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DEVELOPMENT OF A MASH TEST LEVEL 4 STEEL, SIDE-MOUNTED, BEAM-AND-POST, BRIDGE RAIL

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16. Abstract

A new steel, side-mounted, beam-and-post, bridge rail was designed, crash-tested, and evaluated according to safety performance guidelines included in the American Association of State Highway and Transportation Officials *Manual for Assessing Safety Hardware, Second Edition* (MASH 2016) for Test Level 4 (TL-4). The new bridge rail system was designed to be compatible with multiple concrete bridge decks utilized by the States of Illinois and Ohio. Bridge rail configurations were designed and optimized based on weight per foot, constructability, and safety. Post-to-rail and rail-to-rail connections were designed for the new bridge rail. Several concepts for these connections were configured, and after discussion with representatives from Illinois and Ohio Departments of Transportation, a preferred concept was selected for full-scale crash testing with a single-unit truck (SUT), a pickup truck, and a small car. The new bridge rail consisted of three tubular steel rail elements supported by W6x15 (W150x22.5) steel posts mounted to the exterior, vertical edge of the concrete deck and spaced at 8 ft (2.4 m) on centers. The top rail element was an HSS 12-in. x 4-in. x ¼-in. (HSS 304.8-mm x 101.6-mm x 6.4-mm) and the lower two rail elements were HSS 8-in. x 6-in. x ¼-in. (HSS 203.2-mm x 152.4-mm x 6.4-mm). The centerline heights of the rail elements were 37 in. (940 mm), 28 in. (711 mm), and 16 in. (406 mm) above the surface of the deck for the top, middle, and bottom rails, respectively. Four MASH 2016 TL-4 crash tests were performed on the new bridge rail, which successfully contained and redirected each of the MASH 2016 TL-4 vehicles. All occupant risk measures and evaluation criteria were within MASH 2016 limits. In the initial run of test designation no. 4-12, test no. STBR-1 with the SUT, the impact severity did not meet the minimum limit of 142.0 kip-ft (180.6 kJ). Thus, test designation no. 4-12 was re-run in test no. STBR-4, and the results met all MASH 2016 impact safety criteria, ensuring that the new bridge rail meets MASH 2016 TL-4 standards.

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DISCLAIMER STATEMENT

This material is based upon work supported by the Federal Highway Administration, U.S. Department of Transportation, Illinois Department of Transportation, and the Ohio Department of Transportation under TPF-5(193) Supplement #116. The contents of this report reflect the views and opinions of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the University of Nebraska-Lincoln, the Illinois Department of Transportation, the Ohio Department of Transportation, nor the Federal Highway Administration, U.S. Department of Transportation. This report does not constitute a standard, specification, or regulation. Trade or manufacturers' names, which may appear in this report, are cited only because they are considered essential to the objectives of the report. The United States (U.S.) government and the States of Illinois and Ohio do not endorse products or manufacturers.

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.

INDEPENDENT APPROVING AUTHORITY

The Independent Approving Authority for the data contained herein was Dr. Mojdeh Asadollahi Pajouh, Research Assistant Professor.

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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

Bridge rails have been used to contain and safely redirect errant vehicles and prevent motorists from traveling beyond the deck edge, where water hazards and/or vertical drop-offs are located. The majority of bridge rails consist of reinforced concrete parapets or steel beam-and-post systems, often mounted to the top of bridge decks. The use of top-mounted bridge rails requires that bridge engineers increase the overall width of the bridge structure in order to provide the necessary roadway and shoulder widths. Many steel beam-and-post bridge rails can also be side-mounted on the outer vertical edges of the bridge deck, which minimizes the lateral extension of bridge rails above the deck structure and maximizes the traversable deck width. An example of a steel, side-mounted, beam-and-post, bridge rail is shown in Figure 1.

Figure 1. Steel, Side-Mounted, Beam-and-Post, Bridge Rail [1]

Over the past several decades, the Illinois and Ohio Departments of Transportation (DOTs) have often installed steel, side-mounted, beam-and-post, bridge rails to shield motorists from striking hazardous vertical drop-offs associated with elevated bridge superstructures. Steel beam-and-post bridge rails often consist of multiple square or rectangular HSS steel tube rails attached to the front flanges of I-shaped steel posts. Many of these bridge rails have been configured without a lower curb to allow water to drain off the outer edges of the bridge deck. For many bridge rails, the front faces of the rails are positioned to be vertically flush with the exterior deck edge, which eliminated rail extension above the bridge deck and reduced overall deck width.

More recently, bridge railings have been crash-tested and evaluated according to impact safety standards, which have evolved over the last 50 years. State Departments of Transportation have often sought system eligibility and federal reimbursement from the Federal Highway Administration for bridge rails utilized on the National Highway System (NHS). Although system eligibility and crash testing may not be required for all bridge railings found along local roads and non-NHS highways, state DOTs and other government agencies have been proactive in determining the crashworthiness of most bridge railing systems and using systems with acceptable safety performance.

In 1993, the Illinois DOT had a two-rail, beam-and-post, bridge rail subjected to full-scale crash testing, specifically the Illinois Side-Mounted, Bridge Railing [2-3]. The Illinois Side-Mounted, Bridge Rail is shown in Figure 2. This bridge rail consisted of W6x25 (W150x 37.1) steel posts spaced at 6 ft – 3 in. (1,905 mm) centers, which supported a TS 8-in. x 4-in. x 5/16 -in. (203-mm x 102-mm x 8-mm) top rail element and a TS 6-in. x 4-in. x ¼-in. (152-mm x 102-mm x 6.4-mm) bottom rail element. Both rails were mounted to the front flange of the steel posts. Texas A&M Transportation Institute (TTI) researchers successfully crash tested this bridge rail using crash testing criteria published in the American Association of State Highway and Transportation Officials (AASHTO) *1989 Guide Specifications for Bridge Railings* [2-4]. The Illinois Side-Mounted, Bridge Railing was crash tested under Performance Level 2 (PL-2), which involved an 1,800-lb (816-kg) passenger car with an impact speed of 60.0 mph (96.6 km/h) and an impact angle of 20 degrees, a 5,400-lb (2,449-kg) pickup truck with an impact speed of 60.0 mph (96.6 km/h) and an impact angle of 20 degrees, and a 18,000-lb (8,167-kg) single-unit truck (SUT) with an impact speed of 50.0 mph (80.5 km/h) and an impact angle of 15 degrees. All three crash tests met the required evaluation criteria [2-4]. AASHTO PL-2 criteria is considered equivalent to Test Level 4 (TL-4) safety criteria found in the *National Cooperative Highway Research Program* (NCHRP) *Report 350* [5].

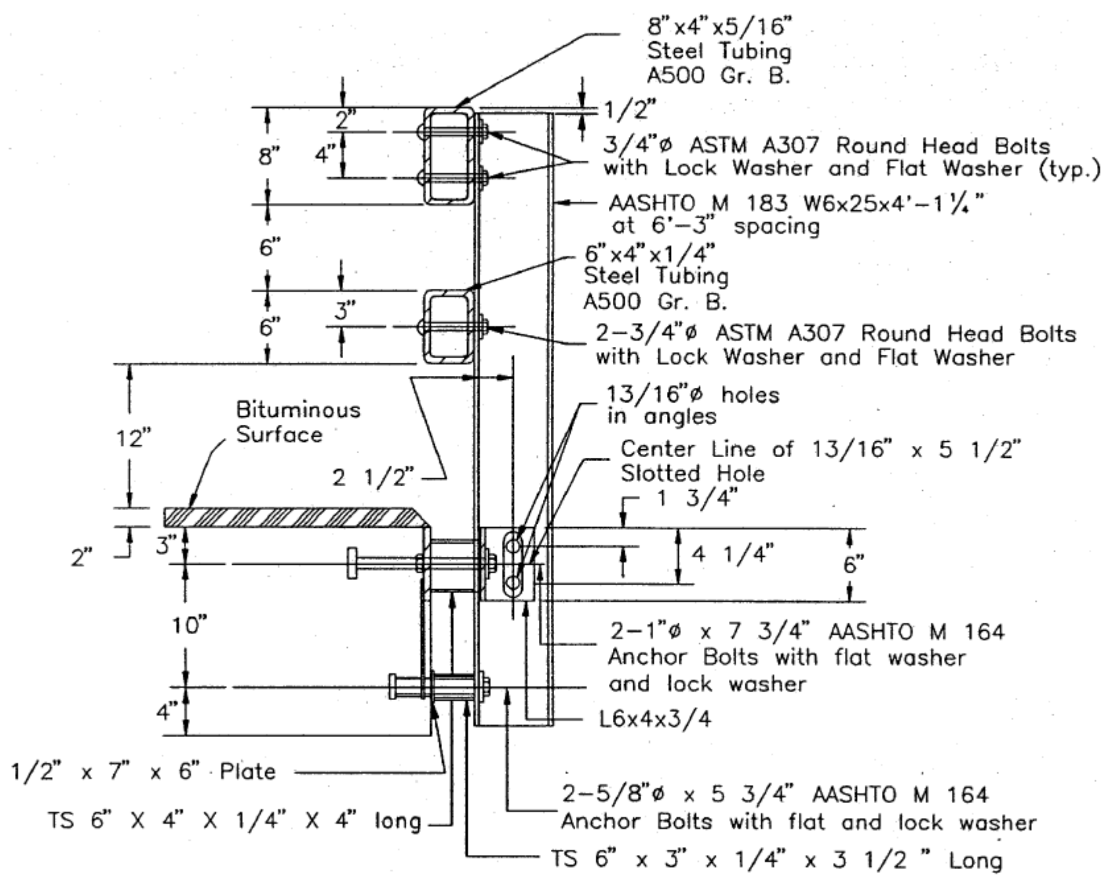


Figure 2. Illinois Side-Mounted, Bridge Rail [2]

In 1999, Ohio DOT started to implement a similar two-rail, beam-and-post, bridge rail [6]. The Ohio Twin Steel-Tube, Bridge Rail is shown in Figure 3. This bridge rail, the Ohio Twin Steel-Tube, Bridge Rail, adopted the W6x25 (W150x37.1) steel posts, and the TS 8-in. x 4-in. x $^{5}/_{16}$-in. (203-mm x 102-mm x 8-mm) top steel rail from the Illinois Side-Mounted, Bridge Rail was used for both rail sections. With the larger and stronger lower rail, the Ohio Twin Steel-Tube, Bridge Rail was deemed to be acceptable under the Test Level 4 (TL-4) safety criteria found in NCHRP Report 350 without further testing [5,7].

In 2009, AASHTO published a new guideline for crash testing and evaluating longitudinal barriers, such as bridge rails, specifically the *Manual for Assessing Safety Hardware* (MASH) [8]. MASH safety criteria supersedes those criteria published in NCHRP Report 350 for the crash testing and evaluation of roadside safety hardware devices. The second edition to MASH was published in 2016 [9].

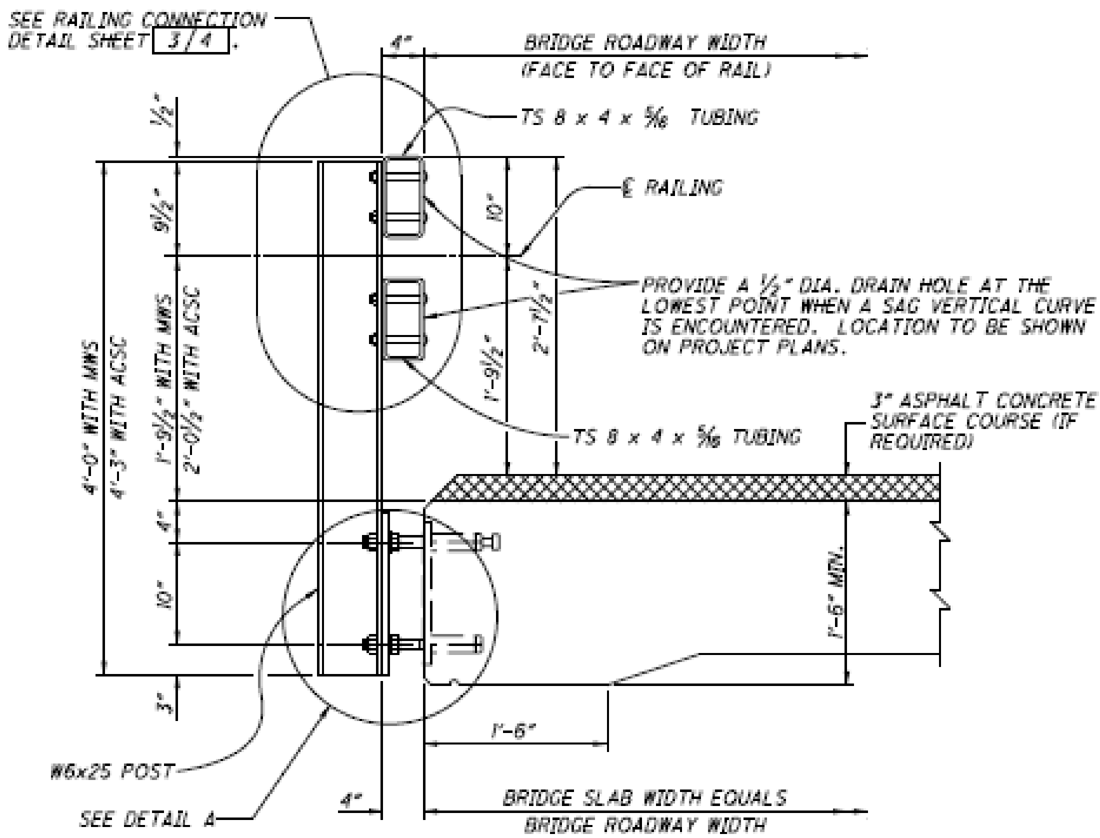


Figure 3. Ohio Twin Steel-Tube, Bridge Rail [6]

In an effort to encourage state DOTs to advance hardware designs, the Federal Highway Administration (FHWA) and AASHTO established a MASH implementation policy, which included sunset dates for existing roadside safety hardware based on hardware category [10]. The implementation policy indicated that all modifications to NCHRP Report 350 crash-tested devices required testing under MASH in order to receive a Federal-aid eligibility letter from FHWA. For road projects involving bridge rails, transitions, and other longitudinal barriers installed on the NHS after December 31, 2019, only safety hardware evaluated according to MASH 2016 would be allowed for use on new permanent installations or as full replacements. Therefore, government agencies must use MASH 2016 crash-tested hardware on all projects after December 31, 2019.

Through initial discussions between the Illinois DOT, Ohio DOT, and the Midwest Roadside Safety Facility (MwRSF), a prototype concept was created for a steel, side-mounted, beam-and-post, bridge rail that satisfies MASH 2016 TL-4 impact safety standards. The Illinois and Ohio MASH 2016 TL-4 Prototype Bridge Rail Concept was modified throughout the discussion process. The Illinois and Ohio MASH 2016 TL-4 Prototype Bridge Rail Concept is shown in Figure 4. The bridge rail concept consisted of three longitudinal steel tube rails attached to W6x15 (W150x22.5) steel posts, which are weaker than the W6x25 (W150x37.1) steel posts utilized in the two Illinois and Ohio bridge rails noted above, as well as in many MASH TL-4 steel, beam-and-post, bridge rails. The W6x15 (W150x 22.5) steel posts were preferred to reduce high loading to the bridge deck and to mitigate bridge deck damage, while deforming after vehicle impact and absorbing much of the vehicle’s kinetic energy. The steel posts are mounted to the outer vertical edge of the bridge deck without a curb and with the front faces of the tubular rails positioned vertically flush with the exterior deck edge to eliminate rail extension above the bridge deck. Additionally, the Illinois and Ohio MASH 2016 TL-4 Prototype Bridge Rail Concept has an overall height of 39 in. (991 mm) above the bridge deck to meet the minimum 36-in. (914-mm) height for MASH 2016 TL-4 barriers after a future 3-in. roadway overlay is placed. Furthermore, it was anticipated that Illinois and Ohio bridge deck types would differ. Therefore, the bridge rail system would need to be adaptable to multiple bridge deck configurations utilized by the States of Illinois and Ohio.

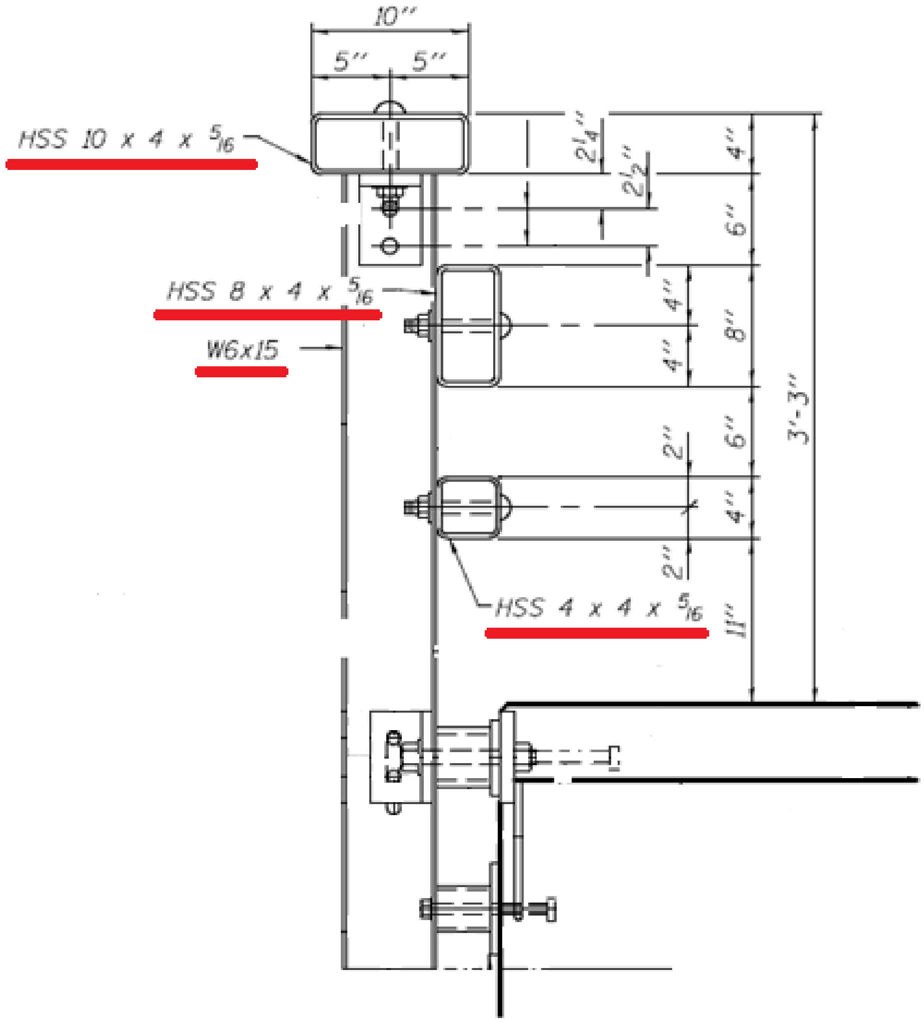


Figure 4. Illinois and Ohio MASH 2016 TL-4 Prototype Bridge Rail Concept

1.2 Objective

The objective of this project was to develop and evaluate a new steel, side-mounted, beam-and-post, bridge rail according to the MASH 2016 TL-4 safety performance criteria. The new steel, side-mounted, beam-and-post, bridge rail was designed to be adaptable to multiple bridge deck configurations utilized by the States of Illinois and Ohio. The system was configured to the minimum 36-in. (914-mm) height for MASH 2016 TL-4 barriers after a future 3-in. (76-mm) roadway overlay has been placed. The front faces of the steel rail tubes were approximately aligned with the exterior vertical edge of the concrete deck to eliminate rail extension above the bridge deck. No curb was utilized. It should be noted that W6x15 (W150x22.5) steel posts were used in lieu of W6x25 (W150x37.1) steel posts to lower the impact loads transferred to the deck, and consequently, reduce the potential for bridge deck damage. Further, adequate post-to-rail and rail-to-rail connection designs were provided.

Additionally, a transition was to be developed to safely connect the bridge rail to adjacent crashworthy thrie beam approach guardrail transition systems [11]. Both the bridge rail and the transition were to be subjected to full-scale vehicle crash testing, as required by MASH 2016. The special transition was to be tested and evaluated according to MASH 2016 TL-3 safety performance criteria, while the bridge rail itself was to be tested and evaluated according to MASH 2016 TL-4 safety performance criteria. Final guidance and implementation of the bridge rail will be provided separately [12].

1.3 Scope

The development of the MASH 2016 TL-4 bridge rail and associated special transition were to be conducted through a two-phase research effort. Phase I focused on the development and testing of the steel, side-mounted, beam-and-post, bridge rail and the corresponding post-to-deck anchorage connection and is discussed within this report. Phase II consisted of the design, simulation, and testing of the special transition [11]. The research effort described in this report focuses only on the design and full-scale crash testing of the new steel, side-mounted, beam-and-post, bridge rail, as noted in Phase I.

Phase I began with a literature review of previous crashworthy steel, beam-and-post, bridge rails that were tested and evaluated using different safety performance standards. The literature review included side-mounted and top-mounted PL-2, PL-3, TL-3, TL-4, and TL-5 bridge rails to study the contribution of posts, rails, post-to-rail connections, and rail-to-rail connections to the crashworthiness of the bridge rail system. Several design considerations, such as bridge rail geometric requirements, design impact loads, and critical deck configurations, were studied to limit the number of variables for the locations and sizes of the three steel rails of the Illinois and Ohio Bridge Rail Concept. Bridge rail design methodologies were investigated to identify a suitable design process for the new bridge rail. Bridge railing configurations that mitigate the potential for vehicle snag while providing adequate strength were developed. Post-to-rail and rail-to-rail connection details were provided to the sponsors for review and comment. Subsequently, final design details were prepared for the new bridge rail.

Although described in greater detail in another Phase I report, dynamic component testing was conducted to evaluate the performance of several post-to-deck connection concepts [13-14]. Six dynamic component tests were performed on individual posts mounted to the side of a pre-stressed, prefabricated, concrete box beam to evaluate the impact behavior of posts, anchorages, and the deck, as well as to identify any damage that may be likely to occur during vehicular impact events. Only a very brief summary is provided herein on this significant effort. Once mounted to a simulated bridge deck or beam, the steel posts were laterally impacted with a bogie vehicle traveling approximately 25 mph (40 km/h). Since the posts and post-to-deck connection hardware may differ between different deck types, the component tests were also utilized to identify the critical configuration for use in the full-scale vehicle crash testing program.

Finally, a steel, side-mounted, beam-and-post, bridge railing system was selected, configured with CAD details, constructed, and subjected to four full-scale vehicle crash tests under MASH 2016 TL-4 impact safety standards to evaluate the safety performance of the bridge rail. Complete conclusions, recommendations, and implementation guidance are provided in summary report [12].

2 LITERATURE REVIEW

2.1 Overview

The first task of the research project consisted of a literature search in order to review and gain knowledge on (1) historical and current crash testing criteria, (2) relevant steel, side-mounted and top-mounted, beam-and-post, bridge rails, (3) prior NCHRP Report 350 TL-4 [5] as well as current MASH TL-4 [8-9] lateral design loading for barriers, and (4) prior and current NCHRP and MASH TL-4 minimum barrier heights. Few steel, beam-and-post, bridge rails have been tested to MASH TL-4 safety performance criteria. Therefore, it was also necessary to review relevant bridge rails that were crash tested and evaluated using safety performance criteria from AASHTO's *Guide Specifications for Bridge Railings* [4] as well as NCHRP Report 230 [15] and 350 [5]. Moreover, studies relevant to lateral and vertical design impact loads and minimum bridge rail heights corresponding to MASH TL-4 test conditions were reviewed.

2.2 Historical and Current Crash Testing Criteria, Matrices, and Conditions

Over the years, numerous documents have been published to provide guidance on the crash testing and evaluation of roadside safety hardware. In these roadside safety guidelines, test impact conditions were provided, including critical impact points, vehicle types, vehicle weights, impact speeds, and impact angles. Test impact conditions within MASH represent the worst practical conditions associated with real-world collisions.

2.2.1 NCHRP Report No. 230

In 1981, NCHRP published Report No. 230, one of the early safety standards that was widely used for the testing and evaluation of roadside barriers, such as bridge rails [15]. For NCHRP Report No. 230, the three primary crash tests for evaluating the length-of-need for longitudinal barriers corresponded to test designation nos. 10, 11, and 12, which were a 4,500-lb (2,041-kg) large sedan, a 2,250-lb (1,021-kg) sub-compact sedan, and a 1,800-lb (816-kg) mini-compact sedan, respectively. The NCHRP Report No. 230 primary crash test conditions for longitudinal barriers are shown in Table 1.

Table 1. NCHRP Report No. 230 Primary Crash Test Conditions for Longitudinal Barriers [15]

Test Designation	Vehicle Type	Impact Speed (mph)	Impact Angle (deg)	Target Impact Severity (kip-ft)
10	4,500S	60	25	97
11	2,250S	60	15	18
12	1,800S	60	15	14

NCHRP Report No. 230 also provided several supplementary crash test conditions for evaluating the length-of-need of longitudinal barriers, including passenger vehicles as well as heavy vehicles. These heavy vehicles included a variety of buses (P), tractor/van truck trailers (A), and tractor/fluid tanker trucks (F). The supplementary test conditions were divided into three multiple service levels (MSL-1, MSL-2, and MSL-3). Supplementary NCHRP Report No. 230 multiple service levels are shown in Table 2.

Table 2. NCHRP Report No. 230 Supplementary Crash Test Conditions for Longitudinal Barriers [15]

Test Designation	Multiple Service Level	Vehicle Type	Impact Speed (mph)	Impact Angle (deg)	Target Impact Severity (kip-ft)
S13	MSL-1	1,800S	60	20	25
S14	MSL-1	4,500S	60	15	36
S15	MSL-3	40,000P	60	15	237
S16	MSL-1	20,000P	45	7	14
S17	MSL-2	20,000P	50	15	77
S18	MSL-2	20,000P	60	15	111
S19	MSL-3	32,000P	60	15	97
S20	MSL-3	80,000A	50	15	(t)
S21	MSL-3	80,000F	50	15	(t)

(t) - Not appropriate for articulated vehicles

2.2.2 AASHTO Guide Specifications for Bridge Railings

In 1989, AASHTO published *Guide Specifications for Bridge Railings* to address the testing and evaluation of bridge railings [4]. This publication contained three crash test performance levels (PLs) for roadside safety hardware: PL-1, PL-2, and PL-3, which are shown in Table 3.

Table 3. AASHTO Guide Specifications for Bridge Railings Testing Conditions [4]

Performance Level	Vehicle Type	Vehicle Weight (lbs)	Nominal Speed (mi/h)	Nominal Angle (deg)	Impact Severity (kip-ft)
PL-1	Small Automobile	1,800	50	20	17.6
	Pickup Truck	5,400	45	20	42.8
PL-2	Small Automobile	1,800	60	20	25.3
	Pickup Truck	5,400	60	20	76.0
	Single-Unit Truck	18,000	50	15	100.8
PL-3	Small Automobile	1,800	60	20	25.3
	Pickup Truck	5,400	60	20	76.0
	Van-Type Tractor Trailer	50,000	50	15	279.9

2.2.3 NCHRP Report 350

In 1993, NCHRP Report 350 [5] was published, superseding the previous crash testing guidelines from AASHTO, specifically the *Guide Specifications for Bridge Railings*. Six different test levels (TLs) were provided to develop a range of roadside safety hardware (i.e., bridge rails) that could be used for different purposes. Test Level 1 was used to evaluate features found in many work zones as well as along low-volume, low-speed, local streets and highways. Test Level 2 was used to evaluate features found on most local and collector roads and many work zones. Test Level 3 was used as the basic level for devices found on high-speed arterial highways. Test Levels 4 through 6 were used for scenarios with higher volumes of trucks and heavy vehicles as well as situations with consequences of penetration beyond the longitudinal barrier. Test Levels 1 through 3 were focused on the impact performance of passenger vehicles varying by impact speed as the test level increased. Test Levels 4 through 6 included the previous passenger vehicles but additionally incorporated various sizes of trucks. Specifically, Test Level 4 involved a 1,808-lb (820-kg) small car impacting the barrier at 62.1 mph (100 km/h) at an impact angle of 20 degrees, a 4,409-lb (2,000-kg) pickup truck impacting the barrier at 62.1 mph (100 km/h) at an impact angle of 25 degrees, and a 17,637-lb (8,000-kg) SUT impacting the barrier at 49.7 mph (80 km/h) at an impact angle of 15 degrees. Test Levels 5 and 6 involved the same passenger vehicles as used in Test Levels 1 through 4, and a 79,366-lb (36,000-kg) van-type tractor trailer and a 79,366-lb (36,000-kg) tractor-tank trailer, respectively. The NCHRP Report 350 testing conditions for the six test levels are shown in Table 4.

Table 4. NCHRP Report 350 Test Impact Conditions [5]

Test Level	Vehicle Type	Vehicle Mass kg (lbs)	Impact Speed km/h (mi/h)	Nominal Angle (deg)	Impact Severity kJ (kip-ft)
1	820C	820 (1,808)	50 (31.1)	20	9.3 (6.8)
	2000P	2,000 (4,409)	50 (31.1)	25	34.5 (25.4)
2	820C	820 (1,808)	70 (43.5)	20	18.1 (13.4)
	2000P	2,000 (4,409)	70 (43.5)	25	67.5 (49.8)
3	820C	820 (1,808)	100 (62.1)	20	37.0 (27.3)
	2000P	2,000 (4,409)	100 (62.1)	25	137.8 (101.6)
4	820C	820 (1,808)	100 (62.1)	20	37.0 (27.3)
	2000P	2,000 (4,409)	100 (62.1)	25	137.8 (101.6)
	8000S	8,000 (17,637)	80 (49.7)	15	132.3 (97.6)
5	820C	820 (1,808)	100 (62.1)	20	37.0 (27.3)
	2000P	2,000 (4,409)	100 (62.1)	25	137.8 (101.6)
	36000V	36,000 (79,366)	80 (49.7)	15	595.4 (439.2)
6	820C	820 (1,808)	100 (62.1)	20	37.0 (27.3)
	2000P	2,000 (4,409)	100 (62.1)	25	137.8 (101.6)
	36000T	36,000 (79,366)	80 (49.7)	15	595.4 (439.2)

2.2.4 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 [16]. No test level equivalencies have been determined for MASH test criteria.

The equivalencies set forth by the FHWA are summarized in Table 5. 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 5. FHWA Crash Test Equivalencies [16]

Bridge Railing Testing Criteria	Testing Level Equivalencies					
NCHRP Report 350 [5]	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
NCHRP Report 230 [15]	N/A	MSL-1 MSL-2	N/A	N/A	N/A	N/A
AASHTO Guide Spec. [4]	N/A	PL-1	N/A	PL-2	PL-3	N/A

N/A = No testing level equivalencies exist amongst standards

2.2.5 Manual for Assessing Safety Hardware (MASH)

In 2008, MwRSF performed NCHRP Project No. 22-14(2) *Improvement of Procedures for the Safety-Performance Evaluation of Roadside Features* [17], which updated the safety performance criteria found in NCHRP Report 350 [5]. The Project No. 22-14(2) research effort culminated in the 2009 *Manual for Assessing Safety Hardware* (MASH) [8] to supersede NCHRP Report 350 [5]. MASH included updated test vehicles to replicate those being produced recently. Test impact conditions were also modified to correct inconsistencies in impact severities. In 2016, the *AASHTO Technical Committee on Roadside Safety* updated the MASH 2009 safety performance guidelines, which added test matrices for cable barriers placed in sloped medians [9]. The changes to the test impact conditions from NCHRP Report 350 to MASH involved several vehicle weight modifications, including a small car increase from 1,808 lb (820 kg) to 2,420 lb (1,100 kg), a pickup truck increase from 4,409 lb (2,000 kg) to 5,000 lb (2,268 kg), and a SUT from 17,637 lb (8,000 kg) to 22,046 lb (10,000 kg). The TL-4 impact speed of the SUT increased from 50 mph (80 km/h) to 56 mph (90 km/h), and the impact angle of the small car increased from 20 degrees to 25 degrees as well.

The MASH testing conditions for the six test levels are shown in Table 6. As shown therein, the MASH TL-4 testing and evaluation criteria for longitudinal barriers consists of three full-scale vehicle crash tests (test nos. 4-10, 4-11, and 4-12). Crash test nos. 4-10 and 4-11 involve the 2,425-lb (1,100-kg) small car and 5,000-lb (2,268-kg) pickup truck, both impacting the barrier system at a speed of 62 mph (100.0 km/h) and an impact angle of 25 degrees, respectively. Test designation no. 4-12 involves the 22,046-lb (10,000-kg) SUT impacting the barrier system at a speed of 56 mph (90.0 km/h) and angle of 15 degrees.

Table 6. MASH 2016 Crash Test Conditions for Longitudinal Barriers [9]

Test Level	Vehicle Type	Vehicle Mass lbs (kg)	Impact Speed mi/h (km/h)	Nominal Angle (deg)	Impact Severity kip-ft (kJ)	Evaluation Criteria
1	1100C	2,425 (1,100)	31 (50.0)	25	14.0 (18.9)	A,D,F,H,I
	2270P	5,000 (2,268)	31 (50.0)	25	28.8 (39.1)	A,D,F,H,I
2	1100C	2,425 (1,100)	44 (70.0)	25	27.4 (37.1)	A,D,F,H,I
	2270P	5,000 (2,268)	44 (70.0)	25	56.5 (76.6)	A,D,F,H,I
3	1100C	2,425 (1,100)	62 (100.0)	25	55.9 (75.8)	A,D,F,H,I
	2270P	5,000 (2,268)	62 (100.0)	25	115.4 (156.4)	A,D,F,H,I
4	1100C	2,425 (1,100)	62 (100.0)	25	55.9 (75.8)	A,D,F,H,I
	2270P	5,000 (2,268)	62 (100.0)	25	115.4 (156.4)	A,D,F,H,I
	10000S	22,046 (10,000)	56 (90.0)	15	154.4 (209.3)	A,D,G
5	1100C	2,425 (1,100)	62 (100.0)	25	55.9 (75.8)	A,D,F,H,I
	2270P	5,000 (2,268)	62 (100.0)	25	115.4 (156.4)	A,D,F,H,I
	36000V	79,336 (36,000)	50 (80.0)	15	439.2 (595.4)	A,D,G
6	1100C	2,425 (1,100)	62 (100.0)	25	55.9 (75.8)	A,D,F,H,I
	2270P	5,000 (2,268)	62 (100.0)	25	115.4 (156.4)	A,D,F,H,I
	36000T	79,336 (36,000)	50 (80.0)	15	439.2 (595.4)	A,D,G

MASH 2016 evaluation criteria for full-scale crash tests is based on three main areas: (1) structural adequacy; (2) occupant risk; and (3) vehicle trajectory after impact. Specific details for the MASH 2016 evaluation criteria are provided in Table 7.

The evaluation of the structural adequacy determines the ability of the bridge rail to contain and redirect errant vehicles. Structural adequacy of roadside hardware, in general, consists of the barrier’s ability to contain and properly redirect impacting vehicles based on its strength and height. If a barrier is not strong enough, the impact vehicle can penetrate it, and if the bridge rail is not tall enough, the vehicle can override it or roll over.

Occupant risk evaluates the level of risk to the occupants of the impacting vehicle, which is required for passenger vehicles and optional for heavier vehicles, such as MASH 2016 TL-4 SUTs. The Post-Impact Head Deceleration (PHD), the Theoretical Head Impact Velocity (THIV), and the Acceleration Severity Index (ASI) are calculated and reported on the corresponding test summary sheet. Supplementary information of PHD, THIV, and ASI is also provided in MASH 2016. The vehicle trajectory after impact is evaluated as the vehicle remains upright during and after collision. For this criterion, the maximum roll and pitch angles are not to exceed 75 degrees.

Table 7. MASH 2016 Evaluation Criteria for Longitudinal Barriers [9]

<table>
<tr><td>Structural Adequacy</td><td>A.</td><td colspan="3">Test article should contain and redirect the vehicle or bring the vehicle to a controlled stop; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.</td></tr>
<tr><td rowspan="12">Occupant Risk</td><td>B.</td><td colspan="3">Detached elements, fragments or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformations of, or intrusions into, the occupant compartment should not exceed limits set forth in Section 5.3 and Appendix E of MASH.</td></tr>
<tr><td>F.</td><td colspan="3">The vehicle should remain upright during and after the collision. The maximum roll and pitch angles are not to exceed 75 degrees.</td></tr>
<tr><td>G.</td><td colspan="3">It is preferable, although not essential, that the vehicle remain upright during and after collision</td></tr>
<tr><td rowspan="4">H.</td><td colspan="3">Occupant Impact Velocity (OIV) (see Appendix A, Section A5.3 of MASH for calculation procedure) should satisfy the following limits:</td></tr>
<tr><td colspan="3">Occupant Impact Velocity Limits</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>30 ft/s
(9.1 m/s)</td><td>40 ft/s
(12.2 m/s)</td></tr>
<tr><td rowspan="4">I.</td><td colspan="3">The Occupant Ridedown Acceleration (ORA) (see Appendix A, Section A5.3 of MASH for calculation procedure) should satisfy the following limits:</td></tr>
<tr><td colspan="3">Occupant Impact Velocity Limits</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>15.0 g’s</td><td>20.49 g’s</td></tr>
</table>

2.2.6 Impact Severity

The severity of an impact event is normally measured in terms of impact severity (IS) for crash tests involving longitudinal barriers [8-9]. Impact severity indicates the portion of the vehicle’s kinetic energy that is imparted perpendicular to the bridge rail’s longitudinal axis. Impact severity is found from the vehicle mass, impact velocity, and the impact angle. MASH 2016 provides an equation to calculate the impact severity (IS) for each test impact condition which is defined in Equation 1.

$$IS = \frac{1}{2}m(v\ sin\theta)^2 \qquad (1)$$

where:

m = vehicle inertial mass (kg)
v = impact velocity (m/s)
θ = impact angle (deg)

Using the test conditions, the impact severity for MASH TL-4 crashes are higher than those provided in NCHRP Report 350 TL-4 crashes. For the three test conditions, the impact severity increased 105 percent for the small car, 14 percent for the pickup truck, and 58 percent for the SUT. These increases in impact severity for the three test conditions could be useful when examining lateral impact forces imparted to longitudinal barriers, subjected to both safety performance guidelines with barriers of similar stiffness, strength, and deformation behavior. For this specific scenario, one may expect proportional increases in lateral loading for corresponding increases in impact severity.

2.3 Steel, Side-Mounted, Beam-and-Post, Bridge Rails

For this research effort, a review of relevant steel, beam-and-post, bridge rails subjected to full-scale vehicle crash testing was performed. The literature review emphasized details and information pertaining to post and rail sections, post spacing, overall system heights, post-to rail connections, rail-to-rail connections, system deflections, vehicle impact performance, crash testing criteria conditions, design load, lateral barrier capacity, and overall crashworthiness of bridge rail systems.

2.3.1 Illinois Side-Mounted Bridge Rail

The Illinois Side-Mounted Bridge Rail consisted of two tubular steel tubes supported by a W6x25 (W150x37) posts spaced at 6 ft - 3 in. (1.91 m) centers, which were side-mounted to the edge of the reinforced concrete bridge deck [2-3]. The Illinois Side-Mounted Bridge Rail is shown in Figure 5. The top rail element consisted of a TS 8-in. x 4-in. x $^{5}/_{16}$-in. (TS 203-mm x 102-mm x 7.9-mm) steel tube attached to the post with two staggered, horizontal ¾-in. (19-mm) diameter ASTM A307 round head bolts. The bottom rail element consisted of a TS 6-in. x 4-in. x ¼-in. (TS 152-mm x 102-mm x 6.4mm) steel tube attached to the post with two horizontal ¾-in. (19-mm) diameter ASTM A307 round head bolts. The overall height of the bridge rail was 32 in. (813 mm) from the top of the upper rail to the bridge deck overlay.

In 1997, the Illinois Side-Mounted Bridge Rail was successfully crash-tested with a small automobile, a pickup truck, and a SUT under the AASHTO Performance Level 2 criteria published in the 1989 AASHTO *Guide Specifications for Bridge Railings*, which is considered equivalent to NCHRP Report 350 TL-4. Acceptable safety performance was demonstrated with a 1,800-lb (817-kg) small car, a 5,400-lb (2452-kg) pickup truck, and an 18,000-lb (8,200-kg) SUT crash test. For this program, minimal to moderate barrier damage was observed in the post flanges at the upper post-to-deck connections.

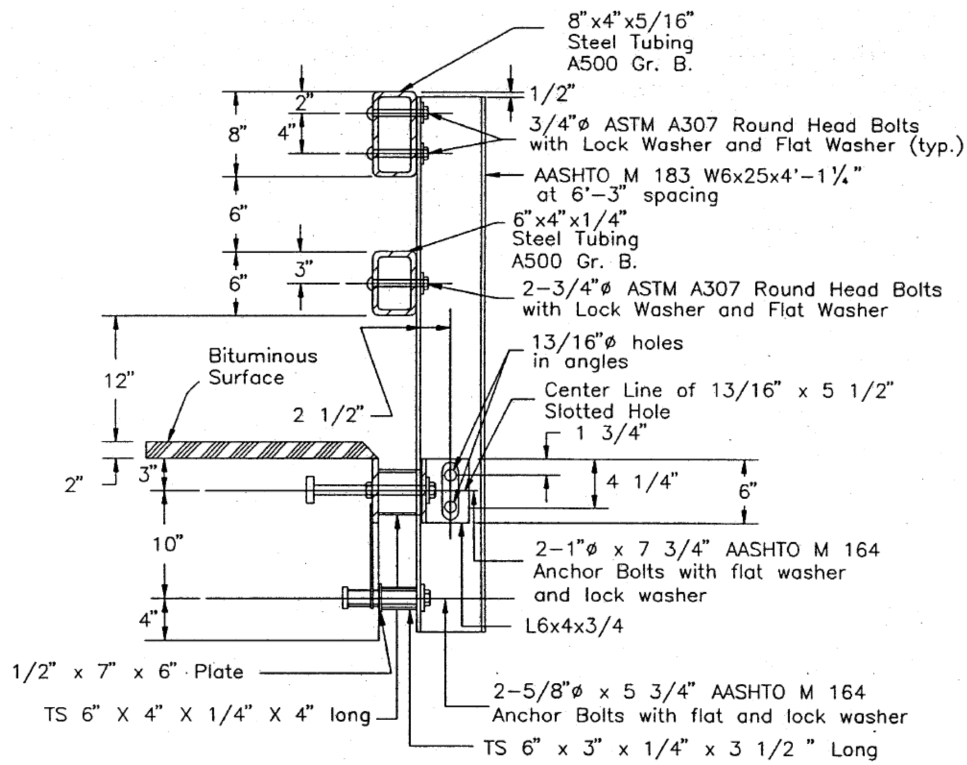


Figure 5. Illinois Side-Mounted Bridge Rail [3]

2.3.2 MwRSF STTR Bridge Rail

In 2002, the MwRSF STTR Bridge Rail was developed for its use on transverse glue-laminated (glulam) timber bridge decks [18-19]. The railing was a combination of a TS 8-in. x 3-in. x $^{3}/_{16}$-in. (TS 203-mm x 76-mm x 4.8-mm) steel top rail made of ASTM A500 Grade B steel, and a 10-gauge (3.43-mm) thrie-beam rail supported by ASTM A36 W6x15 (W150x22.5) structural wide-flange steel posts, as shown in Figure 6. The top tube rail sections were attached to a pair of ASTM A36 L 3½-in. x 3½-in. x $^{5}/_{16}$-in. (L 89-mm x 89-mm x 8-mm) structural steel angles with eight ⅝-in. (16-mm) diameter button head bolts. The structural steel angles were connected to the top of each web for the ASTM A36 W6x15 (W152x22.3) steel spacer blockouts. The thrie beam was attached to the front flanges of each blockout with two ⅝-in. (16-mm) diameter button head bolts. The steel, beam-and-post, bridge rail had an overall height of 36 in. (0.91 m) and a post spacing of 8 ft (2.4 m). The tube rail sections were connected to one another at the ends using a fabricated steel splice tube, which was welded together with two vertical ¼-in. (6.4-mm) and two horizontal ⅜-in. (9.5-mm) thick ASTM A36 steel plates.

Two crash tests were performed on the NCHRP Report 350 TL-4 steel bridge rail utilizing a pickup truck and a SUT. 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 impacted the system at 47.5 mph (76.5 km/h) and at an angle of 14.6 degrees relative to the bridge rail. The bridge railing adequately contained and redirected the pickup truck with a maximum dynamic deflection of 5⅜ in. (137 mm). Minor deformations to the occupant compartment were found inside the pickup truck. The bridge rail also properly contained and redirected the SUT. The system contained and redirected the SUT with a maximum deflection of 8 in. (203 mm). Minor deformations to the occupant compartment were found inside the SUT.

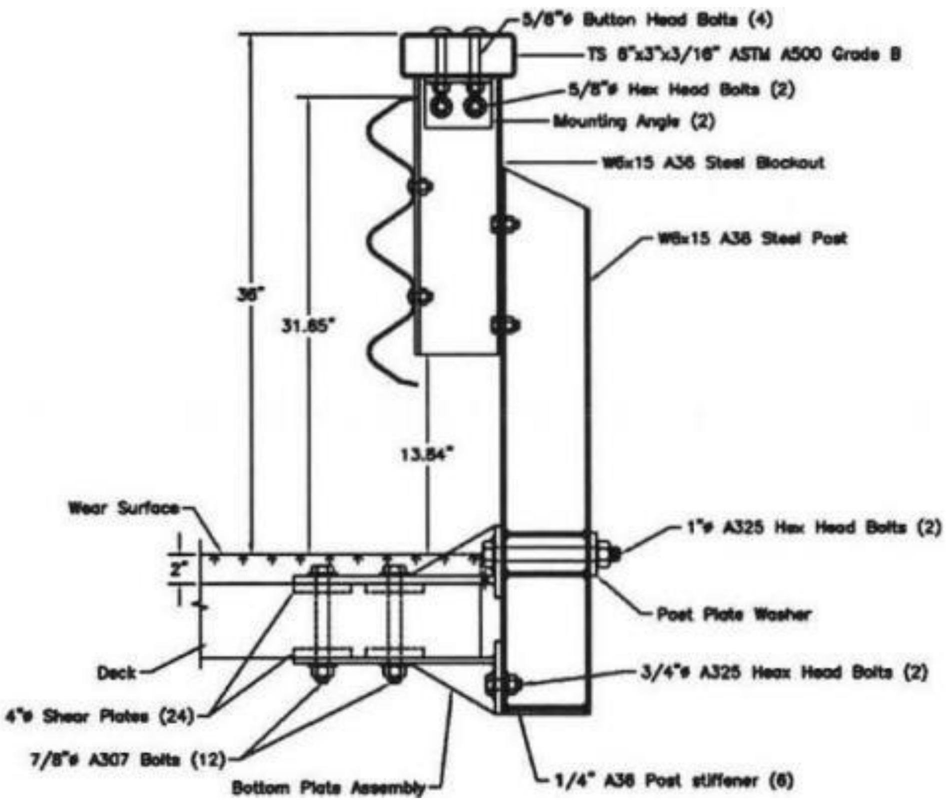


Figure 6. MwRSF STTR Bridge Rail for Transverse, Glulam Timber Decks [18]

2.3.3 California ST-70 Side-Mounted Bridge Rail

The California ST-70, Side-Mounted, Bridge Rail was designed by California Department of Transportation (Caltrans) to fulfill the urgency to develop a MASH TL-4 side-mounted system that can be used in areas where the posted speed limit is greater than 45 mph (70 km/h) [20]. The California ST-70, Side-Mounted, Bridge Rail consisted of four rectangular steel rail elements supported by fabricated steel plate posts mounted to the vertical outer edge of the bridge deck. The posts were spaced on 10 ft (3.0 m) centers, as shown in Figure 7. The overall height of the bridge rail was 42 in. (1,067 mm), as measured between the top of upper rail and the concrete deck surface. The top and bottom rail elements are comprised of ASTM A500 TS 8-in. x 3-in. x $^{5}/_{16}$ -in. (TS 203-mm x 76-mm x 7.9-mm) steel tubes and the middle two rails consisted of ASTM A36 TS 8-in. x 4-in. x $^{5}/_{16}$-in. (TS 203-mm x 102-mm x 7.9-mm) steel tubes. Each rail element was attached to the front of the posts with two ¾-in. diameter stud bolts. The steel posts consisted of two ASTM A36 ¾-in. (19-mm) thick by 5-ft (1.5-m) long plates spaced apart at 8 in. (203 mm) on center. The ends of the rails were connected to each other using ⅜-in. (9.5-mm) thick, customized, welded, rectangular splice tubes.

The California ST-70, Side-Mounted, Bridge Rail was successfully crash-tested under the AASHTO MASH TL-4 safety performance criteria using small car, pickup truck, and SUT test vehicles. The three full-scale vehicle crash tests resulted with minimal post and rail damage. The small car stayed in contact with the bridge rail for about 10 ft (3.0 m) for a maximum dynamic deflection of 0.9 in. (23 mm) and did not snag on the posts. The pickup truck contacted the railing for approximately 14 ft (4.3 m) with a maximum dynamic deflection of 1.6 in. (41 mm) without snagging on the posts. The SUT stayed in contact with the barrier for approximately 50 ft (15 m) for a maximum dynamic deflection of 2.4 in. (61 mm) and did not snag on the posts.

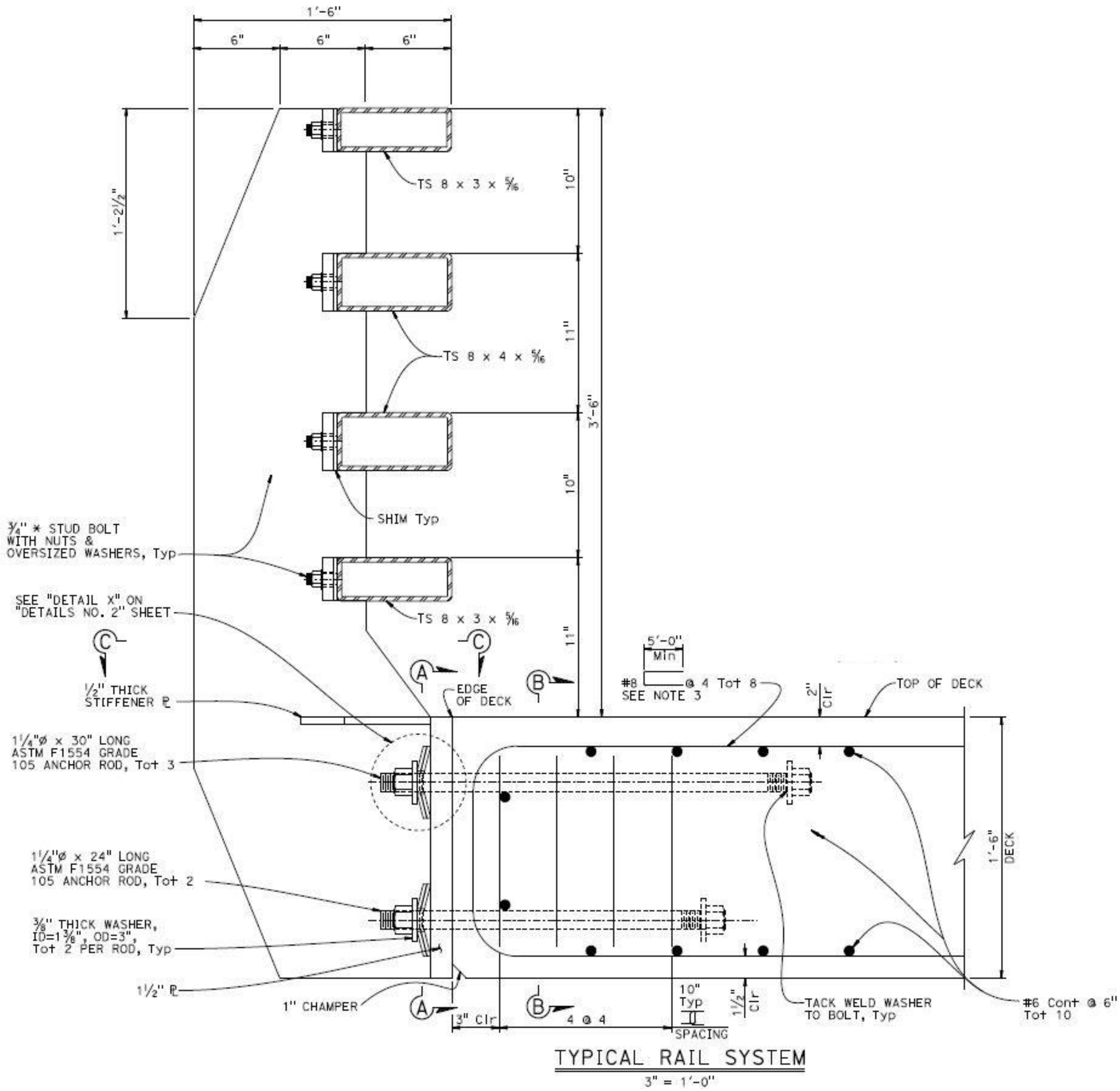


Figure 7. California ST-70, Side-Mounted, Bridge Rail [20]

2.3.4 Verrazano-Narrows Bridge Rail

The Verrazano Narrows Bridge Rail was successfully crash tested under the AASHTO MASH TL-5 safety criteria [21]. The bridge rail was designed for the Verrazano-Narrows Bridge in New York to accommodate large traffic volumes of SUTs and tractor-van trailers on this bridge. The Verrazano-Narrows Bridge Rail consisted of four rail elements mounted to the front faces of custom-welded, steel posts spaced on 8 ft – 3 in. (2.5 m) centers and side-mounted to the outer vertical edge of the bridge deck, as shown in Figure 8. The total height of the bridge rail was 42 in. (1,067 mm), as measured from the top of the upper rail to the roadways surface on the bridge deck. The ASTM A500 Grade B top and bottom rail elements were comprised of HSS 5-in. x 3-in. x ½-in. (HSS 127-mm x 76-mm x 13-mm) steel sections, and the two ASTM A500 Grade B middle rails were HSS 6-in. x 6-in. x ⅜-in. (HSS 152-mm x 152-mm x 10-mm) steel sections. The ASTM A572 Grade 50 steel posts were comprised of W8x28 (W200x41.7) structural steel sections welded to 1¾-in. (44.5-mm) thick steel baseplates, with the tops beveled 1¾-in. (44.5-mm) downward to the field side.

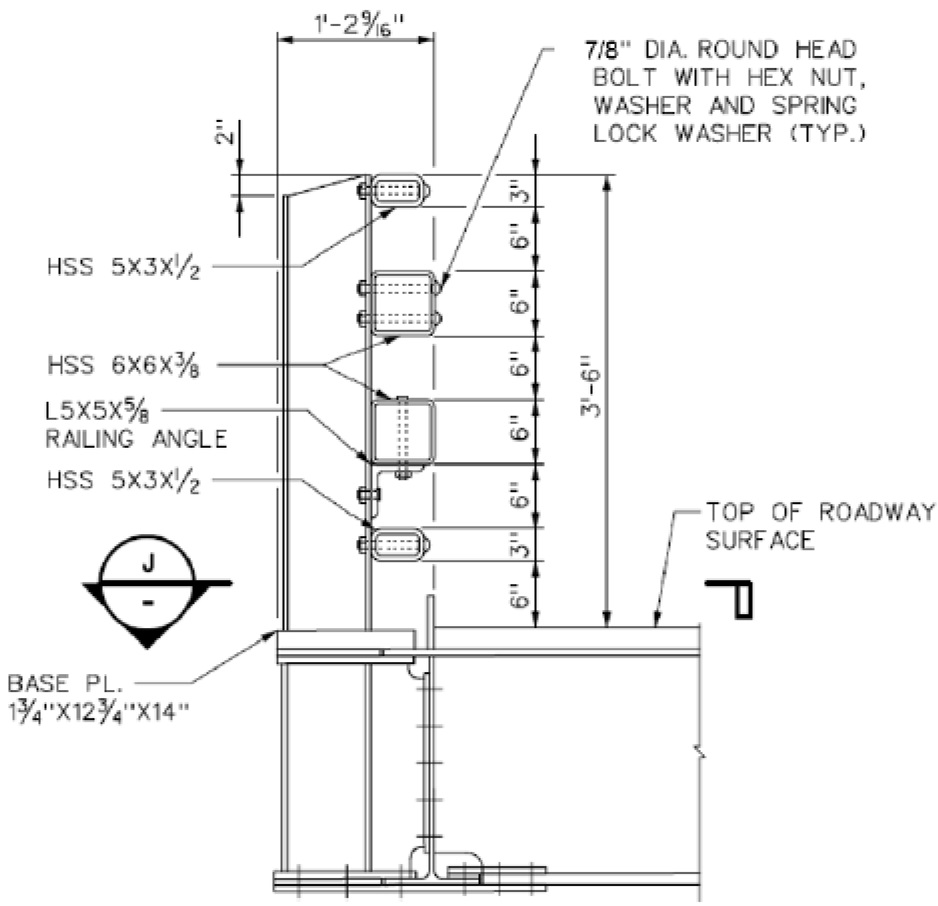


Figure 8. Verrazano-Narrows Bridge Rail [21]

The top rail element was attached to the post with two horizontal ⅞-in. (22-mm) diameter button head bolts. The upper middle rail was attached to the post using two staggered ⅞-in. (22-mm) diameter ASTM A325 button head bolts. The lower middle rail was attached to the post with an L 5-in. x 5-in. x ⅜-in. (L 127-mm x 127-mm x 9.5-mm) steel shelf angle. Two vertical ¾-in.

(19-mm) diameter hex head bolts attached the rail with the shelf angle and two horizontal ¾-in. (19-mm) diameter hex bolts attached the shelf angle to the post. The bottom rail was attached to the post with two ⅞-in. (22-mm) button head bolts. The end sections of the rails were connected to each other using HSS 6-in. x 6-in. x ⅜-in. (HSS 152-mm x 152-mm x 10-mm) steel tubes for the two middle rails and HSS 5-in. x 3-in. x ½-in. (HSS 127-mm x 76-mm x 12-mm) steel tubes for the top and bottom rails.

The bridge rail was found to have satisfactory performance according to the MASH TL-5 safety performance criteria. The small vehicle impacted the bridge railing and resulted with a maximum dynamic deflection of 1.5 in. (38 mm) without vehicle snag nor pocketing. The pickup truck and the tractor trailer vehicles impacted the bridge railing and produced a maximum dynamic deflection of 2 in. (51 mm) without snag on the posts for both test vehicles.

2.4 Steel, Top-Mounted, Beam-and-Post, Bridge Rails

Top-mounted, beam and post, bridge rails relevant to this study were identified to examine their railing elements, geometric characteristics, and safety performance. Some of the noted bridge rails were installed on top of reinforced concrete curbs. Curbs minimize the vertical rail opening between the bottom rail and the roadway surface which can reduce the propensity for wheel snagging on posts. Nevertheless, these systems were included within the bridge railing investigation.

2.4.1 TxDOT T131 Bridge Rail

The TxDOT T131 Bridge Rail consisted of three steel tubular rail elements mounted to the front flanges of W6x20 (W150x29.8) steel posts spaced on 8 ft – 4 in. (2.54 m) centers [22], as shown in Figure 9. The overall height of the bridge rail was 33 in. (838 mm) above the concrete deck. The top rail element was comprised of an ASTM A500 Grade C HSS 10-in. x 6-in. x ¼-in. (HSS 254-mm x 152-mm x 6.4-mm) structural steel tube. The two lower rail elements were ASTM A500 Grade C HSS 4-in. x 4-in. x ¼-in. (HSS 102-mm x 102-mm x 6.4-mm) steel tubes. The top rail element was bolted to a ½-in. (13-mm) thick steel plate that was welded on the top of the posts. The ⅞-in. (22-mm) diameter A307 hex head vertical bolts were used to make the connection. The two lower rails were attached to the front flanges of the steel posts using two ⅝-in. (16-mm) diameter ASTM A307 button head bolts at each post location. The ends of the middle and bottom rails were attached to each other using ASTM A500 Grade C HSS 3-in. x 3-in. x ¼-in. (HSS 76-mm x 76-mm x 6.4-mm) rectangular steel sections. The ends of the top rail sections were attached with ⅜-in. (10-mm) thick, welded steel tubes that were fabricated with two ASTM A572 Grade B bent steel plates.

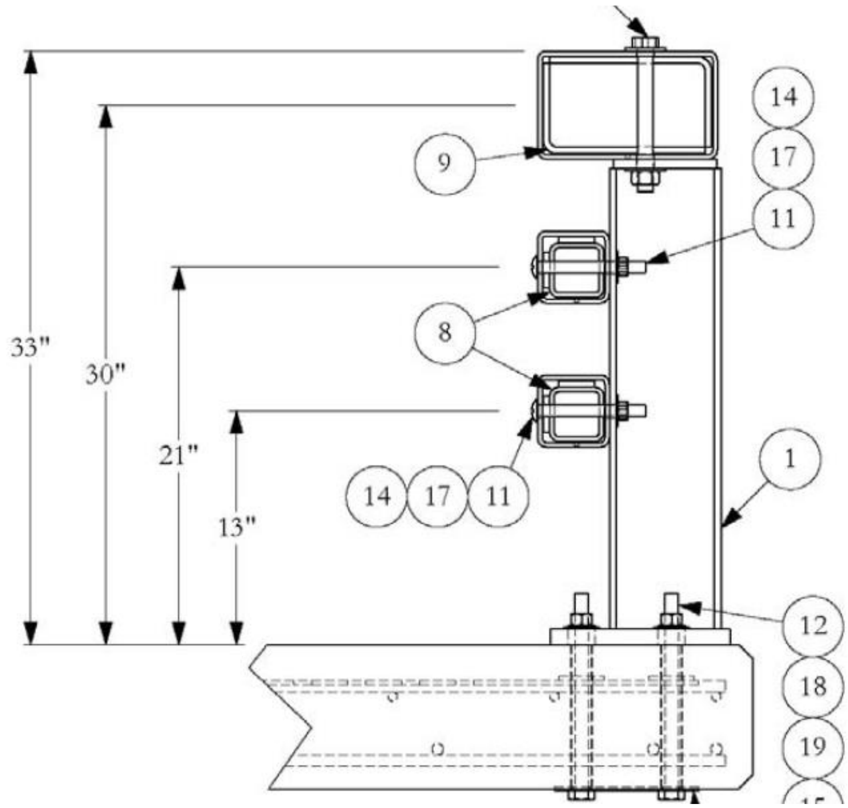


Figure 9. TxDOT T131 Bridge Rail [22]

The TxDOT T131 Bridge Rail was unsuccessfully crash-tested under the MASH TL-3 criteria due to rollover of the 2270P vehicle when one post-to-deck connection gave away during MASH test designation no. 3-11. However, the bridge rail performed adequately according to the MASH test designation no. 3-10 criteria. The maximum deflections for the small car and pickup truck crash tests were 4.8 in. (122 mm) and 10.9 in. (277 mm), respectively. For MASH test no. 3-10, the rails were noted to only have contact marks at the impact location, while the concrete deck was cracked around three posts. For MASH test no. 3-11 crash test, the rails were noted to have contact marks and scraps at the impact location. However, concrete spalling and cracking was observed in the deck near three posts. Consequently, the pickup truck was redirected out of the system but it later rolled over 135 degrees clockwise after loss of contact with the bridge railing.

2.4.2 TxDOT C2P Bridge Rail

The TxDOT C2P Bridge Rail consisted of three steel rails attached to customized steel posts spaced at 8 ft centers and mounted on top of a 9-in. tall concrete curb having an overall height of 42 in. (1,067 mm) [23], as shown in Figure 10. The upper rail elements were comprised of a ASTM A500 Grade B round HSS 4½-in. x $^{3}/_{16}$-in. (HSS 114-mm x 4.8-mm) steel sections, and the middle and bottom rails conformed to ASTM A500 Grade B rectangular HSS 6-in. x 2-in. x ¼-in. (HSS 152-mm x 51-mm x 6.4-mm) steel sections. The posts consisted of two ASTM A572 PL 31¼-in. x 9-in. x ¾-in. (PL794-mm x 229-mm x 19-mm) steel vertical plates spaced at 12 in. (305 mm) centers. Each rail was attached to the front faces of the custom-built posts with two ½-in. diameter ASTM A36 steel U-bolts. The rails were connected with internal splice tubes at the ends of the sections. The splice connection for the top rail used a round HSS 4-in. x ¼-in. (HSS 102-mm x 6.4-mm) steel section, and the splice connection for the lower rails used built-up tubes fabricated with two $^{3}/_{16}$-in. (4.8-mm) thick bent plates.

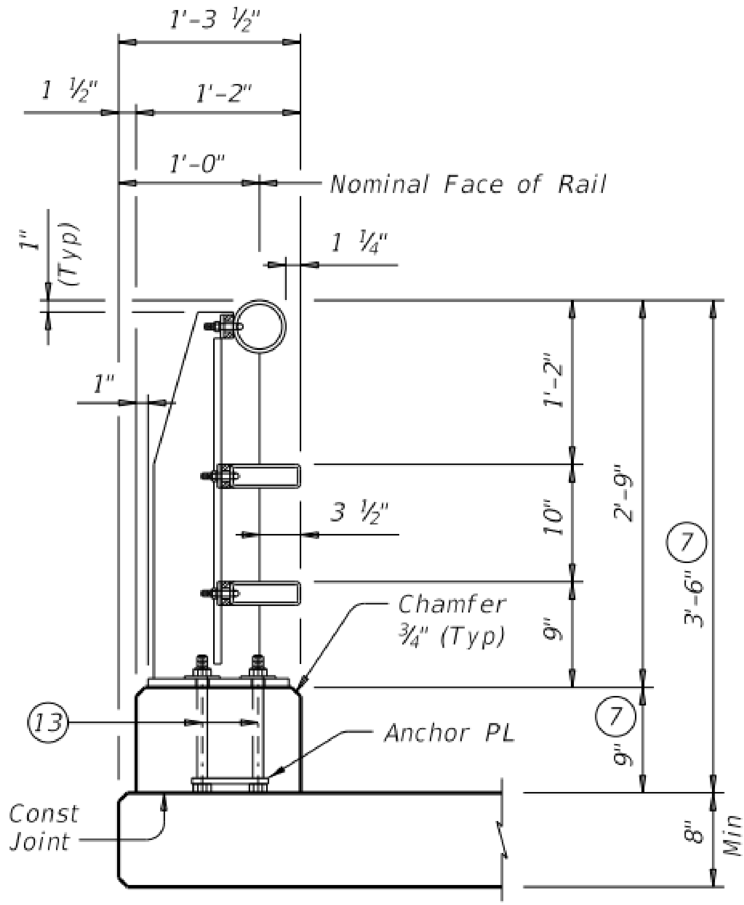


Figure 10. TxDOT C2P Bridge Rail [23]

The TxDOT C2P Bridge Rail was successfully crash tested under MASH TL-4 using three vehicle impact conditions. TxDOT C2P Bridge Rail contained and redirected the three MASH TL-4 vehicles, which did not penetrate, underride, or override the bridge rail installation. The 1100C and 2270P passenger vehicles remained upright during and after the collision event. The 10000S vehicle was properly contained and redirected after losing contact with the bridge rail, resulting in a maximum deflection during the test 11.4 in. (290 mm). For MASH test designation no. 4-12, the welds between the posts and base plates were not properly fabricated according to the design drawings. Consequently, these welds within the impact area immediately ruptured during the SUT impact event.

2.4.3 Massachusetts S3 TL-4 Bridge Rail

The bridge rail consisted of three rectangular steel rails attached to the front flanges of W6x25 (W150x37) steel posts spaced at 6 ft – 7½ in. (2.0 m) centers and mounted to the top of an 8 in. (203 mm) tall, concrete curb [24], as shown in Figure 11. The upper rail element was comprised of HSS 5-in. x 4-in. x ¼-in. (HSS 127-mm x 102-mm x 6.4-mm) steel tubes, while the lower two steel rails were comprised of HSS 5-in. x 5-in. x ¼-in. (HSS 127-mm x 127-mm x 6.4-mm) steel tubes. The overall system height of the bridge rail was 40¼ inches (1,022 mm).

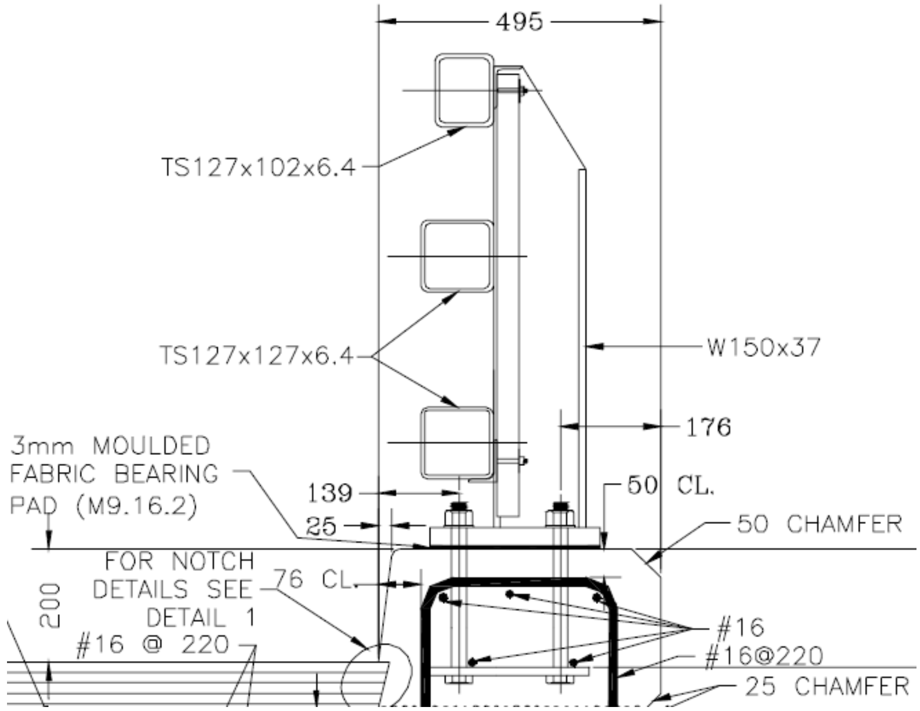


Figure 11. Massachusetts S3-TL4 Bridge Railing [24]

The Massachusetts S3-TL-4 Bridge Railing, which was mounted on a safety curb, met all criteria specified for *NCHRP Report 350* test designations nos. 4-11 and 4-12. During test designation nos. 4-11 and 4-12, the vehicles were contained and safely redirected, remained upright during and after the collision, and resulted in a maximum dynamic deflection of 1½ in. (38 mm) and 2⅛ in. (55 mm), respectively.

2.4.4 Caltrans ST-10 Bridge Rail

The ST-10 Bridge Rail was unsuccessfully crash tested by California Department of Transportation (Caltrans) under MASH test no. 3-11 vehicle impact conditions [25]. The Caltrans ST-10 Bridge Rail consisted of two TS 8-in. x 4-in. x $^5/_{16}$-in. (TS 203-mm x 101-mm x 8-mm) rectangular steel rails that were mounted to built-up steel posts fabricated with two PL 26½-in. x 10-in. x ⅝-in. (PL 673-mm x 254-mm x 16-mm) steel plates, which were spaced 8 in. (203 mm) apart on a baseplate, as shown in Figure 12. The post spacing was 10 ft (3 m) on centers. The posts were installed at the top of a 6-in. (152-mm) tall reinforced concrete curb.

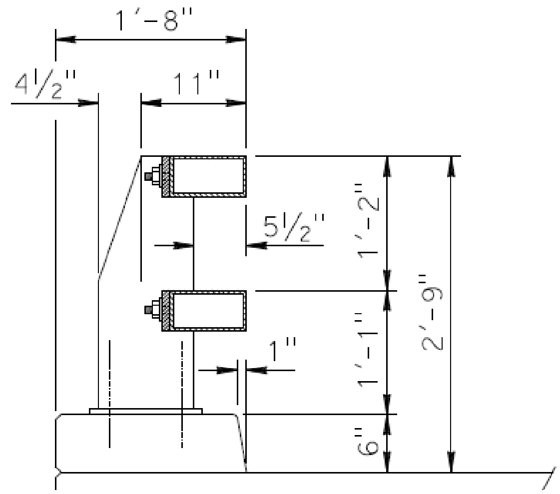


Figure 12. Caltrans ST-10 Bridge Rail [25]

The pickup truck was redirected after impacting the bridge rail. However, after losing contact with the barrier, the pickup rolled onto its side. The crash test of the pickup truck resulted in a maximum roll of 118.5 degrees. Vehicle intrusion between the rails presumably caused the vehicular instability.

2.4.5 PosBarrier-B Bridge Rail

The PosBarrier-B bridge rail consisted of three steel rails attached to customized steel posts spaced at 9 ft – 10 in. (3.0 m) centers and mounted on top of an 11⅞-in. (302-mm) tall concrete curb [26], as shown in Figure 13. The overall height of the bridge rail was 56 in. (1.4 m). The top, middle, and bottom rail elements were formed from flat steel sheets to be 5½ in. (140 mm) deep with a round face on the traffic side and center heights of 53⅛ in. (1.35 m), 34½ in. (876 mm), and 20½ in. (520 mm), respectively. These rails were bolted to hollow built-up posts, formed from a channel section welded to a front steel plate.

The PosBarrier-B Bridge Rail was successfully crash tested under MASH TL-4 impact conditions the three MASH TL-4 vehicles. The crash tests of the small vehicle, pickup truck, and SUT resulted with a maximum dynamic deflection of 1.5 in. (38 mm), 2.25 in. (57 mm), and 3 in. (76 mm), respectively.

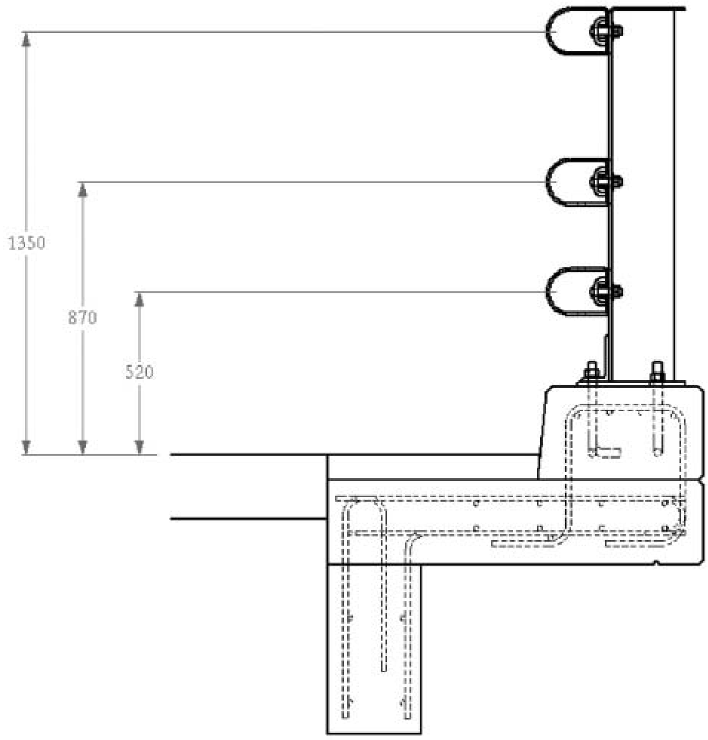


Figure 13. PosBarrier-B Bridge Rail [26]

2.4.6 Caltrans ST-20 Bridge Rail

The Caltrans ST-20 Bridge Rail consisted of four steel, rectangular rails and a steel, tubular handrail [27]. The top and bottom rail elements were ASTM A500 TS 6-in. x 3-in. x $^{5}/_{16}$-in. (TS 152-mm x 76-mm x 7.9-mm) steel sections, and the two middle rail elements were ASTM A500 TS 6-in. x 4-in. x $^{5}/_{16}$-in. (TS 152-mm x 102-mm x 7.9-mm) steel sections, as shown in Figure 14. The tubular handrail was comprised of a ASTM A500 TS 3-in. x 2-in. x $^{3}/_{16}$-in. (TS 76-mm x 51-mm x 4.8-mm) steel sections. The built-up, steel posts were fabricated with two ASTM A36 Grade B PL 40¾-in. x $11^{13}/_{16}$-in. x ⅝-in. (PL 1035-mm x 300-mm x 16-mm) steel plates, which were spaced 8 in. (203 mm) apart on a baseplate. Each of the four main rails were attached to the front faces of the built-up steel posts using two ¾-in. (19 mm) diameter ASTM A108 steel stud bolts. The post spacing along the system was 9 ft – 10 in. (3.0 m), and the overall height of the top rail element was $46^{11}/_{16}$-in. (1,186 mm) above the concrete deck.

The Caltrans ST-20 Bridge Rail was successfully crash tested by California Department of Transportation (Caltrans) under NCHRP Report 350 test no. 3-11 safety performance criteria. The 2000P pickup truck was successfully contained and redirected with a maximum dynamic deflection of 1 in. (25 mm).

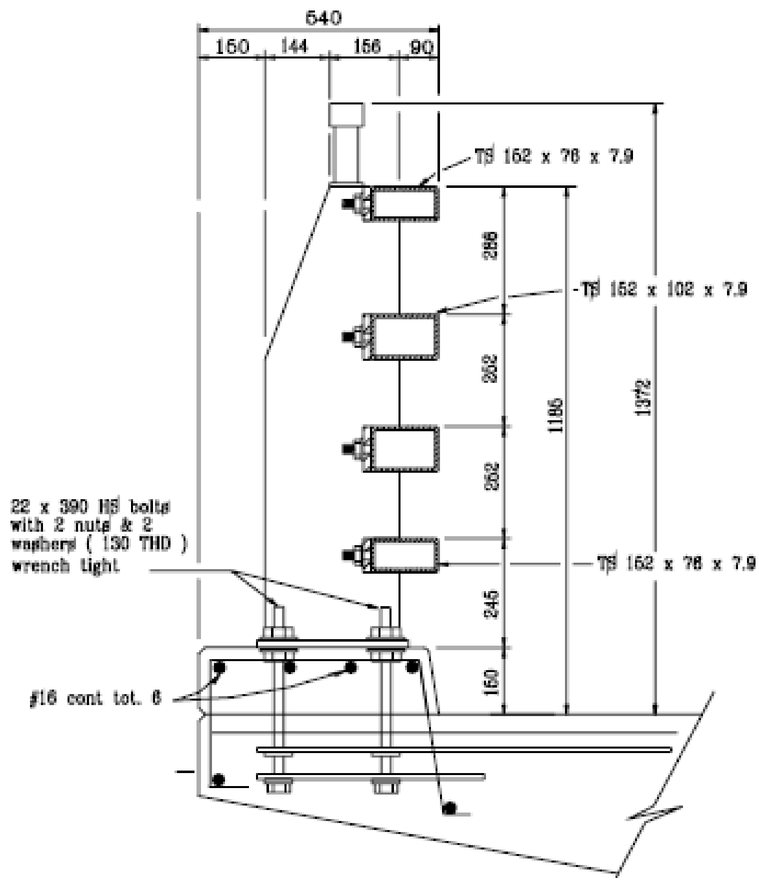


Figure 14. Caltrans ST-20 Bridge Rail [27]

2.4.7 TxDOT T131RC Bridge Rail

The TxDOT T131RC Bridge Rail consisted of two tubular rail elements mounted to the front flanges of W6x20 (W150x29.8) steel posts spaced on 5 ft (1.52 m) centers and mounted on top of an 8-in. (203-mm) tall reinforced concrete curb [28], as shown in Figure 15. Both rail elements were comprised of ASTM A500 Grade C HSS 6-in. x 6-in. x¼-in. (HSS 152-mm x 152-mm x 6.4-mm) structural tubes. The overall height of the bridge rail was 36 in. (914 mm) above the concrete deck. The two rails were attached to the front flanges of the steel posts using two ⅝-in. (16-mm) diameter ASTM A307 button head bolts at each post location. The ends of the top rail sections were attached with ⅜-in. (10-mm) thick, welded steel tubes that were fabricated with two ASTM A572 Grade B bent steel plates. The bridge railing adequately contained and redirected the pickup truck.

The TxDOT T131RC Bridge Rail was successfully crash tested under MASH test designation no. 3-11 impact conditions. The bridge railing safely contained and redirected the 2270P pickup truck. Although the maximum dynamic deflection was not obtainable, the maximum permanent set deflection was 6½ in. (165 mm).

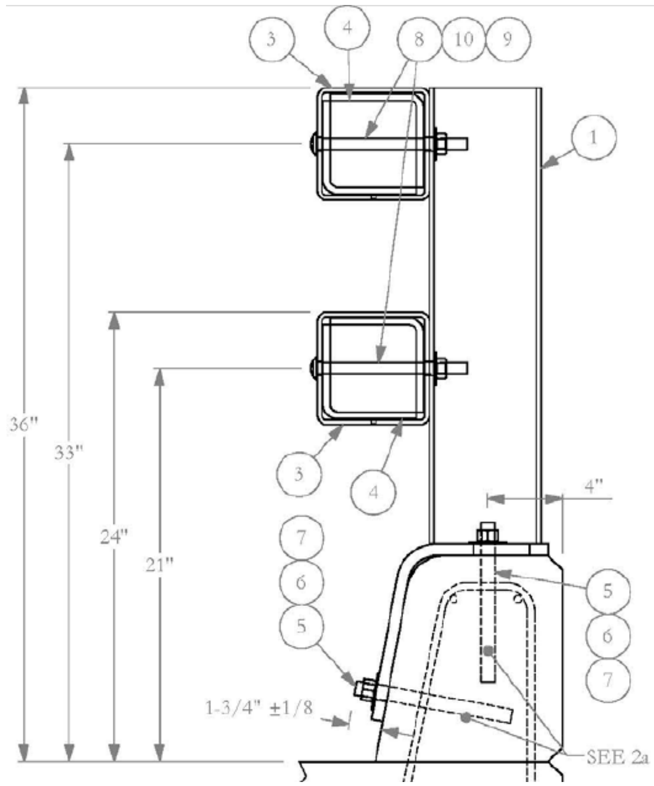


Figure 15. TxDOT T131RC Bridge Rail [28]

2.4.8 TxDOT Picket Bridge Rail

The TxDOT Picket Bridge Rail consisted of three steel rails attached to the front faces of built-up posts, which were mounted to the top of a 9-in. (229-mm) tall reinforced concrete curb [29], as shown in Figure 16. The overall height of the bridge rail was 36 in. (914 mm) above the concrete deck surface, while the built-up posts were spaced on 8 ft (2.44 m) centers. The top rail element consisted of an ASTM A500 Grade B round HSS 4½-in. x $^{3}/_{16}$-in. (HSS 114.3-mm x 4.8-mm) steel tube. The lower two rail elements were comprised of ASTM A500 Grade B HSS 6-in. x 2-in. x ¼-in. (HSS 152-mm x 51-mm x 6.4-mm) rectangular steel tubes. The heights of the top, middle, and bottom steel rail elements were 36 in., 28 in., and 18 in. (914 mm, 711 mm, and 457 mm), respectively, as measured to the top of the tubes. Each rail was attached to each post using a ½-in. (12.7-mm) diameter ASTM A36 bent U-bolt. The built-up posts consisted of two ASTM A572 Grade 50 ¾-in. (19-mm) thick, 9-in. (229-mm) wide, and 26-in. (660-mm) tall steel plates spaced 12½-in. (317-mm) apart. The steel pickets were attached to the field side of the bridge railing and consisted of ASTM A36 ⅝-in. (15.9 mm) square steel bars located at 6 in. (152-mm) on centers. The ends of the rails were attached to each other with internal splice tubes. The top splice tube consisted of a ASTM A500 Grade B HSS 4-in. x ¼-in. (HSS 102-mm x 6.4-mm) round section, and the two lower splice tubes were ASTM A36 $^{3}/_{16}$-in. (4.8-mm) thick, welded steel sections that were fabricated with two bent steel plates.

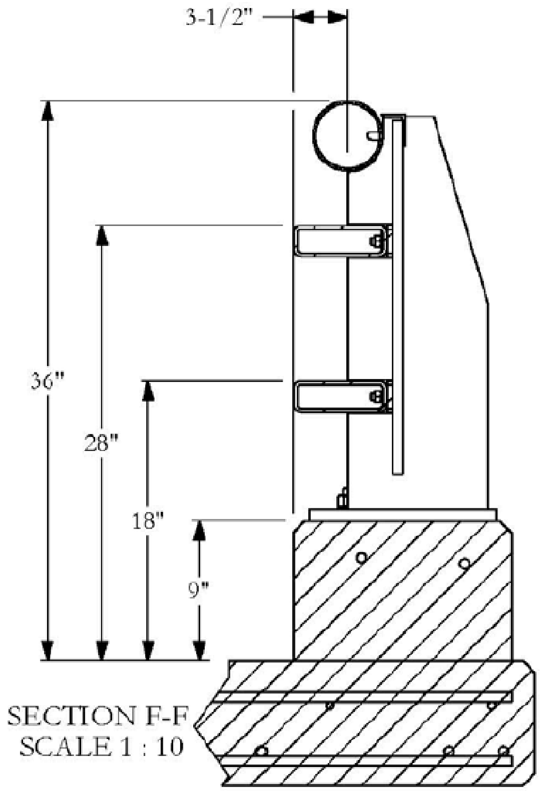


Figure 16. TxDOT Picket Bridge Rail [29]

Two crash tests were performed on the MASH TL-3 steel bridge rail utilizing a pickup truck and a small car. The TxDOT Picket Bridge Rail adequately contained and redirected the small car with a maximum dynamic deflection of 0.9 in. (23 mm). Minor deformations to the occupant compartment were found inside the small vehicle. The bridge rail also properly contained and redirected the pickup truck with a maximum dynamic deflection of 2.8 in. (71 mm). Minor deformations to the occupant compartment were found inside the pickup truck.

2.5 Lateral and Vertical Impact Loading

2.5.1 Overview

To design longitudinal roadside barriers, such as the new MASH TL-4 steel, beam-and-post, side-mounted, bridge rail, it was necessary to identify lateral and vertical impact loadings. Many research studies have investigated the magnitude of impact loading pertaining to TL-4 impact safety standards. Researchers have identified different impact loads based on published design values, physical test results, and simulation results. Therefore, researchers have used different TL-4 design impact loads for the development of longitudinal roadside barriers, including bridge rails.

For the development of the new steel, beam-and-post, side-mounted, bridge rail, lateral and vertical design impact loads were reviewed to configure the system to resist MASH TL-4 pickup truck and SUT impact events.

Design impact forces for configuring roadside barrier systems, including bridge rails, have been published in various editions of the AASHTO LRFD *Bridge Design Specifications*, including the 8th edition [30]. These impact loads were derived using data obtained from two crash testing studies using an instrumented, reinforced concrete wall, which was conducted by TTI researchers during the 1980's [31-33]. The loads measured in these studies were obtained from impacts with rigid barriers, therefore, these load measurements would represent an upper bound of impact forces that would actually be observed in deformable roadside barriers, such as the new steel, side-mounted, beam-and-post, bridge rail.

2.5.2 42-in. Tall, Instrumented, Reinforced Concrete Wall

In 1981, the first impact load study involved a 42-in. (1,067-mm) tall, instrumented, reinforced concrete wall that was constructed and out-fitted with accelerometers and load cells [31]. The wall consisted of four 120-in. x 42-in. (2.9-m x 1.1-m) wall segments with load cells on all four corners and one accelerometer in the center of each wall segment. The instrumented wall was used to measure the impact forces associated with eight full-scale crash tests involving small vehicles, pickup trucks, and intercity buses. The target impact speed in all of the crash tests was 60 mph (96.6 km/h). Each full-scale crash test was divided in two phases in order to provide the resultant loading at the frontal initial impact and the final rear impact or "tail-slap". A summary of results from all the tests provided data for the initial and the final phases of the impact for each test. The summary of the results is shown in Table 8.

Table 8. Distribution of Forces from the 42-in. (1,067-mm) Tall, Instrumented Tall [31]

Test Conditions				Impact Phase	Resultant			
Vehicle Type	Weight (lb)	Speed (mph)	Angle (deg)		Height (in.)	Magnitude (kips)	Contact Height (ft)	Contact Length (ft)
Subcompact Sedan	2,050	59.0	15.5	Initial	17.0	18.4	2.33	5.0
				Final	18.7	8.4	2.58	7.6
Subcompact Sedan	2,090	58.5	21.0	Initial	19.0	21.1	2.67	6.0
				Final	20.7	13.1	3.00	8.0
Compact Sedan	2,800	58.3	15.0	Initial	18.1	18.5	2.50	5.0
				Final	15.3	13.9	2.08	10.8
Compact Sedan	2,830	56.0	18.5	Initial	19.3	22.0	2.92	4.8
				Final	21.3	22.5	3.00	10.2
Full-Sized Sedan	4,680	52.9	15.0	Initial	21.4	52.5	3.08	7.3
				Final	24.0	28.3	3.25	10.7
Full-Sized Sedan	4,740	59.9	24.0	Initial	21.8	59.9	3.17	6.5
				Final	22.5	28.3	3.25	14.5
School Bus	20,030	57.6	15.0	Initial	29.0	63.7	2.17	12.3
				Final	32.7	73.8	1.58	25.5
Intercity Bus	32,020	60.0	15.0	Initial	26.3	85.0	2.58	6.3
				Final	28.4	211.0	2.25	15.0

2.5.3 90-in. Tall, Instrumented, Reinforced Concrete Wall

In 1989, the second impact load study involved a 90-in. (2.3-m) tall, instrumented, reinforced concrete wall that was constructed and out-fitted with accelerometers and load cells [32-33]. The wall consisted of four 120-in. x 90-in. (2.9-m x 2.3-m) wall segments with load cells on all four corners and one accelerometer in the center of each wall segment. Ten vehicles, ranging from small cars and pickup trucks to tractor-van and tractor-tank trailers, were crashed into the barrier. A summary of results from all of the tests is shown in Table 9.

Table 9. Distribution of Forces from the 90-in. (2,3-m) Tall, Instrumented Tall [32]

Vehicle Type	Vehicle Weight (lb)	Impact Velocity (mph)	Impact Angle (deg)	Maximum Impact Force* (kips)	Vertical Height of Resultant (in.)
Automobile	4,500	61.8	25.6	56	19.0
Intercity Bus	40,050	58.6	15.4	386	52.0
Tractor Van-Trailer	80,080	55.0	15.3	220	70.0
Tractor Tank-Trailer	79,900	54.8	16.0	408	56.0
Pickup	5,409	65.8	19.9	45	22.5
Pickup	5,432	46.8	19.0	32	23.0
Suburban	5,400	64.1	19.7	51	20.0
Suburban	5,350	44.7	19.5	28	25.0
Tractor Van-Trailer	50,000	50.4	14.6	150	35.0
Single Unit Truck	18,050	51.6	16.8	90	40.0

*Forces shown are the maximum 0.050-sec average forces measured with the instrumented wall.

2.5.4 AASHTO 1989 Guide Specifications for Bridge Rails - Design Loading

The AASHTO 1989 *Guide Specifications for Bridge Rails* [4] provided a matrix of recommended design loads for the PL-1, PL-2, PL-3, PL-4, and PL-4T performance levels. A recommended design lateral load of 80 kips (356 kN) longitudinally distributed over 28 in. (711 mm) at a height of 17 in. (432 mm) was specified for (PL-2) Performance Level, as shown in Table 10. In addition, the recommended vertical design load was 15 kips (67 kN) downward and 5 kips (22 kN) upward. Details were provided for distributing lateral, vertical, and longitudinal loads to parapets, rails, and posts.

Table 10. AASHTO 1989 *Guide Specifications for Bridge Rails* Bridge Railing Loads [4]

Railing Performance Level	PL-1	PL-2	PL-3	Optimal PL-4	Optimal PL-4T
Horizontal Load	30 kips	80 kips	140 kips	200 kips	200 kips
Downward Load	12 kips	15 kips	18 kips	18 kips	18 kips
Upward Load	4 kips	5 kips	6 kips	6 kips	6 kips
Horizontal Load Height	16 in.	17 in.	18 in.	19 in.	19 in.
Horizontal Load Distributed Length	24 in.	28 in.	32 in.	36 in.	36 in.

2.5.5 AASHTO LRFD Bridge Design Specifications - Design Loading

The recommended design impact loads found in various editions of the *AASHTO LRFD Bridge Design Specifications*, including the 8th edition [30], are shown in Table 11. For the pickup truck, the lateral design impact load was 54 kips (240 kN) at 24 in. (610 mm) above ground level applied on a span of 4 ft (1.22 m), and the vertical impact load was found to be 4.5 kips (20 kN) over a 18-ft (5.5-m) span. For the SUT, the lateral design impact load was found to be 54 kips (240 kN) at 32 in. (813 mm) applied on a span of 3.5 ft (1.06 m), and a vertical load of 18 kips (80 kN) over an 18-ft (5.5-m) span.

Table 11. AASHTO LRFD *Bridge Design Specifications* Forces for Traffic Railings [30]

Design Forces and Designation	Railing Test Levels					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
F_t Transverse (kips)	13.5	27.0	54.0	54.0	124.0	175.0
F_L Longitudinal (kips)	4.5	9.0	18.0	18.0	41.0	58.0
F_v Vertical (kips)	4.5	4.5	4.5	18.0	80.0	80.0
L_t and L_L (ft)	4.0	4.0	4.0	3.5	8.0	8.0
L_v (ft)	18.0	18.0	18.0	18.0	40.0	40.0
H_e (min) (in.)	18.0	20.0	24.0	32.0	42.0	56.0
Minimum *H* Height of rail (in.)	27.0	27.0	27.0	32.0	42.0	90.0

where:

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

2.5.6 32-in. Tall, Vertical, Rigid Barrier Finite Element Simulations

In 2009, TTI researchers performed finite element simulations of a NCHRP Report 350 TL-3 pickup truck impacting a 32-in. (813-mm) tall, vertical rigid barrier [34-35]. The average impact force on the rigid barrier was 55.8 kips (248 kN); similar to the 54-kip (240 kN) recommended lateral design impact load published in the current AASHTO LRFD *Bridge Design Specifications*.

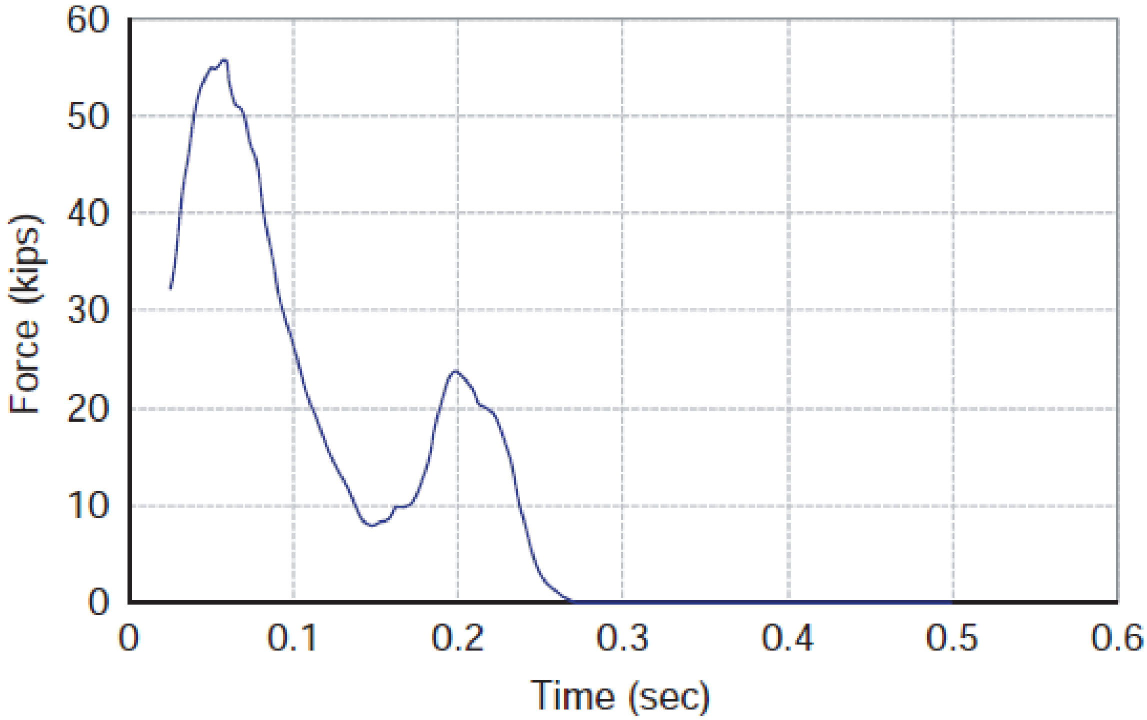


Figure 17. NCHRP Report 350 TL-3 Pickup Truck Time History of Impact Force [34]

TTI researchers also performed finite element simulations of the MASH pickup truck impacting the identical 32-in. (813-mm) tall, vertical rigid barrier. The maximum average force obtained from the MASH pickup truck was approximately 71 kips (316 kN) at a height of 19.5 in. (495 mm) [36]. The two different models of the MASH and NCHRP Report 350 pickup trucks are shown in Figure 18.

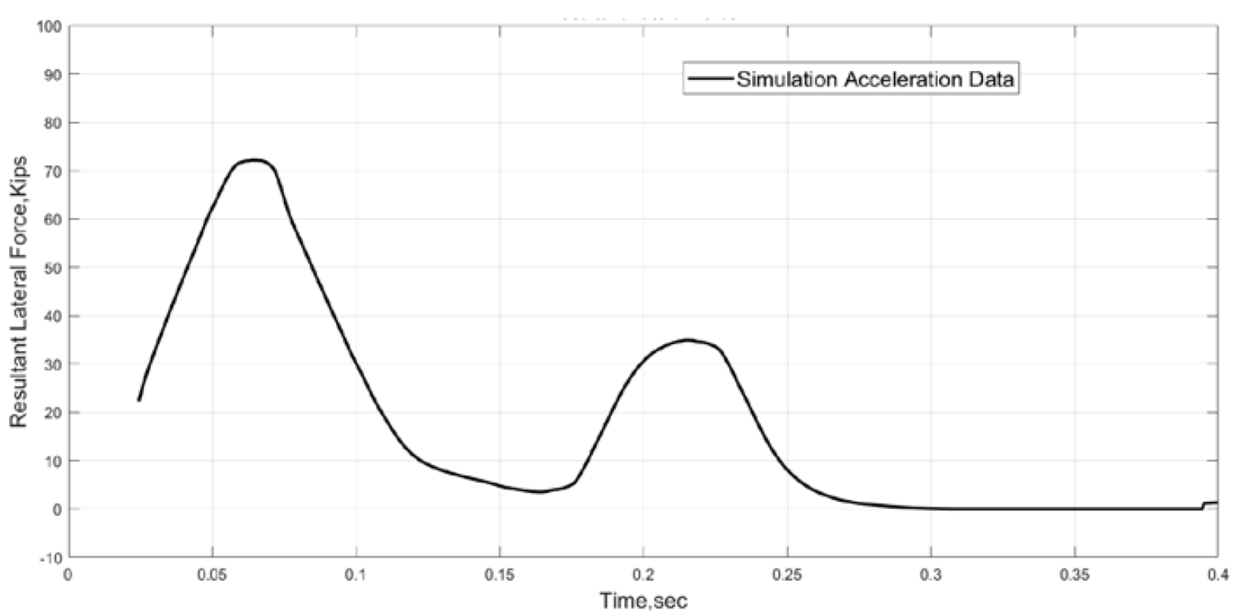


Figure 18. MASH TL-3 Pickup Truck Time History of Impact Force [36]

2.5.7 NCHRP Project No. 22-20(2)

Finite element simulations were conducted with a MASH SUT vehicle model impacting rigid vertical walls with heights of 36 in. (914 mm), 39 in. (991 mm), 42 in. (1.07 m), and 90 in. (2.23 m) [37]. The objective of this study was to obtain MASH TL-4 impact loads on barriers at different heights using finite element impact simulations. As shown in Table 12, as the height of the barrier increases, the applied force increases due to less vehicle roll. Moreover, the magnitude of the vertical forces applied on the barrier decreases as the barrier height increases due to the decrease of vehicle roll.

Table 12. Summary of Resultant Impact Loads for MASH TL-4 Single-Unit Truck [37]

Design Forces and Designations	Barrier Height (in.)			
	36	39	42	90
F_t Transverse (kips)	67.2	72.3	79.1	93.3
F_L Longitudinal (kips)	21.6	23.6	26.8	27.5
F_v Vertical (kips)	37.8	32.7	22	N/A
L_L (ft)	4	5	5	14
H_e (in.)	25.1	28.7	30.2	45.5

N/A = Not Applicable

where:

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

NCHRP Report No. 22-20(2) divides MASH TL-4 recommended design impact loads in two sections according to the heights of the barrier [37]. TL-4-1 was associated with a 36-in. (915-mm) tall, rigid vertical barrier and TL-4-2 correspond a 42-in. (635-mm) tall, rigid vertical barrier. However, TL-4-2 design impact loads were used for the design of longitudinal barriers with an overall height greater than 36 in. (915 mm).

Table 13. Recommendation of Design Impact Loads for MASH TL-4 Traffic Barriers [37]

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

where:

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

The TL-4-1 design loads correspond to a rigid 36-in. (813-mm) tall barrier with a design lateral impact load of 70 kips (311 kN) at 25 in. (635 mm) above grade applied on a 4-ft (1.22-m) span and a design vertical impact load of 38 kips (169 kN) on a 18-ft (5.5-m) span. The TL-4-2 design loads correspond to a barrier greater than 36 in. (914 mm) tall with a design lateral force of 80 kips (356 kN) at 30 in. (762 mm) height applied on 18 ft (5.5 mm) and a design vertical load of 33 kips (146 kN) on an 18-ft (5.5-m) span.

The design load for MASH TL-3 impacts was updated from the recommended 54-kip (240 kN) load obtained from an impact simulation of a NCHRP Report 350 TL-3 pickup truck impacting a 32-in. (813-mm) tall, vertical rigid barrier NCHRP Report No. 663 [34]. Finite element simulations of the MASH 2270P pickup truck impacting the same 32-in. (813-mm) tall, vertical rigid barrier indicated that a lateral load of 70 kips (311 kN) at 24 in. (610 mm) above grade applied on a 4-ft (1.22-m) span represented an upper bound of the lateral design impact load observed on simulation.

2.5.8 36-in. Tall, Single-Slope, Rigid Concrete Barrier

In 2011, TTI performed a satisfactory full-scale crash test of the MASH TL-4 10,000S SUT impacting a 36-in. (914-mm) tall, single-slope, rigid concrete barrier for MASH test designation no. 4-12 [38]. The objective of this study was to recommend a lateral design impact load and a minimum rail height under MASH TL-4 impact conditions.

Impact LS-DYNA simulations were performed with barrier heights of 36, 37, 38, 39, and 42 in. (914, 940, 965, 991, and 1,067 mm). As expected, the 42-in. (914-mm) tall barrier produced the greatest vehicular stability; however, this research was required to establish a minimum height for MASH TL-4 conditions. The 36-in. (914-mm) height was selected for a full-scale crash test. LS-DYNA simulations were also used to calculate lateral loads resulting from simulated SUT impacts into a rigid, single-slope barrier with various heights. The researchers based their recommendation for a lateral design impact load of a 42-in. (1.07-m) height to accommodate a broader range of MASH TL-4 heights. A design load of 80 kips (356 kN) was recommended for MASH TL-4 rails.

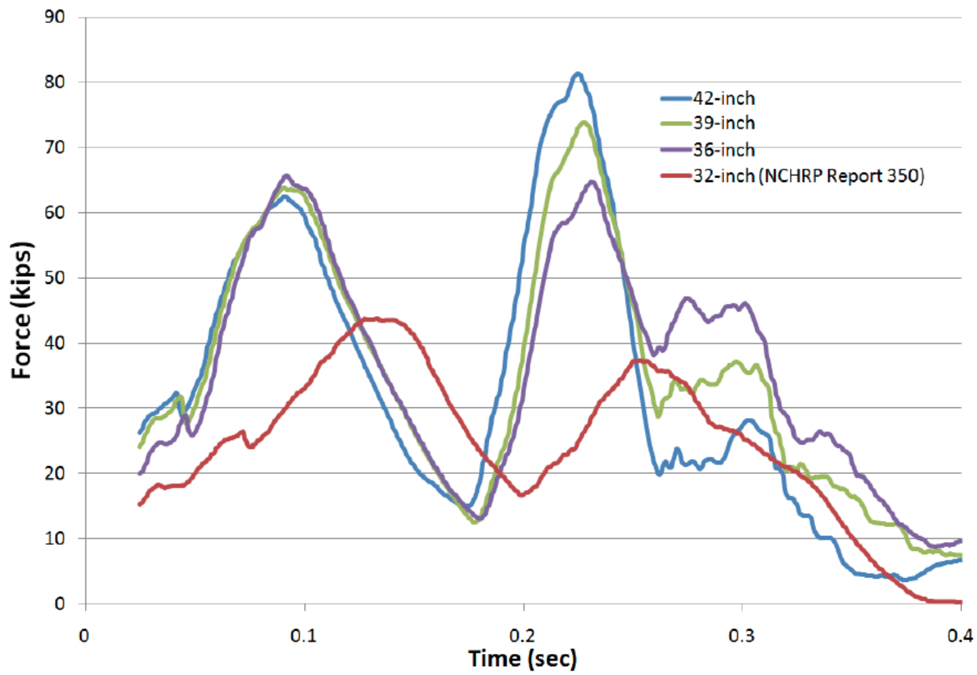


Figure 19. Lateral Impact Loads with Various Rail Heights [38]

2.6 Minimum Bridge Rail Overall Heights

The NCHRP Report 350 crash testing criteria have been used to determine acceptable overall heights for bridge rails, median barriers, and roadside barriers at most levels which allow vehicle capture and redirection without override. Due to the more intense MASH impact conditions for the pickup truck and SUT vehicle, further review was needed to identify minimum barrier heights that would meet MASH TL-4 impact conditions.

