ILOH4-7, Sheet 1 of 4

INFASCO

SET NO.: 2017-26016

ISO 9001 : 2008
ISO / IEC 17025
ISO 14001 : 2004

FASTENER TEST REPORT

(THIS DOCUMENT MAY ONLY BE REPRODUCED IN ITS ENTIRETY, WITH PRIOR WRITTEN APPROVAL BY THE INFASCO LABORATORY)

(THE INFASCO LABORATORY IS ACCREDITED BY THE CCN FOR THE TESTS LISTED AT WWW.CCN.CA)

COMPLIES WITH EN10204:2004 INSPECTION CERTIFICATE 3.1

DATE: 2017-09-01

DESCRIPTION C A325-1+A563-DH NA UNC OS
1-8 X 3 1/2

BOLT A325-1 STRUCTURAL BOLT UNC N HDG
MARKING : HOLLOW TRIANGLE & "A325"

LOT NO.	MANUFACTURED BY	HARDNESS (ROCKWELL)	PROOF LOAD (LB)	TENSILE STRENGTH (LB)	
1706-55498 7739G	INFASCO	HRC 25.0 – HRC 34.0	MIN: 51,500	MIN: 72,700	
MEAN VALUE		29.3	PASS	87,633	

HEAT NO.	C %	Mn %	P %	S %	Si %				
A28910	0.38	0.96	0.007	0.012	0.18				

NUT HVY HEX NUT A563-DH FNA OS. UNC OS HDG
MARKING : TRIANGLE & "DH"

LOT NO.	MANUFACTURED BY	HARDNESS (ROCKWELL)	PROOF LOAD (LB)		
1504-53627 1775G	INFASCO	HRC 24.0 – HRC 38.0	MIN: 90,900		
MEAN VALUE		30.0	PASS		

HEAT NO.	C %	Mn %	P %	S %	Si %	Cu %	Ni %		
C114375	0.45	0.85	0.007	0.011	0.22	0.12	0.04		

HEAT CHEMICAL ANALYSIS PROVIDED BY STEEL SUPPLIER.
THE ASSEMBLY MEETS THE REQUIREMENTS OF ASTM A325.
-NUTS LUBRICATED

THE ABOVE SET HAS BEEN ROTATIONAL CAPACITY TESTED WITH A RANDOM LOT OF GALVANIZED WASHERS TO VALIDATE THE LUBRICITY OF THE NUT. TEST RESULT: PASS

Isabelle Parent, Eng., M.A.Sc.
Quality Assurance Foreman

INFASCO
A division of Ifastgroupe LP
A Heico Company
700 Ouellette, Marieville (Quebec) J3M 1P6
Tel.: (450) 658-8741 Fax: (450) 460-5496

FQ-019-4 Rev. 08 Revision date of test report: 2017-09-05 Page 1 of 1

Figure E-14. 1 in.-8 UNC (M24x3), 3½ in. (89 mm) Long Heavy Hex Head Bolt and Nut, Test Nos. ILOH4-1 to ILOH4-7, Sheet 2 of 4

INFASCO

LOT NO.: **1706-55498**
7739G

ISO 9001, ISO/TS16949
ISO / IEC 17025
ISO 14001

FASTENER TEST REPORT

(THIS DOCUMENT MAY ONLY BE REPRODUCED IN ITS ENTIRETY, WITH PRIOR WRITTEN APPROVAL BY THE INFASCO LABORATORY)
(THE INFASCO LABORATORY IS ACCREDITED BY THE CCN FOR THE TESTS LISTED AT WWW.CCN.CA)
COMPLIES WITH EN10204:2004 INSPECTION CERTIFICATE 3.1

DATE 2017-07-12

DESCRIPTION AND MARKING: A325-1 STRUCTURAL BOLT UNC N HDG
HOLLOW TRIANGLE & "A325"

SIZE	GRADE	QUANTITY
1-8 X 3 1/2	1037M	16,400

HEAT CHEMICAL ANALYSIS (provided by steel supplier)