2.6.1 Impact Simulations of 27, 28, and 29 in. Tall Rigid Barriers

In 2017, an LS-DYNA simulation was performed by TTI researchers to determine minimum heights for MASH TL-3 impact conditions [36]. Finite element simulations of a pickup truck impacting rigid barriers were used to determine the minimum rail height for the MASH TL-3 pickup truck. The height of the rigid barriers were progressively increased to obtain a minimum rail height depending on the vehicle kinematics and stability. The simulations were conducted with a vertical rigid barrier with heights of 27 in. (686 mm), 28 in. (711 mm), and 29 in. (737 mm). The simulation with the 27-in. (686-mm) tall rigid barrier resulted with rollover of the pickup truck. The simulation of the 28-in. (711-mm) tall rigid barrier did not rollover but experienced moderate roll. The simulation with the 29-in. (737-mm) tall rigid barrier showed adequate vehicle kinematics and remained fairly stable after the impact event. Therefore, based on the simulation results, the minimum recommended overall height for MASH TL-3 bridge rails was 29 in. (737 mm).

Figure 20. FE Simulations of MASH Pickup Truck Impacting a 27-in. (left), 28-in. (middle), and 29-in. (right) Tall Rigid Barriers [36]

2.6.2 32-in. Tall, Safety Shape, Concrete Barrier

In 2004, TTI researchers performed a successful full-scale crash test of a NCHRP Report 350 TL-4 SUT impacting a 32-in. (813-mm) tall New Jersey Safety Shape Bridge Rail [39]. The objective of this study was to determine if the 32-in. (813-mm) minimum height requirement for TL-4 vehicles from the AASHTO LRFD *Bridge Design Specifications* was adequate.

Figure 21. 32-in. Tall NJ Safety Shape Barrier under NCHRP 350 Impact Conditions [39]

However, in 2006, MwRSF researchers performed an unsuccessful full-scale crash test of an updated NCHRP Report 350 TL-4 single unit truck impacting the 32-in. tall New Jersey Safety Shape Bridge Rail [40]. The crash test conditions of this full-scale crash test resulted to be identical to the current MASH test designation 4-12 test conditions. During the impact, the SUT rolled over the top of the barrier and came to rest on its side behind the barrier.

Figure 22. 32-in. Tall NJ Safety Shape Barrier under MASH Impact Conditions [40]

2.6.3 36-in. Tall, Single-Slope, Concrete Barrier

In 2011, TTI researchers crash tested a 36-in. (914-mm) tall, Single-Slope, Traffic Rail with a MASH TL-4 SUT to identify a minimum barrier height for MASH TL-4 longitudinal barriers [38]. The barrier successfully contained and redirected the SUT. Therefore, a minimum barrier height of 36 in. (914 mm) was determined for MASH TL-4 impact conditions.

Figure 23. 36-in. (914-mm) Tall, Single-Slope, Traffic Rail Bridge Rail under MASH TL-4 Impact Conditions [38]

3 DESIGN CRITERIA

3.1 Overview

Several design criteria were established for the development of the new steel, beam-and-post, side-mounted, bridge rail. The configuration of the new bridge rail was designed to be adaptable to four different bridge deck types utilized by the States of Illinois and Ohio with or without the installation of future 3-in. (76-mm) thick pavement overlays. The configuration of the bridge rail was also designed to meet minimum rail heights for the three MASH TL-4 test vehicles to prevent vehicle rollover and instabilities. The bridge rail system was expected to safely contain and redirect MASH TL-4 vehicles as well as resist lateral and vertical design impact loadings from small cars, pickup trucks, and SUTs. Furthermore, in order to satisfy MASH TL-4 safety performance criteria, the bridge rail configuration was to mitigate vehicle snag into posts through identifying appropriate vertical clear openings, rail heights, and rail offsets away from posts. After consulting with the Illinois and Ohio DOTs, the sponsors provided design criteria to ease the fabrication and installation efforts for the bridge rail.

3.2 Critical Deck Configuration

The new steel, side-mounted, beam-and-post, bridge rail was designed to be adaptable to four concrete bridge deck configurations utilized by the Illinois and Ohio DOTs. Each configuration has post-to-deck connections that are comprised of a pair of tension and a pair compression steel anchor rods. This connection is used to attach the front flange of each steel post to the exterior vertical edge of the concrete deck.

Four bridge deck configurations and post attachments were initially to be considered in this study, including: a reinforced concrete slab with posts anchored to the slab (Deck #1); a pre-stressed box with a reinforced concrete slab on top with posts anchored to the pre-stressed box and upper slab (Deck #2); a pre-stressed box with a concrete slab on top with posts anchored to the pre-stressed box (Deck #3); and a pre-stressed box with a 2-in. (51-mm) thick asphalt wearing surface placed on top with posts anchored to the pre-stressed box (Deck #4), as shown in Figure 24.

Bridge deck configuration #2 featured a 6-in. (152-mm) thick concrete slab on top of the concrete pre-stressed box girder. Assuming cast-in-place 1¼-in. (32-mm) diameter tension anchor rods, the 6-in. (152-mm) thick concrete slab would have a clear cover of 1¼ in. (32 mm) to the bottom of the slab/top of pre-stressed box girder. This minimal clear cover posed risk for reduced concrete-anchor bond and an increased risk of anchor pullout for the tension anchor rods embedded in the concrete slab. Representatives from the Illinois and Ohio DOTs proceeded to eliminate side-mount anchoring into deck configuration #2 and anchor solely into the bridge deck or into the box beam girders. Therefore, bridge deck configuration #2 was disregarded for post-to-deck attachment designs and for the design of the new bridge railing system.

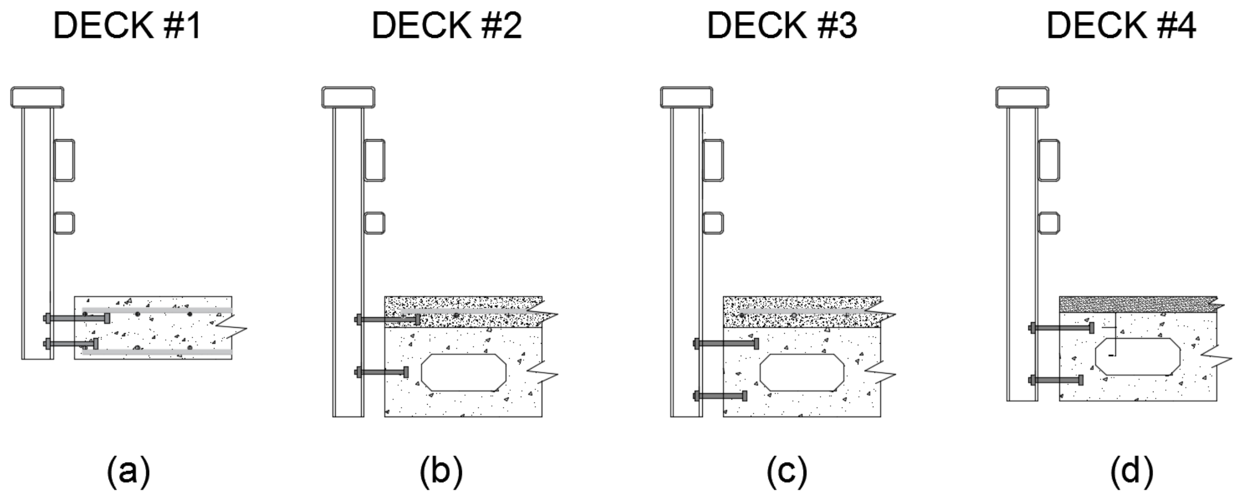


Figure 24. Four Bridge Deck Configurations with Post Attachment: (a) Reinforced Concrete Slab; (b) Pre-Stressed Box with a Reinforced Concrete Slab on Top; (c) Pre-Stressed Box with a Reinforced Concrete Slab on Top; and (d) Pre-Stressed Box with a 2-in. Asphalt Wearing Surface

As noted in Section 1.2, W6x15 (W150x22.5) steel posts were used in the Illinois and Ohio MASH 2016 TL-4 Prototype Bridge Rail to replace W6x25 (W150x37.5) steel posts found in the Illinois Side-Mounted Bridge Rail [3] to lower the impact loads transferred to the deck, and consequently, reduce the potential for bridge deck damage. Further, W6x15 (W150x22.5) posts were also utilized in the MwRSF STTR Bridge Rail [18], which was successfully crash tested under NCHRP Report 350 TL-4 test conditions.

The W6x15 (W150x22.5) steel posts near the impact region were expected to result in plastic deformations in posts at an elevation near the tension anchor rods in the three MASH 2016 TL-4 full-scale crash tests, as shown in Figure 25. Consequently, the plastic hinges at the elevation of the tension anchors would limit the magnitude of the load imparted to the bridge deck and the potential for concrete damage.

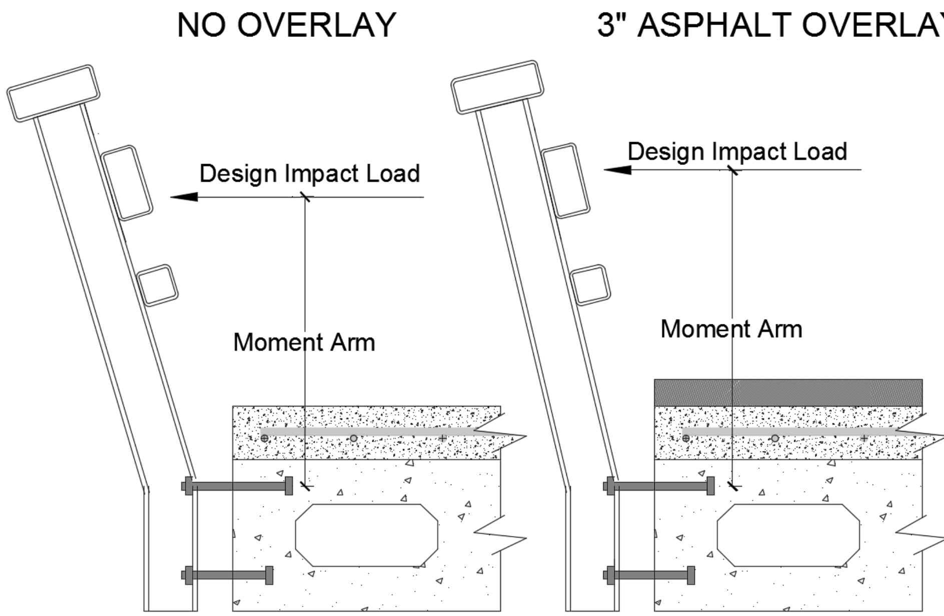


Figure 25. Steel Post Plastic Hinges at Elevation of Tension Anchor Rods

The elevation of the tension anchor rods as well as the roadway pavement overlay dictate the moment arm between the plastic hinge location of the posts and the applied lateral load. The bridge deck configuration with the largest moment arm between the tension anchor rods (i.e., plastic hinge location) and the impact load height was determined to result in the weakest lateral post resistance as well as the largest bridge rail deflection. On the other hand, the bridge deck configuration with the smallest moment arm would result in the strongest lateral post resistance as well as smallest bridge rail deflection, assuming post and/or deck rupture do not occur. The targeted vertical position for the tension anchor rods within each of the four bridge deck configurations was determined with the assistance of Illinois and Ohio DOT personnel, as shown in Figure 26.

As shown in Figure 26, bridge deck configuration #3 with the 3-in. pavement overlay was determined to be the weakest post resistance, and bridge deck configuration #1 without pavement overlay was determined to be the strongest post resistance, assuming post and/or deck rupture do not occur.

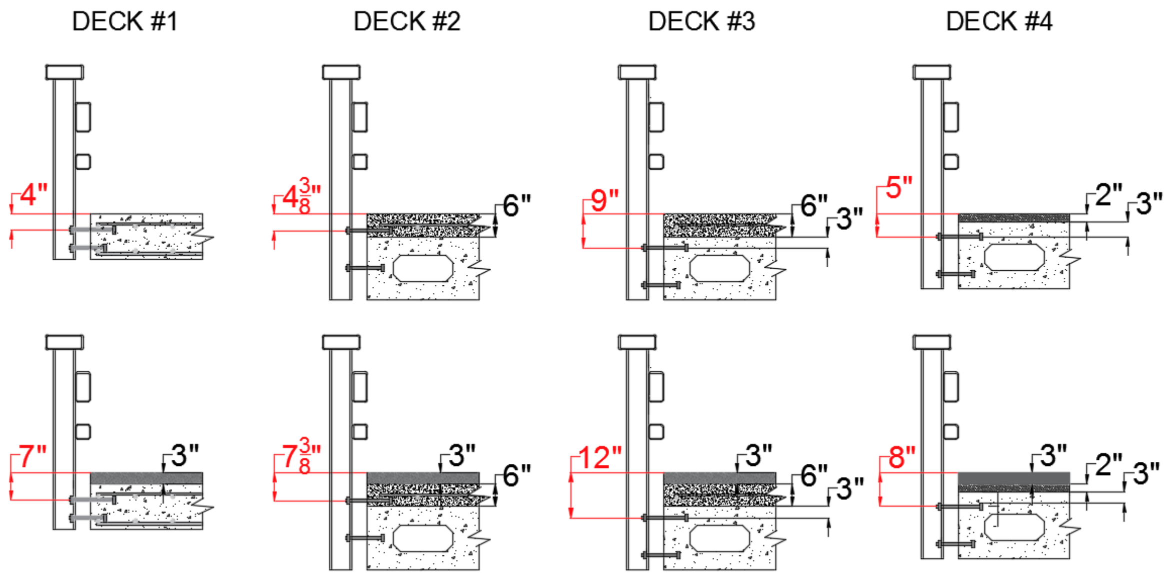


Figure 26. Preliminary Slab Decks and Post Configurations

The MASH 2016 TL-4 full-scale vehicle crash tests were conducted to investigate the barrier's ability to safely contain and redirect the test vehicles and to meet all occupant risk measures. However, the primary concern of the test designation no. 4-10 is vehicle stability and acceptable occupant risk. For test designation nos. 4-11 and 4-12, vehicle containment and stability are evaluated with pickup truck and the SUT, along with acceptable occupant risk with pickup truck. For test designation no. 4-10, the critical bridge deck configuration for full-scale crash testing was bridge deck configuration #1 without a pavement overlay. For test designation nos. 4-11 and 4-12, the critical deck configuration was bridge deck configuration #3 with a 3-in. (76-mm) roadway asphalt overlay to maximize the lateral barrier deflections and the propensity of the pickup truck and the SUT to rollover and/or override the bridge rail. The critical deck configurations with post attachments for all three crash tests are depicted in Figure 27.

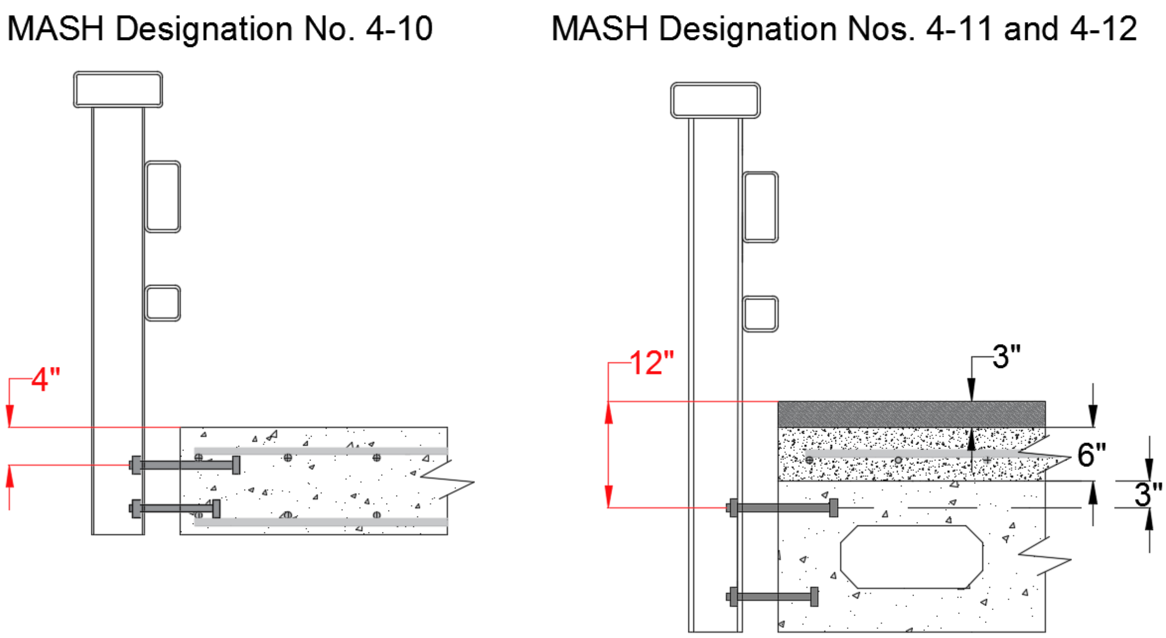


Figure 27. Critical Deck Configurations for Three MASH Crash Test Designations

3.3 Lateral and Vertical Design Impact Loading

As previously discussed in Section 2.5.7, NCHRP Report No. 22-20(2) provided two different design load categories for the MASH TL-4 SUT (TL-4-1 and TL-4-2) to recognize the effect of barrier height on the magnitude of the lateral and vertical loads [37]. TL-4-1 design loads were associated with the configuration of longitudinal barriers with a height of 36 in. (914 mm) and TL-4-2 design loads were applicable for configuring longitudinal barriers with a height greater than 36 in. (914 mm).

The new steel, side-mounted, beam-and-post, bridge rail was configured with a future 3-in. thick asphalt overlay and a resulting total effective height of 36 in. (914 mm) Thus, it was determined that the TL-4-2 lateral design load of 80 kips at a height of 30 in. (762 mm) and distributed over 5 ft (1.5 m) and the TL-4-1 vertical design load of 38 kips distributed over 18 ft (5.5 m) would both be used to create a conservative bridge railing system.

For the MASH TL-3 pickup truck and as noted previously, finite element simulations of the MASH 2270P pickup truck impacting a 32-in. (813-mm) tall, vertical rigid barrier indicated that a lateral load of 70 kips at 24 in. (610 mm) above grade applied on a 4-ft (1.22-m) span represented an upper bound of the lateral design impact load observed on simulation [37]. This lateral design impact load was also used for the development of the new bridge rail discussed herein.

3.4 Minimum Bridge Rail Heights

The minimum bridge rail height of 36 in. (914 mm) was determined for MASH TL-4 rails based on a successful full-scale vehicle crash test on a 36-in. (914-mm) tall, single-slope, concrete barrier using a SUT [41]. Therefore, the IL/OH MASH 2016 TL-4 Bridge Rail Prototype Design would need to be 39 in. (991 mm) tall before placement of a 3-in. (76-mm) thick asphalt overlay. On the other hand, the minimum barrier height for the MASH TL-3/TL-4 pickup truck was determined to be 29 in. (737 mm) based on finite element simulations of the MASH pickup truck impacting rigid barriers [35].

The lateral design impact load of the pickup truck was determined to be 70 kips at a height of 24 in. (610 mm) based on a simulation of a 32-in. (813-mm) tall vertical rigid barrier [36], and the 1-in. top rail setback could decrease and/or eliminate direct loading imparted to the top rail by the pickup truck. Therefore, it was determined to disregard the top rail when considering pickup truck stability, even though the top rail would provide structural capacity to the bridge railing system under these impact scenarios. Thus, the middle rail needed to have a minimum height of 29 in. (737 mm) to contain and redirect the pickup truck using the posts and only the middle and bottom rails. When full-scale crash testing the SUT and the pickup truck vehicles, bridge deck configuration #3 would be configured with a 3-in. (76-mm) thick roadway asphalt overlay. Therefore, a barrier height of 36 in. (914 mm) was recommended to evaluate the pickup truck so that the top of the middle rail would be located 29 in. (737 mm) above any overlay surface, as depicted in Figure 28.

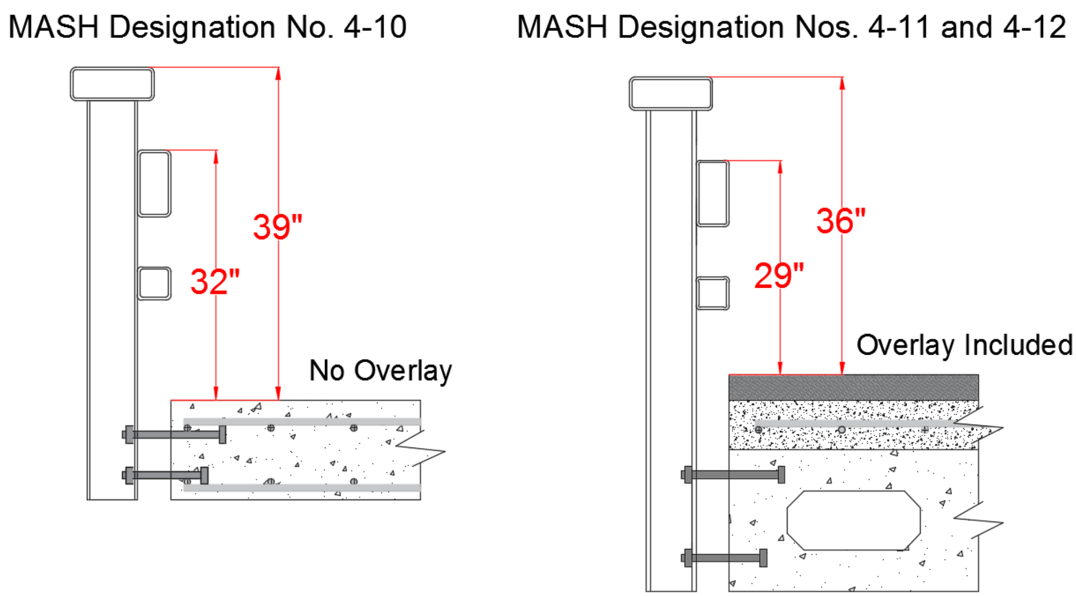


Figure 28. Minimum Rail Heights for the Three MASH TL-4 Test Designations

3.5 Top Rail Setback

From the MASH safety performance evaluation criteria, any bridge rail contact with the side windows and subsequent glass fracture for an impacting vehicle would result in a test failure.

The TxDOT C2P Bridge Rail with a top rail height of 42 in. (1,067 mm) contained and redirected the MASH 1100C small vehicle [23], as shown in Figure 29. However, the head of the dummy in the driver's side of the small car impacted and shattered the side window. The top rail may have also contributed to the shattering of the side window since the top rail contacted the bottom edge of the side window.

Figure 29. Profile and Crash Test Sequentials of MASH Test Designation No. 4-10 [23]

Therefore, the bottom side window heights of the MASH TL-4 passenger vehicles were investigated to identify the potential for the upper railings of the bridge rail to contact and fracture the side windows. The heights of the bottom edge of the side windows for the MASH small car and pickup truck were approximately near 36¼ in. (196 mm) and 52¾ in. (1,340 mm), respectively. Therefore, the small car side window was only exposed to contact with the upper rail, which has a total height of 39 in. (991 mm) when no asphalt overlay existed for MASH test designation no. 4-10. Thus, the upper rail was set back 1 in. (25 mm) to reduce concerns for side window contact with the top horizontal rail, as depicted in Figure 30.

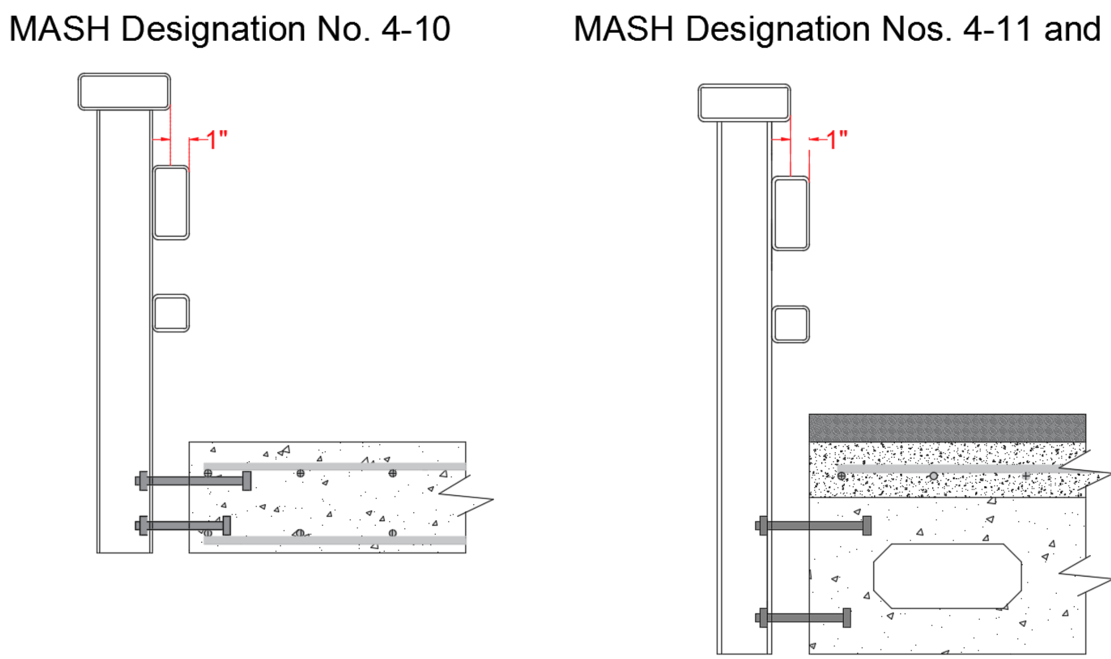


Figure 30. Top Rail Setback

3.6 Potential for Vehicle Snag

When errant vehicles impact a beam-and-post bridge rail, vehicle components, such as wheels, engine hood, and front bumper, may extend between the rails, or even below the bottom rail, and consequently, snag the vertical posts. Vehicle snag is a term used to describe a situation where a structural part of a vehicle contacts a barrier element and results in abrupt decelerations, thus potentially leading to vehicular instability, unsafe redirection or rollover, and/or significant loading to the occupants. The configuration and vertical location of the rails were essential in order to reduce the propensity for MASH TL-4 vehicles to snag on the posts of the new steel, beam-and-post, side-mounted, bridge rail. Therefore, the new bridge rail was configured using optimum rail sizes and vertical locations that would prevent, or at least greatly reduce, wheel snag against the posts under the bottom rail and bumper snag between the horizontal rails. The risk of engine hood and quarter panel snag on posts between the middle and upper rails would be minimized as best as possible with the use of a small vertical opening.

3.6.1 AASHTO LRFD Bridge Design Specifications

The AASHTO LRFD *Bridge Design Specifications* [30] provided preferred geometric relationships for configuring beam-and-post bridge rails in order to reduce the potential for vehicle snag, which was based on data obtained from systems previously crash tested under NCHRP Report No. 230 impact conditions [15]. The geometric relationships included vertical rail openings, ratio of vertical rail contact surface to overall barrier height, and post setback distances for beam-and-post bridge rails. The potential for vehicle snag existed with the vehicle's wheel, bumper, quarter panel, and engine hood, which correlated to the geometry of the railing.

The risk for a vehicle's wheel, bumper, quarter panel, or engine hood to snag on a post between and/or below rails is shown in Figures 31 and 32. The vertical clear opening, C, depicts acceptable rail openings for a beam-and-post bridge rail, which has often varied as a function of vertical position of rails, as noted in Figure 31. Larger openings have been accommodated below the bottom rail, while smaller openings have been used between rails. The propensity for a vehicle to snag on a post with respect to the summation of the depths of the vertical front faces of the rails and/or the depth of the concrete curbs, $\Sigma A_i/H$, is depicted in Figure 32. The definition of post setback, S, pertains to the distance between the front face of the railings to the front face of the posts, as shown in Figures 31 and 32.

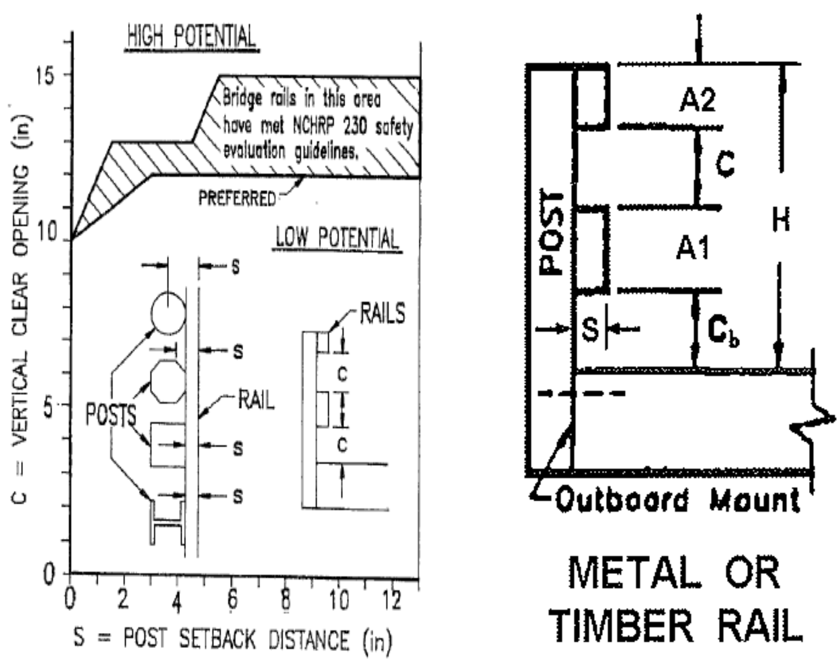


Figure 31. AASHTO LRFD Potential Wheel, Bumper, Quarter Panel, and/or Engine Hood Impact with Post [30]

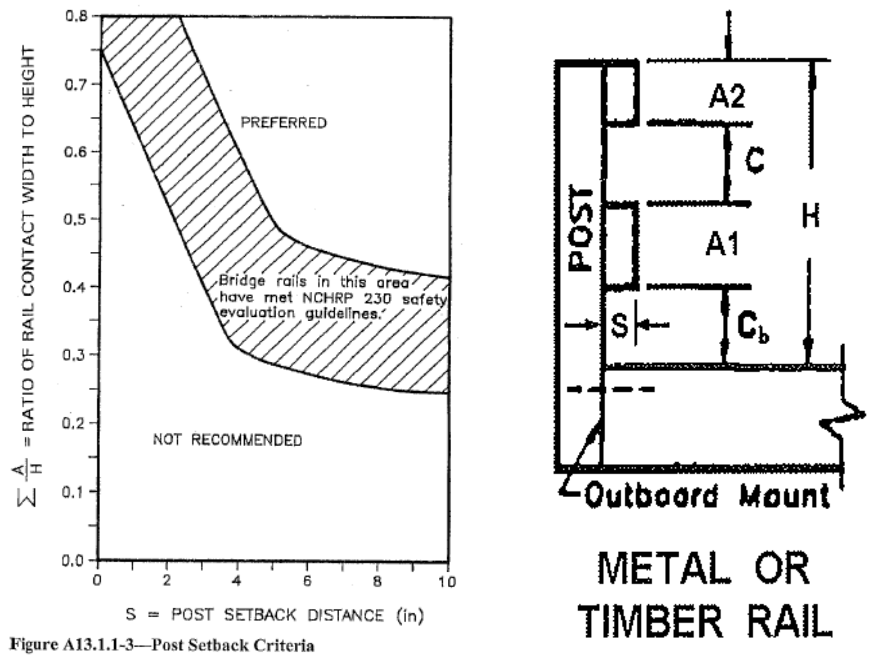


Figure A13.1.1-3—Post Setback Criteria

Figure 32. AASHTO LRFD Post Setback Criteria [30]

The published vehicle snag geometric relationships have not been updated to include crash data corresponding with NCHRP Report 350 and MASH test vehicles and impact conditions. Therefore, a research effort was performed to update the two charts with the bridge rails found in the literature review as well as the IL/OH MASH 2016 TL-4 Bridge Rail Prototype Design to better predict potential snag risks with posts.

For each beam-and-post bridge rail, the vertical clear opening, post setback distance, and the ratio of vertical rail contact width to overall barrier height were determined and displayed in both plots. Bridge rails that were only crash tested with SUTs were disregarded as the smaller impact angle, deeper frontal bumpers, and larger wheel diameters as compared to MASH TL-4 passenger vehicles did not represent a high potential for snag on the posts of the new bridge rail. The geometrics of the additional beam-and-post bridge rails are shown in Table 14. AASHTO guidance plots were updated for the bridge railing geometry and are shown in Figures 33 through 36.

Table 14. Literature Review Beam-and-Post Bridge Rail Geometrics

Beam-and-Post Bridge Rail System	Reference	Post Setback Distance, S, (in.)	Maximum Vertical Clear Opening, C, (in.)	Ratio of Vertical Contact Width to Overall Barrier Height, $\Sigma A_i/H$
Illinois Side-Mounted	3	4.00	12.00	0.44
MwRSF STTR	13	6.00	13.84	0.43
California ST-70	15	6.00	8.00	0.33
Verrazano-Narrows	16	6.00	6.00	0.43
TxDOT T131	17	4.00	11.00	0.42
TxDOT C2P	18	4.00	9.50	0.42
Massachusetts S3 TL-4	19	5.00	8.00	0.57
Caltrans ST-10	20	5.50	10.00	0.42
PosBarrier-B	21	6.00	13.40	0.49
Caltrans ST-20	22	3.50	8.27	0.44
TxDOT T131RC	23	6.00	10.00	0.56
TxDOT Picket Rail	24	3.50	8.00	0.47
IL/OH Prototype Design	-	4.00	11.00	0.41

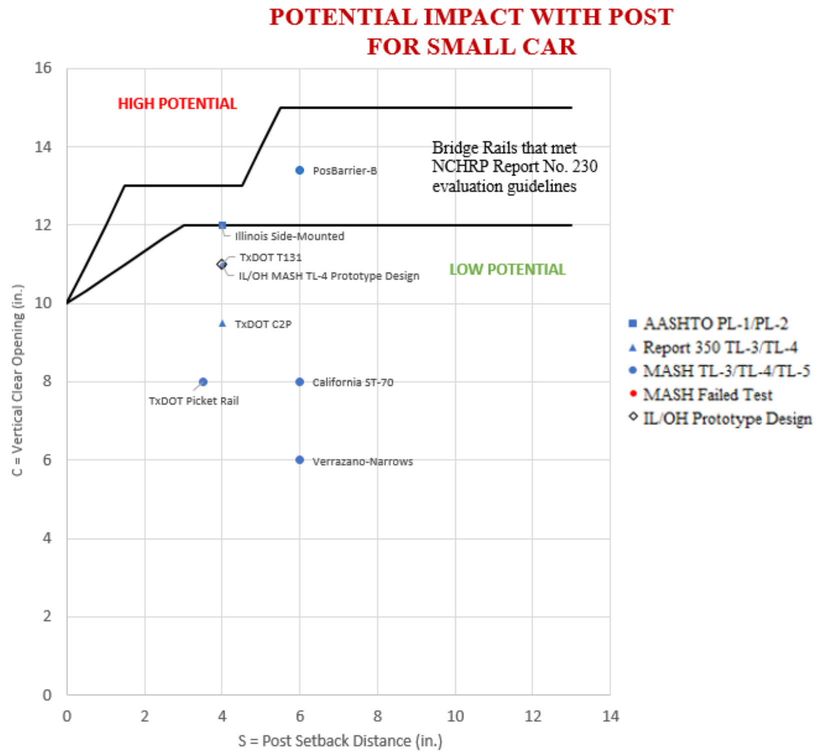


Figure 33. AASHTO LRFD Potential Wheel, Bumper, Quarter Panel, and/or Engine Hood Impact with Post for Small Car

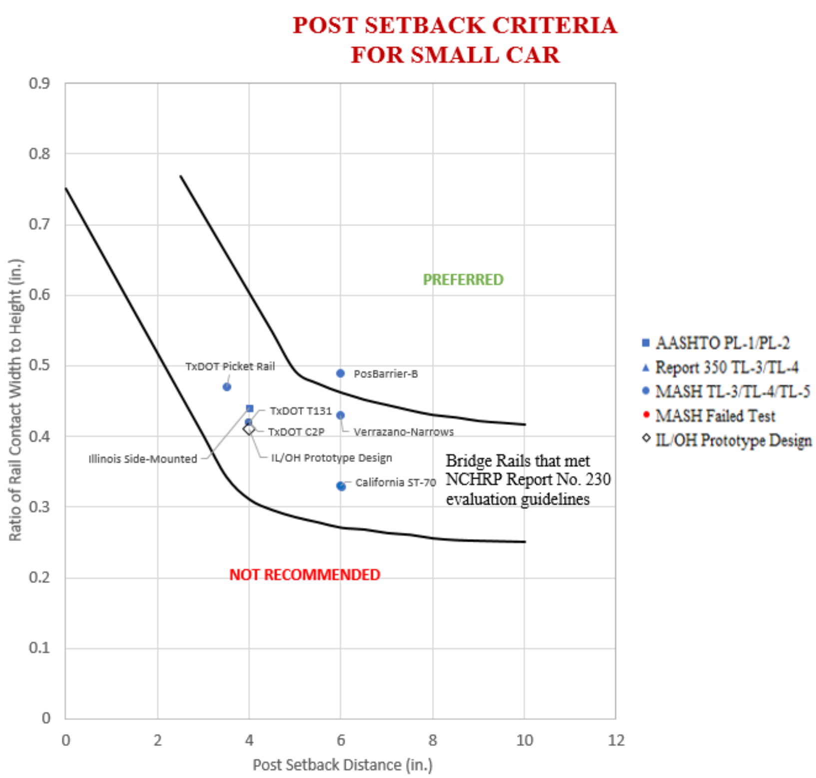


Figure 34. Post Setback Criteria for Small Car

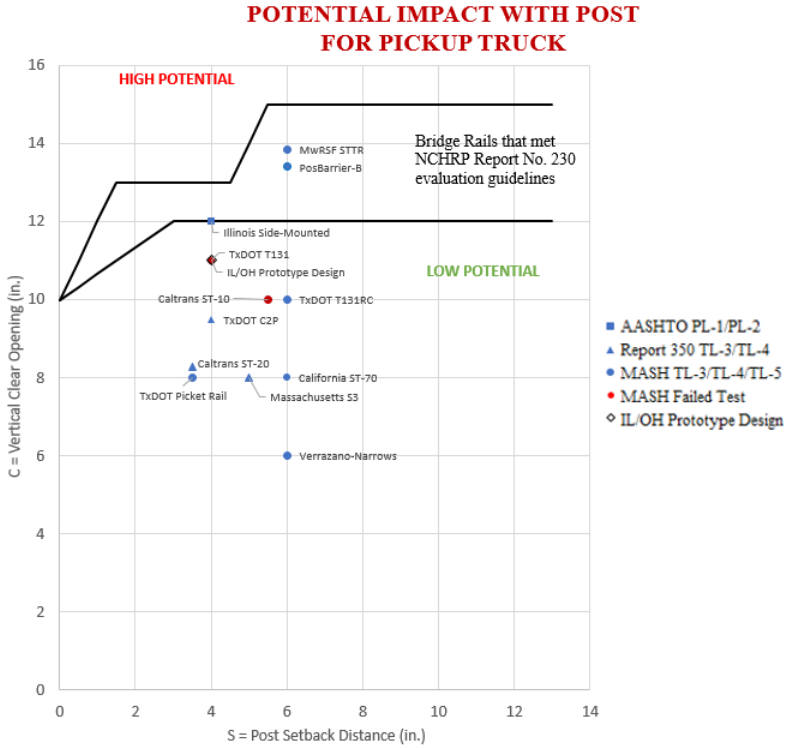


Figure 35. AASHTO LRFD Potential Wheel, Bumper, Quarter Panel, and/or Engine Hood Impact with Post for Pickup Truck

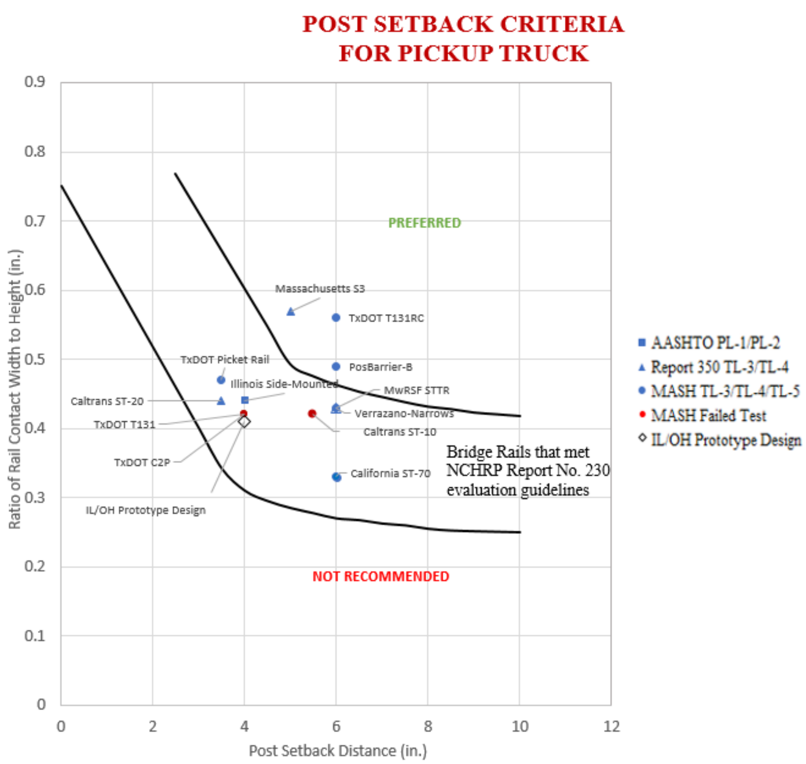


Figure 36. Post Setback Criteria for Pickup Truck

For the "Potential Wheel, Bumper, and/or Engine Hood Impact with Post" and the "Post Setback Criteria" plots, all of the crash tested bridge rail configurations were located outside of the "potential zone" for vehicle snag. Thus, the development of the new bridge rail continued with the selection of a maximum vertical clear opening of 12 in. (305 mm) and a minimum post setback of 4 in. (102 mm), which were deemed appropriate using the "Potential Impact with Posts" plots. Further, a 12-in. (305-mm) vertical clear opening was found in the Illinois Side-Mounted Bridge Rail [3], which was successfully crash tested, and evaluated under AASHTO *Guide Specifications for Bridge Railings* Performance Level 2 (PL-2) impact conditions [4]. Furthermore, a minimum ratio of rail contact width to total height of 0.4 with a minimum post setback of 4 in. (102 mm) was established from the "Post Setback Criteria" plots.

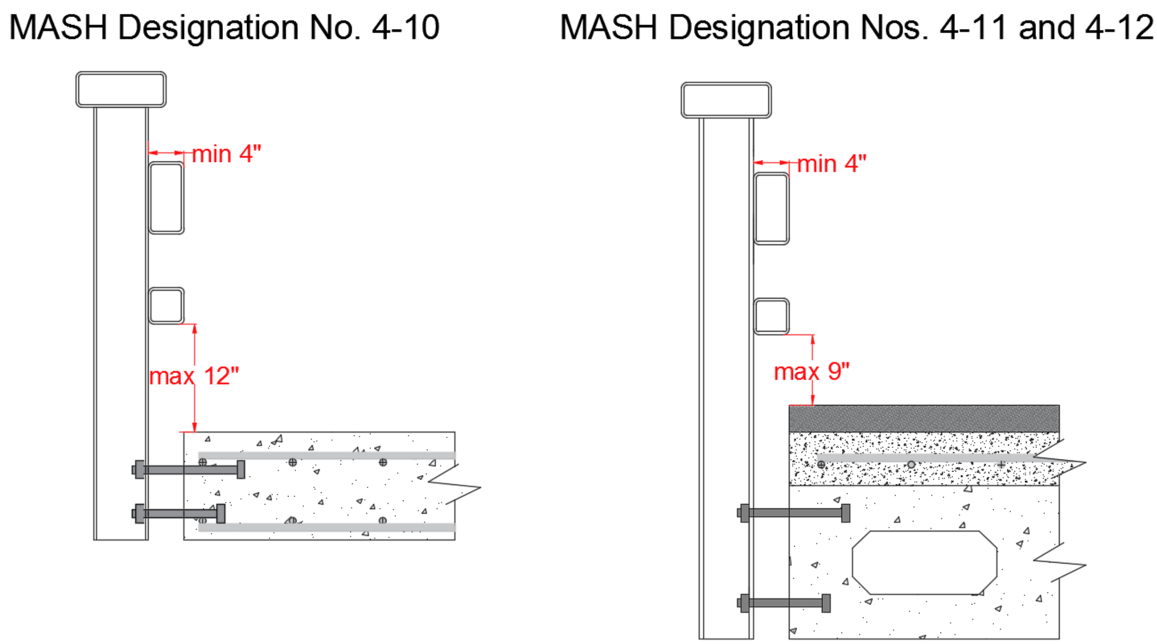


Figure 37. 12-in. Maximum Vertical Clear Opening and 4-in. Minimum Post Setback Based on AASHTO Specification Guidance Plots

3.6.2 Bumper Rigid Body Configurations

Vehicles impacting beam-and-post bridge rails may snag against the posts with their front bumpers extending between rails or getting stuck between rails, thus resulting in vehicle instabilities. Under oblique vehicular impacts, the bumper covers are easily deformed and crushed without providing significant load transfer to the chassis of the vehicle. Consequently, bumper covers may extend between rails and may contact the posts without much threat to the stability of impacting vehicles. As the bumper cover crushes or detaches away from the impacting vehicle, the structural components of the bumpers become exposed to contact with the rails or the posts, thus potentially causing vehicle instability that may lead to rollover or an unsafe vehicle redirection. The configurations of the structural components of the bumpers for the MASH TL-4 vehicles are shown in Figure 38 and further detailed in Table 15.

Figure 38. MASH Small Car, Pickup Truck, and SUT Front Bumper Rigid Bodies

Table 15. Typical Front Bumper Structural Component Heights

Vehicle Type	Description of Structural Components of Front Bumper	Bumper Bottom Edge Height (in.)	Bumper Top Edge Height (in.)
Small Car (1100C)	48-in. x 3⅞-in. x 2-in. Frame	16.25	20.125
Pickup Truck (2270P)	Two 6-in. Square Mounting Brackets	18.375	24.375
Single-Unit Truck (10000S)	38¼-in. x 10⅜-in. x 4½-in. Frame	23.125	33.5

The geometries of the structural components of the front bumper from the three MASH TL-4 vehicles were plotted next to the IL/OH MASH 2016 TL-4 Prototype Bridge Rail with and without a 3-in. (76-mm) thick roadway overlay, as shown in Figure 39. The geometries and heights of the steel rails were analyzed regarding the potential of the vehicle to snag against the posts.

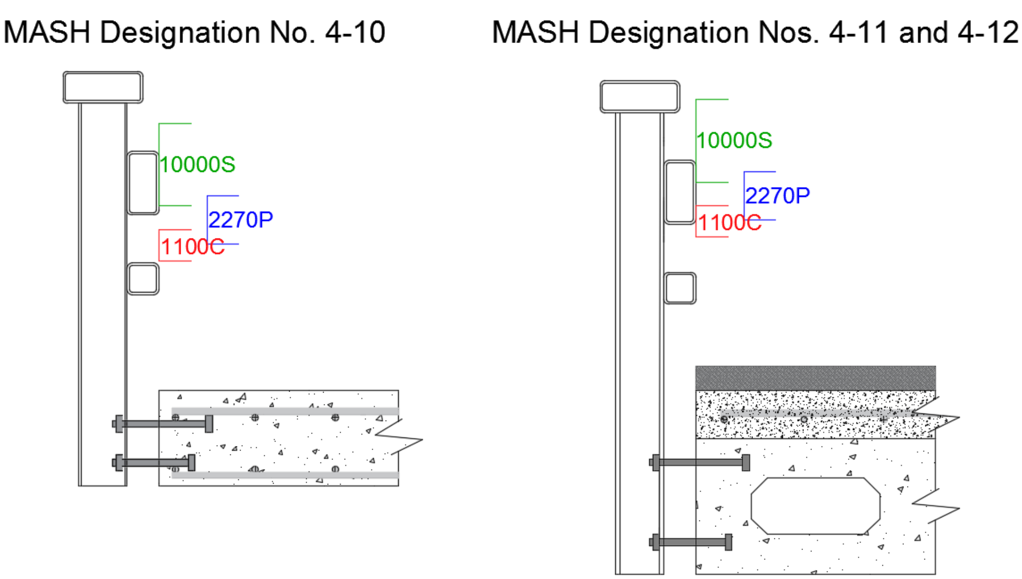


Figure 39. Structural Components of Front Bumper Adjacent to IL/OH MASH 2016 TL-4 Prototype Bridge Rail

As shown in Figure 39, the structural components of the front bumpers for the pickup truck and the SUT would not likely be fully exposed to the front face of the post. The front bumper of the small car would likely be exposed to contact the post of the IL/OH Prototype Bridge Rail between the lower two rails when no roadway overlay existed. Therefore, the geometries and locations of the steel rails were investigated to prevent bumper snag between the lower and middle rails. In order to avoid small car vehicle snag on the posts, the rail opening between the lower and the middle rails would likely need to range between 4 in. (102 mm) to 6 in. (152 mm).

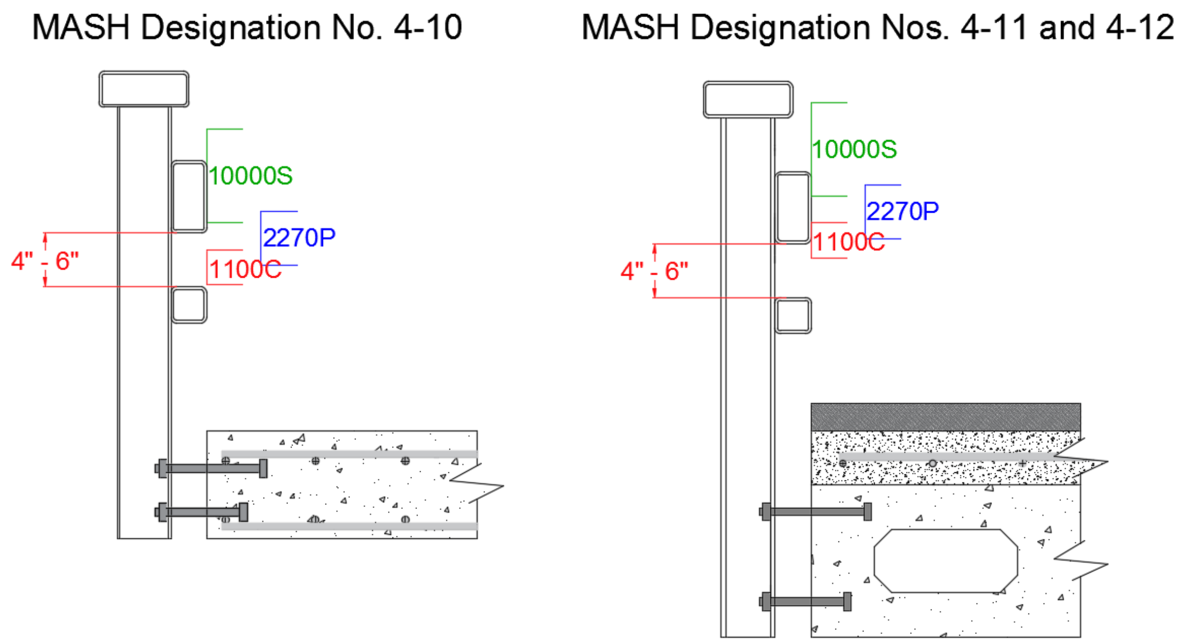


Figure 40. Preferred 4-in. to 6-in. Vertical Rail Opening for Small Car

3.7 Design Criteria from Sponsors

Personnel from the Illinois and Ohio DOTs provided several design criteria for use in the development of the new bridge rail in order to improve constructability, simplify acquisition of material, and reduce system cost. These design criteria would be used to modify the IL/OH Prototype Bridge Rail throughout the research and development effort.

For the installation of the bridge rail, the steel rails would likely be the heaviest components of the system. Depending on rail length and post spacing, the steel rails could be heavy and difficult for workers to carry and install without the use of large machinery. Thus, personnel from the Illinois and Ohio DOTs established a maximum weight for each steel rail of 500 lb in order to not require large machinery on the bridge deck during bridge rail installation, which could pose risks to the structural integrity of the bridge deck. In order to maintain a maximum rail weight of 500 lb, each rail element would likely be limited to one to three increments in the post spacing.

Personnel from the Illinois and Ohio DOTs also requested that the middle and bottom steel rails utilize an identical cross section to standardize as much material as possible, which would simplify material acquisition. Further, the rail height options were 6 in. (152 mm), 7 in. (178 mm), and 8 in. (203 mm), while the rail depths were 4 in. (102 mm), 5 in. (127 mm), and 6 in. (152 mm) to provide adequate post setback. The vertical opening snag potential between rails depend on the rail depths and heights of the horizontal tubes. The IL/OH Prototype Bridge Rail was exposed to vehicle snag with the rigid frame of the small car in the rail opening between the lower and middle rails. The Illinois and Ohio DOT personnel advised the research team to disregard steel rails with odd dimensions (i.e. 7 in. (178 mm) depth, 5 in. (127 mm) width). Therefore, the lower and middle rails were limited to a depth of 8 in. (203 mm) and widths of 4 in. (102 mm) and 6 in. (152 mm).

A minimum rail thickness of ¼ in. (6.4 mm) was also specified for the three steel rails to prevent crushing of steel rails with thicknesses of $^{3}/_{16}$ in. (4.8-mm) or less. The crushing of the steel rails could accentuate large plastic deformations that may lead to excessive vehicle instabilities and rollover.

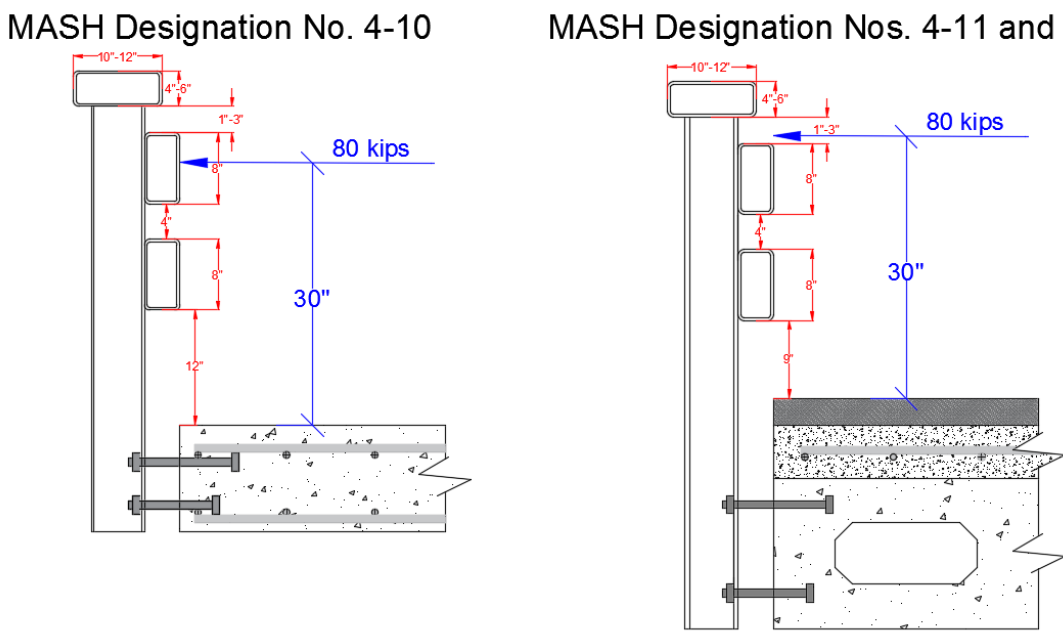


Figure 41. Summary of Design Criteria

4 DESIGN METHODOLOGY

4.1 Introduction

Historically, the two most common analysis methods for configuring steel, beam-and-post bridge rails are based on: (1) 2-D and/or 3-D nonlinear, finite element simulations of vehicle models impacting a barrier system and (2) an inelastic analysis of the collapse mechanism of a bridge rail under design impact loading. The design of the new steel, side-mounted, beam-and-post, bridge rail was based on the plastic collapse mechanism of the bridge rail system. However, the two design methodologies were briefly examined to identify their applications for the development of crashworthy bridge rails.

4.2 2-D and/or 3-D Nonlinear, Finite Element Computer Simulation

Computer simulation with various codes, such as BARRIER VII [41-42] or LS-DYNA [43], have been used by roadside safety researchers to better understand the crashworthiness of bridge rails under impact events. The modeling of nonlinear, physical contact behavior requires great care in full-scale crash test simulations. Nonlinear physical behavior is very complicated, and capturing this behavior with mathematics is not an exact science [44]. However, nonlinear finite element computer simulation plays an important role in the development of roadside safety hardware. It serves as a research tool to identify critical failure modes, such as vehicle rollover, vehicle snag, vehicle pocketing, as well as component fracture, and material yielding.

Researchers utilize computer simulation differently to study impact events with roadside safety hardware. Occasionally, dynamic component testing of bridge rail components are conducted to evaluate specific impact performance. When component test data is available, the researcher may use it to validate the computer model of the bridge rail. On the other hand, if no component test data exists, the researcher may use validated computer simulations from testing on a similar bridge rail to prepare a model for the new prototype to extrapolate system behavior with finite element analysis. If a bridge rail system was subjected to full-scale crash testing, computer simulations could be performed to develop a model that predicts similar behavior. Then, impact simulations could be conducted to evaluate design modifications, minimum rail height, propensity for vehicle snag, occupant risk, barrier deflection, working width, load distribution throughout barrier components, etc.

In one recent example, LS-DYNA computer simulation was used to assist with the design of a combination bridge separation barrier [45]. For this study, Iowa DOT desired that MwRSF researchers design and crash test a combination bridge separation barrier according to the MASH TL-2 safety performance criteria. Nonlinear finite element simulations were performed to determine a recommended height for the vertical concrete parapet and to identify the impacting vehicle’s extent over the front face of the barrier to mitigate its interaction with the posts and rail as well as to properly place the rail away from the parapet face. For the model validation, a vertical concrete parapet model was created to match the crash testing details from a Texas A&M Transportation Institute study that was performed according to the MASH TL-3 safety criteria [46] as shown in Figure 42. With the validated model, simulations were conducted to observe vehicle and barrier performance at varying heights as well as later performance with the attached posts and rail.

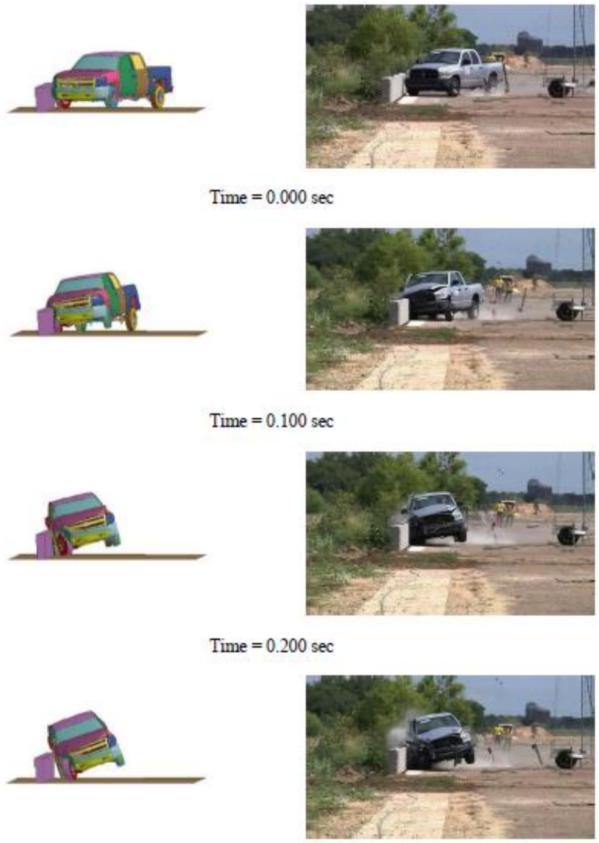


Figure 42. Sample Validation Sequence from Computer Simulation and TTI Crash Test [46]

4.3 Plastic Collapse Mechanism

Historically, the development of steel beam-and-post bridge rails has followed guidance contained in various editions of the AASHTO LRFD *Bridge Design Specifications* [30]. Herein, steel beam-and-post bridge rails were analyzed and designed using an iterative process that determined the system capacity by examining multiple plastic collapse mechanisms for each combination of rail and post sections. The plastic collapse mechanism or inelastic analysis method was used to determine the bridge rail's lateral resistance for each number of spans involved in plastic collapse. Upon review of the findings, the number of affected spans with the lowest lateral capacity was found to provide the critical or controlling bridge railing strength. This method has also been described in various publications from AUSTROADS [47] and TTI [48]. A one-span collapse mechanism involves plastic hinges at the midspan and end sections of the rails located above the two support posts but only in rails, as shown in Figure 43. A two-span collapse mechanism involves plastic hinges in the rails at the midspan of two spans (i.e., middle post) and at the end sections of the rails as well as at the base of the middle post, as depicted in Figure 43. A three-span collapse mechanism involves plastic hinges in the rails at the midspan of three spans and at the end sections of the rails as well as at the bases of the middle two posts, as shown in Figure 43. Note that a bridge railing system with more than three spans was also analyzed.

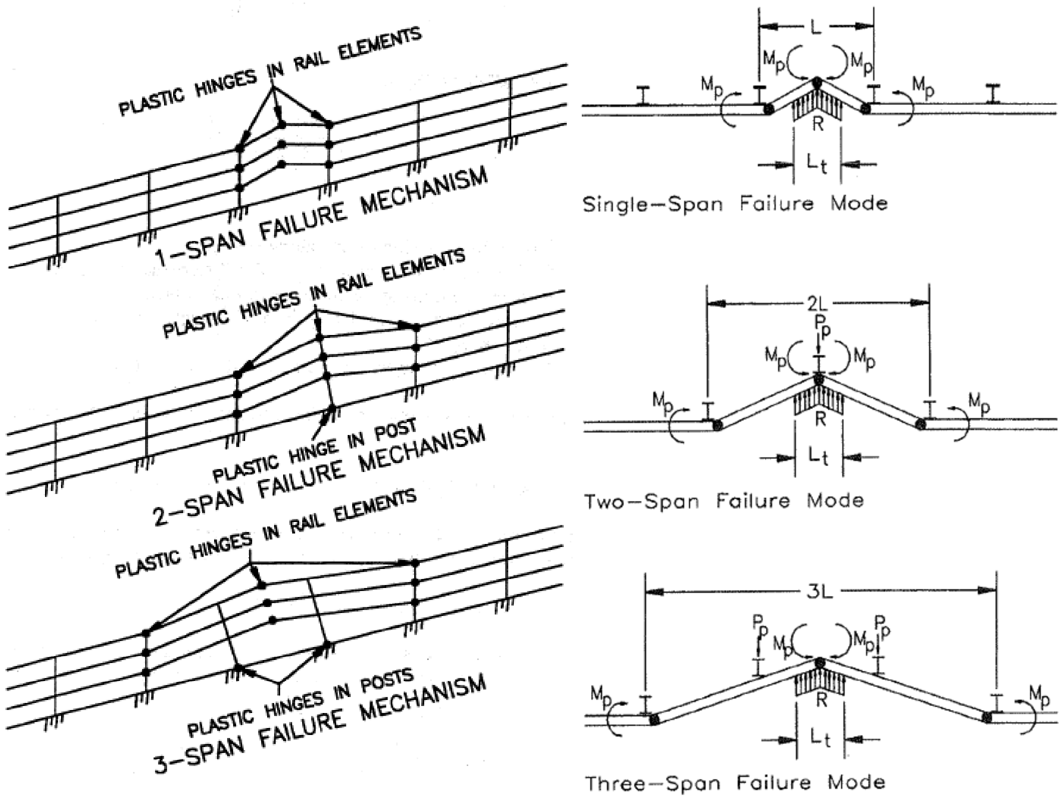


Figure 43. One-Span, Two-Span, and Three-Span Plastic Collapse Mechanisms [30, 33, 48]

The lateral bridge rail resistance with the contribution of the steel rails and posts at a particular height should be taken at the least value determined from Equations 2 and 3. Note that sample units are provided below for the provided variables.

For failure modes involving an odd number of rail spans [30]:

$$R = \frac{16M_{P\ RAILS} + (N-1)(N+1)P_{POST}L}{2NL - L_T} \tag{2}$$

For failure modes involving an even number of rail spans [30]:

$$R = \frac{16M_{P\ RAILS} + N^2 P_{POST}L}{2NL - L_T} \tag{3}$$

where:

N = number of rail spans;

R = total lateral resistance of the rails and posts at effective height of rails, $\overline{Y}_{RAILS}$ (kips);

$M_{P\ RAILS}$ = plastic moment capacity of all rails contributing to a plastic hinge (kip-in.);

P_{POST} = shear force on a single post which corresponds to $M_{P\ POST}$ and located $\overline{Y}_{RAILS}$ above deck or at effective height of rails (kips);

L = post spacing or single span (in.); and

L_T = distributed length of lateral design vehicle impact load (in.).

The plastic moment capacity for all rails, $M_{P\ RAILS}$, is represented by the summation of the individual plastic moments of the rails, as determined in Equation 4 and shown in Figure 44. The individual plastic moment for each rail was determined by equation F7-1 of the AISC Steel Construction Manual [49]. A strength reduction factor, Ø, of 0.9 was used in order to account for uncertainty in material yield strength and cross-section geometries as well as less accurate method of analysis. The horizontal rails were specified to use ASTM A500 Grade C steel material. The specified minimum yield strength, F_Y, of the rails was 50 ksi [49-50]. The plastic section modulus, Z, of each rail was obtained from the AISC *Steel Construction Manual* [49] section properties, specifically, Table 1.11. For the posts, ASTM A992 Grade 50 steel material was specified [49], which pertained to a minimum yield strength of 50 ksi.

$$M_{P\ RAILS} = \Sigma\,[\emptyset\ F_Y\ Z] \qquad (\text{kip}-\text{in.}) \tag{4}$$

where:

Ø = reduction factor, 0.9;
F_Y = minimum specified yield stress, (ksi); and
Z = plastic section modulus of rail, (in.^{3})

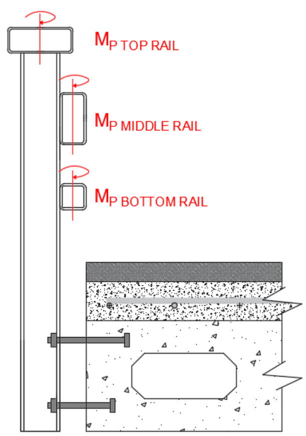


Figure 44: Plastic Moment Capacities of Rails.

The effective height of the rails, $\overline{Y}_{RAILS}$, corresponds to the combined height for all rails depicted in Figure 45 using the plastic moment capacity of each rail at its corresponding height with regard to the plastic moment capacity for each rail, as stated in Equation 5.

$$\overline{Y}_{RAILS} = \frac{\Sigma(M_{Pi} * h_i)}{M_{P\ RAILS}} \tag{5}$$

where:

M_{Pi} = plastic moment capacity of rail ith (kip-in.);
H_i = height of ith rail from location of plastic hinge (in.); and
$M_{P\ RAILS}$ = plastic moment capacity of all rails combined (kip-in.).

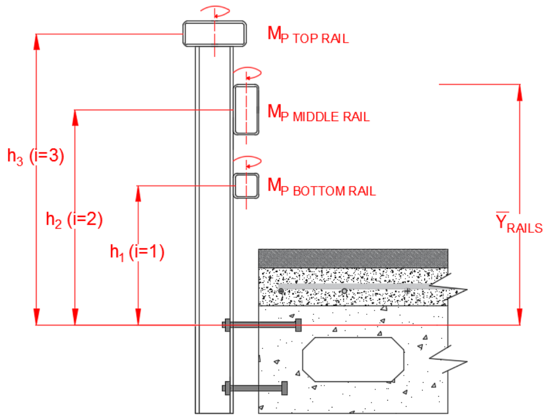


Figure 45. Effective Height of Rails

The shear force on a single post, P_{POST}, corresponds to the plastic moment capacity of the post, $M_{P\ POST}$, divided by the effective height of the rails, $\overline{Y}_{RAILS}$, as stated in Equation 6 and as shown in Figure 46.

$$P_{POST} = \frac{M_{P\ POST}}{\overline{Y}_{RAILS}} \tag{6}$$

where:

$M_{P\ POST}$ = plastic moment capacity of post section (kip-in.) and
$\overline{Y}_{RAILS}$ = effective height of rails (in.).

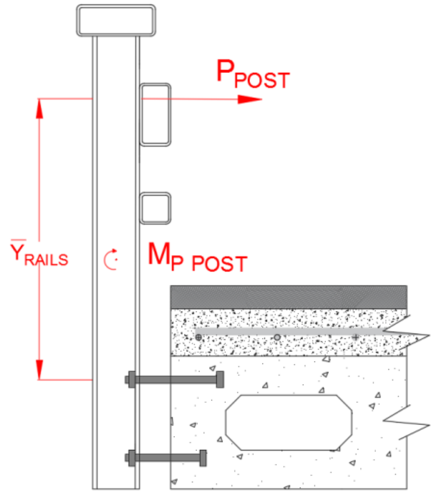


Figure 46. Shear Force on a Single Post

The plastic moment capacity of each post, $M_{P\ POST}$, corresponds to the minimum specified yield stress of the steel, F_Y, multiplied by the plastic section modulus of the rail, Z, and a reduction factor, Ø, of 0.9, as shown in Equation 7.

$$M_{P\ POST} = \text{Ø}\, F_Y\, Z \qquad (\text{kip} - \text{in.}) \tag{7}$$

where:

Ø = reduction factor, 0.9;
F_Y = minimum specified yield stress, (ksi); and
Z = plastic section modulus of rail, (in.^{3})

Equations 2 and 3 were used to determine the lateral resistance of the bridge railing system consisting of rails and posts, R, at the effective height of the rails, $\overline{Y}_{RAILS}$. However, it was also necessary to calculate the lateral resistance of the bridge rail at the height of the design impact load, H_{DESIGN}, for critical vehicles such as the pickup truck and SUT. Since it was determined that the lateral capacity of the bridge rail was linearly proportional to the distance away from the post mounting or yield location, the lateral capacity of the bridge rail at a design impact load height, R_{DESIGN}, is calculated using Equation 8 and shown in Figure 47.

$$R_{DESIGN} = R * \frac{\overline{Y}_{RAILS}}{H_{DESIGN}} \tag{8}$$

where:

R_{DESIGN} = lateral resistance of bridge railing system at design impact load height, H_{DESIGN}, (kips);
R = lateral resistance of bridge railing system (rails and posts) at the effective height of rails, $\overline{Y}_{RAILS}$ (kips);
$\overline{Y}_{RAILS}$ = effective height of rails, (in.); and
H_{DESIGN} = design impact load height (in.).

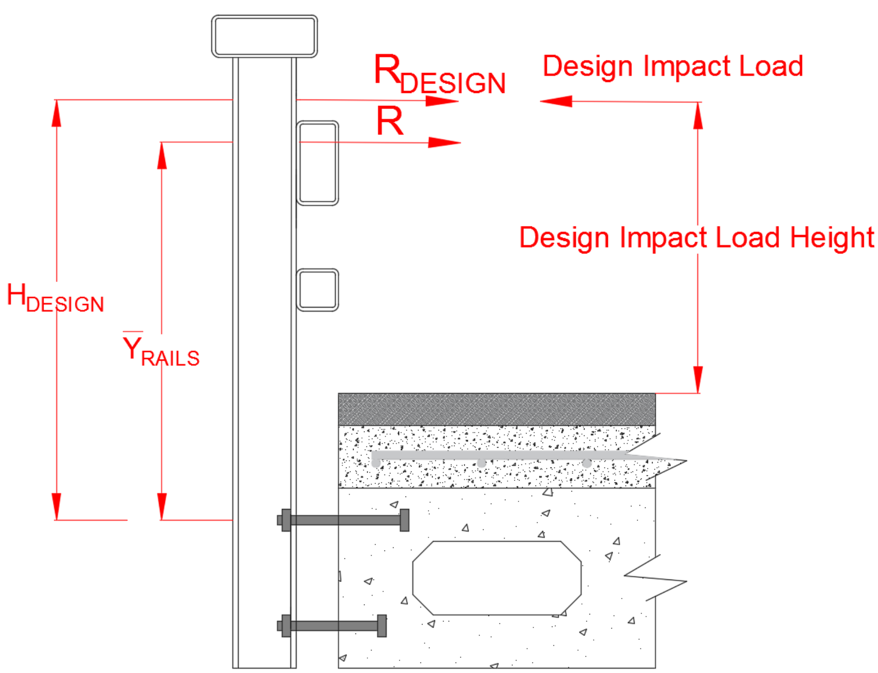


Figure 47. Lateral Resistance of Bridge Railing System at Height of Design Impact Load

5 BRIDGE RAIL ANALYSIS AND DESIGN

5.1 Overview

As noted in Section 4.3, the plastic collapse method or inelastic analysis was used for the analysis and design of steel, side-mounted, beam-and-post, bridge rail configurations capable of resisting MASH TL-4 SUT impact events. Chapter 5 was intended to explain the chronological process of the development of the bridge rail from the beginning of the project. The design criterion defined in Chapter 3 were a product of the completion of the work described herein, which were produced by research findings and sponsor feedback.

The design process for the bridge rail configurations started with the development of guidance plots, which specified the required plastic moment for all of the rails at the design impact load height and for three bridge deck types utilized by the Illinois and Ohio DOTs. These plots provided guidance to design preliminary bridge rail configurations. Improved bridge rail configurations were designed considering the critical bridge deck type for SUT impact events; bridge deck configuration #3 with a 3-in. (76-mm) thick asphalt overlay.

For the development of final bridge rail configurations, an analysis of the lateral bending resistance of the two lower rails within a single span was performed for pickup truck impact events prior to post yielding and impact loading imparted to the top rail. Since the lower two railings were to be equal in size and thickness, this analysis resulted in rail sections that were unable to resist pickup truck design lateral loading within a single span.

After the analysis of system weight per foot and preferences from representatives of Illinois and Ohio DOTs, the most efficient design for the new bridge rail in terms of weight per foot and constructability was identified and later prepared for full-scale crash testing and evaluation.

5.2 Plastic Collapse Mechanism Method for IL/OH Bridge Rail Prototype Design

Using a plastic collapse mechanism or inelastic analysis, the overall lateral resistance of the IL/OH MASH 2016 TL-4 Prototype Design was calculated. Since vehicular impacts transfer dynamic loading to the bridge rail system, a dynamic magnification factor (DMF) was used to account for actual yield strengths higher than nominal values and strain rate effects in select bridge rail components. A DMF of 1.5 has been typically used for posts utilized in steel, beam-and-post, bridge rails [5]. This factor is empirical, and it was based on observations of W6x9 (W150x 13.5) posts with a yield stress of 36 ksi (248 MPa) anchored to rigid foundations subjected to a cantilever load condition [5, 51]. Since the posts of the bridge rail were to be bolted or welded to the mounting brackets on the side of the concrete bridge deck, DMFs of 1.0 and 1.5 were considered for the posts when calculating the lateral redirective capacity of bridge rail configurations. The desired DMF was incorporated into Equation 7 to calculate the plastic moment capacity of a post, as depicted in Equation 18:

$$\mathrm{M_{P\ POST}} = \emptyset * F_Y * \mathrm{Z} \qquad (\mathrm{kip-in.}) \tag{7}$$

$$\mathrm{M_{P\ POST\ DMF}} = \emptyset * DMF * F_Y * \mathrm{Z} \qquad (\mathrm{kip-in.}) \tag{18}$$

where:

Ø = reduction factor, 0.9;
DMF = dynamic magnification factor (1.0, 1.5);
F_Y = minimum specified yield strength, (ksi); and
Z = plastic section modulus of rail, (in.^{3})

The lateral resistances of the IL/OH MASH 2016 TL-4 Prototype Design with DMFs for the posts equal to 1.0 and 1.5 and no asphalt overlay were then calculated. These lateral barrier resistances were generated for comparison to the design impact loading of the pickup truck and the SUT, as specified in Section 3.5. Therefore, the SUT lateral design load of 80 kips (356 kN) at a height of 30 in. (762 mm) was distributed over 5 ft (1.5 m), and the pickup truck lateral design load of 70 kips (311 kN) at a height of 24 in. (610 mm) was distributed over 4 ft (1.2 m) [37].

As stated in Section 3.4 and for design purposes, the top rail was disregarded when considering pickup truck stability due to the design impact load height of 24 in. (610 mm) and the 1-in. (25-mm) top rail setback, even though the top rail would contribute to the structural capacity of the bridge rail system. Therefore, the lateral barrier resistances of the IL/OH Prototype Bridge Rail with DMFs equal to 1.0 and 1.5 were initially calculated using only the contribution of the lower two rails supported by posts. Later, the pickup truck analysis effort included both two and three horizontal rails for determining lateral barrier capacity.

Examples of the analysis and design process using the plastic collapse mechanism or inelastic analysis on the IL/OH Prototype Bridge Rail Design for SUT impacts with the contribution of three rails and pickup truck impacts with the contribution of only the two lower rails as well as all three rails are shown in the following sections when using a DMF of 1.0. Microsoft Excel spreadsheets were also developed to utilize a plastic collapse mechanism to calculate the lateral barrier resistance of the MASH 2016 TL-4 Prototype Bridge Rail with DMFs of 1.0 and 1.5. For the prototype system, a post spacing of 6 ft - 3 in. (1.91 m) and an anchor location 4 in. (102 mm) below the deck's surface were selected. The plastic collapse mechanism spreadsheets for these examples are shown in Table B-1 and Table B-2 in Appendix B.

5.2.1 Example Problem No. 1 – Estimate Barrier Capacity for IL/OH Prototype Bridge Rail for Single-Unit Trucks with Three Rails and DMF=1.0

<u>Step 1 - System information:</u>

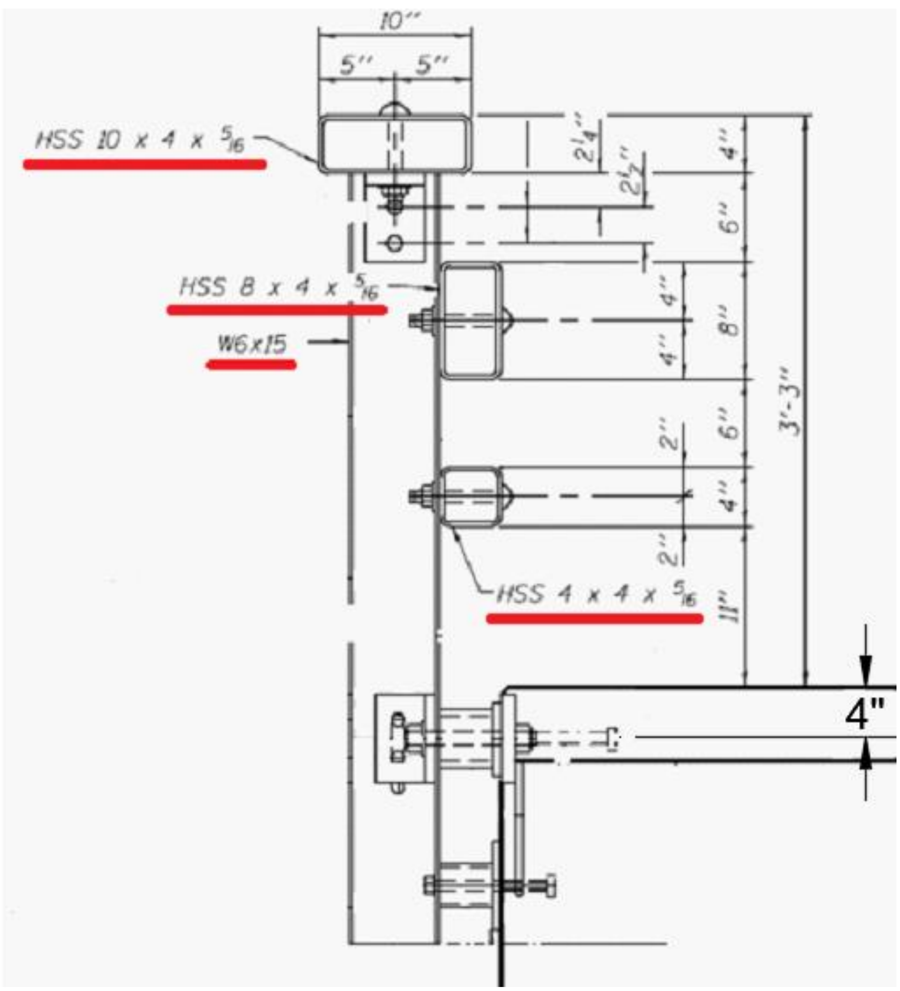
L = 6.25 ft (post spacing)

Top anchor depth = 4 in.

L_T = 5 ft (length of distributed load)

DMF on posts = 1.0

Post: W6x15	(ASTM A992)	$Z_{X\ POST}$ = 10.8 in.	and F_Y = 50 ksi
Top rail: HSS 10-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{X\ TOP\ RAIL}$ = 23.1 in.^{3}	and F_Y = 50 ksi
Middle rail: HSS 8-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{Y\ MID\ RAIL}$ = 9.91 in.^{3}	and F_Y = 50 ksi
Bottom rail: HSS 4-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{Y\ BOT\ RAIL}$ = 5.59 in.^{3}	and F_Y = 50 ksi



Step 2 – Determine distance from center of rails to top anchor, Y_{RAILS}:

$Y_{TOP\ RAIL}$ = 37 in. + 4 in. = 41 in.

$Y_{MID\ RAIL}$ = 25 in. + 4 in. = 29 in.

$Y_{BOT\ RAIL}$ = 13 in. + 4 in. = 17 in.

Step 3 – Determine plastic moment capacity of rails, $\Sigma M_{P\ RAILS}$, and post, $M_{P\ POST}$:

$M_{P\ TOP\ RAIL} = Ø\ F_Y\ Z_X$ = (0.9) (50 ksi) (23.1 in.^{3}) = 1039.5 kip-in.

$M_{P\ MID\ RAIL} = Ø\ F_Y\ Z_Y$ = (0.9) (50 ksi) (9.91 in.^{3}) = 445.95 kip-in.

$M_{P\ BOT\ RAIL} = Ø\ F_Y\ Z_Y$= (0.9) (50 ksi) (5.59 in.^{3}) = 251.55 kip-in.

$\Sigma M_{P\ RAILS}$ = 1737.0 kip-in.

$M_{P\ POST} = Ø\ DMF\ F_Y\ Z_X$ = (0.9) (1.0) (50 ksi) (10.8 in.^{3}) = 486.0 kip-in.

Step 4 - Determine effective height, $\overline{Y}_{RAILS}$, of combined rail plastic moment capacities:

$$\overline{Y}_{RAILS} = \frac{\Sigma(M_{P\ RAIL} * h)}{\Sigma M_{P\ RAILS}}$$

$$\overline{Y}_{RAILS} = \frac{(1039.5\ \text{kip}-\text{in.} * 41\ \text{in.}) + (445.95\ \text{kip}-\text{in.} * 29\ \text{in.}) + (251.55\ \text{kip}-\text{in.} * 17\ \text{in.})}{1737.0\ \text{kip}-\text{in.}}$$

$\overline{Y}_{RAILS}$ = 34.44 in.

Step 5 – Calculate shear force, P_P, for single post corresponding to the effective height of rails, $\overline{Y}_{RAILS}$:

$$P_P = \frac{M_{P\ POST}}{\overline{Y}_{RAILS}} = \frac{486.0\ \text{kip}-\text{in.}}{34.44\ \text{in.}} = 14.11\ \text{kips}$$

<u>Step 6 - Determine minimum strength of rails and posts, R, for multiple spans at effective height of rails, $\bar{Y}_{RAILS}$:</u>

For failure modes involving an odd number of rail spans: $R = \frac{16M_{P\ RAILS}+(N-1)(N+1)P_PL}{2NL-L_T}$

For failure modes involving an even number of rail spans: $R = \frac{16M_{P\ RAILS}+N^2P_PL}{2NL-L_T}$

1 Span: $R = \frac{16 * (1737\ \text{kip-in.}) + (1+1) * (1-1) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 1 * 75\ \text{in.}) - 60\ \text{in.}}$ = 308.8 kips @ 34.44 in.

2 Spans: $R = \frac{16 * (1737\ \text{kip-in.}) + (2^2) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 2 * 75\ \text{in.}) - 60\ \text{in.}}$ = 133.4 kips @ 34.44 in.

3 Spans: $R = \frac{16 * (1737\ \text{kip-in.}) + (3+1) * (3-1) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 3 * 75\ \text{in.}) - 60\ \text{in.}}$ = 93.0 kips @ 34.44 in.

4 Spans: $R = \frac{16 * (1737\ \text{kip-in.}) + (4^2) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 4 * 75\ \text{in.}) - 60\ \text{in.}}$ = 82.8 kips @ 34.44 in.

5 Spans: $R = \frac{16 * (1737\ \text{kip-in.}) + (5+1) * (5-1) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 5 * 75\ \text{in.}) - 60\ \text{in.}}$ = <u>77.1 kips @ 34.44 in.</u> Critical Value

6 Spans: $R = \frac{16 * (1737\ \text{kip-in.}) + (6^2) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 6 * 75\ \text{in.}) - 60\ \text{in.}}$ = 78.4 kips @ 34.44 in.

7 Spans: $R = \frac{16 * (1737\ \text{kip-in.}) + (7+1) * (7-1) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 7 * 75\ \text{in.}) - 60\ \text{in.}}$ = 79.4 kips @ 34.44 in.

8 Spans: $R = \frac{16 * (1737\ \text{kip-in.}) + (8^2) * 14.11\ \text{kips} * 75\ \text{in.}}{(2 * 8 * 75\ \text{in.}) - 60\ \text{in.}}$ = 83.8 kips @ 34.44 in.

<u>Step 7 – Determine horizontal resistance, R_{DESIGN}, for MASH TL-4 single-unit truck at design impact load height, H_{DESIGN}:</u>

$R_{DESIGN} = R * \frac{\bar{Y}_{RAILS}}{H_{DESIGN}} = 77.1\ \text{kips} * \frac{34.44\ \text{in.}}{30\ \text{in.}+4\ \text{in.}}$

R_{DESIGN} = 78.1 kips at 34.0 in. < 80 kips

Barrier inadequate for MASH single-unit truck loading of 80 kips distributed over 5 ft at a height of 30 in. above deck!

5.2.2 Example Problem No. 2 – Estimate Barrier Capacity for IL/OH Prototype Bridge Rail for Pickup Truck with Three Rails and DMF=1.0

Step 1 - System information:

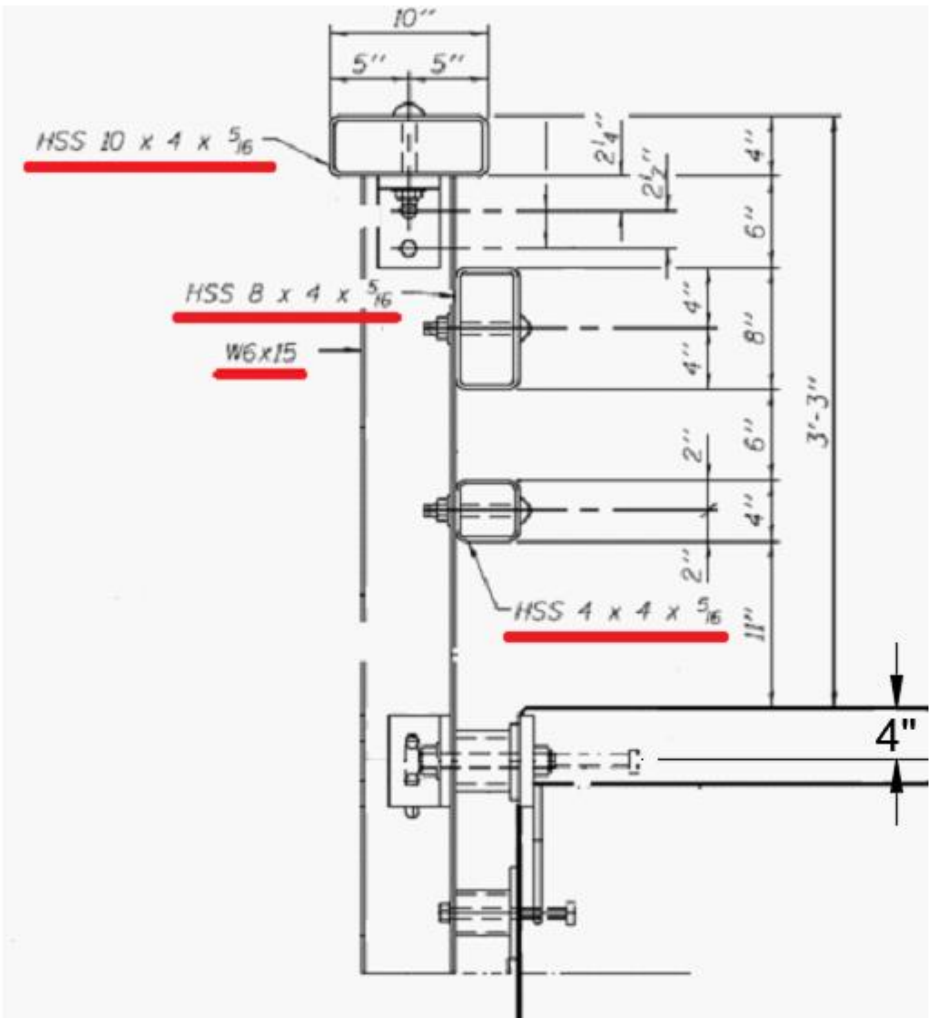
L = 6.25 ft (post spacing)

Top anchor depth = 4 in.

L_T = 4 ft (length of distributed load)

DMF on posts = 1.0

Post: W6x15	(ASTM A992)	Z_{XPOST} = 10.8 in.	and F_Y = 50 ksi
Top rail: HSS 10-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{X\ TOP\ RAIL}$ = 23.1 in.^{3}	and F_Y = 50 ksi
Middle rail: HSS 8-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{Y\ MID\ RAIL}$ = 9.91 in.^{3}	and F_Y = 50 ksi
Bottom rail: HSS 4-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{Y\ BOT\ RAIL}$ = 5.59 in.^{3}	and F_Y = 50 ksi



Step 2 – Determine distance from center of rails to top anchor, Y_{RAILS}:

$Y_{TOP\ RAIL}$ = 37 in. + 4 in. = 41 in.

$Y_{MID\ RAIL}$ = 25 in. + 4 in. = 29 in.

$Y_{BOT\ RAIL}$ = 13 in. + 4 in. = 17 in.

Step 3 – Determine plastic moment capacity of rails, $\Sigma M_{P\ RAILS}$, and post, $M_{P\ POST}$:

$M_{P\ TOP\ RAIL}$ = Ø F_Y Z_X = (0.9) (50 ksi) (23.1 in.^{3}) = 1039.5 kip-in.

$M_{P\ MID\ RAIL}$ = Ø F_Y Z_Y = (0.9) (50 ksi) (9.91 in.^{3}) = 445.95 kip-in.

$M_{P\ BOT\ RAIL}$ = Ø F_Y Z_Y= (0.9) (50 ksi) (5.59 in.^{3}) = 251.55 kip-in.

$\Sigma M_{P\ RAILS}$ = 1737.0 kip-in.

$M_{P\ POST}$ = Ø DMF F_Y Z_X = (0.9) (1.0) (50 ksi) (10.8 in.^{3}) = 486.0 kip-in.

Step - 4 Determine effective height, $\overline{Y}_{RAILS}$, of combined rail plastic moment capacities:

$$\overline{Y}_{RAILS} = \frac{\Sigma(M_{P\ RAIL} * h)}{\Sigma M_{P\ RAILS}}$$

$$\overline{Y}_{RAILS} = \frac{(1039.5 \text{ kip} - \text{in.} \ * 41 \text{ in.}) + (445.95 \text{ kip} - \text{in.} \ * 29 \text{ in.}) + (251.55 \text{ kip} - \text{in.} \ * 17 \text{ in.})}{1737.0 \text{ kip} - \text{in.}}$$

$\overline{Y}_{RAILS}$ = 34.44 in.

Step 5 – Calculate shear force, P_P, for single post corresponding to the effective height of rails, $\overline{Y}_{RAILS}$:

$$P_P = \frac{M_{P\ POST}}{\overline{Y}_{RAILS}} = \frac{486.0 \text{ kip}-\text{in.}}{34.44 \text{ in.}} = 14.11 \text{ kips}$$

Step 6 - Determine minimum strength of rails and posts, R, for multiple spans at effective height of rails, $\overline{Y}_{RAILS}$:

For failure modes involving an odd number of rail spans: $R = \frac{16M_{P\,RAILS}+(N-1)(N+1)P_PL}{2NL-L_T}$

For failure modes involving an even number of rail spans: $R = \frac{16M_{P\,RAILS}+N^2P_PL}{2NL-L_T}$

1 Span: $R = \frac{16*(1737\text{ kip-in.})+(1+1)*(1-1)*14.11\text{ kips}*75\text{ in.}}{(2*1*75\text{ in.})-48\text{ in.}}$ = 272.5 kips @ 34.44 in.

2 Spans: $R = \frac{16*(1737\text{ kip-in.})+(2^2)*14.11\text{ kips}*75\text{ in.}}{(2*2*75\text{ in.})-48\text{ in.}}$ = 127.1 kips @ 34.44 in.

3 Spans: $R = \frac{16*(1737\text{ kip-in.})+(3+1)*(3-1)*14.11\text{ kips}*75\text{ in.}}{(2*3*75\text{ in.})-48\text{ in.}}$ = 90.2 kips @ 34.44 in.

4 Spans: $R = \frac{16*(1737\text{ kip-in.})+(4^2)*14.11\text{ kips}*75\text{ in.}}{(2*4*75\text{ in.})-48\text{ in.}}$ = 81.0 kips @ 34.44 in.

5 Spans: $R = \frac{16*(1737\text{ kip-in.})+(5+1)*(5-1)*14.11\text{ kips}*75\text{ in.}}{(2*5*75\text{ in.})-48\text{ in.}}$ = <u>75.8 kips @ 34.44 in.</u> Critical Value

6 Spans: $R = \frac{16*(1737\text{ kip-in.})+(6^2)*14.11\text{ kips}*75\text{ in.}}{(2*6*75\text{ in.})-48\text{ in.}}$ = 77.3 kips @ 34.44 in

7 Spans: $R = \frac{16*(1737\text{ kip-in.})+(7+1)*(7-1)*14.11\text{ kips}*75\text{ in.}}{(2*7*75\text{ in.})-48\text{ in.}}$ = 78.4 kips @ 34.44 in

8 Spans: $R = \frac{16*(1737\text{ kip-in.})+(8^2)*14.11\text{ kips}*75\text{ in.}}{(2*8*75\text{ in.})-48\text{ in.}}$ = 82.9 kips @ 34.44 in

Step 7 – Determine horizontal resistance, R_{DESIGN}, for MASH TL-4 single-unit truck at design impact load height, H_{DESIGN}:

$R_{DESIGN} = R * \frac{\overline{Y}_{RAILS}}{H_{DESIGN}}$ = 75.8 kips $* \frac{34.44\text{ in.}}{24\text{ in.}+4\text{ in.}}$

R_{DESIGN} = 93.2 kips at 28 in. > 70 kips

Barrier adequate for MASH pickup truck loading of 70 kips distributed over 4 ft at a height of 24 in. above deck when considering three rails!

5.2.3 Example Problem No. 3 – Estimate Barrier Capacity for IL/OH Prototype Bridge Rail for Pickup Truck with Two Lower Rails and DMF=1.0

Step 1 - System information:

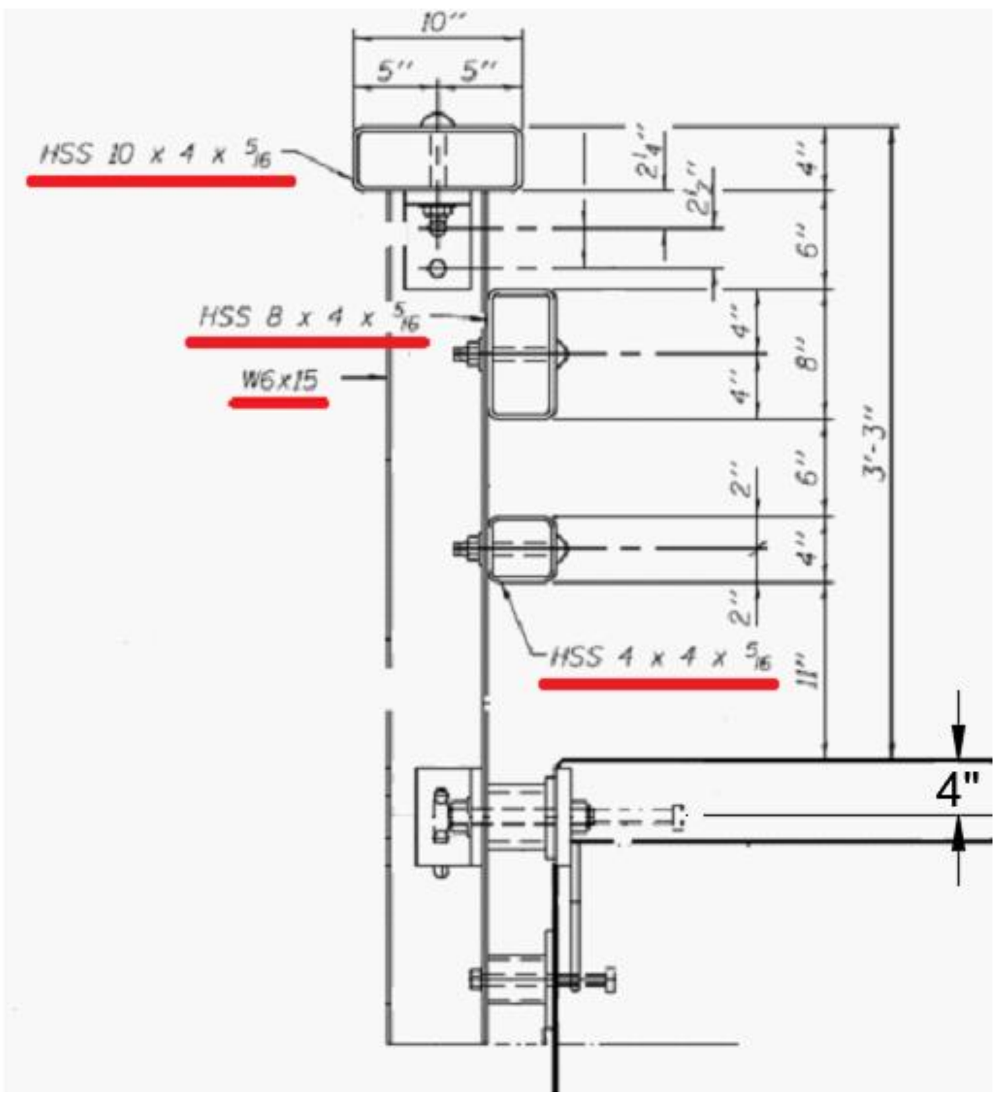
L = 6.25 ft (post spacing)

Top anchor depth = 4 in.

L_T = 4 ft (length of distributed load)

DMF on posts = 1.0

Post: W6x15	(ASTM A992)	Z_{POST} = 10.8 in.	and F_Y = 50ksi
Middle rail: HSS 8-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{Y\ MID\ RAIL}$ = 9.91 in.^{3}	and F_Y = 50ksi
Bottom rail: HSS 4-in. x 4-in. x $^5/_{16}$-in.	(ASTM A500 Grade C)	$Z_{Y\ BOT\ RAIL}$ = 5.59 in.^{3}	and F_Y = 50ksi



Step 2 – Determine distance from center of rails to top anchor, Y_{RAILS}:

$Y_{MID\ RAIL}$ = 25 in. + 4 in. = 29 in.

$Y_{BOT\ RAIL}$ = 13 in. + 4 in. = 17 in.

Step 3 – Determine plastic moment capacity of rails, $\Sigma M_{P\ RAILS}$, and post, $M_{P\ POST}$:

$M_{P\ MID\ RAIL} = Ø\ F_Y\ Z_Y$ = (0.9) (50 ksi) (9.91 in.^{3}) = 445.95 kip-in.

$M_{P\ BOT\ RAIL} = Ø\ F_Y\ Z_Y$= (0.9) (50 ksi) (5.59 in.^{3}) = 251.55 kip-in.

$\Sigma M_{P\ RAILS}$ = 697.5 kip-in.

$M_{P\ POST} = Ø\ DMF\ F_Y\ Z_X$ = (0.9) (1.0) (50 ksi) (10.8 in.^{3}) = 486.0 kip-in.

Step - 4 Determine effective height, $\overline{Y}_{RAILS}$, of combined rail plastic moment capacities:

$$\overline{Y}_{RAILS} = \frac{\Sigma(M_{P\ RAIL} * h)}{\Sigma M_{P\ RAILS}}$$

$$\overline{Y}_{RAILS} = \frac{(445.95\text{ kip} - \text{in.} \ * 29\text{ in.}) + (251.55\text{ kip} - \text{in.} \ * 17\text{ in.})}{697.5\text{ kip} - \text{in.}}$$

$\overline{Y}_{RAILS}$ = 24.67 in.

Step 5 – Calculate shear force, P_P, for single post corresponding to the effective height of rails, $\overline{Y}_{RAILS}$:

$$P_P = \frac{M_{P\ POST}}{\overline{Y}_{RAILS}} = \frac{486.0\text{ kip}-\text{in.}}{24.67\text{ in.}} = 19.7\text{ kips}$$

Step 6 - Determine minimum strength of rails and posts, R, for multiple spans at effective height of rails, $\bar{Y}_{RAILS}$:

For failure modes involving an odd number of rail spans: $R = \frac{16M_{P\,RAILS}+(N-1)(N+1)P_PL}{2NL-L_T}$

For failure modes involving an even number of rail spans: $R = \frac{16M_{P\,RAILS}+N^2P_PL}{2NL-L_T}$

1 Span: $R = \frac{16 * (697.5\text{ kip-in.}) + (1+1) * (1-1) * 19.7\text{ kips} * 75\text{ in.}}{(2 * 1 * 75\text{ in.}) - 48\text{ in.}}$ = 109.4 kips @ 24.67 in.

2 Spans: $R = \frac{16 * (697.5\text{ kip-in.}) + (2^2) * 19.7\text{ kips} * 75\text{ in.}}{(2 * 2 * 75\text{ in.}) - 48\text{ in.}}$ = 67.7 kips @ 24.67 in.

3 Spans: $R = \frac{16 * (697.5\text{ kip-in.}) + (3+1) * (3-1) * 19.7\text{ kips} * 75\text{ in.}}{(2 * 3 * 75\text{ in.}) - 48\text{ in.}}$ = <u>57.2 kips @ 24.67 in.</u> Critical Value

4 Spans: $R = \frac{16 * (697.5\text{ kip-in.}) + (4^2) * 19.7\text{ kips} * 75\text{ in.}}{(2 * 4 * 75\text{ in.}) - 48\text{ in.}}$ = 63.0 kips @ 24.67 in.

5 Spans: $R = \frac{16 * (697.5\text{ kip-in.}) + (5+1) * (5-1) * 19.7\text{ kips} * 75\text{ in.}}{(2 * 5 * 75\text{ in.}) - 48\text{ in.}}$ = 66.4 kips @ 24.67 in.

6 Spans: $R = \frac{16 * (697.5\text{ kip-in.}) + (6^2) * 19.7\text{ kips} * 75\text{ in.}}{(2 * 6 * 75\text{ in.}) - 48\text{ in.}}$ = 75.5 kips @ 24.67 in

7 Spans: $R = \frac{16 * (697.5\text{ kip-in.}) + (7+1) * (7-1) * 19.7\text{kips} * 75\text{ in.}}{(2 * 7 * 75\text{ in.}) - 48\text{ in.}}$ = 81.9 kips @ 24.67 in

8 Spans: $R = \frac{16 * (697.5\text{ kip-in.}) + (8^2) * 19.7\text{ kips} * 75\text{ in.}}{(2 * 8 * 75\text{ in.}) - 48\text{ in.}}$ = 91.8 kips @ 24.67 in

Step 7 – Determine horizontal resistance, R_{DESIGN}, for MASH TL-4 single-unit truck at design impact load height, H_{DESIGN}

$R_{DESIGN} = R * \frac{\bar{Y}_{RAILS}}{H_{DESIGN}}$ = 57.2 kips $* \frac{24.67\text{ in.}}{24\text{ in.}+4\text{ in.}}$

R_{DESIGN} = 50.4 kips at 28 in. < 70 kips

Barrier inadequate for MASH pickup truck loading of 70 kips distributed over 4 ft at a height of 24 in. above deck when considering two lower rails!

5.3 Example Problem Summary

As shown in the previous examples and calculations provided in Section 5.2 as well as in Appendix B, the lateral barrier resistance of the IL/OH MASH 2016 TL-4 Prototype Bridge Rail with a 6 ft - 3 in. (1.91 m) post spacing and DMFs equal to 1.0 and 1.5 for the SUT scenario were calculated to be 78.1 kips (347 kN) for a three-span collapse and 96.7 kips (430 kN) for a five-span collapse, respectively. The barrier lateral resistance of the IL/OH Prototype Bridge Rail when only considering the lower two rails with DMFs of 1.0 and 1.5 for the pickup truck scenario were calculated to be 50.4 kips (224 kN) for a three-span collapse and 63.3 kips (282 kN) for a five-span collapse, respectively. The lateral barrier capacity of the IL/OH MASH 2016 TL-4 Prototype Bridge Rail when considering all three rails with DMFs of 1.0 and 1.5 for the pickup truck scenario were calculated as 93.2 kips (415 kN) for a three-span collapse and 115.5 kips (514 kN) for a five-span collapse, respectively. These results are depicted in Table 16. Based on this analysis, further investigation was performed to configure acceptable systems with varied post spacing and to comply with other design criteria.

Table 16. IL/OH Lateral Barrier Resistance

Design Scenario	No. of Rails Effective	Lateral Barrier Capacity (kips)		% Increase in Barrier Capacity
		DMF = 1.0	DMF = 1.5	
Single-Unit Truck	3	78.1	96.7	23.8
Pickup Truck	2 (Lower & Middle)	50.4	63.3	25.6
Pickup Truck	3	93.2	115.5	23.9

The lateral barrier resistance of the IL/OH Prototype Bridge Rail increased by 23.8%, 25.6%, and 23.9% for the three impact scenarios when considering a DMF equal to 1.5 versus 1.0. Therefore, the lateral barrier resistance was expected to increase approximately 25% for future bridge rail configurations when using a DMF equal to 1.5 versus 1.0.

5.4 Guidance Charts for Preliminary Bridge Rail Configurations

A research effort was performed to identify the required plastic moment capacity for combined number of rails at the height of the selected design impact loading in order to resist both pickup truck and SUT impact events. These guidance plots were created using plastic collapse mechanism calculations with a modified effective height of the combined rails to be located at the same height as the design impact loading for all four bridge deck types generally used by the Illinois and Ohio DOTs, as previously discussed in Section 3.2. A W6x15 (W150x22.5) post section configured with ASTM A992 steel was used to create these guidance plots. An asphalt overlay of 3 in. (76 mm) was considered in order to maximize the moment arm between the heights of the design impact load and the tension anchor rods in the upper regions of the bridge deck or box slabs for both pickup truck and SUT impact events. The distances between the top of the 3 in. (76 mm) asphalt overlay to the tension anchor rods for the four bridge deck types are shown in

Figure 48. The moment arm between the pickup and SUT design loading and the tension anchor rod height are shown in Figure 49 and Figure 50, respectively.

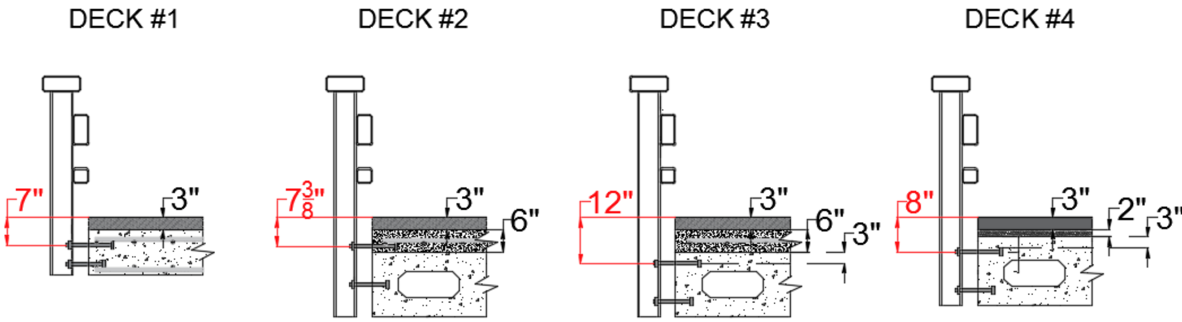


Figure 48. Preliminary Slab Decks and Post Configurations

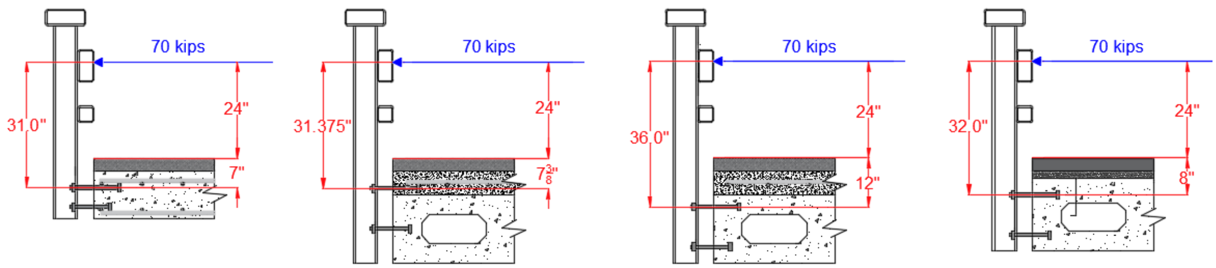


Figure 49. Moment Arm between Pickup Truck Design Impact Loading and Tension Anchor Rods

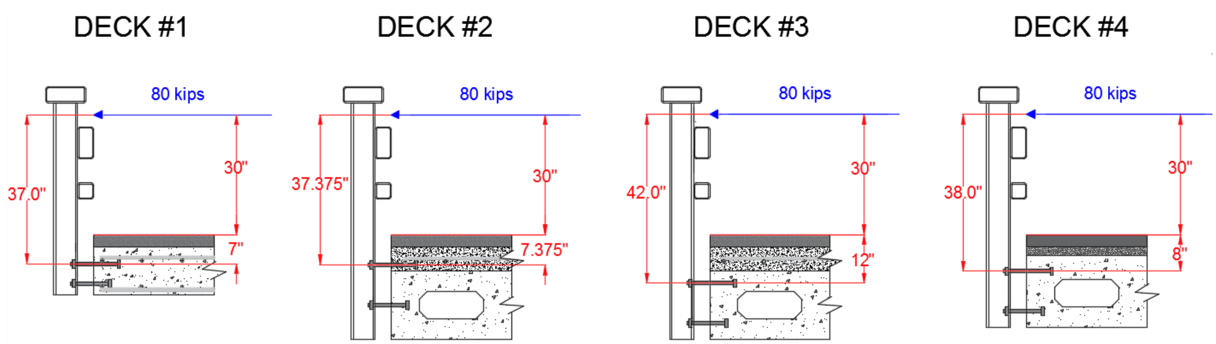


Figure 50. Moment Arm between SUT Design Impact Loading and Tension Anchor Rods

The guidance charts for pickup truck and SUT impact events, DMFs equal to 1.0 and 1.5, W6x15 (W150x22.5) posts, and four different concrete bridge decks commonly used by Illinois and Ohio DOTs, are shown in Tables 17 through 24. The bridge deck configuration, design impact loading, DMF applied to the posts, and post spacing, were required to proceed using guidance charts. The post spacing ranged between 4 ft (1.2 m) and 12 ft (3.7 m). Guidance charts were prepared for each deck type, which corresponded to a different effective height of rails, $\overline{Y}_{RAILS}$. For a defined vehicular impact event scenario, an engineer could select a guidance chart with a known deck type and DMF. Then, the engineer would select a desired post spacing for the bridge rail system. Once in the table and using the appropriate column for post spacing, an engineer would find the green cell to determine the estimated lateral resistance of the barrier exceeding the design impact loading. Green cells represent acceptable lateral barrier resistance (kips) for a bridge rail configuration, while red cells represent unacceptable lateral barrier resistance (kips) for bridge rail configuration. Therefore, an end-user could start selecting railing sections to match the minimum required plastic moment capacity for the combined rails to design crashworthy bridge rail configurations for beam-and-post systems supported by W6x15 (W150x22.5) steel posts for a range of effective height of rails, $\overline{Y}_{RAILS}$, and two DMFs.

Table 17. Guidance Charts for Deck #1 and Pickup Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Deck #1, PICKUP DMF=1.0

Plastic Moment Capacity of Rails, $\Sigma M_{P\ RAILS}$ (kips - in.)	48	60	72	75	96	120	144
Post Spacing, L	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft
2800	129.1	113.6	103.1	101.0	88.7	78.1	71.1
2700	126.9	111.6	101.4	99.4	87.0	76.7	69.9
2600	124.6	109.5	99.8	97.8	85.2	75.3	68.8
2500	122.0	107.5	98.1	96.2	83.5	73.9	67.7
2400	119.4	105.5	96.4	94.6	81.7	72.5	66.5
2300	116.9	103.5	94.8	92.6	80.0	71.1	65.4
2200	114.3	101.5	92.7	90.3	78.2	69.7	64.2
2100	111.7	99.4	90.3	88.1	76.4	68.4	63.1
2000	109.2	97.4	87.9	85.8	74.7	67.0	61.3
1900	106.6	95.4	85.6	83.5	72.9	65.6	59.4
1800	104.0	93.1	83.2	81.2	71.2	64.2	57.4
1700	101.5	90.2	79.5	78.9	69.4	62.8	55.5
1600	98.9	87.3	78.4	76.7	67.7	60.5	53.5
1500	96.3	84.4	76.0	74.4	65.9	58.1	51.5
1400	93.7	81.5	73.6	72.1	64.2	55.7	49.6
1300	90.0	78.6	71.3	69.8	62.2	53.3	47.6
1200	86.3	75.7	68.9	67.5	59.2	51.0	45.7
1100	82.5	72.8	66.5	65.3	56.1	48.6	43.7
1000	78.8	69.9	64.1	63.0	53.1	46.2	41.7
900	75.1	67.0	61.0	59.2	50.1	43.8	39.8
800	71.4	64.1	56.8	55.2	47.0	41.4	37.8
700	67.7	60.0	52.7	51.3	44.0	39.1	35.9
600	64.0	54.9	48.5	47.3	41.0	36.7	33.9
500	58.4	49.8	44.3	43.3	38.0	34.3	31.9
400	51.8	44.6	40.2	39.3	34.9	31.9	26.7
300	45.1	39.5	36.0	35.3	31.9	25.0	20.0
200	38.4	34.4	31.8	31.4	22.2	16.7	13.3
100	31.8	22.2	16.7	15.7	11.1	8.3	6.7

31" Effective height of rails, $\overline{Y}_{RAILS}$

Deck #1, PICKUP DMF=1.5

Plastic Moment Capacity of Rails, $\Sigma M_{P\ RAILS}$ (kips - in.)	48	60	72	75	96	120	144
Post Spacing, L	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft
1900	133.1	116.4	105.7	103.6	91.8	78.8	70.5
1800	129.4	113.5	103.3	101.3	88.8	76.5	68.5
1700	125.7	110.6	99.5	99.0	85.7	74.1	66.5
1600	122.0	107.7	98.6	96.8	82.7	71.7	64.6
1500	118.3	104.8	96.2	94.5	79.7	69.3	62.6
1400	114.6	101.9	93.6	90.8	76.6	66.9	60.7
1300	110.9	99.0	89.4	86.8	73.6	64.5	58.7
1200	107.2	96.1	85.3	82.9	70.6	62.2	56.7
1100	103.5	92.6	81.1	78.9	67.5	59.8	54.8
1000	99.7	87.5	76.9	74.9	64.5	57.4	52.8
900	96.0	82.3	72.8	70.9	61.5	55.0	50.8
800	91.0	77.2	68.6	66.9	58.4	52.6	48.9
700	84.3	72.1	64.4	63.0	55.4	50.3	46.7
600	77.6	66.9	60.3	59.0	52.4	47.9	40.0
500	71.0	61.8	56.1	55.0	49.4	41.7	33.3
400	64.3	56.7	51.9	51.0	44.4	33.3	26.7
300	57.6	51.6	47.8	47.0	33.3	25.0	20.0
200	51.0	44.4	33.3	31.4	22.2	16.7	13.3
100	33.3	22.2	16.7	15.7	11.1	8.3	6.7

31" Effective height of rails, $\overline{Y}_{RAILS}$

Table 18. Guidance Charts for Deck #1 and Single-Unit Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Deck #1, SUT
DMF=1.0

Plastic Moment Capacity of Rails, Σ $M_{P\,RAILS}$ (kips - in.)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4200	151.9	130.7	117.0	114.3	99.5	88.2	80.8
4100	149.6	128.9	115.5	112.8	98.2	87.2	80.0
4000	147.4	127.2	114.0	111.4	97.0	86.2	79.1
3900	145.1	125.4	112.6	110.0	95.7	85.2	78.1
3800	142.9	123.6	111.1	108.6	94.5	84.2	77.0
3700	140.6	121.8	109.6	107.2	93.2	83.2	75.8
3600	138.3	120.0	108.2	105.8	92.0	82.3	74.6
3500	136.1	118.3	106.7	104.3	90.8	81.3	73.5
3400	133.8	116.5	105.2	102.7	89.5	80.3	72.3
3300	131.6	114.7	103.6	101.1	88.3	79.3	71.2
3200	129.3	112.9	101.9	99.5	87.0	78.1	70.0
3100	127.0	111.2	100.2	97.9	85.8	76.7	68.8
3000	124.8	109.4	98.5	96.2	84.5	75.3	67.7
2900	122.5	107.6	96.8	94.6	83.3	73.9	66.5
2800	120.3	105.8	95.1	93.0	82.0	72.5	65.4
2700	118.0	103.9	93.5	91.4	80.8	71.1	64.2
2600	115.8	101.8	91.8	89.8	79.5	69.7	63.0
2500	113.5	99.8	90.1	88.2	78.1	68.3	61.9
2400	111.2	97.7	88.4	86.6	76.3	66.9	60.7
2300	109.0	95.7	86.7	84.9	74.5	65.5	59.6
2200	106.7	93.6	85.0	83.3	72.7	64.1	58.4
2100	104.4	91.6	83.3	81.7	71.0	62.7	57.2
2000	101.7	89.5	81.6	80.1	69.2	61.3	56.1
1900	99.1	87.5	80.0	78.3	67.4	59.9	54.9
1800	96.5	85.4	78.0	76.0	65.6	58.4	53.8
1700	93.9	83.4	74.3	73.7	63.8	57.0	52.6
1600	91.3	81.3	73.2	71.4	62.1	55.6	50.7
1500	88.7	79.3	70.8	69.0	60.3	54.2	48.7
1400	86.1	76.5	68.3	66.7	58.5	52.8	46.7
1300	83.4	73.5	65.9	64.4	56.7	50.6	44.7
1200	80.8	70.6	63.5	62.1	55.0	48.2	42.7
1100	77.9	67.6	61.1	59.8	53.2	45.8	40.7
1000	74.1	64.7	58.6	57.5	50.6	43.3	38.7
900	70.3	61.7	56.2	55.1	47.5	40.9	36.7
800	66.5	58.7	53.8	52.8	44.4	38.5	34.7
700	62.7	55.8	50.4	48.9	41.3	36.1	32.8
600	58.9	52.8	46.1	44.8	38.2	33.7	30.8
500	55.1	47.7	41.8	40.7	35.1	31.2	28.8
400	50.2	42.3	37.5	36.6	32.0	28.8	26.8
300	43.2	37.0	33.2	32.5	28.9	26.4	21.1
200	36.2	31.7	28.9	28.4	24.2	17.8	14.0
100	29.1	26.3	19.0	17.8	12.1	8.9	7.0

Post Spacing, L

37" Effective height of rails, Y_{RAILS}

Deck #1, SUT
DMF=1.5

Plastic Moment Capacity of Rails, Σ $M_{P\,RAILS}$ (kips - in.)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2700	144.8	128.1	117.0	114.0	98.4	87.7	80.6
2600	142.1	126.1	114.6	111.7	96.7	86.3	79.5
2500	139.5	124.0	112.2	109.4	94.9	84.9	78.0
2400	136.9	122.0	109.8	107.1	93.1	83.5	76.0
2300	134.3	119.9	107.3	104.7	91.3	82.1	74.0
2200	131.7	117.7	104.9	102.4	89.6	80.7	72.0
2100	129.1	114.8	102.5	100.1	87.8	79.2	70.0
2000	126.5	111.8	100.1	97.8	86.0	77.1	68.0
1900	123.8	108.8	97.6	95.5	84.2	74.7	66.0
1800	121.2	105.9	95.2	93.1	82.4	72.3	64.1
1700	118.6	102.9	91.4	90.8	80.7	69.9	62.1
1600	115.0	99.9	90.4	88.5	78.9	67.4	60.1
1500	111.2	97.0	87.9	86.2	75.8	65.0	58.1
1400	107.4	94.0	85.5	83.9	72.7	62.6	56.1
1300	103.6	91.1	83.1	81.5	69.6	60.2	54.1
1200	99.8	88.1	80.7	79.2	66.5	57.7	52.1
1100	95.9	85.1	77.8	75.4	63.4	55.3	50.1
1000	92.1	82.2	73.5	71.3	60.3	52.9	48.1
900	88.3	79.2	69.2	67.2	57.2	50.5	46.1
800	84.5	74.2	64.9	63.1	54.1	48.1	44.2
700	80.7	68.9	60.6	59.0	51.0	45.6	42.2
600	75.3	63.5	56.3	54.9	47.9	43.2	40.2
500	68.3	58.2	52.0	50.8	44.8	40.8	35.1
400	61.3	52.9	47.7	46.7	41.7	35.6	28.1
300	54.2	47.5	43.4	42.6	36.4	26.7	21.1
200	47.2	42.2	38.1	35.6	24.2	17.8	14.0
100	40.2	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing, L

37" Effective height of rails, Y_{RAILS}

Table 19. Guidance Charts for Deck #2 and Pickup Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Deck #2, PICKUP
DMF=1.0

Plastic Moment Capacity of Rails, Σ $M_{P\ RAILS}$ (kips - in.)							
2800	128.3	112.9	102.4	100.4	88.3	77.6	70.6
2700	126.1	110.9	100.8	98.8	86.5	76.2	69.5
2600	123.9	108.9	99.1	97.2	84.7	74.8	68.3
2500	121.3	106.8	97.4	95.6	83.0	73.4	67.2
2400	118.7	104.8	95.8	94.0	81.2	72.1	66.0
2300	116.2	102.8	94.1	92.1	79.5	70.7	64.9
2200	113.6	100.8	92.2	89.9	77.7	69.3	63.7
2100	111.0	98.8	89.8	87.6	76.0	67.9	62.6
2000	108.5	96.7	87.5	85.3	74.2	66.5	61.1
1900	105.9	94.7	85.1	83.0	72.5	65.1	59.1
1800	103.3	92.6	82.7	80.7	70.7	63.7	57.2
1700	100.8	89.7	79.9	78.5	69.0	62.3	55.2
1600	98.2	86.8	77.9	76.2	67.2	60.2	53.2
1500	95.7	83.9	75.5	73.9	65.4	57.8	51.3
1400	93.1	81.0	73.2	71.6	63.7	55.5	49.3
1300	89.5	78.1	70.8	69.3	61.9	53.1	47.4
1200	85.8	75.2	68.4	67.1	58.9	50.7	45.4
1100	82.0	72.3	66.0	64.8	55.9	48.3	43.4
1000	78.3	69.4	63.6	62.5	52.8	45.9	41.5
900	74.6	66.5	60.7	58.9	49.8	43.6	39.5
800	70.9	63.6	56.6	55.0	46.8	41.2	37.6
700	67.2	59.7	52.4	51.0	43.7	38.8	35.6
600	63.5	54.6	48.2	47.0	40.7	36.4	33.6
500	58.1	49.5	44.1	43.0	37.7	34.0	31.7
400	51.5	44.3	39.9	39.0	34.7	31.7	26.7
300	44.8	39.2	35.7	35.1	31.6	25.0	20.0
200	38.1	34.1	31.6	31.1	22.2	16.7	13.3
100	31.5	22.2	16.7	15.7	11.1	8.3	6.7
	48	60	72	75	96	120	144
	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft
Post Spacing, L							

31.375" Effective height of rails, $\bar{Y}_{RAILS}$

Deck #2, PICKUP
DMF=1.5

Plastic Moment Capacity of Rails, Σ $M_{P\ RAILS}$ (kips - in.)							
1900	132.3	115.7	105.0	102.9	91.4	78.4	70.1
1800	128.6	112.8	102.6	100.6	88.3	76.1	68.1
1700	124.9	109.9	100.0	98.3	85.3	73.7	66.1
1600	121.2	107.0	97.8	96.0	82.3	71.3	64.2
1500	117.5	104.1	95.5	93.8	79.3	68.9	62.2
1400	113.8	101.2	93.1	90.4	76.2	66.5	60.3
1300	110.1	98.3	89.0	86.4	73.2	64.1	58.3
1200	106.4	95.4	84.9	82.4	70.2	61.8	56.3
1100	102.7	92.2	80.7	78.5	67.1	59.4	54.4
1000	99.0	87.0	76.5	74.5	64.1	57.0	52.4
900	95.3	81.9	72.4	70.5	61.1	54.6	50.4
800	90.5	76.8	68.2	66.5	58.0	52.2	48.5
700	83.8	71.6	64.0	62.5	55.0	49.9	46.5
600	77.2	66.5	59.9	58.6	52.0	47.5	40.0
500	70.5	61.4	55.7	54.6	48.9	41.7	33.3
400	63.8	56.3	51.5	50.6	44.4	33.3	26.7
300	57.2	51.1	47.4	46.6	33.3	25.0	20.0
200	50.5	44.4	33.3	31.4	22.2	16.7	13.3
100	33.3	22.2	16.7	15.7	11.1	8.3	6.7
	48	60	72	75	96	120	144
	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft
Post Spacing, L							

31.375" Effective height of rails, $\bar{Y}_{RAILS}$

Table 20. Guidance Charts for Deck #2 and Single-Unit Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Deck #2, SUT
DMF=1.0

Plastic Moment Capacity of Rails, Σ Mp RAILS (kips - in.) / Post Spacing, L

Σ Mp RAILS (kips - in.)	48 4 ft	60 5 ft	72 6 ft	75 6 ft - 3 in	96 8 ft	120 10 ft	144 12 ft
4200	151.3	130.1	116.4	113.7	99.0	87.7	80.3
4100	149.1	128.4	114.9	112.3	97.8	86.7	79.5
4000	146.8	126.6	113.5	110.9	96.5	85.7	78.7
3900	144.6	124.8	112.0	109.5	95.3	84.8	77.8
3800	142.3	123.0	110.5	108.1	94.0	83.8	76.6
3700	140.0	121.3	109.1	106.7	92.8	82.8	75.5
3600	137.8	119.5	107.6	105.3	91.5	81.8	74.3
3500	135.5	117.7	106.2	103.9	90.3	80.8	73.1
3400	133.3	115.9	104.7	102.2	89.0	79.8	72.0
3300	131.0	114.1	103.1	100.6	87.8	78.8	70.8
3200	128.7	112.4	101.4	99.0	86.5	77.8	69.7
3100	126.5	110.6	99.7	97.4	85.3	76.4	68.5
3000	124.2	108.8	98.0	95.8	84.0	75.0	67.3
2900	122.0	107.0	96.3	94.2	82.8	73.6	66.2
2800	119.7	105.3	94.7	92.5	81.6	72.1	65.0
2700	117.4	103.4	93.0	90.9	80.3	70.7	63.9
2600	115.2	101.3	91.3	89.3	79.1	69.3	62.7
2500	112.9	99.3	89.6	87.7	77.7	67.9	61.6
2400	110.7	97.2	87.9	86.1	76.0	66.5	60.4
2300	108.4	95.2	86.2	84.5	74.2	65.1	59.2
2200	106.1	93.1	84.5	82.8	72.4	63.7	58.1
2100	103.9	91.1	82.8	81.2	70.6	62.3	56.9
2000	101.2	89.0	81.2	79.6	68.8	60.9	55.8
1900	98.6	87.0	79.5	78.0	67.1	59.5	54.6
1800	96.0	84.9	77.7	75.7	65.3	58.1	53.4
1700	93.4	82.9	74.7	73.3	63.5	56.7	52.3
1600	90.8	80.8	72.8	71.0	61.7	55.3	50.5
1500	88.2	78.8	70.4	68.7	60.0	53.9	48.5
1400	85.6	76.2	68.0	66.4	58.2	52.5	46.5
1300	82.9	73.2	65.6	64.1	56.4	50.4	44.5
1200	80.3	70.2	63.1	61.7	54.6	48.0	42.5
1100	77.6	67.3	60.7	59.4	52.8	45.6	40.5
1000	73.8	64.3	58.3	57.1	50.4	43.2	38.5
900	70.0	61.3	55.9	54.8	47.3	40.7	36.5
800	66.1	58.4	53.4	52.5	44.2	38.3	34.6
700	62.3	55.4	50.2	48.7	41.1	35.9	32.6
600	58.5	52.5	45.9	44.6	38.0	33.5	30.6
500	54.7	47.5	41.6	40.5	34.9	31.0	28.6
400	50.0	42.1	37.3	36.4	31.8	28.6	26.6
300	43.0	36.8	33.0	32.3	28.7	26.2	21.1
200	35.9	31.5	28.7	28.2	24.2	17.8	14.0
100	28.9	26.1	19.0	17.8	12.1	8.9	7.0

37.375" Effective height of rails, $\bar{Y}_{RAILS}$

Deck #2, SUT
DMF=1.5

Plastic Moment Capacity of Rails, Σ Mp RAILS (kips - in.) / Post Spacing, L

Σ Mp RAILS (kips - in.)	48 4 ft	60 5 ft	72 6 ft	75 6 ft - 3 in	96 8 ft	120 10 ft	144 12 ft
2700	144.0	127.4	116.5	113.5	97.9	87.2	80.2
2600	141.4	125.4	114.1	111.2	96.2	85.8	79.0
2500	138.8	123.3	111.7	108.9	94.4	84.4	77.7
2400	136.2	121.2	109.2	106.5	92.6	83.0	75.7
2300	133.6	119.2	106.8	104.2	90.8	81.6	73.7
2200	130.9	117.1	104.4	101.9	89.0	80.2	71.7
2100	128.3	114.2	102.0	99.6	87.3	78.7	69.7
2000	125.7	111.3	99.6	97.3	85.5	76.9	67.7
1900	123.1	108.3	97.1	94.9	83.7	74.4	65.8
1800	120.5	105.3	94.7	92.6	81.9	72.0	63.8
1700	117.9	102.4	91.8	90.3	80.2	69.6	61.8
1600	114.5	99.4	89.9	88.0	78.4	67.2	59.8
1500	110.6	96.5	87.4	85.7	75.5	64.7	57.8
1400	106.8	93.5	85.0	83.3	72.4	62.3	55.8
1300	103.0	90.5	82.6	81.0	69.3	59.9	53.8
1200	99.2	87.6	80.2	78.7	66.2	57.5	51.8
1100	95.4	84.6	77.5	75.1	63.1	55.0	49.8
1000	91.6	81.6	73.2	71.0	60.0	52.6	47.8
900	87.8	78.7	68.9	66.9	56.9	50.2	45.9
800	84.0	73.9	64.6	62.8	53.8	47.8	43.9
700	80.2	68.5	60.3	58.7	50.7	45.3	41.9
600	75.0	63.2	56.0	54.6	47.6	42.9	39.9
500	67.9	57.9	51.7	50.5	44.5	40.5	35.1
400	60.9	52.5	47.4	46.4	41.4	35.6	28.1
300	53.9	47.2	43.1	42.3	36.4	26.7	21.1
200	46.9	41.9	38.1	35.6	24.2	17.8	14.0
100	39.9	26.7	19.0	17.8	12.1	8.9	7.0

37.375" Effective height of rails, $\bar{Y}_{RAILS}$

Table 21. Guidance Charts for Deck #3 and Pickup Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Plastic Moment Capacity of Rails, Σ Mp RAILS (kips - in.)

Deck #3, PICKUP DMF=1.0							
3200	128.7	113.0	101.9	99.6	87.5	78.2	70.3
3100	126.5	111.2	100.3	98.0	86.3	76.8	69.1
3000	124.3	109.5	98.6	96.4	85.0	75.4	68.0
2900	122.0	107.7	96.9	94.8	83.8	74.0	66.9
2800	119.8	105.7	95.3	93.2	82.6	72.6	65.7
2700	117.6	103.6	93.6	91.6	81.3	71.3	64.6
2600	115.4	101.6	91.9	90.0	79.7	69.9	63.4
2500	113.2	99.6	90.3	88.4	78.0	68.5	62.3
2400	110.9	97.6	88.6	86.8	76.2	67.1	61.1
2300	108.7	95.6	86.9	85.2	74.5	65.7	60.0
2200	106.3	93.5	85.3	83.6	72.7	64.3	58.8
2100	103.7	91.5	83.6	82.0	70.9	62.9	57.7
2000	101.1	89.5	81.9	80.2	69.2	61.5	56.5
1900	98.6	87.5	80.0	77.9	67.4	60.1	55.4
1800	96.0	85.5	77.6	75.6	65.7	58.8	54.2
1700	93.4	83.4	85.8	73.4	63.9	57.4	52.4
1600	90.9	81.4	72.8	71.1	62.2	56.0	50.4
1500	88.3	78.7	70.4	68.8	60.4	54.6	48.5
1400	85.7	75.8	68.0	66.5	58.7	52.6	46.5
1300	83.2	72.9	65.7	64.2	56.9	50.2	44.5
1200	80.4	70.0	63.3	62.0	55.2	47.9	42.6
1100	76.7	67.1	60.9	59.7	53.0	45.5	40.6
1000	73.0	64.2	58.5	57.4	49.9	43.1	38.7
900	69.3	61.3	56.1	55.1	46.9	40.7	36.7
800	65.6	58.4	53.6	52.0	43.9	38.3	34.7
700	61.9	55.5	49.4	48.0	40.8	36.0	32.8
600	58.2	51.5	45.3	44.0	37.8	33.6	30.8
500	54.5	46.4	41.1	40.0	34.8	31.2	28.9
400	48.3	41.3	36.9	36.1	31.8	28.8	26.7
300	41.6	36.2	32.8	32.1	28.7	25.0	20.0
200	34.9	31.0	28.6	28.1	22.2	16.7	13.3
100	28.3	22.2	16.7	15.7	11.1	8.3	6.7
	48	60	72	75	96	120	144
	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft

Post Spacing, L

36" Effective height of rails, $\bar{Y}_{RAILS}$

Plastic Moment Capacity of Rails, Σ Mp RAILS (kips - in.)

Deck #3, PICKUP DMF=1.5							
2200	131.2	116.6	104.5	102.1	89.8	81.2	71.7
2100	128.6	113.7	102.1	99.8	88.0	78.9	69.8
2000	126.1	110.8	99.7	97.5	86.2	76.5	67.8
1900	123.5	107.9	97.3	95.2	84.5	74.2	65.8
1800	120.7	105.0	94.9	92.9	82.7	71.8	63.9
1700	117.0	102.1	105.8	90.7	81.0	69.4	61.9
1600	113.3	99.2	90.2	88.4	77.9	67.0	60.0
1500	109.6	96.3	87.8	86.1	74.9	64.6	58.0
1400	105.9	93.4	85.4	83.8	71.9	62.3	56.0
1300	102.1	90.5	83.0	81.6	68.8	59.9	54.1
1200	98.4	87.6	80.4	78.0	65.8	57.5	52.1
1100	94.7	84.7	76.2	74.0	62.8	55.1	50.2
1000	91.0	81.8	72.0	70.0	59.8	52.7	48.2
900	87.3	77.3	67.9	66.0	56.7	50.4	46.2
800	83.6	72.2	63.7	62.1	53.7	48.0	44.3
700	79.1	67.1	59.5	58.1	50.7	45.6	42.3
600	72.4	61.9	55.4	54.1	47.6	43.2	40.0
500	65.7	56.8	51.2	50.1	44.6	40.8	33.3
400	59.1	51.7	47.0	46.1	41.6	33.3	26.7
300	52.4	46.5	42.9	42.2	33.3	25.0	20.0
200	45.7	41.4	33.3	31.4	22.2	16.7	13.3
100	33.3	22.2	16.7	15.7	11.1	8.3	6.7
	48	60	72	75	96	120	144
	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft

Post Spacing, L

36" Effective height of rails, $\bar{Y}_{RAILS}$

Table 22. Guidance Charts for Deck #3 and Single-Unit Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Deck #3, SUT
DMF=1.0

Plastic Moment Capacity of Rails, Σ Mp RAILS (kips - in.)	48 4 ft	60 5 ft	72 6 ft	75 6 ft - 3 in	96 8 ft	120 10 ft	144 12 ft
4800	158.7	134.7	119.2	116.1	100.2	88.6	80.2
4700	156.4	132.9	117.7	114.7	99.1	87.6	79.3
4600	154.2	131.1	116.2	113.3	98.0	86.6	78.5
4500	151.9	129.4	114.8	111.9	96.9	85.6	77.7
4400	149.6	127.6	113.3	110.5	95.9	84.6	76.9
4300	147.4	125.8	111.8	109.1	94.8	83.6	76.1
4200	145.1	124.0	110.4	107.7	93.7	82.6	75.2
4100	142.9	122.3	108.9	106.3	92.6	81.6	74.4
4000	140.6	120.5	107.4	104.9	91.4	80.6	73.6
3900	138.3	118.7	106.0	103.5	90.1	79.7	72.8
3800	136.1	116.9	104.5	102.1	88.9	78.7	72.0
3700	133.8	115.1	103.0	100.7	87.6	77.7	71.2
3600	131.6	113.4	101.6	99.2	86.4	76.7	70.3
3500	129.3	111.6	100.1	97.8	85.1	75.7	69.5
3400	127.0	109.8	98.6	96.4	83.9	74.7	68.4
3300	124.8	108.0	97.2	95.0	82.6	73.7	67.2
3200	122.5	106.3	95.7	93.6	81.4	72.7	66.1
3100	120.3	104.5	94.3	92.2	80.2	71.8	64.9
3000	118.0	102.7	92.8	90.6	78.9	70.8	63.8
2900	115.7	100.9	91.1	88.9	77.7	69.8	62.6
2800	113.5	99.1	89.4	87.3	76.4	68.5	61.4
2700	111.2	97.4	87.8	85.7	75.2	67.1	60.3
2600	109.0	95.6	86.1	84.1	73.9	65.7	59.1
2500	106.7	93.8	84.4	82.5	72.7	64.3	58.0
2400	104.4	92.0	82.7	80.9	71.4	62.9	56.8
2300	102.2	89.9	81.0	79.2	70.2	61.5	55.6
2200	99.9	87.9	79.3	77.6	68.7	60.1	54.5
2100	97.7	85.8	77.6	76.0	67.0	58.7	53.3
2000	95.4	83.8	75.9	74.4	65.2	57.3	52.2
1900	93.1	81.7	74.3	72.8	63.4	55.9	51.0
1800	90.6	79.6	72.6	71.2	61.6	54.5	49.8
1700	88.0	77.6	79.4	69.6	59.8	53.1	48.7
1600	85.4	75.5	69.1	67.3	58.1	51.7	47.5
1500	82.8	73.5	66.7	65.0	56.3	50.3	46.4
1400	80.2	71.4	64.2	62.7	54.5	48.9	44.4
1300	77.5	69.4	61.8	60.3	52.7	47.5	42.5
1200	74.9	66.4	59.4	58.0	51.0	45.9	40.5
1100	72.3	63.4	57.0	55.7	49.2	43.5	38.5
1000	69.7	60.5	54.5	53.4	47.4	41.1	36.5
900	66.0	57.5	52.1	51.1	45.1	38.6	34.5
800	62.2	54.6	49.7	48.7	42.0	36.2	32.5
700	58.4	51.6	47.3	46.4	38.9	33.8	30.5
600	54.6	48.6	43.7	42.4	35.8	31.4	28.5
500	50.8	45.2	39.4	38.3	32.7	29.0	26.5
400	47.0	39.8	35.1	34.2	29.6	26.5	24.5
300	40.5	34.5	30.8	30.1	26.5	24.1	21.1
200	33.5	29.2	26.5	26.0	23.4	17.8	14.0
100	26.5	23.8	19.0	17.8	12.1	8.9	7.0

Post Spacing, L

42" Effective height of rails, $\bar{Y}_{RAILS}$

Deck #3, SUT
DMF=1.5

Plastic Moment Capacity of Rails, Σ Mp RAILS (kips - in.)	48 4 ft	60 5 ft	72 6 ft	75 6 ft - 3 in	96 8 ft	120 10 ft	144 12 ft
3200	147.6	129.7	117.3	114.8	101.3	88.8	80.6
3100	145.4	127.7	115.6	113.2	99.5	87.4	79.4
3000	143.1	125.6	113.9	111.6	97.8	86.0	78.3
2900	140.8	123.6	112.2	110.0	96.0	84.6	77.1
2800	138.5	121.5	110.5	108.4	94.2	83.1	75.9
2700	135.9	119.5	108.8	106.8	92.4	81.7	74.8
2600	133.3	117.4	107.2	105.1	90.7	80.3	73.6
2500	130.7	115.4	105.5	103.3	88.9	78.9	72.5
2400	128.1	113.3	103.6	100.9	87.1	77.5	71.3
2300	125.5	111.3	101.2	98.6	85.3	76.1	70.1
2200	122.9	109.2	98.8	96.3	83.5	74.7	68.7
2100	120.2	107.2	96.4	94.0	81.8	73.3	66.7
2000	117.6	105.1	93.9	91.7	80.0	71.9	64.7
1900	115.0	102.6	91.5	89.3	78.2	70.5	62.7
1800	112.4	99.6	89.1	87.0	76.4	68.9	60.7
1700	109.8	96.7	96.8	84.7	74.7	66.5	58.7
1600	107.2	93.7	84.2	82.4	72.9	64.0	56.7
1500	104.6	90.7	81.8	80.1	71.1	61.6	54.7
1400	100.9	87.8	79.4	77.7	69.2	59.2	52.7
1300	97.1	84.8	77.0	75.4	66.1	56.8	50.7
1200	93.3	81.8	74.5	73.1	63.0	54.3	48.8
1100	89.5	78.9	72.1	70.8	59.9	51.9	46.8
1000	85.7	75.9	69.7	67.7	56.8	49.5	44.8
900	81.9	73.0	65.6	63.6	53.7	47.1	42.8
800	78.1	70.0	61.3	59.5	50.6	44.6	40.8
700	74.3	65.1	57.0	55.4	47.5	42.2	38.8
600	70.5	59.8	52.7	51.3	44.4	39.8	36.8
500	64.3	54.4	48.4	47.2	41.3	37.4	34.8
400	57.3	49.1	44.1	43.1	38.2	34.9	28.1
300	50.3	43.8	39.8	39.0	35.1	26.7	21.1
200	43.3	38.4	35.5	34.9	24.2	17.8	14.0
100	36.3	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing, L

42" Effective height of rails, $\bar{Y}_{RAILS}$

Table 23. Guidance Charts for Deck #4 and Pickup Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Deck #4, PICKUP DMF=1.0

Plastic Moment Capacity of Rails, Σ $M_{P\ RAILS}$ (kips - in.)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2900	129.2	113.8	103.0	100.9	89.2	78.2	71.0
2800	127.0	111.8	101.3	99.3	87.5	76.9	69.9
2700	124.8	109.8	99.7	97.7	85.7	75.5	68.7
2600	122.6	107.8	98.0	96.1	84.0	74.1	67.6
2500	120.2	105.7	96.3	94.5	82.2	72.7	66.4
2400	117.6	103.7	94.7	92.9	80.5	71.3	65.3
2300	115.1	101.7	93.0	91.3	78.7	69.9	64.1
2200	112.5	99.7	91.3	89.1	77.0	68.5	63.0
2100	109.9	97.7	89.1	86.8	75.2	67.1	61.8
2000	107.4	95.6	86.7	84.5	73.5	65.7	60.7
1900	104.8	93.6	84.3	82.2	71.7	64.4	58.7
1800	102.2	91.6	81.9	80.0	69.9	63.0	56.7
1700	99.7	88.9	80.7	77.7	68.2	61.6	54.8
1600	97.1	86.0	77.1	75.4	66.4	59.8	52.8
1500	94.5	83.1	74.8	73.1	64.7	57.4	50.9
1400	92.0	80.2	72.4	70.9	62.9	55.0	48.9
1300	88.6	77.3	70.0	68.6	61.2	52.6	46.9
1200	84.9	74.4	67.6	66.3	58.5	50.3	45.0
1100	81.2	71.5	65.2	64.0	55.4	47.9	43.0
1000	77.5	68.6	62.9	61.7	52.4	45.5	41.0
900	73.8	65.7	60.3	58.5	49.4	43.1	39.1
800	70.1	62.8	56.1	54.5	46.3	40.7	37.1
700	66.4	59.3	51.9	50.5	43.3	38.4	35.2
600	62.7	54.1	47.8	46.5	40.3	36.0	33.2
500	57.6	49.0	43.6	42.6	37.2	33.6	31.2
400	51.0	43.9	39.4	38.6	34.2	31.2	26.7
300	44.3	38.8	35.3	34.6	31.2	25.0	20.0
200	37.6	33.6	31.1	30.6	22.2	16.7	13.3
100	31.0	22.2	16.7	15.7	11.1	8.3	6.7

Post Spacing, L

32" Effective height of rails, $\bar{Y}_{RAILS}$

Deck #4, PICKUP DMF=1.5

Plastic Moment Capacity of Rails, Σ $M_{P\ RAILS}$ (kips - in.)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2000	134.8	117.4	106.2	104.0	92.6	80.2	71.4
1900	131.1	114.5	103.8	101.7	90.7	77.8	69.4
1800	127.4	111.6	101.4	99.4	87.7	75.4	67.5
1700	123.7	108.7	100.8	97.2	84.7	73.0	65.5
1600	120.0	105.8	96.7	94.9	81.6	70.6	63.5
1500	116.3	102.9	94.3	92.6	78.6	68.3	61.6
1400	112.6	100.0	91.9	89.7	75.6	65.9	59.6
1300	108.9	97.1	88.3	85.7	72.5	63.5	57.7
1200	105.2	94.2	84.2	81.8	69.5	61.1	55.7
1100	101.5	91.3	80.0	77.8	66.5	58.7	53.7
1000	97.8	86.3	75.8	73.8	63.4	56.4	51.8
900	94.1	81.2	71.7	69.8	60.4	54.0	49.8
800	89.8	76.1	67.5	65.8	57.4	51.6	47.8
700	83.1	70.9	63.3	61.9	54.3	49.2	45.9
600	76.5	65.8	59.2	57.9	51.3	46.8	40.0
500	69.8	60.7	55.0	53.9	48.3	41.7	33.3
400	63.1	55.6	50.8	49.9	44.4	33.3	26.7
300	56.5	50.4	46.7	45.9	33.3	25.0	20.0
200	49.8	44.4	33.3	31.4	22.2	16.7	13.3
100	33.3	22.2	16.7	15.7	11.1	8.3	6.7

Post Spacing, L

32" Effective height of rails, $\bar{Y}_{RAILS}$

Table 24. Guidance Charts for Deck #4 and Single-Unit Truck Design Impact Loading with W6x15 (W150x22.5) Steel Posts

Deck #4, SUT
DMF=1.0

Plastic Moment Capacity of Rails, Σ $M_{P\,RAILS}$ (kips - in.)	48 4 ft	60 5 ft	72 6 ft	75 6 ft - 3 in	96 8 ft	120 10 ft	144 12 ft
4300	152.7	131.0	117.0	114.2	99.5	87.9	80.4
4200	150.4	129.2	115.5	112.8	98.2	87.0	79.6
4100	148.1	127.5	114.0	111.4	97.0	86.0	78.7
4000	145.9	125.7	112.6	110.0	95.7	85.0	77.9
3900	143.6	123.9	111.1	108.6	94.5	84.0	77.1
3800	141.4	122.1	109.6	107.2	93.3	83.0	76.1
3700	139.1	120.3	108.2	105.8	92.0	82.0	74.9
3600	136.8	118.6	106.7	104.4	90.8	81.0	73.8
3500	134.6	116.8	105.3	103.0	89.5	80.0	72.6
3400	132.3	115.0	103.8	101.5	88.3	79.1	71.4
3300	130.1	113.2	102.3	99.8	87.0	78.1	70.3
3200	127.8	111.5	100.6	98.2	85.8	77.1	69.1
3100	125.5	109.7	98.9	96.6	84.5	75.8	68.0
3000	123.3	107.9	97.3	95.0	83.3	74.4	66.8
2900	121.0	106.1	95.6	93.4	82.0	73.0	65.7
2800	118.8	104.3	93.9	91.8	80.8	71.6	64.5
2700	116.5	102.6	92.2	90.1	79.5	70.2	63.3
2600	114.3	100.6	90.5	88.5	78.3	68.8	62.2
2500	112.0	98.5	88.8	86.9	77.1	67.4	61.0
2400	109.7	96.5	87.1	85.3	75.4	66.0	59.9
2300	107.5	94.4	85.4	83.7	73.6	64.6	58.7
2200	105.2	92.4	83.8	82.1	71.9	63.2	57.5
2100	103.0	90.3	82.1	80.4	70.1	61.8	56.4
2000	100.4	88.2	80.4	78.8	68.3	60.4	55.2
1900	97.8	86.2	78.7	77.2	66.5	59.0	54.1
1800	95.2	84.1	77.0	75.1	64.7	57.6	52.9
1700	92.6	82.1	75.4	72.8	63.0	56.2	51.7
1600	90.0	80.0	72.3	70.5	61.2	54.8	50.2
1500	87.4	78.0	69.8	68.1	59.4	53.4	48.2
1400	84.7	75.6	67.4	65.8	57.6	52.0	46.2
1300	82.1	72.6	65.0	63.5	55.9	50.1	44.2
1200	79.5	69.7	62.6	61.2	54.1	47.7	42.2
1100	76.9	66.7	60.2	58.9	52.3	45.3	40.2
1000	73.2	63.7	57.7	56.6	50.0	42.8	38.2
900	69.4	60.8	55.3	54.2	46.9	40.4	36.2
800	65.6	57.8	52.9	51.9	43.8	38.0	34.2
700	61.7	54.8	49.9	48.4	40.7	35.6	32.3
600	57.9	51.9	45.6	44.3	37.6	33.1	30.3
500	54.1	47.1	41.3	40.2	34.5	30.7	28.3
400	49.6	41.8	37.0	36.1	31.4	28.3	26.3
300	42.6	36.5	32.7	32.0	28.3	25.9	21.1
200	35.6	31.1	28.4	27.9	24.2	17.8	14.0
100	28.6	25.8	19.0	17.8	12.1	8.9	7.0

Post Spacing, L

38" Effective height of rails, $\bar{Y}_{RAILS}$

Deck #4, SUT
DMF=1.5

Plastic Moment Capacity of Rails, Σ $M_{P\,RAILS}$ (kips - in.)	48 4 ft	60 5 ft	72 6 ft	75 6 ft - 3 in	96 8 ft	120 10 ft	144 12 ft
2800	145.4	128.3	117.2	115.0	98.9	87.8	80.5
2700	142.8	126.2	115.5	112.7	97.1	86.4	79.3
2600	140.2	124.2	113.3	110.3	95.3	85.0	78.2
2500	137.6	122.1	110.8	108.0	93.6	83.6	77.0
2400	135.0	120.1	108.4	105.7	91.8	82.1	75.2
2300	132.4	118.0	106.0	103.4	90.0	80.7	73.3
2200	129.7	116.0	103.6	101.1	88.2	79.3	71.3
2100	127.1	113.4	101.1	98.7	86.4	77.9	69.3
2000	124.5	110.4	98.7	96.4	84.7	76.4	67.3
1900	121.9	107.5	96.3	94.1	82.9	74.0	65.3
1800	119.3	104.5	93.9	91.8	81.1	71.5	63.3
1700	116.7	101.5	92.5	89.5	79.3	69.1	61.3
1600	113.6	98.6	89.0	87.1	77.6	66.7	59.3
1500	109.8	95.6	86.6	84.8	75.1	64.3	57.3
1400	106.0	92.6	84.2	82.5	72.0	61.8	55.3
1300	102.1	89.7	81.7	80.2	68.9	59.4	53.4
1200	98.3	86.7	79.3	77.9	65.8	57.0	51.4
1100	94.5	83.8	76.9	74.6	62.7	54.6	49.4
1000	90.7	80.8	72.7	70.5	59.6	52.1	47.4
900	86.9	77.8	68.4	66.4	56.5	49.7	45.4
800	83.1	73.4	64.1	62.3	53.4	47.3	43.4
700	79.3	68.0	59.8	58.2	50.3	44.9	41.4
600	74.4	62.7	55.5	54.1	47.2	42.4	39.4
500	67.4	57.4	51.2	50.0	44.1	40.0	35.1
400	60.4	52.0	46.9	45.9	41.0	35.6	28.1
300	53.4	46.7	42.6	41.8	36.4	26.7	21.1
200	46.3	41.4	38.1	35.6	24.2	17.8	14.0
100	39.3	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing, L

38" Effective height of rails, $\bar{Y}_{RAILS}$

Additional guidance charts for SUT impact events were created to provide assistance with configuring future steel, beam-and-post, bridge rails. Similarly to previous guidance charts, the MASH TL-4 SUT design impact load was used with DMFs equal to 1.0 and 1.5 for a range of effective height of rails, $\bar{Y}_{RAILS}$, and W6x15 (W150x22.5) steel posts. The effective height of the rails ranged from 30 in. (762 mm) through 42 in. (1067 mm) in order to top-mounted posts with baseplates as well as side-mounted posts with the tension anchor rods located at a depth of 12 in. (305 mm) below the top of the concrete deck, as depicted in Figure 51. The 13 additional guidance charts are shown in Table B-3 through Table B-15 of Appendix B.

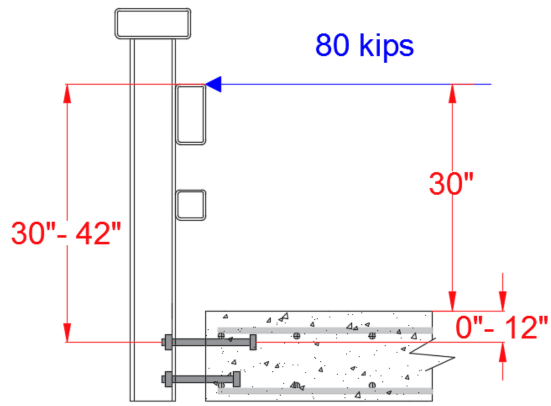


Figure 51. SUT Design Impact Loading for Use with Additional Guidance Charts in Appendix B

5.5 Preliminary Bridge Rail Configurations

Preliminary bridge rail configurations were developed using the guidance charts for four concrete deck types to resist only SUT impacts; since, they represented the most critical impact conditions for the three MASH TL-4 crash tests. The post spacing options were reduced to 6 ft (1.8 m), 8 ft (2.4 m), and 10 ft (3.0 m) based on feedback obtained from representatives with the Illinois and Ohio DOTs. An asphalt overlay of 3 in. (76 mm) was used to maximize the moment arm between the design impact load and the tension anchor rods.

One of the design objectives for the bridge rail was to have the front face of the rails aligned flush with the exterior vertical edge of the concrete deck. Thus, preliminary bridge rail configurations were created for 4-in. (102-mm), 5-in. (127-mm), and 6-in. (152-mm) lateral offsets between the post flange and deck edge, as provided in Tables 25 through 36. Note that minimum combined moment capacities are shown in parentheses. For each horizontal rail, the plastic section modulus was obtained from the AISC *Steel Construction Manual* [49]. The thicknesses of the three rail sections were initially intended to be equal for simplification purposes. The vertical heights of the lower two rails were also intended to be equal, but it was not always possible due to limited sizes available for 5-in. (127-mm) wide rectangular HSS sections. Therefore, the lower two rails for bridge rail systems configured with a 5-in. (127-mm) wide mounting bracket had different vertical depths.

Table 25. Preliminary Bridge Rail Configurations for Deck #1 and 4-in. (102-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	12x4x5/16	1408.5	31.84	8x4x1/4	598.5	19.02
	Middle	8x4x5/16	446.0	23.34	8x4x1/4	369.0	19.02
	Bottom	8x4x5/16	446.0	23.34	8x4x1/4	369.0	19.02
			∑=2300.5 (2000)	***∑=78.52***		***∑=1336.5 (1200)***	***∑=57.06***
8	Top	10x4x1/2	1534.5	42.05	10x4x5/16	1039.5	27.59
	Middle	8x4x1/2	643.5	35.24	8x4x5/16	446.0	23.34
	Bottom	8x4x1/2	643.5	35.24	8x4x5/16	446.0	23.34
			∑=2821.5 (2700)	***∑=112.53***		***∑=1931.5 (1700)***	***∑=74.27***
10	Top	12x4x5/8	2497.5	59.32	12x4x5/16	1408.5	31.84
	Middle	8x4x5/8	747.0	42.30	8x4x5/16	446.0	23.34
	Bottom	8x4x5/8	747.0	42.30	8x4x5/16	446.0	23.34
			∑=3991.5 (3400)	***∑=143.92***		***∑=2300.5 (2200)***	***∑=78.52***

[1] – Minimum combined moment capacities shown in parentheses.

Table 26. Preliminary Bridge Rail Configurations for Deck #1 and 5-in. (127-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	9x5x5/16	990.0	27.59	10x4x3/16	657.0	17.08
	Middle	9x5x5/16	657.0	27.59	9x5x3/16	416.2	17.08
	Bottom	7x5x5/16	535.5	23.34	7x5x3/16	340.6	14.53
			∑=2182.5 (2000)	***∑=78.52***		***∑=1413.9 (1200)***	***∑=48.69***
8	Top	12x6x5/16	1714.5	36.1	10x5x1/4	958.5	24.12
	Middle	9x5x5/16	657.0	27.59	9x5x1/4	540.0	22.42
	Bottom	7x5x5/16	535.5	23.34	7x5x1/4	442.3	19.02
			∑=2907 (2700)	***∑=87.03***		***∑=1940.8 (1700)***	***∑=65.56***
10	Top	12x4x1/2	2101.5	48.85	12x6x1/4	1399.5	29.23
	Middle	9x5x1/2	967.5	42.05	9x5x1/4	540.0	22.42
	Bottom	7x5x1/2	778.5	35.24	7x5x1/4	442.3	19.02
			∑=3847.5 (3400)	***∑=126.14***		***∑=2381.85 (2200)***	***∑=70.67***

[1] – Minimum combined moment capacities shown in parentheses.

Table 27. Preliminary Bridge Rail Configurations for Deck #1 and 6-in. (152-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	10x5x1/4	958.5	24.12	8x6x3/16	585.0	17.08
	Middle	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
	Bottom	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
			∑=2209.5 (2000)	***∑=68.96***		***∑=1548.0 (1200)***	***∑=51.24***
8	Top	12x4x5/16	1408.5	31.84	10x6x3/16	810.0	19.63
	Middle	8x6x5/16	760.5	27.59	8x6x3/16	481.5	17.08
	Bottom	8x6x5/16	760.5	27.59	8x6x3/16	481.5	17.08
			∑=2929.5 (2700)	***∑=87.02***		***∑=1773.0 (1700)***	***∑=53.79***
10	Top	12x6x3/8	2016.0	42.79	10x6x1/4	1062	25.82
	Middle	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
	Bottom	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
			∑=3798 (3400)	***∑=107.95***		***∑=2313 (2200)***	***∑=70.66***

[1] – Minimum combined moment capacities shown in parentheses.

Table 28. Preliminary Bridge Rail Configurations for Deck #2 and 4-in. (102-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	12x4x5/16	1408.5	31.84	8x4x1/4	598.5	19.02
	Middle	8x4x5/16	446.0	23.34	8x4x1/4	369	19.02
	Bottom	8x4x5/16	446.0	23.34	8x4x1/4	369	19.02
			∑=2300.5 (2000)	***∑=78.52***		***∑=1336.5 (1200)***	***∑=57.06***
8	Top	10x4x1/2	1534.5	42.05	10x4x5/16	1039.5	27.59
	Middle	8x4x1/2	643.5	35.24	8x4x5/16	446.0	23.34
	Bottom	8x4x1/2	643.5	35.24	8x4x5/16	446.0	23.34
			∑=2821.5 (2700)	***∑=112.53***		***∑=1931.5 (1700)***	***∑=74.27***
10	Top	12x4x5/8	2497.5	59.32	12x4x5/16	1408.5	31.84
	Middle	8x4x5/8	747	42.3	8x4x5/16	446.0	23.34
	Bottom	8x4x5/8	747	42.3	8x4x5/16	446.0	23.34
			∑=3991.5 (3500)	***∑=143.92***		***∑=2300.5 (2200)***	***∑=78.52***

[1] – Minimum combined moment capacities shown in parentheses.

Table 29. Preliminary Bridge Rail Configurations for Deck #2 and 5-in. (127-mm) Wide Lower Rails.

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	9x5x5/16	990	27.59	10x4x3/16	657	17.08
	Middle	9x5x5/16	657	27.59	9x5x3/16	416.25	17.08
	Bottom	7x5x5/16	535.5	23.34	7x5x3/16	340.65	14.53
			∑=2182.5 (2000)	***∑=78.52***		***∑=1413.9 (1200)***	***∑=48.69***
8	Top	12x6x5/16	1714.5	36.1	10x5x1/4	958.5	24.12
	Middle	9x5x5/16	657	27.59	9x5x1/4	540.0	22.42
	Bottom	7x5x5/16	535.5	23.34	7x5x1/4	442.3	19.02
			∑=2907 (2700)	***∑=87.03***		***∑=1940.8 (1700)***	***∑=65.56***
10	Top	12x4x1/2	2101.5	48.85	12x6x1/4	1399.5	29.23
	Middle	9x5x1/2	967.5	42.05	9x5x1/4	540.0	22.42
	Bottom	7x5x1/2	778.5	35.24	7x5x1/4	442.3	19.02
			∑=3847.5 (3500)	***∑=126.14***		***∑=2381.8 (2200)***	***∑=70.67***

[1] – Minimum combined moment capacities shown in parentheses.

Table 30. Preliminary Bridge Rail Configurations for Deck #2 and 6-in. (152-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	10x5x1/4	958.5	24.12	8x6x3/16	585.0	17.08
	Middle	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
	Bottom	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
			∑=2209.5 (2000)	***∑=68.96***		***∑=1548.0 (1200)***	***∑=51.24***
8	Top	12x4x5/16	1408.5	31.84	10x6x3/16	1062	19.63
	Middle	8x6x5/16	760.5	27.59	8x6x3/16	625.5	17.08
	Bottom	8x6x5/16	760.5	27.59	8x6x3/16	625.5	17.08
			∑=2929.5 (2700)	***∑=87.02***		***∑=1773 (1700)***	***∑=53.79***
10	Top	12x6x3/8	2016.0	42.79	10x6x1/4	1062.0	25.82
	Middle	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
	Bottom	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
			∑=3798 (3500)	***∑=107.95***		***∑=2313.0 (2200)***	***∑=70.66***

[1] – Minimum combined moment capacities shown in parentheses.

Table 31. Preliminary Bridge Rail Configurations for Deck #3 and 4-in. (102-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	12x4x3/8	1651.5	37.69	10x4x1/4	855.0	22.42
	Middle	8x4x3/8	517.5	27.48	8x4x1/4	369.0	19.02
	Bottom	8x4x3/8	517.5	27.48	8x4x1/4	369.0	19.02
			∑=2686.5 (2300)	***∑=92.65***		***∑=1593 (1500)***	***∑=60.46***
8	Top	12x4x1/2	2101.5	48.85	10x4x3/8	1215.0	32.58
	Middle	8x4x1/2	643.5	35.24	8x4x3/8	517.5	27.48
	Bottom	8x4x1/2	643.5	35.24	8x4x3/8	517.5	27.48
			∑=3388.5 (3100)	***∑=119.33***		***∑=2250 (2100)***	***∑=87.54***
10	Top	12x6x5/8	3096.0	67.82	12x4x3/8	1651.5	37.69
	Middle	8x4x5/8	747.0	42.3	8x4x3/8	517.5	27.48
	Bottom	8x4x5/8	747.0	42.3	8x4x3/8	517.5	27.48
			∑=4590 (4000)	***∑=152.42***		***∑=2686.5 (2600)***	***∑=92.65***

[1] – Minimum combined moment capacities shown in parentheses.

Table 32. Preliminary Bridge Rail Configurations for Deck #3 and 5-in. (127-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	10x4x3/8	1215.0	32.58	10x4x1/4	855.0	22.42
	Middle	9x5x3/8	769.5	32.58	7x5x1/4	442.4	19.02
	Bottom	7x5x3/8	621.0	27.48	7x5x1/4	442.4	19.02
			∑=2605.5 (2300)	***∑=92.64***		***∑=1739.8 (1500)***	***∑=60.46***
8	Top	12x6x3/8	2016.0	42.79	10x5x5/16	1170.0	29.72
	Middle	9x5x3/8	769.5	32.58	9x5x5/16	657.0	27.59
	Bottom	7x5x3/8	621.0	27.48	7x5x5/16	535.5	23.34
			∑=3406.5 (3100)	***∑=102.85***		***∑=2362.5 (2100)***	***∑=80.65***
10	Top	12x6x1/2	2583.0	55.66	10x6x3/8	1521.0	37.69
	Middle	9x5x1/2	967.5	42.05	9x5x3/8	769.5	32.58
	Bottom	7x5x1/2	778.5	35.24	7x5x3/8	621.0	27.48
			∑=4590.0 (4000)	***∑=132.95***		***∑=2911.5 (2600)***	***∑=97.75***

[1] – Minimum combined moment capacities shown in parentheses.

Table 33. Preliminary Bridge Rail Configurations for Deck #3 and 6-in. (152-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	12x6x1/4	1399.5	29.23	10x6x3/16	810.0	19.63
	Middle	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
	Bottom	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
			∑=2650.5 (2300)	***∑=74.07***		***∑=1773 (1500)***	***∑=53.79***
8	Top	12x4x3/8	1651.5	37.69	10x6x1/4	1062.0	25.82
	Middle	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
	Bottom	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
			∑=3433.5 (3100)	***∑=102.85***		***∑=2313.0 (2100)***	***∑=70.66***
10	Top	12x4x1/2	2101.5	48.85	12x4x5/16	1408.5	31.84
	Middle	8x6x1/2	1120.5	42.05	8x6x5/16	760.5	27.59
	Bottom	8x6x1/2	1120.5	42.05	8x6x5/16	760.5	27.59
			∑=4342.5 (4000)	***∑=132.95***		***∑=2929.5 (2600)***	***∑=87.02***

[1] – Minimum combined moment capacities shown in parentheses.

Table 34. Preliminary Bridge Rail Configurations for Deck #4 and 4-in. (102-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	12x4x5/16	1408.5	31.84	8x4x1/4	598.5	19.02
	Middle	8x4x5/16	446.0	23.34	8x4x1/4	369.0	19.02
	Bottom	8x4x5/16	446.0	23.34	8x4x1/4	369.0	19.02
			∑=2300.5 (2000)	***∑=78.52***		***∑=1336.5 (1300)***	***∑=57.06***
8	Top	10x4x1/2	1534.5	42.05	10x4x5/16	1039.5	27.59
	Middle	8x4x1/2	643.5	35.24	8x4x5/16	446.0	23.34
	Bottom	8x4x1/2	643.5	35.24	8x4x5/16	446.0	23.34
			∑=2821.5 (2800)	***∑=112.53***		***∑= 1931.4 (1800)***	***∑=74.27***
10	Top	10x6x5/8	2308.5	59.32	12x4x5/16	1408.5	31.84
	Middle	8x4x5/8	747.0	42.3	8x4x5/16	446.0	23.34
	Bottom	8x4x5/8	747.0	42.3	8x4x5/16	446.0	23.34
			∑=3802.5 (3500)	***∑=143.8***		***∑=2300.5 (2300)***	***∑=78.52***

[1] – Minimum combined moment capacities shown in parentheses.

Table 35. Preliminary Bridge Rail Configurations for Deck #4 and 5-in. (127-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	9x5x5/16	990.0	27.59	10x4x3/16	657.0	17.08
	Middle	9x5x5/16	657.0	27.59	9x5x3/16	416.3	17.08
	Bottom	7x5x5/16	535.5	23.34	7x5x3/16	340.7	14.53
			∑=2182.5 (2000)	***∑=78.52***		***∑= 1413.6 (1300)***	***∑=48.69***
8	Top	12x6x5/16	1714.5	36.10	10x5x1/4	958.5	24.12
	Middle	9x5x5/16	657.0	27.59	9x5x1/4	540.0	22.42
	Bottom	7x5x5/16	535.5	23.34	7x5x1/4	442.4	19.02
			∑=2907 (2800)	***∑=87.03***		***∑=1940.9 (1800)***	***∑=65.56***
10	Top	12x4x1/2	2101.5	48.85	12x6x1/4	1399.5	29.23
	Middle	9x5x1/2	967.5	42.05	9x5x1/4	540	22.42
	Bottom	7x5x1/2	778.5	35.24	7x5x1/4	442.35	19.02
			∑=3847.5 (3500)	***∑=126.14***		***∑=2381.85 (2300)***	***∑=70.67***

[1] – Minimum combined moment capacities shown in parentheses.

Table 36. Preliminary Bridge Rail Configurations for Deck #4 and 6-in. (152-mm) Wide Lower Rails

Post Spacing (ft)	Rail Position	Post DMF = 1.0			Post DMF = 1.5		
		HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)	HSS Rail	M_p[1] (k-in.)	Weight (lb/ft)
6	Top	10x5x1/4	958.5	24.12	8x6x3/16	585.0	17.08
	Middle	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
	Bottom	8x6x1/4	625.5	22.42	8x6x3/16	481.5	17.08
			∑=2209.5 (2000)	***∑=68.96***		***∑=1548.0 (1300)***	***∑=51.24***
8	Top	12x4x5/16	1408.5	31.84	10x6x3/16	810.0	19.63
	Middle	8x6x5/16	760.5	27.59	8x6x3/16	481.5	17.08
	Bottom	8x6x5/16	760.5	27.59	8x6x3/16	481.5	17.08
			∑=2929.5 (2800)	***∑=87.02***		***∑=1773.0 (1800)***	***∑=53.79***
10	Top	12x6x3/8	2016.0	42.79	10x6x1/4	1062.0	25.82
	Middle	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
	Bottom	8x6x3/8	891.0	32.58	8x6x1/4	625.5	22.42
			∑=3798.0 (3500)	***∑=107.95***		***∑=2313.0 (2300)***	***∑=70.66***

[1] – Minimum combined moment capacities shown in parentheses.

From the preliminary bridge rail configurations shown in Tables 25 through 36, it is noticeable that deck #3 resulted in the highest weight per foot of the three rails. This observation correlates with the interpretation defined in Section 3.2 that bridge deck configuration #3 with a 3-in. (76-mm) thick asphalt overlay corresponded to the most critical bridge deck, which had the largest moment arm between the design loading height and the location of the tension anchor rods. Moreover, it was recognized that an increase in the post spacing generally resulted in a greater weight per foot for the three rails.

5.6 Improved Bridge Rail Configurations for Critical Bridge Deck

The rail sections of preliminary bridge rail configurations were modified to improve the weight per foot of the three rails. The size and thickness of the three rail sections were not required to be equal for these system configurations. These bridge rail configurations were designed for bridge deck configuration #3 with a 3-in. (76-mm) thick roadway asphalt overlay, which would likely maximize lateral barrier deflections as well as the propensity for vehicle instability, rollover, and/or override of the bridge rail.

In order to create improved bridge rail configurations, the rail sections were optimized based on reducing the overall weight per foot of the system. The weight per foot of the bridge rail was based on the weight per foot of the three rails, the length of the posts, and the estimated weight of the post-to-deck connections. The rail splice hardware and all other connection hardware would also contribute to the overall weight of the system weight per foot but were omitted at this time. The length of the posts was assumed to start from the bottom edge of the concrete deck and end at the bottom edge of the top rail. The depth (i.e., thickness) of the concrete deck was initially assumed to be 26 in. (660 mm). Therefore, the length of the posts was 56 in. (1422 mm) or 58 in. (1,473 mm) when the top rail depth (i.e., height) was 4 in. (102 mm) or 6 in. (152 mm), respectively. According to the AISC *Steel Construction Manual* [49], W6x15 (W150x22.5) steel sections weigh 15 lb/ft (2.1 kg/m). Therefore, the weight of each post was assumed to be 72.5 lb (32.9 kg) when the top rail depth was 4 in. (102 mm) and 70 lb (31.8 kg) with a top rail depth of 6 in. (152 mm). Moreover, the post-to-deck connection hardware (i.e., steel plates) were assumed to weigh approximately 50 lb (22.7 kg). The improved bridge rail configurations are depicted in Tables 37 through 45. The bridge rail configurations with the smallest weight per foot for each impact event scenario are shown in Tables 37 through 45 using yellow highlighting. The lowest-weight configurations are summarized in Table 46, which were then subjected to further analysis.

Table 37. Improved Bridge Rail Configurations for 4-in. (102-mm) Wide Lower Rails and Post Spacing of 6 ft (1.8 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
4	6	58	1	Rails	Top	HSS10X4X1/2		42.1	34.1	1534.5		SEVEN-SPAN	81.7
					Middle	HSS8X4X5/16		23.3	9.91	446.0			
					Bottom	HSS8X4X5/16		23.3	9.91	446.0			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 109.1		Σ = 2426.4	2300		
				Rails	Top	HSS10X6X3/8		37.7	33.8	1521.0		SEVEN-SPAN	80.8
					Middle	HSS8X4X5/16		23.3	9.91	446.0			
					Bottom	HSS8X4X5/16		23.3	9.91	446.0			
				Post		W6x15	70	11.7					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 104.4		Σ = 2412.9	2300		
				Rails	Top	HSS10X6X3/8		37.7	33.8	1521.0		SEVEN-SPAN	80.2
					Middle	HSS8X4X5/16		23.3	9.91	446.0			
					Bottom	HSS7X4X5/16		21.2	8.83	397.4			
				Post		W6x15	70	11.7					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 102.2		Σ = 2364.3	2300		
			1.5	Rails	Top	HSS10X4X1/4		22.4	19	855.0		FIVE-SPAN	81.4
					Middle	HSS8X4X1/4		19.0	8.2	369.0			
					Bottom	HSS8X4X1/4		19.0	8.2	369.0			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 80.9		Σ = 1593.0	1500		
				Rails	Top	HSS10X4X1/4		22.4	19	855.0		FIVE-SPAN	80.7
					Middle	HSS8X4X1/4		19.0	8.2	369.0			
					Bottom	HSS7X4X1/4		17.3	7.33	329.9			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 79.2		Σ = 1553.9	1500		
				Rails	Top	HSS12X4X3/16		19.6	19.6	882.0		FIVE-SPAN	80.8
					Middle	HSS8X4X1/4		19.0	8.2	369.0			
					Bottom	HSS6X4X1/4		15.6	6.45	290.3			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 74.7		Σ = 1541.3	1500		

Table 38. Improved Bridge Rail Configurations for 4-in. (102-mm) Wide Lower Rails and Post Spacing of 8 ft (2.4 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
4	8	58	1	Rails	Top	HSS12X4X1/2		48.9	46.7	2101.5		SEVEN-SPAN	80.2
					Middle	HSS8X4X1/2		35.2	14.3	643.5			
					Bottom	HSS7X4X5/16		21.2	8.83	397.4			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 120.6		Σ = 3142.4	3100		
				Rails	Top	HSS12X4X1/2		48.9	46.7	2101.5		SEVEN-SPAN	82.0
					Middle	HSS8X4X1/2		35.2	14.3	643.5			
					Bottom	HSS8X4X1/2		35.2	14.3	643.5			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 134.6		Σ = 3388.5	3100		
				Rails	Top	HSS10X6X5/8		59.3	51.3	2308.5		SEVEN-SPAN	80.1
					Middle	HSS8X4X5/16		23.3	9.91	446.0			
					Bottom	HSS7X4X5/16		21.2	8.83	397.4			
				Post		W6x15	70	8.8					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 118.9		Σ = 3151.8	3100		
			1.5	Rails	Top	HSS10X4X3/8		32.6	27	1215.0		FIVE-SPAN	81.7
					Middle	HSS8X4X3/8		27.5	11.5	517.5			
					Bottom	HSS8X4X3/8		27.5	11.5	517.5			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 102.9		Σ = 2250.0	2100		
				Rails	Top	HSS10X6X3/8		37.7	33.8	1521.0		FIVE-SPAN	83.9
					Middle	HSS8X4X5/16		23.3	9.91	446.0			
					Bottom	HSS6X4X5/16		19.1	7.75	348.8			
				Post		W6x15	70	8.8					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 95.1		Σ = 2315.7	2100		
				Rails	Top	HSS12X4X5/16		31.8	31.3	1408.5		FIVE-SPAN	81.5
					Middle	HSS8X4X1/4		19.0	8.2	369.0			
					Bottom	HSS8X4X1/4		19.0	8.2	369.0			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 85.2		Σ = 2146.5	2100		

Table 39. Improved Bridge Rail Configurations for 4-in. (102-mm) Wide Lower Rails and Post Spacing of 10 ft (3.0 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
4	10	58	1	Rails	Top	HSS12X6X5/8		67.8	68.8	3096.0		SEVEN-SPAN	83.3
					Middle	HSS8X4X1/2		35.2	14.3	643.5			
					Bottom	HSS8X4X1/2		35.2	14.3	643.5			
				Post	W6x15		70	7.0					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = 150.3		Σ = 4383.0	4000		
				Rails	Top	HSS12X6X5/8		67.8	68.8	3096.0		SEVEN-SPAN	83.7
					Middle	HSS8X4X5/8		42.3	16.6	747.0			
					Bottom	HSS7X4X1/2		31.8	12.6	567.0			
				Post	W6x15		70	7.0					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = 154.0		Σ = 4410.0	4000		
				Rails	Top	HSS12X6X5/8		67.8	68.8	3096.0		SEVEN-SPAN	84.2
					Middle	HSS8X4X5/8		42.3	16.6	747.0			
					Bottom	HSS8X4X1/2		35.2	14.3	643.5			
				Post	W6x15		70	7.0					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = 157.4		Σ = 4486.5	4000		
			1.5	Rails	Top	HSS10X4X1/2		42.1	34.1	1534.5		FIVE-SPAN	80.8
					Middle	HSS8X4X1/2		35.2	14.3	643.5			
					Bottom	HSS8X4X1/2		35.2	14.3	643.5			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = 124.8		Σ = 2821.5	2600		
				Rails	Top	HSS12X4X3/8		37.7	36.7	1651.5		FIVE-SPAN	81.3
					Middle	HSS8X4X1/2		35.2	14.3	643.5			
					Bottom	HSS6X4X1/2		28.4	11	495.0			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = 113.6		Σ = 2790.0	2600		
				Rails	Top	HSS12X4X1/2		48.9	46.7	2101.5		FIVE-SPAN	83.0
					Middle	HSS8X4X1/4		19.0	8.2	369.0			
					Bottom	HSS6X4X1/4		15.6	6.45	290.3			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = 95.7		Σ = 2760.8	2600		

Table 40. Improved Bridge Rail Configurations for 5-in. (127-mm) Wide Lower Rails and Post Spacing of 6 ft (1.8 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
5	6	58	1	Rails	Top	HSS10X4X3/8		32.6	27	1215.0		SEVEN-SPAN	82.3
					Middle	HSS9X5X3/8		32.6	17.1	769.5			
					Bottom	HSS7X5X3/8		27.5	13.8	621.0			
				Post	W6x15		72.5	12.1					
				Post-to-Deck Conn.	TBD		50	8.3					
								Σ = 113.1		Σ = 2605.5	2300		
				Rails	Top	HSS10X5X3/8		35.1	30.4	1368.0		SEVEN-SPAN	82.3
					Middle	HSS9X5X5/16		27.6	14.6	657.0			
					Bottom	HSS7X5X5/16		23.3	11.9	535.5			
				Post	W6x15		71.25	11.9					
				Post-to-Deck Conn.	TBD		50	8.3					
								Σ = 106.3		Σ = 2560.5	2300		
				Rails	Top	HSS12X4X5/16		31.8	31.3	1408.5		SEVEN-SPAN	80.4
					Middle	HSS8X4X3/8		27.5	11.5	517.5			
					Bottom	HSS8X4X5/16		23.3	9.91	446.0			
				Post	W6x15		72.5	12.1					
				Post-to-Deck Conn.	TBD		50	8.3					
								Σ = 103.1		Σ = 2372.0	2300		
			1.5	Rails	Top	HSS10X4X1/4		22.4	19	855.0		FIVE-SPAN	81.5
					Middle	HSS9X5X3/16		17.1	9.25	416.3			
					Bottom	HSS7X5X3/16		14.5	7.57	340.7			
				Post	W6x15		72.5	12.1					
				Post-to-Deck Conn.	TBD		50	8.3					
								Σ = 74.4		Σ = 1611.9	1500		
				Rails	Top	HSS10X4X1/4		22.4	19	855.0		FIVE-SPAN	81.5
					Middle	HSS9X5X3/16		17.1	9.25	416.3			
					Bottom	HSS6X5X3/16		13.3	6.73	302.9			
				Post	W6x15		72.5	12.1					
				Post-to-Deck Conn.	TBD		50	8.3					
								Σ = 73.2		Σ = 1574.1	1500		
				Rails	Top	HSS10X4X1/4		22.4	19	855.0		FIVE-SPAN	84.0
					Middle	HSS7X5X1/4		19.0	9.83	442.4			
					Bottom	HSS7X5X1/4		19.0	9.83	442.4			
				Post	W6x15		72.5	12.1					
				Post-to-Deck Conn.	TBD		50	8.3					
								Σ = 80.9		Σ = 1739.7	1500		

Table 41. Improved Bridge Rail Configurations for 5-in. (127-mm) Wide Lower Rails and Post Spacing of 8 ft (2.4 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
5	8	58	1	Rails	Top	HSS10X4X5/8		50.8	40.3	1813.5		SEVEN-SPAN	81.6
					Middle	HSS9X5X1/2		42.1	21.5	967.5			
					Bottom	HSS7X5X3/8		27.5	13.8	621.0			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 135.7		Σ = 3402.0	3100		
				Rails	Top	HSS12X4X1/2		48.9	46.7	2101.5		SEVEN-SPAN	80.7
					Middle	HSS9X5X5/16		27.6	14.6	657.0			
					Bottom	HSS7X5X1/4		19.0	9.83	442.4			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 110.8		Σ = 3200.9	3100		
			1.5	Rails	Top	HSS10X4X3/8		32.6	27	1215.0		FIVE-SPAN	81.3
					Middle	HSS9X5X1/4		22.4	12	540.0			
					Bottom	HSS7X5X1/4		19.0	9.83	442.4			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 89.3		Σ = 2197.4	2100		
				Rails	Top	HSS12X4X5/16		31.8	31.3	1408.5		FIVE-SPAN	82.0
					Middle	HSS9X5X3/16		17.1	9.25	416.3			
					Bottom	HSS7X5X3/16		14.5	7.57	340.7			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 78.8		Σ = 2165.4	2100		
				Rails	Top	HSS10X5X3/8		35.1	30.4	1368.0		FIVE-SPAN	80.9
					Middle	HSS9X5X3/16		17.1	9.25	416.3			
					Bottom	HSS7X5X3/16		14.5	7.57	340.7			
				Post		W6x15	71.25	8.9					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 81.9		Σ = 2124.9	2100		

Table 42. Improved Bridge Rail Configurations for 5-in. (157-mm) Wide Lower Rails and Post Spacing of 10 ft (3.0 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
5	10	58	1	Rails	Top	HSS12X4X5/8		59.3	55.5	2497.5		SEVEN-SPAN	81.2
					Middle	HSS9X5X1/2		42.1	21.5	967.5			
					Bottom	HSS7X5X1/2		35.2	17.3	778.5			
				Post		W6x15	72.5	7.3					
				Post-to-Deck Conn.		TBD	50	5.0					
								Σ = 148.9		Σ = 4243.5	4000		
				Rails	Top	HSS12X6X1/2		55.7	57.4	2583.0		SEVEN-SPAN	81.5
					Middle	HSS9X5X1/2		42.1	21.5	967.5			
					Bottom	HSS7X5X1/2		35.2	17.3	778.5			
				Post		W6x15	70	7.0					
				Post-to-Deck Conn.		TBD	50	5.0					
								Σ = 145.0		Σ = 4329.0	4000		
			1.5	Rails	Top	HSS10X4X1/2		42.1	34.1	1534.5		FIVE-SPAN	82.2
					Middle	HSS9X5X3/8		32.6	17.1	769.5			
					Bottom	HSS7X5X3/8		27.5	13.8	621.0			
				Post		W6x15	72.5	7.3					
				Post-to-Deck Conn.		TBD	50	5.0					
								Σ = 114.4		Σ = 2925.0	2600		
				Rails	Top	HSS12X4X1/2		48.9	46.7	2101.5		FIVE-SPAN	84.2
					Middle	HSS9X5X3/16		17.1	9.25	416.3			
					Bottom	HSS7X5X3/16		14.5	7.57	340.7			
				Post		W6x15	72.5	7.3					
				Post-to-Deck Conn.		TBD	50	5.0					
								Σ = 92.7		Σ = 2858.4	2600		
				Rails	Top	HSS10X6X3/8		37.7	33.8	1521.0		FIVE-SPAN	81.4
					Middle	HSS9X5X3/8		32.6	17.1	769.5			
					Bottom	HSS7X5X3/8		27.5	13.8	621.0			
				Post		W6x15	70	7.0					
				Post-to-Deck Conn.		TBD	50	5.0					
								Σ = 109.8		Σ = 2911.5	2600		

Table 43. Improved Bridge Rail Configurations for 6-in. (152-mm) Wide Lower Rails and Post Spacing of 6 ft (1.8 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
6	6	58	1	Rails	Top	HSS10X4X5/16		27.6	23.1	1039.5		SEVEN-SPAN	80.3
					Middle	HSS8X6X5/16		27.6	16.9	760.5			
					Bottom	HSS8X6X5/16		27.6	16.9	760.5			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 103.2		Σ = 2560.5	2300		
				Rails	Top	HSS12X4X1/4		25.8	25.6	1152.0		SEVEN-SPAN	80.7
					Middle	HSS8X6X5/16		27.6	16.9	760.5			
					Bottom	HSS6X6X5/16		23.3	13.6	612.0			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 97.2		Σ = 2524.5	2300		
				Rails	Top	HSS12X4X5/16		31.8	31.3	1408.5		SEVEN-SPAN	80.2
					Middle	HSS8X6X3/16		17.1	10.7	481.5			
					Bottom	HSS8X6X3/16		17.1	10.7	481.5			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 86.4		Σ = 2371.5	2300		
			1.5	Rails	Top	HSS10X4X1/4		22.4	19	855.0		FIVE-SPAN	83.8
					Middle	HSS8X6X3/16		17.1	10.7	481.5			
					Bottom	HSS6X6X3/16		14.5	8.63	388.4			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 74.4		Σ = 1724.9	1500		
				Rails	Top	HSS12X4X3/16		19.6	19.6	882.0		FIVE-SPAN	84.5
					Middle	HSS8X6X3/16		17.1	10.7	481.5			
					Bottom	HSS6X6X3/16		14.5	8.63	388.4			
				Post		W6x15	72.5	12.1					
				Post-to-Deck Conn.		TBD	50	8.3					
								Σ = 71.7		Σ = 1751.9	1500		

Table 44. Improved Bridge Rail Configurations for 6-in. (152-mm) Wide Lower Rails and Post Spacing of 8 ft (2.4 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
6	8	58	1	Rails	Top	HSS10X6X1/2		48.9	43	1935.0		SEVEN-SPAN	83.8
					Middle	HSS8X6X3/8		32.6	19.8	891.0			
					Bottom	HSS8X6X3/8		32.6	19.8	891.0			
				Post		W6x15	70	8.8					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 129.0		Σ = 3717.0	3100		
				Rails	Top	HSS12X4X1/2		48.9	46.7	2101.5		SEVEN-SPAN	81.7
					Middle	HSS8X6X1/4		22.4	13.9	625.5			
					Bottom	HSS8X6X1/4		22.4	13.9	625.5			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 109.0		Σ = 3352.5	3100		
				Rails	Top	HSS10X6X1/2		48.9	43	1935.0		SEVEN-SPAN	82.2
					Middle	HSS8X6X3/8		32.6	19.8	891.0			
					Bottom	HSS6X6X3/8		27.5	15.8	711.0			
				Post		W6x15	70	8.8					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 123.9		Σ = 3537.0	3100		
			1.5	Rails	Top	HSS10X4X3/8		32.6	27	1215.0		FIVE-SPAN	83.0
					Middle	HSS8X6X1/4		22.4	13.9	625.5			
					Bottom	HSS6X6X1/4		19.0	11.2	504.0			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 89.3		Σ = 2344.5	2100		
				Rails	Top	HSS12X4X5/16		31.8	31.3	1408.5		FIVE-SPAN	82.1
					Middle	HSS8X6X3/16		17.1	10.7	481.5			
					Bottom	HSS6X6X1/8		9.9	5.92	266.4			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 74.1		Σ = 2156.4	2100		
				Rails	Top	HSS12X4X5/16		31.8	31.3	1408.5		FIVE-SPAN	80.6
					Middle	HSS8X4X1/4		19.0	8.2	369.0			
					Bottom	HSS6X4X1/4		15.6	6.45	290.3			
				Post		W6x15	72.5	9.1					
				Post-to-Deck Conn.		TBD	50	6.3					
								Σ = 81.8		Σ = 2067.8	2100		

Table 45. Improved Bridge Rail Configurations for 6-in. (152-mm) Wide Lower Rails and Post Spacing of 10 ft (3.0 m)

Spacer Length (in.)	Post Spacing (ft)	Post Length (in.)	Dynamic Magnification Factor	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft.)	Z (in.^{3})	Mp (kip-in.)	Mp required (kip-in.)	Span-Mechanism	Strength Capacity at Load Height (kips)
6	10	58	1	Rails	Top	HSS10X6X5/8		59.3	51.3	2308.5		SEVEN-SPAN	81.9
					Middle	HSS8X6X1/2		42.1	24.9	1120.5			
					Bottom	HSS8X6X1/2		42.1	24.9	1120.5			
				Post	W6x15		70	7.0					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = *155.4*		Σ = *4549.5*	4000		
				Rails	Top	HSS12X4X5/8		59.3	55.5	2497.5		SEVEN-SPAN	82.9
					Middle	HSS8X6X1/2		42.1	24.9	1120.5			
					Bottom	HSS6X6X1/2		35.2	19.8	891.0			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = *148.9*		Σ = *4509.0*	4000		
				Rails	Top	HSS12X4X5/8		59.3	55.5	2497.5		SEVEN-SPAN	81.1
					Middle	HSS8X6X3/8		32.6	19.8	891.0			
					Bottom	HSS8X6X3/8		32.6	19.8	891.0			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = *136.7*		Σ = *4279.5*	4000		
			1.5	Rails	Top	HSS10X4X1/2		42.1	34.1	1534.5		FIVE-SPAN	80.4
					Middle	HSS8X6X1/4		22.4	13.9	625.5			
					Bottom	HSS8X6X1/4		22.4	13.9	625.5			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = *99.1*		Σ = *2785.5*	2600		
				Rails	Top	HSS12X4X3/8		37.7	36.7	1651.5		FIVE-SPAN	81.0
					Middle	HSS8X6X1/4		22.4	13.9	625.5			
					Bottom	HSS6X6X1/4		19.0	11.2	504.0			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = *91.4*		Σ = *2781.0*	2600		
				Rails	Top	HSS12X4X3/8		37.7	36.7	1651.5		FIVE-SPAN	82.2
					Middle	HSS8X6X1/4		22.4	13.9	625.5			
					Bottom	HSS8X6X1/4		22.4	13.9	625.5			
				Post	W6x15		72.5	7.3					
				Post-to-Deck Conn.	TBD		50	5.0					
								Σ = *94.8*		Σ = *2902.5*	2600		

Table 46. Minimum Weight per Foot for Improved Bridge Rail Configurations

Post Offset (in.)	Post Spacing (ft)	DMF	Weight per Foot (lb/ft)	Post Offset (in.)	Post Spacing (ft)	DMF	Weight per Foot (lb/ft)	Post Offset (in.)	Post Spacing (ft)	DMF	Weight per Foot (lb/ft)
4	6	1	102.2	5	6	1	103.1	6	6	1	86.4
		1.5	74.7			1.5	73.2			1.5	71.7
	8	1	118.9		8	1	110.8		8	1	109.0
		1.5	85.2			1.5	78.8			1.5	74.1
	10	1	150.3		10	1	145.0		10	1	136.7
		1.5	95.7			1.5	92.7			1.5	91.4

After comparing the weight per foot for the lightest bridge rail configurations, it was observed that using a DMF equal to 1.5 versus 1.0 reduced the overall weight per foot of the systems by approximately 20% to 30%. Moreover, an increase in lateral post offset resulted in decreased the weight per foot of the bridge rail system. Furthermore, an increased post spacing resulted in increased weight per foot of the system. However, it should be noted that a reduced post spacing requires more posts and post-to-deck connections, thus likely resulting in a longer and more labor-intensive installation process. It should also be noted that the post-to-deck connection hardware would likely increase in weight for greater lateral post offsets. However, the same post-to-deck connection hardware and weight were used for these calculations. It was noticeable that thinner lower rails were needed for 6-in. (152-mm) wide lower rails configurations, thus, lower system weight per foot could be expected. Five-span collapse mechanisms occurred when the DMF was equal to 1.5, and seven-span collapse mechanisms occurred when the DMF was equal to 1.0 in all the cases.

5.7 Modified Bridge Rail Configurations Considering Post-to-Rail Connection Holes

After design variables were established for generating final bridge rail configurations, it was necessary to decrease the plastic section moduli of the horizontal rails due to inclusion of post-to-rail connection bolt holes. The general configurations for the post-to-rail connections were initially based on the IL/OH MASH 2016 TL-4 Prototype Bridge Rail, which consisted of a pair of horizontal round bolts for each of the two lower rails and one pair of vertical round bolts for the top rail, as depicted in Figure 52. The round bolt holes in the vertical and horizontal faces of the top rail and lower rails, respectively, reduced the cross-sectional areas and plastic section moduli of the rails. These reductions were calculated and subtracted from tabulated data published for the three rails within the AISC *Steel Construction Manual* [49]. The post properties were not affected at this time as the system was expected to utilize a connection detail that would not weaken the support post.

The axes of bending for the three rails are shown in Figure 52. To solve for the reduction in plastic section moduli for holes in the top, middle, and bottom rails, the plastic section moduli for the pair of rectangular cross-sections were calculated using Equations 19 through 21, which and were obtained from the AISC Steel Construction Manual [49], specifically AISC Table 17-27. The sample calculations for the rail reductions of plastic section moduli are shown below for the initial IL/OH Prototype Bridge Rail and its three rail sections. Note that this procedure was replicated for modifying plastic section moduli for other rail combinations used in the design process.

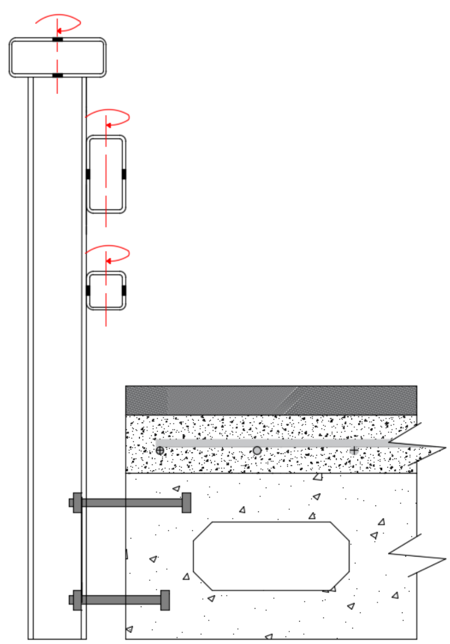


Figure 52. Axis of Bending of Rails Under Lateral Loading

The reduction of plastic section modulus for the top rail section for a single bolt hole with axis of bending through center was calculated using Equation 19. The reduction of plastic section modulus must be multiplied by two to capture both holes for the top rail, which is shown in Figure 53.

$$Z_{TOP\ RAIL\ HOLE} = \frac{b * d^2}{4} \tag{19}$$

$$Z_{TOP\ RAIL\ 2\ HOLES} = (2)\frac{b * d^2}{4} = \frac{b * d^2}{2} \tag{20}$$

where:

$Z_{\text{TOP RAIL REDUCTION}}$ = Reduction of plastic section modulus of top rail (in.^{3})
b = thickness of one hole (in.); and
d = width of one hole (in.).

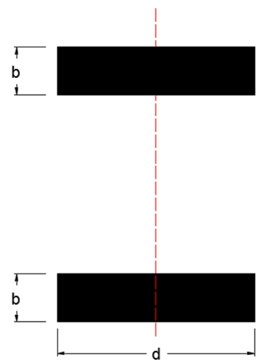


Figure 53. Plastic Section Modulus Schematic for Two Holes Bending About Vertical Axis in Top Rail

The reduction of plastic section modulus for the middle and bottom rail sections due to a pair of bolt holes with axis of bending through center of gravity, as depicted in Figure 54, was solved using Equation 21:

$$Z_{LOWER\ RAIL\ HOLES} = \frac{b}{4}(d^2 - d_1^2) \tag{21}$$

where:

$Z_{\text{LOWER RAIL REDUCTION}}$ = Reduction of plastic section modulus of lower rails (in.^{3})
b = width of holes (in.);
d = outside distance between holes (in.); and
d_1 = inside distance between holes (in.).

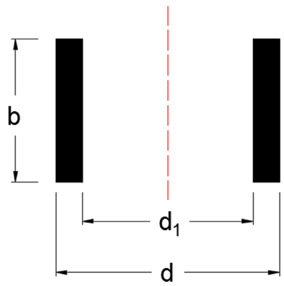


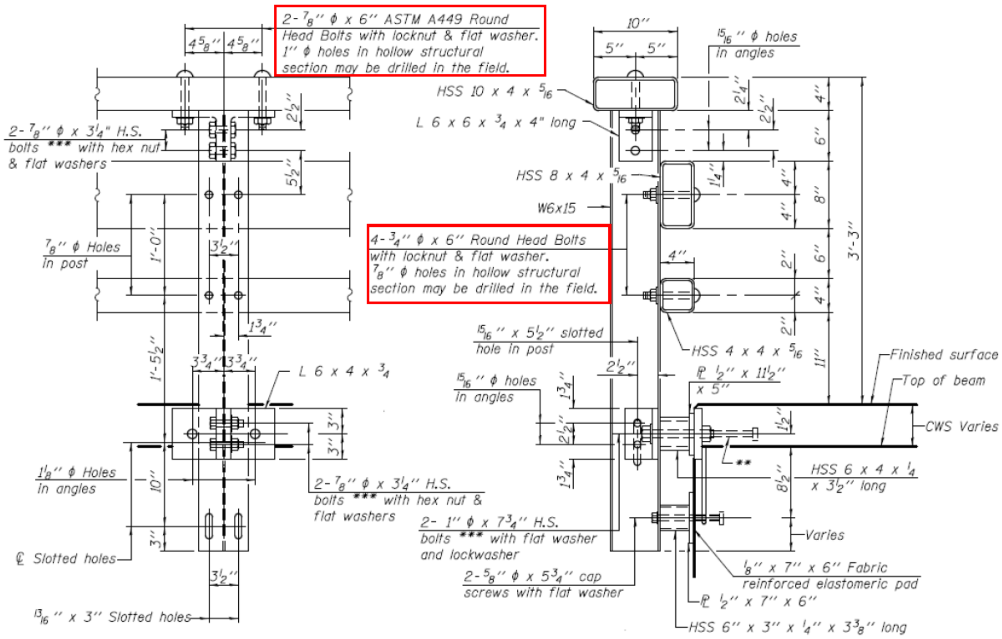
Figure 54. Plastic Section Modulus Schematic for Two Holes Bending About Vertical Axis in Lower Two Rails

The reduced plastic section modulus for each rail section was modified by subtracting the plastic section moduli of the bolt holes away from the tabulated plastic section modulus, as denoted in Equation 22.

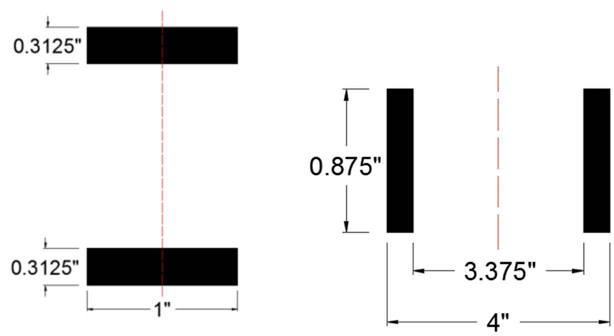
$$Z_{REDUCED} = Z_{TABULATED} - Z_{HOLES} \tag{22}$$

5.7.1 Example Problem No. 4 – Calculate Modified Plastic Section Modulus for Three Rails in IL/OH Prototype Bridge Rail

The calculations for the reduced plastic section moduli for the three rails in the IL/OH Prototype Bridge Rail are provided below. The configuration utilized a pair of ⅞-in. (22-mm) diameter bolts for the top rail and a pair ¾-in. (19-mm) diameter bolts for the lower two rails. The top rail used 1-in. (25-mm) diameter bolt holes, and ⅞-in. (22-mm) diameter bolt holes were used for the lower two rails.



(a)



(b)

Figure 55. IL/OH Prototype Bridge Rail (a) CAD Details and (b) Dimension for Each Pair of Holes

Step 1 – Calculate top rail reduction of plastic section modulus, $Z_{X\ RED\ TOP\ RAIL}$:

$$Z_{X\ RED\ TOP\ RAIL} = (2)\,\frac{b * d^2}{4} = \frac{0.3125\text{ in.} * (1\text{ in.})^2}{2} = 0.16\text{ in.}^3$$

Step 2 – Calculate middle rail reduction of plastic section modulus, $Z_{Y\ RED\ MIDDLE\ RAILS}$:

$$Z_{Y\ RED\ MIDDLE\ RAIL} = \frac{b}{4}(d^2 - d_1^2) = \frac{0.875\text{ in.}}{4}\ (4\text{ in.}^2 - 3.375\text{ in.}^2) = 1.01\text{ in.}^3$$

Step 3 – Calculate middle rail reduction of plastic section modulus, $Z_{Y\ RED\ MIDDLE\ RAILS}$:

$$Z_{Y\ RED\ BOTTOM\ RAIL} = \frac{b}{4}(d^2 - d_1^2) = \frac{0.875\text{ in.}}{4}\ (4\text{ in.}^2 - 3.375\text{ in.}^2) = 1.01\text{ in.}^3$$

<u>Step 4 – Calculate reduced plastic section modulus for three rail sections, $Z_{REDUCED}$:</u>

$$Z_{REDUCED} = Z_{TABULATED} - Z_{HOLES}$$

$$Z_{REDUCED\ TOP\ RAIL} = 23.10\ in.^3 - 0.16\ \text{in.}^3 = 22.94\ in.^3$$

$$Z_{REDUCED\ MIDDLE\ RAIL} = 9.91\ in.^3 - 1.01\ \text{in.}^3 = 8.90\ in.^3$$

$$Z_{REDUCED\ TOP\ RAIL} = 5.59\ in.^3 - 1.01\ \text{in.}^3 = 4.58\ in.^3$$

5.7.2 Preliminary Plastic Section Moduli Reduction for Final Bridge Rail Configuration

The reduced plastic section moduli of the three rails were calculated in order to design final bridge rail configurations. The initial configurations for the post-to-rail connections were based on the connections used in the IL/OH Prototype Bridge Rail. The configuration utilized a pair of ⅞-in. (22-mm) diameter bolts for the top rail and a pair ¾-in. (19-mm) diameter bolts for the lower two rails. The top rail used 1-in. (25-mm) diameter bolt holes, and ⅞-in. (22-mm) diameter bolt holes were used for the lower two rails. The tabulated plastic section moduli for the top and lower rail sections due to inclusion of bolt holes are shown in Tables 47 and 50, respectively. The reduction of plastic section modulus of the top and lower rail sections are shown in Tables 48 and 51, respectively. Lastly, the reduced plastic section moduli for the top and lower rail sections are shown in Tables 49 and 52, respectively.

Table 47. AISC Tabulated Section Modulus for Top Rail [49]

6-in. Rail Depth		4-in. Rail Depth	
HSS Shape	Z_X (in.^{3})	HSS Shape	Z_X (in.^{3})
12x6x⅝	68.8	12x4x⅝	55.5
12x6x½	57.4	12x4x½	46.7
12x6x⅜	44.8	12x4x⅜	36.7
12x6x5/16	38.1	12x4x5/16	31.3
12x6x¼	31.1	12x4x¼	25.6
10x6x⅝	51.3	10x4x⅝	40.3
10x6x½	43.0	10x4x½	34.1
10x6x⅜	33.8	10x4x⅜	27.0
10x6x5/16	28.8	10x4x5/16	23.1
10x6x¼	23.6	10x4x¼	19.0

Table 48. Reduction of Plastic Section Modulus Holes for Top Rail

6-in. Post Offset		4-in. Post Offset	
HSS Shape	Z_X (in.^{3})	HSS Shape	Z_X (in.^{3})
12x6x⅝	0.31	12x4x⅝	0.31
12x6x½	0.25	12x4x½	0.25
12x6x⅜	0.19	12x4x⅜	0.19
12x6x5/16	0.16	12x4x5/16	0.16
12x6x¼	0.13	12x4x¼	0.13
10x6x⅝	0.31	10x4x⅝	0.31
10x6x½	0.25	10x4x½	0.25
10x6x⅜	0.19	10x4x⅜	0.19
10x6x5/16	0.16	10x4x5/16	0.16
10x6x¼	0.13	10x4x¼	0.13

Table 49. Reduced Plastic Section Modulus for Top Rail

6-in. Post Offset		4-in. Post Offset	
HSS Shape	Z_X (in.^{3})	HSS Shape	Z_X (in.^{3})
12x6x⅝	68.49	12x4x⅝	55.19
12x6x½	57.15	12x4x½	46.45
12x6x⅜	44.61	12x4x⅜	36.51
12x6x5/16	37.94	12x4x5/16	31.14
12x6x¼	30.97	12x4x¼	25.47
10x6x⅝	50.99	10x4x⅝	39.99
10x6x½	42.75	10x4x½	33.85
10x6x⅜	33.61	10x4x⅜	26.81
10x6x5/16	28.64	10x4x5/16	22.94
10x6x¼	23.47	10x4x¼	18.87

Table 50. AISC Tabulated Plastic Section Modulus for Lower Rails [49]

6-in. Post Offset		4-in. Post Offset	
HSS Shape	Z_Y (in.^{3})	HSS Shape	Z_Y (in.^{3})
8x6x⅝	29.5	8x4x⅝	16.6
8x6x½	24.9	8x4x½	14.3
8x6x⅜	19.8	8x4x⅜	11.5
8x6x5/16	16.9	8x4x5/16	9.91
8x6x¼	13.9	8x4x¼	8.2

Table 51. Reduction of Plastic Section Modulus Holes for Lower Rails

6-in. Post Offset		4-in. Post Offset	
HSS Shape	Z_Y (in.^{3})	HSS Shape	Z_Y (in.^{3})
8x6x⅝	3.36	8x4x⅝	2.11
8x6x½	2.75	8x4x½	1.75
8x6x⅜	2.11	8x4x⅜	1.36
8x6x$^5/_{16}$	1.78	8x4x$^5/_{16}$	1.15
8x6x¼	1.44	8x4x¼	0.94

Table 52. Reduced Plastic Section Modulus for Lower Rails

6-in. Post Offset		4-in. Post Offset	
HSS Shape	Z_Y (in.^{3})	HSS Shape	Z_Y (in.^{3})
8x6x⅝	26.14	8x4x⅝	14.49
8x6x½	22.15	8x4x½	12.55
8x6x⅜	17.69	8x4x⅜	10.14
8x6x$^5/_{16}$	15.12	8x4x$^5/_{16}$	8.76
8x6x¼	12.46	8x4x¼	7.26

5.8 Single-Span Check for 2270P Pickup Trucks for Lower Two Rails

The 70-kip (311-kN) lateral design impact load for the pickup truck utilizes a height of 24 in. (610 mm) and is distributed over 4 ft (1.2 m) [37]. This condition, as well as the 1-in. (25-mm) top rail setback, led researchers to consider only the two lower rails for containing and redirecting the pickup truck under impact events. Therefore, an analysis of the lower two rails was performed to analyze the horizontal bending capacity of the bridge rail for the pickup truck within a single span prior to post yielding and without loading the top rail. It was determined that the contribution of the two lower rails to resist bending forces was equally distributed to simplify this additional investigation.

Previously, AASHTO's *Standard Specifications for Highway Bridges* [52] recommended that bridge railing members be designed to resist a moment under concentrated loads at the center of a single span of PL/6. The intention of using PL/6 was to consider the average of maximum moments under concentrated loads of a simply-supported beam as well as a fixed-end beam, resulting in a maximum moment equal to PL/4 and PL/8, respectively.

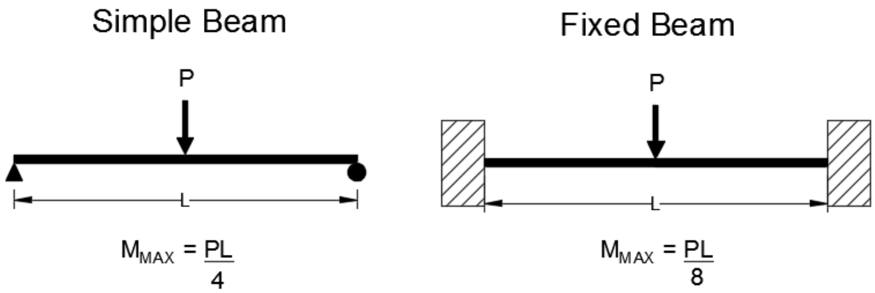


Figure 56. Maximum Moments of Simply-Supported Beam and Fixed-End Beam with a Concentrated Load at Midspan

Since the design impact load for the pickup truck actually uses a distributed length equal to 4 ft (1.2 m), the single-span check was intended to consider a uniform partially-distributed load at the midspan location and applied over the lower two rails. Based on AASHTO's *Standard Specifications for Highway Bridges* [52], the maximum moment of the span for the single-span check was determined to be the average of simply-supported beam and a fixed-end beam maximum moments under a uniform, partially-distributed load at midspan, as shown in Figure 57. The maximum moment for a simply-supported beam with uniform, partially-distributed load is shown in Equation 23 [49], and the maximum moment of a fixed-end beam with a partially-distributed load at midspan is shown in Equation 24 [50].

$$M_{\mathrm{MAX\ SIMPLE}} = R_1\left(a - \frac{L_T}{2} + \frac{R_1}{2W}\right) \tag{23}$$

where:

$M_{\mathrm{MAX\ SIMPLE}}$ = maximum moment for a simply-supported beam with a uniform, partially-distributed load at midspan (kip-in.);
R_1 = vertical shear reaction at each support (kip);
R_2 = vertical shear reaction at each support (kip);
W = distributed load (kip/ft);
L_T = length of distributed lateral load (ft);
a = center of distributed load to the left (ft); and
b = center of distributed load to the right (ft).

$$M_{\mathrm{MAX\ FIXED}} = \frac{WL_T}{L^2}\left(ab^2 + \frac{(a-2b){L_T}^2}{12}\right) \tag{24}$$

where:

$M_{\mathrm{MAX\ FIXED}}$ = maximum moment for a fixed-end beam with uniform, partially-distributed load at midspan (kip-in.)
R_1 = vertical shear reaction at each support (kip);
R_2 = vertical shear reaction at each support (kip);
W = distributed load (kip/ft);
L_T = length of distributed load (ft);
a = center of distributed load to the left (ft); and
b = center of distributed load to the right (ft).