HEAT NO.	C %	Mn %	P %	S %	Si %				
A28910	0.38	0.96	0.007	0.012	0.18				

METHOD	ASTM F606	ASTM F606			ASTM F606		ASTM E376
	PROOF LOAD (psi)	WEDGE TENSILE STRENGTH (psi)	SHEAR STRENGTH	SURFACE HARDNESS (HR 30N)	CORE HARDNESS (ROCKWELL)	MICRO HARDNESS	COATING THICKNESS (0.001 in)
SPEC. MIN.	85,000	120,000			HRC 25.0		2.00
SPEC. MAX:					HRC 34.0		
SAMPLE NO.1	85,000	144,000			HRC 29.7		4.77
NO.2	85,000	145,000			28.9		4.54
NO.3	85,000	145,000			29.3		4.52
NO.4					29.1		4.53
NO.5							4.95
NO.6							4.15
NO.7							3.65
NO.8							3.87
NO.9							3.91
NO.10							3.68
NO.11							3.52
NO.12							3.96
NO.13							3.21
NO.14							3.84
NO.15							3.29

THE ABOVE TESTED SAMPLES HAVE BEEN INSPECTED FOR VISUAL DISCONTINUITIES AND FOUND ACCEPTABLE.
THEY COMPLY IN ALL RESPECTS WITH THE LATEST EDITION OF THE FOLLOWING SPECS:
ASTM A325 TYPE 1 ASME B18.2.6, THREADS PER ASME B1.1 CLASS 2A. UNLESS OTHERWISE SPECIFIED.
MEETS THE SURFACE DISCONTINUITIES REQUIREMENTS
ASTM F2329, ASTM-A-153 Class C
THESE FASTENERS WERE OIL QUENCHED AND TEMPERED AT A TEMP. ABOVE 800°F.
NO BISMUTH, SELENIUM, TELLURIUM OR LEAD HAVE BEEN INTENTIONALLY ADDED

COATING THICKNESS VALUES PROVIDED BY COATING SUPPLIER.

MANUFACTURED IN : CANADA
The steel was melted and rolled
in North America and is mercury and asbestos-free.

Isabelle Parent, Eng., M.A.Sc.
Quality Assurance Foreman

INFASCO
A division of Ifastgroupe LP
A Heico Company
700 Ouellette, Marieville (Quebec) J3M 1P6
Tel.: (450) 658-8741 Fax: (450) 460-5496

FQ-019-2 Rev. 09 Revision date of test report: 2017-08-25 Page 1 of 1

Figure E-15. 1 in.-8 UNC (M24x3), 3½ in. (89 mm) Long Heavy Hex Head Bolt and Nut, Test Nos. ILOH4-1 to ILOH4-7, Sheet 3 of 4

INFASCO

ISO 9001, ISO/TS16949
ISO / IEC 17025
ISO 14001

LOT NO.: 1504-53627
1775G

FASTENER TEST REPORT

(THIS DOCUMENT MAY ONLY BE REPRODUCED IN ITS ENTIRETY, WITH PRIOR WRITTEN APPROVAL BY THE INFASCO LABORATORY)
(THE INFASCO LABORATORY IS ACCREDITED BY THE CCN FOR THE TESTS LISTED AT WWW.CCN.CA)
COMPLIES WITH EN10204:2004 INSPECTION CERTIFICATE 3.1

DATE 2015-08-19

DESCRIPTION AND MARKING: HVY HEX NUT A563-DH FNA OS. UNC OS HDG+L
TRIANGLE & "DH"

SIZE	GRADE	QUANTITY
1-8 .024 OS	1046	103,000

HEAT CHEMICAL ANALYSIS (provided by steel supplier)

HEAT NO.	C %	Mn %	P %	S %	Si %	Cu %	Ni %		
C114375	0.45	0.85	0.007	0.011	0.22	0.12	0.04		

METHOD	ASTM F606 PROOF LOAD (psi)	WEDGE TENSILE STRENGTH	SHEAR STRENGTH	SURFACE HARDNESS (HR 30N)	ASTM F606 CORE HARDNESS (ROCKWELL)	MICRO HARDNESS	ASTM E376 COATING THICKNESS (0.001 in)
SPEC. MIN.	150,000				HRC 24.0		2.00
SPEC. MAX:					HRC 38.0		
SAMPLE NO.1	155,000				HRC 29.8		2.29
NO.2	154,000				30.5		2.73
NO.3	155,000				30.8		2.95
NO.4	154,000				29.6		3.70
NO.5	154,000				29.4		3.46
NO.6							3.03
NO.7							3.92
NO.8							2.32
NO.9							3.93
NO.10							3.21
NO.11							4.59
NO.12							3.23
NO.13							3.07
NO.14							3.27
NO.15							3.52

THE ABOVE TESTED SAMPLES HAVE BEEN INSPECTED FOR VISUAL DISCONTINUITIES AND FOUND ACCEPTABLE.
THEY COMPLY IN ALL RESPECTS WITH THE LATEST EDITION OF THE FOLLOWING SPECS:
ASTM A563 DH AND ASME B18.2.2, THREADS PER ASME B1.1 CLASS 2B UNLESS OTHERWISE SPECIFIED.