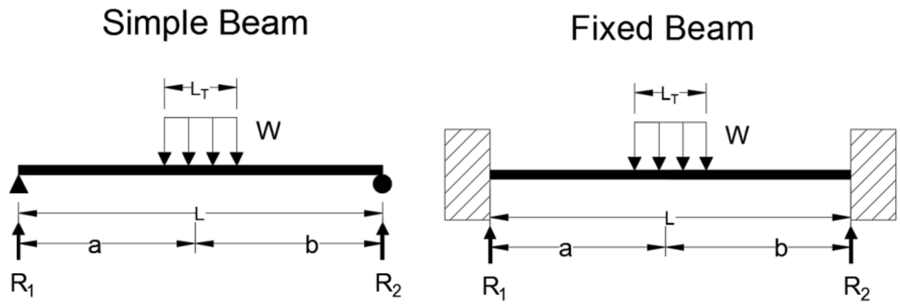


Figure 57. Maximum Moments for a Simply-Supported Beam and Fixed-End Beam with Uniform Partially-Distributed Loads at Midspan

For the single-span check and similar to AASHTO's approach for point loading [52], the maximum moments for simply-supported and fixed-end beams with partially-distributed loads at midspan were calculated to determine the average maximum moment. This average maximum moment was to be resisted by the lower two rail sections with post spacings of 6 ft (1.8 m), 8 ft (2.4 m), and 10 ft (3.0 m). The maximum moment equations were solved using a lateral design load for the pickup truck of 70 kips (311 kN) at a height of 24 in. (610 mm) and distributed over 4 ft (1.2 m).

Table 53. Average Maximum Moments for 6 ft (1.8 m), 8 ft (2.4 m), and 10 ft (3.0 m) Post Spacings

Post Spacing (ft)	Simply-Supported Beam Maximum Moment, M_{MAX} (kip-in.)	Fixed-End Beam Maximum Moment, M_{MAX} (kip-in.)	Average Maximum Moment, M_{MAX} (kip-in.)
6	840.0	536.7	688.4
8	1260.0	770.0	1015.0
10	1680.0	994.0	1337.0

The plastic moment capacity of a rail was previously defined in Equation 4. Based on Equation 4, the required plastic section moduli for the middle and bottom rails was found using Equation 24. Note that the distributed loading was to be resisted equally by the two lower rails. The required plastic section moduli of the middle and bottom rails for 6 ft (1.8 m), 8 ft (2.4 m), and 10 ft (3.0 m) post spacing were 15.3 in.^{3} (250,722 mm^3), 22.6 in.^{3} (370,347 mm^3), and 29.7 in.^{3} (486,696 mm^3), respectively, as shown in Table 54.

$$M_{P\ RAILS} = \text{Ø}\, F_Y\, Z \tag{4}$$

$$Z_{\ REQUIRED} = \frac{Average\ M_{MAX}}{\text{Ø}F_Y} \tag{25}$$

where:

$Z_{REQUIRED}$ = required plastic section modulus for single-span check (in.^{3});
Average M_{MAX}= average maximum moment applied to both lower rails (kip-in.);
Ø = reduction factor, 0.9; and
F_Y = minimum specified yield strength, ksi.

Table 54. Required Plastic Section Modulus of Middle and Bottom Rails for Single-Span Check

Post Spacing (ft)	Average Maximum Moment, M_{MAX} (kip-in.)	Required Plastic Section Modulus for Middle and Bottom Rails, Z_{REQ} (in.^{3})
6	688.4	15.3
8	1015.0	22.6
10	1337.0	29.7

The plastic section moduli for the lower rail sections shown in Table 52 that did not satisfy the required plastic section moduli for the lower two rails, $Z_{REQUIRED}$, were not considered for the new bridge rail. The eliminated lower rail cross-sections with and without the reduction of plastic section moduli were identified, as shown in Tables 55 and 56.

Table 55. Lower Rail Sections Eliminated by Single-Span Check without Plastic Section Modulus Reduction for Holes

Post Offset	Post Spacing		
	6 ft	8 ft	10 ft
4 in.	N/A	HSS 8x4x5/16 HSS 8x4x¼	HSS 8x4x½ HSS 8x4x⅜ HSS 8x4x5/16 HSS 8x4x¼
6 in.	N/A	N/A	HSS 8x6x¼

Table 56. Lower Rail Sections Eliminated by Single-Span Check Using Plastic Section Modulus Reduction for Holes

Post Offset	Post Spacing		
	6 ft	8 ft	10 ft
4 in.	HSS 8x4x¼	HSS 8x4x⅜ HSS 8x4x5/16 HSS 8x4x¼	HSS 8x4x⅝ HSS 8x4x½ HSS 8x4x⅜ HSS 8x4x5/16 HSS 8x4x¼
6 in.	N/A	N/A	HSS 8x6x¼

5.9 Other Design Considerations for Final Bridge Rail

For the installation of the bridge rail, the steel rails would likely be the heaviest system components. Depending on rail length and post spacing, the steel rails could be heavy and difficult for workers to carry and install without the use of large machinery. Thus, personnel from the Illinois and Ohio DOTs established a maximum weight for each steel rail segment equal to 500 lb (227 kg) in order to not require large machinery on the bridge deck during bridge rail installation, which could pose risks to the structural integrity of the bridge deck. In order to maintain a maximum rail segment weight of 500 lb (227 kg), each rail element was limited to two increments in the post spacing.

Later in the research process and after consulting with representatives from the Illinois and Ohio DOTs, a minimum rail thickness of ¼ in. (6.4 mm) was also specified for the three steel rails to prevent crushing of thinner wall sections that could accentuate large plastic deformations lead to excessive vehicle instabilities and rollover.

Representatives from the Illinois and Ohio DOTs preferred the lower two rails to use equal thickness and size and to disregard steel rails with odd dimensions (i.e., 7 in. height, 5 in. width) in order to simplify installation and improve material availability. Therefore, the bottom and middle rails were limited to a height of 6 in. (152 mm) and 8 in. (203 mm) and widths of 4 in. (102 mm) and 6 in. (152 mm).

However, the use of two identical lower rails with a height of 6 in. (152 mm) was disregarded due to a vertical spacing of 6 in. (152 mm) between the bottom and middle rails, falling short of the minimum lower rail height of 29 in. (737 mm) for the middle rail when a 3-in. (76-mm) thick asphalt overlay is applied, and a large 5-in. (127-mm) vertical opening between the middle and top rails, as shown in Figure 58. Note that a vertical spacing or opening between the lower two rails of 6 in. (152 mm) versus 4 in. (102 mm) could lead to an increased potential for the structural component of the small car to wedge between and/or snag on the vertical posts. The vertical height of the bumpers structural components was 3⅞ in. (98 mm) deep.

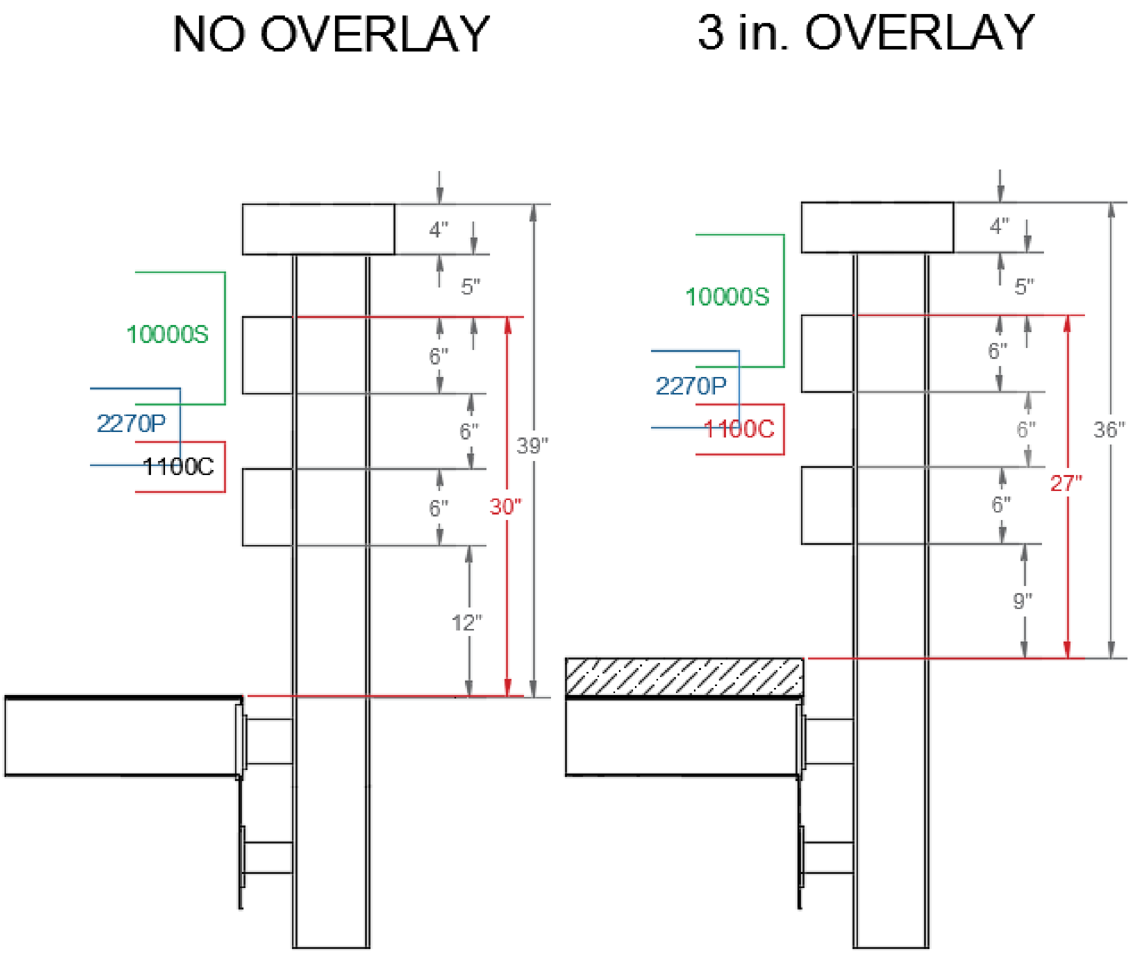


Figure 58. Prototype Bridge Rail Geometry with 6-in. (152-mm) Deep Lower Rails

In order to satisfy the design criteria for the new bridge rail, the bottom and middle rails were limited to a height of 8 in. (203 mm) using widths of 4 in. (102 mm) or 6 in. (152 mm), while the top rail was limited to widths of 10 in. (254 mm) or 12 in. (305 mm) using heights of 4 in. (102 mm) or 6 in. (152 mm), as shown in Figure 59. Again, minimum rail thickness equal to ¼ in. (6.4 mm) was also specified for the three steel rails.

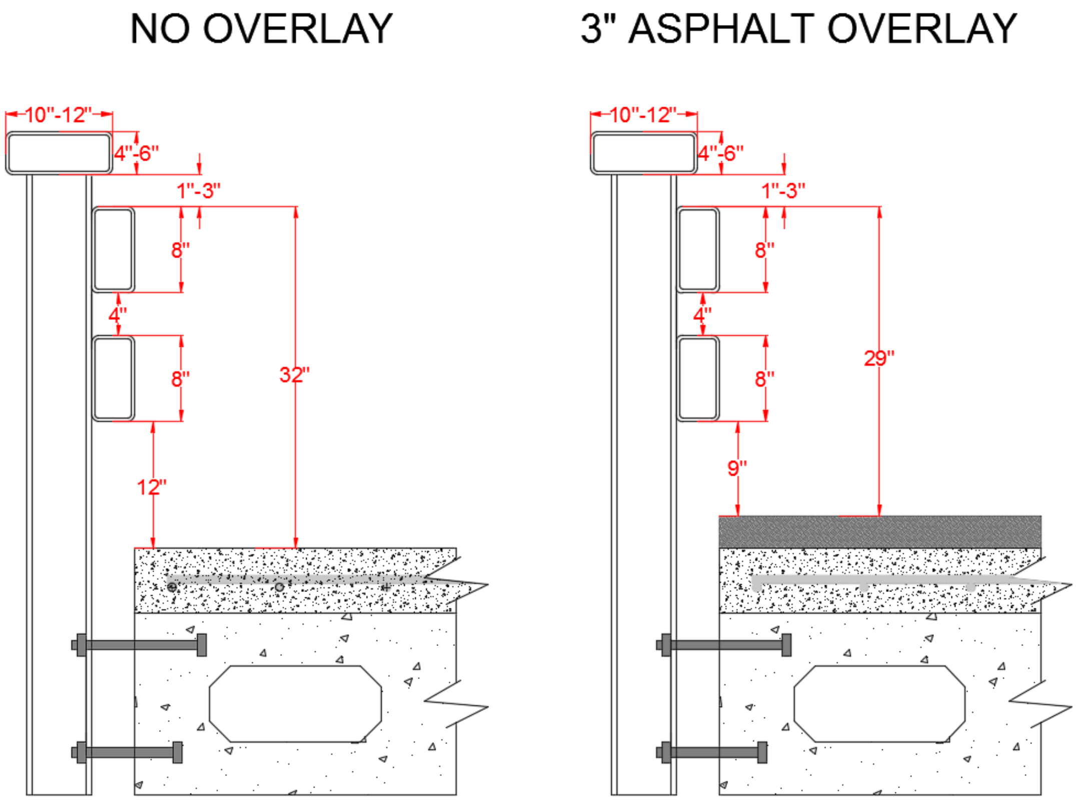


Figure 59. Design Criteria Summary

5.10 Vertical Bending Capacity and Deflection – Top Rail

The vertical design impact load for the SUT is represented as a 38-kip (169-kN) distributed load over 18 ft (5.5 m), which occurs as a downward load applied by the roll motion of the SUT. An analysis effort was performed to determine if the top rail would remain elastic as well as calculate its maximum deflection under vertical design loading. For the prototype bridge rail, the post spacings were 6 ft (1.8 m), 8 ft (2.4 m), and 10 ft (3.0 m). Thus, a conservative simply-supported beam with a length equal to 10 ft (3.0 m) were considered to be subjected to a design downward loading using the weakest selected HSS shape for the top rail, specifically a HSS 10-in. x 4-in. x ¼-in. (HSS 254-mm x 101.6-mm x 6.4-mm). The simplified beam is shown in Figure 60. The beam analysis was performed using Equations 26 and 27.

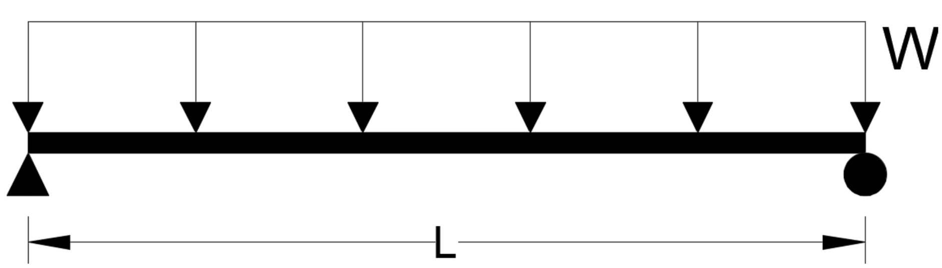


Figure 60. Simply-Supported Beam with Uniformly Distributed Load

$$\mathrm{M_{MAX}} = \frac{\mathrm{WL}^2}{8} \tag{26}$$

where:

M_{MAX} = maximum moment of simply-supported beam with uniform load (kip-in.);
W= distributed load (kip/ft); and
L= length of the beam (ft).

$$\mathrm{M_{MAX}} = \frac{\left(\frac{38\ \mathrm{kips}}{18\ \mathrm{ft}}\right)(10\ \mathrm{ft})^2}{8} = 26.39\ \mathrm{k-ft} = 317\ \mathrm{k-in.}$$

$$M_{P\ TOP\ RAIL} = \emptyset * F_Y * S_Y \tag{27}$$

where:

$M_{P\ TOP\ RAIL}$ = maximum moment for simply-supported beam with uniform load (kip-in.);
Ø = 0.9;
F_Y = yield strength of top rail, (ksi); and
S_Y = elastic section modulus for weak axis (in.^{3}).

M_P= (0.9) (50 ksi) (8.87 in.^{3}) = 399 k-in. $\quad$ $\mathrm{M_{P\ TOP\ RAIL}} > \mathrm{M_{MAX}}$

$$\Delta_{\mathrm{MAX}} = \frac{5\mathrm{wl}^4}{384\mathrm{EI}} \tag{28}$$

where:

Δ_{MAX} = maximum deflection of simple beam at midspan (in.)
W= distributed load (kip/in.)
L= length of the beam (in.)
E= modulus of elasticity of top rail (ksi)
I = moment of inertia (in.^{4})

$$\Delta_{\mathrm{MAX}} = \frac{5\left(\frac{38\ \mathrm{kips}}{216\ \mathrm{in.}}\right)(120\ \mathrm{in.})^4}{384(29000\ \mathrm{ksi})(17.7\ \mathrm{in.}^4)}$$

$$\Delta_{\mathrm{MAX}} = 0.92\ \mathrm{in.}$$

The analysis showed that the top rail would remain elastic under vertical loading for all preferred top rail options. The elastic moment was found to be 399 kip-in. (45 kN-m), while the design moment was 317 kip-in. (36 kN-m). The vertical deflection for the HSS 10-in. x 4-in. x ¼-in. (HSS 254-mm x 101.6-mm x 6.4-mm) top rail with 10 ft (3.0 m) post spacing, and simply-supported beam, was determined using Equation 28 [49]. The maximum midspan deflection for a simply-supported beam was found to be 0.92 in. (24-mm) using Equation 28.

5.11 Final Bridge Rail Configurations

A maximum rail segment weight of 500 lb (227 kg), a reduction in preliminary plastic section modulus for the three rails, results from single-span check, and other considerations defined throughout the design process, were used to configure the final bridge rail. Final bridge rail configurations are shown in Tables 57 and 58. Bridge rail configurations shown with an asterisk represent systems with lower rail sections that violated single-span check discussed previously. Moreover, the last column of both tables contain the weight of the top rail segment over two spans. Bridge rail configurations with red shading in the last column depict a system with a top rail segment heavier than 500 lb (227 kg).

As depicted in Table 57 and with green shading, bridge rail configurations with a 4-in. (102-mm) lateral offset only met the weight limitations for the top rail when using a 6-ft (1.8-m) post spacing at both DMFs (1.0 and 1.5). For systems using 8-ft (2.4-m) and 10-ft (3.0-m) post spacings, the top and/or bottom rails exceeded 500 lb (227 kg), as depicted with red shading. Only the two systems depicted in green shading were moved forward for consideration as a refined bridge rail option.

As depicted in Table 58 and with green shading, bridge rail configurations with a 6-in. (152-mm) lateral offset only met the weight for the top rail when using a 6-ft (1.8 m) post spacing at both DMFs (1.0 and 1.5) and a 8-ft (2.4-m) post spacing at a DMF equal to 1.5. For systems using 8-ft (2.4-m) post spacing at a DMF equal to 1.0 and 10-ft (3.0-m) post spacing, the top and/or bottom rails exceeded 500 lb (227 kg), as depicted with red shading. The three systems depicted in green shading were moved forward for consideration as a refined bridge rail option.

The remaining acceptable bridge rail configurations included four options with a 6-ft (1.8 m) post spacing, and one option with a post offset equal to 6 in. (152 mm), a post spacing of 8 ft (2.4 m), and a DMF equal to 1.5.

Table 57. Final Bridge Rail Configurations with Post Offset Equal to 4 in. (102 mm)

Post Offset (in.)	Post Spacing (ft)	Dynamic Magnification Factor (Post-Only)	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft)	Plastic Section Modulus, (in.^{3})	Critical Span-Mechanism	Lateral Barrier Resistance at Load Height (kips)	Weight of Top Rail Over Two Spans (lb)
4	6	1	Rails	Top	HSS12X4X3/8		37.7	36.51	SEVEN SPAN	82.1	452.3
				Middle	HSS8X4X5/16		23.3	8.76			
				Bottom	HSS8X4X5/16		23.3	8.76			
			Post	W6x15		72.5	12.1				
			Post-Deck Conn.			87.5	14.6				
							Σ = ***111.0***				
		1.5	Rails	Top	HSS10X4X1/4		22.4	18.87	FIVE SPAN	82.1	269.0
				Middle	HSS8X4X5/16		23.3	8.76			
				Bottom	HSS8X4X5/16		23.3	8.76			
			Post	W6x15		72.5	12.1				
			Post-Deck Conn.			87.5	14.6				
							Σ = ***95.8***				
	8	1*	Rails	Top	HSS12X4X1/2		48.9	46.45	SEVEN SPAN	80.4	781.6
				Middle	HSS8X4X1/2		35.2	12.55			
				Bottom	HSS8X4X1/2		35.2	12.55			
			Post	W6x15		72.5	9.1				
			Post-Deck Conn.			87.5	10.9				
							Σ = ***139.3***				
		1.5*	Rails	Top	HSS10X4X3/8		32.6	26.81	FIVE SPAN	82.7	521.3
				Middle	HSS8X4X1/2		35.2	12.55			
				Bottom	HSS8X4X1/2		35.2	12.55			
			Post	W6x15		72.5	9.1				
			Post-Deck Conn.			87.5	10.9				
							Σ = ***123.1***				
	10	1*	Rails	Top	HSS12X6X5/8		67.8	68.49	SEVEN SPAN	83.3	1356.4
				Middle	HSS8X4X5/8		42.3	14.49			
				Bottom	HSS8X4X5/8		42.3	14.49			
			Post	W6x15		70	11.7				
			Post-Deck Conn.			87.5	14.6				
							Σ = ***178.7***				
		1.5*	Rails	Top	HSS12X4X3/8		37.7	36.51	FIVE SPAN	82.6	753.8
				Middle	HSS8X4X5/8		42.3	14.49			
				Bottom	HSS8X4X5/8		42.3	14.49			
			Post	W6x15		72.5	12.1				
			Post-Deck Conn.			87.5	14.6				
							Σ = ***149.0***				

* - Bridge rail configurations with lower rails violating single-span check.

Table 58. Final Bridge Rail Configurations with Post Offset Equal to 6 in. (152 mm)

Post Offset (in.)	Post Spacing (ft)	Dynamic Magnification Factor (Post-Only)	Bridge Rail Hardware Category	Section		Section Weight (lb)	Weight per foot (lb/ft)	Plastic Section Modulus, (in.^{3})	Critical Span-Mechanism	Lateral Barrier Resistance at Load Height (kips)	Weight of Top Rail Over Two Spans (lb)
6	6	1	Rails	Top	HSS10X6X3/8		37.7	33.61	SEVEN SPAN	83.5	452.3
				Middle	HSS8X6X1/4		22.4	12.46			
				Bottom	HSS8X6X1/4		22.4	12.46			
			Post	W6x15		72.5	12.1				
			Post-Deck Conn.			87.5	14.6				
							Σ = ***109.2***				
		1.5	Rails	Top	HSS10X4X1/4		22.4	18.87	FIVE SPAN	88.1	269.0
				Middle	HSS8X6X1/4		22.4	12.46			
				Bottom	HSS8X6X1/4		22.4	12.46			
			Post	W6x15		72.5	12.1				
			Post-Deck Conn.			87.5	14.6				
							Σ = ***93.9***				
	8	1	Rails	Top	HSS12X4X5/16		31.8	31.14	SEVEN SPAN	82.3	509.4
				Middle	HSS8X6X5/8		50.8	26.14			
				Bottom	HSS8X6X5/8		50.8	26.14			
			Post	W6x15		72.5	9.1				
			Post-Deck Conn.			87.5	10.9				
							Σ = ***153.5***				
		1.5	Rails	Top	HSS12X4X1/4		25.8	25.47	FIVE SPAN	81.5	413.1
				Middle	HSS8X6X1/4		22.4	12.46			
				Bottom	HSS8X6X1/4		22.4	12.46			
			Post	W6x15		72.5	9.1				
			Post-Deck Conn.			87.5	10.9				
							Σ = ***90.7***				
	10	1	Rails	Top	HSS12X4X1/2		48.9	46.45	SEVEN SPAN	80.9	977.0
				Middle	HSS8X6X5/8		50.8	26.14			
				Bottom	HSS8X6X5/8		50.8	26.14			
			Post	W6x15		72.5	7.3				
			Post-Deck Conn.			87.5	8.8				
							Σ = ***166.5***				
		1.5	Rails	Top	HSS12X4X5/16		31.8	31.14	FIVE SPAN	81.9	636.8
				Middle	HSS8X6X3/8		32.6	17.69			
				Bottom	HSS8X6X3/8		32.6	17.69			
			Post	W6x15		72.5	7.3				
			Post-Deck Conn.			87.5	8.8				
							Σ = ***113.0***				

A post spacing equal to 8 ft (2.4 m) was preferred in order to lower the number of post-to-deck connections. Moreover, a DMF equal to 1.5 was desired to lower the weight per foot of the system by approximately 20% to 30%. Therefore, the preferred configuration for the MASH 2016 TL-4 steel, side-mounted, beam-and-post, bridge rail consisted of a HSS 12-in. x 4-in. x ¼-in. (HSS 304.8-mm x 101.6-mm x 6.4-mm) section for the top rail, HSS 8-in. x 6-in. x ¼-in. (HSS 203.2-mm x 152.4-mm x 6.4-mm) sections for the lower rails, a post spacing of 8 ft (2.4 m), and a DMF for the posts equal to 1.5, as depicted in Figure 61. For SUT impact scenarios, the lateral barrier resistance when considering all of the three rails was 81.5 kips (362.5 kN) for a five-span collapse. For pickup truck impact scenarios, lateral barrier resistances were 67.1 kips (298.5 kN) for a three-span collapse when considering the lower two rails and 107.2 kips (476.9 kN) for a five-span collapse when considering all three of the rails.

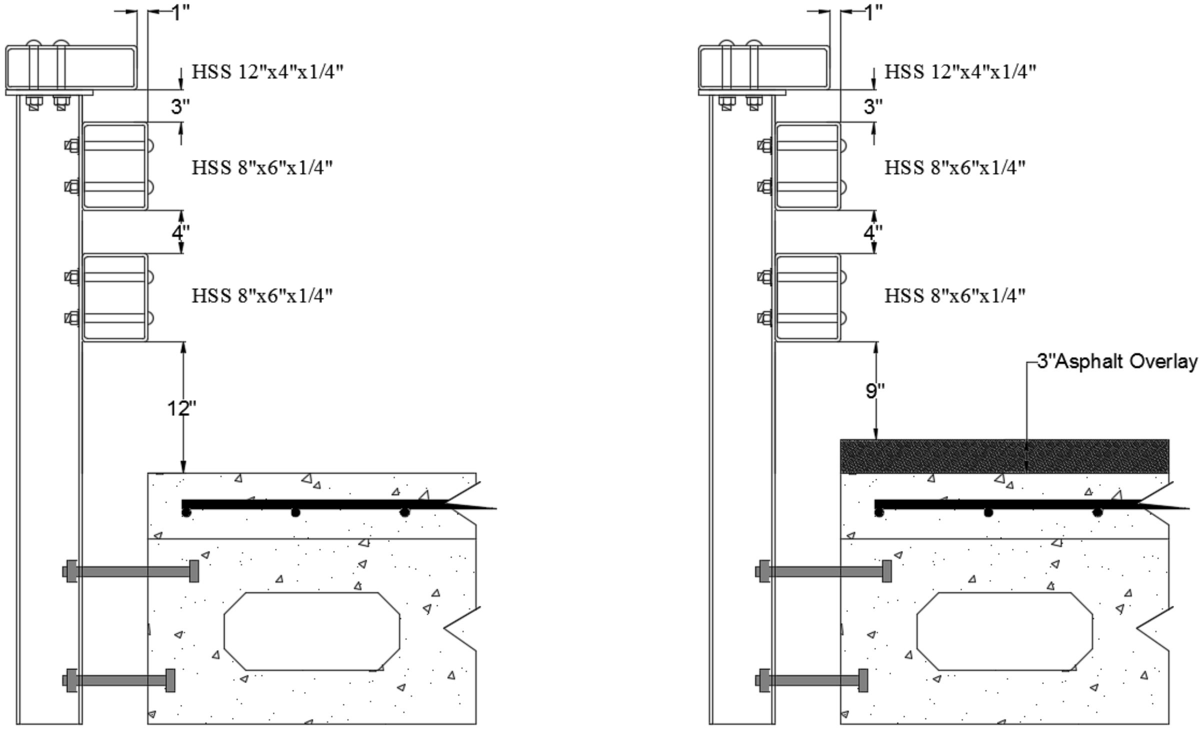


Figure 61. Final Bridge Rail Configuration on Deck #3 for New MASH 2016 TL-4 Bridge Rail

6 DESIGN OF BRIDGE RAIL CONNECTIONS

6.1 Overview

Post-to-rail and rail-to-rail connections were designed for the new bridge rail. In an attempt to meet design preferences from the Illinois and Ohio DOTs, horizontal slotted bolt holes were used for post-to-rail and rail-to-rail connections. These horizontal slotted bolt holes provided a workable longitudinal, construction tolerance of ⅝ in. (16 mm) for the installation and removal of the system. These slotted bolt holes were located in the front flange of the post for the middle and bottom rails and in the top rail mounting brackets. Slotted bolt holes were also provided in both ends of the three rails to connect the splice tubes to adjacent rails.

For the post-to-rail connections, a double-angle bracket was initially suggested for attaching the top rail to the top region of the post within the MASH 2016 TL-4 Prototype Bridge Rail. However, an alternative configuration consisted of a steel plate welded to the top of each vertical post. For the middle and bottom rails, a pair of staggered round bolts were used to attach each rail to the front flange of each post. For the rail-to-rail connections, both rectangular HSS steel section tubes with external shim plates and welded, built-up steel tubes were designed to properly connect the ends of the three rails, while providing continuity across the joints. These connections are described in the following sections.

6.2 Post-to-Rail Connections

A longitudinal tolerance of ⅝ in. (16 mm) was provided in the post-to-rail connections to facilitate the installation process. Consequently, horizontal slotted bolt holes were located in the front flanges of the posts, and round bolt holes were provided in the rails for the post-to-rail connections. The slotted bolt holes and round bolt holes were configured based on the size of the bolts. The bolt holes were determined to be ⅛ in. (3.2 mm) larger than the bolt diameter. The post-to-rail connection bolts for the three rails utilized round heads to reduce the potential for vehicle components to snag on the heads.

6.2.1 Top Rail Mounting Bracket

6.2.1.1 Lateral Design Loading for Top Rail Mounting Bracket

The design lateral loading imparted to the interface between the mounting bracket and the bottom of the top rail was calculated to design the vertical bolts against shear. The lateral load applied to the interface was estimated using a worst-case, simplified model that represents the bridge rail system with a hinge at the base of the post, which disregards its cantilevered bending contribution. Similarly to Section 5.8, the rail spans were assumed to resist a maximum bending moment under concentrated loads at the center of the span equal to PL/6, which represents an intermediate bending condition between simply-supported and fixed-end beams. The three rails with a hinged support were assumed to be 16 ft (4.8 m) long to represent the length of two spans. The lower rails consisted of HSS 8-in. x 6-in. x ¼-in. (HSS 203.2-mm x 152.4-mm x 6.4-mm) sections with a plastic section modulus of 13.9 in.^{3} (227,780 mm^3). The maximum concentrated load that the lower rails could resist with a hinged support post was based on the maximum plastic moment capacity for the rails, calculated with Equations 29 through 31. Using Equation 31, the maximum load that can be applied to each of the lower two rails was based on the plastic bending capacity. For this example, it was calculated as 21.7 kips (96.5 kN).

$$M_P = \frac{P_{MAX}\, L}{6} \tag{29}$$

where:

M_P = maximum plastic moment for two-span beams with intermediate hinged post, end posts without translation and concentrated load at midspan (kip-in.);
P_{MAX} = maximum concentrated load applied at rail midspan (kips); and
L= length of the beam (in.) for two spans.

$$P_{MAX} = \frac{6\, M_P}{L} \tag{30}$$

$$P_{MAX} = \frac{6\, F_Y\, Z_Y}{L} \tag{31}$$

where:

P_{MAX} = maximum concentrated load applied at rail midspan (kips);
M_{MAX} = maximum plastic moment for two-span beams with intermediate hinged post, end posts without translation, and concentrated load at midspan (kip-in.);
L= length of the beam (in.);
F_Y = yield strength of lower rails, (ksi);
Z_Y = plastic section modulus for weak axis (in.^{3}).

$$\mathrm{P_{MAX}} = \frac{6\ \mathrm{F_Y\ Z_Y}}{\mathrm{L}} = \frac{6\ (50\ \mathrm{ksi})\ (13.9\ \mathrm{in.}^3)}{192\ \mathrm{in.}} = 21.7\ \mathrm{kips}$$

The simplified model was developed using the heights of the lower two rails relative to a bridge deck configuration with the shortest post length. The center of the bottom and middle rails were 20 in. (508 mm) and 32 in. (813 mm), respectively, above the location of the tension anchor rods, as depicted in Figure 62. The maximum loading applied to the interface between the bottom of the top rail and the mounting bracket was estimated by summing of moments around the base of assumed hinged post using Equation 32.

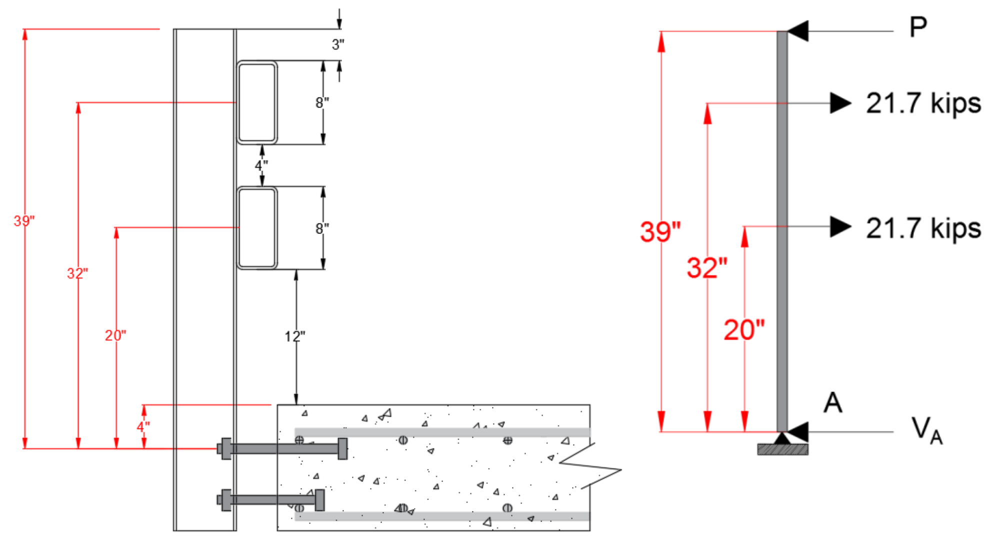


Figure 62: Simplified Model for Mounting Bracket Interface Loading.

$$\Sigma M_A = 0 = \frac{P_{INTERFACE} * H_{INTERFACE}}{\Sigma(P_{MAX} * H_i)} \tag{32}$$

where:

$P_{INTERFACE}$ = maximum concentrated load applied at the interface between the mounting bracket and the top rail (kips);
$H_{INTERFACE}$ = distance between interface to tension anchor rods (in.);
P_{MAX} = maximum concentrated load applied at each rail midspan (kips); and
H_i = distance between of each rail to tension anchor rods (in.).

$$P_{INTERFACE} * H_{INTERFACE} = \Sigma(P_{MAX} * H_i)$$

$$P_{INTERFACE} = \frac{\Sigma(P_{MAX} * H_i)}{H_{INTERFACE}} = \frac{(21.7 \text{ kips} * 20 \text{ in.}) + (21.7 \text{ kips} * 32 \text{ in.})}{39 \text{ in.}} = 28.9 \text{ kips}$$

The maximum transverse shear load at the interface between the mounting bracket and the top rail was calculated as 28.9 kips (128.6 kN). Therefore, the mounting bracket bolts had to provide transverse shear capacity equal to or greater than 28.9 kips (128.6 kN).

6.2.1.2 Review Concepts

Two main concepts were produced to attach the top rail to the post. The first concept consisted of a double-angle bracket bolted between the top rail and each post's web. The second concept consisted of a ⅜-in. (10-mm) thick, fully-welded, horizontal steel plate anchored to the top of each post. This plate had longitudinal slotted bolt holes to connect the top rail to posts.

6.2.1.3 Double-Angle Bracket Concept

The double-angle bracket concept was based on the IL/OH Prototype Bridge Rail. Two L 5-in. x 5-in. x ½-in. (L 127-mm x 127-mm x 12.7-mm) by 4½-in. (114-mm) long sections were selected to properly attach the rails to the top of the posts and allow for the angles to fit between the post flanges. Two bolt configurations were designed, as shown in Figure 63. For both options, each double-angle bracket was bolted to the post's web with two ASTM A449, ¾-in. (19-mm) diameter by 1¾-in. (44-mm) long, round-head steel bolts. Option 1 included two ASTM A449 ⅞-in. (22-mm) diameter by 6-in. (152-mm) long, vertical round-head steel bolts. Option 2 included four ASTM A449 ¾-in. (19-mm) diameter by 6-in. (152-mm) long, vertical round-head steel bolts. The bolts used ASTM F436 round SAE washers and ASTM A563 Grade DH heavy hex nuts. The sizes of the slotted holes were 1 in. (25 mm) diameter by 1½ in. (38 mm) long for Option 1 and ⅞ in. (22 mm) diameter by 1½ in. (38 mm) long for Option 2, which provided the ⅝ in. (16 mm) desired longitudinal construction tolerance.

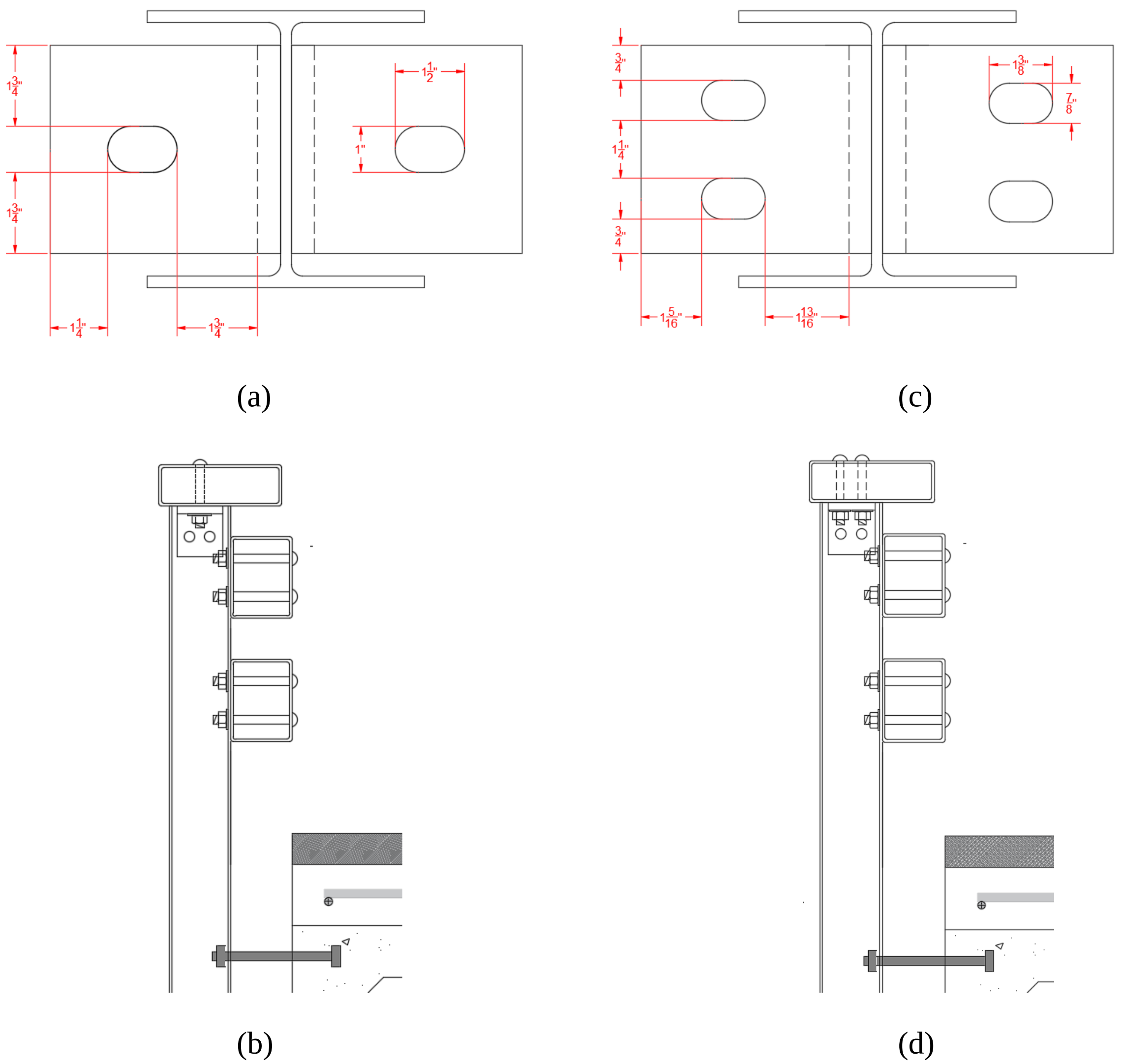


Figure 63: Top Rail Double-Angle Bracket Concept - (a) Plan View Option 1 without Top Rail, (b) Side View Option 1, (c) Plan View Option 2 without Top Rail, and (d) Side View Option 2

The bolt tear-out and bearing strength capacities in the transverse and longitudinal axes as well as the tensile and shear capacities of the bolts for Options 1 and 2 were calculated using equations found in the AISC *Steel Construction Manual* [49]. Equation J3-6C from the AISC Steel Construction Manual [49] was used to calculate the available bearing strength and bolt tear-out at the slotted hole, $ØR_n$, using (a) clear distances in the direction of the force between the edge of the hole and the edge of the adjacent hole or edge of the material, l_c, (b) the thickness of the steel angle, t, equal to ½ in. (12.5 mm), and (c) the specified minimum tensile strength of the steel angle, F_u, as shown in Equations 33 and 34. The tensile capacity and shear capacity of the bolts were calculated using Equations 35 and 36, which were found in Section J3-1 of the AISC *Steel Construction Manual*. An example of Option 1 plate tear-out, plate bearing, bolt shear, and bolt tensile capacities are shown below, in Section 6.2.1.3.1.

For bolt tear-out:

$$ØR_n = Ø\ 1.0\ l_c\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{33}$$

where:

$ØR_n$ = factored available tear-out (kips);
Ø = reduction factor, 0.75;
l_c = clear distance between the edge of the slotted bolt hole and the edge of the material (in.);
t = thickness of connected material (in.); and
F_u = specified minimum tensile strength of the connected material (ksi).

For bearing strength:

$$ØR_n = Ø\ 2.0\ d\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{34}$$

where:

$ØR_n$ = factored available bearing strength (kips);
Ø = reduction factor, 0.75;
d = nominal bolt diameter (in.);
t = thickness of connected material (in.); and
F_u = specified minimum tensile strength of the connected material (ksi).

For bolt tensile capacity:

$$ØR_n = Ø\ F_{nt}\ A_b \qquad \text{[AISC J3-1]} \tag{35}$$

where:

$ØR_n$ = factored available tensile capacity (kips);
Ø = reduction factor, 0.75;
F_{nt} = nominal tensile strength (ksi); and
A_b = nominal unthreaded area of bolt (in.^{2}).

For bolt shear capacity:

$$\emptyset R_n = \emptyset\, m\, F_{nv}\, A_b \qquad \text{[AISC J3-1]} \qquad (36)$$

where:

$\emptyset R_n$ = factored available shear capacity (kips);
$\emptyset$ = reduction factor, 0.75;
m = number of shear planes;
F_{nv} = nominal shear strength (ksi); and
A_b = nominal unthreaded area of bolt (in.^{2}).

6.2.1.3.1 Example Problem No. 5 – Estimate Plate Tear-Out, Plate Bearing, Bolt Shear, and Bolt Tensile Capacities of Double-Angle Mounting Bracket – Option 1

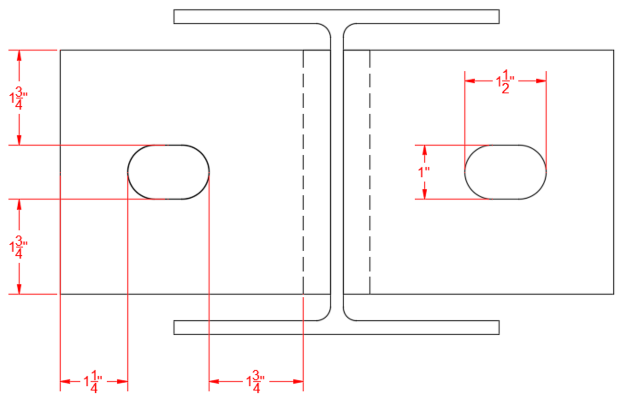


Figure 64: Plan View of Double-Angle Bracket without Top Rail

Step 1 – Calculate Bearing and Tear-Out Capacities of Mounting Bracket in Transverse Axis:

For bolt tear-out:

$$\emptyset R_n = \emptyset\ 1.0\ l_c\ t\ F_u \qquad \text{[AISC J3-6C]} \qquad (33)$$

$$\emptyset R_n = (0.75)(1.0)(1.75\text{ in.})\,(0.5\text{ in.})\,(65\text{ ksi}) = 42.6\text{ kips per bolt}$$

$$\emptyset R_n = 85.2\text{ kips per two bolts}$$

For bearing strength:

$$\emptyset R_n = \emptyset\ 2.0\ d\ t\ F_u \qquad \text{[AISC J3-6C]} \qquad (34)$$

$$\emptyset R_n = (0.75)(2.0)(0.875\text{ in.})\,(0.5\text{ in.})\,(65\text{ ksi}) = 42.7\text{ kips per bolt}$$

$$\emptyset R_n = 85.4\text{ kips per two bolts}$$

<u>Step 2 – Calculate Bearing and Tear-Out Capacities of Mounting Bracket in Longitudinal Axis:</u>

The web of the post would prevent one of the two bolts from tear-out in the horizontal double-angle in the longitudinal axis. However, the post web was disregarded for calculations. For this case, the bolt tear-out calculations considered l_{c1}=1¼ in. (32 mm) and l_{c2}=1¾ in. (44 mm). For bolt tear-out:

$$\emptyset R_n = \emptyset\ 1.0\ l_c\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{33}$$

$$\emptyset R_n = (0.75)\ (1.0)\ (1.25\text{ in.})\ (0.5\text{ in.})\ (65\text{ ksi}) = 30.5\text{ kips for bolt}$$

$$\emptyset R_n = (0.75)\ (1.0)\ (1.75\text{ in.})\ (0.5\text{ in.})\ (65\text{ ksi}) = 42.7\text{ kips for bolt}$$

$$\emptyset R_n = 73.2\text{ kips per two bolts}$$

For bearing strength:

$$\emptyset R_n = \emptyset\ 2.0\ d\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{34}$$

$$\emptyset R_n = (0.75)\ (2.0)\ (0.875\text{ in.})\ (0.5\text{ in.})\ (65\text{ ksi}) = 42.7\text{ kips per bolt}$$

$$\emptyset R_n = 85.4\text{ kips per two bolts}$$

<u>Step 3 – Calculate Shear and Tensile Capacities of Bolts:</u>

For bolt tensile capacity:

$$\emptyset R_n = \emptyset\ F_{nt} A_b \qquad \text{[AISC J3-1]} \tag{35}$$

$$\emptyset R_n = (0.75)(90\text{ ksi})(0.60\text{ in.}) = 40.5\text{ kips per bolt}$$

$$\emptyset R_n = 81.0\text{ kips per two bolts}$$

For bolt shear strength:

$$\emptyset R_n = \emptyset\ F_{nv} A_b \qquad \text{[AISC J3-1]} \tag{36}$$

$$\emptyset R_n = (0.75)(54\text{ ksi})(0.60\text{ in.}) = 24.3\text{ kips per bolt}$$

$$\emptyset R_n = 48.6\text{ kips per two bolts}$$

6.2.1.3.2 Double-Angle Mounting Bracket Summary

Similarly to the double-angle mounting bracket option 1, plate tear-out, plate bearing, bolt shear and bolt tensile capacities for double-angle mounting bracket option 2 were made. The results are depicted in Table 59. As depicted in Table 59, plate tear-out, transverse plate bearing, bolt shear and bolt tensile capacities were greater than 28.9 kips (128.6 kN) , which were satisfactory for lateral design loading. Although, other smaller bolt sizes and/or quantities would likely meet the required capacities, only two configurations are shown herein and seem to better fit with the structural components.

Table 59. Double-Angle Mounting Bracket Tear-Out, Bearing Capacity, Bolt Shear and Tensile Capacities

Double-Angle Bracket Option	Bolt Diameter	Transverse Axis		Longitudinal Axis		Bolts	
		Bolt Tear-Out, (kips)	Bearing Strength, (kips)	Bolt Tear-Out, (kips)	Bearing Strength, (kips)	Tensile Capacity, (kips)	Shear Capacity, (kips)
1	7/8 in., two bolts	85.2	85.4	73.2	85.4	81.0	48.6
2	3/4 in., four bolts	97.6	146.4	152.4	128.0	118.8	71.2

6.2.1.4 Fully-Welded Plate Concept

The fully-welded plate concept was based on the TxDOT T131 Bridge Rail, which was unsuccessfully crash tested with the MASH 2270P pickup truck due to a roll angle of 135 degrees [28]. However, the welded plate performed well and maintained connectivity between the top rail and the post during the impact event.

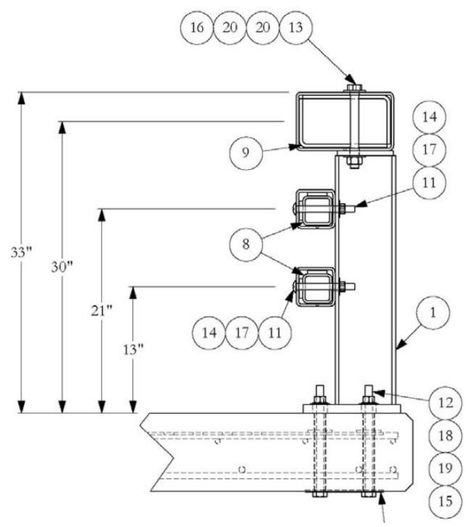


Figure 65. Schematic of TxDOT T131 Bridge Rail [28]

For the design of the fully-welded plate, the plate dimensions were increased to allow for edges to extend 1 in. (25 mm) beyond the post edges. This extension permitted to the back edge of the plate to be flush with the back face of the top rail. The 1-in. (25-mm) extension in the front provided additional resistance for vertical loading at the front of the top rail. The fully-welded plate consisted of an ASTM A572 Grade 50 steel plate measuring PL 8 in. x 8 in. x ⅜ in. (PL 203 mm x 203 mm x 10 mm), which was welded to the post with an all-around, $^{3}/_{16}$-in. (4.8-mm) fillet weld. Two options of bolt configurations were designed, as shown in Figure 66. Option 1 included two ASTM A449 ⅞-in. (22-mm) diameter by 6-in. (152-mm) long, round-head steel bolts. Option 2 included four ASTM A449 ¾-in. (19-mm) diameter by 6-in. (152-mm) long, round-head steel bolts. The bolts used ASTM F436 round SAE washers and ASTM A563 Grade DH heavy hex nuts. The sizes of the slotted holes were 1 in. (25 mm) diameter by 1½ in. (38 mm) long for Option 1 and ⅞ in. (22 mm) diameter by 1½ in. (38 mm) long for Option 2, which provided the ⅝ in. (16 mm) desired horizontal construction tolerance preferred by representatives from the Illinois and Ohio DOTs.

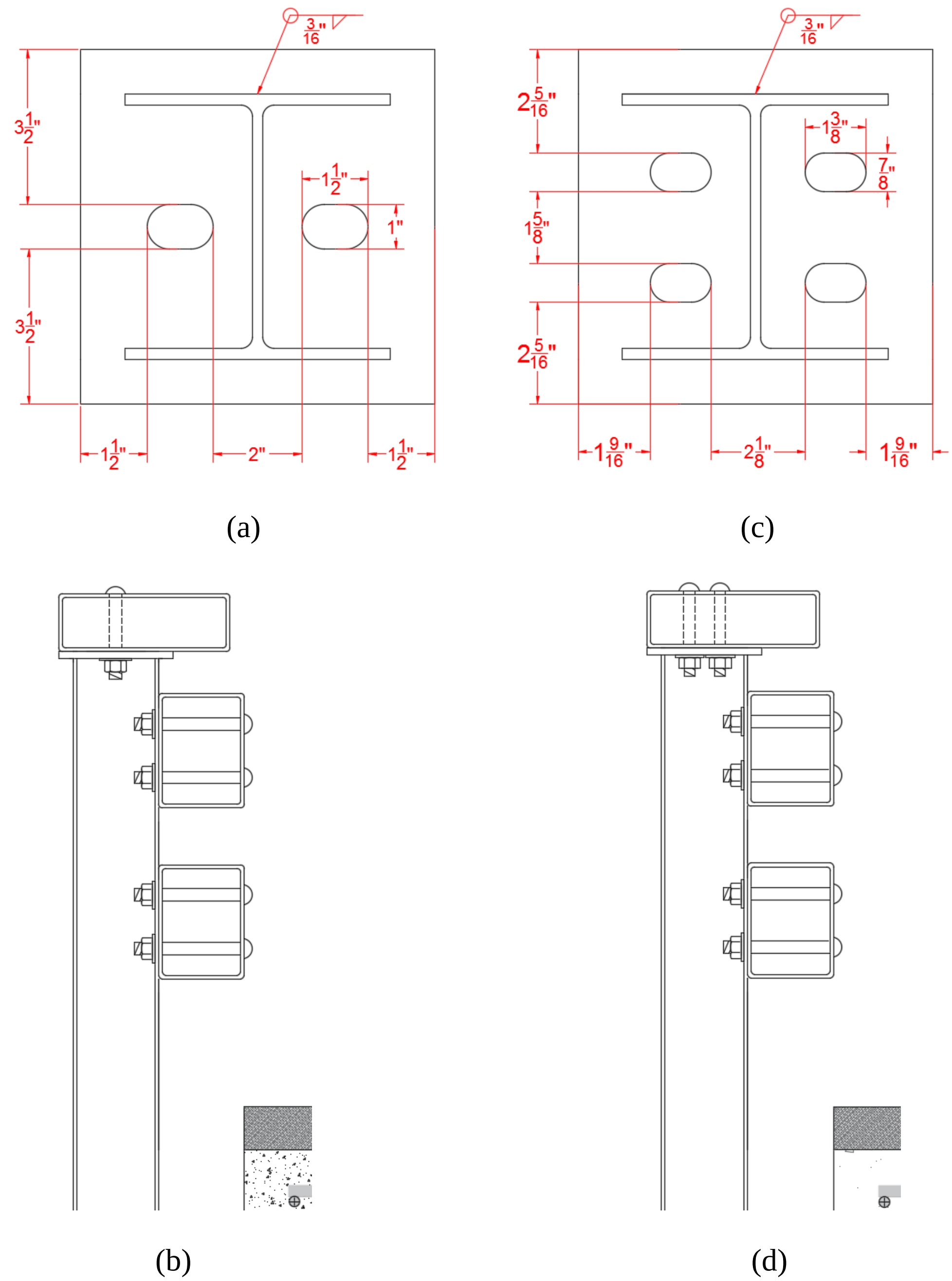


Figure 66. Top Rail Fully-Welded Plate Concept - (a) Plan View Option 1 without Top Rail, (b) Side View Option 1, (c) Plan View Option 2 without Top Rail, and (d) Side View Option 2

The bolt tear-out and bearing strength capacities in the transverse and longitudinal axes as well as the tensile and shear capacities of the bolts for Options 1 and 2 were calculated using equations found in the AISC *Steel Construction Manual* [49].

6.2.1.4.1 Example Problem No. 6 – Estimate Plate Tear-Out, Plate Bearing, Bolt Shear, and Bolt Tensile Capacities of Fully-Welded Plate Mounting Bracket – Option 2

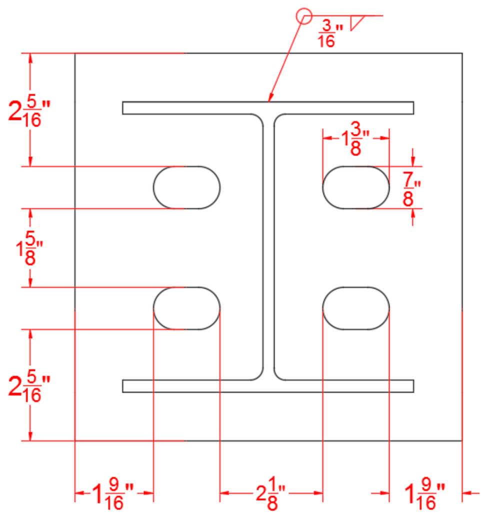


Figure 67. Plan View of Fully-Welded Plate Bracket without Top Rail

Step 1 – Calculate Bearing and Tear-Out Capacities of Mounting Bracket in Transverse Axis:

The flanges of the post would prevent two of four bolts from tear-out the fully-welded plate in the transverse axis. For this case, the bolt tear-out calculations considered l_{c1}=1⅝ in. (41 mm) and $l_{c2}=2^5/_{16}$ in. (58 mm).

For bolt tear-out:

$$\emptyset R_n = \emptyset\ 1.0\ l_c\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{33}$$

$$\emptyset R_n = (0.75)\ (1.0)\ (1.625\text{ in.})\ (0.375\text{ in.})\ (65\text{ ksi}) = 29.7\text{ kips per bolt}$$

$$\emptyset R_n = 59.4\text{ kips per two bolts}$$

$$\emptyset R_n = (0.75)\ (1.0)\ (2.3125\text{ in.})\ (0.375\text{ in.})\ (65\text{ ksi}) = 42.3\text{ kips per bolt}$$

$$\emptyset R_n = 84.6\text{ kips per two bolts}$$

$$\emptyset R_n = 144.0\text{ kips per four bolts}$$

For bearing strength:

$$\emptyset R_n = \emptyset\ 2.0\ d\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{34}$$

$$\emptyset R_n = (0.75)\ (2.0)\ (0.75\ \text{in.})\ (0.375\ \text{in.})\ (65\ \text{ksi}) = 27.4\ \text{kips per bolt}$$

$$\emptyset R_n = 109.6\ \text{kips per four bolts}$$

Step 2 – Calculate Bearing and Tear-Out Capacities of Mounting Bracket in Longitudinal Axis:

The web of the post would prevent two of the four bolts from tear-out in the horizontal plate in the longitudinal axis. However, the post web was disregarded for calculations. For this case, the bolt tear-out calculations were made considering the l_{c1}=1$^{9}/_{16}$ in. (39.7 mm) and l_{c2}= $^{15}/_{16}$ in. (23.8 mm).

For bolt tear-out:

$$\emptyset R_n = \emptyset\ 1.0\ l_c\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{33}$$

$$\emptyset R_n = (0.75)(1.0)(1.5625\ \text{in.})(0.375\ \text{in.})(65\ \text{ksi}) = 28.6\ \text{kips per bolt}$$

$$\emptyset R_n = 57.2\ \text{kips per two bolts}$$

$$\emptyset R_n = (0.75)(1.0)(0.9375\ \text{in.})(0.375\ \text{in.})(65\ \text{ksi}) = 17.1\ \text{kips per bolt}$$

$$\emptyset R_n = 34.2\ \text{kips per two bolts}$$

$$\emptyset R_n = 91.4\ \text{kips per four bolts}$$

For bearing strength:

$$\emptyset R_n = \emptyset\ 2.0\ d\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{34}$$

$$\emptyset R_n = (0.75)(2.0)(0.75\ \text{in.})(0.375\ \text{in.})(65\ \text{ksi}) = 27.4\ \text{kips per bolt}$$

$$\emptyset R_n = 109.6\ \text{kips per four bolts}$$

Step 3 – Calculate Shear and Tensile Capacities of Bolts:

For bolt tensile capacity:

$$\emptyset R_n = \emptyset\ F_{nt} A_b \qquad \text{[AISC J3-1]} \tag{35}$$

$$\emptyset R_n = (0.75)(90\ \text{ksi})(0.44\ \text{in.}) = 29.7\ \text{kips per bolt}$$

$$\emptyset R_n = 118.8\ \text{kips per four bolts}$$

For bolt shear strength:

$$\emptyset R_n = \emptyset\, F_{nv} A_b \qquad \text{[AISC J3-1]} \tag{36}$$

$$\emptyset R_n = (0.75)(54\ \text{ksi})(0.44\ \text{in.}) = 17.8\ \text{kips per bolt}$$

$$\emptyset R_n = 71.2\ \text{kips per four bolts}$$

6.2.1.5 Fully-Welded Plate Mounting Bracket Summary

The plate tear-out, plate bearing, bolt shear, and bolt tension capacities were calculated for Options 1 and 2. The results are depicted in Table 60. As depicted in Table 60, plate tear-out, plate bearing, bolt shear, and bolt tensile capacities were greater than 28.9 kips (128.6 kN), which were satisfactory for lateral design loading from Section 6.2.1.1.

Table 60. Fully-Welded Plate Mounting Bracket Plate Tear-Out, Plate Bearing, Bolt Shear, and Bolt Tensile Capacities

Fully-Welded Plate Bracket Option	Bolt Diameter	Transverse Axis		Longitudinal Axis		Bolts	
		Bolt Tear-Out, (kips)	Bearing Strength, (kips)	Bolt Tear-Out, (kips)	Bearing Strength,(kips)	Tensile Capacity, (kips)	Shear Capacity, (kips)
1	7/8 in., two bolts	118.8	64.0	63.9	64.0	81.0	48.6
2	3/4 in., four bolts	144.0	109.6	134.8	109.6	118.8	71.2

6.2.1.6 Final Selection of Top Rail Mounting Bracket

After discussion with the Illinois and Ohio DOTs, it was decided to use the fully-welded, steel plate with four ¾-in. (19-mm) diameter bolts; since, it represented a simpler field connection for installing the rail to the posts. Moreover, the welded plate was expected to provide a more uniform tensile and shear capacity than the double-angle bolted bracket. Further, the fully-welded, top steel plate also used fewer bolts (i.e., no bolts through web), which facilitated the installation process, and it was considered to be more aesthetic from backside vantage points.

In addition, tear-out and bearing strength of the rails in the transverse and longitudinal axes as well as the bolt shear and bolt tensile capacities were calculated for the mounting bracket final selection, as depicted in Table 61. Note that the rails used ⅞-in. (22.2-mm) diameter round holes for the vertical bolts. As depicted in Table 61, plate tear-out, transverse plate bearing, bolt shear and bolt tensile capacities were greater than 28.9 kips (128.6 kN), which were satisfactory for lateral design loading.

Table 61. Top Rail Tear-Out, Bearing, Bolt Shear, and Bolt Tensile Capacities

Top Rail HSS Section	Bolt Diameter	Transverse Axis		Longitudinal Axis		Bolts	
		Bolt Tear-Out, (kips)	Bearing Strength, (kips)	Bolt Tear-Out, (kips)	Bearing Strength,(kips)	Tensile Capacity, (kips)	Shear Capacity, (kips)
	3/4 in., four bolts	192.0	146.3	146.2	146.3	118.8	71.2

6.2.1.7 Combined Shear and Tension Loading for Fully-Welded Plate Design

After the selection of the fully-welded plate Option 2, a design vertical loading was analyzed in order to ensure that the four ASTM A449 ¾-in. (19-mm) diameter by 6-in. (152-mm) long, round-head steel bolts were able to sustain a combined shear and tension loading. Based on MASH TL-4 vertical design loading specified in Section 3.3, a 38-kip (169-kN) vertical loading, which was distributed over 18 ft (5.5 m) was considered for this analysis [37]. Each post location was approximately subjected to 50 percent of total vertical load (≈19 kips), as shown in Figure 68. The design load applied on each mounting bracket was estimated as 19 kips (85 kN) due to the assumption of having two posts sustaining the loading.

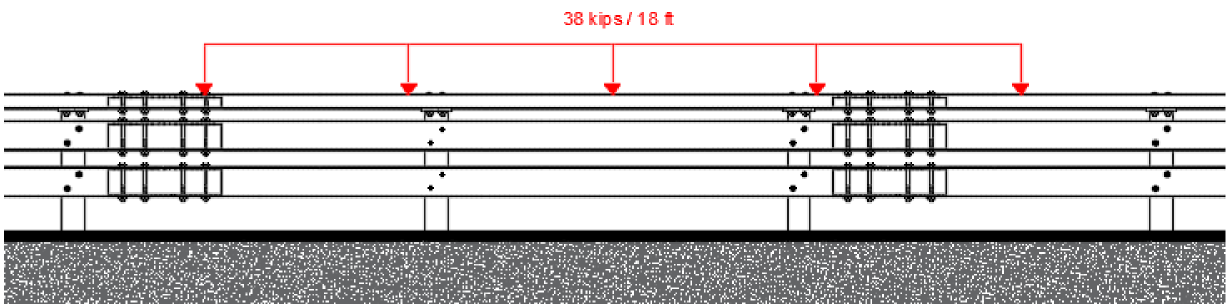


Figure 68. Vertical Design Load Distributed Over Two Spans

When analyzing the bolts subjected to downward (i.e., vertical) loading on the front face of top rail, as shown in Figure 69, an uneven loading was expected. The tensile loading applied to the four bolts of the welded plate varied depending on the relative distance between the vertical design loading and the lateral location of the bolt row. Assuming rigid top rail and fully-welded plate, the tensile loading applied to the two bolt rows was calculated using a linear load distribution, as shown in Figure 69. Considering the top rail would rotate at the right tip of the welded plate (i.e., a pin support), Equation 37, was used to find the tensile loading to the two bolt rows.

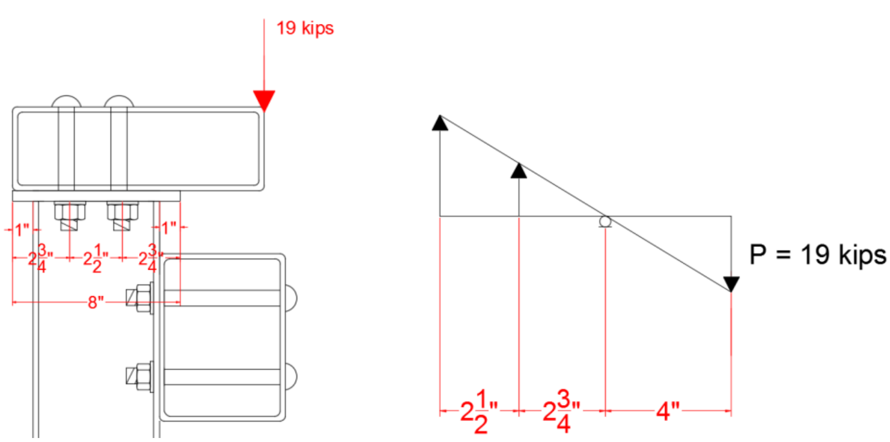


Figure 69. Top Rail Fully-Welded Plate Final Design - (a) Profile View, (b) and Linear Loading Distribution

$$(T_1)(d_1) + (T_1)\left(\frac{d_1}{d_2}\right)(d_1) = (P)(d_3) \tag{37}$$

where:

T_1 = tensile reaction of outer bolt (kips);
P = vertical loading (kips);
d_1= distance between pin support and inner bolt location (in.);
d_2= distance between pin support and outer bolt location (in.); and
d_3= distance between pin support and vertical loading (in.).

$$(T_1)(5.25\text{ in.}) + (T_1)\left(\frac{2.75\text{ in.}}{5.25\text{ in.}}\right)(2.75\text{ in.}) = (19\text{ kips})(4\text{ in.})$$

$$T_1 = 11.4\text{ kips for two outer bolts} = 5.7\text{ kips per outer bolt}$$

$$T_2 = 7.6\text{ kips for two inner bolts} = 3.8\text{ kips per inner bolt}$$

The tensile loading applied to the outer bolt row was then calculated as 5.7 kips (25.4 kN) for each outer bolt. A combined shear and tension loading analysis was then conducted on vertical bolts used with the fully-welded plate to ensure the performance of the fully-welded plate bolts. AISC *Steel Construction Manual* Equations J3-2 and J3.3a [49] were used to calculate the available tensile strength of the outer bolts subjected to tension and shear, as shown in Equations 38 through 40.

$$ØR_n = F'_{nt}\,A_b \qquad [\text{AISC J3-2}] \tag{38}$$

where:

F'_{nt} = nominal tensile stress modified to include the effects of shear stress (ksi); and
A_b = area of the bolt (in.^{2}).

$$F'_{nt} = 1.3\,F_{nt} - \frac{F_{nt}}{ØF_{nv}} f_{rv} \le F_{nt} \qquad [\text{AISC J3-3a}] \tag{39}$$

where:

F'_{nt} = nominal tensile stress modified to include the effects of shear stress (ksi);
F_{nt} = nominal tensile stress (ksi);
F_{nv} = nominal shear stress (ksi);
Ø = reduction factor, 0.75; and
f_{rv} = required shear stress (ksi).

$$f_{rv} = \frac{V_u}{A_b} \qquad [\text{AISC J3-3a}] \tag{40}$$

where:

f_{rv} = required shear stress (ksi).
V_u = maximum shear stress applied in one bolt (ksi); and
A_b = area of bolt (in.^{2})

$$f_{rv} = \frac{\frac{(28.9 \text{ kips})}{4 \text{ bolts}}}{0.44 \text{ in.}^2} = 16.42 \text{ ksi}$$

$$F'_{nt} = 1.3\ (90 \text{ ksi}) - \frac{90 \text{ ksi}}{(0.75)\ (54 \text{ ksi})}(16.42 \text{ ksi}) \leq 90\text{ksi}$$

$$F'_{nt} = 80.51 \text{ ksi}$$

$$ØR_n = F'_{nt}\ A_b \qquad [\text{AISC J3}-2] \tag{38}$$

$$ØR_n = (80.51 \text{ ksi})(0.44 \text{ in.}^2\ ) = 35.4 \text{ kips}$$

As shown above, the nominal tensile stress of a bolt subjected to combined tension and shear was calculated as 80.51 ksi (555.1 MPa). Therefore, the modified or available tensile strength of a bolt subjected to tension and shear loading was calculated as 35.4 kips (157.5 kN). This bolt tensile strength was greater than a 5.7-kip (25.4 kN) tensile load applied each outer bolt by the 19-kip (85-kN) vertical load at a post location, which was satisfactory for combined shear and tension loading.

6.2.2 Middle and Bottom Post-to-Rail Connections

A pair of staggered, horizontal, ASTM A449 ¾-in. (19.1-mm) diameter by 7½-in. (190.5-mm) long round-head bolts with ASTM F436 round SAE washers and ASTM A563 Grade DH heavy hex nuts were used to attach the vertical faces of the middle and bottom rails to the front flanges of the posts. In order to provide a desired ⅝-in. (16-mm) longitudinal construction tolerance, a pair of staggered 1⅜-in. (35-mm) long by ⅞-in. (22.2-mm) diameter slotted bolt holes were provided to attach the middle and the bottom rails, as shown in Figure 70. Note that these slotted bolt holes were intended to be staggered to prevent having more than one hole in any cross-section of the rails.

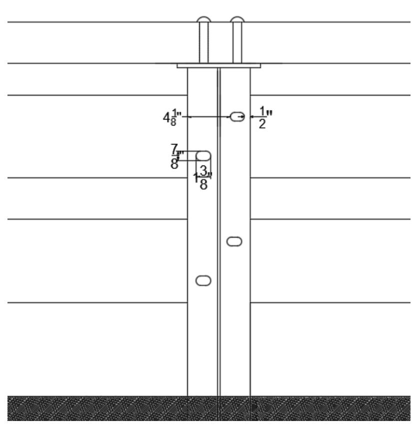


Figure 70. Post Slots Middle and Bottom Rail Locations for Post-to-Rail Connections

The rail bolt tear-out and bearing capacities in the longitudinal axis and the tensile and shear capacities of the bolts were calculated using the AISC *Steel Construction Manual* [49] and are shown below. For this case, the bolt tear-out calculations considered l_{c1}=½ in. (12.5 mm) and l_{c2}=4⅛ in. (105 mm). Note that these design ranges of bolt sizes for post-to-rail connections are commonly found in bridge rail systems.

For bolt tear-out in longitudinal axis:

$$\emptyset R_n = \emptyset\ 1.0\ l_c\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{33}$$

$$\emptyset R_n = (0.75)(1.0)(0.5\text{ in.})(0.26\text{ in.})(65\text{ ksi}) = 6.3\text{ kips per bolt}$$

$$\emptyset R_n = (0.75)(1.0)(4.125\text{ in.})(0.26\text{ in.})(65\text{ ksi}) = 52.3\text{ kips per bolt}$$

$$\emptyset R_n = 58.6\text{ kips per two bolts}$$

For bearing strength in longitudinal axis:

$$\emptyset R_n = \emptyset\ 2.0\ d\ t\ F_u \qquad \text{[AISC J3-6C]} \tag{34}$$

$$\emptyset R_n = (0.75)(2.0)(0.75\text{ in.})(0.26\text{ in.})(65\text{ ksi}) = 19.0\text{ kips per bolt}$$

$$\emptyset R_n = 38.0\text{ kips per two bolts}$$

For bolt tensile capacity:

$$\emptyset R_n = \emptyset\ F_{nt}\ A_b \qquad \text{[AISC J3-1]} \tag{35}$$

$$\emptyset R_n = (0.75)\ (90\text{ ksi})\ (0.44\text{in.}^2) = 29.7\text{ kips per bolt}$$

$$\emptyset R_n = 59.4\text{ kips per two bolts}$$

For bolt shear strength:

$$\emptyset R_n = \emptyset\ F_{nv}\ A_b \qquad \text{[AISC J3-1]} \tag{36}$$

$$\emptyset R_n = (0.75)\ (54\text{ ksi})\ (0.44\text{ in.}^2) = 17.8\text{ kips per bolt}$$

$$\emptyset R_n = 35.6\text{ kips per two bolts}$$

The bolt tear-out, rail bearing, bolt shear along each axis, and bolt tensile capacities of the bolts were calculated for the middle and bottom railing sections. The results are depicted in Table 62. Note that the middle and bottom rails used ⅞-in. (22.2-mm) diameter round holes for the horizontal bolts. The clear distance of both rails was based on the front flange bolt configuration for the bolt tear-out calculations considered l_{c1}= ¾ in. (19.1 mm) and l_{c2}=4⅜ in. (111.1 mm). The rail tear-out, transverse plate bearing, bolt shear and bolt tensile capacities were greater than 28.9 kips (128.6 kN), which were satisfactory for lateral design loading.

Table 62. Middle and Bottom Railing Sections Post-to-Rail Connections

Middle and Bottom Rail HSS Sections	Bolt Diameter	Longitudinal Axis		Bolts	
		Bolt Tear-Out, (kips)	Bearing Strength, (kips)	Tensile Capacity, (kips)	Shear Capacity, (kips)
	3/4 in., two bolts	112.7	73.1	59.4	35.6

6.3 Rail-to-Rail Connections

6.3.1 Rail Splices

Splice tube were designed to connect the ends sections of the three rails. The location of the splice tubes was planned to be longitudinally aligned at a ¼-span location, as shown in Figure 71. This ¼-span location represents an approximate location of the inflection point for the moment corresponding to a uniformly-loaded, fixed-end beam. For crash testing, the three splice tubes would be longitudinally aligned. If crash testing is later found to be successful, then rail splices could be located at any ¼- or ¾-span location.

Three splice tube concepts were created to provide a variety of options to meet the needs of the Illinois and Ohio DOTs as well as their fabricators and/or installers. The concepts consisted of (1) an HSS tube with welded shims, (2) a built-up, welded tube made with steel plates, and (3) a built-up, welded tube made with two-bent plates.

A ¾-in. (19-mm) expansion gap was incorporated at the three splice locations to account for the steel thermal expansion and contraction of the rails as well as construction tolerance. Further, a ¼-in. (6.4-mm) total inner construction tolerance gap was provided between the inner faces of the rails and outer faces of the splice tubes.

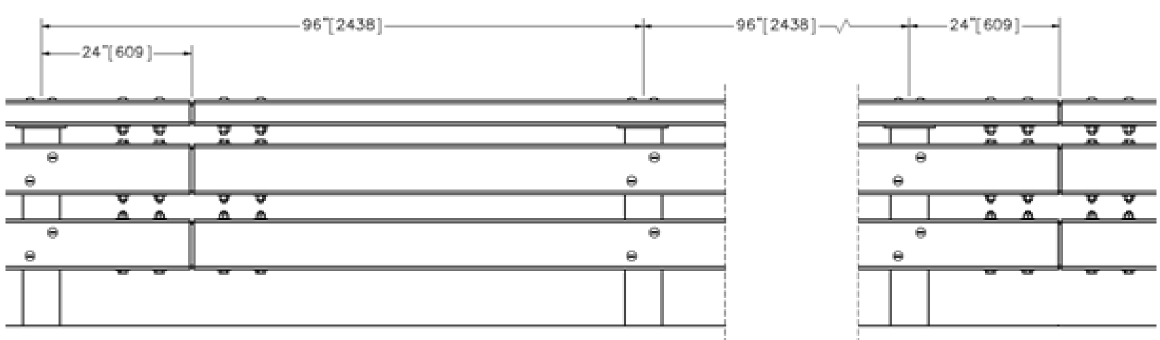


Figure 71. Location of Splice Tubes for Full-Scale Crash Tests

In order to prevent excessive joint rotations of splice tubes inside rails, which may lead to excessive rail deformations, it was determined to provide a maximum rotation angle equal to one degree within each rail end, as depicted in Figure 72. A one-degree rotation angle and satisfactory performance of splice tubes from MASH TL-3 TxDOT T131 Bridge Rail [28] and MASH TL-5 Verrazano-Narrows Bridge Rail [21] led to the selection. The length of each splice tube was determined to be 30 in. (762 mm) for the final configuration based on calculations below. This assumption was based on the acceptable performance of splice tubes observed in prior of crash-tested bridge rails. The rotation angle was calculated using Equation 41. A rotation angle was calculated as 0.97 degrees.

$$\theta_{ROTATION} = \tan^{-1}\left(\frac{x}{L}\right) \tag{41}$$

where:

x = total inner construction tolerance (in.); and
L = leg length (in.)

$$\tan(1°) = \frac{x}{L}$$

$$L = \frac{x}{\tan{(1°)}} = \frac{0.25 \text{ in.}}{\tan{(1°)}} = 14.32 \text{ in.}$$

$$2L = 14.3 \text{ in.}\,(2) = 28.64 \text{ in.}$$

$$\text{Length} \approx 28.64 \text{ in.} + \frac{3}{4} \text{ in.} = 29.39 \text{ in.} \quad \text{Use length equal to 30 in.}$$

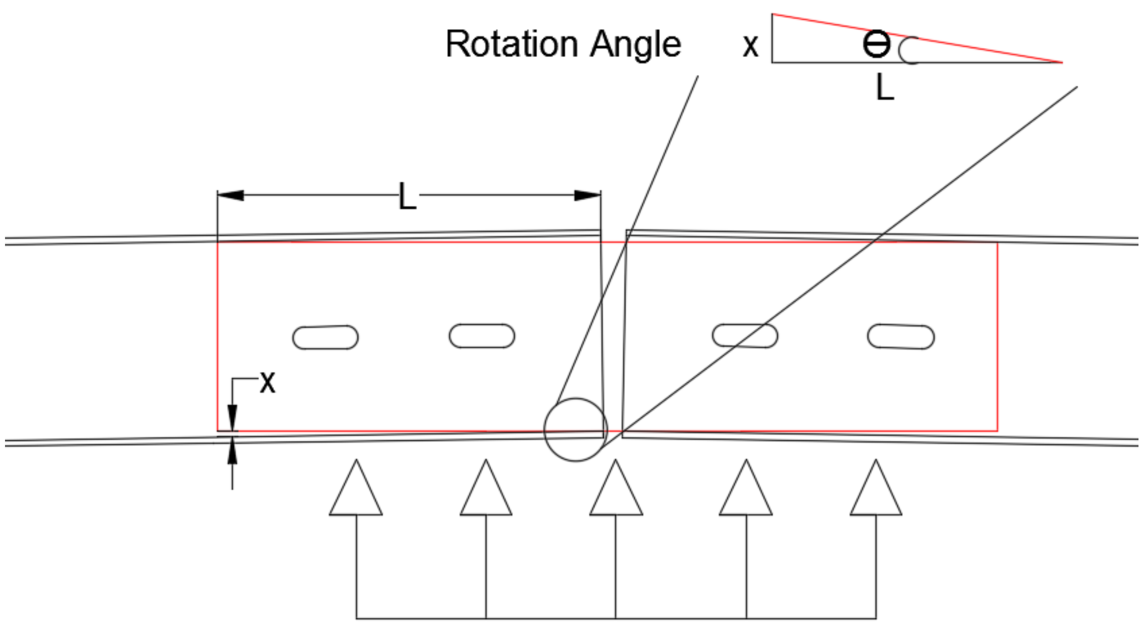


Figure 72. Rotation Angle of Splice Tube and Rail

6.3.2 HSS Section Tubes

The HSS top splice tube consisted of a rectangular HSS 10-in. x 3-in. x ⅜-in. (HSS 254-mm x 76.2-mm x 10-mm) section with a PL ¼-in. x 10-in. (PL 6.4-mm x 254-mm) top shim and two PL ⅝-in. x 1¾-in. (PL 15.9-mm x 44.5-mm) side shims. The HSS middle and bottom splice tubes consisted of a rectangular HSS 7-in. x 5-in. x ⅜-in. (HSS 178-mm x 127-mm x 10-mm) with a PL ¼-in. x 4-in. (PL 6.4-mm x 102-mm) top shim and two PL ⅛-in. x 6-in. (PL 3.2-mm x 152-mm) side shims. For the three splice tubes, as depicted in Figure 73, the side shims were attached to the splice tubes with ¼-in. (6.4-mm) long stitched fillet welds.

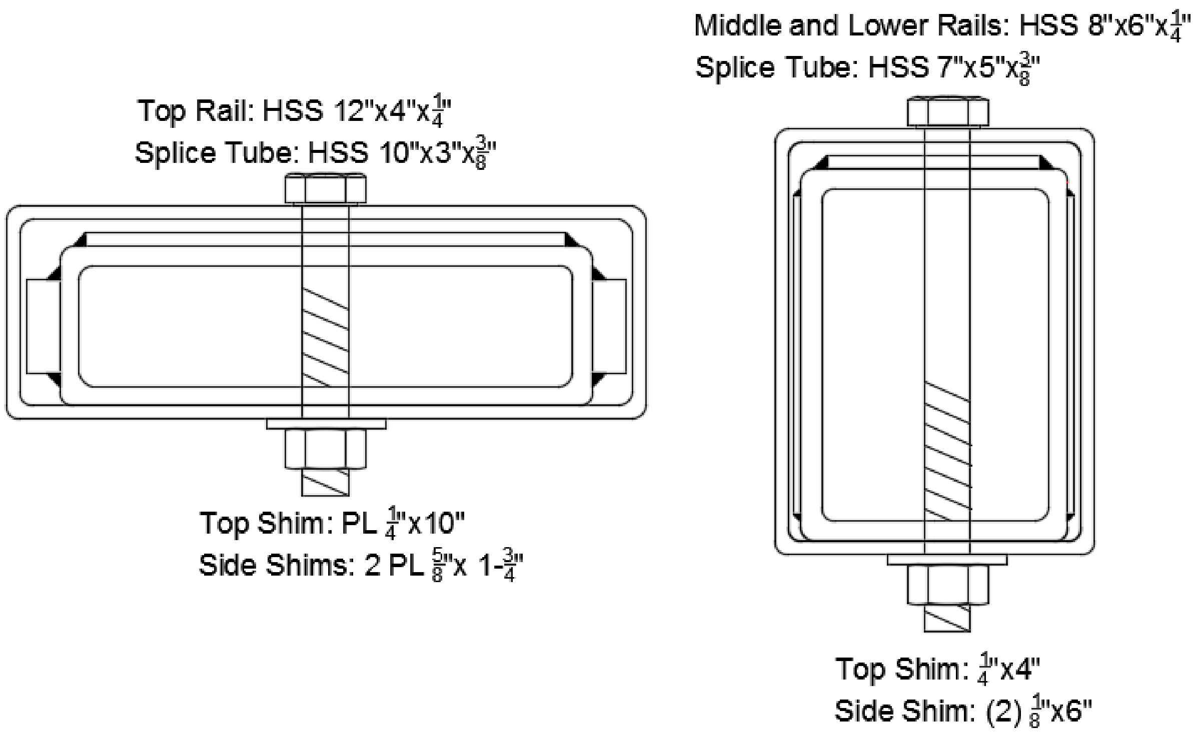


Figure 73. Rectangular HSS Tubes

6.3.3 Four-Plate (Built-Up) Welded Tubes

The top four-plate welded tubes for top rail consisted of two PL 30-in. x 10⅝-in. x 5/16-in. (PL 762-mm x 270-mm 8-mm) horizontal steel plates and two PL 30-in. x 2⅝-in. x ⅜-in. (PL 762-mm x 67-mm x 10-mm) vertical steel plates for the top rail. The four-plate welded tubes for the middle and bottom rails consisted of two PL 30-in. x 6⅝-in. x ⅜-in. (PL 762-mm x 168-mm 10-mm) horizontal plates and two PL 30-in. x 4⅝-in. x 5/16-in. (PL 762-mm x 117-mm x 8-mm) vertical plates, as shown in Figure 74.

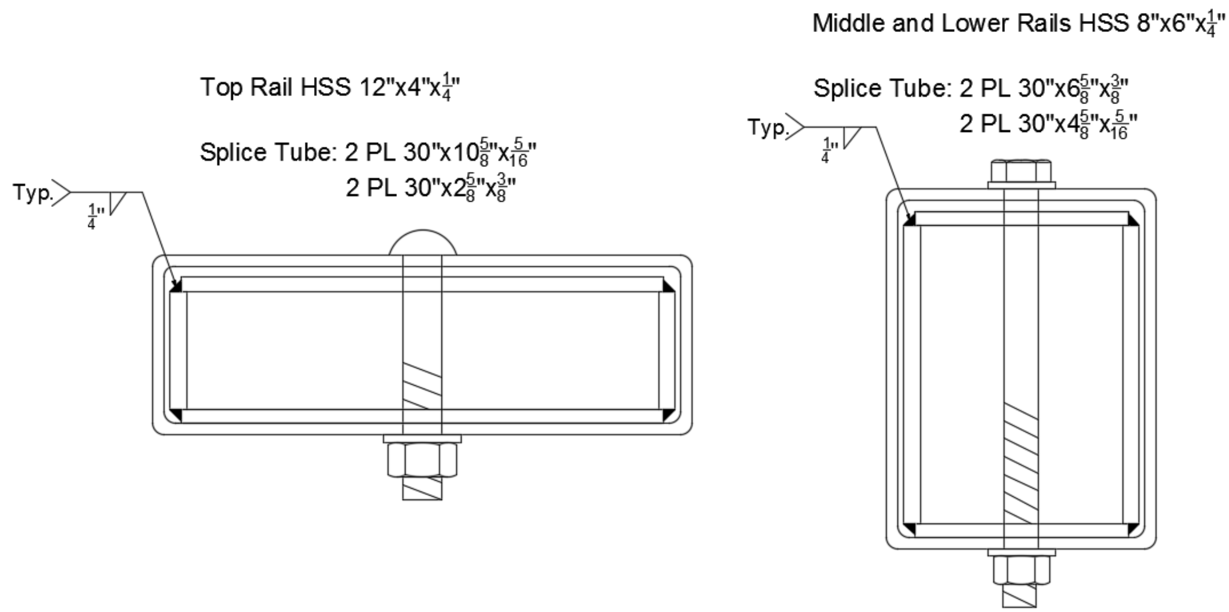


Figure 74. Four-Plate (Built-Up) Welded Tube

6.3.4 Two-Bent Plate Tubes

Two bent ⅜-in. (10-mm) thick plates was a second option for built-up, welded splice tubes. These bent L-plates were joined together with two ¼-in. (6.4-mm) continuous, partial-joint-penetration groove welds. The top splice tube consisted of two PL 30-in. x 13½-in. x ⅜-in. (PL 762-mm x 343-mm x 10-mm), and the middle and bottom splice tubes consisted of two PL 30-in. x 11½-in. x ⅜-in. (PL 762-mm x 292-mm x 10-mm). The two, welded, bent L-plate tubes are shown in Figure 75.

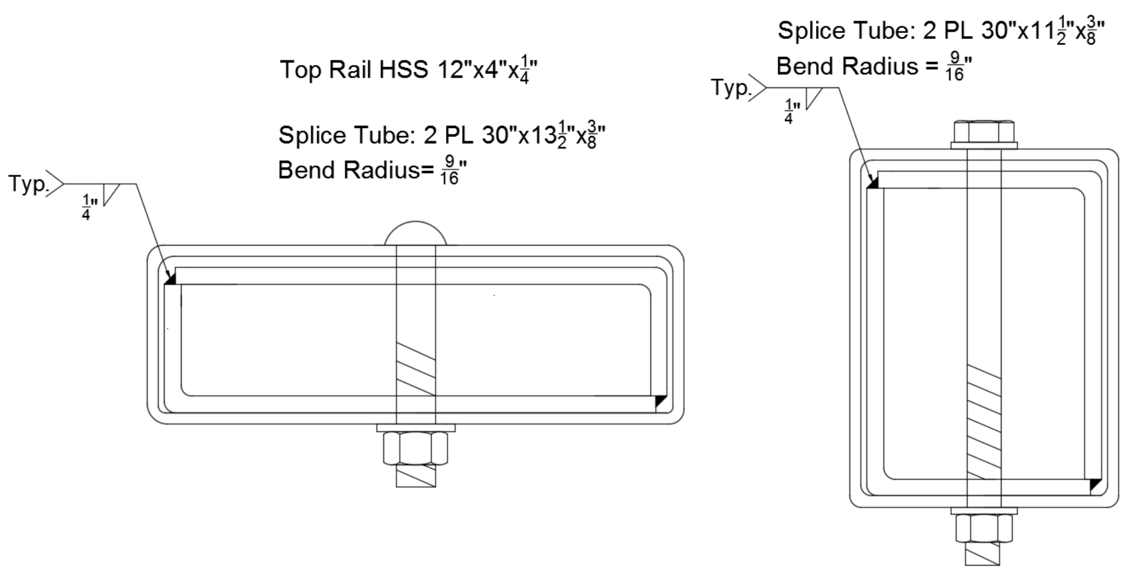


Figure 75. Two Welded Bent Plate Tube

6.3.5 Final Splice Tube Design

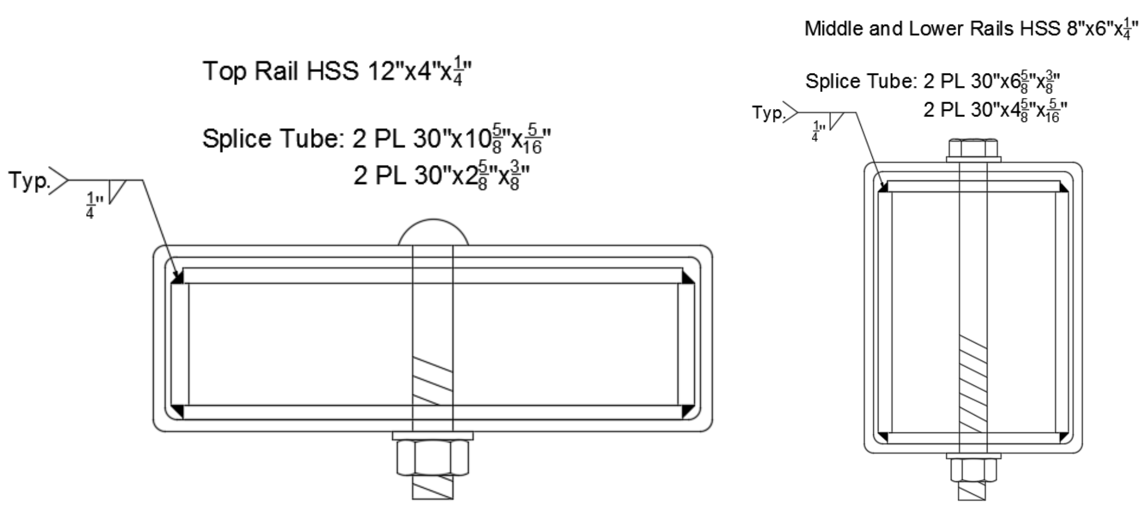
After meeting with representatives from the Illinois and Ohio DOTs, HSS splice tubes were disregarded for final design; since, they required a ⅝-in. (15.9-mm) thick shim to maintain a ¼-in. (6.4-mm) total inner construction tolerance. Furthermore, the HSS tubes shown in Figure 73 may cause uneven load distribution and lead to increased sidewall deformation, rail hinging at the splice tube locations, increased bridge rail deflections, reduced rail horizontal capacity, and failure for the full-scale crash tests. Built-up tubes were preferred for the final design of the new MASH 2016 TL-4 bridge rail.

The four-plate welded tube concept was selected for crash testing. Four ASTM A449 ¾-in. (19-mm) diameter by 6-in. (152-mm) long, round-head bolts with flats were selected for the top splice tube, while four ASTM A449 ¾-in. (19-mm) diameter by 9½-in. (241-mm) long hex-head bolts were selected for the middle and bottom splice tubes. Therefore, the splice tube options used ⅞-in. (22-mm) diameter, round bolt holes. The fabricator, Midwest Steel Works, Inc., used tack-welded, internal gusset as a method to maintain the internal shape of the splice assembly through the welding, as shown in Appendix C. However, the use of removable shim blocks and other jigging methods are acceptable to aid with the fabrication of the splice tube assemblies.

6.3.5.1 Moment and Shear Transfer of Splice Tube Selected

For the splice tubes connecting the ends of the three rails, each splice tube was required to provide equal or higher bending and shearing resistances than offered by the rail sections. Inadequate bending capacity of the splice tubes could lead to excessive deformation and hinging of the rails, which may also lead to increased vehicle instability, barrier override, and increased likelihood of rollover.

The bending moment capacity of the splice tubes was dependent on their plastic section moduli, since the yield strength of the rails and splice tubes was equal to 50 ksi (345 MPa). The plastic section moduli for the top and lower sections of the four-plate welded splice tubes were calculated using Equations 20 and 21. The plastic section moduli of the splice tubes in the horizontal direction were greater than that provided by the rail and therefore were satisfactory for moment transfer.



For top splice tube:

$$\text{For horizontal plates: } Z = (2)\frac{b * d^2}{2} \tag{20}$$

$$Z = (2)\frac{(0.3125 \text{ in.})(10.625 \text{ in.})^2}{4} \rightarrow Z = 8.8 \text{ in.}^3$$

$$\text{For vertical plates: } Z = (2)\frac{b}{4}(d^2 - d_1^2) \tag{21}$$

$$Z = (2)\frac{2.625 \text{ in.}}{4}(11.25\text{in.}^2 - 10.5 \text{ in.}^2) \rightarrow Z = 21.4 \text{ in.}^3$$

$$\Sigma Z_{TOP\ SPLICE} = 30.2 \text{ in.}^3 > Z_{TOP\ RAIL} = 25.6 \text{ in.}^3 \text{ OK!}$$

For middle and bottom splice tubes:

$$\text{For horizontal plates: } Z = (2)\frac{b * d^2}{4} \tag{20}$$

$$Z = (2)\frac{(0.3125 \text{ in.})(4.625 \text{ in.})^2}{4} \rightarrow Z = 3.3 \text{ in.}^3$$

$$\text{For vertical plates: } Z = (2)\frac{b}{4}(d^2 - d_1^2) \tag{21}$$

$$\rightarrow Z = (2)\frac{6.625 \text{ in.}}{4}(5.25 \text{ in.}^2 - 4.5 \text{ in.}^2) \rightarrow Z = 24.2 \text{ in.}^3$$

$$\Sigma Z_{LOWER\ SPLICE} = 27.5 \text{ in.}^3 > Z_{LOWER\ RAIL} = 13.9 \text{ in.}^3 \text{ OK!}$$

For shear transfer, the gross area of the splice tubes was required to be greater than the gross area of the rails. The gross area of the top splice tube was calculated as 8.6 in.^{2} (5,548 mm^2), while the gross area of the top rail was 7.1 in.^{2} (4,581 mm^2). The gross area of the middle and bottom splice tubes was calculated as 7.9 in.^{2} (5,097 mm^2) while the gross area of the middle and bottom rails was 6.2 in.^{2} (4,000 mm^2). Therefore, the shear transfer provided by the three splice tubes was deemed to be satisfactory.

6.3.6 Installation of Splice Tubes

After the four-plate, welded tube option was selected, it was determined that ⅜-in. (10-mm) cranking holes spaced at 2 in. (51 mm) intervals were required to more easily slide the splice tubes using a steel rod for installation and removal purposes. The installation of splice tubes could start with the bottom, then proceed to the middle, and finally finish at the top tube assembly. The splice tube could be installed into one end section of the rails. Then, the other end sections of the rails could be installed. The splice tube could be slid from one rail end section to its final position in the center of the expansion gap, as depicted in Figure 76.

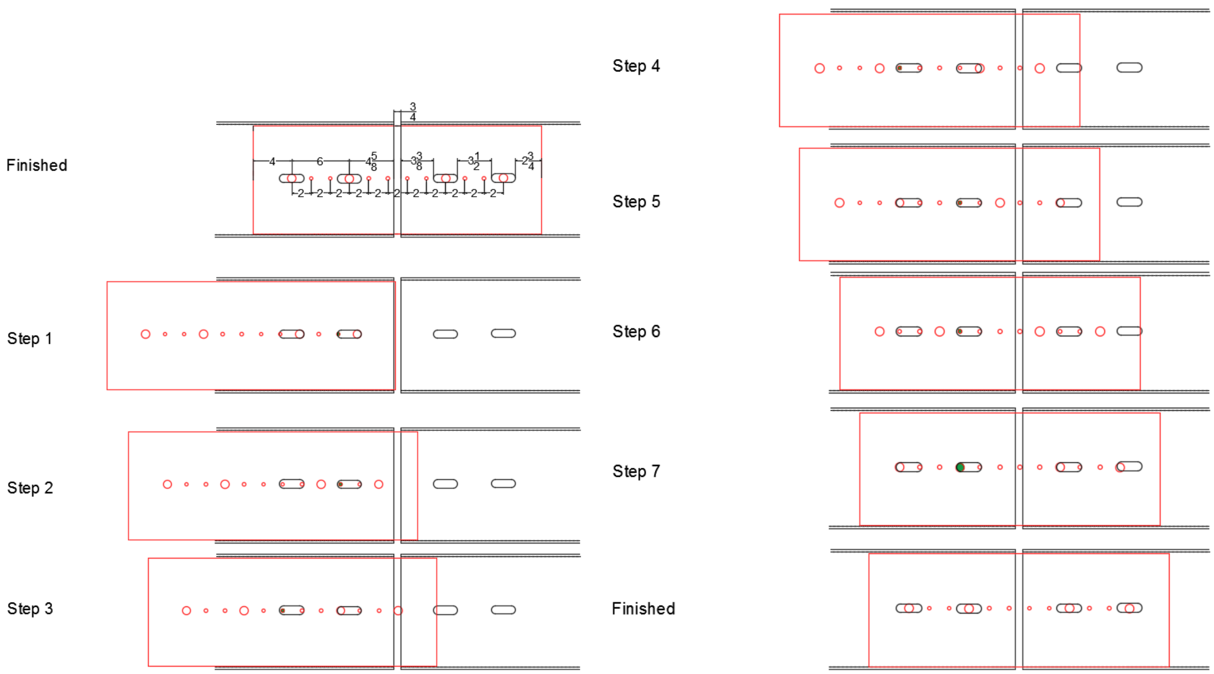


Figure 76. Example Incremental Installation Process for Splice Tubes

6.3.7 Installation and Removal of Splice Tube Bolts

Two methods were provided for the installation and removal of the bolts within the splice tubes. As shown in Figure 77, the first method consisted of releasing the two closest middle rail post-to-rail connection bolts that are closest left and right to the splice. Then, the unbolted rails can be pulled away 3 in. (76 mm) as the outer splices will give 3½ in. (89 mm) of expansion per side. At this point, the vertical slots of the middle rail splices become visible and ready for installation or removal of the vertical bolts. Finally, the bottom splice bolts can be installed or removed vertically upward and the top splice bolts vertically downwards. Vehicle snag on the two ends of the lower bolts in the lower rail opening was not deemed to be critical as it is only 4 in. (102 mm) tall.

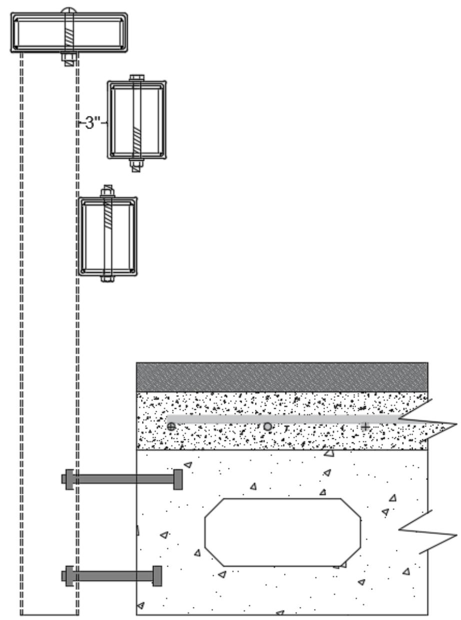


Figure 77. First Method for the Installation and Removal of Splice Tubes

As shown in Figure 78, the second method consisted of releasing the two top rail, post-to-rail bolts that are closest left and right to the splice. Again, the outer splices will provide 3½ in. (89 mm) of expansion per side. The top rail can then be lifted at least 7 in. (178 mm) to fit the middle splice bolts between the rails for installation or removal. The bottom splice bolts can then be installed vertically upward. Again, vehicle snag on the two ends of the lower bolts in the lower rail opening was not deemed critical as it is only 4 in. (102 mm) tall.

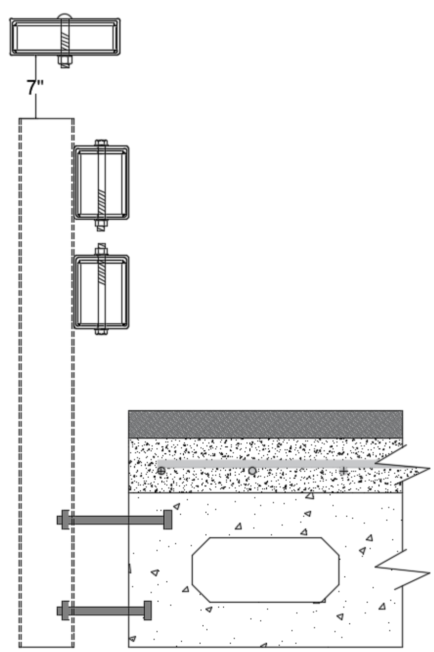


Figure 78. Second Method for Installation and Removal of Splice Tubes

7 COMPONENT TESTING RESULTS, DISCUSSION, AND SYSTEM MODIFICATIONS

7.1 Overview

Simultaneous to the design of the bridge railing Mauricio was conducting research to develop the post-to-deck attachment hardware, as well as adapt that hardware to multiple deck types. That research and development program is discussed in greater detail within a technical report [13] and thesis [14]. A very brief overview of the dynamic component testing program is highlighted in Chapter 7 and should not be considered as a complete summary of that program. Instead, some key findings from the component testing program were only noted herein for use in recalculating the lateral redirective capacity of the bridge rail.

Seven dynamic bogie tests were conducted to evaluate the behavior of Mauricio's preferred concept for the post-to-deck attachment hardware, which was anchored into a critical, reinforced-concrete, box-beam girder. The critical bridge deck configuration for the component testing program was selected from the four different bridge deck configurations commonly used by the Illinois and Ohio DOTs. The post-to-deck configuration was selected to allow for the highest lateral loading to be imparted to the deck. The concrete box-beam girder utilized by the Ohio DOT, as shown in Figure 79, was determined to provide a critical loading scenario to the upper thin slab of the box-beam as well as to its thin side wall. Although not described herein, several tensile anchor rod lengths, diameters, and embedment conditions, were tested and evaluated by Mauricio [13-14].

Based on the results from each of the tests, the design concept was either further refined or abandoned. Posts with varied post-to-deck attachment hardware were dynamically tested to determine the lateral resistive forces that would be developed, examine the energy that would be absorbed by the hardware, and evaluate whether damage would occur to the hardware and concrete box-beam girder. All dynamic component tests were conducted at MwRSF's Outdoor Proving Grounds located in Lincoln, Nebraska.