ASTM F2329, ASTM-A-153 CLASS C + LUBRICANT

COATING THICKNESS VALUES PROVIDED BY COATING SUPPLIER.

MANUFACTURED IN : CANADA

The steel was melted and rolled
in North America and is mercury and asbestos-free.

Daniel Guilbault
Quality Assurance Foreman

INFASCO
A division of Ifastgroupe LP 700 Ouellette, Marieville (Quebec) J3M 1P6
A Heico Company Tel.: (450) 658-8741 Fax: (450) 460-5496

FQ-019-2 Rev. 09 Revision date of test report: 2015-08-20 Page 1 of 1

Figure E-16. 1 in.-8 UNC (M24x3), 3½ in. (89 mm) Long Heavy Hex Head Bolt and Nut, Test Nos. ILOH4-1 to ILOH4-7, Sheet 4 of 4

INFASCO

MANUFACTURER'S LOT REFERENCE

This is to confirm that according to Infasco's database, Ifastgroupe 2004 L.P. and/or its divisions or subsidiaries have supplied the following lot number(s) to:

FASTENAL CO INC (WINONA MN)

201724620

in connection with purchase order **120297131** and part number **19371**

Figure E-17. 1 in.-8 UNC (M24x3), 2 in. (51 mm) Long Heavy Hex Head Bolt, Test Nos. ILOH4-1 to ILOH4-7, Sheet 1 of 4

INFASCO

SET NO.: **2017-24620**

ISO 9001 : 2008
ISO / IEC 17025
ISO 14001 : 2004

FASTENER TEST REPORT

(THIS DOCUMENT MAY ONLY BE REPRODUCED IN ITS ENTIRETY, WITH PRIOR WRITTEN APPROVAL BY THE INFASCO LABORATORY)

(THE INFASCO LABORATORY IS ACCREDITED BY THE CCN FOR THE TESTS LISTED AT WWW.CCN.CA)

COMPLIES WITH EN10204:2004 INSPECTION CERTIFICATE 3.1

DATE: 2017-03-09

DESCRIPTION	C A325-1+A563-DH NA UNC OS 1-8 X 2

BOLT A325-1 STRUCTURAL BOLT UNC FT HDG
MARKING : HOLLOW TRIANGLE & "A325T"

LOT NO.	MANUFACTURED BY	HARDNESS (ROCKWELL)	PROOF LOAD (LB)	TENSILE STRENGTH (LB)	
1701-50259 4072G	INFASCO	HRC 25.0 - HRC 34.0	MIN: 51,500	MIN: 72,700	
MEAN VALUE		30.4	PASS	87,100	

HEAT NO.	C %	Mn %	P %	S %	Si %				
10440690	0.39	1.01	0.008	0.010	0.22				

NUT HVY HEX NUT A563-DH FNA OS. UNC OS HDG
MARKING : TRIANGLE & "DH"

LOT NO.	MANUFACTURED BY	HARDNESS (ROCKWELL)	PROOF LOAD (LB)		
1503-52478 2950G	INFASCO	HRC 24.0 - HRC 38.0	MIN: 90,900		
MEAN VALUE		30.0	PASS		

HEAT NO.	C %	Mn %	P %	S %	Si %	Cu %	Ni %		
C114375	0.45	0.85	0.007	0.011	0.22	0.12	0.04		

HEAT CHEMICAL ANALYSIS PROVIDED BY STEEL SUPPLIER.
THE ASSEMBLY MEETS THE REQUIREMENTS OF ASTM A325
- NUTS LUBRICATED

THE ABOVE SET HAS BEEN ROTATIONAL CAPACITY TESTED WITH A RANDOM LOT OF GALVANIZED WASHERS TO VALIDATE THE LUBRICITY OF THE NUT. TEST RESULT: PASS