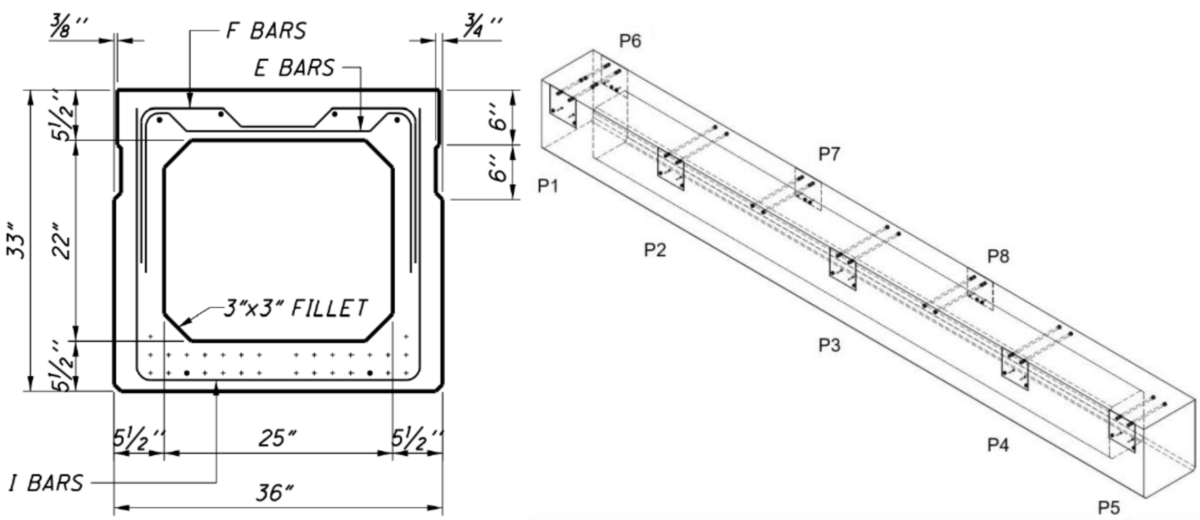


Figure 79. Critical Concrete Box-Beam Used in Dynamic Component Testing Program

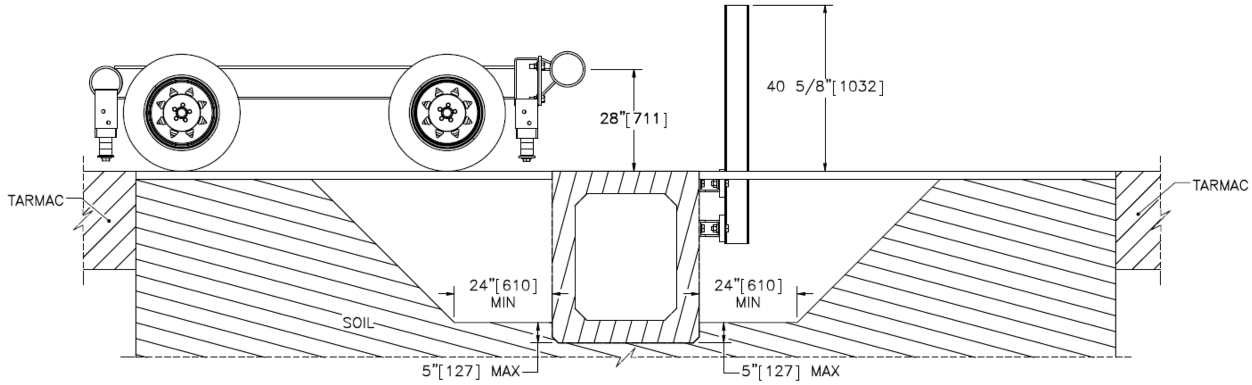


Figure 80. Bogie Testing Setup – End View of Box-Beam with Side-Mounted Post

7.2 Component Testing Conditions and Instrumentation

The target impact conditions consisted of an impact speed of approximately 20 mph (32 km/h) and an impact angle of 0 degrees, creating a "head-on" or full-frontal impact to the strong axis of bending for the post. The posts were impacted 28 in. (711 mm) above the top of the concrete box-beam girder for several reasons. The impact height was intended to represent the 2270P pickup truck, which has a minimum center of gravity equal to 28 in. (711 mm) above the ground line. In the final configuration of the new MASH 2016 TL-4 bridge rail, the middle railing had a center height located at 28 in (711 mm) above the top of the bridge deck when no asphalt overlay has been applied, as shown in Figure 81. Further, this height guaranteed that the W6x15 (W150x22.5) post would develop a plastic hinge due to the impact weight and velocity of the bogie. The weight of the bogie with the addition of the mountable impact head and accelerometers was 2,000 lb (907 kg) for the first two component tests. For test nos. ILOH 4-3 through ILOH 4-7, the bogie's weight was increased to 2,500 lb (1,132 kg) after observing that the impact head was sliding upward along the post as the bogie overrode the post. The posts of all of the dynamic component tests were mounted to a box-beam without a reinforced concrete slab nor an asphalt overlay to minimize the moment arm between the impact load height of the bogie and the tension anchor rod with a cover of 3 in. (76 mm), as shown in Figure 81.

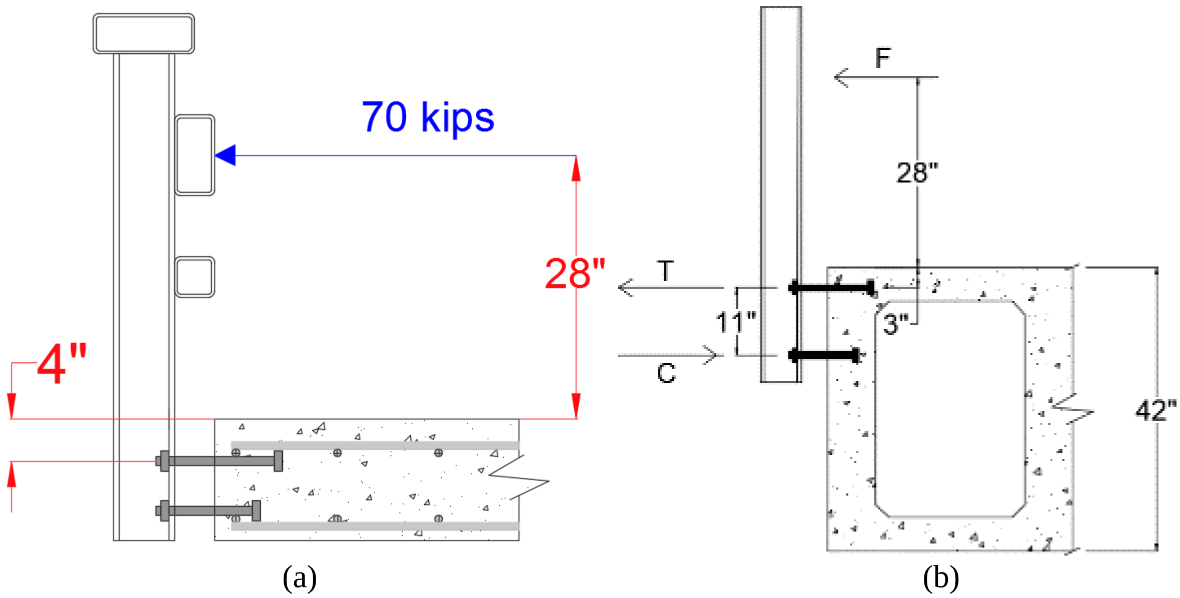


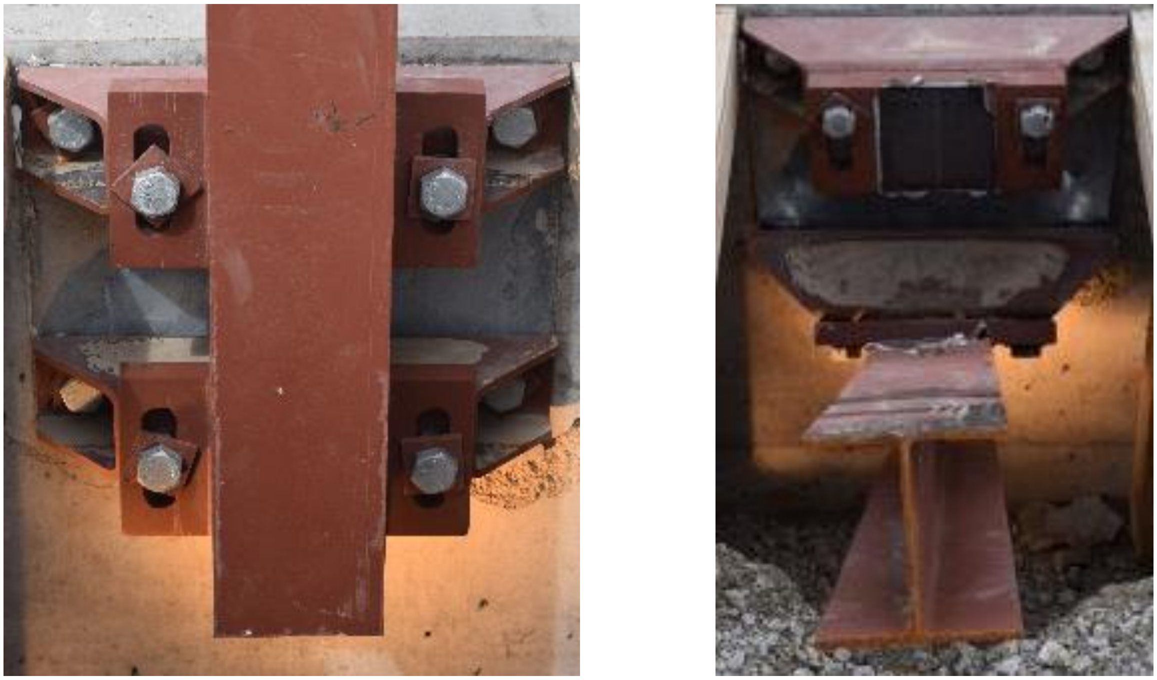
Figure 81. (a) Critical Slab Deck without Asphalt Overlay for Component Testing with Center of Gravity of 2270P Pickup Truck (b) Bogie Impact Height for Dynamic Component Tests

7.3 Dynamic Component Tests

Accelerometers were used and mounted to the center of gravity of the bogie to determine estimated impact forces. The accelerometer data was used to create force vs. deflection and energy vs. deflection graphs, which are shown for each component test. For all component tests, the post-to-deck attachments were side-mounted to the concrete box beam girder utilizing 1-in. (25-mm) diameter anchor rods as the anchorage system. The tension anchor embedment length was 32 in. (813 mm) for test nos. ILOH4-1 through ILOH4-5. In test no. ILOH4-6 the tension anchor embedment length was reduced from 32 in. (813 mm) to 24 in. (610 mm), and later, the embedment length was further reduced to 15 in (381 mm) in test no. ILOH4-7. The seven component tests varied on stirrup spacing, which depended on post location along the box-beam girder.

7.3.1 Test No. ILOH4-1

The first bogie test, test no. ILOH4-1, was performed on a 1¼-in. (32-mm) thick two-plate attachment with two HSS 5-in. x 4-in. x ⅜-in. (HSS 127-mm x 102-mm x 9.5-mm) longitudinal tube spacers. The simulated box-girder stirrups were spaced at 9 in. (229 mm). Upon bogie impact, the W6x15 (W150x22.5) post briefly rotated backward until weld failure occurred at the interface between the vertical mounting plates and the front flange of the steel post, thus resulting in significant post rotation with complete override by the bogie. Pre-test and post-test photographs are shown in Figure 82. Inspection of the post assembly and deck attachment after the test revealed that the post had minimal bending deformation prior to tensile weld rupture of the top plate attachment. The results showed a peak force of 26.9 kips (119.7 kN) over the first few inches of deflection, as shown in Figure 83.



(a) (b)

Figure 82. Test No. ILOH4-1 Photographs - (a) Pre-Test, and (b) Post-Test

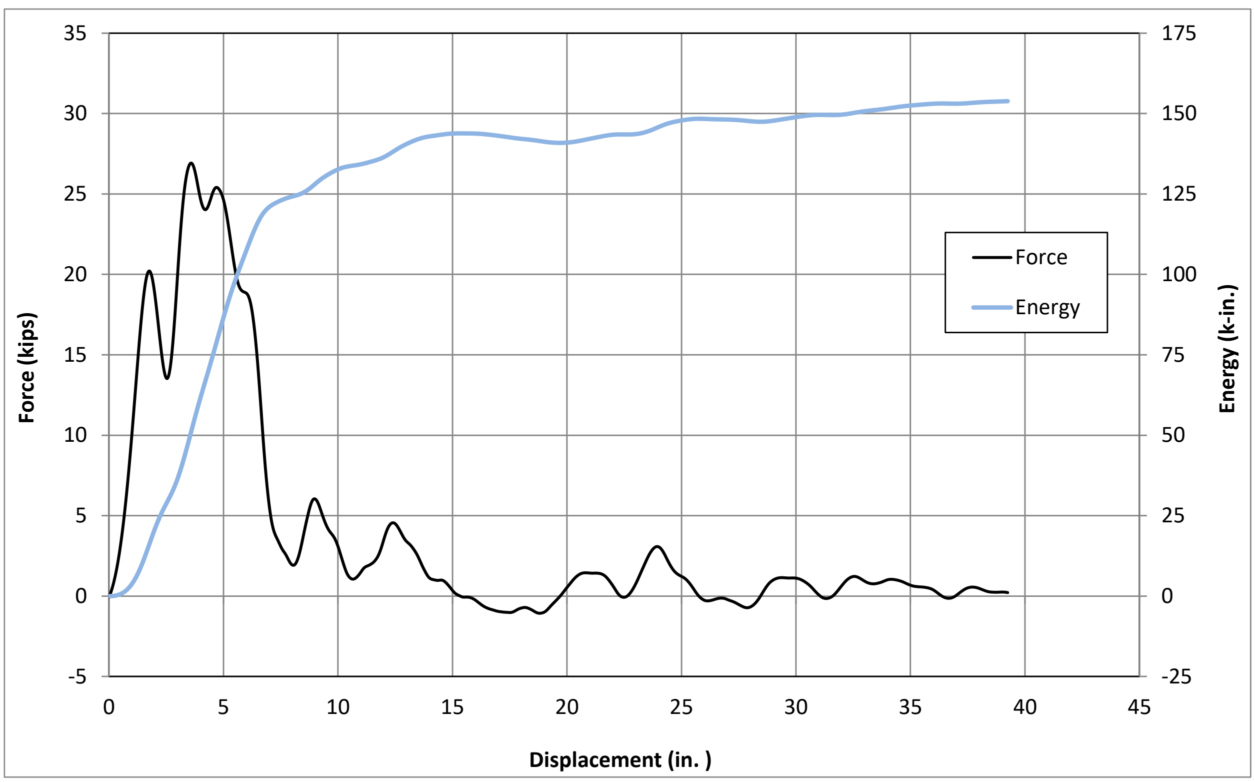


Figure 83. Force vs. Deflection and Energy vs. Deflection Graph, Test No. ILOH4-1

7.3.2 Test No. ILOH4-2

Two gussets were added at the tensile anchor rod height to reinforce the plate-to-flange welded connection and prevent brittle weld failure. The stirrup spacing was 4½ in. (114 mm). Upon bogie impact, the W6x15 (W150x22.5) post 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. Pre-test and post-test photographs are shown in Figure 84. 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. The test results showed a peak impact load very near to the results observed in the first test, at approximately 25.7 kips (114.3 kN), as depicted in Figure 85.

(a) (b)

Figure 84. Test No. ILOH4-2 Photographs - (a) Pre-Test, and (b) Post-Test

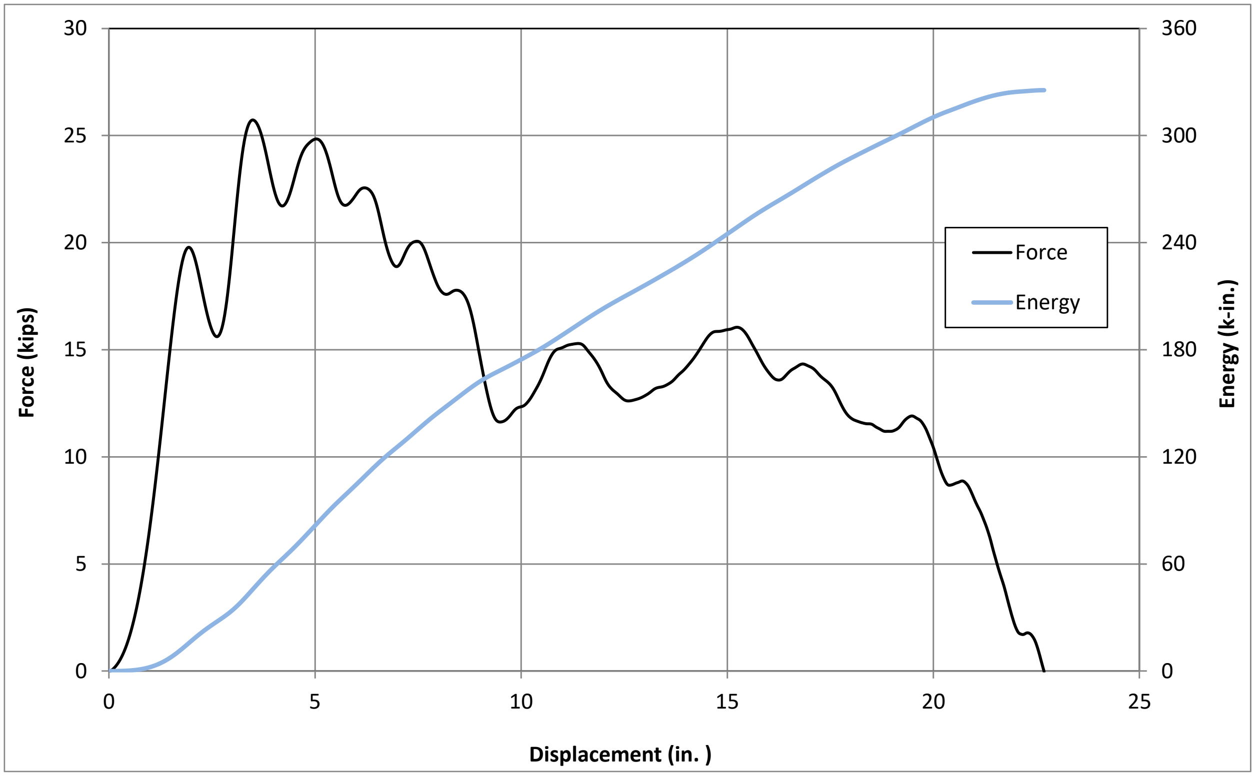


Figure 85. Force vs. Deflection and Energy vs. Deflection Graph, Test No. ILOH4-2

7.3.3 Test No. ILOH4-3

The two 1¼-in. (32-mm) thick attachment plates were replaced with one plate in order to provide continuous front flange support and prevent localized post deformations between tension and compression anchor rods. The thickness of the HSS 5-in. x 4-in. x ⅜-in. (HSS 127-mm x 102-mm x 9.5-mm) tube spacers was increased to ½ in. (12.2 mm) to prevent bowing outward. The current stirrup spacing was 4½ in. (114 mm). Moreover, two gussets were added at the compression anchor rods to prevent localized web buckling. Pre-test and post-test photographs are shown in Figure 86. Upon bogie impact, the horizontal fillet welds for the top and bottom gussets as well as the vertical fillet welds between the attachment plate and the front flange of the post sheared off, and the post rotated backward and came to rest along the tarmac. The post was not bent or deformed as the welds completely failed, and the post detached and rotated backward as the bogie overrode it. 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. Force vs. deflection and energy vs. deflection curves were generated from the accelerometer data, as shown in Figure 87. The peak impact load was higher than the previous two tests being approximately 36.9 kips (164.1 kN).

(a) (b)

Figure 86. Test No. ILOH4-3 Photographs - (a) Pre-Test, and (b) Post-Test

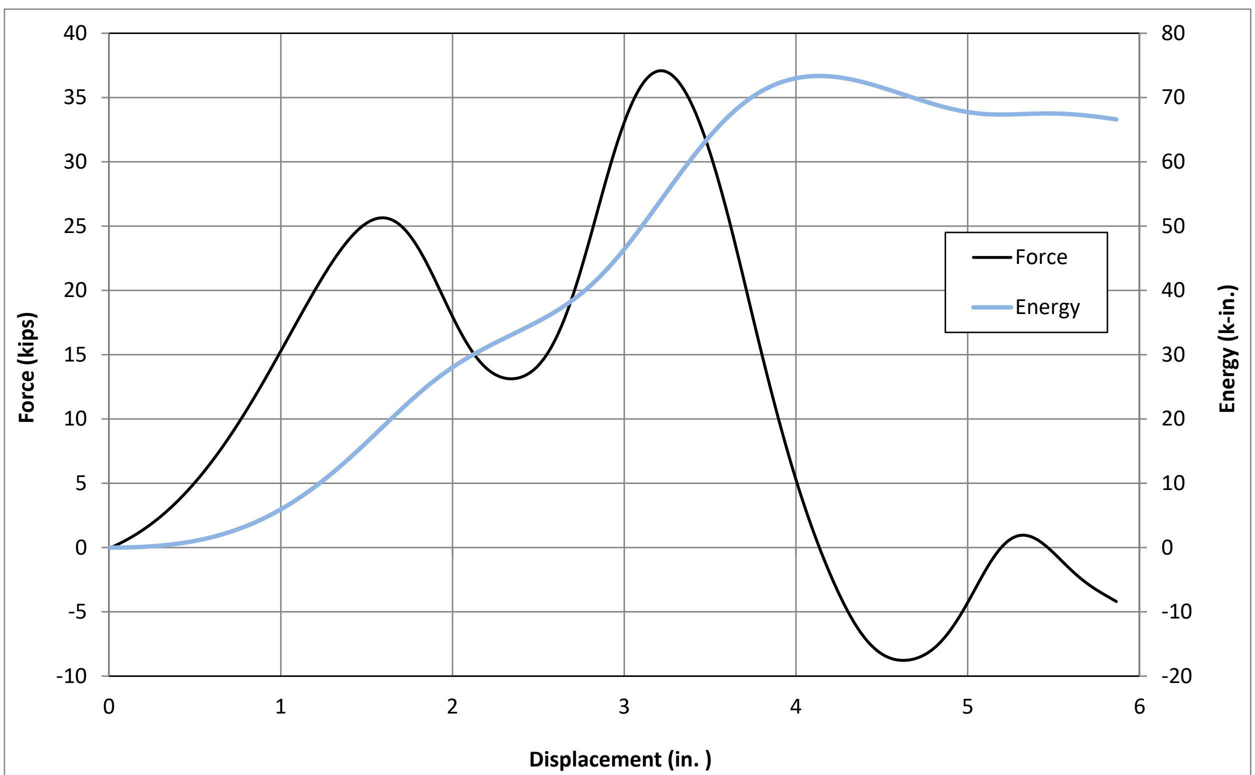


Figure 87. Force vs. Deflection and Energy vs. Deflection Graph, Test No. ILOH4-3

7.3.4 Test No. ILOH4-4

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 with 34½-in. (876-mm) rod embedment depth at 4½-in. (114-mm) stirrup spacing. Pre-test and post-test photographs of test no. ILOH4-4 are shown in Figure 88. Upon bogie impact, the post tore at a location starting right above the 6-in. (152-mm) long, horizontal weld between the front flange of the post and the top of the plate attachment. It also diagonally tore upward along the post web until ending at the back flange. Buckling of the back flange was observed right above the tensile gussets. It was assumed that the post tore at a stress concentration condition due to an overload condition. The test results showed a peak loading of 39.6 kips (176.1 kN) and an average loading of 20 kips (89 kN) through rupture at 17 in. (432 mm) of deflection, as depicted in Figure 89.

(a)

(b)

Figure 88. Test No. ILOH4-4 Photographs - (a) Pre-Test, and (b) Post-Test

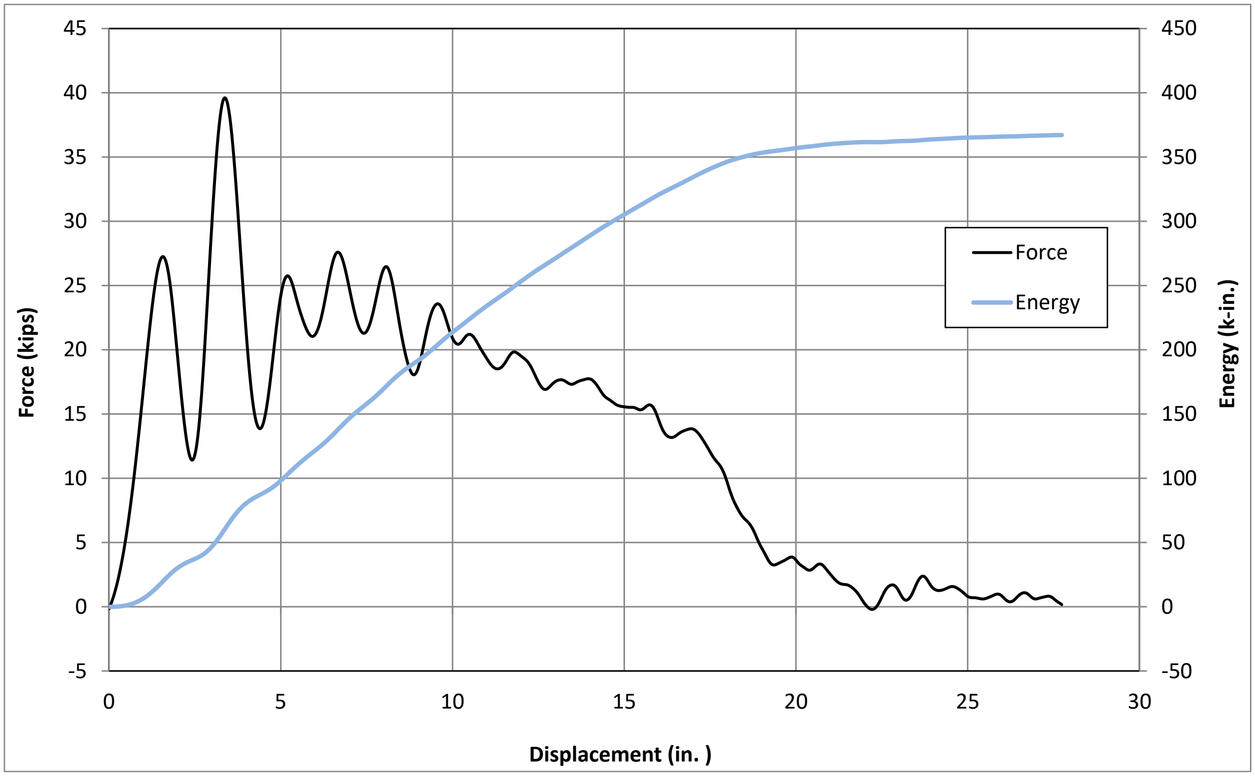


Figure 89. Force vs. Deflection and Energy vs. Deflection Graph, Test No. ILOH4-4

7.3.5 Test No. ILOH4-5

The only modification with respect to test no. ILOH4-4 was a stirrup spacing of 9 in. (229 mm). Pre-test and post-test photographs are shown in Figure 90. In this component test, a plastic hinge of the post was developed starting in the front flange above the top edge of the 1-in. (25-mm) thick, vertical attachment plate and extending through the back flange at the height of the tension gusset plates. No other post deformations were observed. The test results showed a peak loading of 37.6 kips (167.3 kN) and an average loading of 21 kips (93 kN) over a 10-in. (254-mm) deflection, as shown in Figure 91.

(a)

(b)

Figure 90. Test No. ILOH4-5 Photographs - (a) Pre-Test, and (b) Post-Test

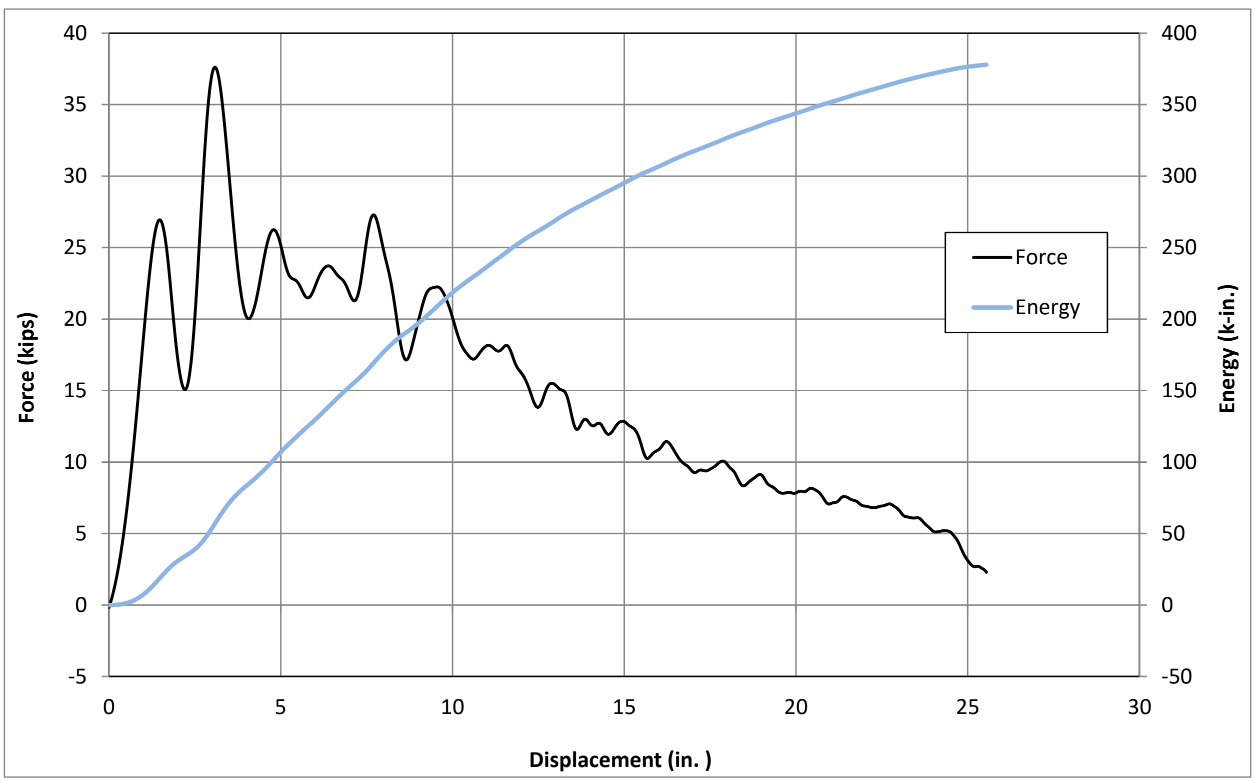


Figure 91. Force vs. Deflection and Energy vs. Deflection Graph, Test No. ILOH4-5

7.3.6 Test No. ILOH4-6

The tension rod embedment length was reduced from 32 in. (813 mm) to 24 in. (610 mm). Pre-test and post-test photographs are shown in Figure 92. Upon bogie impact, a plastic hinge developed starting in the front flange above the top edge of the 1-in. (25-mm) thick, vertical attachment plate extending to the back flange at the height of the tension gusset plates. No other plastic deformation was observed in the post nor the post-to-deck attachment hardware. The test results showed a peak loading of 33.9 kips (150.8 kN) over the first 5 in. (127 mm) of lateral deflection, as shown in Figure 93.

(a) (b)

Figure 92. Test No. ILOH4-6 Photographs - (a) pre-test, (b) post-test

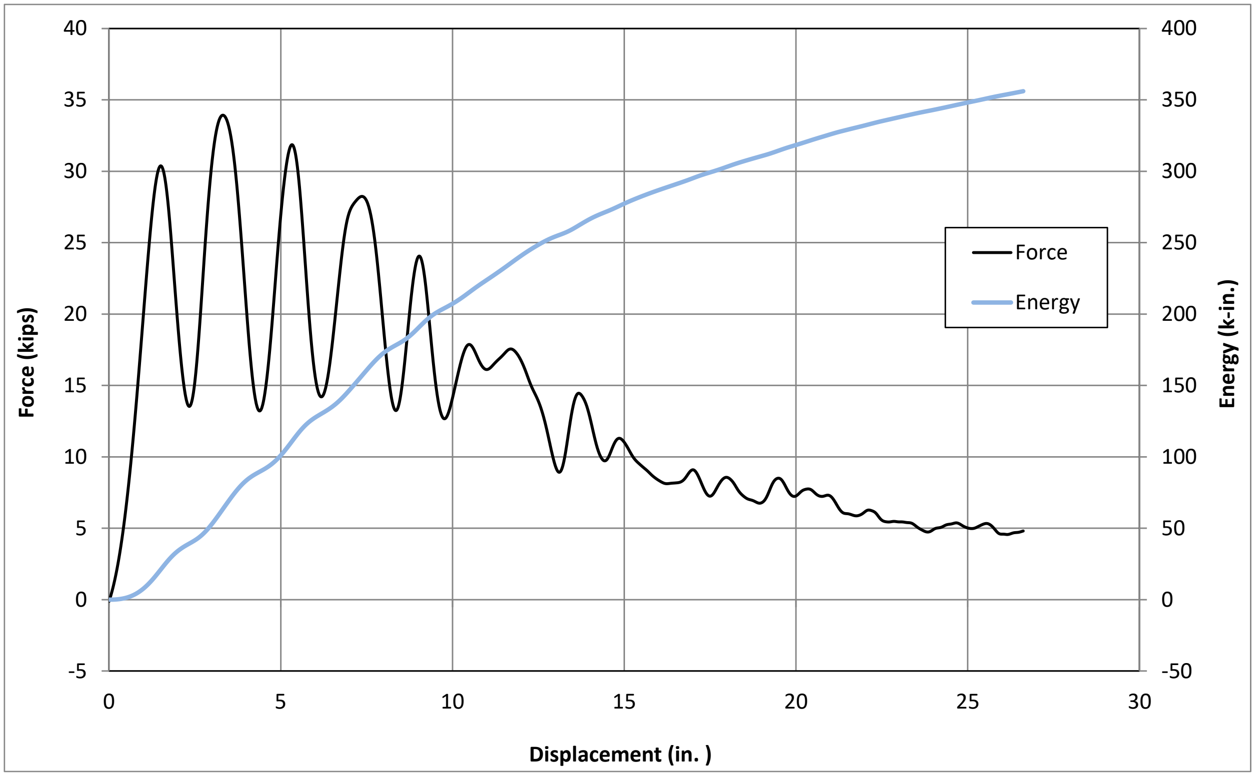


Figure 93. Force vs. Deflection and Energy vs. Deflection Graph, Test No. ILOH4-6

7.3.7 Test No. ILOH4-7

The tension rod embedment length was reduced from 24 in. (610 mm) to 15 in. (381 mm). The thickness of the vertical attachment plate was reduced from 1 in. (25 mm) to ¾ in. (19 mm). Pre-test and post-test photographs are shown in Figure 94. Upon bogie impact, the post developed a plastic hinge starting at the front flange near the top edge of the vertical plate attachment extending to the back flange at the height of the tension gusset plates. The ¾ in. (19 mm) thick, vertical plate attachment was slightly bent at the height of the tension anchor rods. The test results showed a peak loading of 29.2 kips (129.9 kN) through the first 5 in. (117 mm) of lateral deflection, as shown in Figure 95.

(a) (b)

Figure 94. ILOH4-7 Photographs - (a) pre-test, (b) post-test

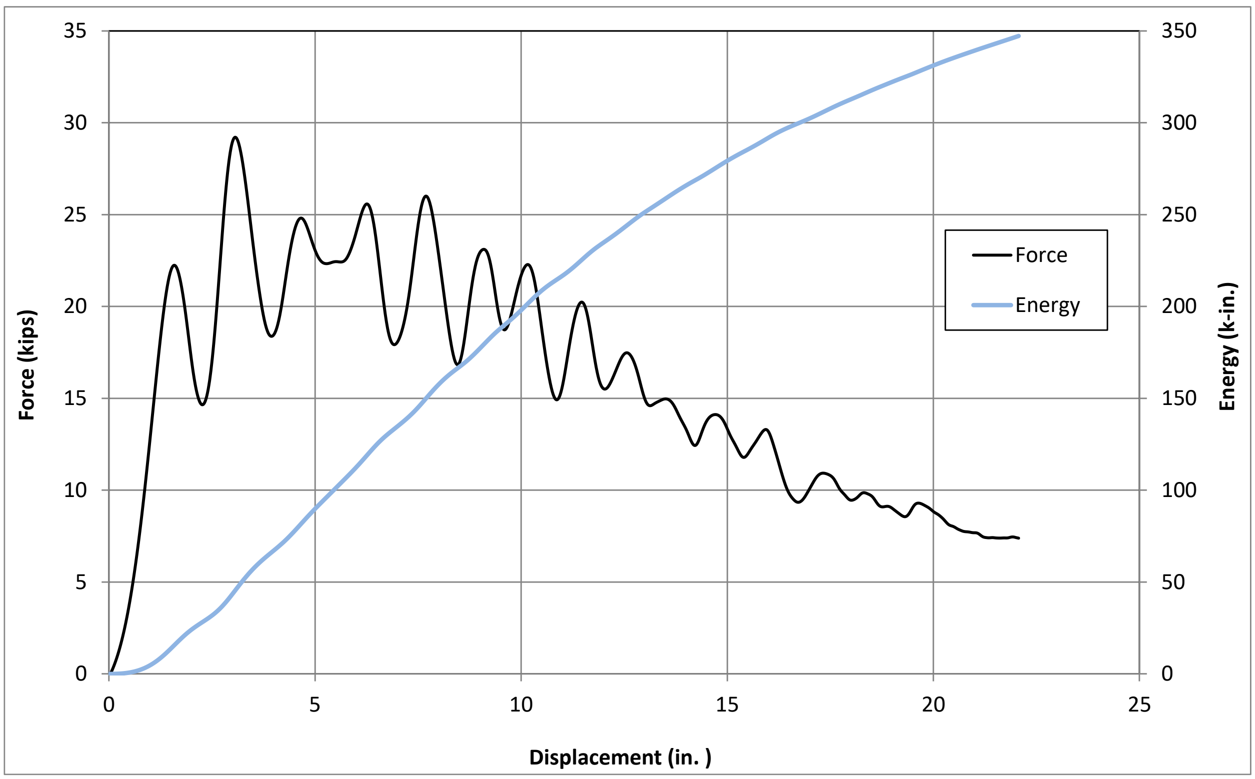


Figure 95. Force vs. Deflection and Energy vs. Deflection Graph, Test No. ILOH4-7

7.3.8 Test Results Summary and Discussion

The lateral forces observed during the test nos. ILOH4-4 through ILOH4-6 were quite uniform when the post remained attached to the welded bracket (i.e., no weld connection failure). Test nos. ILOH4-4 to ILOH4-6 performed adequately without post-to-deck connection hardware deformations, while developing plastic hinges in the posts above the tension anchor rod height, specifically near the top post stiffeners. These post-to-deck prototypes utilized top and bottom post stiffeners that were welded between the post and vertical mounting plate. Therefore, the final configuration of the post assemblies was determined to require bottom and top post stiffeners. Moreover, it was determined that the post-to-deck attachment concept consisted of the 1-in. (25-mm) thick singular plate attachment with top and bottom gusset plates. Further, the two HSS 5-in. x 4-in. x ½-in. (HSS 127-mm x 102-mm x 12.2-mm) deck spacers were found to not impart excessive loading that would critically damage the sidewall of the concrete box-beam girder. Thus, the deck anchorage in the tension region would remain the same, which consisted of 1-in. (25-mm) ASTM F1554 Grade 105 all-thread anchor rods with ASTM A563DH heavy hex huts and coupling nuts. The vertical deck plate thickness was increased from ⅛ in. (3.2 mm) to $^{3}/_{16}$ in. (4.8 mm). The deck anchorage in the compression region was configured with 1-in. (25-mm) diameter ASTM A449 anchor bolts, coupling nuts, and a 3-in. (76-mm) square washer plate. Therefore, these post-to-deck components were implemented into the final configuration for the MASH 2016 TL-4 bridge rail, as shown in Figure 96. The average forces at a determined lateral post deflection and specified impact height were derived from the force vs. deflection and energy vs. deflection graphs for each component test. The average forces at lateral deflections of 5 in. (127 mm), 10 in. (254 mm), 15 in. (381 mm), and 20 in. (508 mm) are shown in Table 63.

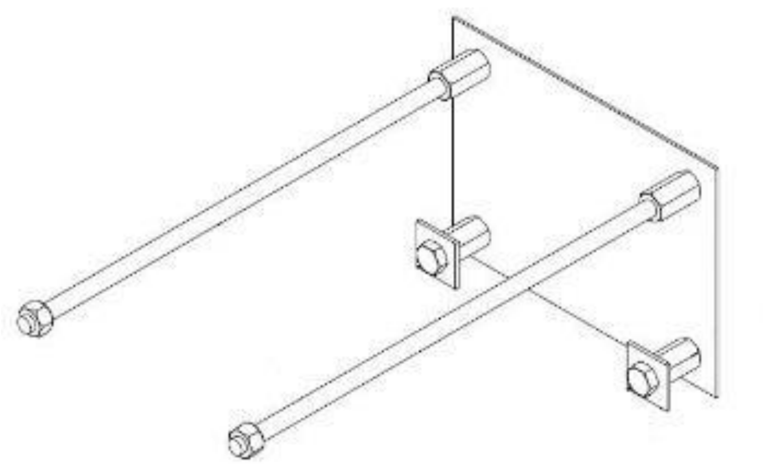


Figure 96. Proposed Deck Anchorage Plate with Embedded Hardware

Table 63. Bogie Test Results

Test No.	Total Dissipated Energy, (k-in.)	Component Failure Mode	Average Impact Force (kips)			
			5-in. Lateral Deflection	10-in. Lateral Deflection	15-in. Lateral Deflection	20-in. Lateral Deflection
ILOH4-1	159	Top Plate Welding	18	14	7	7
ILOH4-2	329	Post Buckling	17	18	17	16
ILOH4-3	77	Welding	14	-	-	-
ILOH4-4	367	Post Rupture	20	21	20	17
ILOH4-5	378	N/A	21	22	20	17
ILOH4-6	356	N/A	20	21	19	16
ILOH4-7	347	N/A	18	20	19	16

7.4 Further Analysis of Bridge Railing Capacity

Based on the review of previous successful TL-4 crash-tested, beam-and-post, bridge rails, a maximum lateral deflection between 10 in. (305 mm) to 12 in. (381 mm) was anticipated for the new bridge rail. Therefore, it was necessary to determine the average impact force through 10-in. (305-mm) and 12-in. (381-mm) lateral deflections for test nos. ILOH4-4 through ILOH4-6, which adequately developed plastic hinges near the tension anchor rods without visible deformations of the post-to-deck connection hardware. The average impact forces at a 10-in. (254-mm) and a 12-in. (305-mm) lateral deflection were obtained for test nos. ILOH4-4 through ILOH4-6 and are shown in Table 64. The average impact force was determined to be 20.4 kips (90.7 kN).

Table 64. Average Force at a Lateral Deflection equal to 10 in. (254 mm) and 12 in. (305 mm)

Component Test No.	F_{AVE} @ 10 in. Lateral Deflection, (kips)	F_{AVE} @ 12 in. Lateral Deflection, (kips)
ILOH 4-4	21.0	20.6
ILOH 4-5	22.0	20.8
ILOH 4-6	21.0	19.8
Average	21.3	20.4

Note that the yield strength of the posts that were used in the component testing was 56 ksi (386 MPa) instead of 50 ksi (345 MPa). These average impact forces at 10-in. (305-mm) and 12-in. (381-mm) lateral deflections were modified to obtain an average impact force for a yield strength of 50 ksi (345 MPa), which was used in design calculations for the new bridge rail using Equation 42.

For $\mathrm{F_{AVE}}\ (\mathrm{F_Y} = 56\ \mathrm{ksi}) = 21.3$ kips at 10-in. lateral deflection:

$$\mathrm{F_{AVE\ MODIFIED}} = \mathrm{F}_{AVE} * \frac{F_{Y\ DESIGN}}{F_{Y\ ACTUAL}} \tag{42}$$

$$\mathrm{F_{AVE\ MODIFIED}}\ (\mathrm{F_Y} = 50\ \mathrm{ksi}) = 21.3\ \mathrm{kips} * \frac{50\ \mathrm{ksi}}{56\ \mathrm{ksi}} = 19.0\ \mathrm{kips}$$

$$\mathrm{F_{AVE\ MODIFIED}} = 19.0\ \text{kips at 10-in. lateral deflection}$$

For F_{AVE} ($F_Y = 56$ ksi) $= 20.4$ kips at a 12-in. lateral deflection:

$$F_{AVE\ MODIFIED} = F_{AVE} * \frac{F_{Y\ DESIGN}}{F_{Y\ ACTUAL}} \tag{42}$$

$$F_{AVE\ MODIFIED}\ (F_Y = 50\ \text{ksi}) = 20.4\ \text{kips} * \frac{50\ \text{ksi}}{56\ \text{ksi}} = 18.2\ \text{kips}$$

$$F_{AVE\ MODIFIED} = 18.2\ \text{kips at } 12\text{-- in. lateral deflection}$$

The modified impact forces resisted by a post with a yield strength of 50 ksi (345 MPa) were calculated as 19.0 kips (84.5 kN) and 18.2 kips (81.0 kN) for 10-in. (305-mm) and 12-in. (381-mm) lateral deflections, respectively. These modified impact forces were compared to design calculations for the lateral impact force sustained by a post to validate the application of a DMF equal to 1.5 in design calculations of the final bridge rail configuration.

The lateral impact force sustained by a post was calculated for the validation of the application of a DMF equal to 1.5, as shown in Equation 43. From observations, the moment arm between the impact load height and the location of the plastic hinges for these tests was determined to be approximately 27¾ in. (705 mm), as shown in Figure 97. The estimated impact force sustained by a 50-ksi (345-MPa) steel post following bridge rail design calculations was calculated as 19.4 kips (86.3 kN). The modified impact forces resisted by a post equal to 19.0 kips (84.5 kN) and 18.2 kips (81.0 kN) for 10-in. (305-mm) and 12-in. (381-mm) lateral deflections were compared with the 19.4 kips (86.3 kN) to validate the application of a DMF equal to 1.5 in design calculations of the final bridge rail configuration.

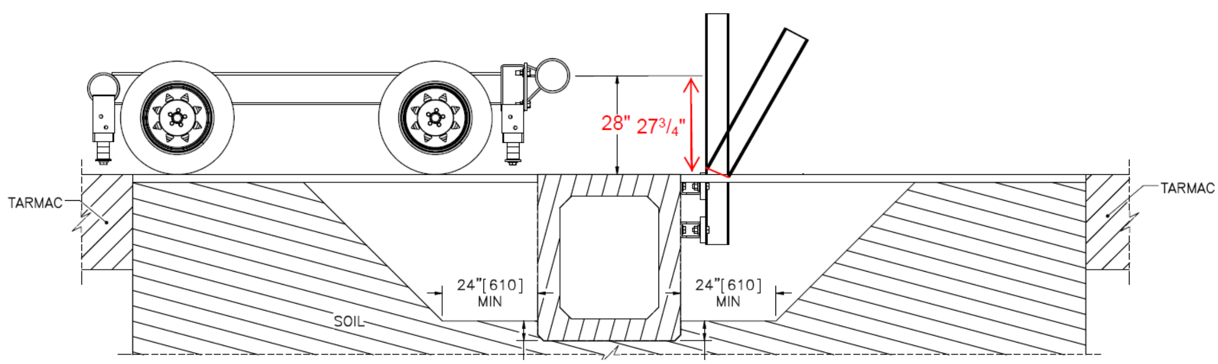


Figure 97. Moment Arm between Bogie Impact Height and Plastic Hinge Location

$$\mathrm{P_{POST}} = \frac{\mathrm{F_Y} * \mathrm{Z_X}}{\mathrm{d}} \tag{43}$$

where:

P_{POST} = lateral impact force sustained by a post (kips),
F_Y = yield strength of the steel post (ksi);
Z_X = plastic section modulus of steel post (in.^{3}); and
d = moment arm between loading height and plastic hinge (in.).

$$\mathrm{P}_{POST} = \frac{50\ \mathrm{ksi} * 10.8\ \mathrm{in.}^3}{27.75\ \mathrm{in}} = 19.4\ \mathrm{kips}$$

$$\mathrm{F_{AVE\ MODIFIED}} = 19.0\ \mathrm{kips\ at\ 10\ in.\ lateral\ deflection}$$

$$\frac{19.0\ \mathrm{kips} - 19.4\ \mathrm{kips}}{19.4\ \mathrm{kips}} = -2.1\%$$

$$\mathrm{F_{AVE\ MODIFIED}} = 18.2\ \mathrm{kips\ at\ 12\ in.\ lateral\ deflection}$$

$$\frac{18.2\ \mathrm{kips} - 19.4\ \mathrm{kips}}{19.4\ \mathrm{kips}} = -6.2\%$$

These comparisons indicated that strain rates did not increase the lateral impact resistance of the posts after updating for the actual yield from component testing. Therefore, it was determined to use a DMF equal to 1.0 for calculating barrier capacity. Using a plastic collapse mechanism, a lateral barrier resistance of 81.5 kips (362.5 kN) at the design impact load height was initially calculated for the final configuration of the new bridge rail using a DMF equal to 1.5. When using a DMF equal to 1.0 for the final capacity and for full-scale crash testing, a lateral barrier resistance of the bridge rail was reduced to 66.7 kips (296.7 kN) at the design impact load height, which resulted in a lower capacity than the 80-kip (356 kN) design loading.

Thus, the research team identified and reviewed successfully crash-tested, beam-and-post, bridge rails that met either AASHTO PL-2 [4] and NCHRP Report 350 [5] but would not meet current design impact loading based on the plastic collapse mechanism. For these systems, the lateral barrier resistance was calculated and compared to prior design impact loading and impact severity.

The NCHRP Report 350 TL-4 STTR bridge rail [18-19] consisted of a TS 8-in. x 3-in. x $^3/_{16}$-in. (TS 203-mm x 76-mm x 4.8-mm) ASTM A500 Grade B steel top rail and a 10-gauge (3.43-mm) AASHTO M180 thrie-beam rail (Grade 50) supported by ASTM A36 W6x15 (W152x22.3) wide-flange structural steel posts with a total rail height of 36 in. (914 mm), as shown in Figure 98. The full-scale crash test with the NCHRP Report 350 TL-4 SUT was successful, where the maximum dynamic deflection was equal to 8.0 in. (203 mm). Based on an inelastic plastic collapse mechanism analysis, the lateral barrier resistance for the STTR bridge rail was 41 kips (182 kN) at the NCHRP Report 350 design impact load height equal to 32 in. (813 mm) above the deck surface. The AASHTO LRFD lateral design impact load associated with the NCHRP Report 350 TL-4 SUT and pickup truck has been previously shown as 54 kips (240 kN).

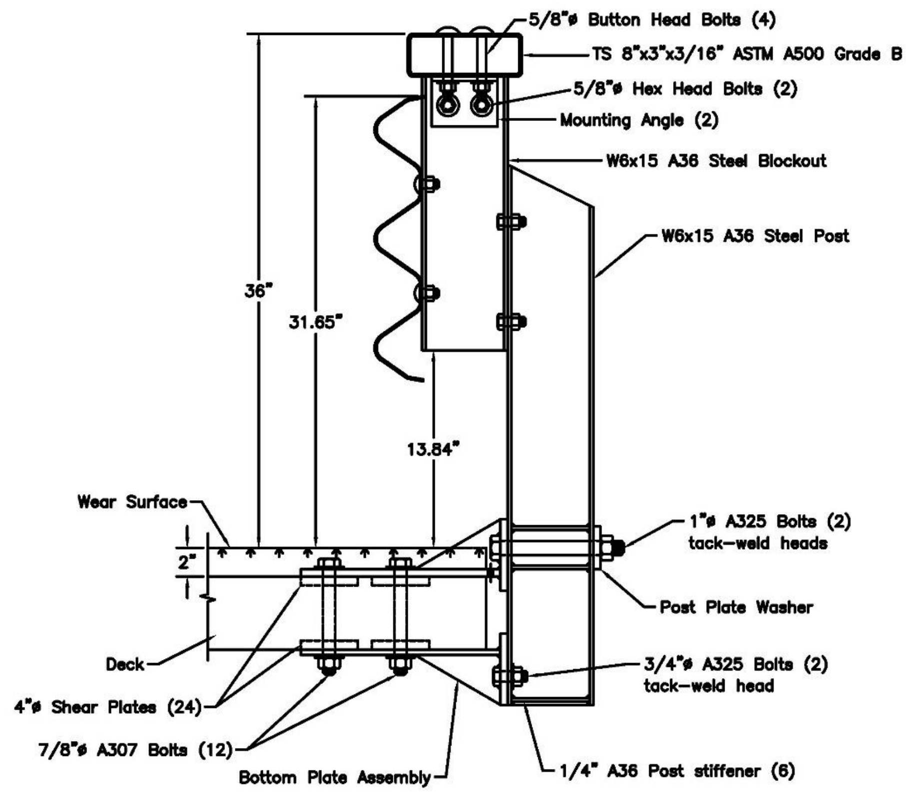


Figure 98. NCHRP Report 350 TL-4 STTR Bridge Rail for Transverse, Glulam Timber Decks [18]

The AASHTO PL-2 TBC-8000 bridge rail [53] consisted on a ASTM A36 C8-in. x 11.5-in. (C200-mm x 17-mm) steel channel section with the web bolted to the top of a W6-in. x 15-in. (W152-mm x 22.3-mm) spacer blocks at post locations and with a 10-gauge (3.43-mm) AASHTO M180 thrie-beam rail bolted to its front flange. The posts consisted of ASTM A36 W6x15 (W152x22.5) steel sections bolted to the side of the bridge deck, as shown in Figure 99. The total height of the bridge rail was 33¼ in. (845 mm). The full-scale crash test with a SUT was successful, where the maximum dynamic deflection was equal to 9.0 in. (229 mm). Based on an inelastic plastic collapse mechanism analysis, the lateral barrier resistance for the TBC-80000 bridge rail was 46 kips (205 kN) at a height of 17 in. (432 mm) above the deck surface. The lateral design impact load for the AASHTO PL-2 SUT was 80 kips (356 kN) at a height of 17 in. (432 mm).

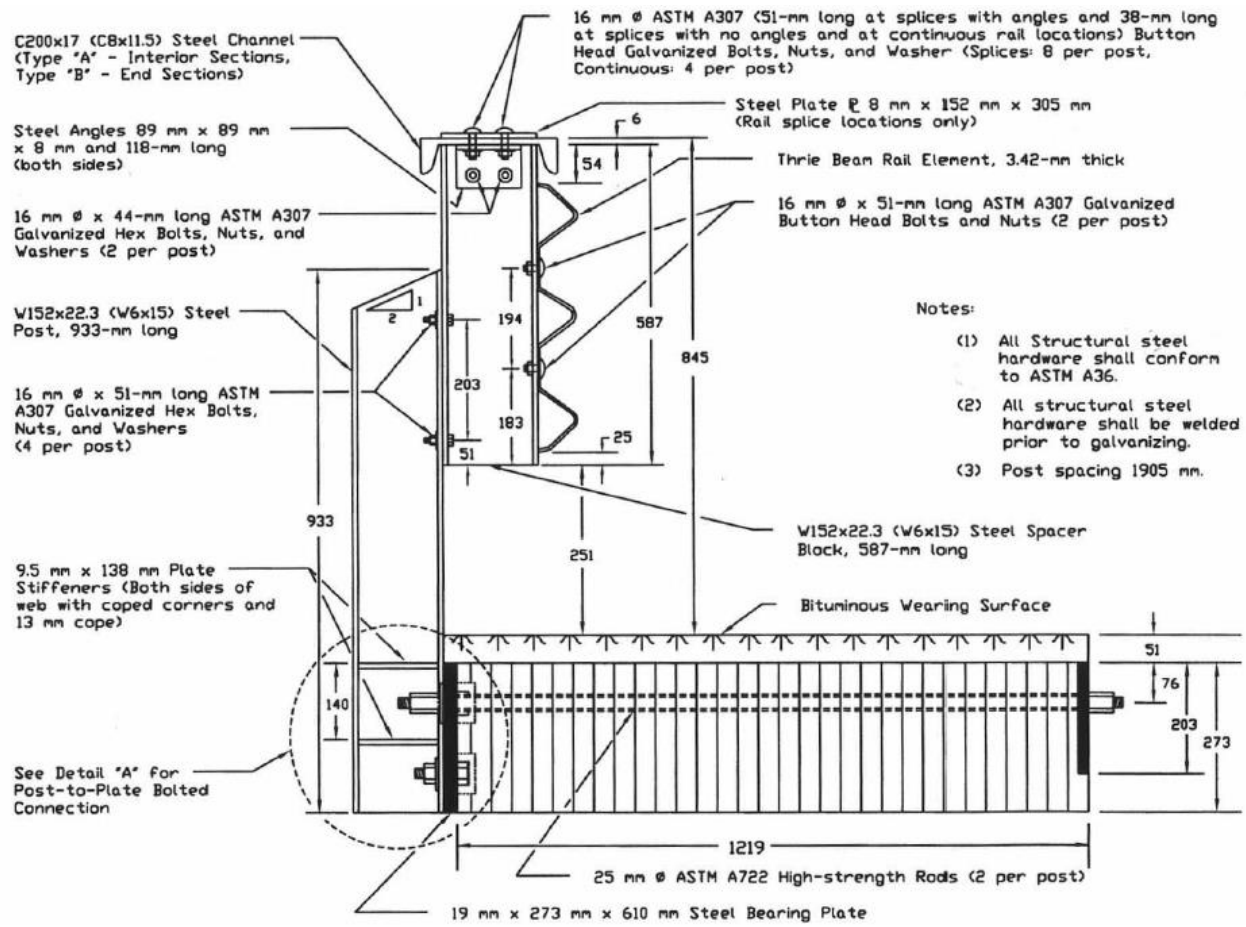


Figure 99. AASHTO PL-2 TBC-8000 Bridge Rail for Longitudinal, Glulam Timber Decks [53]

After analyses of the two successfully crash-tested, bridge rails, the research team and representatives from the Illinois and Ohio DOTs decided to not modify the final prototype bridge rail to again provide an 80-kip (356-kN) lateral barrier capacity but rather proceed with MASH 2016 TL-4 crash testing. Using a DMF equal to 1.0 for the posts, the lateral barrier resistance was reduced. For SUT impact scenarios, the lateral barrier resistance when considering all the three rails was 66.7 kips (296.7 kN) for a five-span collapse. For pickup truck impact scenarios, lateral barrier resistances were 55.9 kips (248.7 kN) for a three-span collapse when considering the lower two rails and 87.7 kips (390.1 kN) for a five-span collapse when considering all the three rails.

For the top rail, and at post locations only, the final prototype bridge rail utilized post-to-rail connections configured with four ⅞-in. (22-mm) diameter round bolt holes versus two 1-in. (25-mm) diameter round bolt holes shown in the original IL/OH Prototype Bridge Rail. The reduced final plastic section modulus of the HSS 12-in. x 4-in. x ¼-in. (HSS 304.8-mm x 101.6-mm x 6.4-mm) top rail section was reduced to 24.5 in.^{3} (401,483 mm^3), as shown in Figure 100 and Equation 21. Therefore, for SUT impact scenarios, the lateral barrier resistance when considering all the three rails was 65.8 kips (292.7 kN) for a five-span collapse. For pickup truck impact scenarios, lateral barrier resistances were 55.9 kips (248.7 kN) for a three-span collapse when considering the lower two rails and 86.6 kips (385.2 kN) for a five-span collapse when considering all the three rails. However, it should be noted that the reduced cross section only occurs at post locations.

The calculations for the final reduced plastic section moduli for the top rail with the welded plate mounting bracket are provided below. The configuration utilized four ¾-in. (19-mm) diameter bolts. The top rail used ⅞-in. (22-mm) diameter by 1⅜-in. (35-mm) long slotted holes.

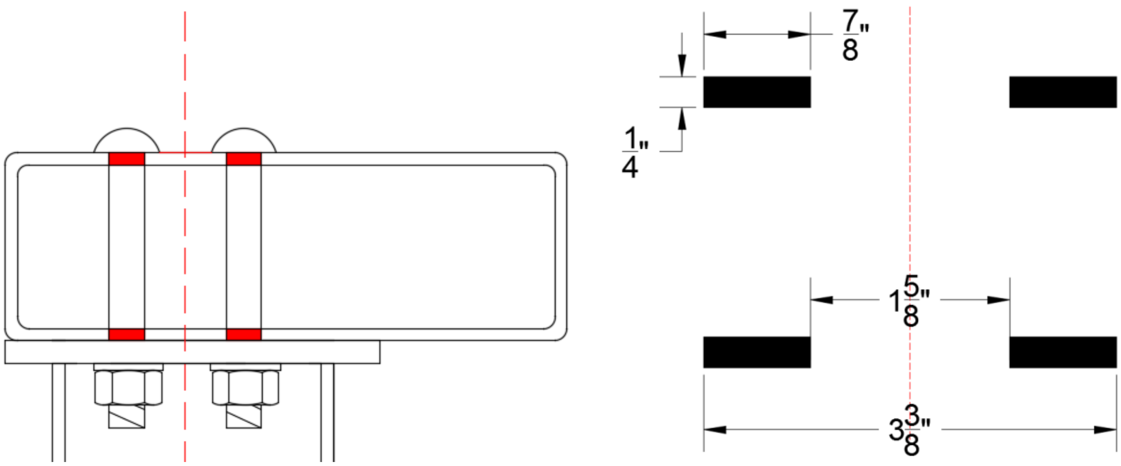


Figure 100. Schematic of Top Rail Bolt Configuration

$$Z_{X\ \text{RED TOP RAIL}} = (2)\,\frac{b}{4}\left(d^2 - d_1^2\right) \tag{21}$$

$$Z_{X\ \text{RED TOP RAIL}} = (2)\,\frac{0.25\text{ in.}}{4}\left(3.375\text{ in.}^2 - 1.625\text{ in.}^2\right) = 1.1\text{ in.}^3$$

$$Z_{\text{REDUCED TOP RAIL}} = 25.6\ in.^3 - 1.1\text{ in.}^3 = 24.5\ in.^3$$

8 SURROGATE CONCRETE BRIDGE DECK

The post-to-deck attachment hardware, tension and compression anchorage rods, and critical bridge deck configuration were tested and evaluated during the dynamic component testing program. The test results demonstrated that the critical box-beam girder did not have excessive damage that would degrade barrier performance nor affect its structural integrity. During post rebound in the dynamic component testing program, minor concrete spalling was observed at the bottom of the vertical deck plate near the lower two attachment bolts. Only minor modifications were incorporated into the anchorage hardware to reduce surface damage to the side wall of the concrete box-beam girder surrounding the embedded, vertical deck plate within the compression region. Therefore, only the bridge railing would be evaluated with the full-scale crash testing program, and all bridge deck configurations would be acceptable for use with the MASH 2016 TL-4 beam-and-post bridge rail. The critical bridge deck configurations are depicted in Figure 101.

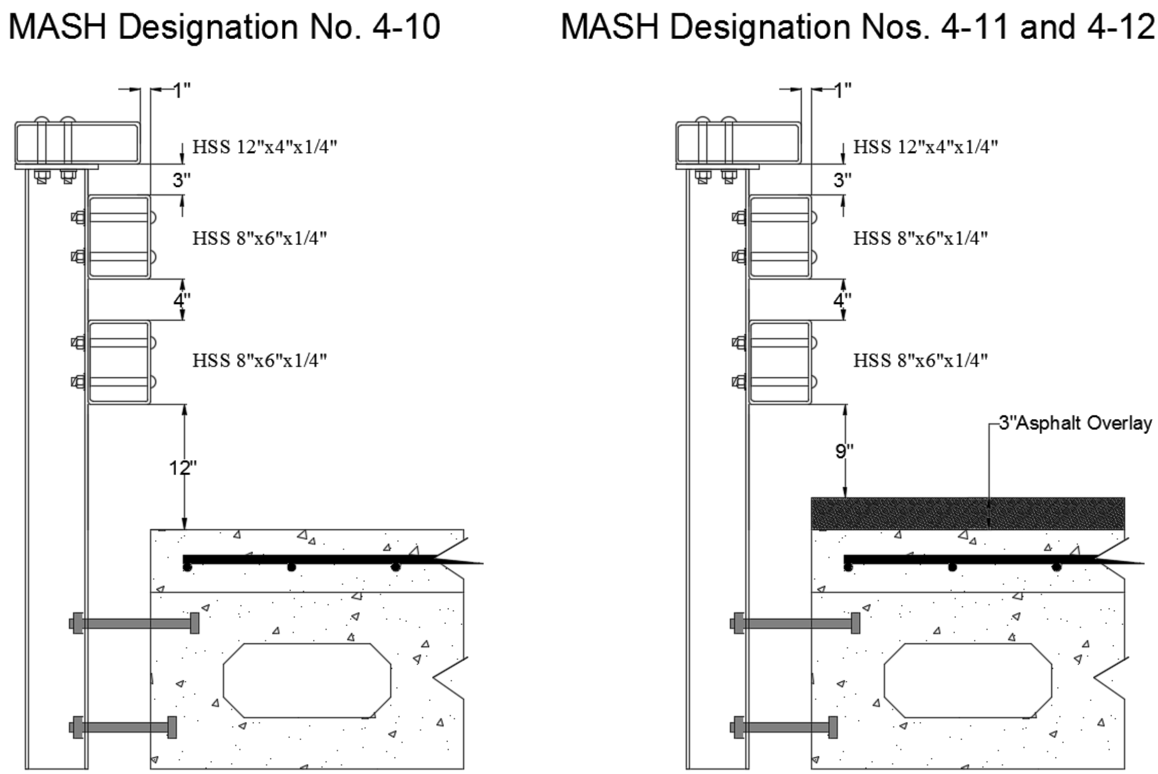


Figure 101. Critical Deck Configurations for MASH Crash Test Designation Nos. 4-10, 4-11, and 4-12

As noted in Section 3.2, and for MASH 2016 test designation no. 4-10, it was determined that critical concerns included wheel snag below the bottom rail and against the post as well as elevated occupant ridedown accelerations produced by the snag event. Therefore, the critical deck configuration needed to incorporate the largest vertical rail opening in combination with the strongest post (i.e., shortest moment arm from the tension anchor location in bridge decks utilized by Illinois and Ohio). This configuration had a 12-in. (305-mm) vertical rail opening and a 4-in. (102-mm) vertical distance between the top of the concrete deck and the centerline of the tension anchors.

For MASH 2016 test designation nos. 4-11 and 4-12, several critical concerns included vehicle override or rollover, excessive barrier deflections, and critical impact loading to the post-to-deck hardware and anchorage system for the SUT and pickup crash events. Since bridge deck loading was already evaluated in the dynamic component testing program, an emphasis was placed on selecting a critical configuration to evaluate vehicle override. Thus, the critical deck configuration must have the shortest overall rail height of 36 in. (914 mm) but the most flexible post due to the largest moment arm. The critical bridge deck consisted of 6-in. (152-mm) deep concrete slab placed on top of a box-beam and a future 3-in. (76-mm) asphalt overlay, resulting in a 36-in. (914-mm) overall top railing height. Further, a 12 in. (305 mm) distance would exist between the top of the asphalt overlay and the centerline of the tension anchors.

A surrogate bridge deck was then designed to allow for only one bridge deck to be constructed for testing and evaluating the three full-scale vehicle crash tests, as shown in Figure 102. This surrogate bridge deck had a depth of 26 in. (660 mm) to allow for the installation of both post-to-deck connections at their appropriate heights. One critical configuration simulated the concrete slab bridge deck for the MASH test designation no. 4-10 (see Figure 103), and another critical configuration simulated the box-beam bridge deck with a concrete slab asphalt overlay on top for MASH test designation nos. 4-11 and 4-12 (see Figure 104).

Figure 102. Surrogate Concrete Bridge Deck

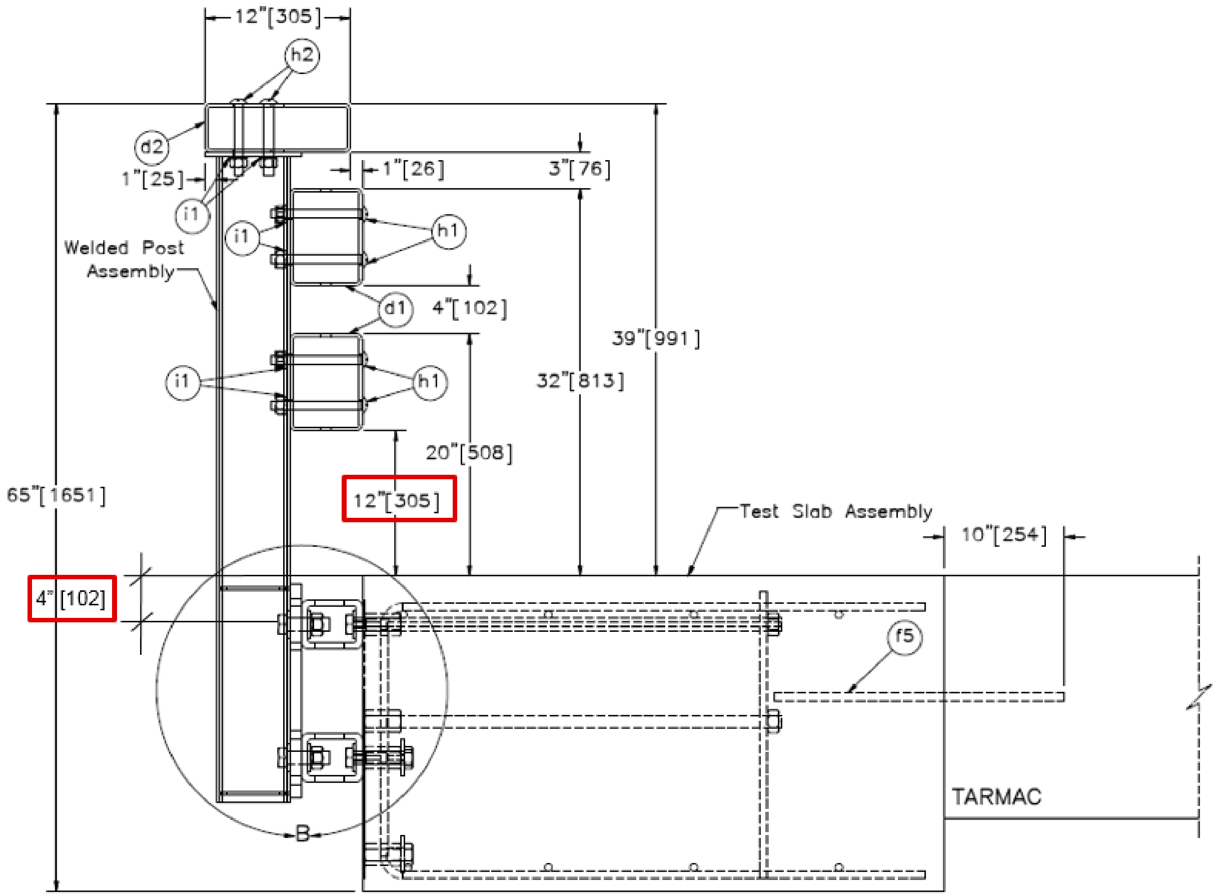


Figure 103. Surrogate Bridge Deck Profile View for MASH 2016 Test Designation No. 4-10

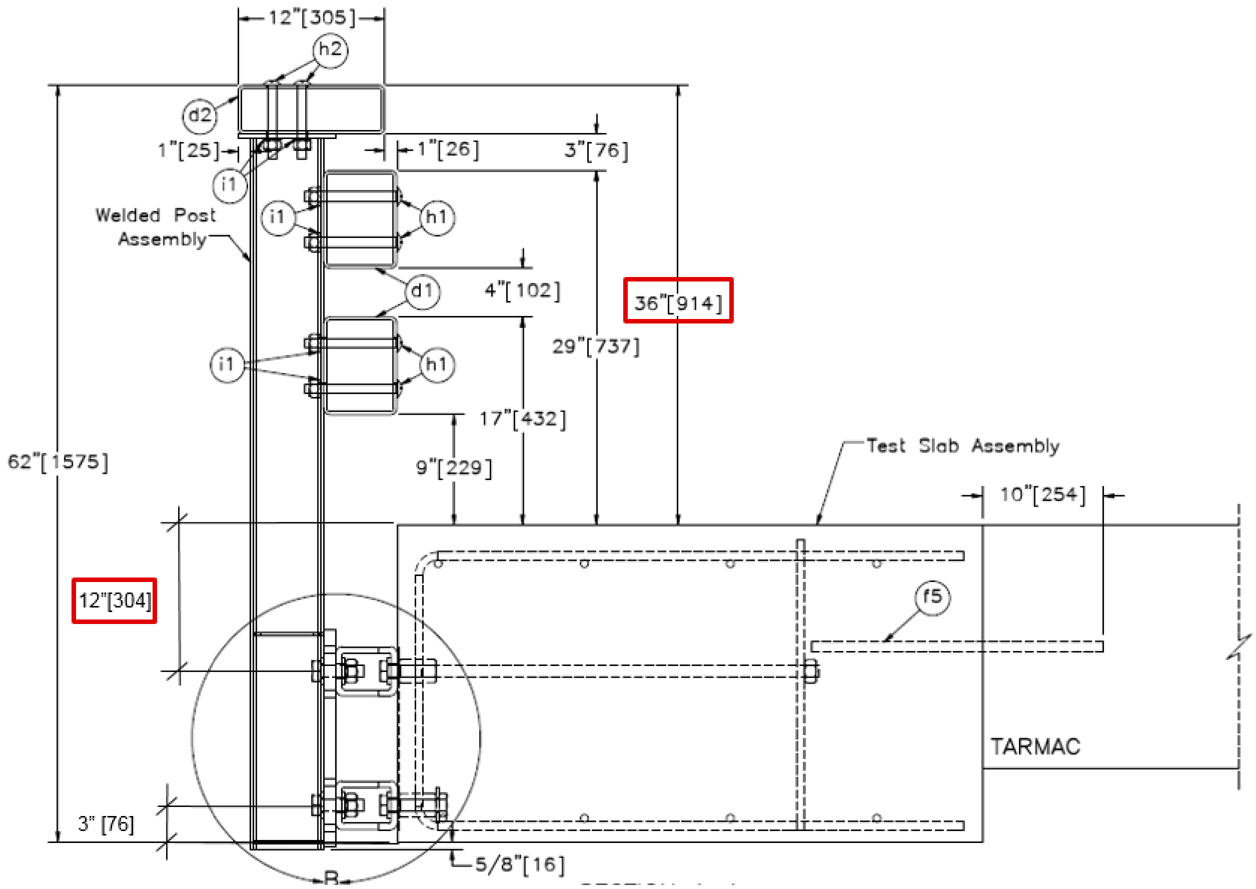


Figure 104. Surrogate Bridge Deck Profile View for MASH 2016 Test Designation Nos. 4-11 and 4-12

The surrogate concrete bridge deck consisted of a 48 in. (1,219 mm) wide by 26 in. (660 mm) deep by 108 ft (32.9 m) long reinforced-concrete full slab with a minimum compressive strength of 4,000 psi. Longitudinal no. 5 rebar were located in the top and bottom of the bridge deck spaced at 12 in. (305 mm) on center. Transverse no. 5 U-shaped bent rebar was spaced at 12 in. (305 mm) on center. These rebar were tied with vertical no. 5 rebar coming from the compacted soil. The stages of construction of the surrogate concrete bridge deck are shown in Figure 105. Additionally, for the construction of the surrogate concrete bridge deck, the anchorage hardware was embedded within the form with welded coupling nuts to the vertical plate and with the use of welded studs. When the form was removed, some vertical plates slightly detached from the exterior, vertical edge of the slab, which may lead to the fall of these exterior steel plates, which was determined to be unacceptable.

Figure 105. Construction of Surrogate Concrete Bridge Deck

9 TEST REQUIREMENTS AND EVALUATION CRITERIA

9.1 Test Requirements

Longitudinal barriers, such as beam-and-post bridge rails, must satisfy impact safety standards in order to be declared eligible for federal reimbursement by the Federal Highway Administration (FHWA) for use on the National Highway System (NHS). For new hardware, these safety standards consist of the guidelines and procedures published in MASH 2016 [9]. According to Test Level 4 (TL-4) of MASH 2016, beam-and-post bridge rails must be subjected to three full-scale vehicle crash tests, as summarized in Table 65. Note that there is little difference between MASH 2009 and MASH 2016 for longitudinal barriers, specifically the bridge railing tested and evaluated in this project, except that there are additional occupant compartment deformation standards and documentation required by MASH 2016.

Table 65. MASH 2016 TL-4 Crash Test Conditions for Longitudinal Barriers [9]

Test Article	Test Designation No.	Test Vehicle	Vehicle Weight, lb (kg)	Impact Conditions		Evaluation Criteria [1]
				Speed, mph (km/h)	Angle, deg.	
Longitudinal Barrier	4-10	1100C	2,425 (1,100)	62 (100)	25	A,D,F,H,I
	4-11	2270P	5,000 (2,270)	62 (100)	25	A,D,F,H,I
	4-12	10000S	22,000 (10,000)	56 (90.0)	15	A,D,G

[1] Evaluation criteria explained in Table 66.

9.2 Evaluation Criteria

Evaluation criteria for full-scale vehicle crash testing are based on three appraisal areas: (1) structural adequacy; (2) occupant risk; and (3) vehicle trajectory after collision. Criteria for structural adequacy are intended to evaluate the ability of the bridge rail to contain and redirect impacting vehicles. In addition, controlled lateral deflection of the test article is acceptable. Occupant risk evaluates the degree of hazard to occupants in the impacting vehicle. Post-impact vehicle trajectory is a measure of the potential of the vehicle to result in a secondary collision with other vehicles and/or fixed objects, thereby increasing the risk of injury to the occupants of the impacting vehicle and/or other vehicles. These evaluation criteria are summarized in Table 66 and discussed in greater detail in MASH 2016. The full-scale vehicle crash test was conducted and reported in accordance with the procedures provided in MASH 2016.

In addition to the standard occupant risk measures, the Post-Impact Head Deceleration (PHD), the Theoretical Head Impact Velocity (THIV), and the Acceleration Severity Index (ASI) were determined and reported. Additional discussion on PHD, THIV and ASI is provided in MASH 2016.

9.3 Critical Impact Point (CIP)

MASH 2016 specifies that post-and-beam longitudinal barriers may have two potential critical impact points (CIPs), one associated with wheel snagging and pocketing on a post (i.e., hard point) and another that induces a maximum loading to a critical portion of the system, such as a rail splice [9]. When splices are coincident with a hard point, a single test can be conducted to evaluate both critical points. If splices are spaced away from a hard point, it may be necessary to conduct two full-scale crash tests with a particular vehicle to properly evaluate CIPs. However, it should be noted that only the 2270P vehicle crash test needs to be repeated as it produces the greatest splice loading and hence the greatest chance for structural failure. Due to the fact that rail splices within the new bridge rail are centered only 2 ft (610 mm) away from the centerline of the posts, it was believed that vehicle snagging on a post and/or splice as well as maximum loading on a splice could be evaluated with one test of each of the two passenger vehicle types.

Table 66. MASH 2016 Evaluation Criteria for Longitudinal Barriers

<table>
<tr><td>Structural Adequacy</td><td>A.</td><td colspan="3">Test article should contain and redirect the vehicle or bring the vehicle to a controlled stop; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.</td></tr>
<tr><td rowspan="12">Occupant Risk</td><td>D.</td><td colspan="3">Detached elements, fragments or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformations of, or intrusions into, the occupant compartment should not exceed limits set forth in Section 5.2.2 and Appendix E of MASH 2016.</td></tr>
<tr><td>F.</td><td colspan="3">The vehicle should remain upright during and after collision. The maximum roll and pitch angles are not to exceed 75 degrees.</td></tr>
<tr><td>G.</td><td colspan="3">It is preferable, although not essential, that the vehicle remain upright during and after collision.</td></tr>
<tr><td rowspan="4">H.</td><td colspan="3">Occupant Impact Velocity (OIV) (see Appendix A, Section A5.2.2 of MASH 2016 for calculation procedure) should satisfy the following limits:</td></tr>
<tr><td colspan="3">Occupant Impact Velocity Limits</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>30 ft/s
(9.1 m/s)</td><td>40 ft/s
(12.2 m/s)</td></tr>
<tr><td rowspan="4">I.</td><td colspan="3">The Occupant Ridedown Acceleration (ORA) (see Appendix A, Section A5.2.2 of MASH 2016 for calculation procedure) should satisfy the following limits:</td></tr>
<tr><td colspan="3">Occupant Ridedown Acceleration Limits</td></tr>
<tr><td>Component</td><td>Preferred</td><td>Maximum</td></tr>
<tr><td>Longitudinal and Lateral</td><td>15.0 g’s</td><td>20.49 g’s</td></tr>
</table>

For the small car and pickup truck crash tests, computer simulations have demonstrated that CIPs are often controlled by the wheel snagging on a post [14]. MASH 2016 provides charts for determining the CIP for test nos. 4-10 and 4-11, as shown in Figures 2-8 and 2-11, respectively. With the new bridge railing expected to provide dynamic deflections similar to those observed with TL-3 approach guardrail transitions, Figures 2-14 and 2-17 were used for determining CIPs for test nos. 4-10 and 4-11, respectively. From those charts, the small car CIP was approximated to occur 3 ft (914 mm) upstream from a rail splice and 5 ft (1.5 m) upstream from a post. From those charts, the pickup truck CIP was approximated to occur 5 ft (1.5 m) upstream from a rail splice and 7 ft (2.1 m) upstream from a post. For the SUT crash test, a CIP location should be chosen to maximize loading into railing components, such as rail splices. According to MASH 2016 Table 2-8, the CIP for a post-and-beam bridge rail impacted by a SUT should be 5 ft (1.5 m) upstream from a rail splice tube location, or 7 ft (2.1 m) upstream from a post.

10 TEST CONDITIONS

10.1 Test Facility

MwRSF's Outdoor Test Site is located at the Lincoln Air Park on the northwest side of the Lincoln Municipal Airport, which is approximately 5 miles (8.0 km) northwest of the University of Nebraska-Lincoln.

10.2 Vehicle Tow and Guidance System

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

A vehicle guidance system developed by Hinch [54] was used to steer the test vehicle. A guide flag, which was attached to the right-front wheel and the guide cable, sheared off before impact with the barrier system. The ⅜-in. (10-mm) diameter guide cable was tensioned to approximately 3,500 lb (15.6 kN) and supported both laterally and vertically every 100 ft (30.5 m) by hinged stanchions. The hinged stanchions stood upright while holding up the guide cable. As the vehicle was towed down the line, the guide flag struck and knocked each stanchion to the ground.

10.3 Test Vehicles

For test no. STBR-1, a 2007 Freightliner M2 106 SUT was used as the test vehicle. The curb, test inertial, and gross static vehicle weights were 13,725 lb (6,226 kg), 22,124 lb (10,035 kg), and 22,277 lb (10,105 kg), respectively. The test vehicle is shown in Figures 106 and 107, and vehicle dimensions are shown in Figure 108.

For test no. STBR-2, a 2011 Dodge Ram 1500 pickup truck was used as the test vehicle. The curb, test inertial, and gross static vehicle weights were 4,938 lb (2,240 kg), 4,492 lb (2,264 kg), and 5,157 lb (2,339 kg), respectively. The test vehicle is shown in Figures 109 and 110, and vehicle dimensions are shown in Figure 111. MASH 2016 requires test vehicles used in crash testing to be no more than six model years old. It should be noted that the test vehicle used was within 6 years of the research project contract date, which was 2017.

For test no. STBR-3, a 2009 Kia Rio small vehicle was used as the test vehicle. The curb, test inertial, and gross static vehicle weights were 2,456 lb (1,114 kg), 2,408 lb (1,092 kg), and 2,569 lb (1,165 kg), respectively. The test vehicle is shown in Figures 112 and 113, and vehicle dimensions are shown in Figure 114. MASH 2016 requires test vehicles used in crash testing to be no more than six model years old. A 2009 model was used for this test because the vehicle geometry of newer models did not comply with recommended vehicle dimension ranges specified in Table 4.1 of MASH 2016. The use of older test vehicles due to recent small car vehicle properties falling outside of MASH 2016 recommendations was allowed by FHWA and AASHTO in MASH implementation guidance dated May of 2018 [55].

For test no. STBR-4, a 2007 Freightliner M2 106 SUT F was used as the test vehicle. The curb, test inertial, and gross static vehicle weights were 13,884 lb (6,298 kg), 22,152 lb (10,048 kg), and 22,314 lb (10,121 kg), respectively. The test vehicle is shown in Figures 115 and 116, and vehicle dimensions are shown in Figure 117.

The longitudinal component of the center of gravity (c.g.) was determined using the measured axle weights for all three vehicle types. The Elevated Axle Method [56] was used to determine the vertical component of the c.g. for each of the 10000S vehicles. This method converted measured wheel weights at different elevations to the location of the vertical component of the c.g. The Suspension Method [57] was used to determine the vertical component of the c.g. for the pickup truck. This method is based on the principle that the c.g. of any freely-suspended body is in the vertical plane through the point of suspension. The vehicle was suspended successively in three positions, and the respective planes containing the c.g. were established. The intersection of these planes pinpointed the final c.g. location for the test inertial condition. The vertical component of the c.g. for the 1100C vehicle was determined utilizing a procedure published by SAE [58]. The location of the final c.g. for test no. STBR-1 is shown in Figures 108 and 118. The location of the final c.g. for test no. STBR-2 is shown in Figures 111 and 119. The location of the final c.g. for test no. STBR-3 is shown in Figures 114 and 120. The location of the final c.g. for test no. STBR-4 is shown in Figures 117 and 121. Data used to calculate the locations of the c.g. are shown in Appendix E.

Square, black- and white-checkered targets were placed on the vehicle for reference to be viewed from the high-speed digital video cameras and aid in the video analysis, as shown in Figure 118 through 121. Round, checkered targets were placed at the c.g. on the left-side door, the right-side door, and the roof of the vehicle.

The front wheels of the test vehicles were aligned to vehicle standards, except the toe-in value was adjusted to zero such that the vehicles would track properly along the guide cable. A 5B flash bulb was mounted on the vehicle's left-side dash for all four tests and was fired by a pressure tape switch mounted at the impact corner of the bumper. The flash bulb was fired upon initial impact with the test article to create a visual indicator of the precise time of impact on the high-speed digital videos. A remote-controlled brake system was installed in the test vehicles so the vehicles could be brought safely to a stop after each test.

For test no. STBR-1, the left and right frame rails were set up symmetrically. A total of four shear plates were attached to the frame to provide for extra support. The front shear plates measured 4 in. x 17 in. x ⅜ in. (102 mm x 432 mm x 10 mm) mounted at a 50-degree angle away from horizontal axis on the right side and at a 60-degree angle on the left side with the top ahead of the bottom. The back shear plates were installed approximately 39 in. (991 mm) from the rear end of the frame, as shown in Figure 122. The front shear plates were connected with one ⅝-in. (16-mm) diameter bolt through the van body subframe, and two ⅝-in. (16-mm) diameter bolts passed through the truck frame. The rear shear plates were measured 6 in. x 14 in. x ⅜ in. (152 mm x 356 mm x 10 mm) and were mounted in a vertical position. The rear shear plates were connected with one ⅝-in. (16-mm) diameter bolt passed through the van body subframe, and three ⅝-in. (16-mm) diameter bolts passed through the truck frame. The subframe was welded to the flat edge sections of the shear plate and not in the corners. The truck frame was not welded. Eight U-bolts were installed between the box and the frame rail to provide additional strength. These bolts were ⅝-in. (16-mm) diameter with 6-in. x 1½-in. x ½-in. (152-mm x 38-mm x 13-mm) steel caps.

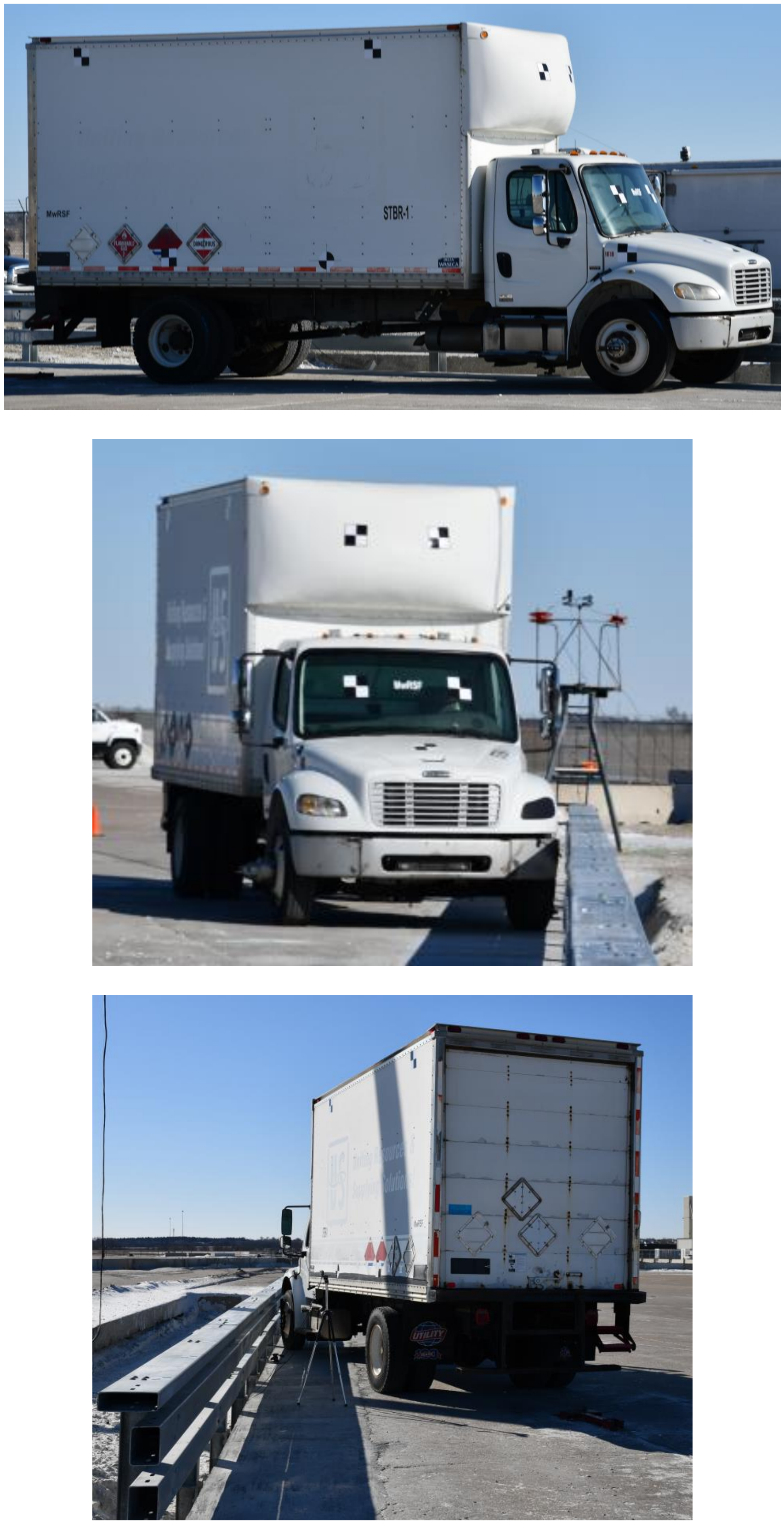


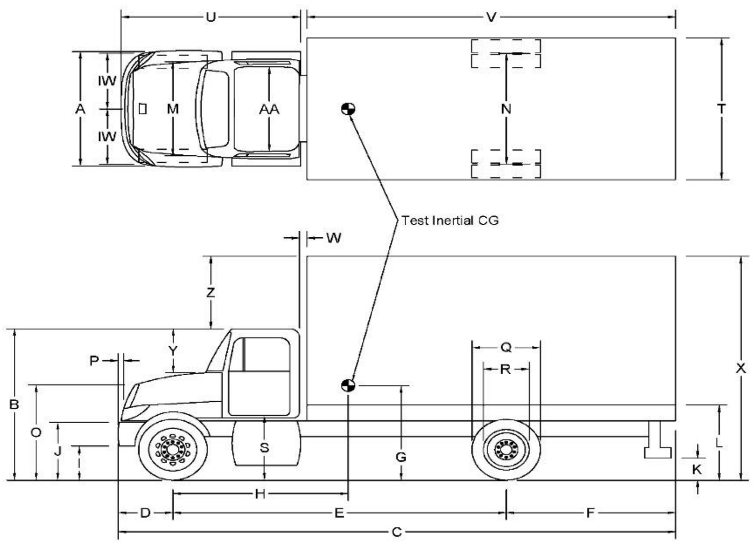
Figure 106. Test Vehicle, Test No. STBR-1

Figure 107. Test Vehicle’s Undercarriage, Test No. STBR-1

Date: 2/8/2019 **Test Name:** STBR-1 **VIN No:** 1FVACXCS57HX61818

Year: 2007 **Make:** Freightliner **Model:** M2 106

Tire Size: 275/80R22.5 **Tire Inflation Pressure:** 110 Psi **Odometer:** 195866



Vehicle Geometry - in. (mm)
Target Ranges listed below

A:	93 1/4	(2369)	B:	98 1/2	(2502)
C:	338	(8585)	D:	29 1/2	(749)
	Max: 394 (10000)				
E:	216 5/8	(5502)	F:	80 1/2	(2045)
	Max: 240 (6100)				
G:	54 11/16	(1389)	H:	137 13/16	(3500)
I:	18 1/4	(464)	J:	31 3/4	(806)
K:	16 3/4	(425)	L:	50 1/2	(1283)
				49±2 (1245±50)	
M:	82 5/8	(2099)	N:	72 1/2	(1842)
O:	54 3/4	(1391)	P:	1	(25)
Q:	39 1/2	(1003)	R:	23 1/2	(597)
S:	35 1/4	(895)	T:	95 3/8	(2423)
U:	106 1/4	(2699)	V:	229	(5817)
W:	2 3/4	(70)	X:	153	(3886)
Y:	33 1/8	(841)	Z:	54 1/2	(1384)
IW (Impact Width):	93 1/4	(2369)	AA:	72	(1829)

Ballast

Weight: 8525 (3867)

CG height: 63 2/3 (1617)
63±2 (1600±50)

Mass Distribution lb (kg)

Gross Static	LF	3945 (1789)	RF	4220 (1914)
	LR	6950 (3152)	RR	7162 (3249)

Wheel Center Height (Front):	20	(508)
Wheel Center Height (Rear):	20	(508)
Wheel Well Clearance (Front):	48	(1219)
Wheel Well Clearance (Rear):	39 1/4	(997)
Bottom Frame Height (Front):	28 1/4	(718)
Bottom Frame Height (Rear):	29	(737)

Weights lb (kg)	Curb	Test Inertial	Gross Static
W-front	6530 (2962)	8048 (3651)	8165 (3704)
W-rear	7195 (3264)	14076 (6385)	14112 (6401)
W-total	13725 (6226) 13200±2200 (6000±1000)	22124 (10035) 22046±660 (10000±300)	22277 (10105)

GVWR Ratings lb

Front	12000
Rear	21000
Total	33000

Surrogate Occupant Data

Type:	Hybrid II
Mass:	153 lb
Seat Position:	Left/Driver

Engine Type:	Diesel
Engine Size:	6.4L I6
Transmission Type:	Automatic
Drive Type:	RWD

Note any damage prior to test: None

Figure 108. Vehicle Dimensions, Test No. STBR-1

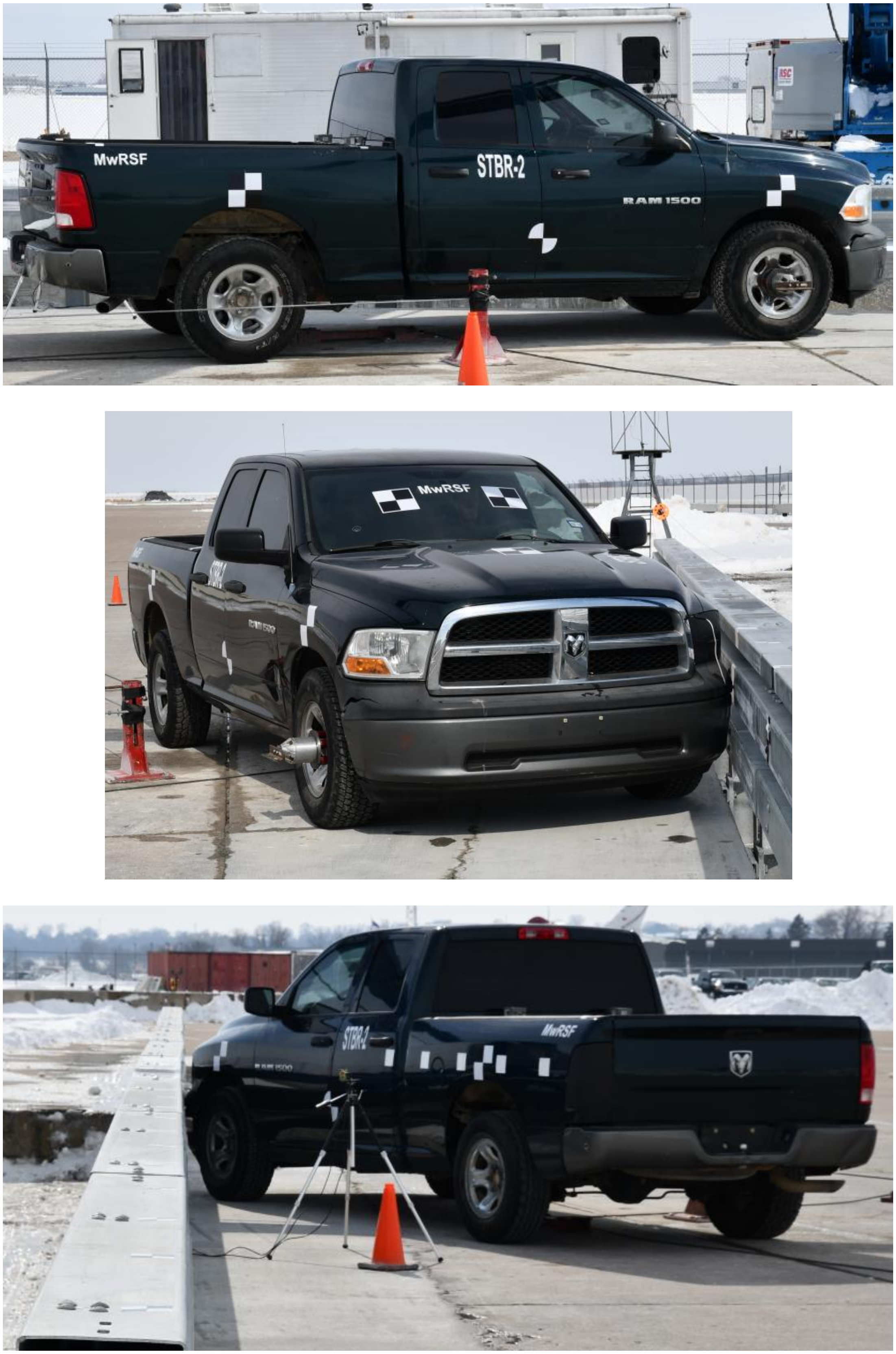


Figure 109. Test Vehicle, Test No. STBR-2

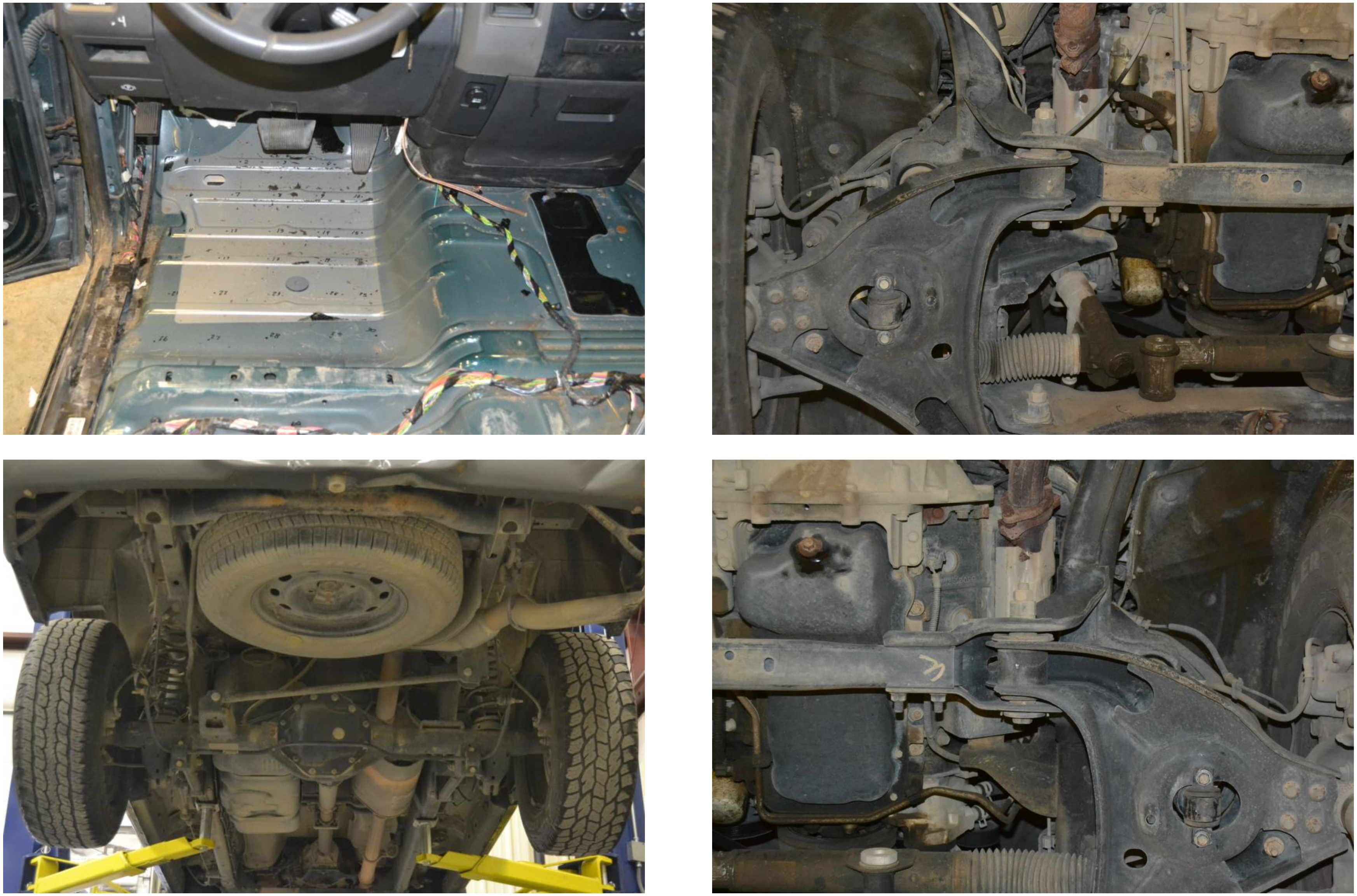
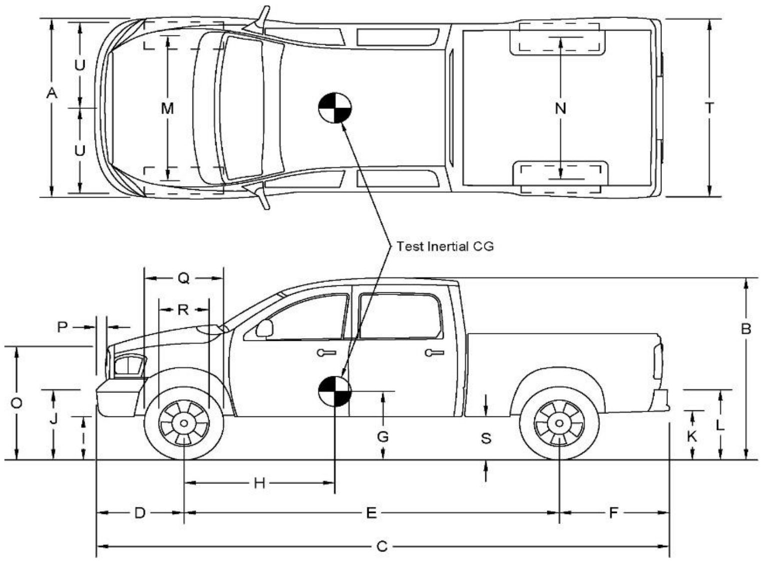


Figure 110. Test Vehicle's Interior Floorboards and Undercarriage, Test No. STBR-2

Date:	2/22/2019	**Test Name:**	STBR-2	**VIN No:**	1D7RB1GK0BS554408
Year:	2011	**Make:**	Dodge	**Model:**	Ram 1500
Tire Size:	P265/70R17	**Tire Inflation Pressure:**	40 Psi	**Odometer:**	191913



Vehicle Geometry - in. (mm)
Target Ranges listed below

A:	78 1/4	(1988)	B:	74 1/8	(1883)
	78±2 (1950±50)				
C:	229 1/8	(5820)	D:	39 3/8	(1000)
	237±13 (6020±325)			39±3 (1000±75)	
E:	140 1/2	(3569)	F:	46 1/8	(1172)
	148±12 (3760±300)				
G:	28	(711)	H:	64 7/8	(1648)
	min: 28 (710)			63±4 (1575±100)	
I:	12	(305)	J:	24	(610)
K:	20	(508)	L:	29	(737)
M:	68 1/4	(1734)	N:	67 1/4	(1708)
	67±1.5 (1700±38)			67±1.5 (1700±38)	
O:	43 1/4	(1099)	P:	4	(102)
	43±4 (1100±75)				
Q:	30 1/2	(775)	R:	18 1/2	(470)
S:	13 3/4	(349)	T:	77 3/8	(1965)

U (impact width):	36 1/2	(927)
Wheel Center Height (Front):	15 1/4	(387)
Wheel Center Height (Rear):	15 1/4	(387)
Wheel Well Clearance (Front):	34 1/2	(876)
Wheel Well Clearance (Rear):	37 1/2	(953)
Bottom Frame Height (Front):	17	(432)
Bottom Frame Height (Rear):	24 3/4	(629)
Engine Type:	Gasoline	
Engine Size:	3.6L V6	
Transmission Type:	Automatic	
Drive Type:	RWD	
Cab Style:	Quad Cab	
Bed Length:	76"	

Mass Distribution lb (kg)

Gross Static	LF	1435	(651)	RF	1357	(616)
	LR	1192	(541)	RR	1173	(532)

Weights lb (kg)	Curb	Test Inertial	Gross Static
W-front	2708 (1228)	2687 (1219)	2792 (1266)
W-rear	2230 (1012)	2305 (1046)	2365 (1073)
W-total	4938 (2240)	4992 (2264) 5000±110 (2270±50)	5157 (2339) 5165±110 (2343±50)

GVWR Ratings lb		Surrogate Occupant Data	
Front	3700	Type:	Hybrid II
Rear	3900	Mass:	165 lb
Total	6700	Seat Position:	Left/Driver

Note any damage prior to test: None

Figure 111. Vehicle Dimensions, Test No. STBR-2

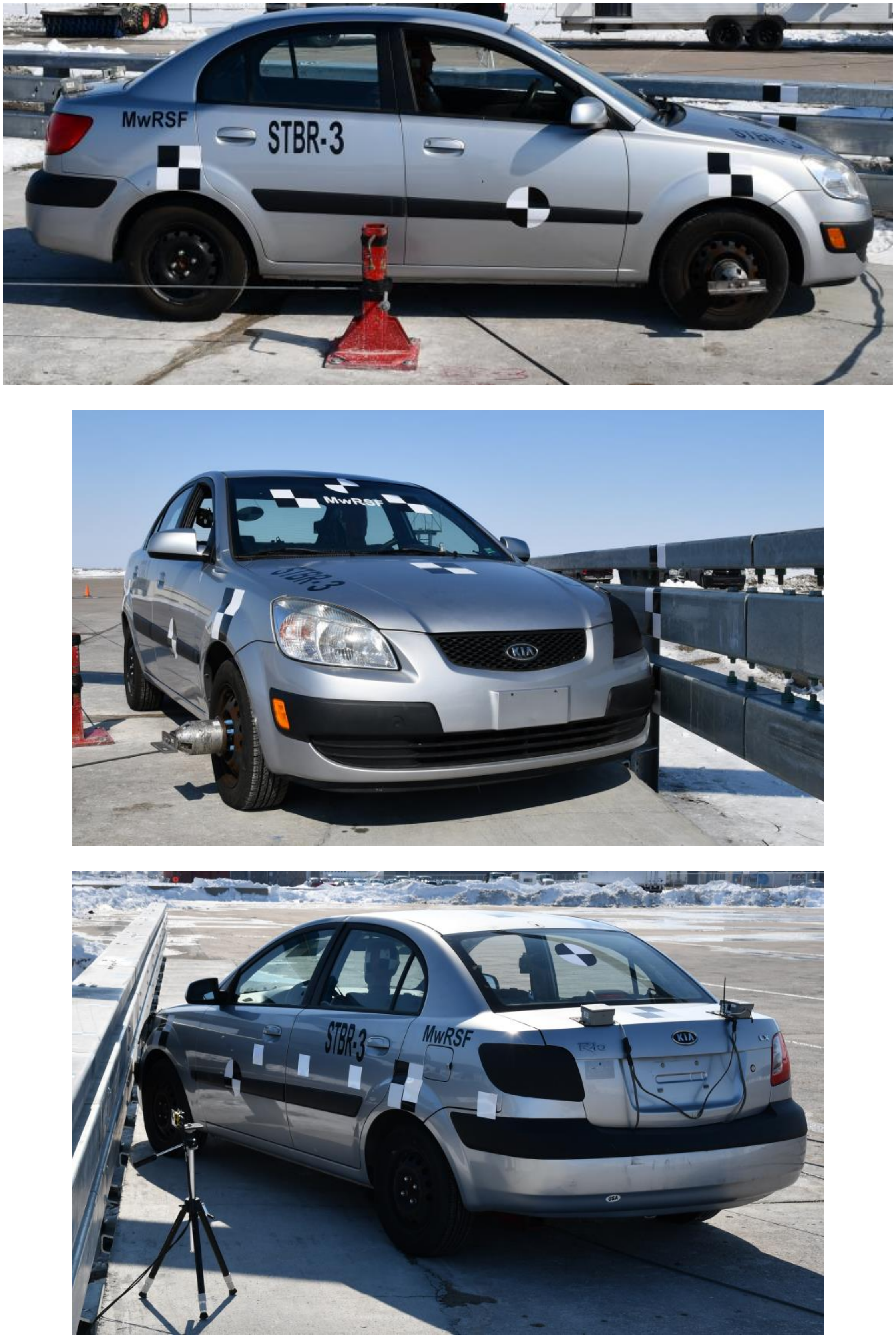


Figure 112. Test Vehicle, Test No. STBR-3

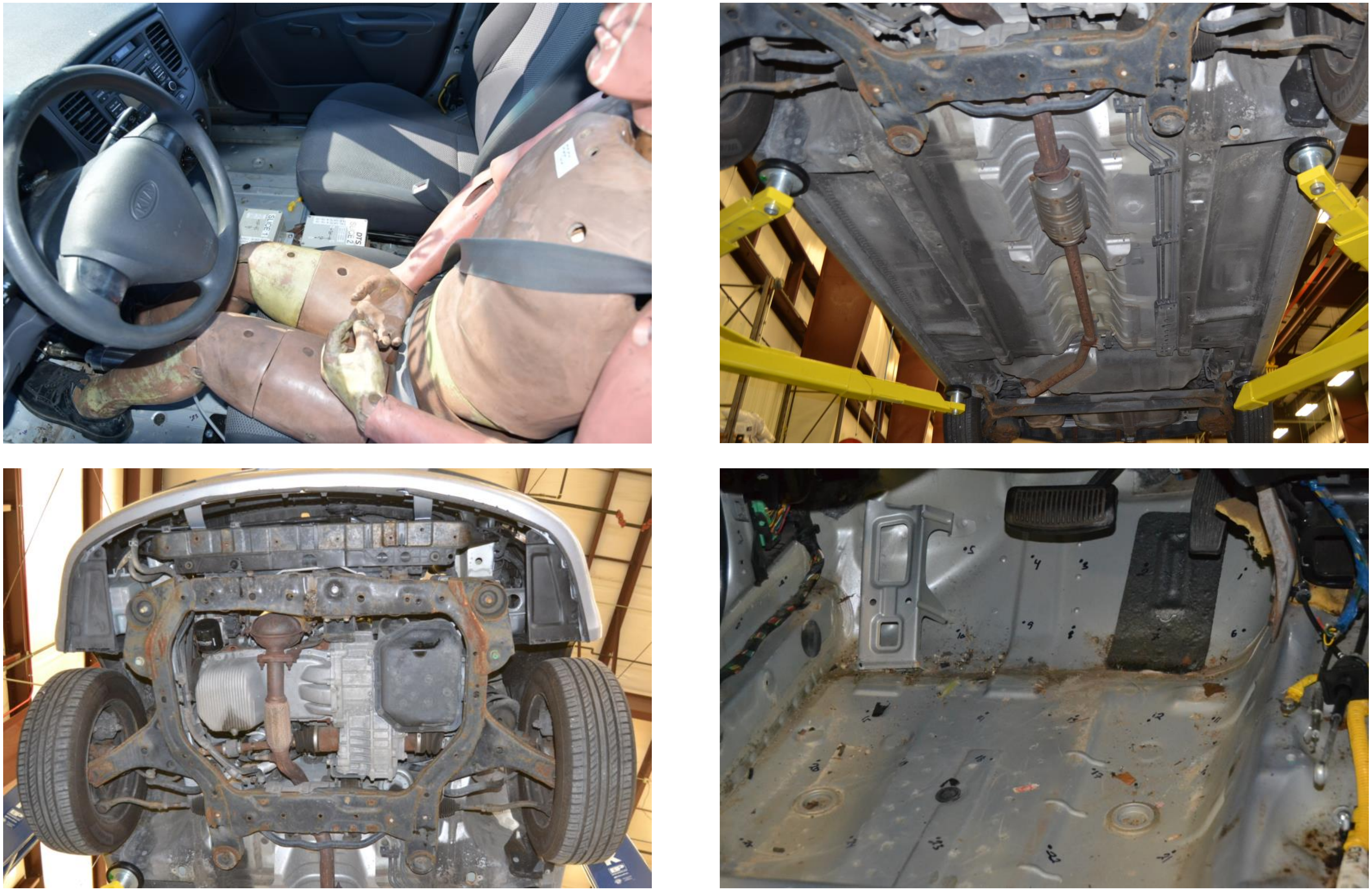
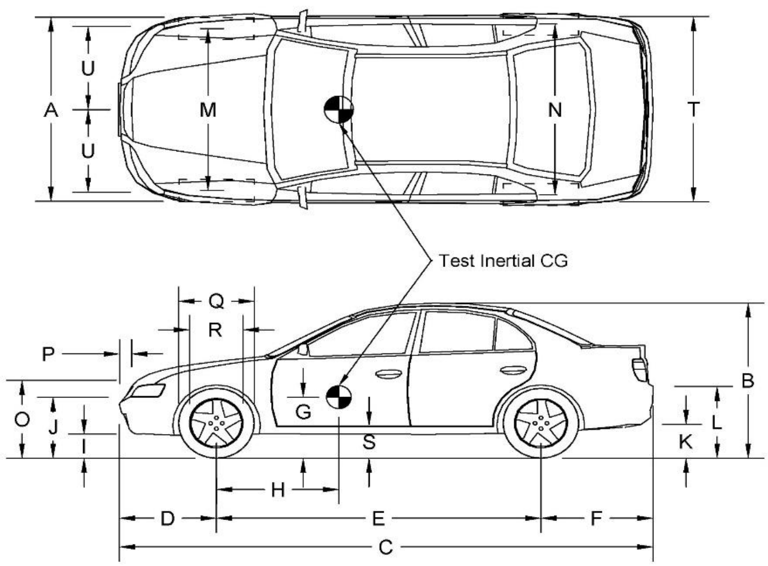


Figure 113. Test Vehicle's Interior Floorboards and Undercarriage, Test No. STBR-3

Date:	2/28/2019	**Test Name:**	STBR-3	**VIN No:**	KNADE223996504334
Year:	2009	**Make:**	Kia	**Model:**	Rio
Tire Size:	185/65R14	**Tire Inflation Pressure:**	32 Psi	**Odometer:**	134652



Vehicle Geometry - in. (mm)
Target Ranges listed below

A:	64 3/4	(1645)	B:	57	(1448)
	65±3 (1650±75)				
C:	167 3/8	(4251)	D:	32 1/2	(826)
	169±8 (4300±200)			35±4 (900±100)	
E:	98 1/2	(2502)	F:	38	(965)
	98±5 (2500±125)				
G:	22 1/4	(565)	H:	36 1/16	(916)
				39±4 (990±100)	
I:	7	(178)	J:	21	(533)
K:	11	(279)	L:	21 1/2	(546)
M:	58	(1473)	N:	57 3/8	(1457)
	56±2 (1425±50)			56±2 (1425±50)	
O:	26 1/2	(673)	P:	1 3/4	(44)
	24±4 (600±100)				
Q:	22 3/4	(578)	R:	15 3/8	(391)
S:	11 1/8	(283)	T:	64 3/4	(1645)

U (impact width):	28 3/8	(721)
Top of radiator core support:	28 7/8	(733)
Wheel Center Height (Front):	11	(279)
Wheel Center Height (Rear):	11 1/2	(292)
Wheel Well Clearance (Front):	24 7/8	(632)
Wheel Well Clearance (Rear):	25 3/8	(645)
Bottom Frame Height (Front):	6 1/2	(165)
Bottom Frame Height (Rear):	8 1/2	(216)
Engine Type:	Gasoline	
Engine Size:	1.6 L 4 Cyl	
Transmission Type:	Automatic	
Drive Type:	FWD	

Mass Distribution lb (kg)

Mass Distrib	LF	845	(383)	RF	761	(345)
	LR	480	(218)	RR	483	(219)

Weights lb (kg)	Curb		Test Inertial		Gross Static	
W-front	1574	(714)	1527	(693)	1606	(728)
W-rear	882	(400)	881	(400)	963	(437)
W-total	2456	(1114)	2408	(1092)	2569	(1165)
			2420±55 (1100±25)		2585±55 (1175±50)	

GVWR Ratings lb

Front	1918
Rear	1874
Total	3638

Surrogate Occupant Data

Type:	Hybrid II
Mass:	161 lb
Seat Position:	Left/Driver

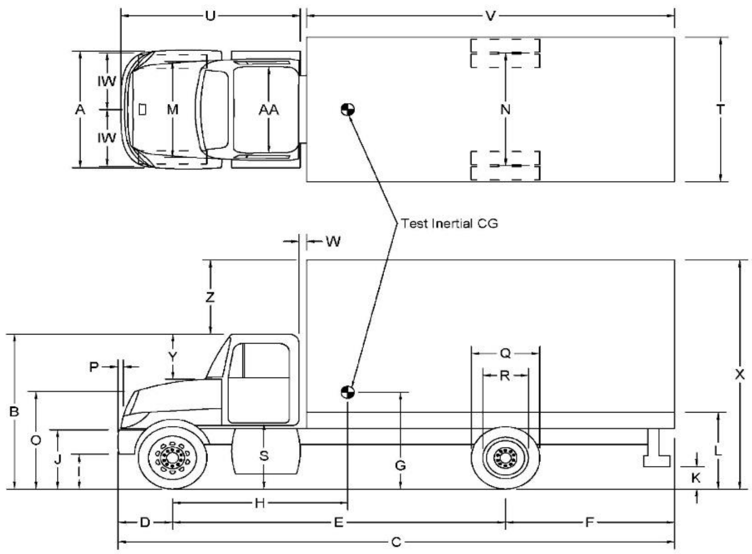
Note any damage prior to test: None

Figure 114. Vehicle Dimensions, Test No. STBR-3

Figure 115. Test Vehicle, Test No. STBR-4

Figure 116. Test Vehicle’s Interior Floorboards and Undercarriage, Test No. STBR-4

Date:	5/22/2019	Test Name:	STBR-4	VIN No:	1FVACXCS37HX61820
Year:	2007	Make:	Freightliner	Model:	M2 106
Tire Size:	295/75R22.5	Tire Inflation Pressure:	110 psi	Odometer:	317533



Vehicle Geometry - in. (mm)
Target Ranges listed below

A:	93 5/8	(2378)	B:	100	(2540)
C:	337 1/2	(8573) Max: 394 (10000)	D:	39	(991)
E:	216 1/4	(5493) Max: 240 (6100)	F:	81	(2057)
G:	52 5/8	(1337)	H:	136 7/16	(3466)
I:	18 7/8	(479)	J:	32 1/8	(816)
K:	16 3/4	(425)	L:	50 1/2	(1283) 49±2 (1245±50)
M:	82 5/8	(2099)	N:	72 5/8	(1845)
O:	53	(1346)	P:	1	(25)
Q:	38 7/8	(987)	R:	23 3/8	(594)
S:	35 1/4	(895)	T:	95 1/2	(2426)
U:	105 1/2	(2680)	V:	228 7/8	(5813)
W:	2 3/4	(70)	X:	153	(3886)
Y:	34	(864)	Z:	54 1/2	(1384)
IW (Impact Width):	94	(2388)	AA:	71 7/8	(1826)

Ballast

Weight lb (kg): 8654 (3925)

CG height in. (mm): 63 5/6 (1621)
63±2 (1600±50)

Mass Distribution - lb (kg)

Gross Static	LF	4210 (1910)	RF	4094 (1857)
	LR	6730 (3053)	RR	7280 (3302)

Wheel Center Height (Front):	19 3/8	(492)
Wheel Center Height (Rear):	19 1/2	(495)
Wheel Well Clearance (Front):	47 3/4	(1213)
Wheel Well Clearance (Rear):	44 3/4	(1137)
Bottom Frame Height (Front):	27 7/8	(708)
Bottom Frame Height (Rear):	29 1/2	(749)

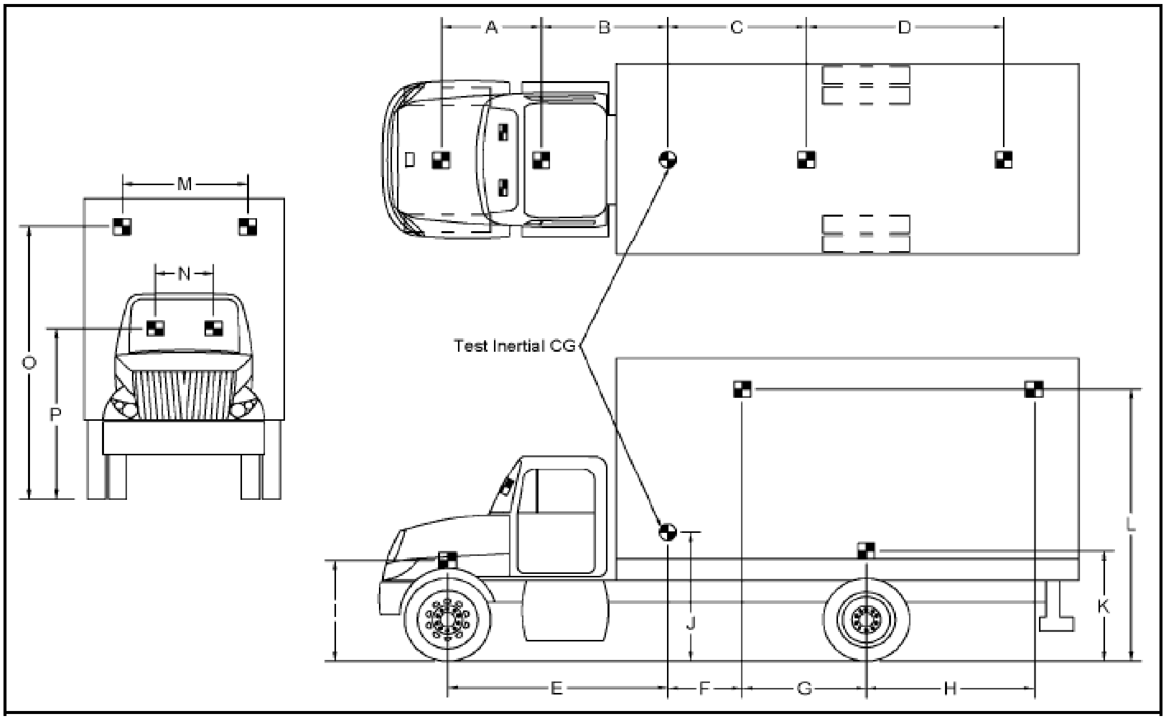
Weights lb (kg)	Curb	Test Inertial	Gross Static
W-front	6686 (3033)	8174 (3708)	8304 (3767)
W-rear	7198 (3265)	13978 (6340)	14010 (6355)
W-total	13884 (6298) 13200±2200 (6000±1000)	22152 (10048) 22046±660 (10000±300)	22314 (10121)

GVWR Ratings - lb		Surrogate Occupant Data			
Front	12000	Type:	Hybrid II	Engine Type:	Diesel
Rear	21000	Mass:	162 lb	Engine Size:	4.6L I6
Total	33000	Seat Position:	Left/Driver	Transmission Type:	Automatic
				Drive Type:	RWD

Note any damage prior to test: None

Figure 117. Vehicle Dimensions, Test No. STBR-4

Date:	1/25/2019	Test Name:	STBR-1	VIN:	1FVACXCS57HX61818
Year:	2007	Make:	Freightliner	Model:	M2 106

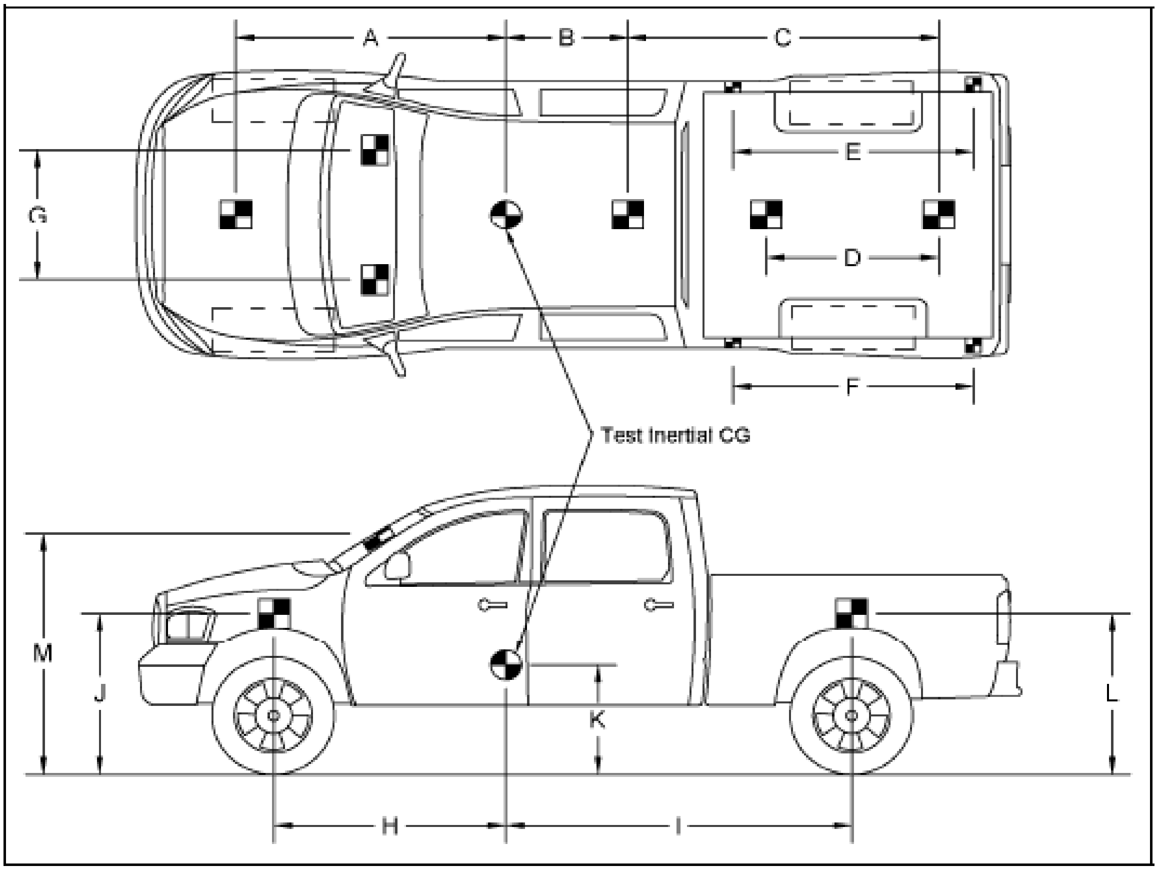


TARGET GEOMETRY-- in. (mm)

A:	57	(1448)	F:	23 1/4	(591)	L:	146 3/8	(3718)
B:	99 7/16	(2526)	G:	101 5/8	(2581)	M:	30	(762)
C:	54 3/8	(1381)	H:	45 1/2	(1156)	N:	36 1/2	(927)
D:	67 1/4	(1708)	I:	59	(1499)	O:	135 1/2	(3442)
E:	137 7/8	(3502)	J:	54 5/8	(1387)	P:	82	(2083)
			K:	57 1/4	(1454)			

Figure 118. Target Geometry, Test No. STBR-1

Date:	2/14/2019	Test Name:	STBR-2	VIN:	1D7RB1GK0BS554408
Year:	2011	Make:	Dodge	Model:	Ram 1500

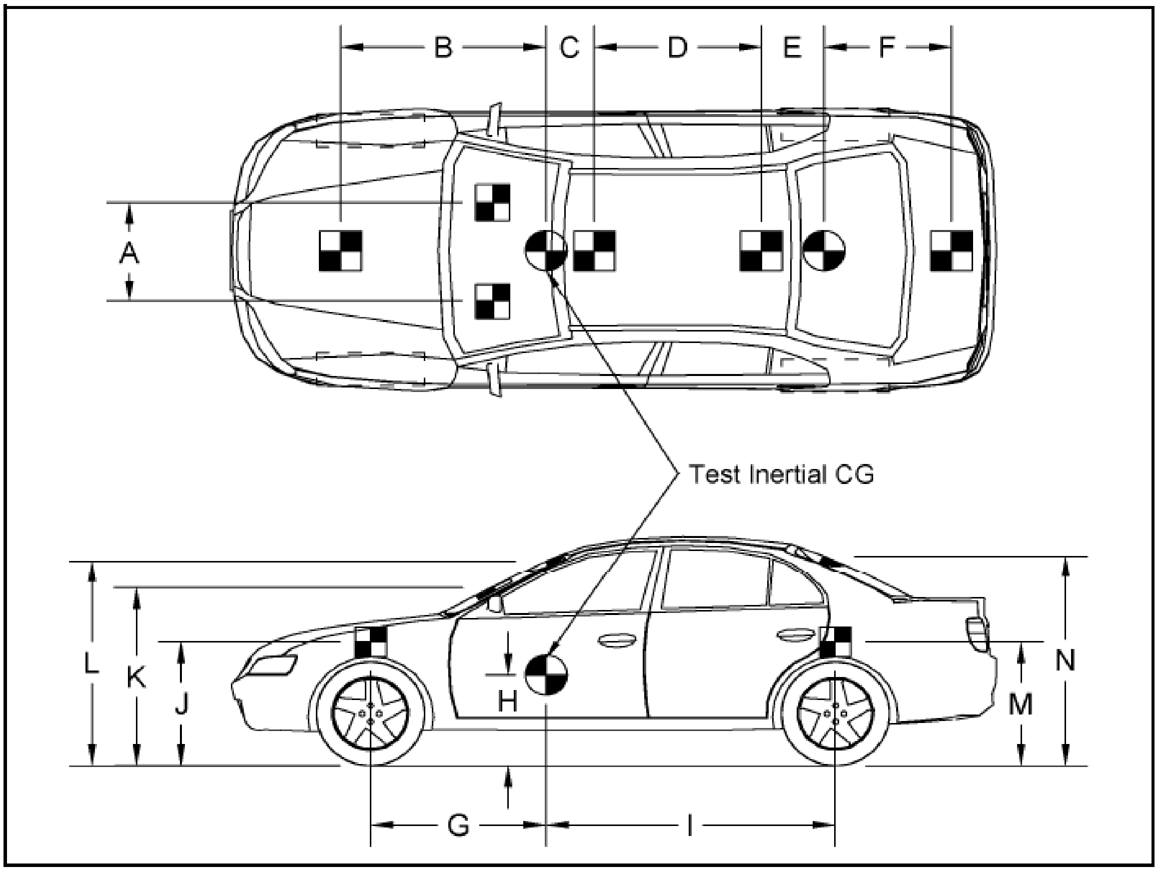


TARGET GEOMETRY-- in. (mm)

A:	74 1/2 (1892)	E:	63 1/2 (1613)	J:	38 3/4 (984)
B:	24 1/4 (616)	F:	63 1/2 (1613)	K:	28 (711)
C:	78 (1981)	G:	24 (610)	L:	41 5/8 (1057)
D:	41 (1041)	H:	64 7/8 (1648)	M:	64 1/2 (1638)
		I:	76 3/8 (1940)		

Figure 119. Target Geometry, Test No. STBR-2

Date:	2/28/2019	Test Name:	STBR-3	VIN:	KNADE223996504334
Year:	2009	Make:	Kia	Model:	Rio

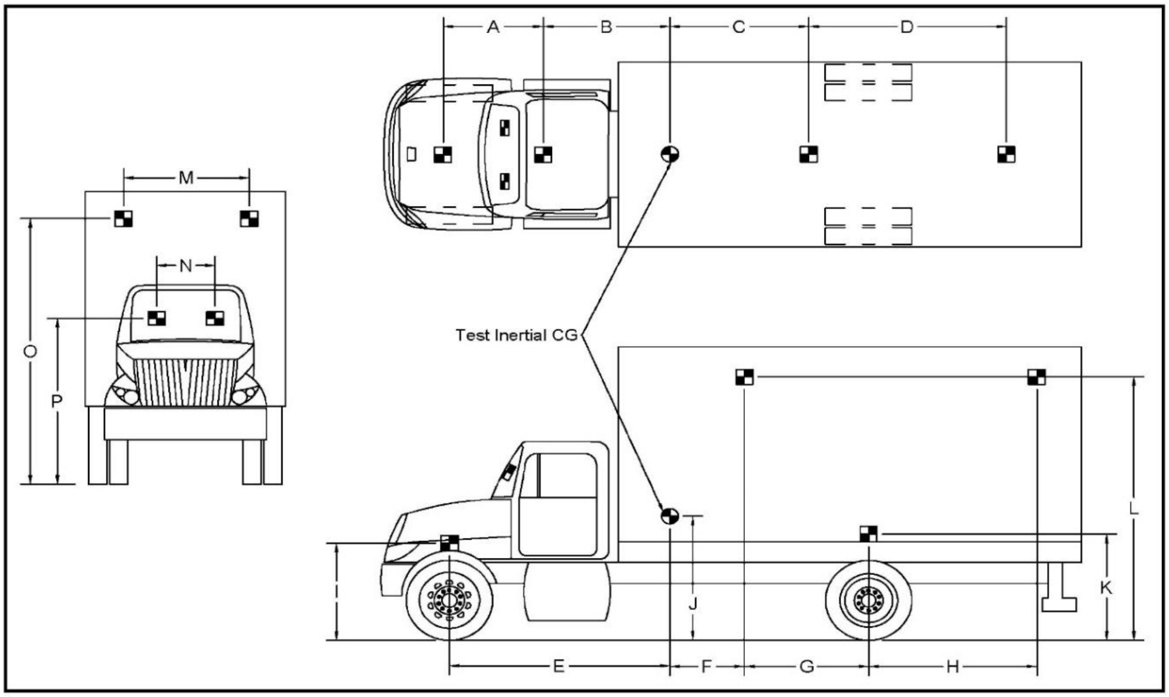


TARGET GEOMETRY-- in. (mm)

A:	23 1/4	(591)	F:	18 5/8	(473)	K: Windshield Target	48 1/8	(1222)
B:	42 3/4	(1086)	G:	36	(914)	L: Front round CG target	51 1/2	(1308)
C:	16 1/4	(413)	H:	22 1/4	(565)	M:	28 3/4	(730)
D:	28 3/4	(730)	I:	62 1/4	(1581)	N: Rear Round target	51 1/2	(1308)
E:	21	(533)	J:	28 1/2	(724)			

Figure 120. Target Geometry, Test No. STBR-3

Date:	5/22/2019	Test Name:	STBR-4	VIN:	1FVACXCS37HX61820
Year:	2007	Make:	Freightliner	Model:	M2 106



TARGET GEOMETRY-- in. (mm)

A:	64 1/8	(1629)	F:	22	(559)	L:	146 3/8	(3718)
B:	91 3/4	(2330)	G:	102 7/8	(2613)	M:	30	(762)
C:	56 1/8	(1426)	H:	44 1/2	(1130)	N:	39 1/4	(997)
D:	67 1/4	(1708)	I:	58 1/4	(1480)	O:	135 1/2	(3442)
E:	136 5/8	(3470)	J:	52 5/8	(1337)	P:	87 3/8	(2219)
			K:	54 3/8	(1381)			

Figure 121. Target Geometry, Test No. STBR-4

Left-Rear Shear Plate and U-Bolt

Left-Front Shear Plate

Figure 122. Shear Plate and U-Bolt Installation, Test No. STBR-1

In test no. STBR-1, approximately 8,525 lb (3,867 kg) of ballast was added to the van body, as can be seen in Figure 123. One safety shape concrete barrier and four steel plates were attached to the van floor. The 4,868-lb (2,208-kg) concrete barrier was attached through the floor and to the subframe with six 1¼-in. (32-mm) diameter threaded rods. Four rectangular, steel plates weighing 203 lb (92 kg) were attached with two 1¼-in. (32-mm) diameter threaded rods, and two circular, steel plates weighing 45-lb (20-kg), were each attached with one 1¼-in. (32-mm) diameter threaded rod through the center of the plates. Foam blocks were used to stabilize the concrete barrier during impact. Nylon straps attached to the side walls of the truck bed box were connected to the front and back faces of the concrete barrier to prevent translation or rotation during impact.

For test no. STBR-4, the left and right frame rails were set up symmetrically. A total of four shear plates were attached to the frame to provide for extra support. The front shear plates measured 4 in. x 14 in. x ⅜ in. (102 mm x 356 mm x 10 mm) mounted at a 50-degree angle away from horizontal axis on the right side and at a 60-degree angle on the left side with the top ahead of the bottom. The back shear plates were installed approximately 31 in. (787 mm) from the rear end of the frame, as shown in Figure 124. The front shear plates were connected with one ⅝-in. (16-mm) diameter bolt through the van body subframe, and two ⅝-in. (16-mm) diameter bolts passed through the truck frame. The rear shear plates were measured 6 in. x 13¼ in. x ⅜ in. (152 mm x 337 mm x 10 mm) and were mounted in a vertical position. The rear shear plates were connected with one ⅝-in. (16-mm) diameter bolt through the van body subframe, and three ⅝-in. (16-mm) diameter bolts passed through the truck frame. The subframe was welded to the flat edge sections of the shear plate except for in the corners. The truck frame was not welded. Eight U-bolts were installed between the box and the frame rail to provide additional strength. These bolts were ⅝-in. (16-mm) diameter with 6-in. x 1½-in. x ½-in. (152-mm x 38-mm x 13-mm) steel caps.

In test no. STBR-4, approximately 8,483 lb (3,848 kg) of ballast was added to the van body, as can be seen in Figure 125. One safety shape concrete barrier, two concrete blocks, and one concrete rail were attached to the van floor. The 4,979-lb (2,258-kg) concrete barrier was attached through the floor and to the subframe with six 1¼-in. (32-mm) diameter threaded rods. The two concrete blocks, each weighing 645 lb (293 kg), were attached with two 1¼-in. (32-mm) diameter threaded rods. The 1,224 lb (555 kg) concrete rail was attached using five 1¼-in. (32-mm) diameter threaded rods. Four rectangular, steel plates, each weighing 203 lb (92 kg), were attached with two 1¼-in. (32-mm) diameter threaded rods, and two circular, steel plates weighing 45-lb (20-kg), were each attached with one 1¼-in. (32-mm) diameter threaded rod through the center of the plates. One rectangular and one circular steel plate were placed on each concrete block, and the other two rectangular steel plates were placed on the concrete rail. Foam blocks were used to stabilize the concrete barrier during impact. Nylon straps attached to the side walls of the truck bed box were connected to the front and back faces of the concrete barrier to prevent translation or rotation during impact.

Figure 123. Nylon Straps and Ballast Installation, Test No. STBR-1

Left-Rear Shear Plate and U-Bolt

Left-Front Shear Plate

Figure 124. Shear Plate and U-Bolt Installation, Test No. STBR-4

Figure 125. Nylon Straps and Ballast Installation, Test No. STBR-4

10.4 Simulated Occupant

For test nos. STBR-1 through STBR-4, a Hybrid II 50th-Percentile, Adult Male Dummy equipped with footwear was placed in the left-front seat of the test vehicle with the seat belt fastened. The simulated occupant had a final weight of 153 lb, 165 lb, 161 lb, and 162 lb (69.4, 74.8, 73.0 kg, and 73.5 kg) for test nos. STBR-1 through STBR-4, respectively. As recommended by MASH 2016, the simulated occupant was not included in calculating the c.g. location.

10.5 Data Acquisition Systems

10.5.1 Accelerometers

Two environmental shock and vibration sensor/recorder systems were used to measure the accelerations in the longitudinal, lateral, and vertical directions for test nos. STBR-2 and STBR-3. An additional environmental shock and vibration sensor/recorder system was used for test nos. STBR-1 and STBR-4, which was mounted inside the cab of each SUT. The four tests had accelerometers systems mounted near the c.g. of the test vehicles. The electronic accelerometer data obtained in dynamic testing was filtered using the SAE Class 60 and the SAE Class 180 Butterworth filter conforming to the SAE J211/1 specifications [59].

The two accelerometer systems used in all four tests, the SLICE-1 and SLICE-2 units, were modular data acquisition systems manufactured by Diversified Technical Systems, Inc. (DTS) of Seal Beach, California. The SLICE-1 unit was designated as the primary system for test nos. STBR-1, STBR-3, and STBR-4, while the SLICE-2 unit served as the primary system for test no. STBR-2. 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.

The additional system used in test nos. STBR-1 and STBR-4 was a two-arm piezoresistive accelerometer system manufactured by Endevco of San Juan Capistrano, California. Three accelerometers were used to measure each of the longitudinal, lateral, and vertical accelerations independently at a sample rate of 10,000 Hz. The accelerometers were configured and controlled using a system developed and manufactured by DTS of Seal Beach, California. More specifically, data was collected using a DTS Sensor Input Module (SIM), Model TDAS3-SIM-16M. The SIM was configured with 16 MB SRAM and 8 sensor input channels with 250 kB SRAM/channel. The SIM was mounted on a TDAS3-R4 module rack. The module rack was configured with isolated power/event/communications, 10BaseT Ethernet and RS232 communication, and an internal backup battery. Both the SIM and module rack were crashworthy. The "DTS TDAS Control" computer software program and a customized Microsoft Excel worksheet were used to analyze and plot the accelerometer data.

10.5.2 Rate Transducers

Two identical angular rate sensor systems which were mounted inside the bodies of the SLICE-1 and SLICE-2 event data recorders were used to measure the rates of rotation of each test vehicle. Each SLICE MICRO Triax ARS had a range of 1,500 degrees/sec in each of the three directions (roll, pitch, and yaw) and recorded data at 10,000 Hz to the onboard microprocessors. The raw data measurements were then downloaded, converted to the proper Euler angles for analysis, and plotted. The “SLICEWare” computer software program and a customized Microsoft Excel worksheet were used to analyze and plot the angular rate sensor data.

A third angular rate sensor, the ARS-1500, with a range of 1,500 degrees/sec in each of the three directions (roll, pitch, and yaw) was used to measure the rates of rotation of the test vehicles. The angular rate sensor was mounted on an aluminum block inside the test vehicle near the c.g. and recorded data at 10,000 Hz to the DTS SIM. The raw data measurements were then downloaded, converted to the proper Euler angles for analysis, and plotted. The “DTS TDAS Control” computer software program and a customized Microsoft Excel worksheet were used to analyze and plot the angular rate sensor data.

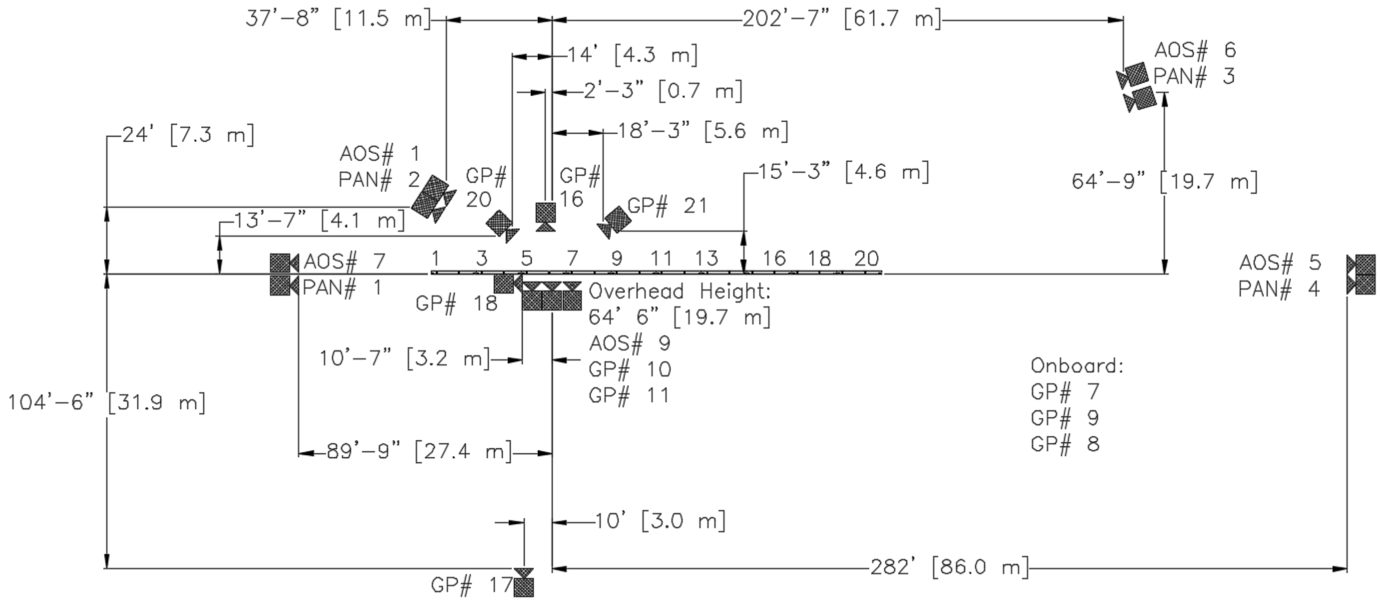
10.5.3 Retroreflective Optic Speed Trap

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

10.5.4 Digital Photography

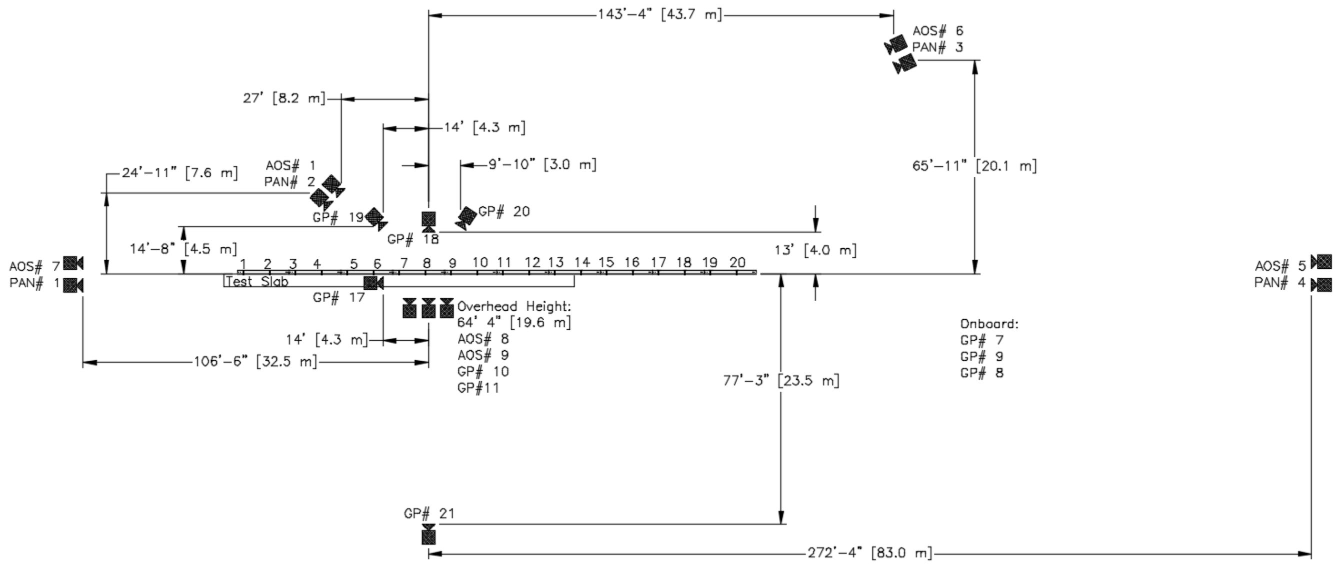
Five AOS high-speed digital video cameras, ten GoPro digital video cameras, and four Panasonic digital video cameras were utilized to film test no. STBR-1. Six AOS high-speed digital video cameras, ten GoPro digital video cameras, and four Panasonic digital video cameras were utilized to film test no. STBR-2. Six AOS high-speed digital video cameras, six GoPro digital video cameras, and four Panasonic digital video cameras were utilized to film test no. STBR-3. Six AOS high-speed digital video cameras, ten GoPro digital video cameras, and four Panasonic digital video cameras were utilized to film test no. STBR-4. Camera details, camera operating speeds, lens information, and a schematic of the camera locations relative to the system are shown in Figures 126, 127, 128, and 129, respectively.

The high-speed videos were analyzed using TEMA Motion and Redlake MotionScope software programs. Actual camera speed and camera divergence factors were considered in the analysis of the high-speed videos. A digital still camera was also used to document pre- and post-test conditions for all tests.



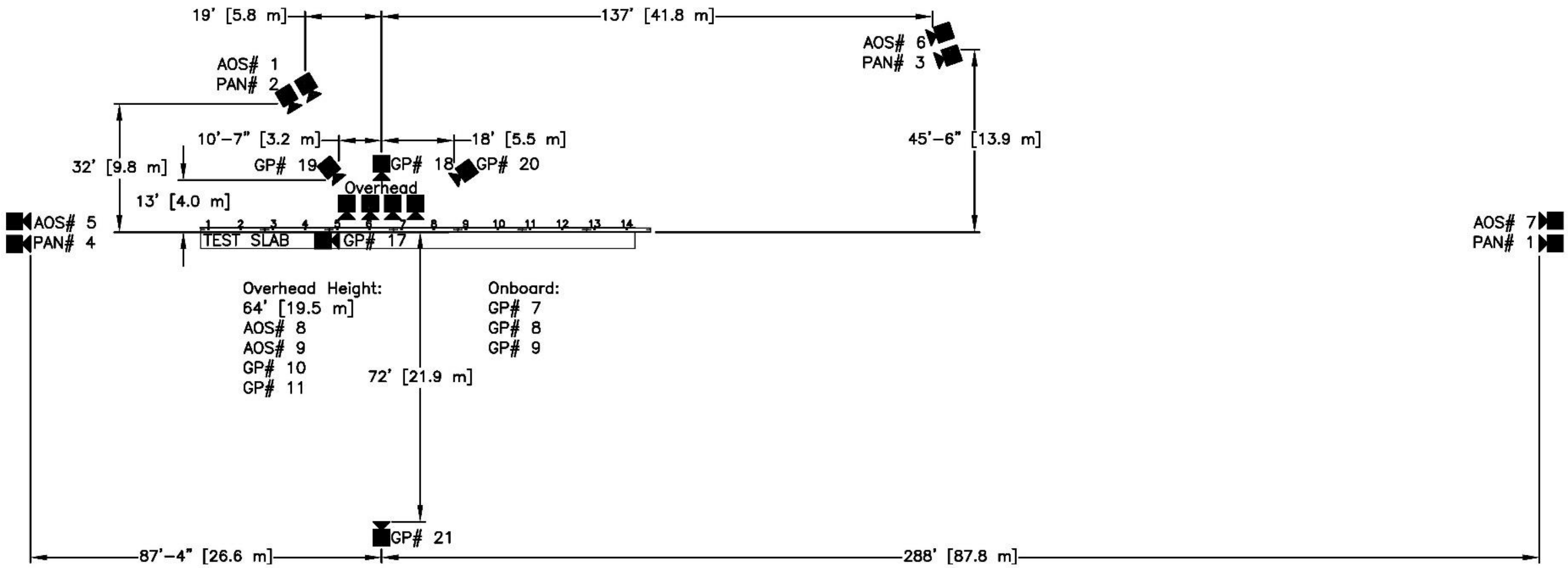
No.	Type	Operating Speed (frames/sec)	Lens	Lens Setting
AOS-1	AOS Vitcam CTM	500	Kowa 16 mm Fixed	
AOS-5	AOS X-PRI	500	100 mm Fixed	
AOS-6	AOS X-PRI	500	Fujinon 75 mm Fixed	
AOS-7	AOS X-PRI	500	Fujinon 50 mm Fixed	
AOS-9	AOS TRI-VIT	500	Kowa 12 mm Fixed	
GP-7	GoPro Hero 4	120		
GP-8	GoPro Hero 4	120		
GP-9	GoPro Hero 4	120		
GP-10	GoPro Hero 4	120		
GP-11	GoPro Hero 4	240		
GP-16	GoPro Hero 4	240		
GP-17	GoPro Hero 4	120		
GP-18	GoPro Hero 6	240		
GP-20	GoPro Hero 6	240		
GP-21	GoPro Hero 6	120		
PAN-1	Panasonic HC-V770	120		
PAN-2	Panasonic HC-V770	120		
PAN-3	Panasonic HC-V770	120		
PAN-4	Panasonic HC-V770	120		

Figure 126. Camera Locations, Speeds, and Lens Settings, Test No. STBR-1



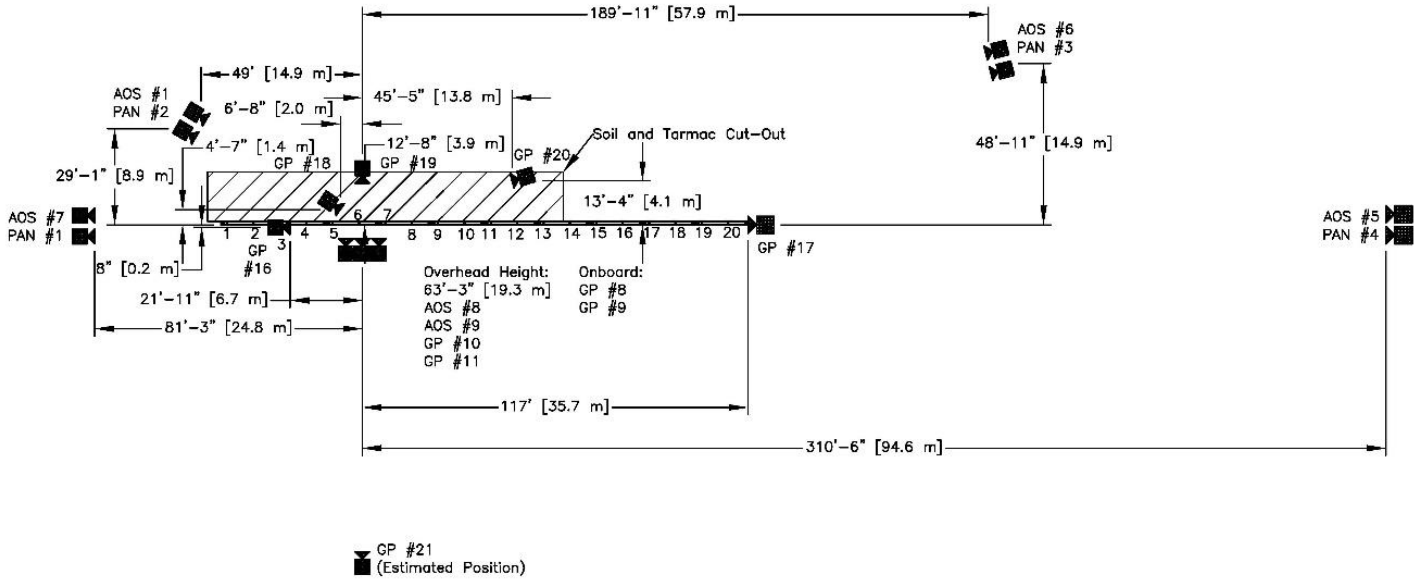
No.	Type	Operating Speed (frames/sec)	Lens	Lens Setting
AOS-1	AOS Vitcam	500	Kowa 25 mm	
AOS-5	AOS X-PRI	500	100 mm	
AOS-6	AOS X-PRI	500	Fujinon 50 mm	
AOS-7	AOS X-PRI	500	Fujinon 75 mm	
AOS-8	AOS S-VIT	500	Kowa 16 mm	
AOS-9	AOS TRI-VIT	500	Kowa 12 mm	
GP-7	GoPro Hero 4	120		
GP-8	GoPro Hero 4	120		
GP-9	GoPro Hero 4	120		
GP-10	GoPro Hero 4	120		
GP-11	GoPro Hero 4	240		
GP-17	GoPro Hero 4	240		
GP-18	GoPro Hero 6	240		
GP-19	GoPro Hero 6	240		
GP-20	GoPro Hero 6	240		
GP-21	GoPro Hero 6	240		
PAN-1	Panasonic HC-V770	120		
PAN-2	Panasonic HC-V770	120		
PAN-3	Panasonic HC-V770	120		
PAN-4	Panasonic HC-V770	120		

Figure 127. Camera Locations, Speeds, and Lens Settings, Test No. STBR-2



No.	Type	Operating Speed (frames/sec)	Lens	Lens Setting
AOS-1	AOS Vitcam CTM	500	Kowa 25 mm	
AOS-5	AOS X-PRI Gigabit	500	100 mm	
AOS-6	AOS X-PRI Gigabit	500	Fujinon 75 mm	
AOS-7	AOS X-PRI Gigabit	500	Fujinon 50 mm	
AOS-8	AOS S-VIT	500	Kowa 16 mm	
AOS-9	AOS TRI-VIT	500	Kowa 12 mm	
GP-10	GoPro Hero 4	120		
GP-11	GoPro Hero 4	240		
GP-17	GoPro Hero 4	240		
GP-19	GoPro Hero 6	240		
GP-20	GoPro Hero 6	240		
GP-21	GoPro Hero 6	120		
PAN-1	Panasonic HC-V770	120		
PAN-2	Panasonic HC-V770	120		
PAN-3	Panasonic HC-V770	120		
PAN-4	Panasonic HC-V770	120		

Figure 128. Camera Locations, Speeds, and Lens Settings, Test No. STBR-3



No.	Type	Operating Speed (frames/sec)	Lens	Lens Setting
AOS-1	AOS Vitcam CTM	500	Kowa 25 mm	
AOS-5	AOS X-PRI Gigabit	500	100 mm	
AOS-6	AOS X-PRI Gigabit	500	Fujinon 35 mm	
AOS-7	AOS X-PRI Gigabit	500	Fujinon 50 mm	
AOS-8	AOS S-VIT	500	Kowa 16 mm	
AOS-9	AOS TRI-VIT	500	Kowa 12 mm	
GP-8	GoPro Hero 4	120		
GP-9	GoPro Hero 4	120		
GP-10	GoPro Hero 4	120		
GP-11	GoPro Hero 4	240		
GP-16	GoPro Hero 6	240		
GP-17	GoPro Hero 6	240		
GP-18	GoPro Hero 6	240		
GP-19	GoPro Hero 6	240		
GP-20	GoPro Hero 6	240		
GP-21	GoPro Hero 6	240		
PAN-1	Panasonic HC-V770	120		
PAN-2	Panasonic HC-V770	120		
PAN-3	Panasonic HC-V770	120		
PAN-4	Panasonic HC-V770	120		

Figure 129. Camera Locations, Speeds, and Lens Settings, Test No. STBR-4

11 CONSTRUCTION DETAILS - TEST NO. STBR-1

The test installation for the bridge rail system consisted of steel rails, posts assemblies, post-to-rail and rail-to-rail connections, as well as a surrogate concrete bridge deck, as shown in Figures 130 through 154. The total length of the bridge rail was 159 ft – 11¼ in. long (48.7 m). Photographs of the test installation are shown in Figures 155 through 159. Material specifications, mill certifications, and certificates of conformity for the system materials are shown in Appendix D.

The system was constructed with twenty galvanized ASTM A992, W6x15 (W150x22.5) steel post assemblies spaced on 96-in. (2,438-mm) centers. Post assembly nos. 1 through 13 were side-mounted to the vertical side edge of the surrogate, reinforced-concrete bridge deck. For the construction of the surrogate concrete bridge deck, the threaded rod and coupling nuts were held in place to the embedded vertical plates by placing bolts through the formwork, rather than utilizing the option of welding the coupling nuts to the embedded vertical plates, as shown in Figure 150. Post assembly nos. 14 through 20 were surface-mounted to the top of existing concrete tarmac, which provided the necessary system length for vehicle redirection and were used for testing-purposes only.

Post assembly nos. 1 through 13 were 58⅞ in. (1,495 mm) long. An ASTM A572 Grade 50 steel plate PL 8-in. x 8-in. x ⅜-in. (PL 203-mm x 203-mm x 10-mm) was attached to the top of each post assembly with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Similarly, an ASTM A572 Grade 50 steel, vertical plate PL 13-in. x 17¾-in. x 1-in. (PL 330-mm x 451-mm x 25-mm) was attached to bottom of the front flange of each post with all-around ¼-in. (6.4-mm) fillet welds. Four gusset plates, fabricated with ASTM A572 Grade 50 steel plate, measuring PL 6⅛ -in. x 5$^{11}/_{16}$-in. x ¼-in. (PL 156-mm x 144-mm x 6-mm), were welded to the top and bottom of the vertical plates, inner faces of post flanges, and web with all-around ¼-in. (6.4-mm) fillet welds. Post assembly nos. 1 through 13 were bolted to the tension and compression sides of the vertical plates with ASTM A500 Grade 50 horizontal spacer tubes, which were specified as HSS 5-in. x 4-in. x ½-in. (HSS 127-mm x 102-mm x 13-mm) sections.

Post assembly nos. 1 through 13 were bolted to the horizontal spacer tubes using ASTM F3125 Grade A325 1-in. (25.4-mm) diameter by 3½-in. (88.9-mm) long, heavy hex-head bolts with ¼-in. (6.4-mm) thick, ASTM A36 steel square washers and 1-in. (25.4-mm) diameter ASTM A563DH heavy hex nuts. The deck anchorage in the tension region consisted of two 1-in. (25.4-mm) diameter by 32¾-in. (832-mm) long, ASTM F1554 Grade 105 all-thread anchor rods with ASTM A563DH heavy hex coupling nuts, and ASTM A563DH heavy hex nuts. The deck anchorage in the compression region consisted of two 1-in. (25.4-mm) diameter by 1½-in. (38.1-mm) long, ASTM A449 anchor bolts with ¼-in. (6.4-mm) thick, 3-in. (76-mm) ASTM A36 steel square washers, and ASTM A563DH heavy hex coupling nuts. A $^{3}/_{16}$-in. (4.8-mm) thick, vertical embedment plate was used at every post location.

Post assembly nos. 14 through 20 were 32 in. (813 mm) long. Post assembly nos. 14 through 20 consisted of three parts – a base plate, a top plate, and a vertical post. The top plate consisted of an ASTM A572 Grade 50 steel plate measuring PL 8-in. x 8-in. x ⅜-in. (PL 203-mm x 203-mm x 10-mm) with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Similarly, the bottom plate consisted of an ASTM A572 Grade 50 steel plate measuring PL 12-in. x 12-in. x ¾-in. (PL 305-mm x 305-mm x 19-mm) with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Finally, the post was fabricated with ASTM A992 W6x15 (W150x22.5) sections measuring 30⅞ in. (784 mm) long. The post assembly nos. 14 through 20 were anchored to the existing tarmac with four ¾-in. (19.1-mm) diameter by 12-in. (305-mm) long ASTM F1554 Grade 36 all-thread anchor rods with ¼-in. (6.4-mm) thick, ASTM A36 steel square washers, and ASTM A563DH heavy hex nuts.

The three rail elements consisted on an ASTM A500 Grade C HSS 12-in. x 4-in. x ¼-in. (HSS 304.8-mm x 101.6-mm x 6.4-mm) section for the top rail and ASTM A500 Grade C HSS 8-in. x 6-in. x ¼-in. (HSS 203.2-mm x 152.4-mm x 6.4-mm) section for the lower two rails. Rail-to-rail connections were located 2 ft (610 mm) downstream from every other post location. The top rails were attached to the post assemblies with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long, ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom rails were attached to the front flanges of the posts with two staggered ¾-in. (19-mm) diameter by 7½-in. (191-mm) long ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

The splice tube for the top rails consisted of two horizontal PL 30-in. x 10⅝-in. x $^{5}/_{16}$-in. (PL 762-mm x 270-mm 8-mm) and two vertical PL 30-in. x 2⅝-in. x ⅜-in. (PL 762-mm x 67-mm x 10-mm) attached with ¼-in. (6.4-mm) fillet welds. The splice tubes for the middle and bottom rails consisted on two vertical PL 30-in. x 6⅝-in. x ⅜-in. (PL 762-mm x 168-mm 10-mm) and two horizontal PL 30-in. x 4⅝-in. x $^{5}/_{16}$-in. (PL 762-mm x 117-mm x 8-mm) attached with ¼-in. (6.4-mm) fillet welds. The top splice tubes were attached to the top rail end sections with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long, ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom splice tubes were attached to the rail end sections with two ¾-in. (19-mm) diameter by 9½-in. (241-mm) long, ASTM A449 hex-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

After test no. STBR-1, post nos. 5, 6, 7, and 8, the nearest two railing elements for each of the three rails, and the three splice tube location connecting these rails were replaced for test no. STBR-2.

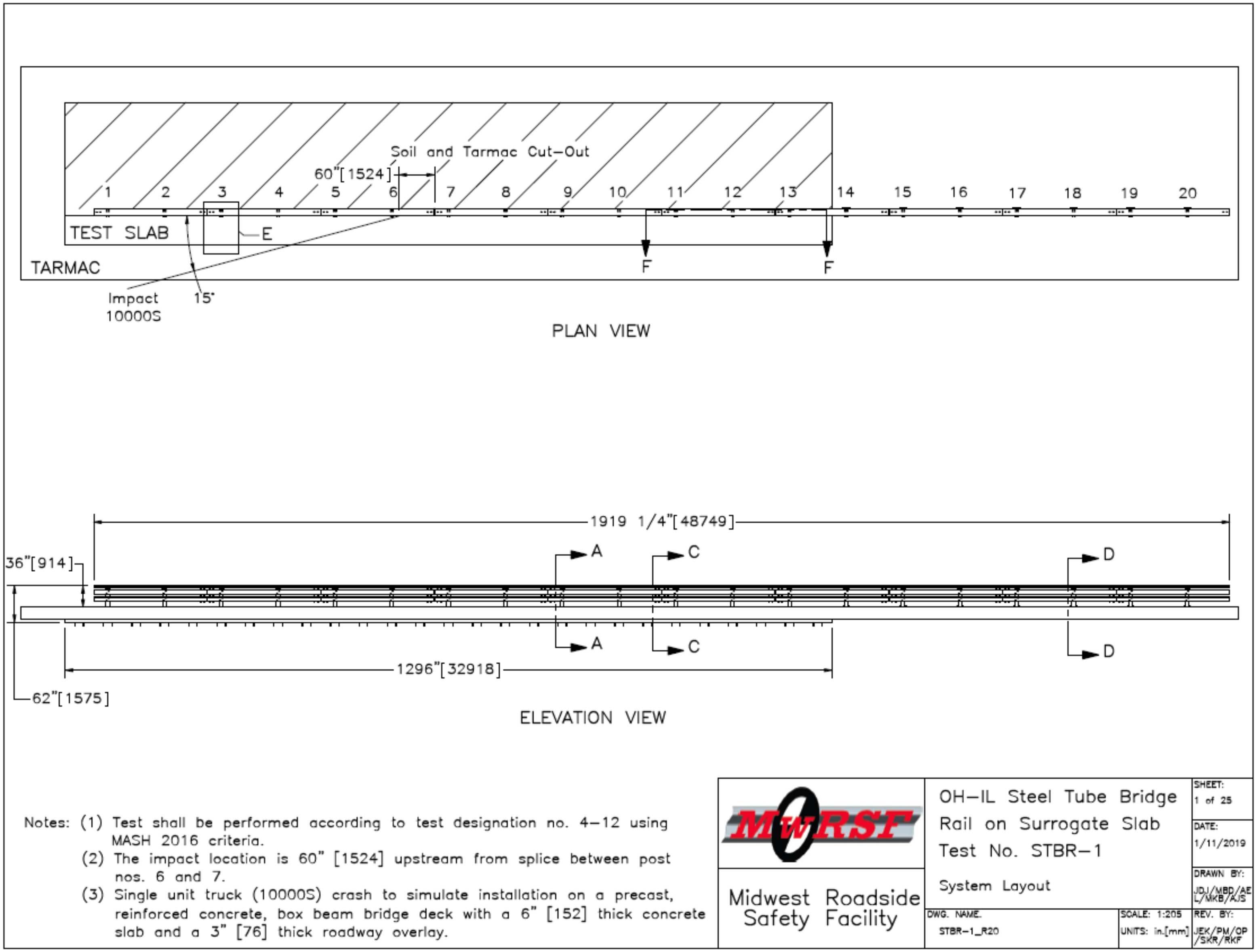


Figure 130. System Layout and Impact Location, Test No. STBR-1

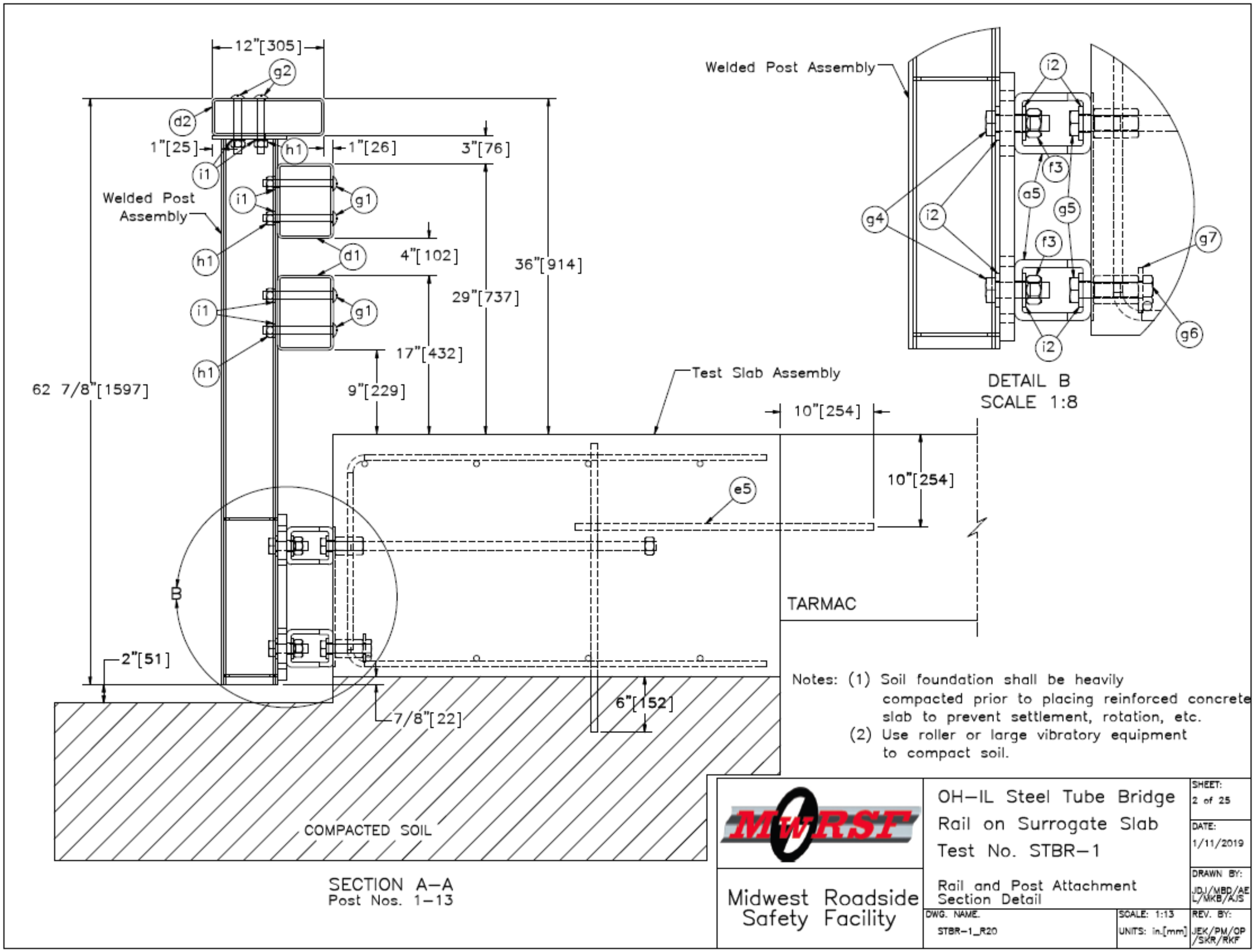


Figure 131. Rail and Post Attachment Section Detail, Test No. STBR-1

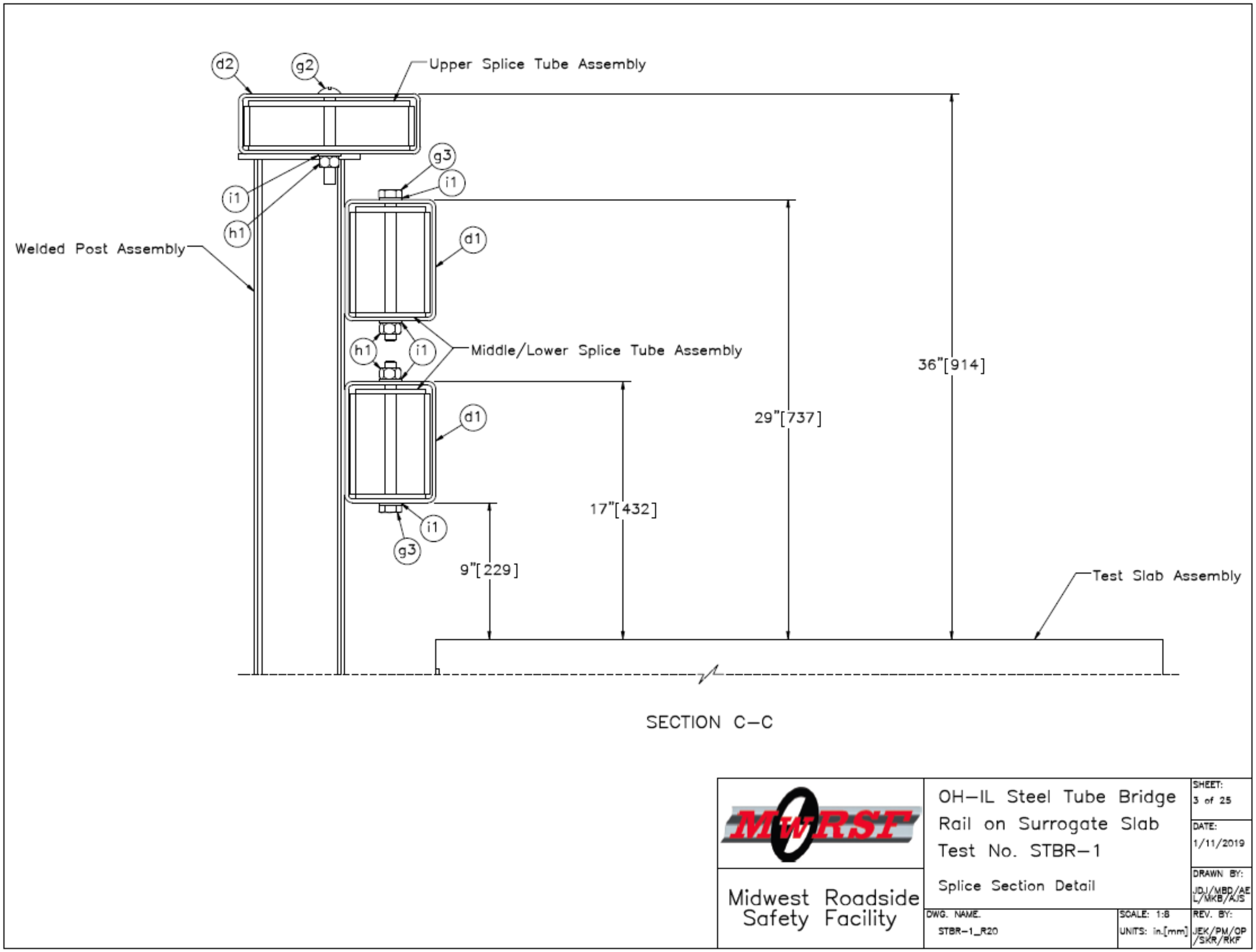


Figure 132. Splice Section Detail, Test No. STBR-1

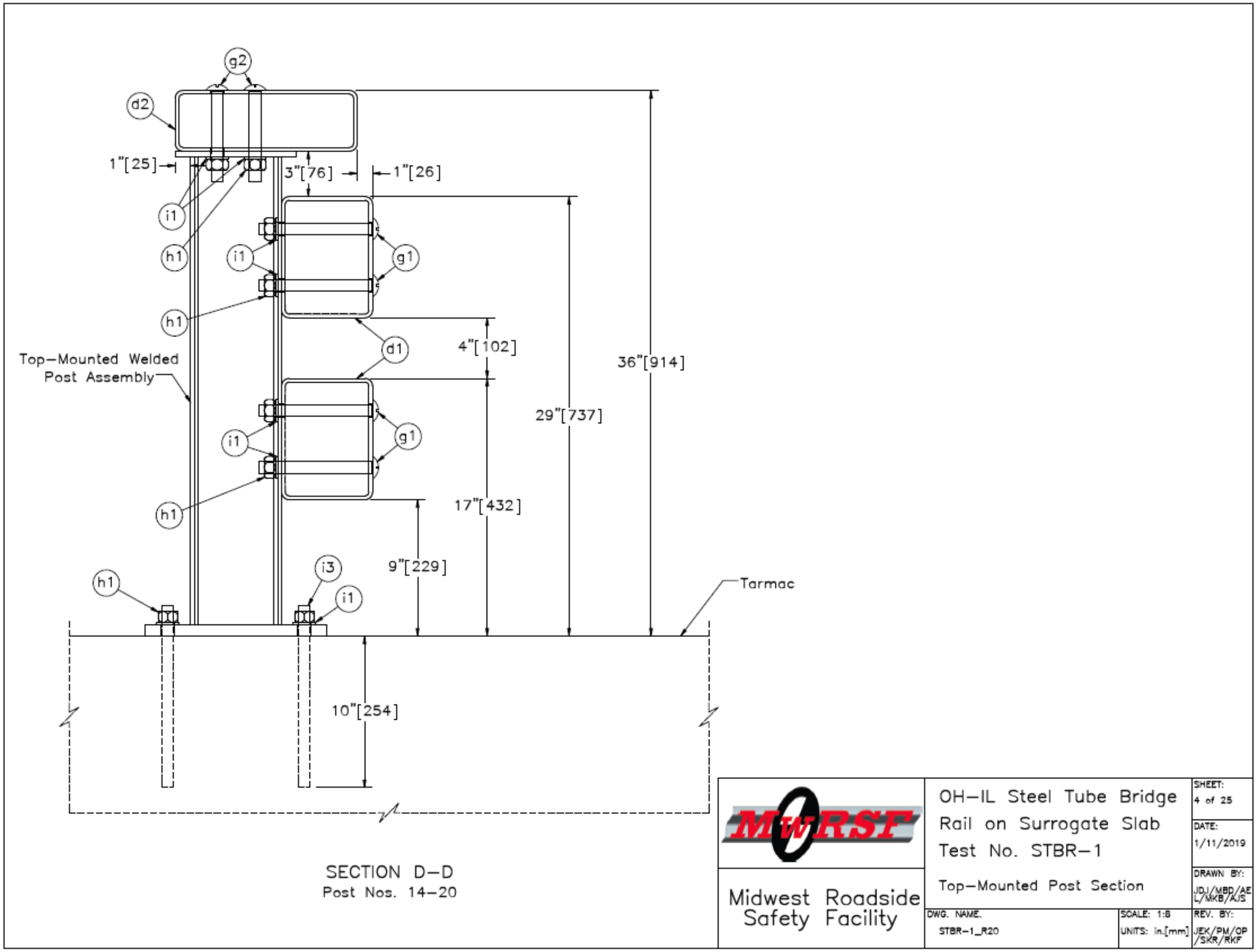


Figure 133. Top-Mounted Post Section, Test No. STBR-1

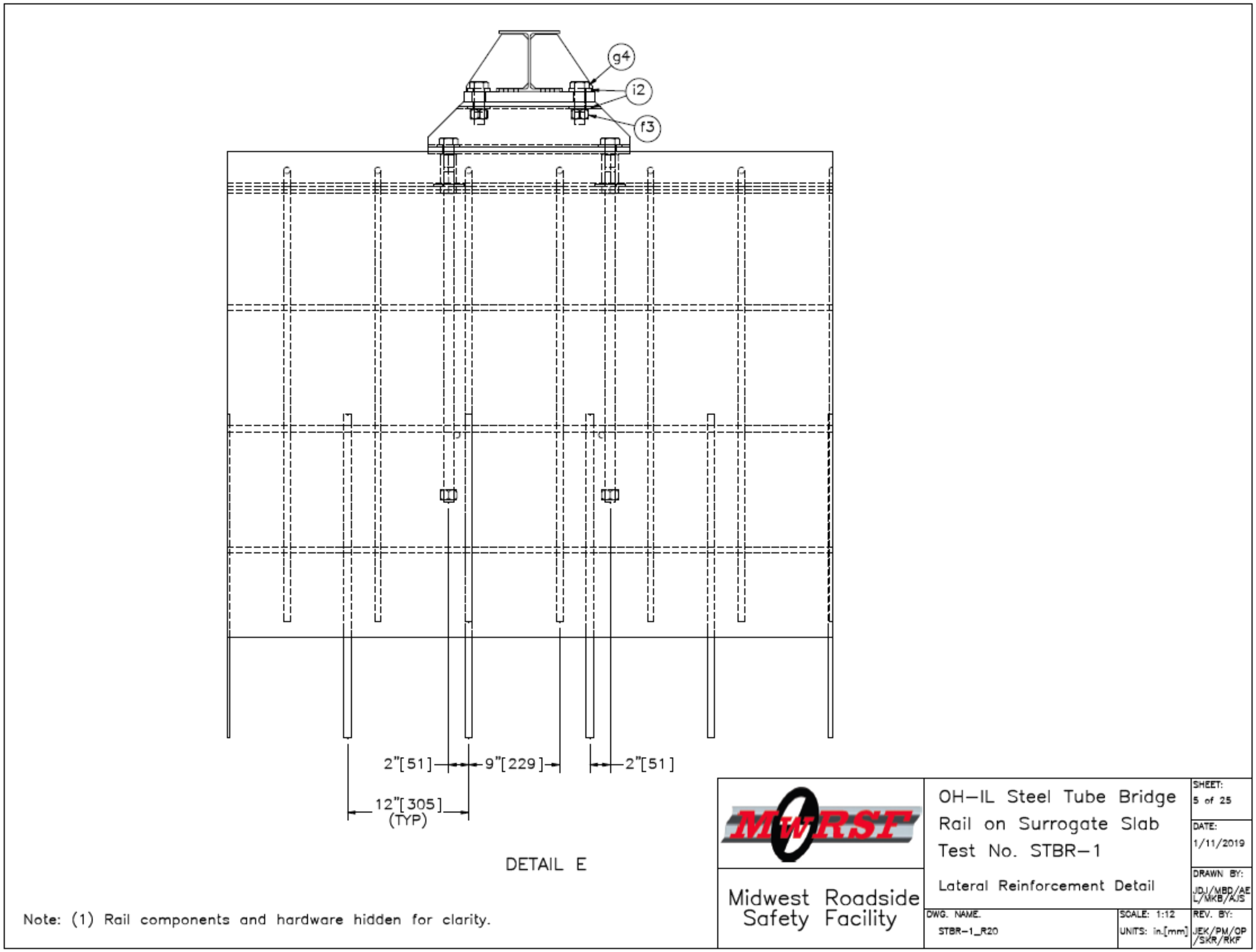


Figure 134. Lateral Reinforcement Detail, Test No. STBR-1

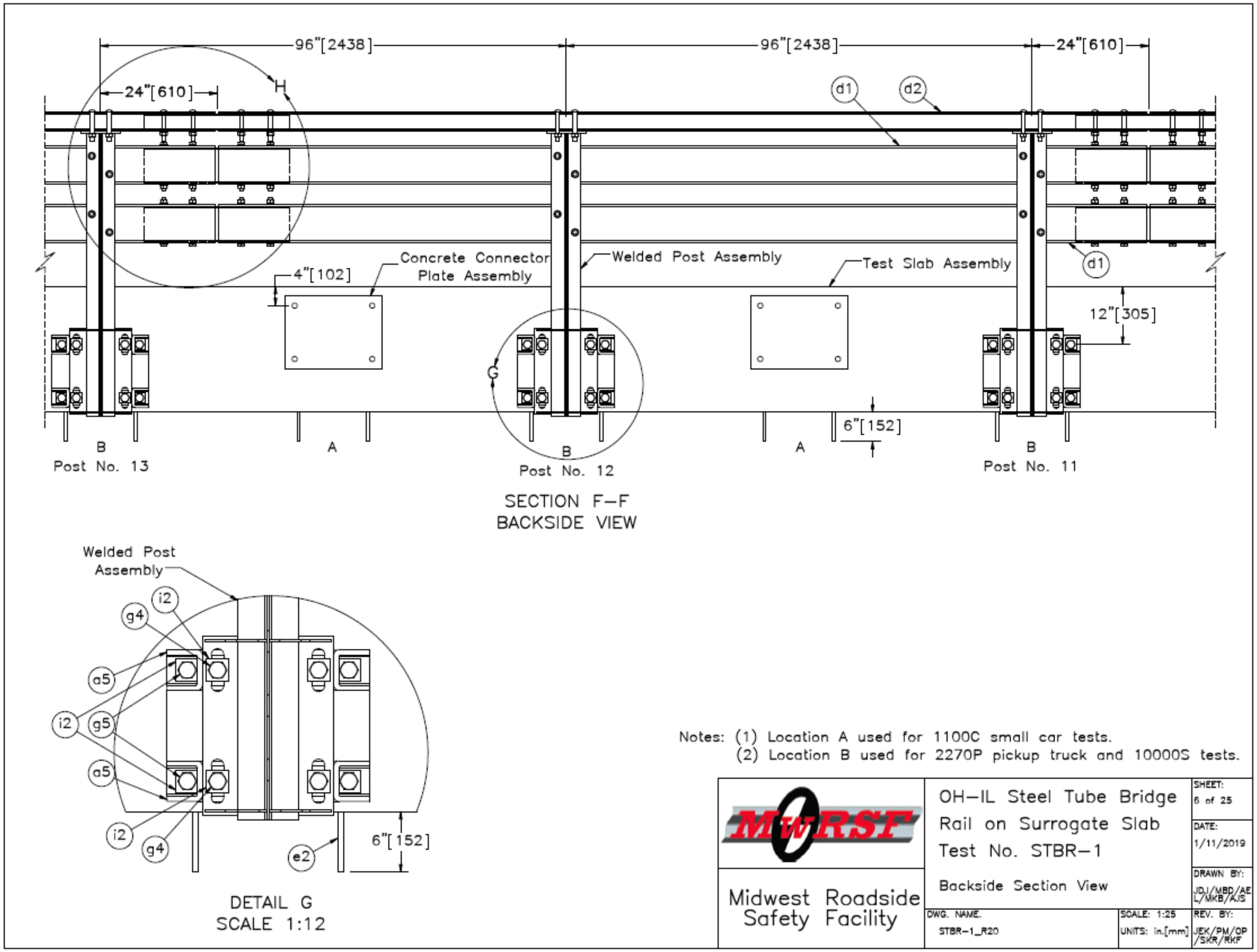


Figure 135. Backside Section View, Test No. STBR-1

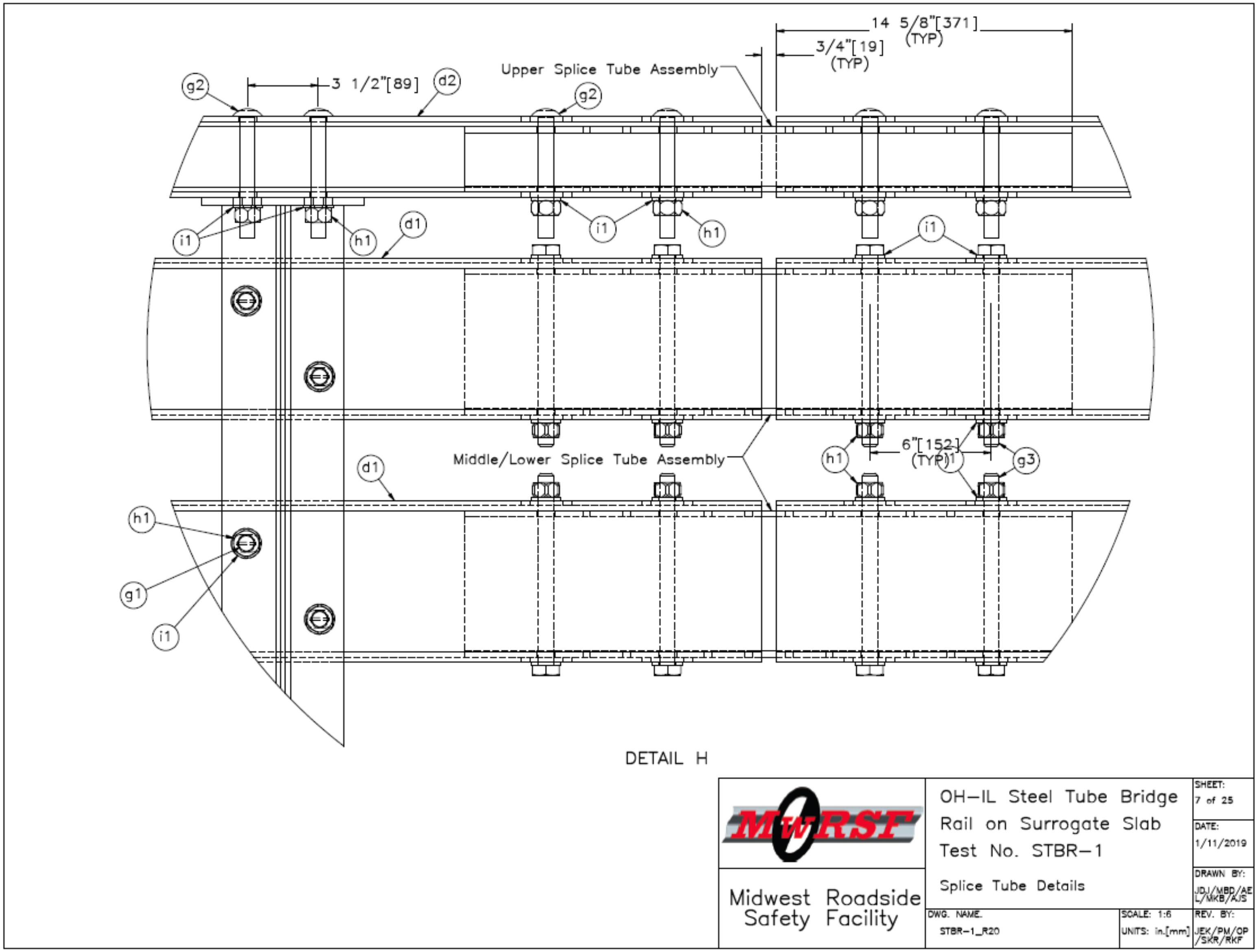
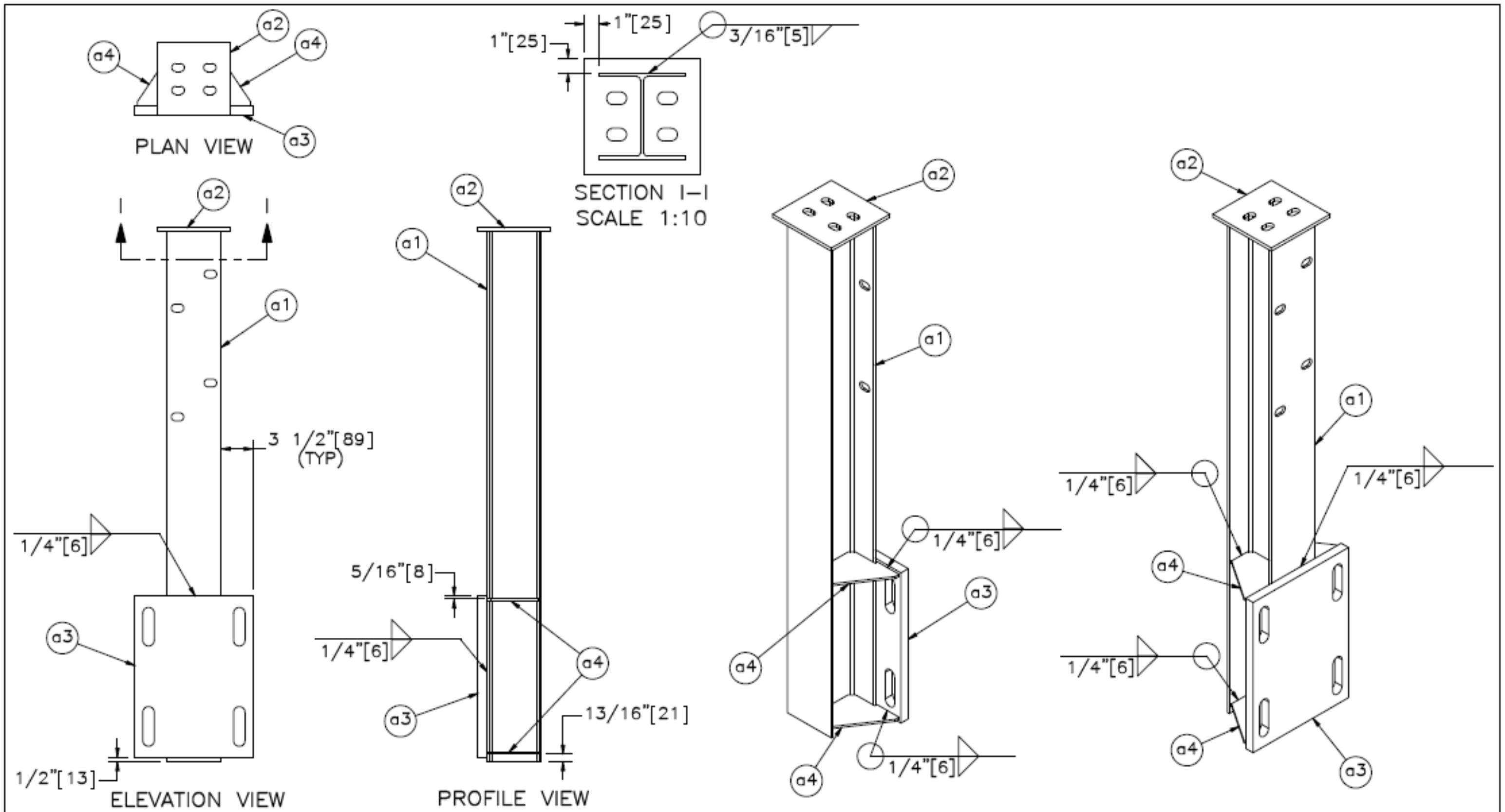


Figure 136. Splice Tube Section Details, Test No. STBR-1



Notes: (1) Gussets on both sides of post.
(2) To prevent welds to end at edges of gussets, leave 5/16" [8] of base material beyond weld to edge of welded part.
(3) Leave coped corners of gussets open for drainage. Do not fill with welds.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	13	Welded Post Assembly	–	ASTM A123
a1	1	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	See Assembly
a2	1	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly
a3	1	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly
a4	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–1

Welded Post Assembly Details

SHEET:	8 of 25
DATE:	1/11/2019
DRAWN BY:	JDJ/MBD/AE L/MKB/AJS
REV. BY:	JEK/PM/OP /SKR/RKF
DWG. NAME.	STBR–1_R20
SCALE:	1:16
UNITS:	in.[mm]

Figure 137. Welded Post Assembly Details, Test No. STBR-1

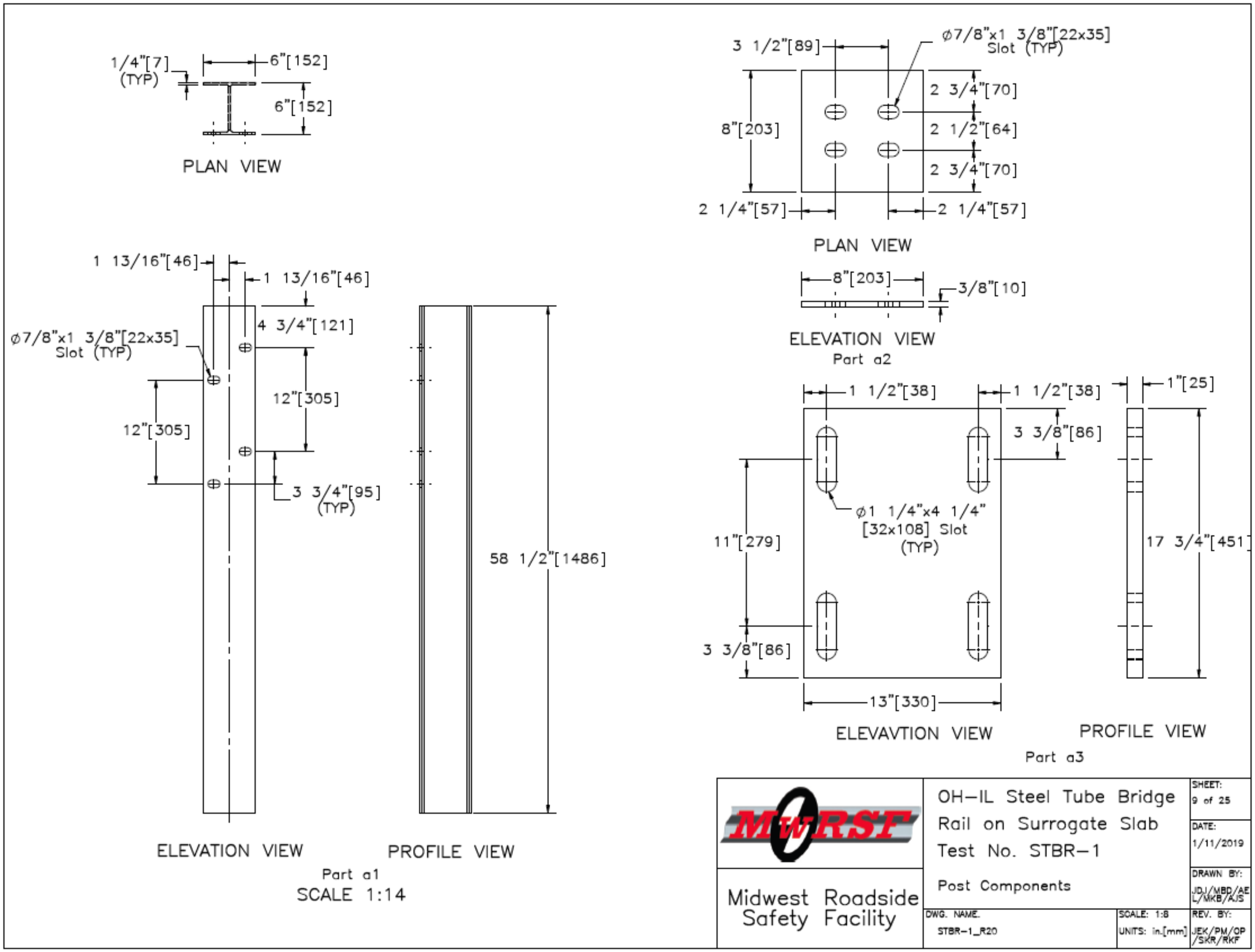


Figure 138. Post Components, Test No. STBR-1

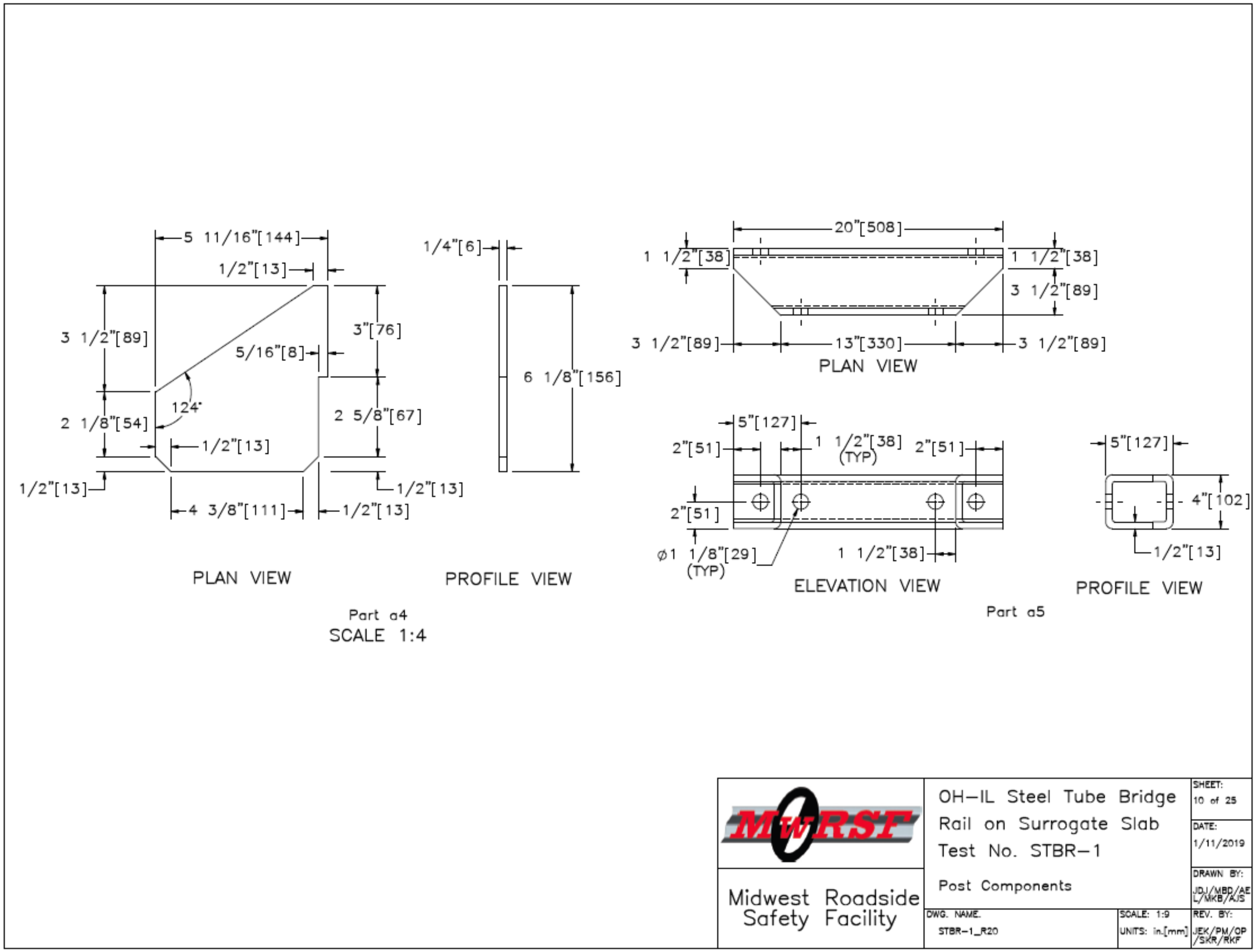
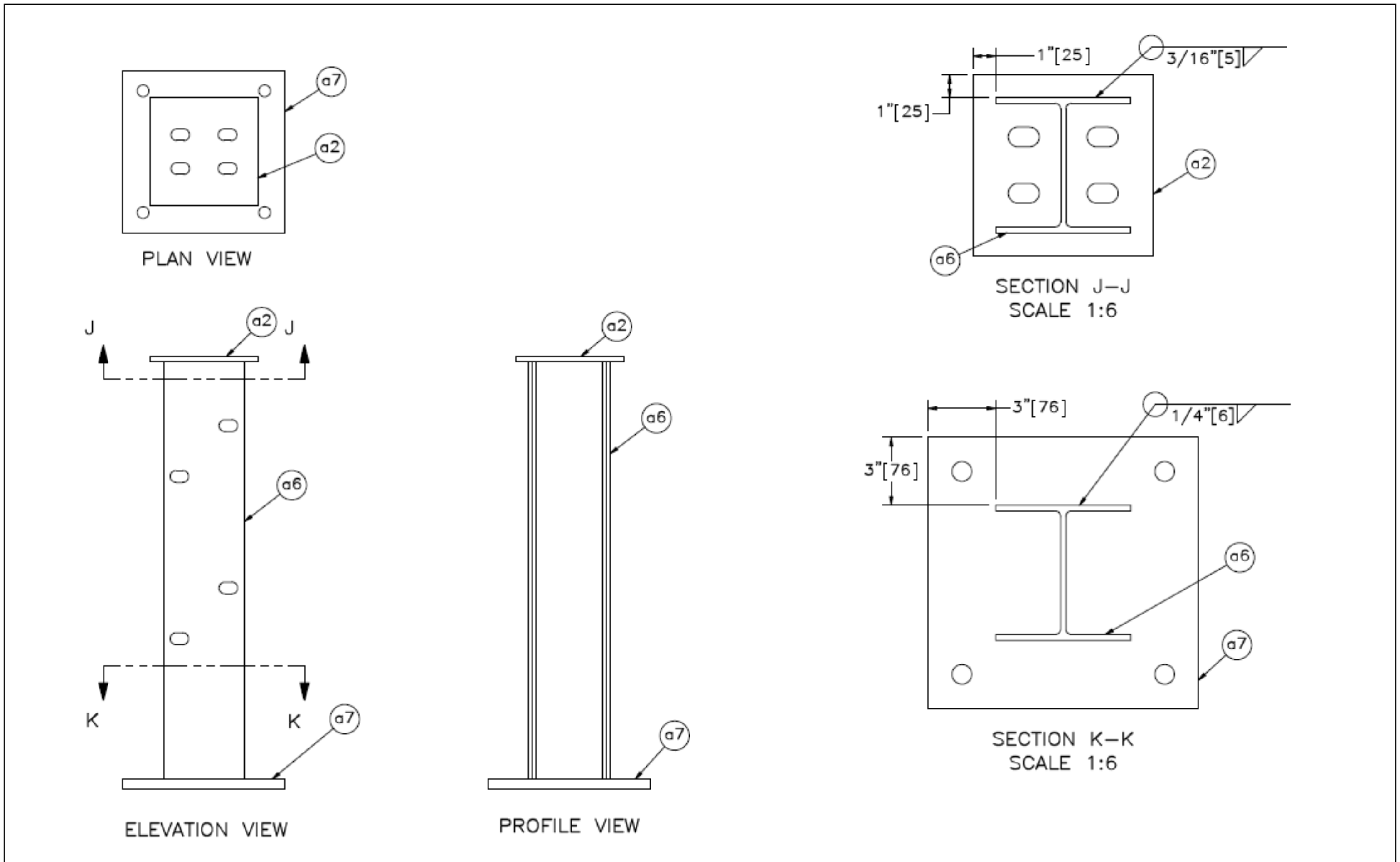


Figure 139. Post Components, Test No. STBR-1



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	7	Top–Mounted Welded Post Assembly	–	ASTM A123
a2	1	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly
a6	1	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	See Assembly
a7	1	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	See Assembly

Midwest Roadside Safety Facility	OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–1	SHEET: 11 of 25
		DATE: 1/11/2019
	Top–Mounted Welded Post Assembly	DRAWN BY: JDJ/MBD/AE L/MKB/AJS
	DWG. NAME. STBR–1_R20	SCALE: 1:10 UNITS: in.[mm]
		REV. BY: JEK/PM/OP /SKR/RKF