Isabelle Parent, Eng., M.A.Sc.
Quality Assurance Foreman

INFASCO
A division of Ifastgroupe LP
A Heico Company

700 Ouellette, Marieville (Quebec) J3M 1P6
Tel.: (450) 658-8741 Fax: (450) 460-5496

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Figure E-18. 1 in.-8 UNC (M24x3), 2 in. (51 mm) Long Heavy Hex Head Bolt, Test Nos. ILOH4-1 to ILOH4-7, Sheet 2 of 4

INFASCO

LOT NO.: 1701-50259
4072G

ISO 9001, ISO/TS16949
ISO / IEC 17025
ISO 14001

FASTENER TEST REPORT

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COMPLIES WITH EN10204:2004 INSPECTION CERTIFICATE 3.1

DATE 2017-01-31

DESCRIPTION AND MARKING: A325-1 STRUCTURAL BOLT UNC FT HDG
HOLLOW TRIANGLE & "A325T"
FULL THREAD WITH "T" MARKING

SIZE	GRADE	QUANTITY
1-8 X 2	1037MS	3,900

HEAT CHEMICAL ANALYSIS (provided by steel supplier)

HEAT NO.	C %	Mn %	P %	S %	Si %				
10440690	0.39	1.01	0.008	0.010	0.22				

METHOD	ASTM F606	ASTM F606			ASTM F606		ASTM E376
	PROOF LOAD (psi)	WEDGE TENSILE STRENGTH (psi)	SHEAR STRENGTH	SURFACE HARDNESS (HR 30N)	CORE HARDNESS (ROCKWELL)	MICRO HARDNESS	COATING THICKNESS (0.001 in)
SPEC. MIN.	85,000	120,000			HRC 25.0		2.00
SPEC. MAX:					HRC 34.0		
SAMPLE NO.1	86,000	144,000			HRC 29.5		3.46
NO.2	85,000	143,000			31.0		4.58
NO.3	85,000	144,000			30.5		3.48
NO.4					30.5		3.24
NO.5							3.99
NO.6							3.01
NO.7							5.52
NO.8							4.33
NO.9							4.41
NO.10							5.64
NO.11							4.16
NO.12							3.56
NO.13							4.30
NO.14							4.98
NO.15							4.47

THE ABOVE TESTED SAMPLES HAVE BEEN INSPECTED FOR VISUAL DISCONTINUITIES AND FOUND ACCEPTABLE.
THEY COMPLY IN ALL RESPECTS WITH THE LATEST EDITION OF THE FOLLOWING SPECS:
ASTM A325 TYPE 1, ASME B18.2.6, THREADS PER ASME B1.1 CLASS 2A UNLESS OTHERWISE SPECIFIED.
MEETS THE SURFACE DISCONTINUITIES REQUIREMENTS.
ASTM F2329, ASTM-A-153 Class C
THESE FASTENERS WERE OIL QUENCHED AND TEMPERED AT A TEMP. ABOVE 800°F.
NO BISMUTH, SELENIUM, TELLURIUM OR LEAD HAVE BEEN INTENTIONALLY ADDED

COATING THICKNESS VALUES PROVIDED BY COATING SUPPLIER.

MANUFACTURED IN: CANADA
The steel was melted and rolled
in North America and is mercury and asbestos-free.

Isabelle Parent, Eng., M.A.Sc.
Quality Assurance Foreman

INFASCO
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Figure E-19. 1 in.-8 UNC (M24x3), 2 in. (51 mm) Long Heavy Hex Head Bolt, Test Nos. ILOH4-1 to ILOH4-7, Sheet 3 of 4

INFASCO

LOT NO.: **1503-52478**
2950G

ISO 9001, ISO/TS16949
ISO / IEC 17025
ISO 14001

FASTENER TEST REPORT

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(THE INFASCO LABORATORY IS ACCREDITED BY THE CCN FOR THE TESTS LISTED AT WWW.CCN.CA)
COMPLIES WITH EN10204:2004 INSPECTION CERTIFICATE 3.1

DATE 2015-09-14

DESCRIPTION AND MARKING: HVY HEX NUT A563-DH FNA OS. UNC OS HDG+L
TRIANGLE & "DH"

SIZE	GRADE	QUANTITY
1-8 .024 OS	1046	103,000

HEAT CHEMICAL ANALYSIS (provided by steel supplier)