Figure 140. Top-Mounted Welded Post Assembly, Test No. STBR-1

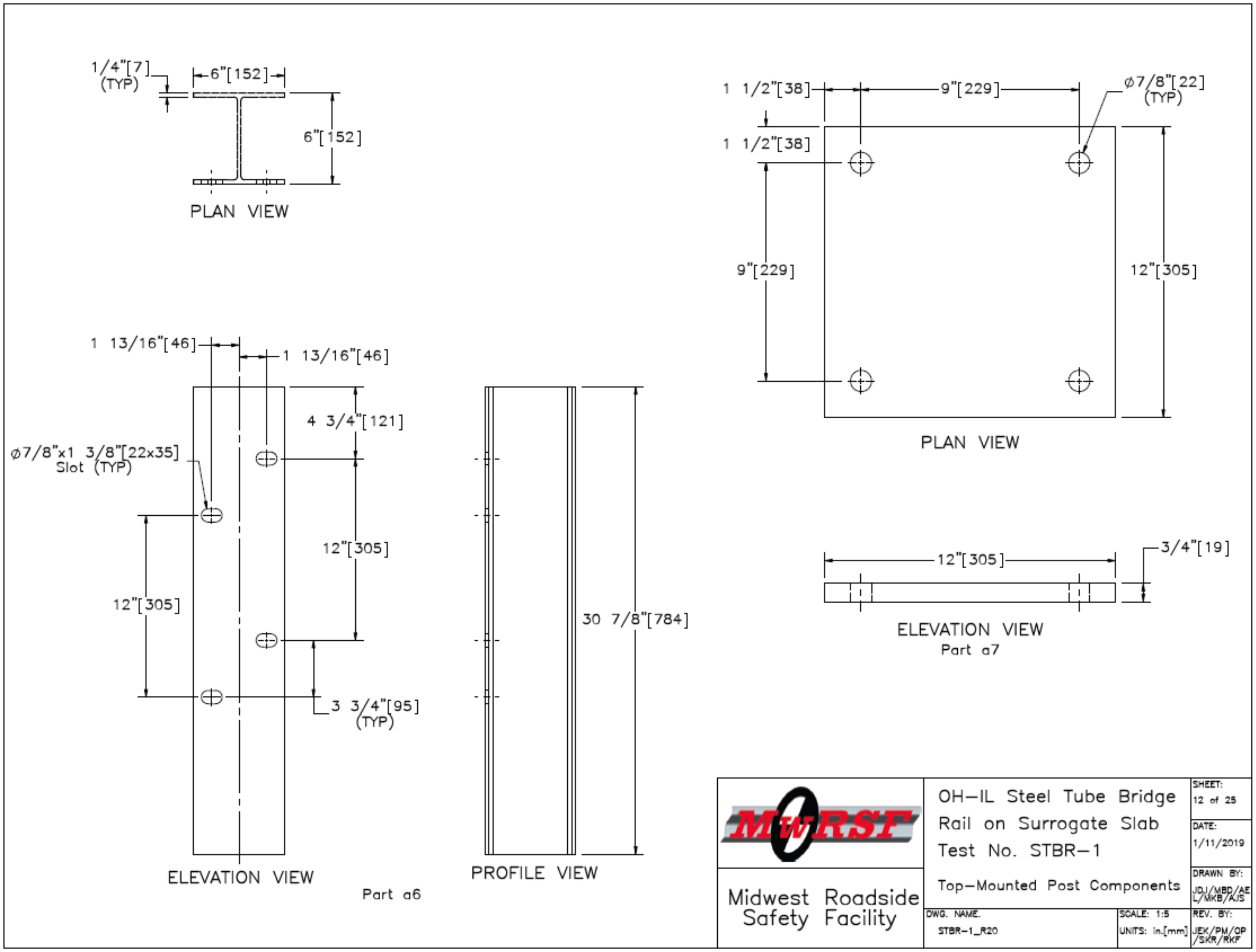
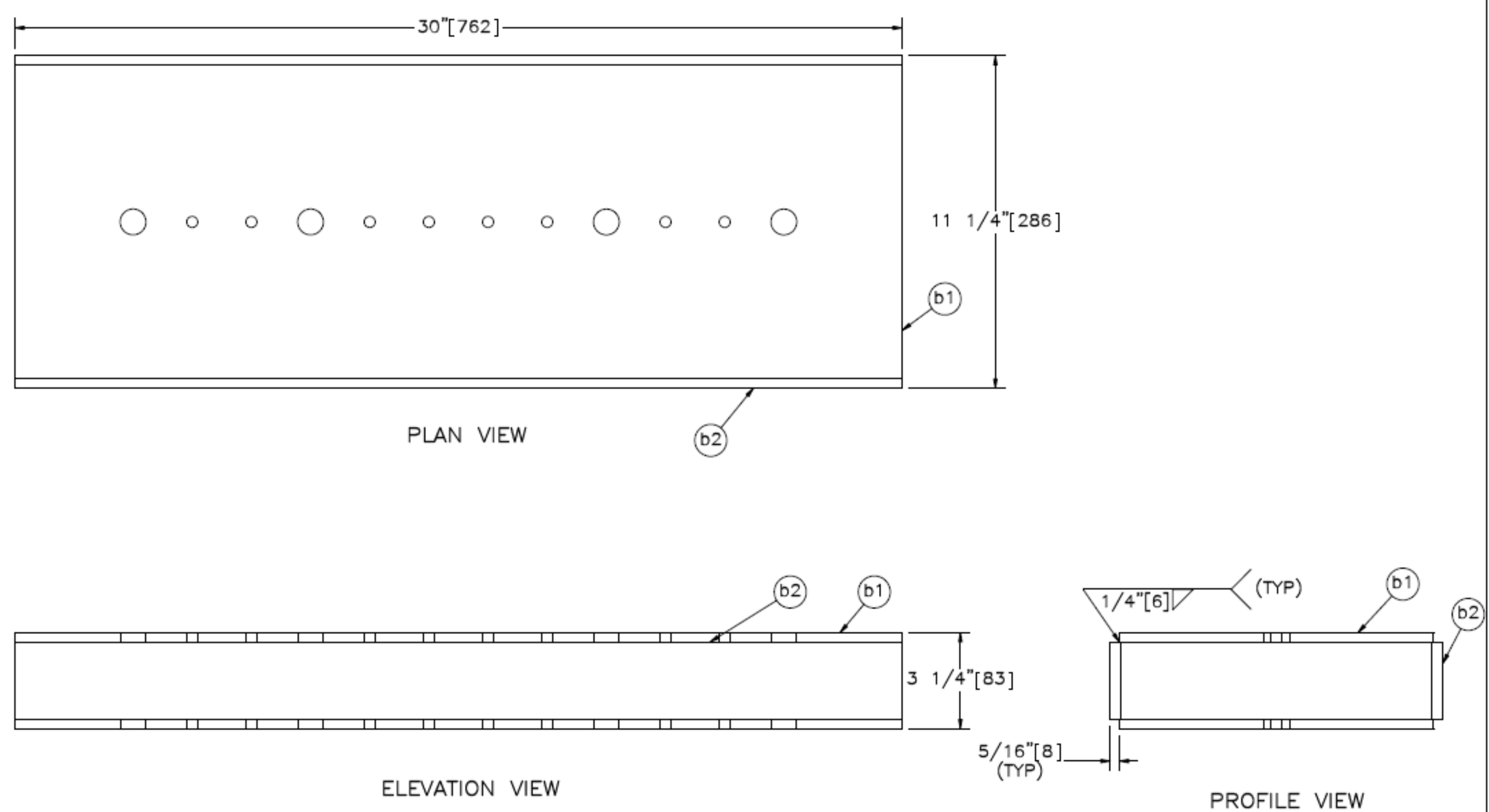


Figure 141. Top-Mounted Post Components, Test No. STBR-1



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	9	Upper Splice Tube Assembly – Option 1	–	ASTM A123
b1	2	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly
b2	2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–1

Upper Splice Tube Assembly

SHEET: 13 of 25
DATE: 1/11/2019
DRAWN BY: JDJ/MBD/AEL/MKB/AJS
DWG. NAME. STBR–1_R20
SCALE: 1:5
UNITS: in.[mm]
REV. BY: JEK/PM/OP/SKR/RKF

Figure 142. Upper Splice Tube Assembly, Test No. STBR-1

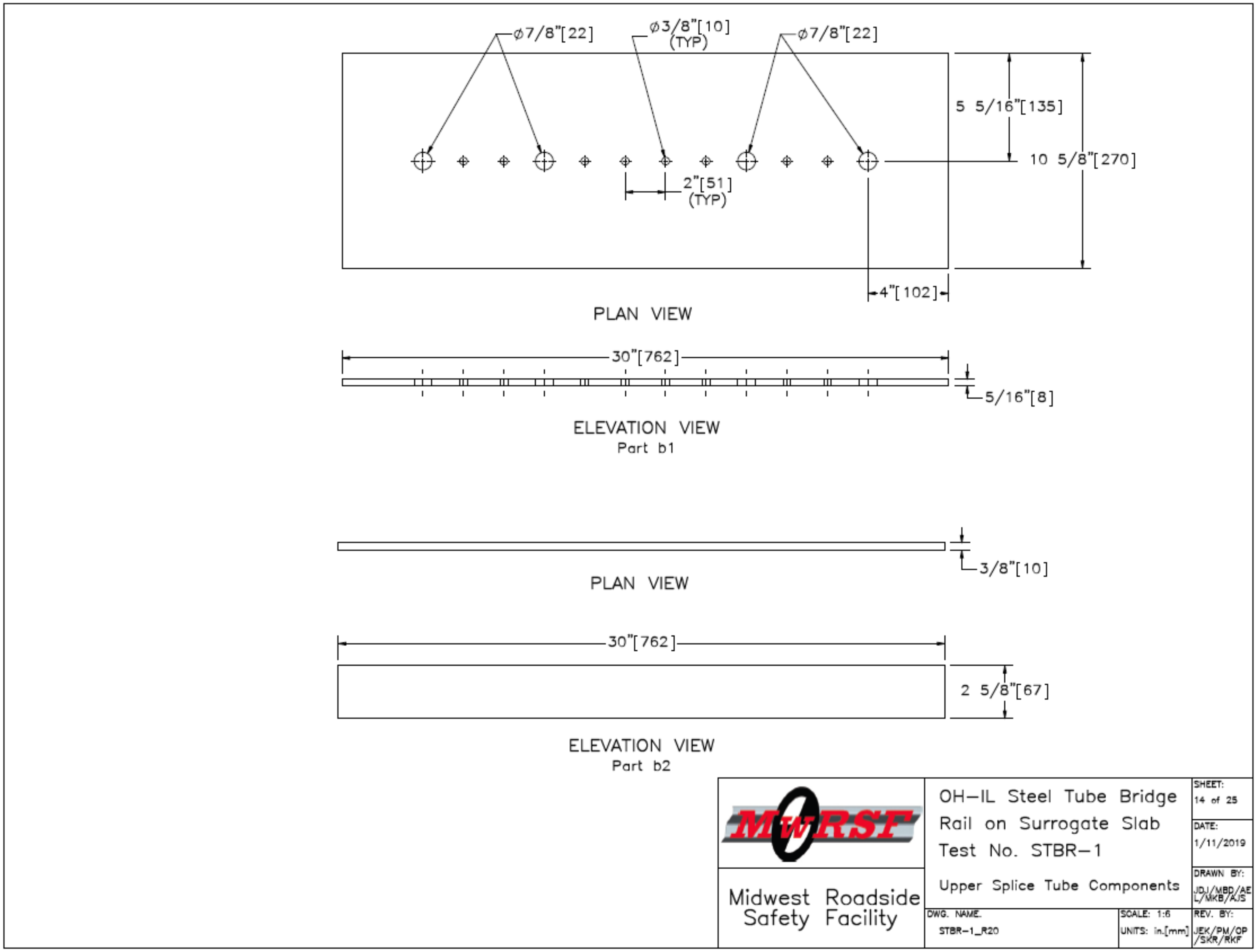
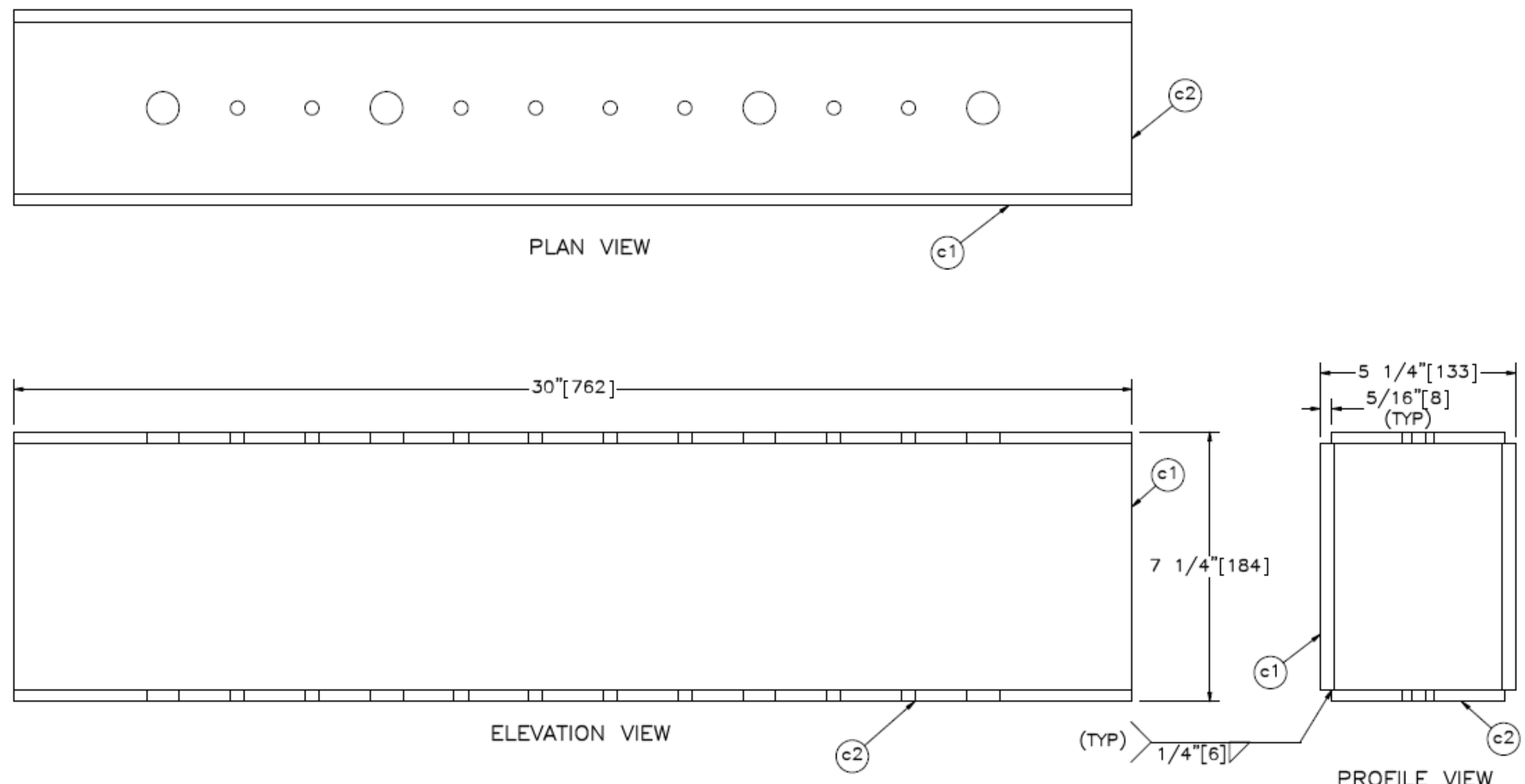


Figure 143. Upper Splice Tube Components, Test No. STBR-1



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	18	Middle/Lower Splice Tube Assembly – Option 1	–	ASTM A123
c1	2	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly
c2	2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–1

Middle/Lower Splice Tube Assembly

SHEET: 15 of 25
DATE: 1/11/2019
DRAWN BY: JDJ/MBD/AEL/MKB/AJS
DWG. NAME. STBR–1_R20
SCALE: 1:4
UNITS: in.[mm]
REV. BY: JEK/PM/OP/SKR/RKF

Figure 144. Middle/Lower Splice Tube Assembly, Test No. STBR-1

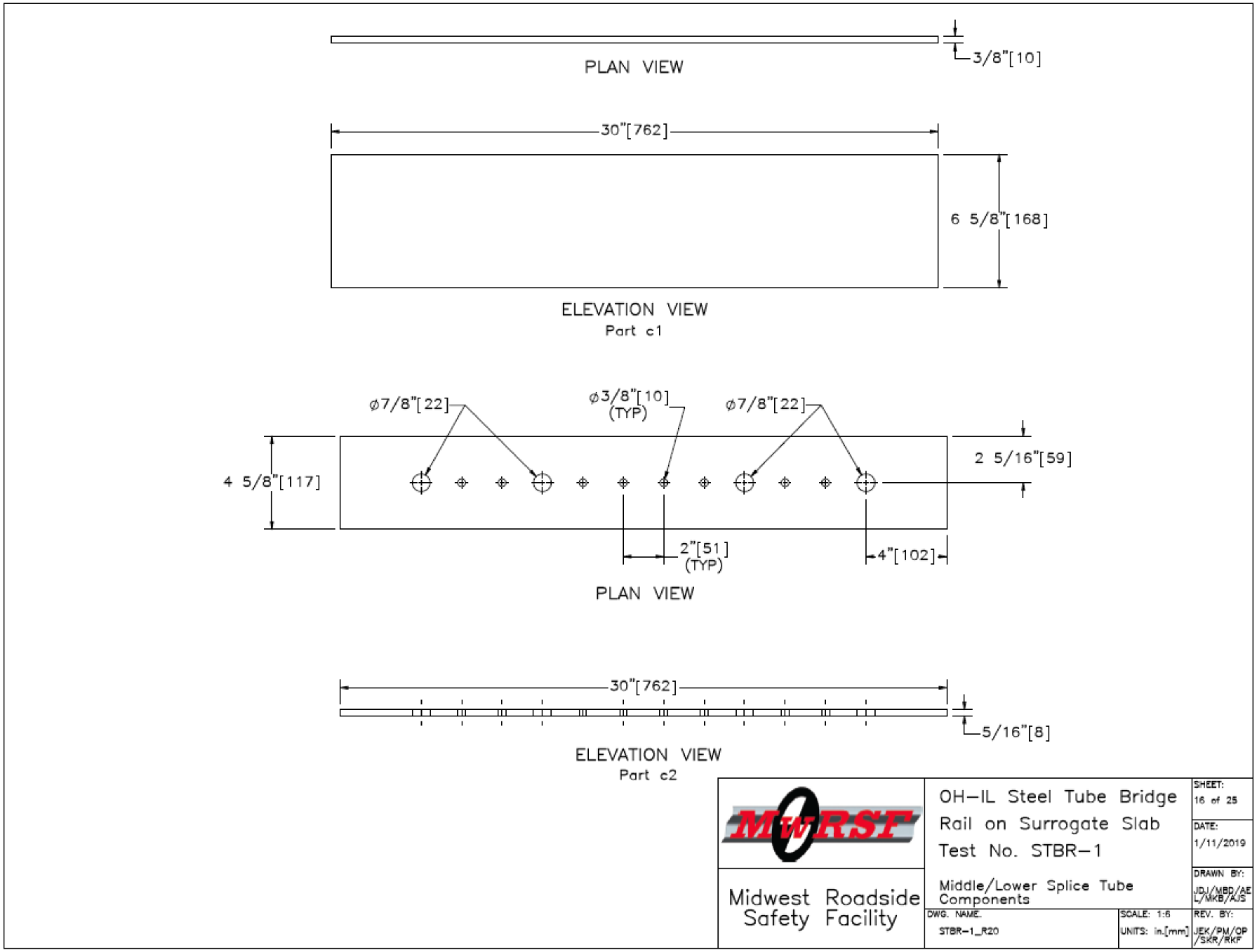


Figure 145. Middle/Lower Splice Tube Components, Test No. STBR-1

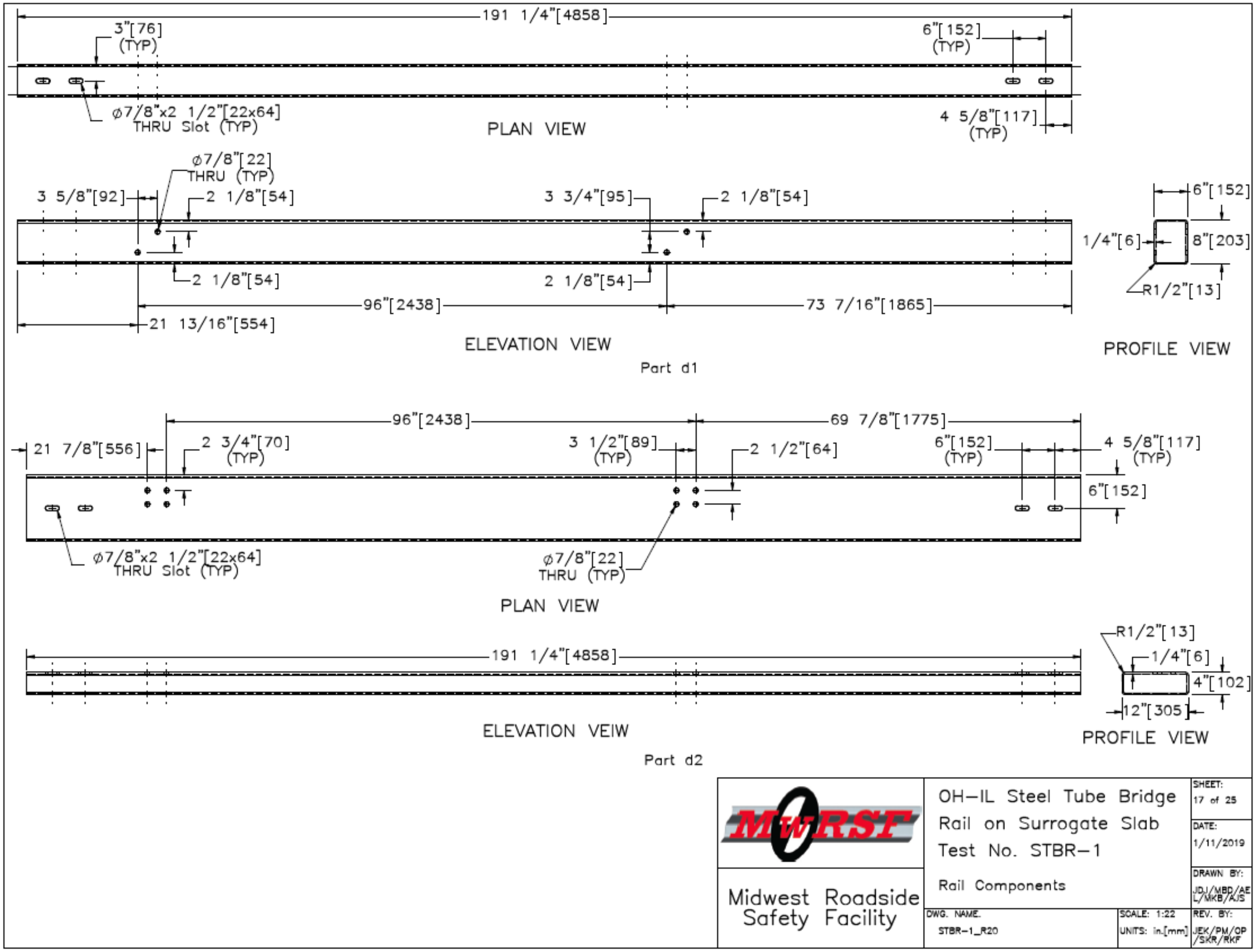


Figure 146. Rail Components, Test No. STBR-1

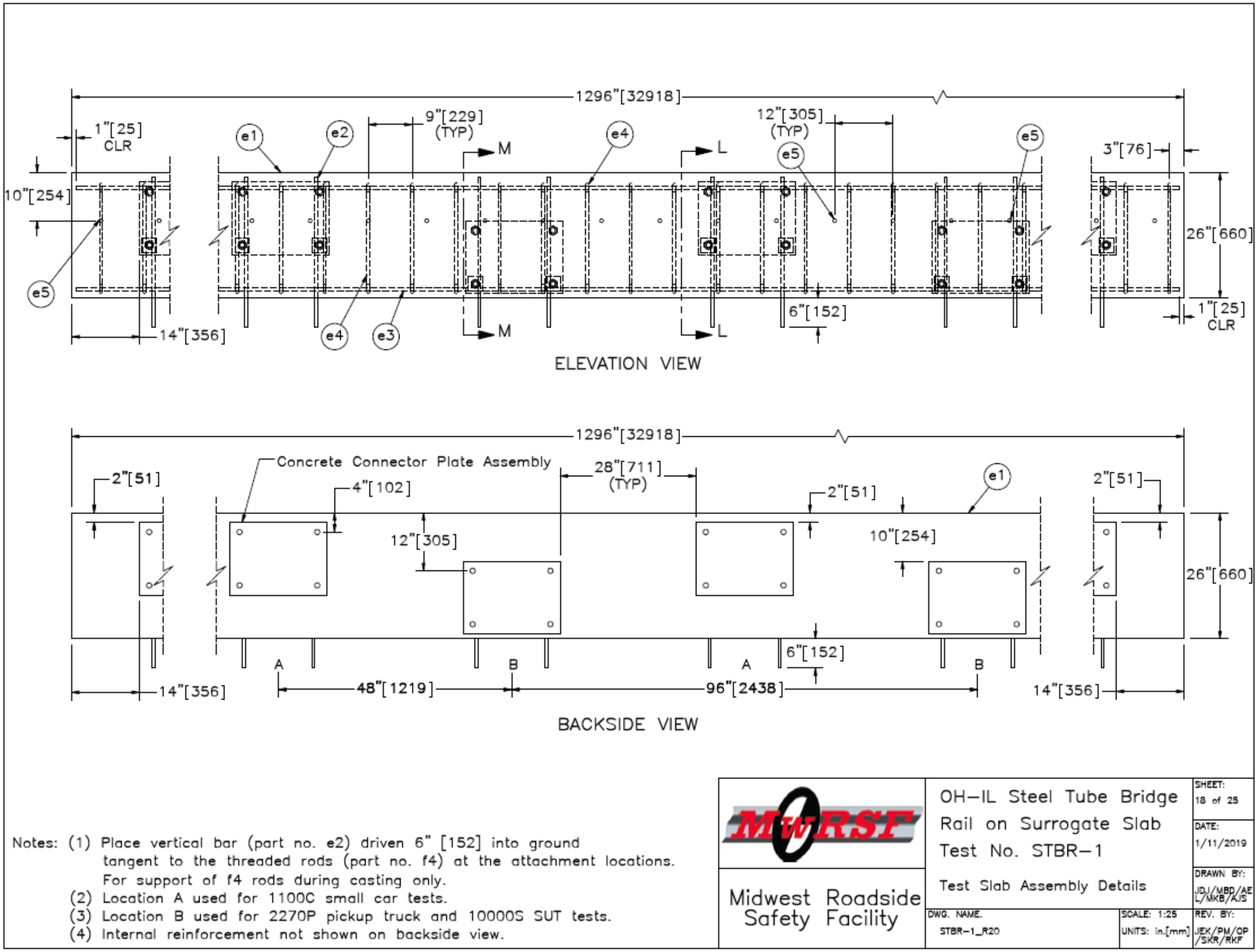
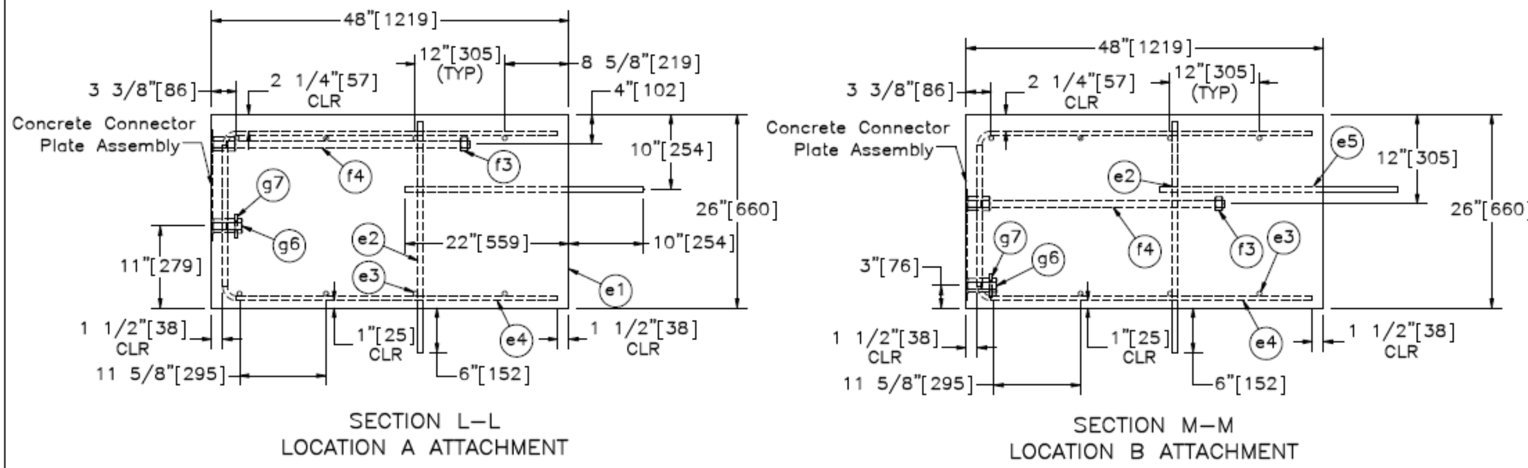


Figure 147. Test Slab Assembly Details, Test No. STBR-1



Item No.	QTY.	Description	Material Specification	Treatment Specification
–		Test Slab Assembly	–	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
–	27	Concrete Connector Plate Assembly	–	ASTM A123
f3	54	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
g6	54	1"–8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329
g7	54	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123

Notes:
(1) Location A used for 1100C small car tests.
(2) Location B used for 2270P pickup truck and 10000 SUT tests.
(3) Place vertical bar (part no. e2) driven 6" [152] into ground tangent to the threaded rods (part no. f4) at the attachment locations. For support during casting.
(4) Part f4 is threaded 1 1/4" [32] into part f2 (coupling nut) on Concrete Connector Plate Assembly.

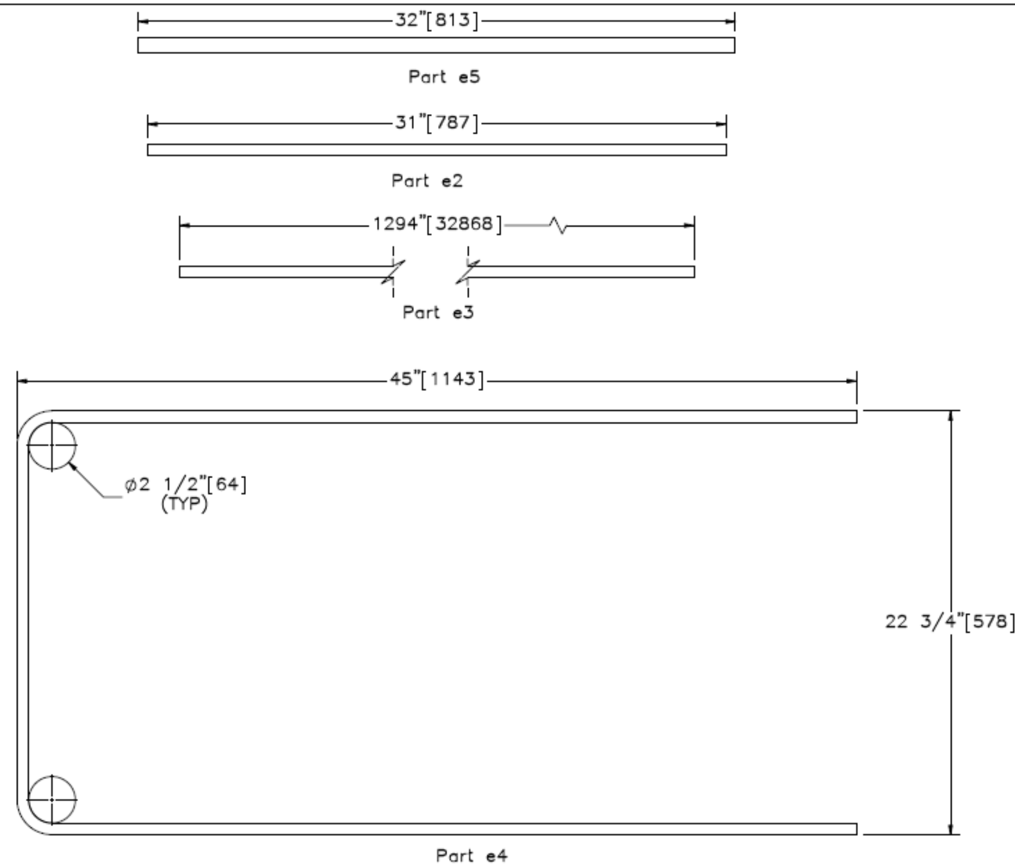
Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–1

Test Slab Assembly Details

SHEET: 19 of 25
DATE: 1/11/2019
DRAWN BY: JDJ/MBD/AE L/MKB/AJS
DWG. NAME. STBR–1_R20
SCALE: 1:20
UNITS: in.[mm]
REV. BY: JEK/PM/OP /SKR/RKF

Figure 148. Test Slab Assembly Details, Test No. STBR-1



Note: (1) Minimum lap length for a #5 bar is 18" [457].

Bill of Bars

Bar	QTY.	Size	Total Unbent Length	Material Specification	Treatment Specification
e2	54	#5	31" [787]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)
e3	8	#5	1294" [32868]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)
e4	144	#5	110 3/16" [2799]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)
e5	108	#6	32" [813]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)

Midwest Roadside Safety Facility

OH-IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR-1

Reinforcement Details

SHEET: 20 of 25

DATE: 1/11/2019

DRAWN BY: JDJ/MBD/AEL/MKB/AJS

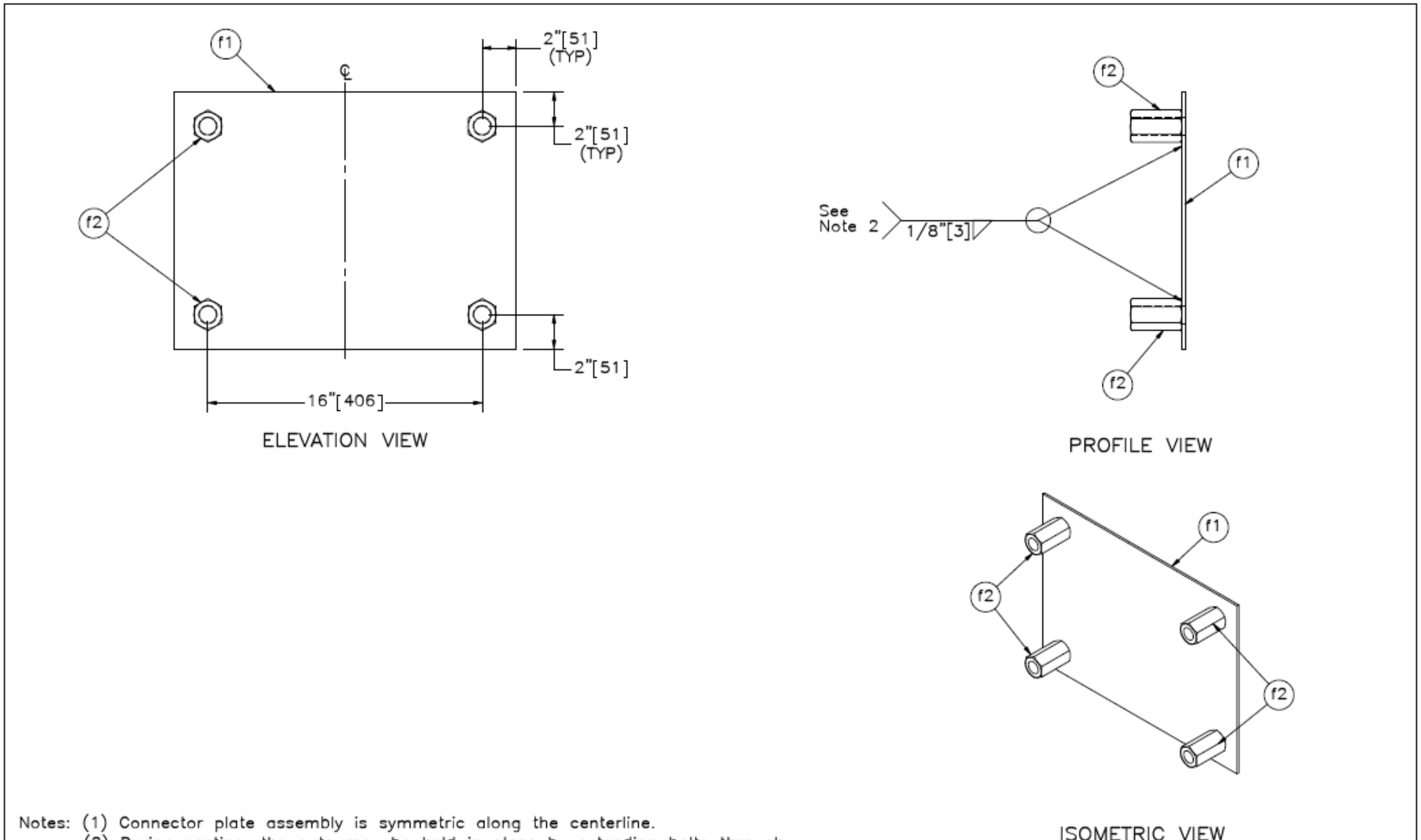
DWG. NAME. STBR-1_R20

SCALE: 1:8

UNITS: in.[mm]

REV. BY: JEK/PM/OP/SKR/RKF

Figure 149. Reinforcement Details, Test No. STBR-1



Notes: (1) Connector plate assembly is symmetric along the centerline.
(2) During casting, the nuts may be held in place by extending bolts through the formwork into the nuts, or by tack welding nuts to inside face of the plate.The welds are not structurally important.

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
–	27	Concrete Connector Plate Assembly	–	ASTM A123	–
f1	1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	See Assembly	–
f2	4	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–

MwRSF
Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–1

Concrete Connector Plate Assembly Details

SHEET: 21 of 25
DATE: 1/11/2019
DRAWN BY: JDJ/MBD/AEL/MKB/AJS
REV. BY: JEK/PM/OP/SKR/RKF
DWG. NAME. STBR–1_R20
SCALE: 1:10
UNITS: in.[mm]

Figure 150. Concrete Connector Plate Assembly Details, Test No. STBR-1

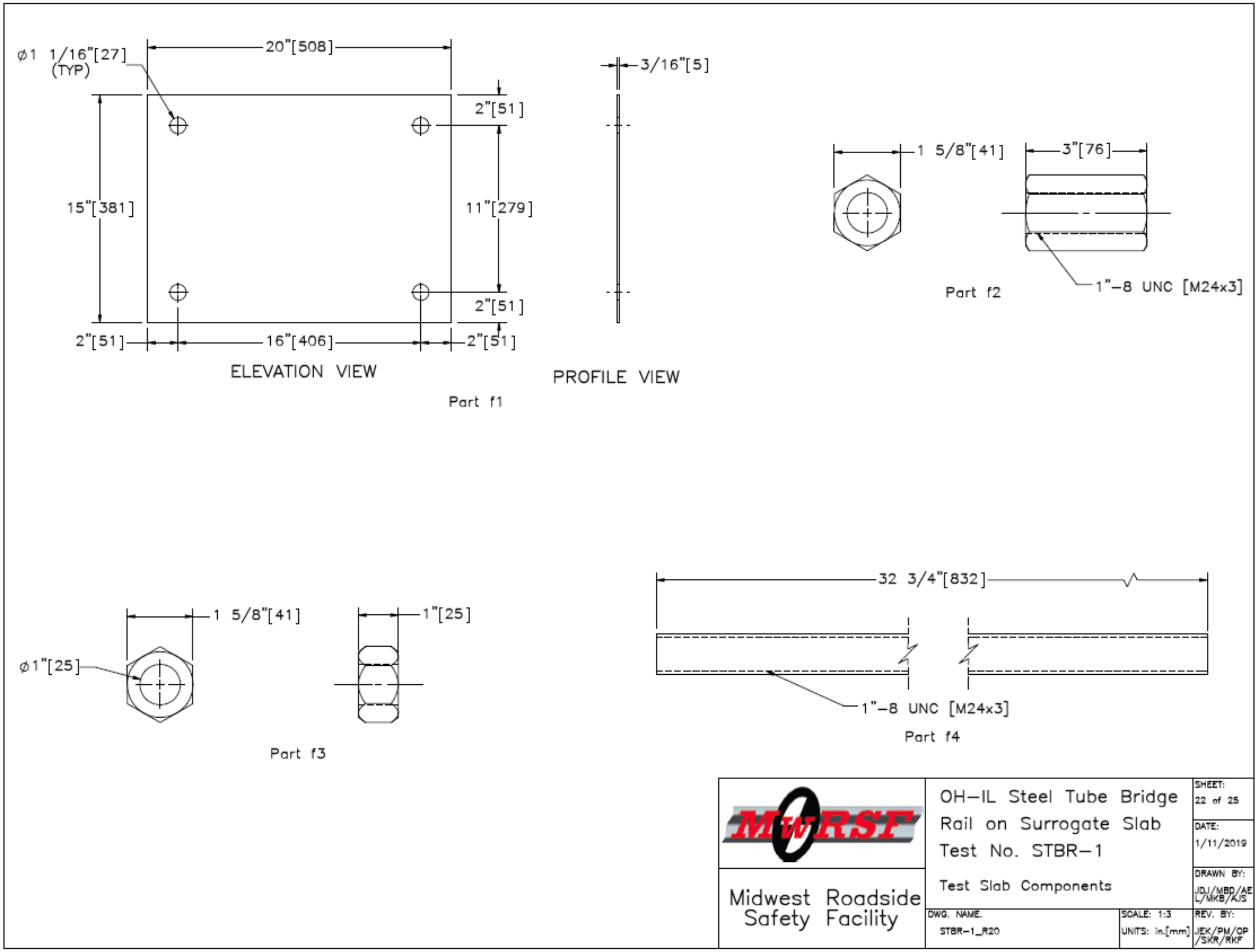


Figure 151. Test Slab Components, Test No. STBR-1

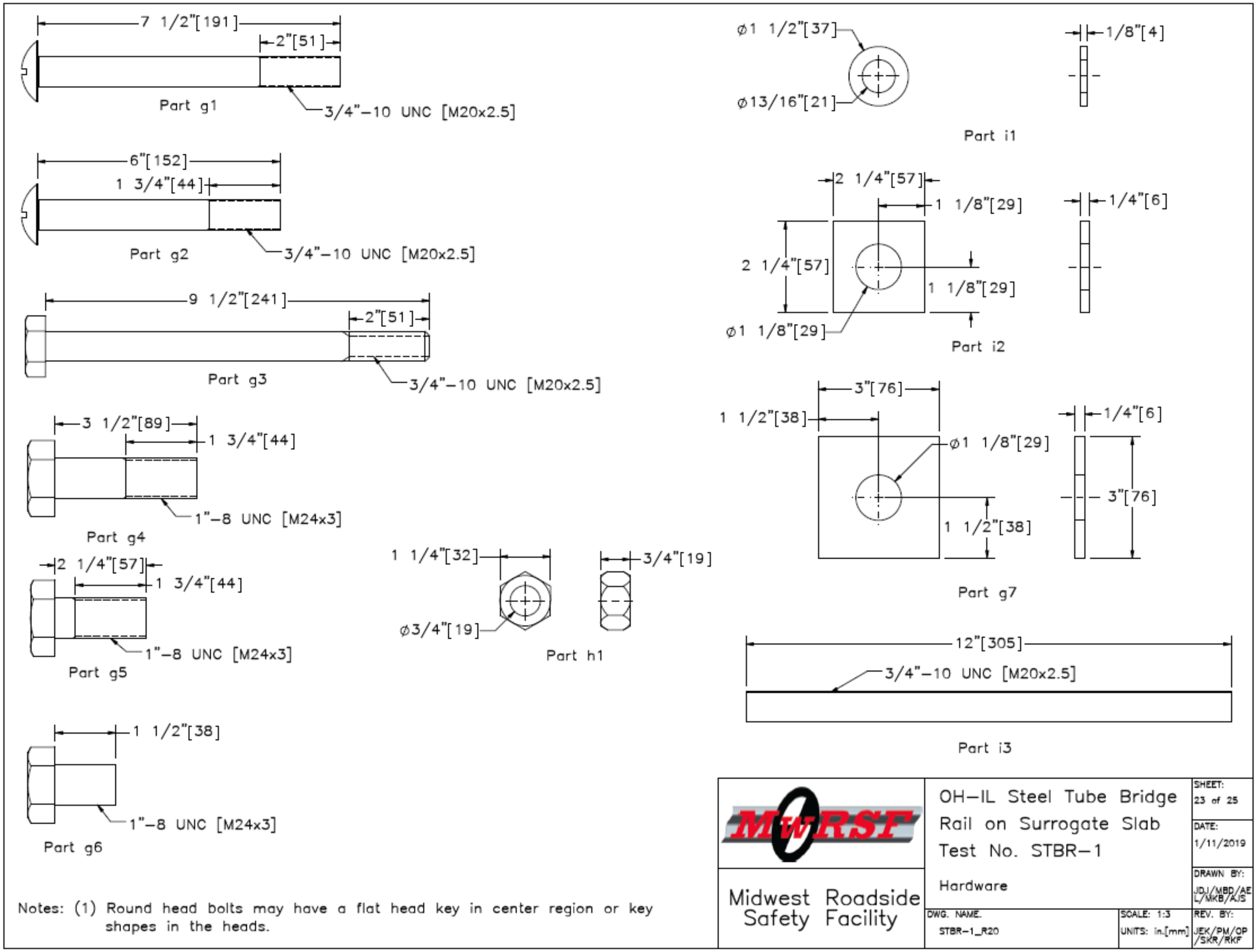


Figure 152. Hardware, Test No. STBR-1

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
a1	13	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	See Assembly	–
a2	20	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly	–
a3	13	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly	–
a4	52	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly	–
a5	26	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	ASTM A123	–
a6	7	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	See Assembly	–
a7	7	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	See Assembly	–
b1	18	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly	–
b2	18	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c1	36	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c2	36	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly	–
d1	20	HSS 8"x6"x1/4" [203x152x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
d2	10	HSS 12"x4"x1/4" [305x102x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
f1	27	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	ASTM A123	–
f2	108	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–
f3	106	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX24b
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329	FRR24b

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–1

Bill of Materials

SHEET: 24 of 25
DATE: 1/11/2019
DRAWN BY: JDJ/MBD/AE L/MKB/AJS
DWG. NAME. STBR–1_R20
SCALE: None
UNITS: in.[mm]
REV. BY: JEK/PM/OP /SKR/RKF

Figure 153. Bill of Materials, Test No. STBR-1

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
g1	80	3/4"–10 UNC [M20x2.5], 7 1/2" [191] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g2	116	3/4"–10 UNC [M20x2.5], 6" [152] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g3	72	3/4"–10 UNC [M20x2.5], 9 1/2" [241] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F2329	FBX20b
g4	52	1"–8 UNC [M24x3], 3 1/2" [89] 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
g5	52	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
g6	54	1"–8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX24b
g7	54	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123	–
h1	296	3/4 –10 UNC [M20x2.5] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX20b
i1	368	3/4" [19] Dia. Hardened Flat Washer	ASTM F436	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329	FWC20b
i2	156	2 1/4"x2 1/4"x1/4" [57x57x6] Square Washer	ASTM A36	ASTM A123	–
i3	28	3/4"–10 UNC [M20x2.5], 12" [305] Long Threaded Rod	ASTM F1554 Gr. 36	ASTM A153 or B695 Class 55 or F2329	FRR20a

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–1

Bill of Materials

SHEET: 25 of 25
DATE: 1/11/2019
DRAWN BY: JDJ/MBD/AEL/MKB/AJS
REV. BY: JEK/PM/OP/SKR/RKF
DWG. NAME. STBR–1_R20
SCALE: None
UNITS: in.[mm]

Figure 154. Bill of Materials, Test No. STBR-1

Figure 155. Test Installation Photographs, Test No. STBR-1

Figure 156. Test Installation Photographs, Side-Mounted and Top-Mounted Posts, Test No. STBR-1

Figure 157. Bridge Railing End Views, Test No. STBR-1

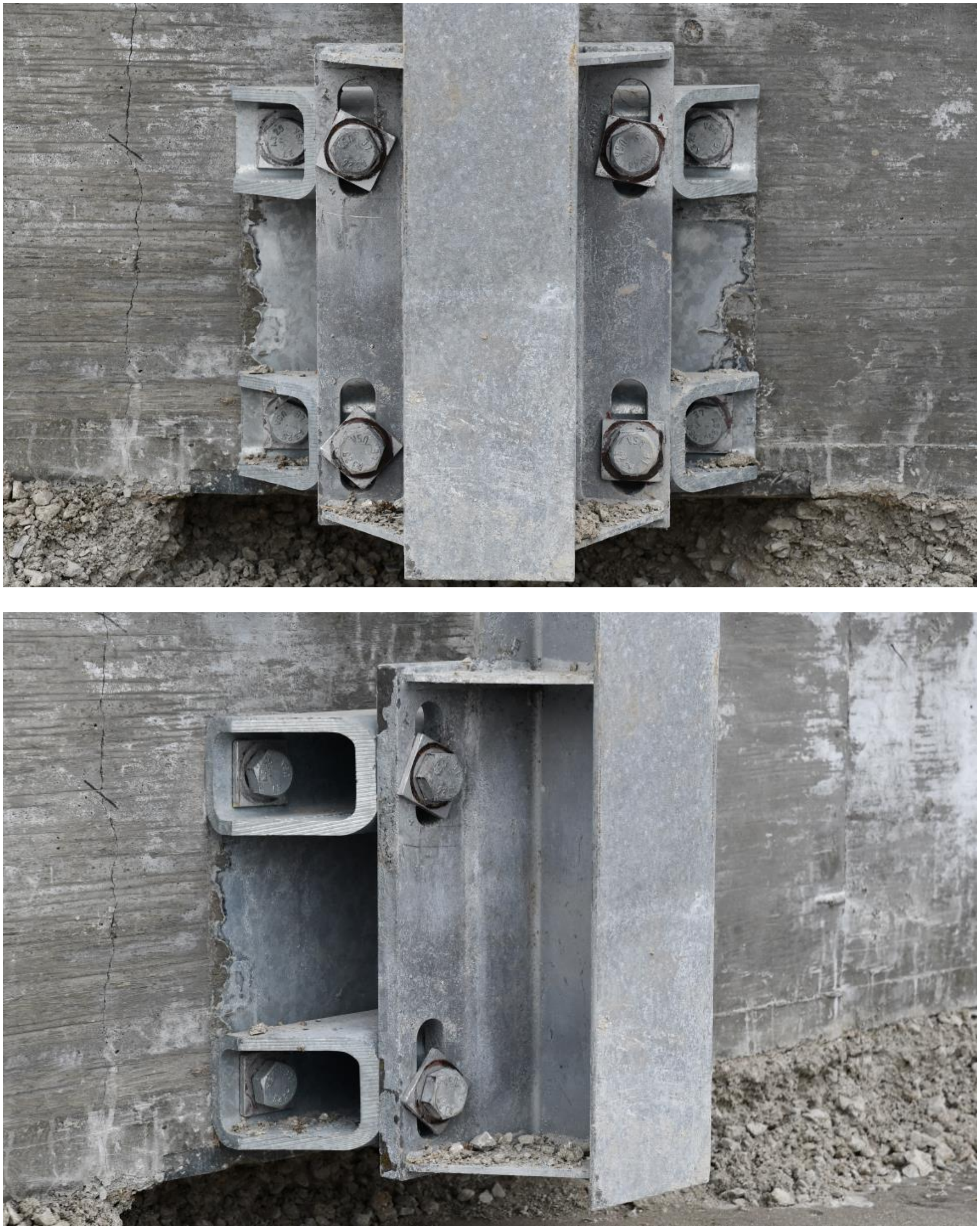


Figure 158. Side-Mounted Post-to-Deck Connections, Test No. STBR-1

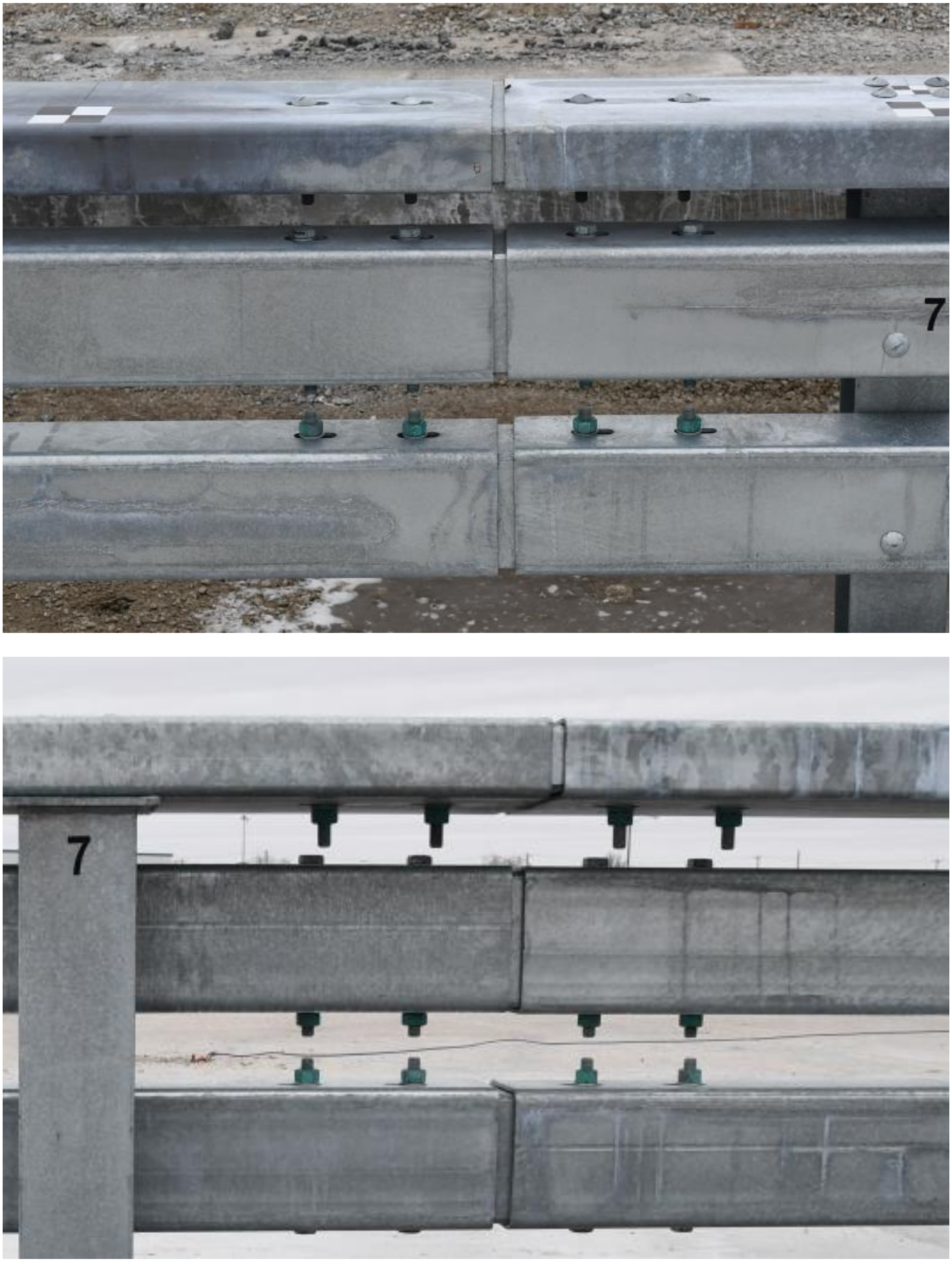


Figure 159. Splice Tubes, Test No. STBR-1

12 FULL-SCALE CRASH TEST NO. STBR-1

12.1 Weather Conditions

Test no. STBR-1 was conducted on February 8, 2019 at approximately 2:15 p.m. The weather conditions as per the National Oceanic and Atmospheric Administration (station 14939/LNK) were reported and are shown in Table 67.

Table 67. Weather Conditions, Test No. STBR-1

Temperature	18° F
Humidity	41%
Wind Speed	4 mph
Wind Direction	90° from True North
Sky Conditions	Sunny
Visibility	10 Statute Miles
Pavement Surface	Dry
Previous 3-Day Precipitation	2.26 in.
Previous 7-Day Precipitation	2.26 in.

12.2 Test Description

Initial vehicle impact was to occur 60 in. (1.5 m) upstream from the splice between post nos. 6 and 7, as shown in Figure 160, which was selected as discussed in Chapter 9. In test no. STBR-1, the 22,277-lb (10,105-kg) SUT impacted the bridge rail at a speed of 53.6 mph (86.2 km/h) and at an angle of 14.5 degrees with an impact severity of 133.2 kip-ft (180.6 KJ), which was below the lower allowable limit of 142.0 kip-ft (192.5 KJ) provided in MASH 2016. Thus, test no. STBR-1 was determined to be an invalid test according to the required MASH 2016 impact severity for test designation no. 4-12. The actual point of impact was 50.5 in. (1,283 mm) upstream from the splice between post nos. 6 and 7. The vehicle came to rest 290 ft – 9 in. (88.6 m) downstream from the original impact point and laterally 41 ft – 7 in. (12.7 m) in front of the bridge rail.

The crash testing parameters are summarized in Table 68. A detailed description of the sequential impact events is contained in Table 69. Sequential photographs are shown in Figures 161 and 162. Documentary photographs of the crash test are shown in Figure 163. The vehicle trajectory and final position are shown in Figure 164.

Figure 160. Impact Location, Test No. STBR-1

Table 68. Actual, Lower-Bound, and Target Crash Test Parameters, Test No. STBR-1

Test Parameter	Actual	Lower-Bound	Target
Speed	53.6 mph	53.5 mph	56.0 mph
Angle	14.5 deg.	13.5 deg.	15.0 deg.
Impact Severity	133.2 kip-ft	142.0 kip-ft	154.4 kip-ft

Table 69. Sequential Description of Impact Events, Test No. STBR-1

Time (sec):	**Event Description**
0.000	Vehicle's left-front bumper contacted rail between post nos. 6 and 7.
0.002	Vehicle's left-front bumper deformed.
0.006	Vehicle's left-front tire contacted rail.
0.008	Vehicle's left fender contacted rail.
0.010	Post no. 6 deflected backward. Vehicle's left fender deformed.
0.014	Post no. 7 deflected backward.
0.018	Post no. 8 deflected backward.
0.020	Vehicle pitched upward.
0.022	Post no. 6 bent backward. Vehicle yawed away from system. Vehicle's left-front tire became airborne.
0.026	Vehicle rolled toward system.
0.032	Post no. 7 bent backward. Post no. 9 deflected forward.
0.036	Vehicle's fuel tank deformed.
0.120	Vehicle's right-front tire became airborne.
0.162	Post no. 9 deflected backward.
0.250	Vehicle's right-rear tire became airborne.
0.292	Vehicle's left-rear cargo box corner contacted rail.
0.294	Vehicle was parallel to system at a speed of 47.3 mph (76.1 km/h).
0.304	Vehicle pitched downward.
0.314	Vehicle's left-rear wheel contacted rail.
0.342	Vehicle's left-front tire regained contact with ground.
0.514	Vehicle's left-rear tire became airborne.
0.772	Vehicle rolled away from system.
0.798	Vehicle pitched upward.
0.892	Vehicle's left-rear bumper contacted rail.
0.904	Vehicle's left-rear bumper deformed.
1.056	Post no. 12 deflected backward.
1.060	Post no. 13 deflected backward.
1.082	Post no. 12 deflected forward. Post no. 13 deflected forward.
1.086	Vehicle's left-rear tire regained contact with ground.
1.176	Vehicle's left cargo box side contacted rail.
1.250	Vehicle pitched downward.
1.326	Vehicle exited system at a speed of 42.4 mph (68.3 km/h).
1.336	Vehicle pitched upward.
1.368	Vehicle came to rest.
1.434	Vehicle's right-front tire regained contact with ground.
1.798	Vehicle's right-rear tire regained contact with ground.
1.852	Vehicle rolled toward system.
1.962	Vehicle yawed toward system.
2.242	Vehicle rolled away from system.
2.260	Vehicle pitched downward. Vehicle pitched upward.


0.000 sec


0.000 sec


0.200 sec

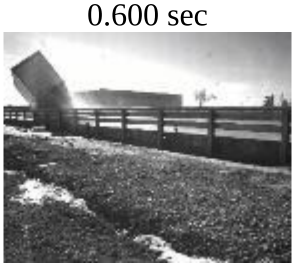

0.200 sec


0.400 sec


0.400 sec


0.600 sec


0.600 sec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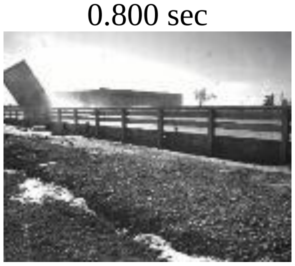
1.000 sec


1.000 sec

Figure 161. Sequential Photographs, Test No. STBR-1


0.000 sec


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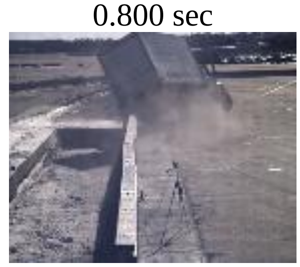
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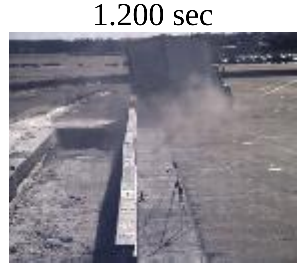
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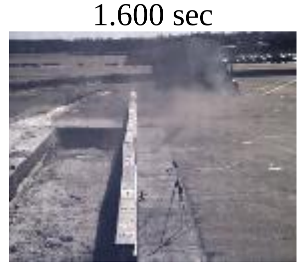

2.000 sec

Figure 162. Additional Sequential Photographs, Test No. STBR-1

Figure 163. Documentary Photographs, Test No. STBR-1

Figure 164. Vehicle Final Position and Trajectory Marks, Test No. STBR-1

12.3 System Damage

Damage to the bridge rail was minimal, as shown in Figures 165 through 172. Note that shrinkage cracks were observed in the surrogate slab prior to the test and denoted with a black marker. System damage consisted of contact marks, scrapes, gouges, and dents on the rails, a plastic hinge at post no. 7 for a two-span collapse, and minimal concrete spalling near the post-to-deck connection of post nos. 7 and 8. The length of vehicle contact along the barrier was approximately 71 ft – 2 in. (21.7 m), which spanned from 11½ in. (292 mm) upstream from the centerline of post no. 6 to 21½ in. (546 mm) upstream from the centerline of post no. 15.

A plastic hinge was found 5 in. (127 mm) above the location of the tension anchor rods for post no. 7. Rail gouging extended 15 in. (381 mm) downstream starting from the impact point along the front face of the middle rail. Gouging was also found along the front face of the bottom rail, extending 7 in. (178 mm) downstream from the impact point. Denting was found in the front face of the middle rail located 29½ in. (749 mm) upstream from the splice tube between posts nos. 6 and 7. Tire marks were visible on the front faces of all three rails starting at 51½ in. (1,308 mm) upstream from the splice tube between post nos. 6 and 7 and extending 10½ in. (267 mm) downstream from the splice tube between post nos. 8 and 9. Scuff marks were found on the top-front corner of the top rail, extending from the splice tube between post nos. 6 and 7 to 17 in. (431 mm) downstream from post no. 7.

Figure 165. System Damage, Test No. STBR-1

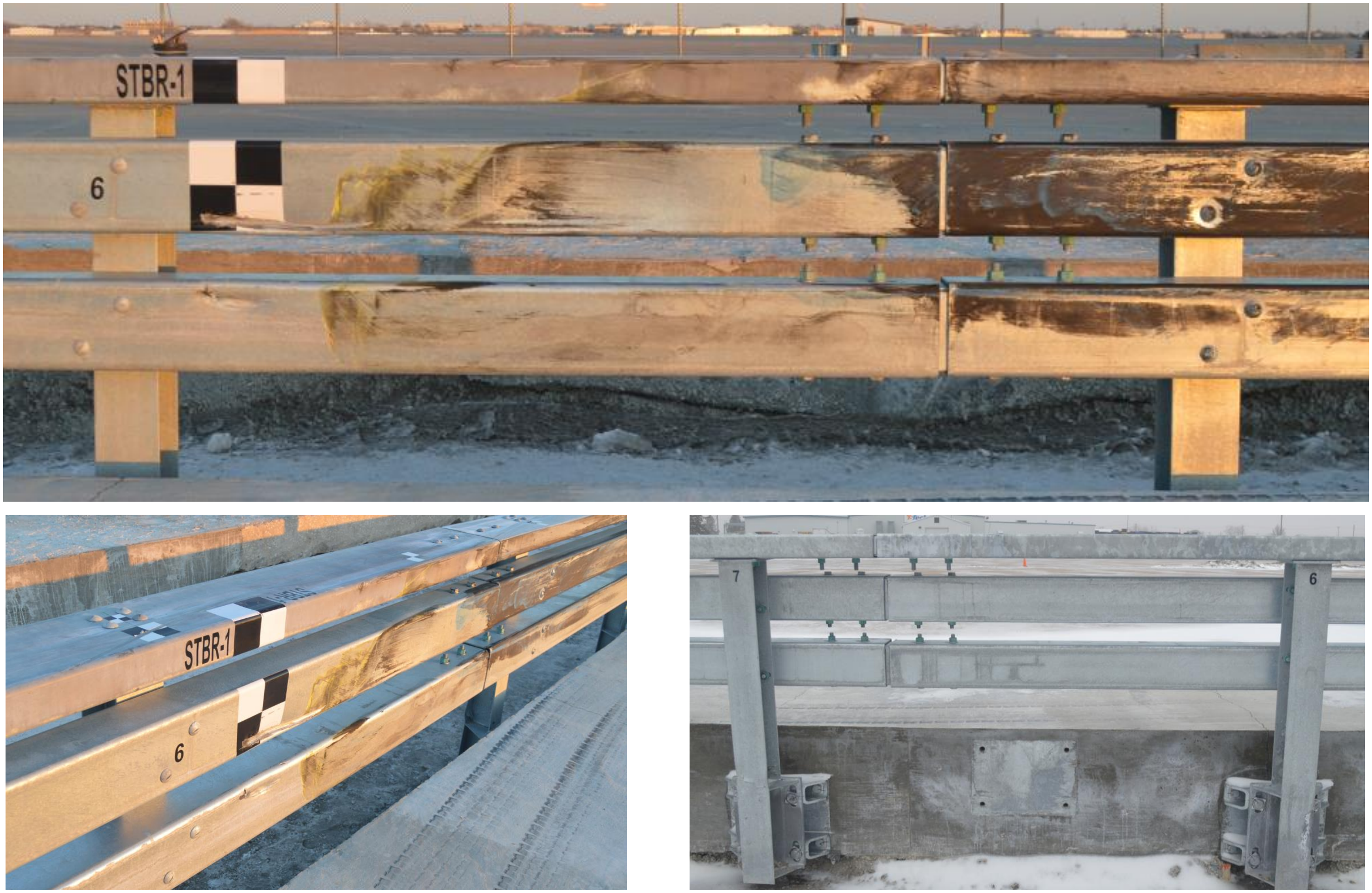


Figure 166. Damage to Rail Span Between Posts Nos. 6 and 7, Test No. STBR-1

Figure 167. Damage to Rail Span Between Posts Nos. 7 and 8, Test No. STBR-1

Figure 168. Damage to Rail Span Between Posts Nos. 8 and 9, Test No. STBR-1

Figure 169. Post No. 7 Damage, Test No. STBR-1

Figure 170. Post No. 8 Damage, Test No. STBR-1

Figure 171. Concrete Damage at Post No. 7, Test No. STBR-1

Figure 172. Concrete Damage at Post No. 8, Test No. STBR-1

Scuff marks were also found on the upper front edge of the top rail starting at 11½ in. (292 mm) upstream from post no. 6 and extending to 15 in. (381 mm) downstream from post no. 6. Minimal denting was observed on the front flange of the middle rail bolt locations.

Minimal concrete spalling, measuring ¼ in. (6.4 mm) deep by 3¼ in. (79 mm) long by 1⅜ in. (35 mm) tall, was found in the surrogate slab on the top right corner of the embedded plate of post no. 7. Concrete spalling, measuring ¼ in. (6.4 mm) deep by 8¼ in. (210 mm) long by 3½ in. (89 mm) tall, was found at the top right corner of the embedded plate for post no. 8.

The maximum lateral permanent set of the barrier system was measured to be 2.7 in. (69 mm). The maximum lateral dynamic barrier deflection was 4.3 in. (109 mm) at the top rail between post nos. 6 and 7, as determined from high-speed digital video analysis. The working width of the system was found to be 69.2 in. (1,757 mm), also determined from high-speed digital video analysis. A schematic of the permanent set deflection, dynamic deflection, and working width is shown in Figure 173.

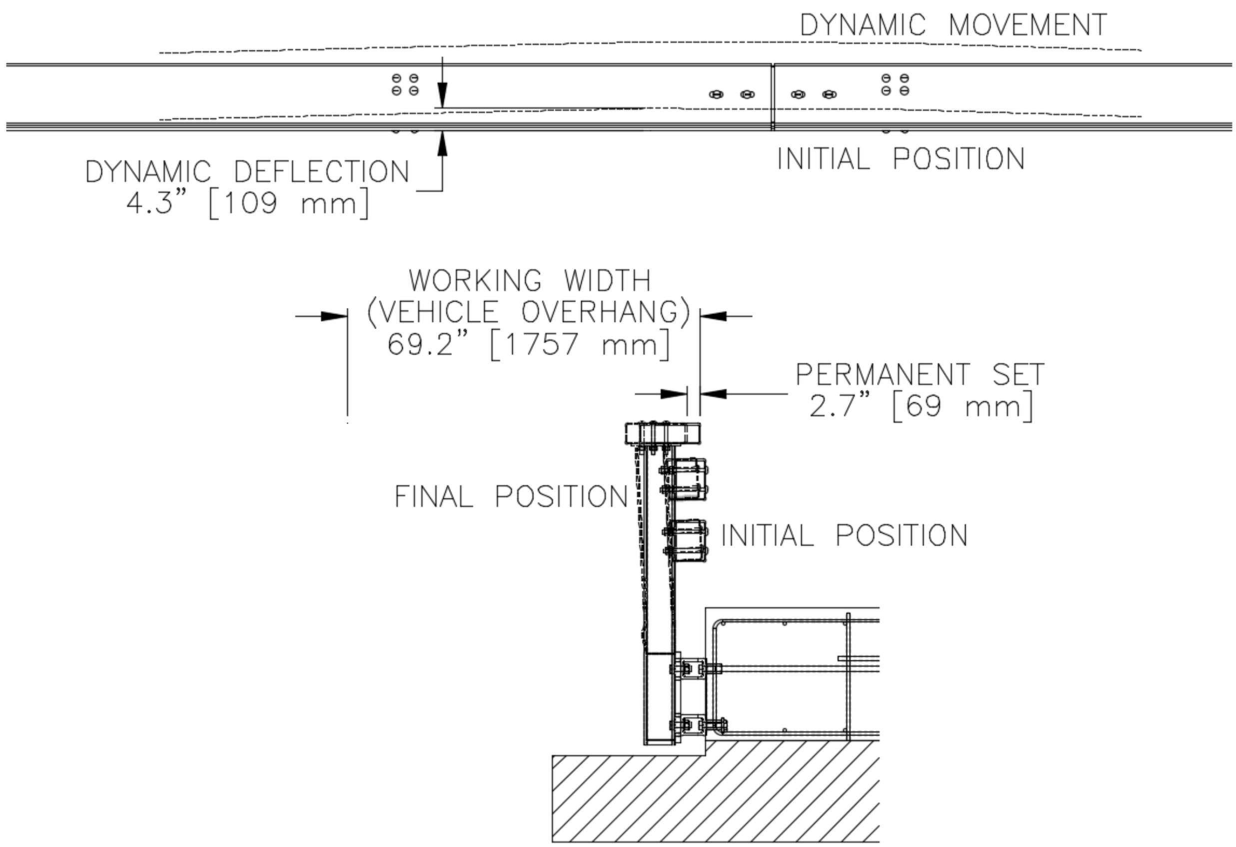


Figure 173. Permanent Set Deflection, Dynamic Deflection, and Working Width, Test No. STBR-1

12.4 Vehicle Damage

The damage to the vehicle was minimal, as shown in Figures 174 through 179. The maximum occupant compartment intrusions are listed in Table 70 along with the intrusion limits established in MASH 2016 for various areas of the occupant compartment. MASH 2016 defines intrusion or deformation as the occupant compartment being deformed and reduced in size with no observed penetration. Note that none of the established MASH 2016 deformation limits were violated. The entire B-pillar (lateral), side front panel, side door (above and below seat), and roof deformed slightly outward. Outward deformations are not considered crush toward the occupant, are denoted as negative numbers in Table 70, and are not evaluated by MASH 2016 criteria. Complete occupant compartment and vehicle deformations and the corresponding locations are provided in Appendix F.

The majority of damage was concentrated on the left-front corner of the vehicle where the impact occurred. The left side of the front bumper was dented inward and backward for 8¼ in. (210 mm). The left-front fender was detached from the vehicle. The gas tank located at the left side of the vehicle was dented 1 in. (25 mm). The left-side shock was dented, the bump stop bushing was disengaged, and the leaf spring mounting bracket was broken. The right-side leaf spring band was bent. The steering control arm was sheared off and disengaged from the steering gear box. The crossover link was broken and disengaged from the left-side steering knuckle. The oil pan of the drivetrain was cracked. The box sheer plates of the chassis were bent. The left-side floor pan was dented into the cab. Images of the damage done to the vehicle can be seen in Figures 174 through 179.

Figure 174. Vehicle Damage, Front and Rear Views, Test No. STBR-1

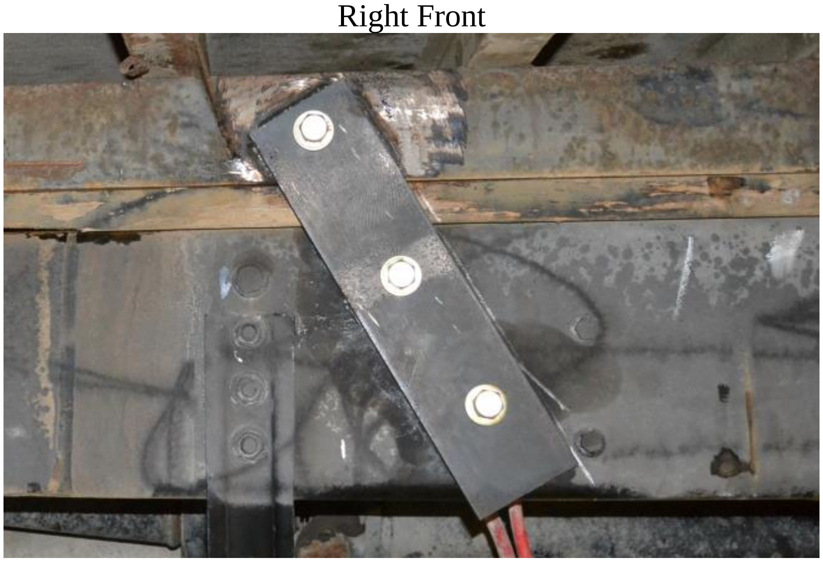
Figure 175. Vehicle Damage, Right and Left Views, Test No. STBR-1

Figure 176. Vehicle Damage, Right Corner Views, Test No. STBR-1

Figure 177. Vehicle Damage, Left Corner Views, Test No. STBR-1

Figure 178. Vehicle Damage, Floor Pan and Undercarriage, Test No. STBR-1

Right Front



Left Front

Figure 179. Vehicle Damage, Left-Front and Shear Plate Damage Views, Test No. STBR-1

Table 70. Maximum Occupant Compartment Intrusions by Location, Test No. STBR-1

LOCATION	MAXIMUM INTRUSION in. (mm)	MASH 2016 ALLOWABLE INTRUSION in. (mm)
Wheel Well & Toe Pan	2.7 (68.6)	≤ 9 (229)
Floor Pan & Transmission Tunnel	1.0 (25.4)	≤ 12 (305)
A-Pillar	0.1 (0.3)	≤ 5 (127)
A-Pillar (Lateral)	0.1 (0.3)	≤ 3 (76)
B-Pillar	0.0 (0.0)	≤ 5 (127)
B-Pillar (Lateral)	-0.1 (-2.5)	N/A[2]
Side Front Panel (in Front of A-Pillar)	-0.1 (-2.5)	N/A[2]
Side Door (Above Seat)	-0.3 (-7.6)	N/A[2]
Side Door (Below Seat)	-0.3 (-7.6)	N/A[2]
Roof	-0.2 (5.1)	N/A[2]
Windshield	0.0 (0.0)	≤ 3 (76)
Side Window	Intact	No shattering due to contact with structural member of test article
Dash	0.2 (5.1)	N/A[1]

Note: Negative values denote outward deformation
N/A[1] – No MASH 2016 criteria exist for this location
N/A[2] – MASH 2016 criteria are not applicable when deformation is outward

12.5 Occupant Risk

Occupant risk values are not required evaluation criteria for test designation no. 4-12. However, the occupant risk values were calculated with the same procedure as used for the 1100C and 2270P vehicles in order to make comparisons. The calculated occupant impact velocities (OIVs) and maximum 0.010-sec average occupant ridedown accelerations (ORAs) in both the longitudinal and lateral directions, as determined from the accelerometer data, are shown in Table 71. The calculated THIV, PHD, and ASI values are also shown in Table 71. The recorded data from the accelerometers and the rate transducers are shown graphically in Appendix G. Note, the SLICE-1 unit was designated as the primary unit during this test as it was mounted closer to the c.g. of the vehicle. The SLICE-2 unit was mounted in the vehicle's cab. The data from the DTS unit was not used in the occupant risk calculations due to the unit's distance from the vehicle's c.g.

Table 71. Summary of OIV, ORA, THIV, PHD, and ASI Values, Test No. STBR-1

Evaluation Criteria		Transducer		MASH 2016 Limits
		SLICE-1 (primary)	SLICE-2	
OIV ft/s (m/s)	Longitudinal	-7.29 (-2.22)	-5.22 (-1.59)	not required
	Lateral	11.55 (3.52)	13.71 (4.18)	not required
ORA g's	Longitudinal	-6.54	-4.90	not required
	Lateral	9.22	5.34	not required
MAXIMUM ANGULAR DISPLACEMENT deg.	Roll	-36.7	-28.9	not required
	Pitch	-7.4	-7.2	not required
	Yaw	42.0	40.2	not required
THIV ft/s (m/s)		13.83 (4.21)	14.79 (4.51)	not required
PHD g's		9.27	5.35	not required
ASI		0.27	0.22	not required

12.6 10,000S Peak Lateral Impact Force Calculation

The longitudinal and lateral vehicle accelerations, as measured at the vehicle's c.g., were also processed using a SAE CFC-60 filter and a 50-msec moving average. The 50-msec moving average vehicle accelerations were then combined with the uncoupled yaw angle versus time data in order to estimate the vehicular loading applied to the barrier system. From the data analysis, the perpendicular impact forces were determined for the bridge rail, as shown in Figures 180 and 181. The maximum perpendicular (i.e., lateral) load imparted to the barrier was 96.1 kips (427 kN) and 102.4 kips (455 kN), as determined by the SLICE-1 (primary) unit and TDAS, respectively.

12.7 Discussion

The analysis of the test results for test no. STBR-1 showed that the system adequately contained and redirected the 10000S vehicle with controlled lateral displacements of the barrier. A summary of the test results and sequential photographs are shown in Figure 182. Detached elements, fragments, or other debris from the test article did not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or work-zone personnel. Deformations of, or intrusions into, the occupant compartment that could have caused serious injury did not occur. The test vehicle did not penetrate nor ride over the barrier and remained upright during and after the collision. Vehicle roll, pitch, and yaw angular displacements, as shown in Appendix G, were deemed acceptable as they did not adversely influence occupant risk nor cause rollover. After impact, the vehicle exited the barrier at an orientation angle of 47.5 degrees, and its trajectory did not violate the bounds of the exit box. Although the test results were acceptable, test no. STBR-1 was determined to not be a valid test according to the required MASH 2016 impact severity for test designation no. 4-12. The actual impact severity was 133.2 kip-ft (180.6 KJ), which was below the lower bound of impact severity equal to 142.0 kip-ft (192.5 KJ), as noted in MASH 2016.

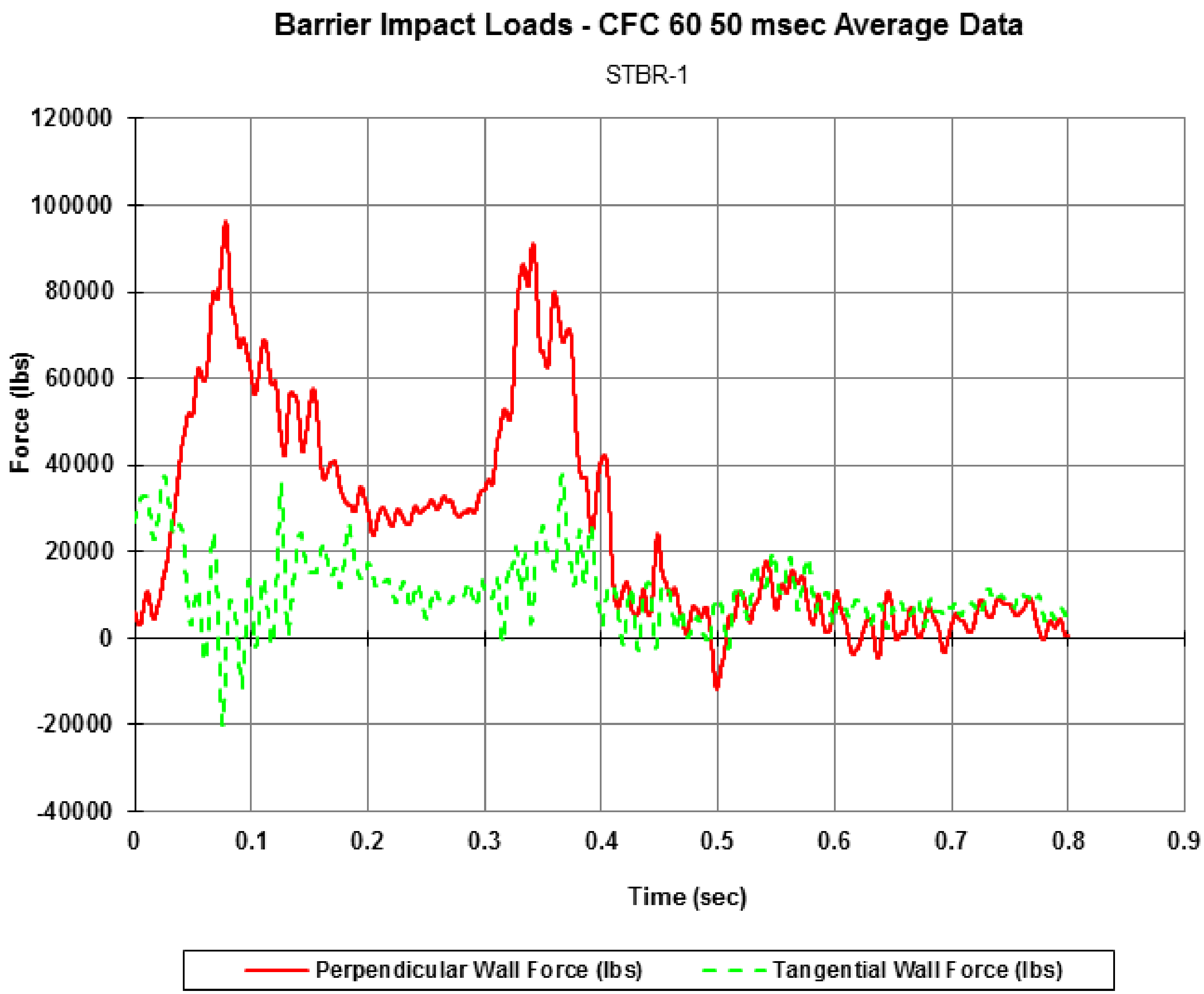


Figure 180. Perpendicular and Tangential Forces Imparted to the Barrier System (SLICE-1), Test No. STBR-1

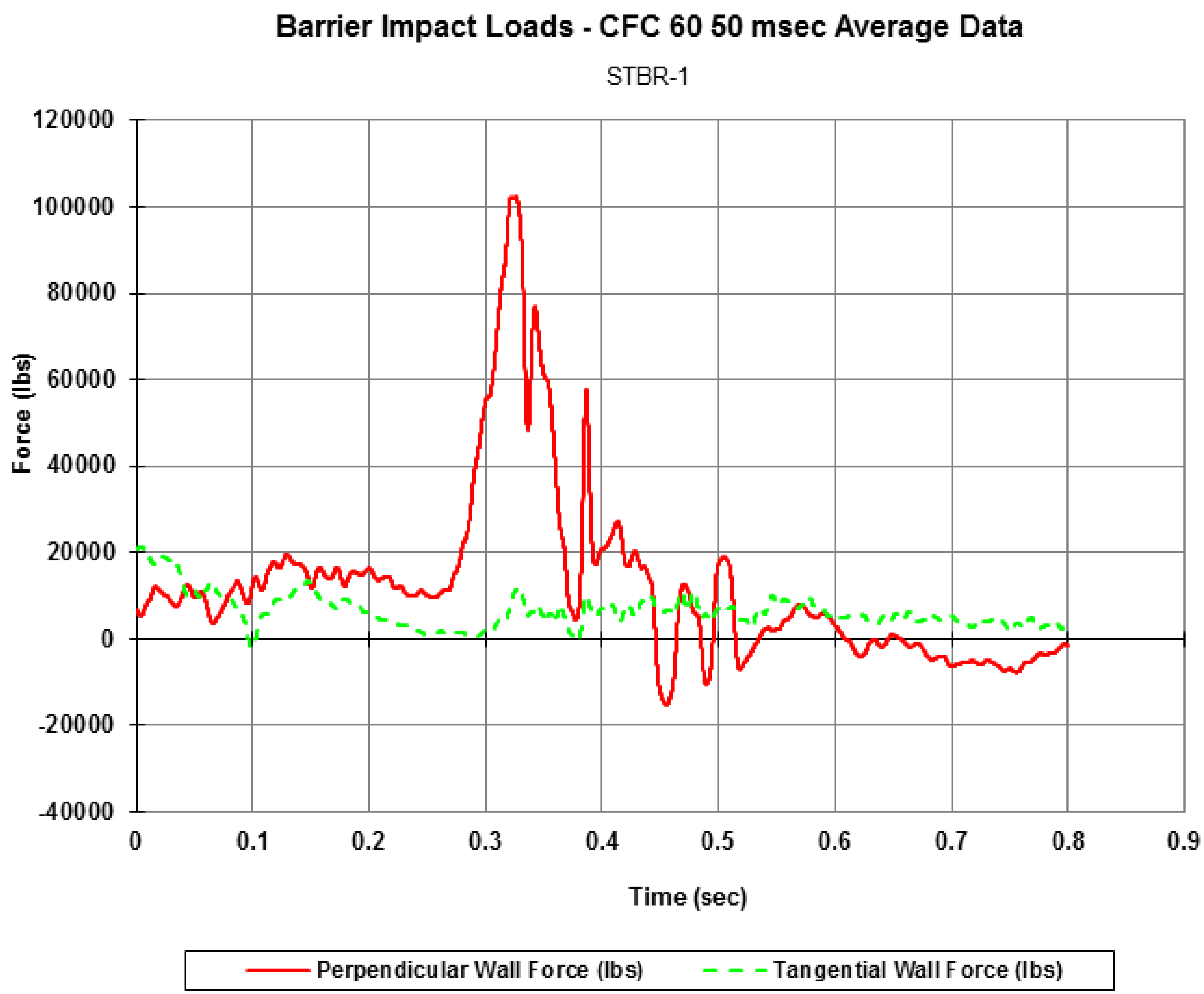


Figure 181. Perpendicular and Tangential Forces Imparted to the Barrier System (DTS), Test No. STBR-1

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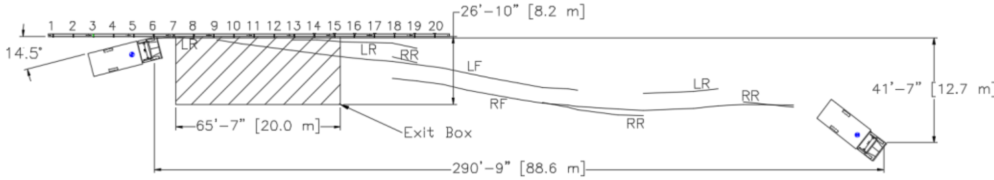
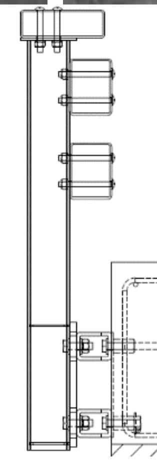




- Test Agency .... MwRSF
- Test Number STBR-1
- Date 2/8/2019
- MASH 2016 Test Designation No. 4-12
- Test Article .... Steel, Side-Mounted, Beam-and-Post, Bridge Rail
- Total Length .... 159 ft – 11¼ in. (48.7 m)
- Key Component – Top Rail
  - Length .... 191¼ in. (4,858 mm)
  - Width .... 12 in. (305 mm)
  - Depth .... 4 in. (102 mm)
- Key Component - Post
  - Length .... 58½in. (1,486 mm)
  - Width .... 6 in. (152 mm)
  - Spacing .... 8 ft (2.4 m)
- Soil Type .... N/A
- Vehicle Make /Model .... 2007 Freightliner M2 106
  - Curb .... 13,725 lb (6,226 kg)
  - Test Inertial .... 22,124 lb (10,035 kg)
  - Gross Static .... 22,277 lb (10,105 kg)
- Impact Conditions
  - Speed .... 53.6 mph (86.2 km/h)
  - Angle .... 14.5 deg.
  - Impact Location ..... 50.5 in. (1.3 m) upstream from splice between post nos. 6 and 7.
- Impact Severity 133.2 kip-ft (180.6 kJ) < 142.0 kip-ft (192.5 kJ) limit from MASH 2016
- Exit Conditions
  - Speed .... 42.4 mph (68.3 km/h)
  - Angle Approximation .... ~10 deg. (based on tire skid marks at exit)
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 290 ft – 9 in. (88.6 m) downstream from impact
  .... Laterally 41 ft – 7 in. (12.7 m) in front of bridge rail
- Vehicle Damage .... Minimal
  - VDS [56] 11-LFQ-4
  - CDC [57] 11-LFAW-4
  - Maximum Interior Deformation .... 2.7 in. (69 mm)
- Test Article Damage .... Minimal
- Maximum Test Article Deflections
  - Permanent Set .... 2.7 in. (69 mm)
  - Dynamic .... 4.3 in. (109 mm)
  - Working Width .... 69.2 in. (1,758 mm)
- Transducer Data

Evaluation Criteria		Transducer		MASH 2016 Limit
		SLICE-1 (primary)	SLICE-2	
OIV ft/s (m/s)	Longitudinal	-7.29 (-2.22)	-5.22 (-1.59)	Not required
	Lateral	11.55 (3.52)	13.71 (4.18)	Not required
ORA g's	Longitudinal	-6.54	-4.90	Not required
	Lateral	9.22	5.34	Not required
MAX ANGULAR DISP. deg.	Roll	-36.7	-28.9	Not required
	Pitch	-7.4	-7.2	Not required
	Yaw	42.0	40.2	Not required
THIV – ft/s (m/s)		13.83 (4.21)	14.79 (4.51)	Not required
PHD – g's		9.27	5.35	Not required
ASI		0.27	0.22	Not required

*Data discarded due to equipment malfunction

Figure 182. Summary of Test Results and Sequential Photographs, Test No. STBR-1

13 CONSTRUCTION DETAILS TEST NO. STBR-2

The test installation for the bridge rail system consisted of steel rails, posts assemblies, post-to-rail and rail-to-rail connections, as well as a surrogate concrete bridge deck, as shown in Figures 183 through 207. The total length of the bridge rail was 159 ft – 11¼ in. (48.7 m). Photographs of the test installation are shown in Figures 208 through 212. Material specifications, mill certifications, and certificates of conformity for the system materials are shown in Appendix D. After test no. STBR-1, post nos. 5, 6, 7, and 8, the nearest two railing elements for each of the three rails, and the three splice tube location connecting these rails, were replaced for test no. STBR-2.

The system was constructed with 20 galvanized ASTM A992, W6x15 (W150x22.5) steel post assemblies spaced on 96-in. (2,438-mm) centers. Post assembly nos. 1 through 13 were side-mounted to the vertical side edge of the surrogate, reinforced-concrete bridge deck. For the construction of the surrogate concrete bridge deck, the threaded rod and coupling nuts were held in place to the embedded vertical plates by placing bolts through the formwork, rather than utilizing the option of welding the coupling nuts to the embedded vertical plates, as shown in Figure 203. Post assembly nos. 14 through 20 were surface-mounted to the top of existing concrete tarmac, which provided the necessary system length for vehicle redirection and were used for testing-purposes only.

Post assembly nos. 1 through 13 were 58⅞ in. (1,495 mm) long. An ASTM A572 Grade 50 steel plate PL 8 in. x 8 in. x ⅜ in. (PL 203 mm x 203 mm x 10 mm) was attached to the top of each post assembly with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Similarly, an ASTM A572 Grade 50 steel, vertical plate PL 13 in. x 17¾ in. x 1 in. (PL 330 mm x 451 mm x 25 mm) was attached to the bottom of the front flange of each post with all-around ¼-in. (6.4-mm) fillet welds. Four gusset plates, fabricated with ASTM A572 Grade 50 steel plate and measuring PL 6⅛ in. x $5^{11}/_{16}$ in. x ¼ in. (PL 156 mm x 144 mm x 6 mm), were welded to the top and bottom of the vertical plates, inner faces of post flanges, and web with all-around ¼-in. (6.4-mm) fillet welds. Post assembly nos. 1 through 13 were bolted to the tension and compression sides of the vertical plates with ASTM A500 Grade 50 horizontal spacer tubes, which were specified as HSS 5-in. x 4-in. x ½-in. (HSS 127-mm x 102-mm x 13-mm) sections.

Post assembly nos. 1 through 13 were bolted to the horizontal spacer tubes using ASTM F3125 Grade A325 1-in. (25-mm) diameter by 3½-in. (89-mm) long, heavy hex-head bolts with ¼-in. (6.4-mm) thick, ASTM A36 steel square washers and 1-in. (25-mm) diameter ASTM A563DH heavy hex nuts. The deck anchorage in the tension region consisted of two 1-in. (25-mm) diameter by 32¾-in. (832-mm) long, ASTM F1554 Grade 105 all-thread anchor rods with ASTM A563DH heavy hex coupling nuts, and ASTM A563DH heavy hex nuts. The deck anchorage in the compression region consisted of two 1-in. (25-mm) diameter, ASTM A449 anchor bolts with ¼-in. (6.4-mm) thick, 3-in. (76-mm) ASTM A36 steel square washers, and ASTM A563DH heavy hex coupling nuts. A $^{3}/_{16}$-in. (4.8-mm) thick, vertical embedment plate was used at every post location.

Post assembly nos. 14 through 20 were 32 in. (813 mm) long. Post assembly nos. 14 through 20 consisted of three parts – a base plate, a top plate, and a vertical post. The top plate consisted of an ASTM A572 Grade 50 steel plate measuring PL 8 in. x 8 in. x ⅜ in. (PL 203 mm x 203 mm x 10 mm) with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Similarly, the bottom plate consisted of an ASTM A572 Grade 50 steel plate measuring PL 12 in. x 12 in. x ¾ in. (PL 305 mm x 305 mm x 19 mm) with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Finally, the post was fabricated with ASTM A992 W6x15 (W150x22.5) sections measuring 30⅞ in. (784 mm) long. The post assembly nos. 14 through 20 were anchored to the existing tarmac with four ¾-in. (19-mm) diameter by 12-in. (305-mm) long ASTM F1554 Grade 36 all-thread anchor rods with ¼-in. (6.4-mm) thick, ASTM A36 steel square washers, and ASTM A563DH heavy hex nuts.

The three rail elements consisted on an ASTM A500 Grade C HSS 12 in. x 4 in. x ¼ in. (HSS 304.8-mm x 101.6-mm x 6.4-mm) section for the top rail and ASTM A500 Grade C HSS 8-in. x 6-in. x ¼-in. (HSS 203.2-mm x 152.4-mm x 6.4-mm) section for the lower two rails. Rail-to-rail connections were located 2 ft (610 mm) downstream from every other post location. The top rails were attached to the post assemblies with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long, ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom rails were attached to the front flanges of the posts with two staggered ¾-in. (19-mm) diameter by 7½-in. (191-mm) long ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

The splice tube for the top rails consisted of two horizontal PL 30 in. x 10⅝ in. x $^{5}/_{16}$ in. (PL 762-mm x 270-mm 8-mm) and two vertical PL 30-in. x 2⅝-in. x ⅜-in. (PL 762-mm x 67-mm x 10-mm) attached with ¼-in. (6.4-mm) fillet welds. The splice tubes for the middle and bottom rails consisted on two vertical PL 30-in. x 6⅝-in. x ⅜-in. (PL 762-mm x 168-mm 10-mm) and two horizontal PL 30-in. x 4⅝-in. x $^{5}/_{16}$-in. (PL 762-mm x 117-mm x 8-mm) attached with ¼-in. (6.4-mm) fillet welds. The top splice tubes were attached to the top rail end sections with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long, ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom splice tubes were attached to the rail end sections with two ¾-in. (19-mm) diameter by 9½-in. (241-mm) long, ASTM A449 hex-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

After test no. STBR-2, post nos. 7, 8, 9, and 10, and the nearest two railing elements for each of the three rails were replaced for test no. STBR-3.