HEAT NO.	C %	Mn %	P %	S %	Si %	Cu %	Ni %		
C114375	0.45	0.85	0.007	0.011	0.22	0.12	0.04		

METHOD	ASTM F606				ASTM F606		ASTM E376
	PROOF LOAD (psi)	WEDGE TENSILE STRENGTH	SHEAR STRENGTH	SURFACE HARDNESS (HR 30N)	CORE HARDNESS (ROCKWELL)	MICRO HARDNESS	COATING THICKNESS (0.001 in)
SPEC. MIN.	150,000				HRC 24.0		2.00
SPEC. MAX:					HRC 38.0		
SAMPLE NO.1	154,000				HRC 30.2		2.92
NO.2	155,000				31.0		3.27
NO.3	156,000				28.6		3.04
NO.4	154,000				29.9		3.11
NO.5	154,000				30.1		3.33
NO.6							4.19
NO.7							3.03
NO.8							2.98
NO.9							3.28
NO.10							3.12
NO.11							3.48
NO.12							3.65
NO.13							2.69
NO.14							3.18
NO.15							2.97

THE ABOVE TESTED SAMPLES HAVE BEEN INSPECTED FOR VISUAL DISCONTINUITIES AND FOUND ACCEPTABLE.
THEY COMPLY IN ALL RESPECTS WITH THE LATEST EDITION OF THE FOLLOWING SPECS:
ASTM A563 DH AND ASME B18.2.2, THREADS PER ASME B1.1 CLASS 2B UNLESS OTHERWISE SPECIFIED.

ASTM F2329, ASTM-A-153 CLASS C + LUBRICANT

COATING THICKNESS VALUES PROVIDED BY COATING SUPPLIER.

MANUFACTURED IN : CANADA
The steel was melted and rolled
in North America and is mercury and asbestos-free.

Daniel Guilbault
Quality Assurance Foreman

INFASCO
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A Heico Company
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Figure E-20. 1 in.-8 UNC (M24x3), 2 in. (51 mm) Long Heavy Hex Head Bolt, Test Nos. ILOH4-1 to ILOH4-7, Sheet 4 of

STEEL AND PIPE SUPPLY
SPS Coil Processing Tulsa
5275 Bird Creek Ave.
Port of Catoosa, OK 74015

METALLURGICAL TEST REPORT

PAGE 1 of 1
DATE 05/14/2018
TIME 08:51:25
USER WF-BATCH

SOLD TO
66031-1127

SHIP TO
13716
Kansas City Warehouse
401 New Century Parkway
NEW CENTURY KS

Order	Material No.	Description	Quantity	Weight	Customer Part	Customer PO	Ship Date
40308183-0010	70872144TM	1/4 72 X 144 A36 TEMPERPASS STPMLPL	3	2,205.360			05/11/2018

Chemical Analysis

Heat No. A8C270 Vendor SSAB - MONTPELIER WORKS DOMESTIC Mill SSAB - MONTPELIER WORKS **Melted and Manufactured in the USA**
Produced from Coil

Carbon	Manganese	Phosphorus	Sulphur	Silicon	Nickel	Chromium	Molybdenum	Boron	Copper	Aluminum	Titanium	Vanadium	Columbium	Nitrogen	Tin
0.1600	0.8500	0.0090	0.0040	0.0300	0.1200	0.1100	0.0300	0.0000	0.2700	0.0300	0.0070	0.0030	0.0020	0.0075	0.0000

Mechanical / Physical Properties

Mill Coil No. A8C2700664

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
77700.000	59700.000	25.40			0	NA			
71100.000	54100.000	29.00			0	NA			
73400.000	54100.000	29.20			0	NA			
72000.000	53700.000	27.00			0	NA			

Batch 0005292009 3 EA 2,205.360 LB Batch 0005292007 13 EA 9,556.560 LB

THE CHEMICAL, PHYSICAL, OR MECHANICAL TESTS REPORTED ABOVE ACCURATELY REFLECT INFORMATION AS CONTAINED IN THE RECORDS OF THE CORPORATION.
The material is in compliance with EN 10204 Section 4.1 Inspection Certificate Type 3.1

Figure E-21. 1-in. (25-mm) Square Washer, Test Nos. ILOH4-1 to ILOH4-7

END OF DOCUMENT