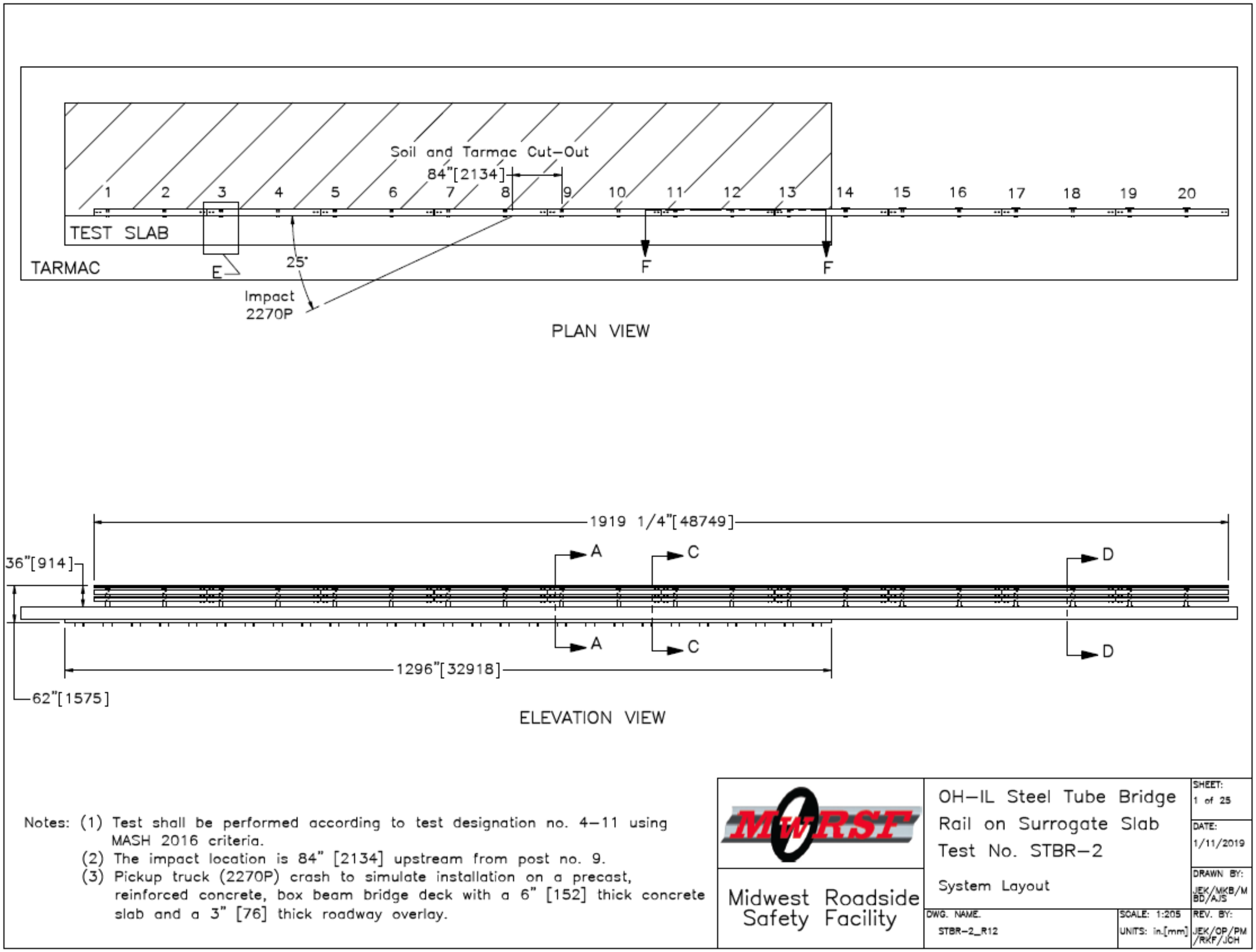


Figure 183. System Layout and Impact Location, Test No. STBR-2

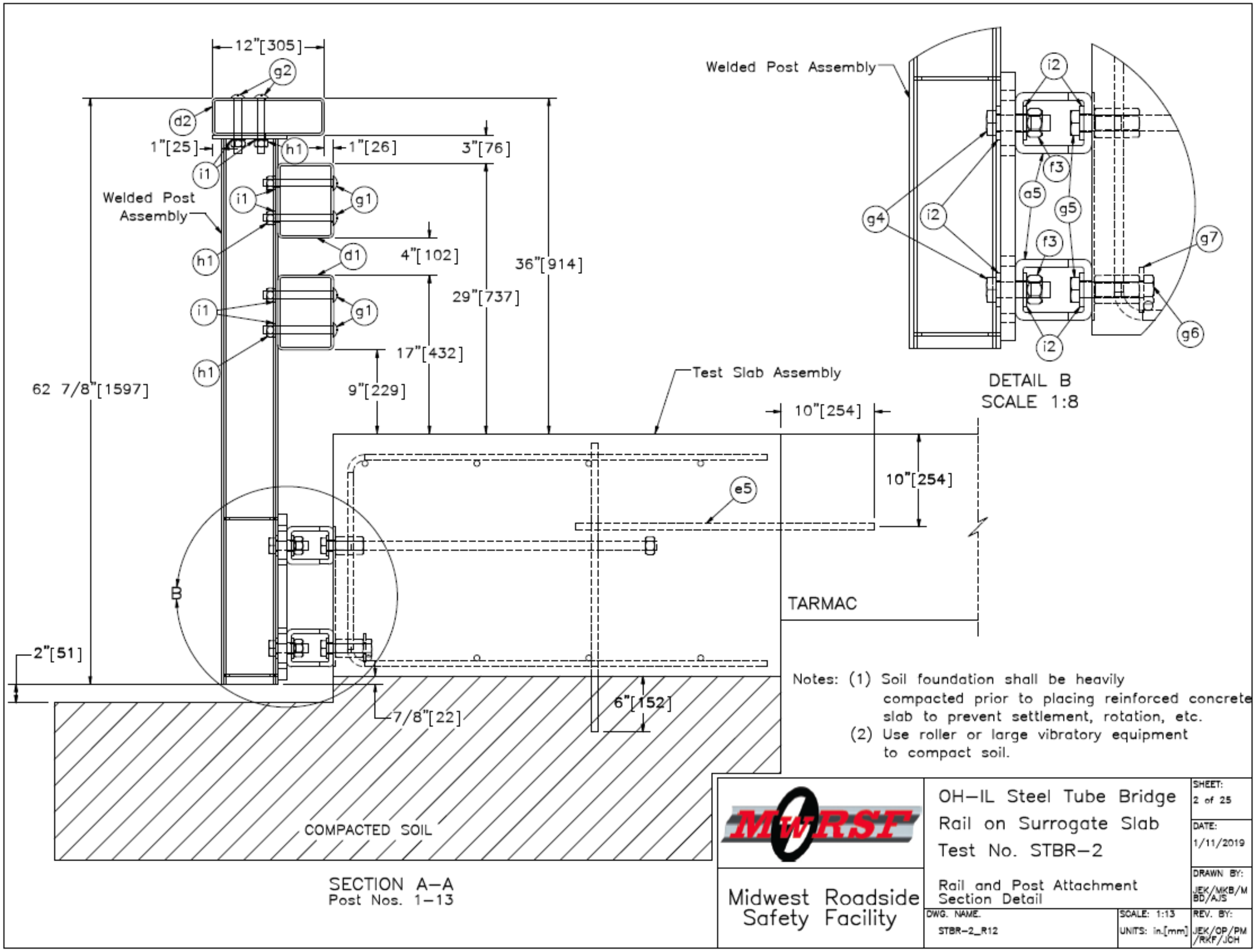


Figure 184. Rail and Post Attachment Section Detail, Test No. STBR-2

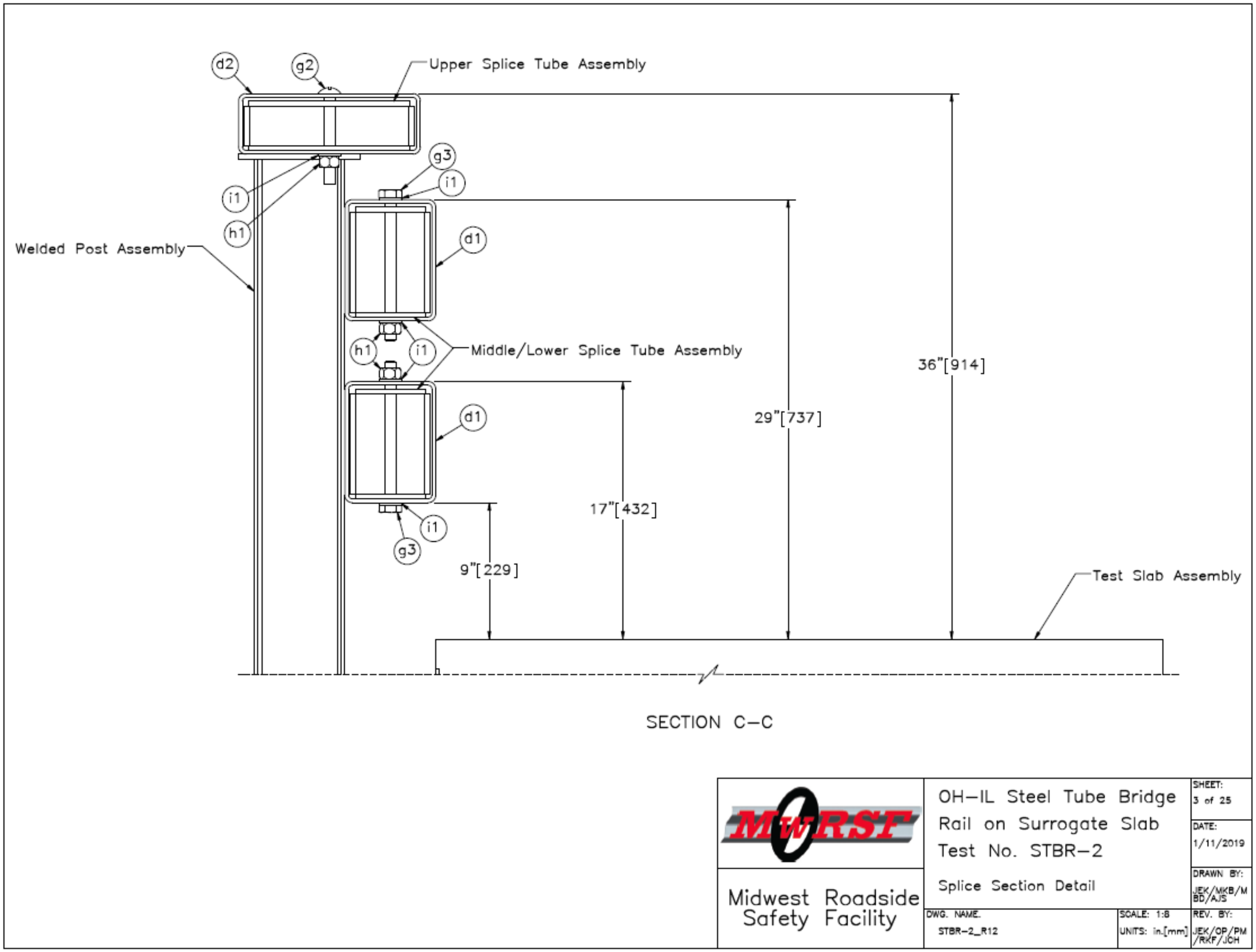


Figure 185. Splice Section Detail, Test No. STBR-2

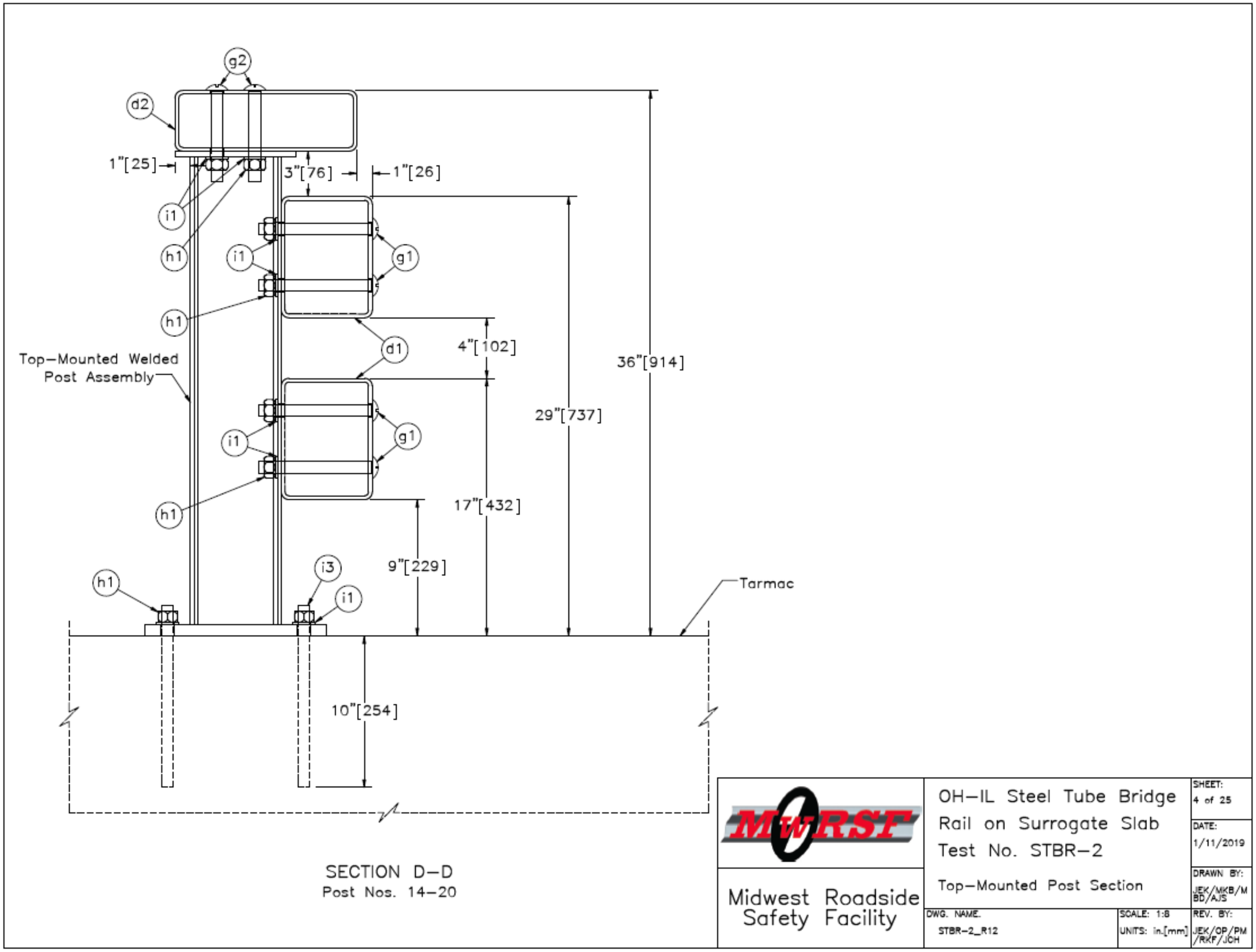


Figure 186. Top-Mounted Post Section, Test No. STBR-2

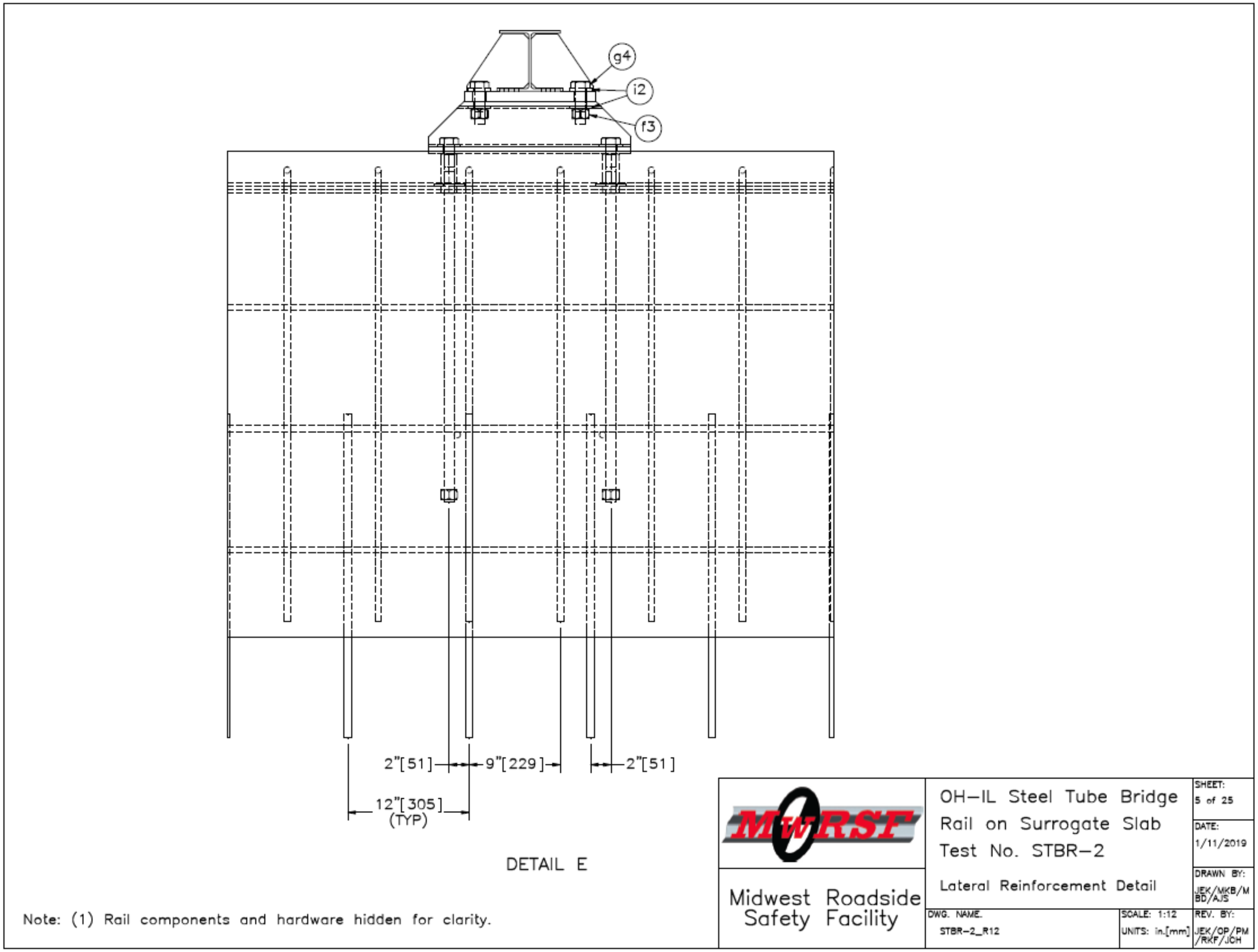


Figure 187. Lateral Reinforcement Detail, Test No. STBR-2

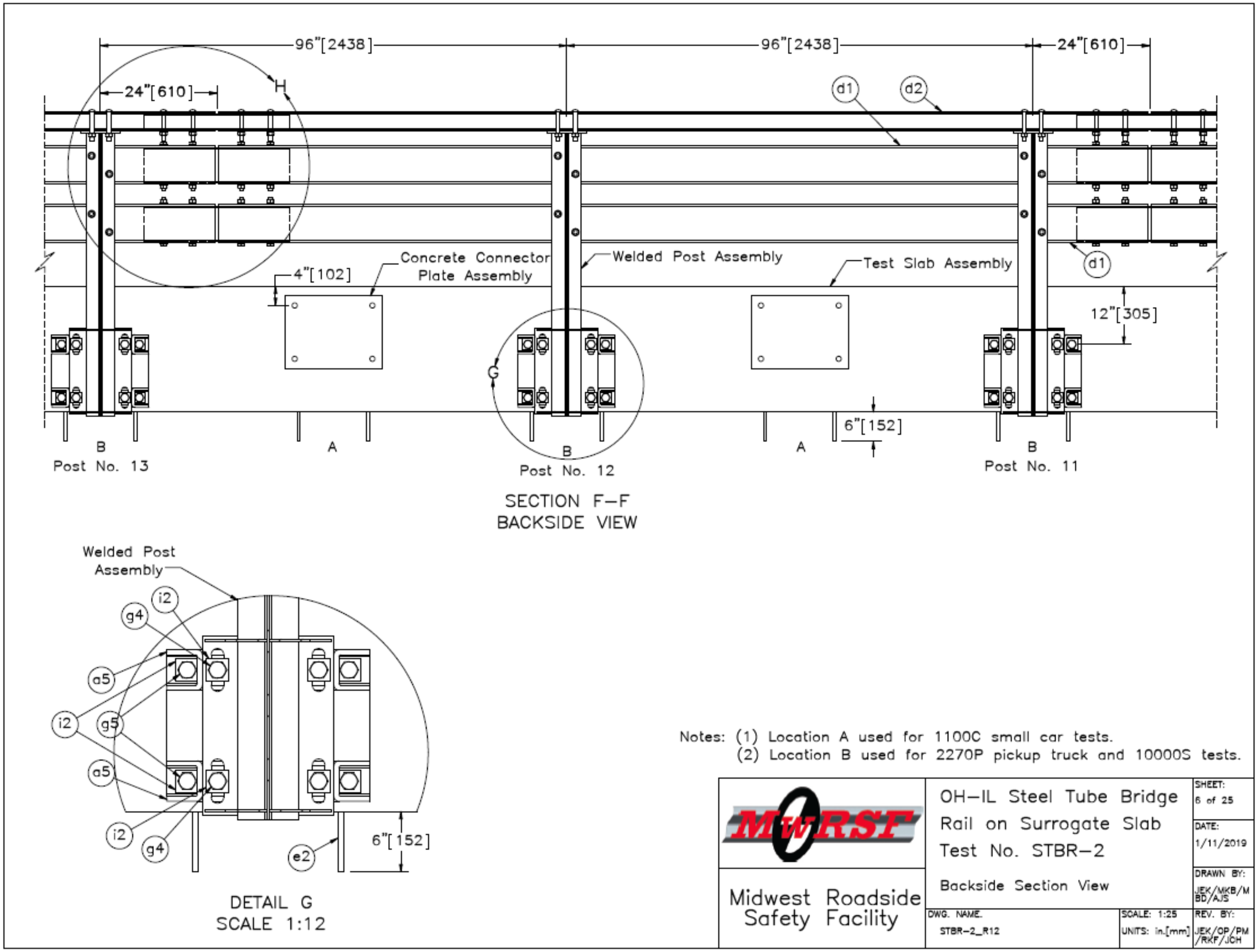


Figure 188. Backside Section View, Test No. STBR-2

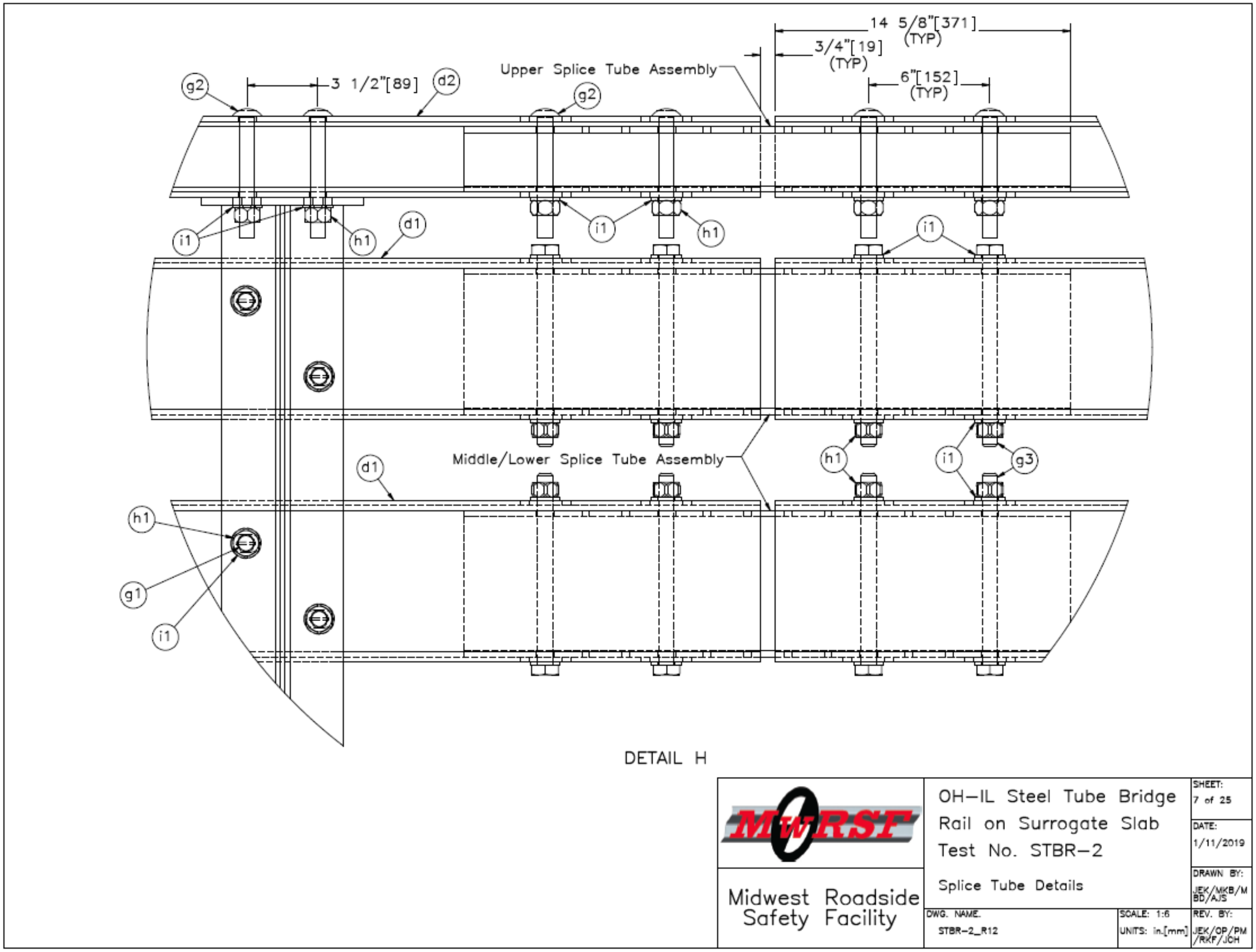
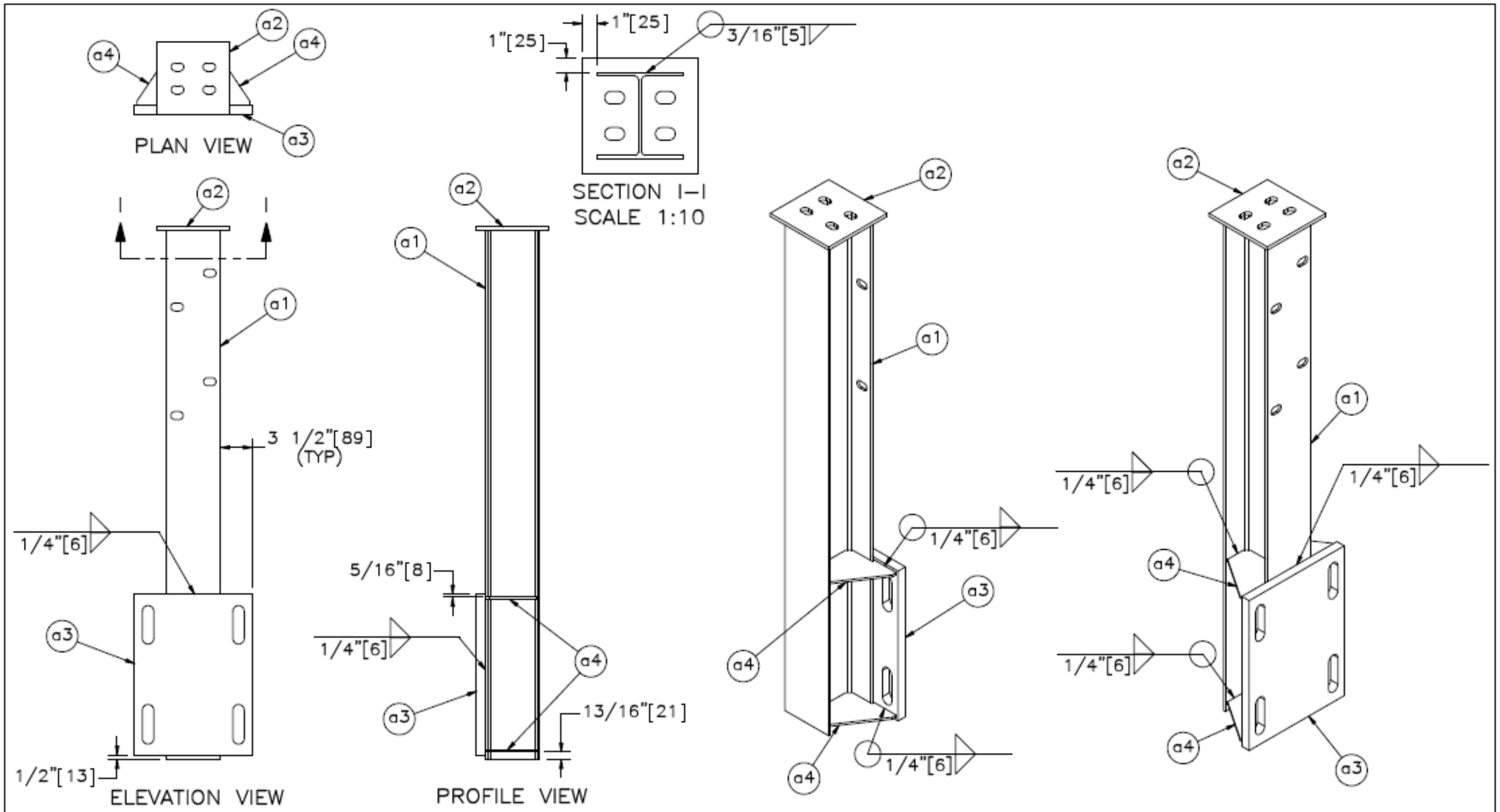


Figure 189. Splice Tube Section Details, Test No. STBR-2



Notes: (1) Gussets on both sides of post.
(2) To prevent welds to end at edges of gussets, leave 5/16" [8] of base material beyond weld to edge of welded part.
(3) Leave coped corners of gussets open for drainage. Do not fill with welds.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	13	Welded Post Assembly	–	ASTM A123
a1	1	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	See Assembly
a2	1	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly
a3	1	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly
a4	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly

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OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–2

Welded Post Assembly Details

SHEET: 8 of 25
DATE: 1/11/2019
DRAWN BY: JEK/MKB/M BD/AJS
DWG. NAME. STBR–2_R12
SCALE: 1:16
UNITS: in.[mm]
REV. BY: JEK/OP/PM /RKF/JOH

Figure 190. Welded Post Assembly Details, Test No. STBR-2

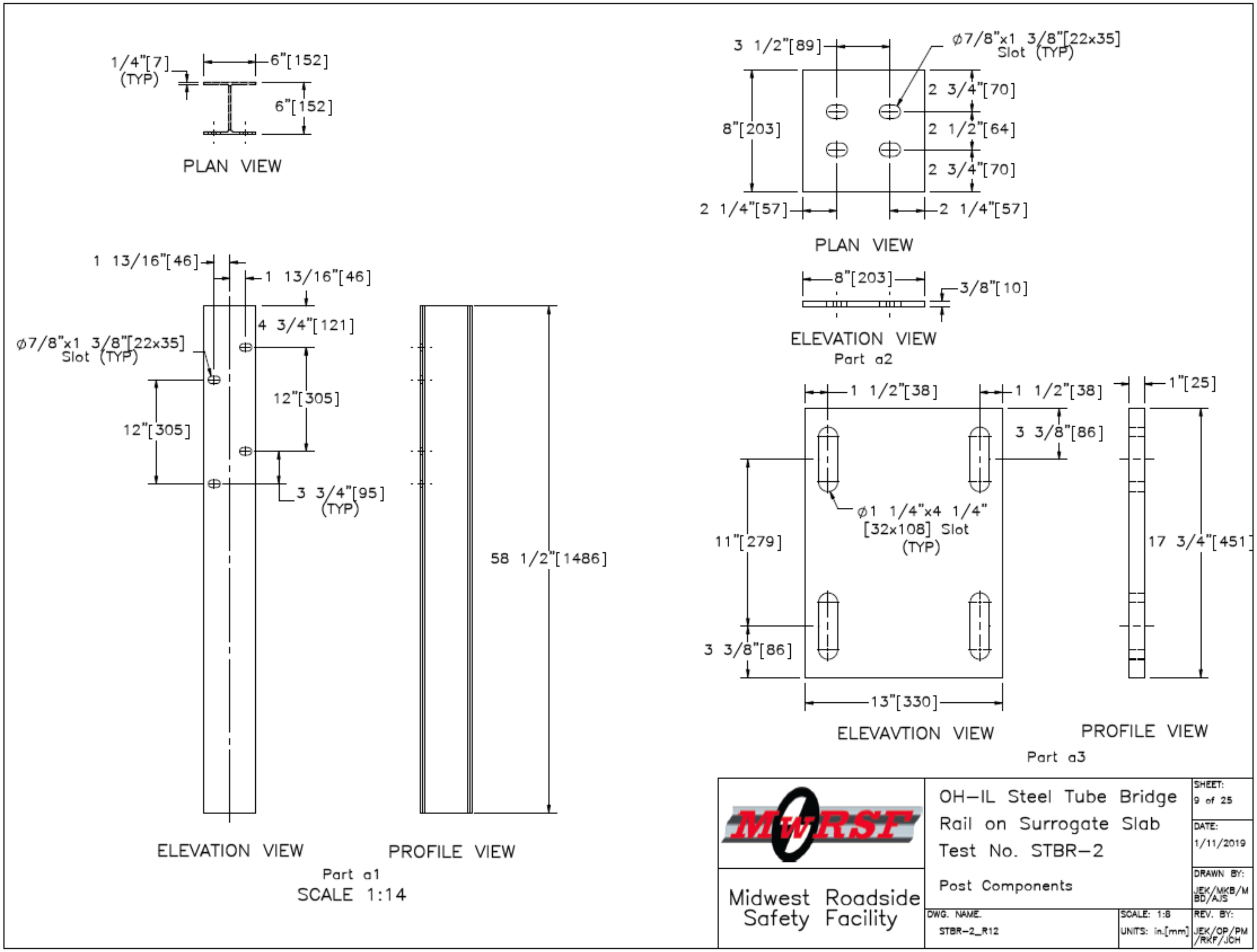


Figure 191. Post Components, Test No. STBR-2

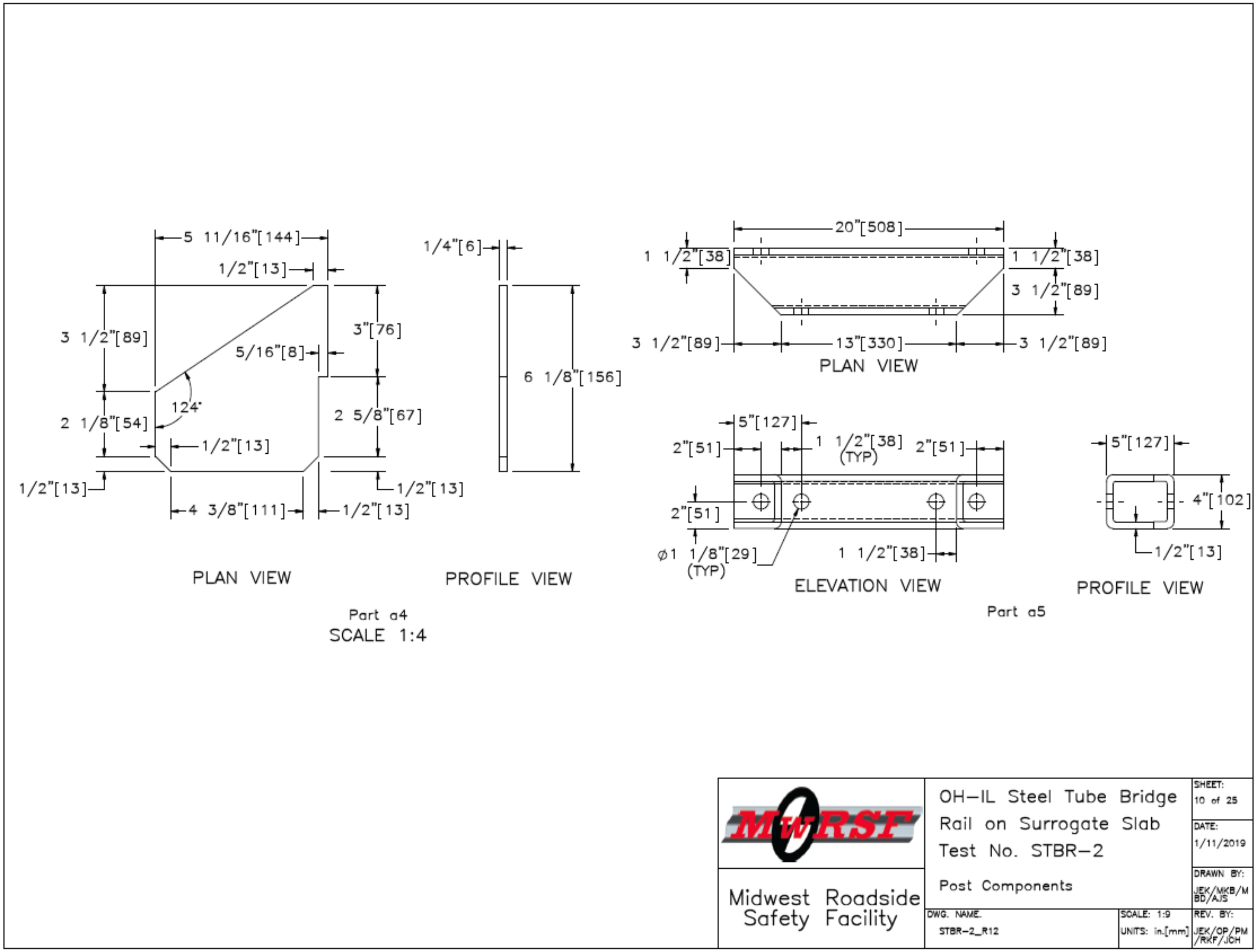
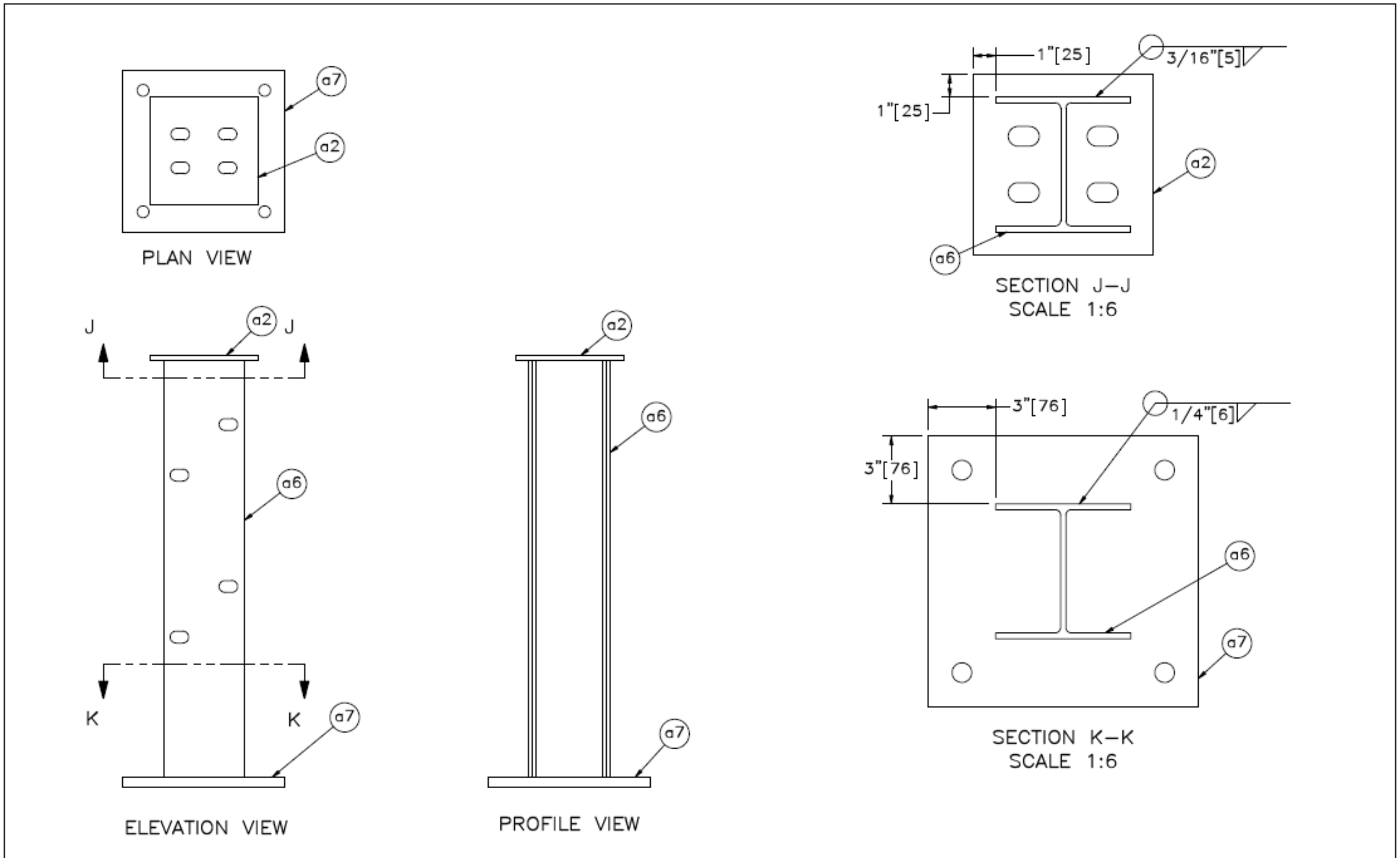


Figure 192. Post Components, Test No. STBR-2



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	7	Top–Mounted Welded Post Assembly	–	ASTM A123
a2	1	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly
a6	1	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	–
a7	1	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	–

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OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–2

Top–Mounted Welded Post Assembly

SHEET: 11 of 25
DATE: 1/11/2019
DRAWN BY: JEK/MKB/M BD/AJS
DWG. NAME. STBR–2_R12
SCALE: 1:10
UNITS: in.[mm]
REV. BY: JEK/OP/PM /RKF/JCH

Figure 193. Top-Mounted Welded Post Assembly, Test No. STBR-2

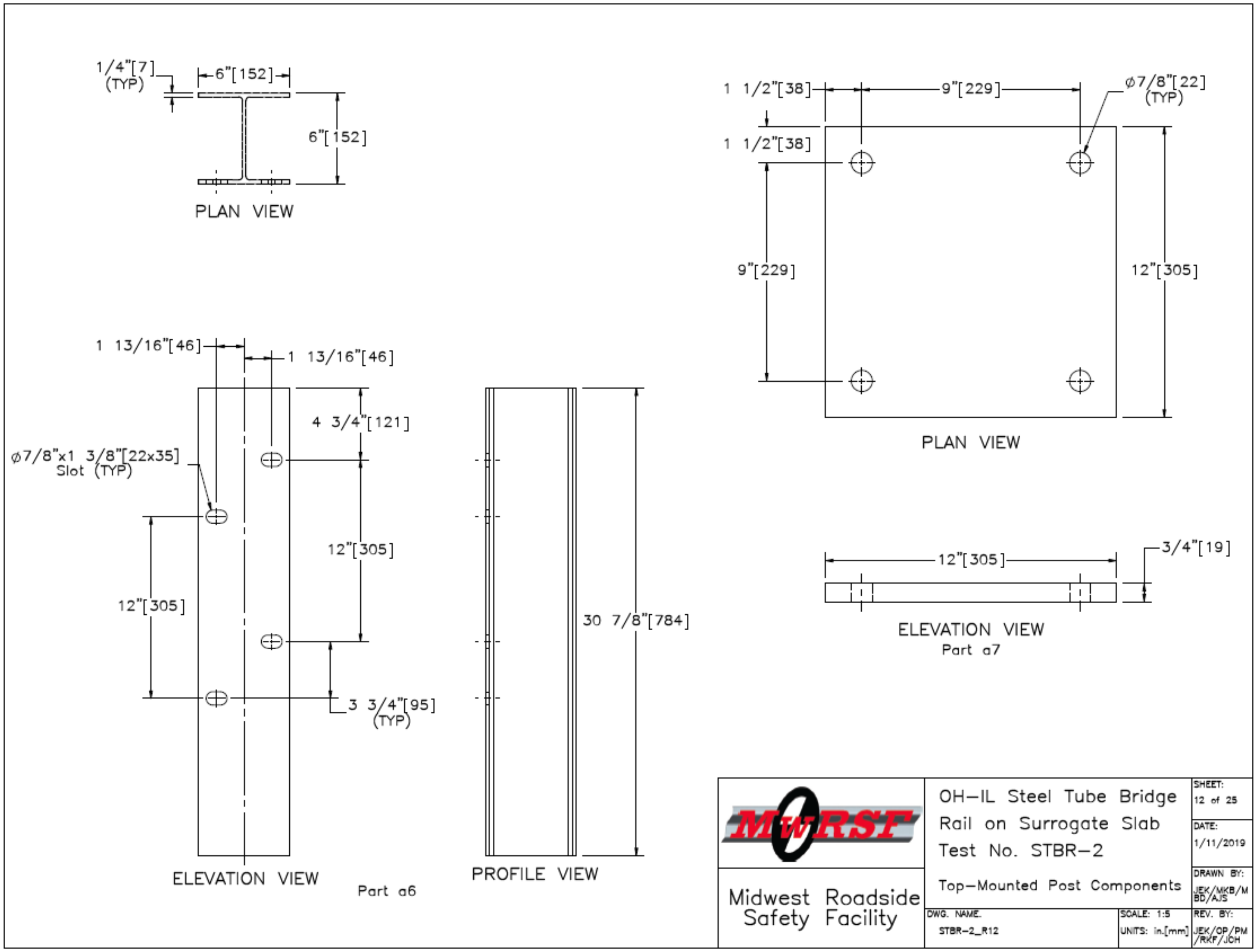
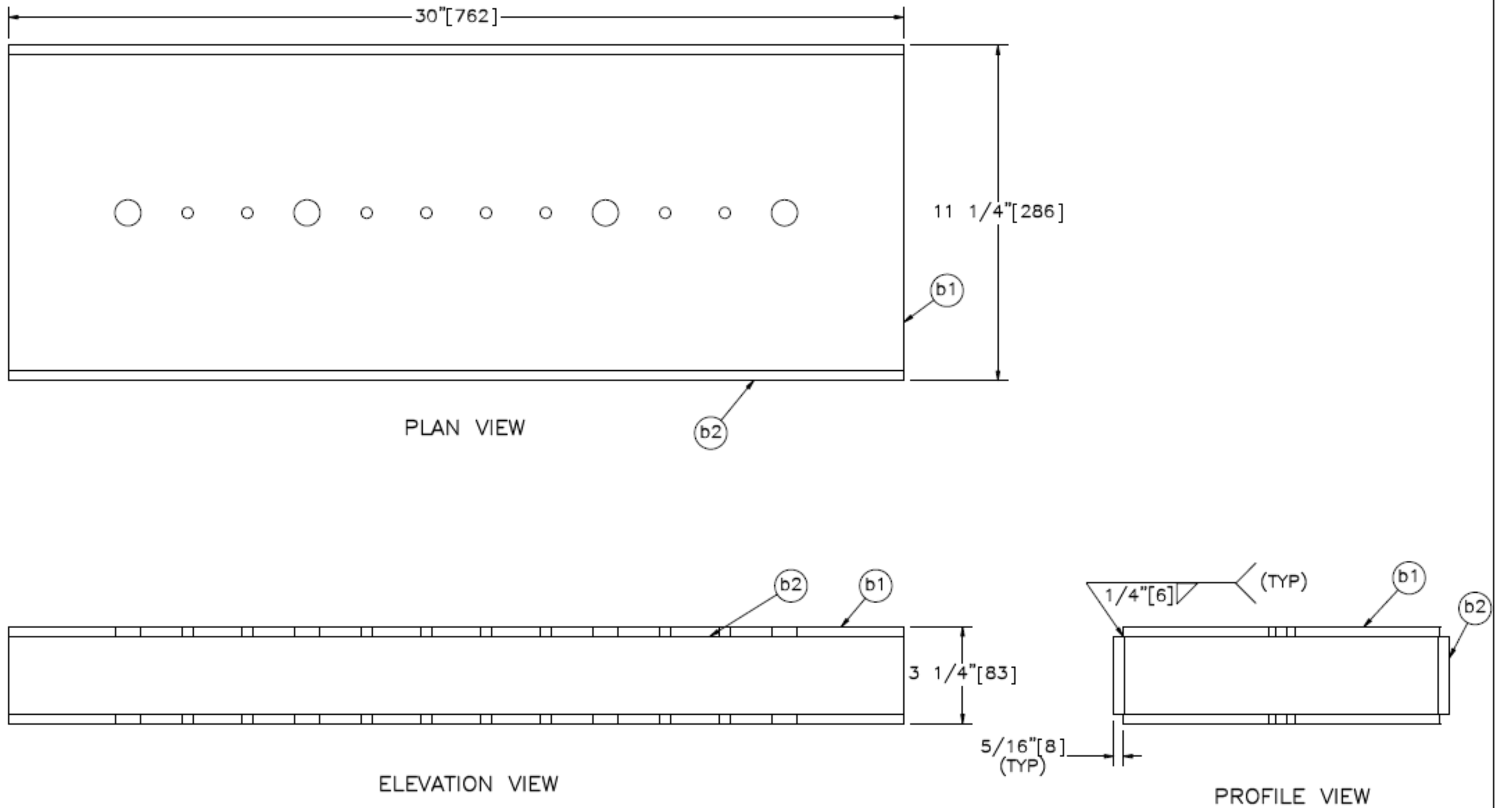


Figure 194. Top-Mounted Post Components, Test No. STBR-2



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	9	Upper Splice Tube Assembly – Option 1	–	ASTM A123
b1	2	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly
b2	2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly

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OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–2

Upper Splice Tube Assembly

SHEET: 13 of 25
DATE: 1/11/2019
DRAWN BY: JEK/MKB/MBD/AJS
REV. BY: JEK/OP/PM/RKF/JCH
DWG. NAME. STBR–2_R12
SCALE: 1:5
UNITS: in.[mm]

Figure 195. Upper Splice Tube Assembly, Test No. STBR-2

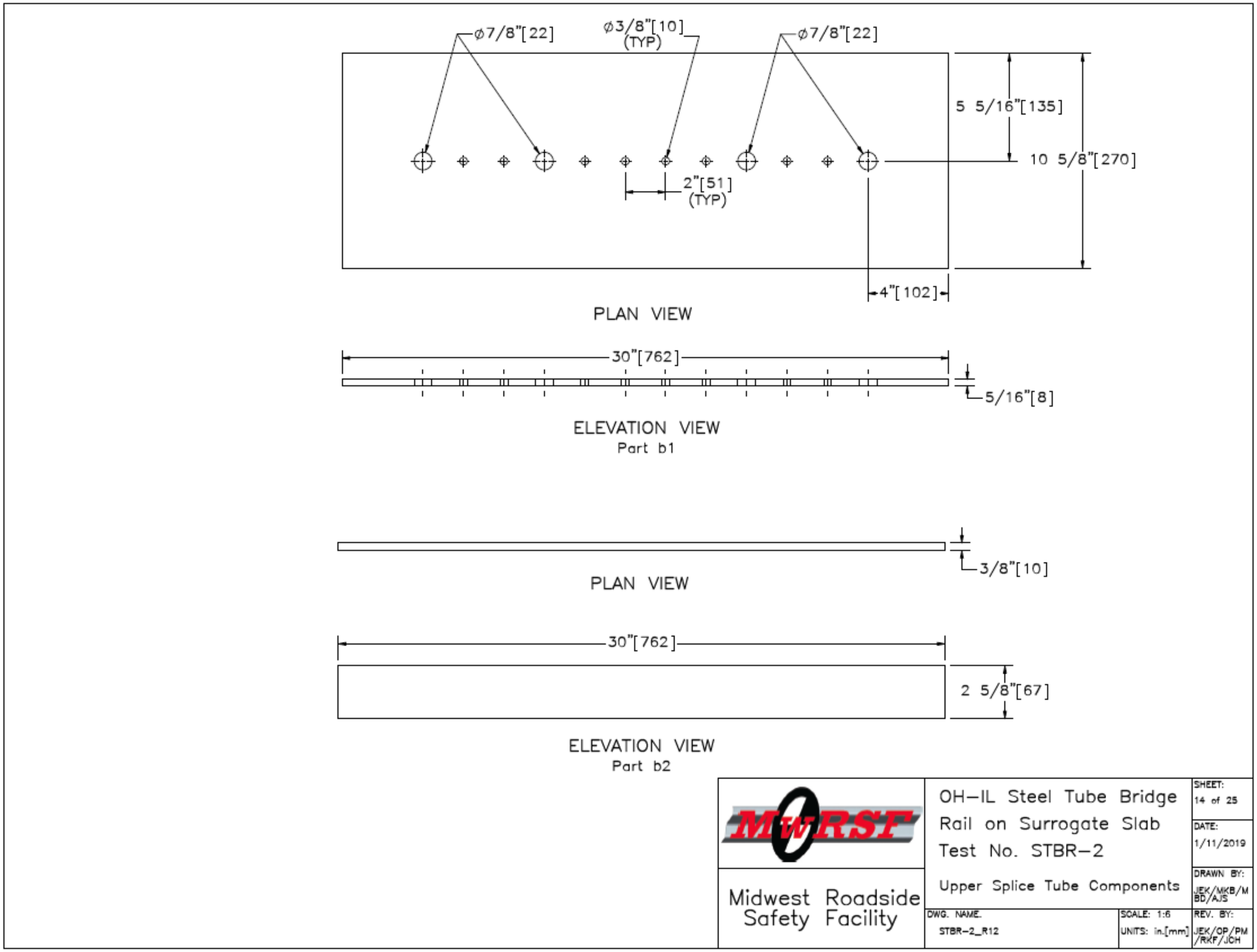
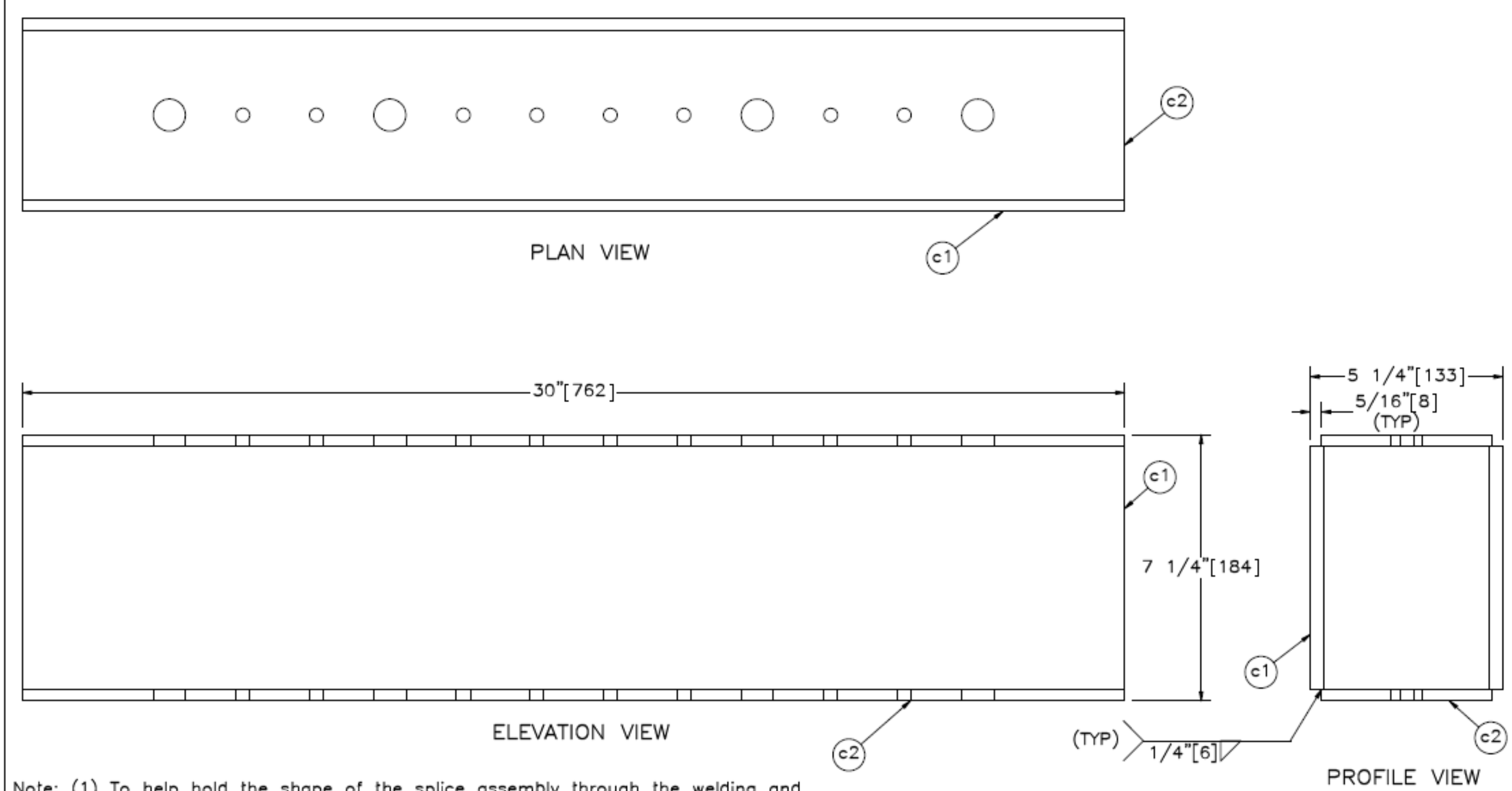


Figure 196. Upper Splice Tube Components, Test No. STBR-2



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	18	Middle/Lower Splice Tube Assembly – Option 1	–	ASTM A123
c1	2	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly
c2	2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly

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OH-IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR-2

Middle/Lower Splice Tube Assembly

SHEET: 15 of 25
DATE: 1/11/2019
DRAWN BY: JEK/MKB/MBD/AJS
DWG. NAME. STBR-2_R12
SCALE: 1:4
UNITS: in.[mm]
REV. BY: JEK/OP/PM/RKF/JOH

Figure 197. Middle/Lower Splice Tube Assembly, Test No. STBR-2

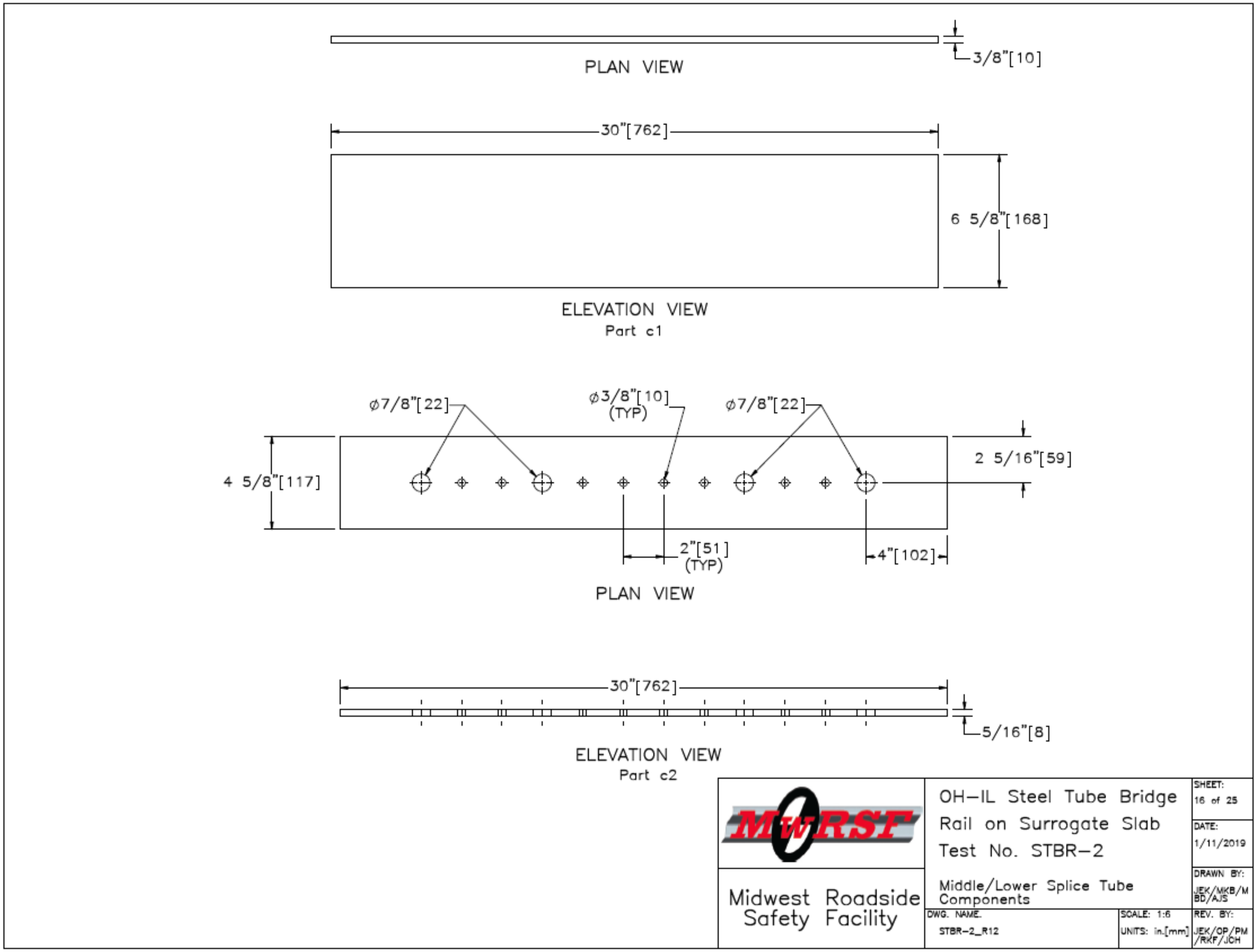


Figure 198. Middle/Lower Splice Tube Components, Test No. STBR-2

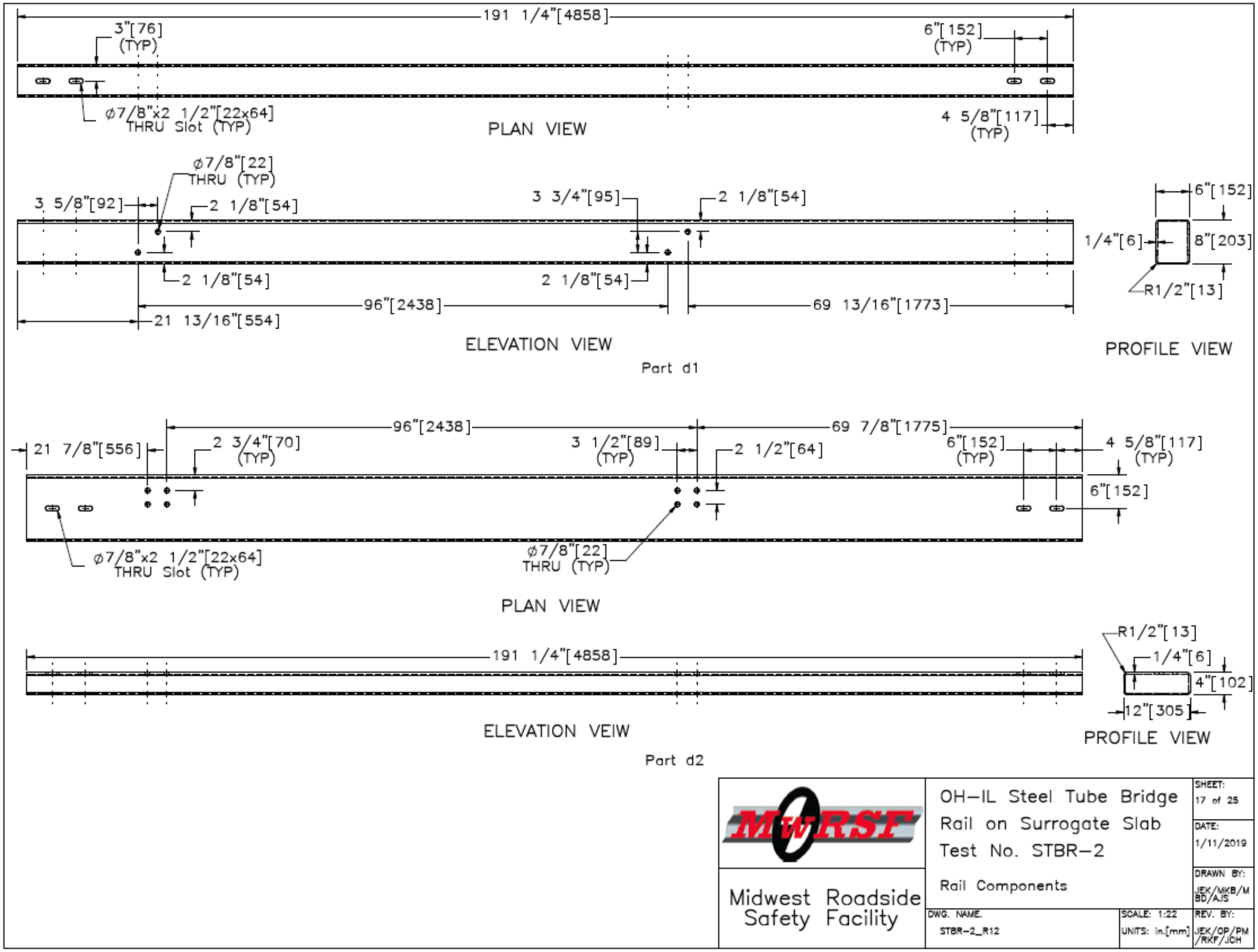


Figure 199. Rail Components, Test No. STBR-2

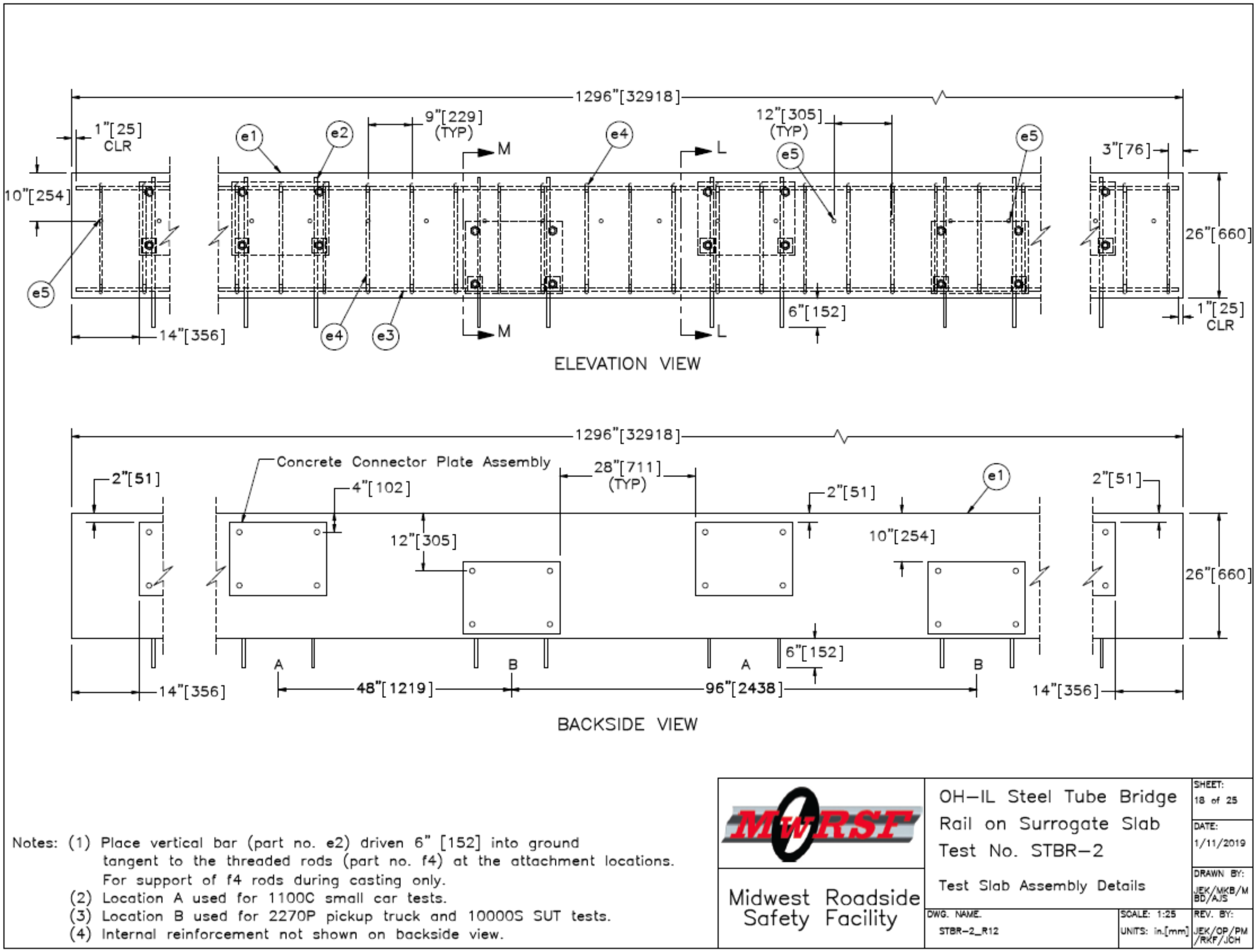
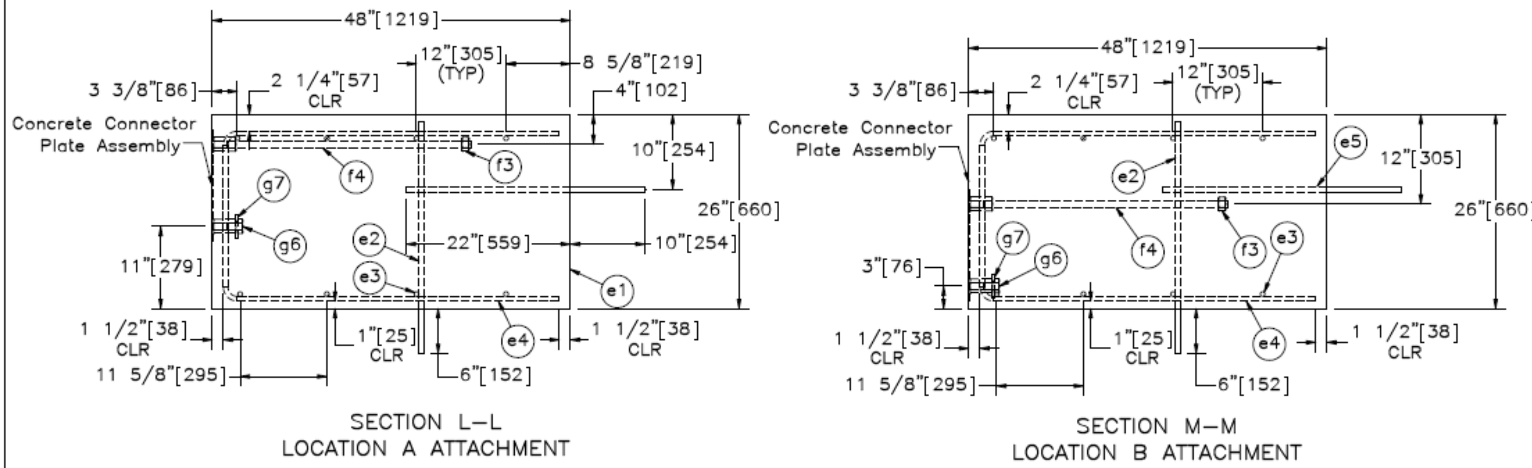


Figure 200. Test Slab Assembly Details, Test No. STBR-2



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Test Slab Assembly	–	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
–	27	Concrete Connector Plate Assembly	–	ASTM A123
f3	54	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
g6	54	1"–8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329
g7	54	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123

Notes:
(1) Location A used for 1100C small car tests.
(2) Location B used for 2270P pickup truck and 10000 SUT tests.
(3) Place vertical bar (part no. e2) driven 6" [152] into ground tangent to the threaded rods (part no. f4) at the attachment locations. For support during casting.
(4) Part f4 is threaded 1 1/4" [32] into part f2 (coupling nut) on Concrete Connector Plate Assembly.

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–2

Test Slab Assembly Details

SHEET: 19 of 25

DATE: 1/11/2019

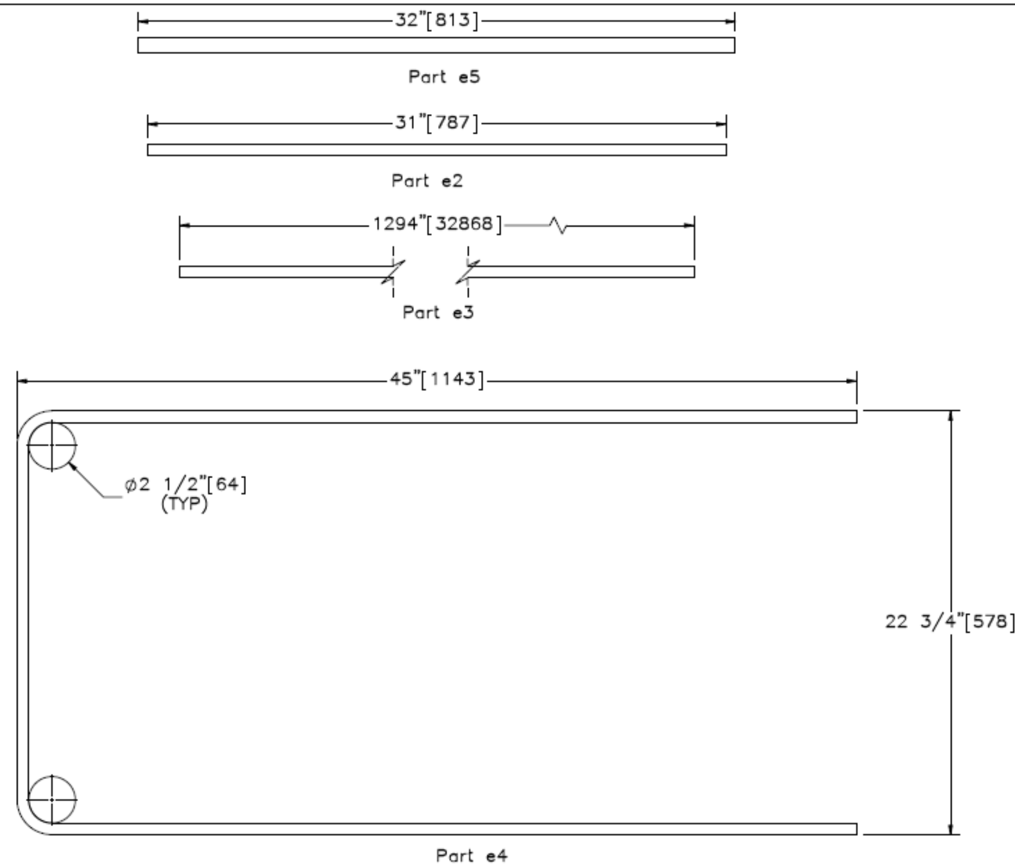
DRAWN BY: JEK/MKB/MBD/AJS

DWG. NAME. STBR–2_R12

SCALE: 1:20 UNITS: in.[mm]

REV. BY: JEK/OP/PM/RKF/JCH

Figure 201. Test Slab Assembly Details, Test No. STBR-2



Note: (1) Minimum lap length for a #5 bar is 18" [457].

Bill of Bars

Bar	QTY.	Size	Total Unbent Length	Material Specification	Treatment Specification
e2	54	#5	31" [787]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)
e3	8	#5	1294" [32868]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)
e4	144	#5	110 3/16" [2799]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)
e5	108	#6	32" [813]	ASTM A615 Gr. 60	Epoxy-Coated (ASTM A775 or ASTM A934)

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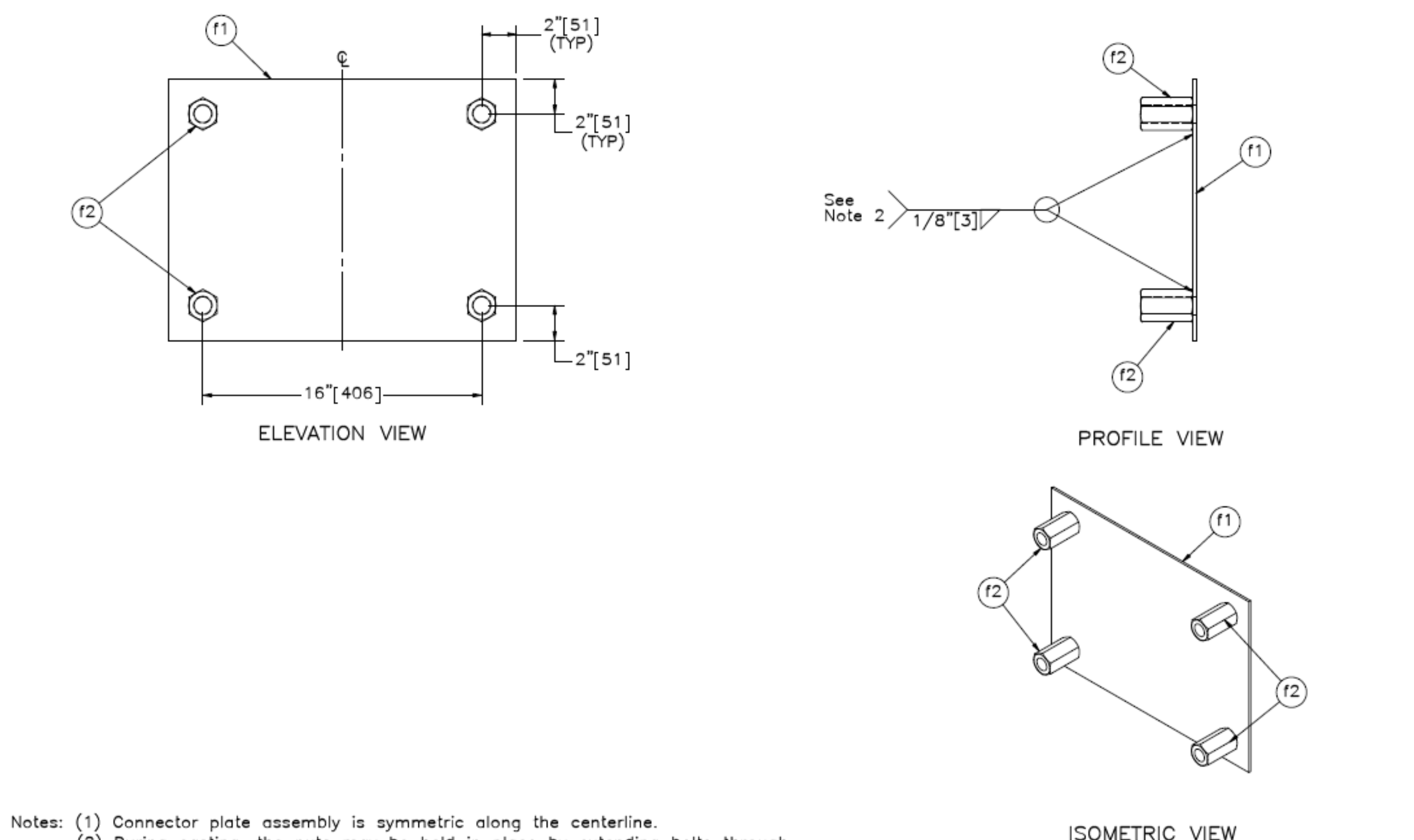
OH-IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR-2

Reinforcement Details

SHEET: 20 of 25
DATE: 1/11/2019
DRAWN BY: JEK/MKB/MBD/AJS
DWG. NAME. STBR-2_R12
SCALE: 1:8
UNITS: in.[mm]
REV. BY: JEK/OP/PM/RKF/JCH

Figure 202. Reinforcement Details, Test No. STBR-2



Notes: (1) Connector plate assembly is symmetric along the centerline.
(2) During casting, the nuts may be held in place by extending bolts through the formwork into the nuts, or by tack welding nuts to inside face of the plate.The welds are not structurally important.

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
–	27	Concrete Connector Plate Assembly	–	ASTM A123	–
f1	1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	See Assembly	–
f2	4	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–2

Concrete Connector Plate Assembly Details

SHEET: 21 of 25
DATE: 1/11/2019
DRAWN BY: JEK/MKB/MBD/AJS
DWG. NAME. STBR–2_R12
SCALE: 1:10
UNITS: in.[mm]
REV. BY: JEK/OP/PM/RKF/JCH

Figure 203. Concrete Connector Plate Assembly Details, Test No. STBR-2

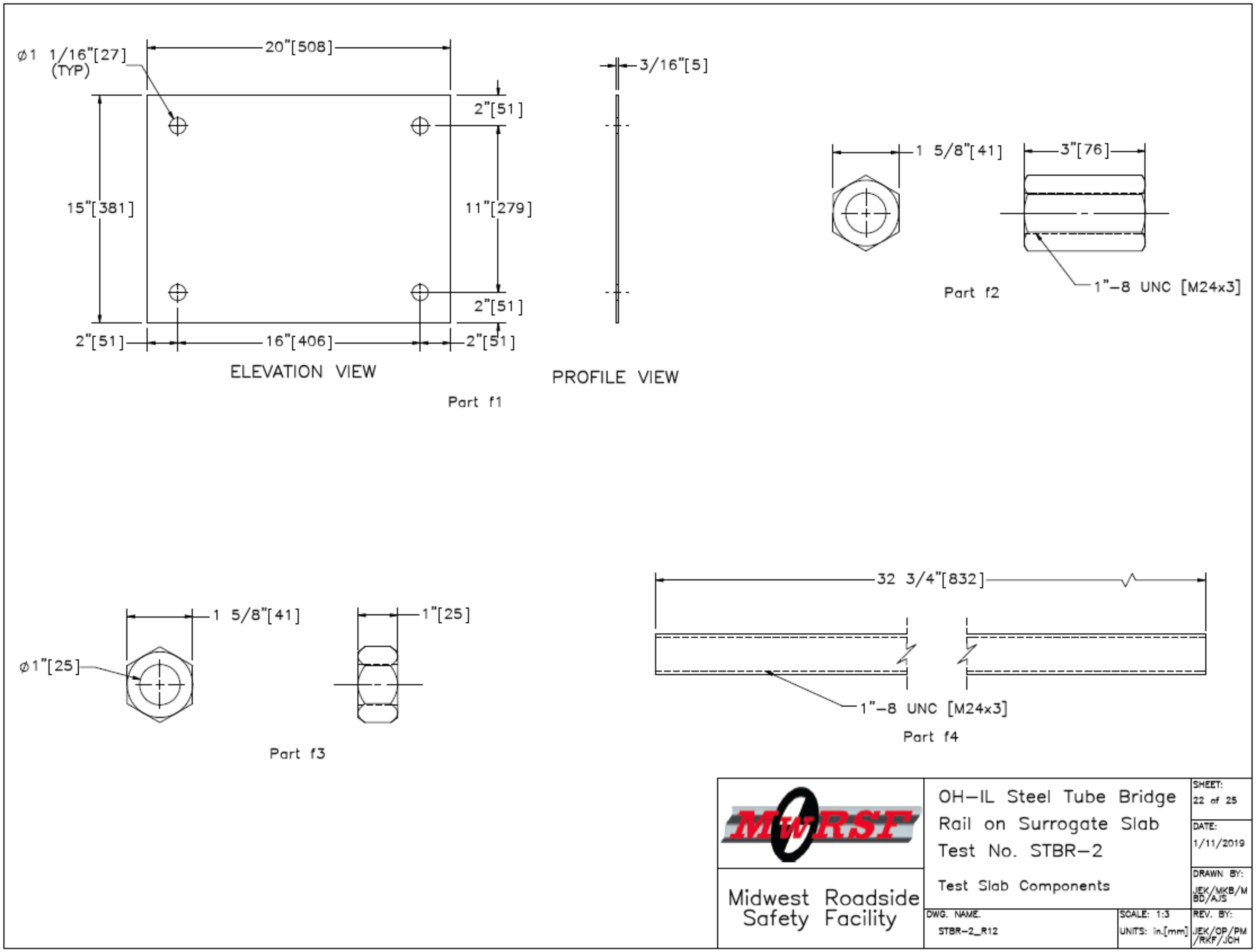


Figure 204. Test Slab Components, Test No. STBR-2

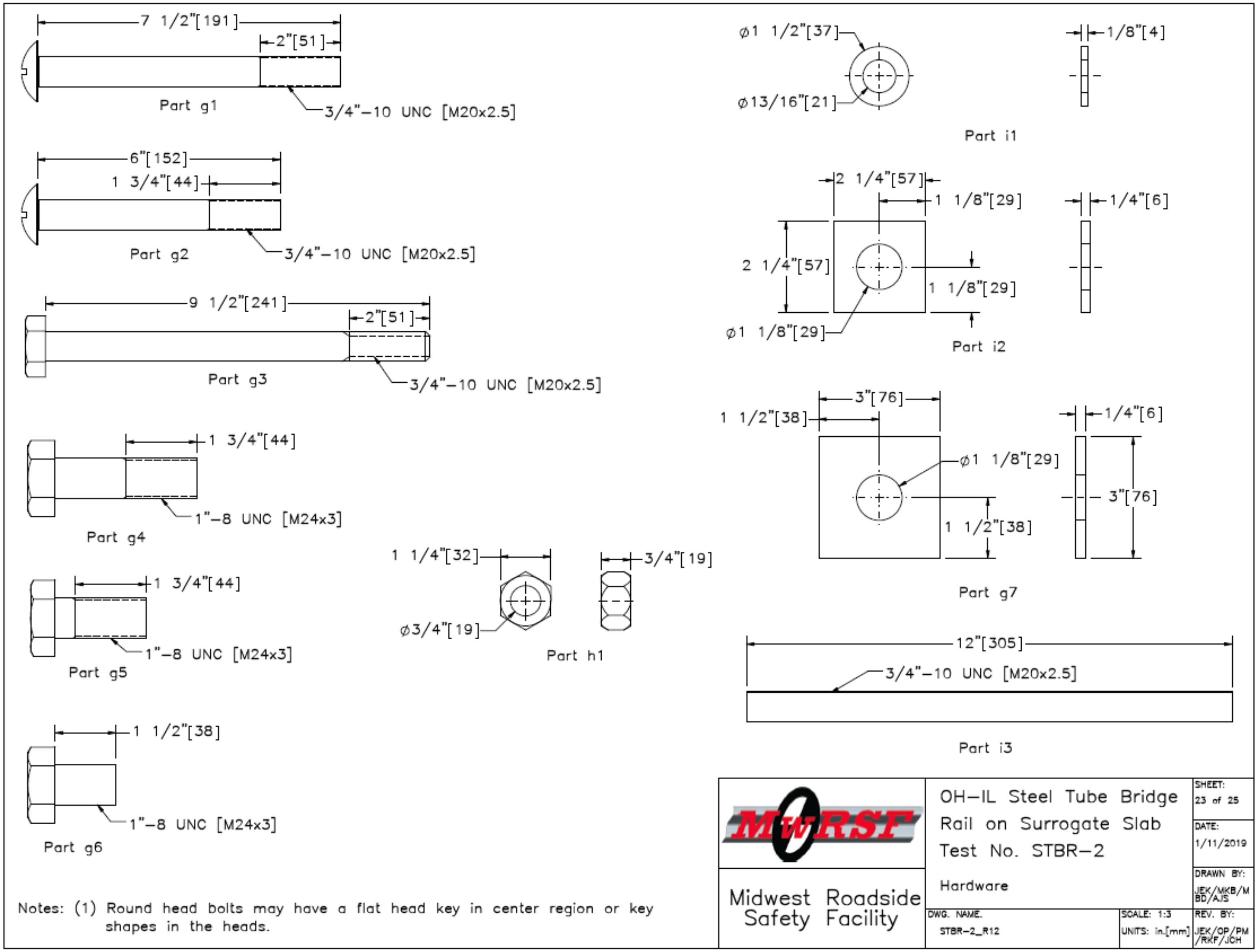


Figure 205. Hardware, Test No. STBR-2

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
a1	13	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	See Assembly	–
a2	20	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly	–
a3	13	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly	–
a4	52	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly	–
a5	26	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	ASTM A123	–
a6	7	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	–	–
a7	7	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	–	–
b1	18	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly	–
b2	18	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c1	36	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c2	36	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly	–
d1	20	HSS 8"x6"x1/4" [203x152x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
d2	10	HSS 12"x4"x1/4" [305x102x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
f1	27	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	ASTM A123	–
f2	108	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–
f3	106	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX24b
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329	FRR24b

MwRSF
Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–2

Bill of Materials

SHEET: 24 of 25
DATE: 1/11/2019
DRAWN BY: JEK/MKB/MBD/AJS
DWG. NAME. STBR–2_R12
SCALE: None
UNITS: in.[mm]
REV. BY: JEK/OP/PM/RKF/JCH

Figure 206. Bill of Materials, Test No. STBR-2

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
g1	80	3/4"–10 UNC [M20x2.5], 7 1/2" [191] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g2	116	3/4"–10 UNC [M20x2.5], 6" [152] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g3	72	3/4"–10 UNC [M20x2.5], 9 1/2" [241] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F2329	FBX20b
g4	52	1"–8 UNC [M24x3], 3 1/2" [89] 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
g5	52	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
g6	54	1"–8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX24b
g7	54	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123	–
h1	296	3/4 –10 UNC [M20x2.5] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX20b
i1	368	3/4" [19] Dia. Hardened Flat Washer	ASTM F436	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329	FWC20b
i2	156	2 1/4"x2 1/4"x1/4" [57x57x6] Square Washer	ASTM A36	ASTM A123	–
i3	28	3/4"–10 UNC [M20x2.5], 12" [305] Long Threaded Rod	ASTM F1554 Gr. 36	ASTM A153 or B695 Class 55 or F2329	FRR20a

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–2

Bill of Materials

SHEET: 25 of 25

DATE: 1/11/2019

DRAWN BY: JEK/MKB/MBD/AJS

REV. BY: JEK/OP/PM/RKF/JCH

DWG. NAME. STBR–2_R12

SCALE: None

UNITS: in.[mm]

Figure 207. Bill of Materials, Test No. STBR-2

Figure 208. Test Installation Photographs, Test No. STBR-2

Figure 209. Test Installation Photographs, Side-Mounted and Top-Mounted Posts, Test No. STBR-2

Figure 210. Bridge Railing End Views, Test No. STBR-2

Figure 211. Side-Mounted Post-to-Deck Connections, Test No. STBR-2

Figure 212. Splice Tubes, Test No. STBR-2

14 FULL-SCALE CRASH TEST NO. STBR-2

14.1 Weather Conditions

Test no. STBR-2 was conducted on February 22, 2019 at approximately 2:30 p.m. The weather conditions as per the National Oceanic and Atmospheric Administration (station 14939/LNK) were reported and are shown in Table 72.

Table 72. Weather Conditions, Test No. STBR-2

Temperature	33° F
Humidity	58%
Wind Speed	9 mph
Wind Direction	90° from True North
Sky Conditions	Overcast
Visibility	9 Statute Miles
Pavement Surface	Dry
Previous 3-Day Precipitation	5.4 in.
Previous 7-Day Precipitation	13.0 in.

14.2 Test Description

Initial vehicle impact was to occur 7 ft (2.1 m) upstream from post no. 9, as shown in Figure 213, which was selected as discussed in Chapter 9. In test no. STBR-2, the 5,157-lb (2,339-kg) Dodge quad cab pickup truck impacted the bridge rail at a speed of 64.5 mph (103.8 km/h) and at an angle of 24.7 degrees. The actual point of impact was 6 ft – 10 in. (2.1 m) upstream from post no. 9. The vehicle came to rest 248 ft – 6 in. (75.7 m) downstream from the original impact point and laterally 30 ft – 5 in. (9.3 m) in front of the bridge rail after brakes were applied.

The crash testing parameters are summarized in Table 73. A detailed description of the sequential impact events is contained in Table 74. Sequential photographs are shown in Figures 214 and 215. Documentary photographs of the crash test are shown in Figure 216. The vehicle trajectory and final position are shown in Figure 217.

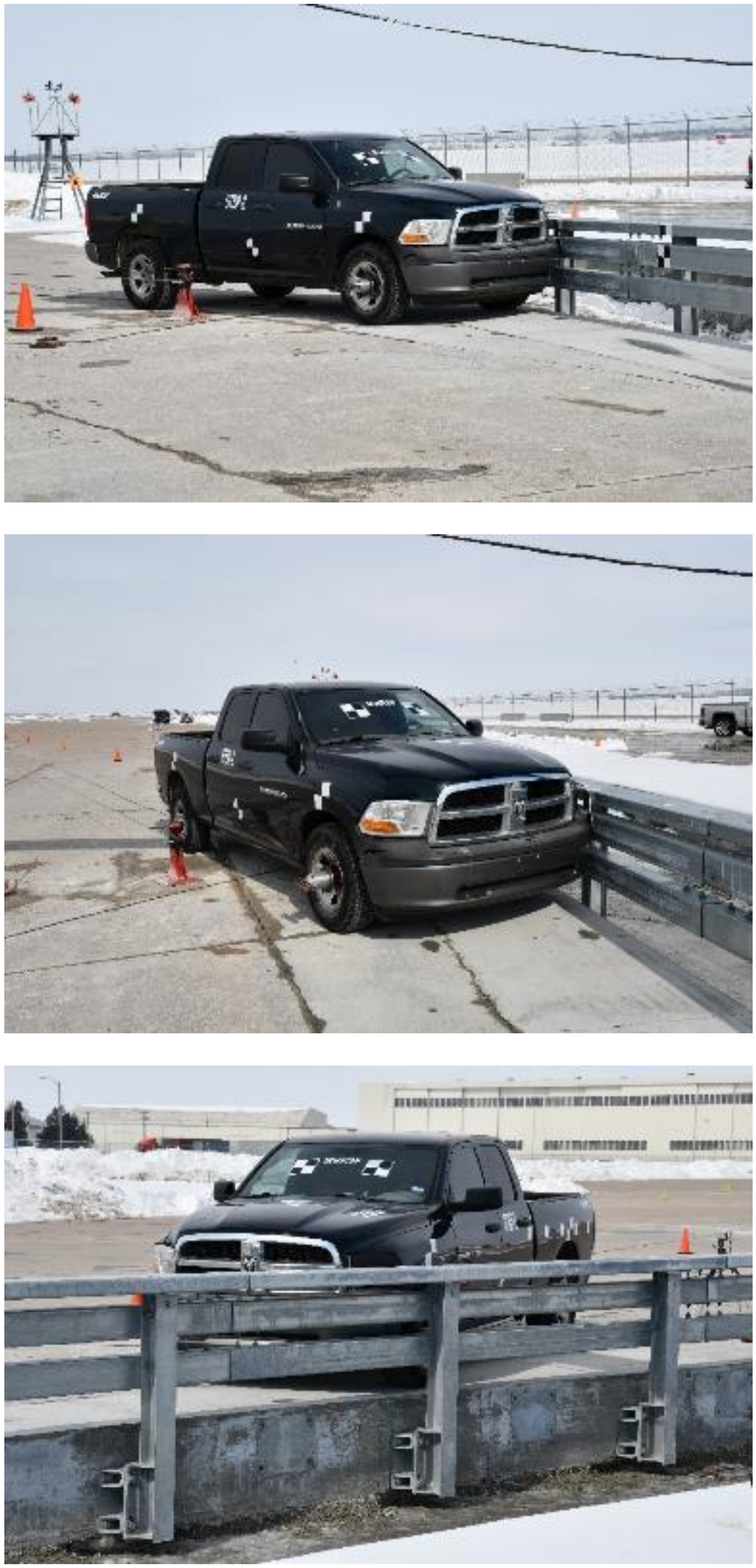


Figure 213. Impact Location, Test No. STBR-2

Table 73. Actual, Lower-Bound, and Target Crash Test Parameters, Test No. STBR-2

Test Parameter	Actual	Lower-Bound	Target
Speed	64.5 mph	59.5 mph	62.0 mph
Angle	24.6 deg.	23.5 deg.	25.0 deg.
Impact Severity	120.9 kip-ft	105.6 kip-ft	115.4 kip-ft

Table 74. Sequential Description of Impact Events, Test No. STBR-2

Time (sec):	**Event Description**
0.000	Vehicle front bumper contacted rail between post nos. 8 and 9.
0.002	Vehicle bumper deformed.
0.004	Vehicle left headlight contacted rail.
0.006	Vehicle left fender contacted rail.
0.008	Vehicle left fender deformed. Vehicle left-front tire contacted rail.
0.010	Post no. 8 deflected backward.
0.016	Post no. 9 deflected backward.
0.020	Vehicle grille contacted rail.
0.024	Post no. 10 deflected backward, Vehicle yawed away from barrier.
0.030	Post no. 7 deflected backward and vehicle rolled toward barrier.
0.048	Vehicle left-front door flexed away from door frame. Vehicle left-front door contacted rail.
0.054	Vehicle left-front door deformed. Vehicle left-rear door flexed away from door frame.
0.056	Post no. 11 deflected backward.
0.070	Vehicle grille became disengaged.
0.078	Vehicle right-front tire became airborne.
0.100	Vehicle right-rear tire became airborne.
0.102	Vehicle left headlight cracked.
0.118	Vehicle right headlight became disengaged.
0.122	Vehicle left headlight shattered.
0.142	Vehicle left-rear door contacted rail.
0.146	Vehicle was parallel to system at a speed of 53.9 mph (86.7 km/h).
0.148	Vehicle left quarter panel contacted rail.
0.150	Vehicle left quarter panel deformed. Vehicle left taillight contacted rail.
0.154	Vehicle rear bumper contacted rail.
0.170	Vehicle left headlight became disengaged.
0.202	Vehicle yawed toward the barrier.
0.224	Vehicle pitched downward.
0.252	Vehicle left-rear tire became airborne.
0.326	Vehicle exited system at a speed of 53.1 mph (85.4 km/h).
0.354	Vehicle left-rear tire regained contact with ground.
0.426	Vehicle rolled away from the barrier.
0.566	Vehicle pitched upward.
0.640	Vehicle right-rear tire regained contact with ground.
0.668	Vehicle right-front tire regained contact with ground.
0.814	Vehicle rolled toward the barrier.
0.932	Vehicle pitched downward.
0.106	Vehicle right-front tire became airborne.
0.109	Vehicle rolled away from the barrier.
0.116	Vehicle right-front tire regained contact with ground.
0.130	Vehicle rolled toward the barrier.

0.000 sec

0.200 sec

0.400 sec

0.600 sec

0.800 sec

1.000 sec

0.000 sec

0.200 sec

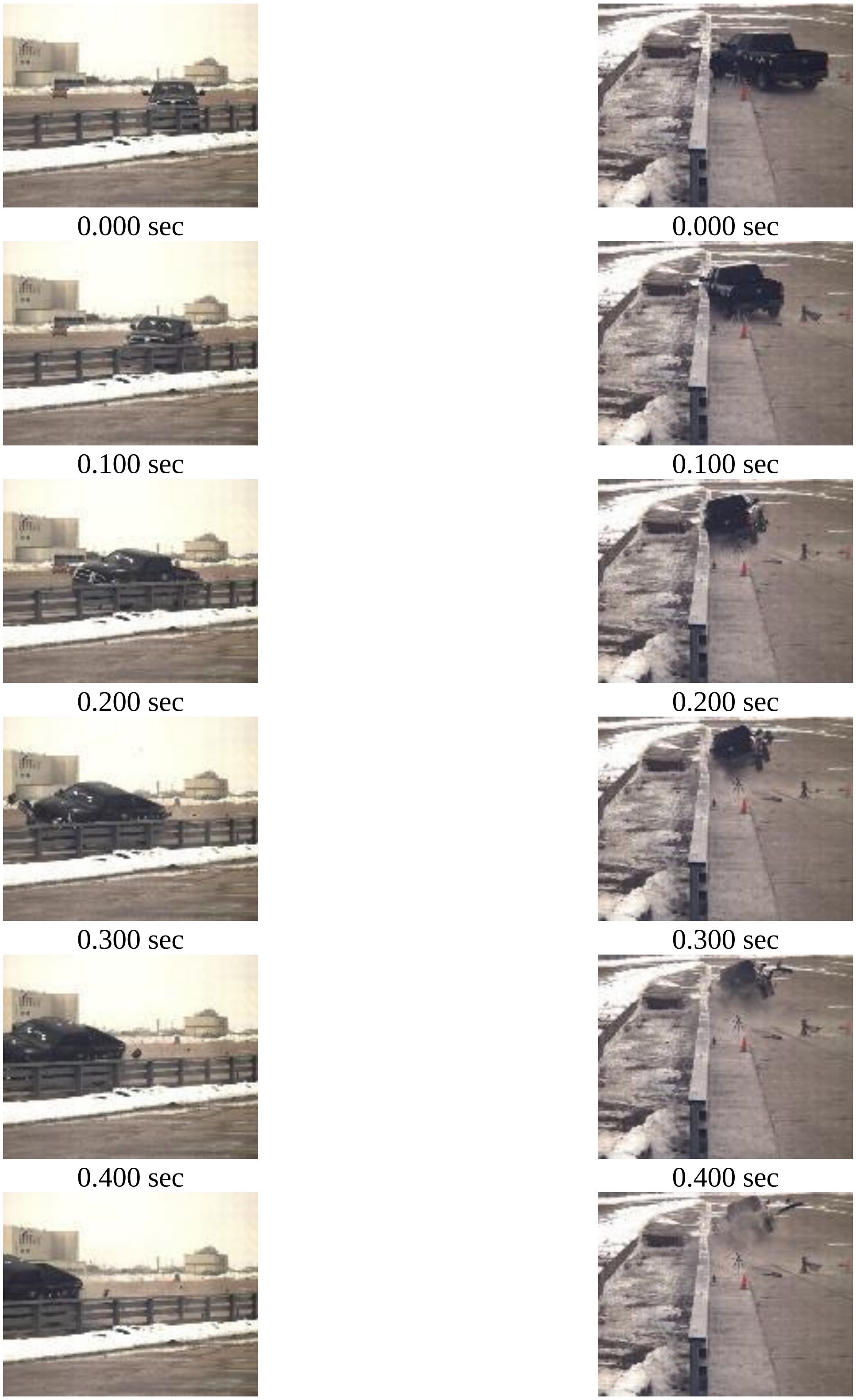
0.400 sec

0.600 sec

0.800 sec

1.000 sec

Figure 214. Sequential Photographs, Test No. STBR-2



0.000 sec | 0.000 sec
0.100 sec | 0.100 sec
0.200 sec | 0.200 sec
0.300 sec | 0.300 sec
0.400 sec | 0.400 sec
0.500 sec | 0.500 sec

Figure 215. Additional Sequential Photographs, Test No. STBR-2

Figure 216. Documentary Photographs, Test No. STBR-2

Figure 217. Vehicle Final Position and Trajectory Marks, Test No. STBR-2

14.3 System Damage

Damage to the barrier was minimal, as shown in Figures 218 through 227. Note that shrinkage cracks were observed in the surrogate slab prior to the test and denoted with a black marker. System damage consisted of contact marks, scrapes, and dents on the rails, plastic hinges at post nos. 8 and 9 for a three-span collapse, and concrete cracks near the post-to-deck connections of post nos. 9 and 10. The length of vehicle contact along the barrier was approximately 14 ft – 7½ in. (4.5 m), which spanned from 1 ft – 7¼ in. (0.5 m) upstream from the center of post no. 8 to 5 ft – ¼ in. (1.5 m) downstream from the center of post no. 9.

Plastic hinges were found 4 in. (102 mm) above the location of the tension anchor rods for post nos. 8 and 9. Contact marks were visible on the front faces of the top and middle rails starting at 19¼ in. (489 mm) upstream form the center of post no. 8 and extending to 5 ft – ¼ in. (1.5 m) downstream from the center of post no. 9. Tire marks were found on the front face of the bottom rail starting 2 in. (51 mm) upstream from post no. 8 center and extending to 3 ft – 8¼ in. (1.1 m) downstream from the center of post no. 9. Denting was found in the front face of the bottom rail located 1 ft – 2 in. (0.4 m) downstream from the centerline of post no. 8 and ending 2 ft – 5½ in. (0.7 m) upstream from post no. 9. Scuff marks were also found on the front face of the bottom rail starting 1 ft – 2 in. (0.4 m) downstream from the centerline of post no. 8 and extending to 2 ft – 4 in. (0.7 m) upstream from the centerline of post no. 9. Further, 2-in. (51-mm) tall tire marks were observed at the left side of the front flange of post no. 9 located 13 in. (330 mm) above the tension anchor rods. Post no. 10 slightly bent backward at the height of the top stiffeners.

Concrete spalling cracks were found at the bottom edge of the concrete deck extending 4 ft – ½ in. (1.2 m) longitudinally and 11 in. (279 mm) above the bottom edge of the deck at post no. 9. Hairline concrete cracks were found at the top-left and top-right corners of the embedded plate of post no. 10.

Figure 218. System Damage, Test No. STBR-2

Figure 219. System Damage, Rail Span Between Posts Nos. 7 and 8, Test No. STBR-2

Figure 220. System Damage, Rail Span Between Posts Nos. 8 and 9, Test No. STBR-2

Figure 221. System Damage, Rail Span Between Posts Nos. 9 and 10, STBR-2

Figure 222. System Damage, Post No. 8, Test No. STBR-2

Figure 223. System Damage, Post No. 9, Test STBR-2

Figure 224. System Damage, Post No. 10, Test No. STBR-2

Figure 225. System Damage, Concrete Damage at Post No. 9, Test No. STBR-2

Figure 226. System Damage, Concrete Damage at Post No. 9, Test No. STBR-2

Figure 227. System Damage, Concrete Damage at Post No. 10, Test No. STBR-2

The maximum lateral permanent set of the barrier system was 3.5 in. (89 mm). The maximum lateral dynamic barrier deflection was 7.0 in. (178 mm) at the top rail expansion gap between post nos. 8 and 9, as determined from high-speed digital video analysis. The working width of the system was found to be 19.0 in. (483 mm), also determined from high-speed digital video analysis. A schematic of the permanent set deflection, dynamic deflection, and working width is shown in Figure 228.

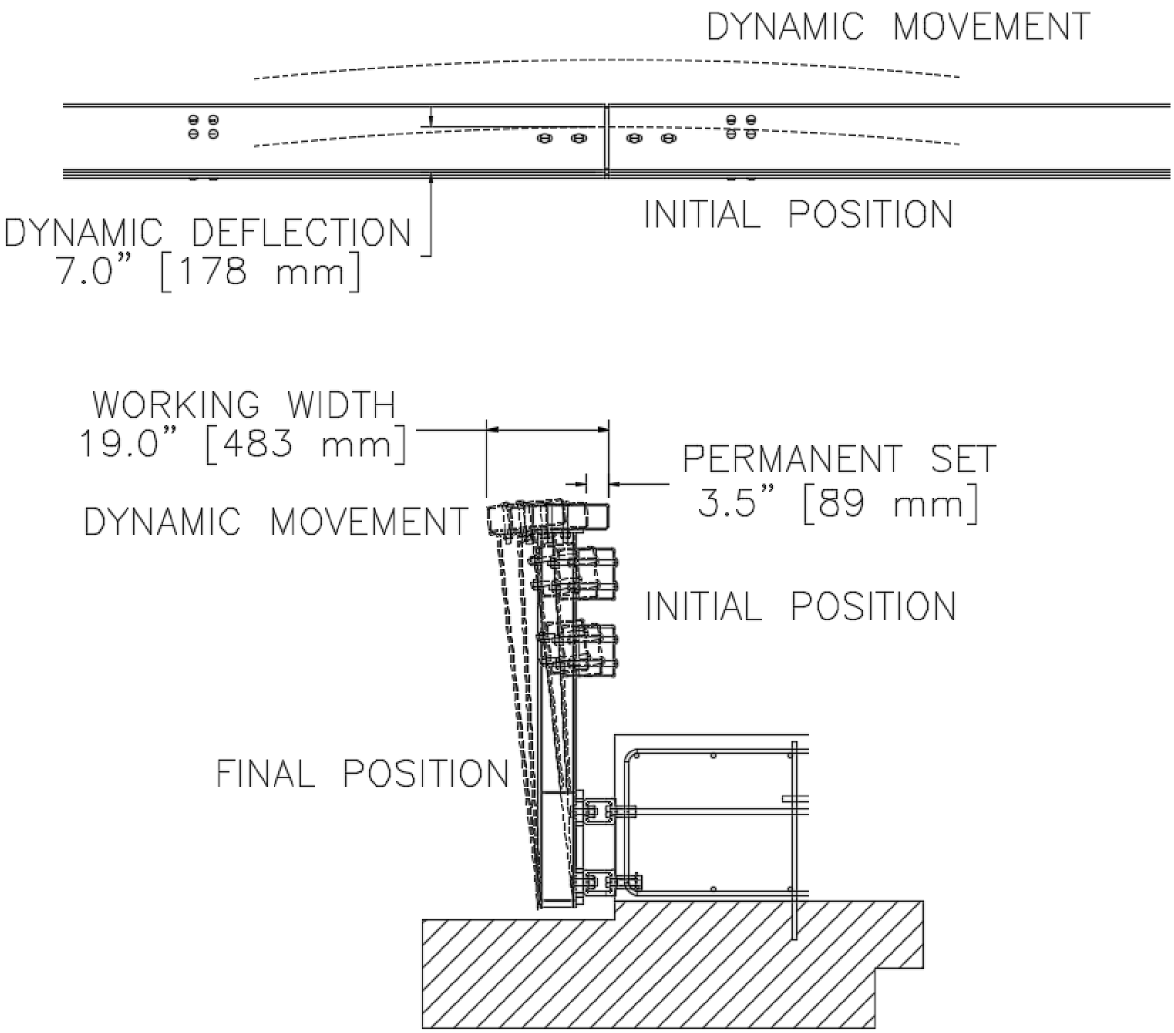


Figure 228. Permanent Set Deflection, Dynamic Deflection, and Working Width, Test No. STBR-2

14.4 Vehicle Damage

The damage to the vehicle was moderate, as shown in Figures 229 through 233. The maximum occupant compartment deformations are listed in Table 75 along with the intrusion limits established in MASH 2016 for various areas of the occupant compartment. MASH 2016 defines intrusion or deformation as the occupant compartment being deformed and reduced in size with no observed penetration. Note that none of the established MASH 2016 deformation limits were violated. Complete occupant compartment and vehicle deformations and the corresponding locations are provided in Appendix F.

Majority of the damage was concentrated on the left-front corner, left-front fender, and left side of the box where the impact had occurred. The left-front corner of the bumper was crushed inward and back. The left-front fender was pushed upward near the door panel and was dented and torn behind the left-front wheel. The left-front steel rim was deformed with tears and significant crushing. The left-side and right-side headlights were removed from the vehicle. The left-front door was dented and scraped. The left-rear door was crushed approximately 1 in. (25 mm).

Denting and scraping were observed along the entire left side of the vehicle. The right-front door was ajar, and creases were found in the door's sheet metal. The right-rear wheel assembly was deformed inward. The left taillight was removed. The left side of the rear bumper was dented and scuffed. The left-front tie rod was bent, and the steering rack was broken at the pinion gear. The transmission slightly shifted and rotated. The oil pan shifted with the transmission and engine. The engine mount was broken and disengaged. The cross members of the engine and transmission were severely bent due to compression from impact load. The left-side frame horn was bent toward the center of the vehicle slightly. The rear cab mount was slightly bent.

Figure 229. Vehicle Damage, Front and Rear Views, Test No. STBR-2

Figure 230. Vehicle Damage, Right and Left Views, Test No. STBR-2

Figure 231. Vehicle Damage, Right-Corner Views, Test No. STBR-2

Figure 232. Vehicle Damage, Left-Corner Views, Test No. STBR-2

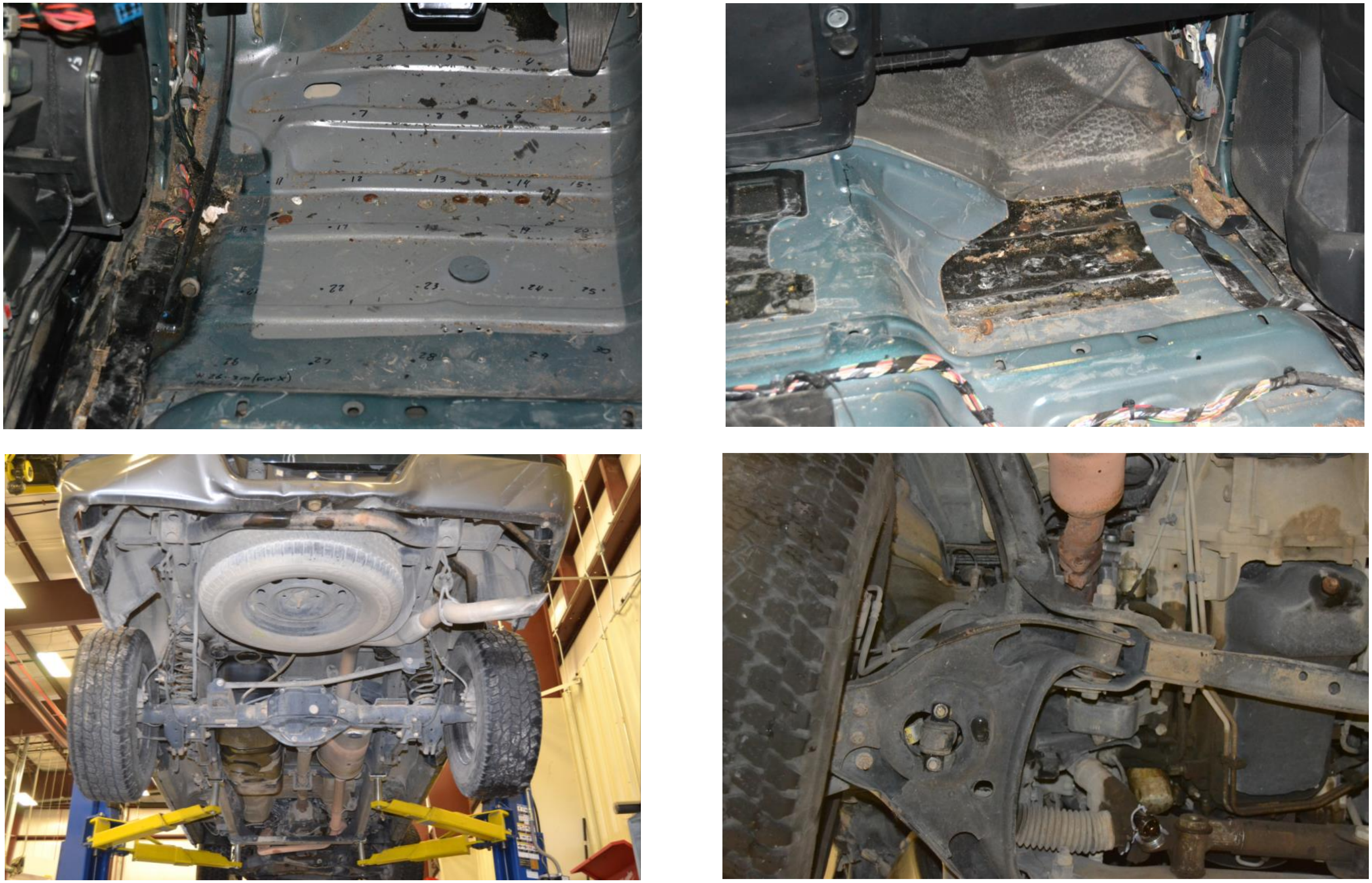


Figure 233. Vehicle Damage, Floor Pan and Undercarriage, Test No. STBR-2

Table 75. Maximum Occupant Compartment Intrusions by Location, Test No. STBR-2

LOCATION	MAXIMUM INTRUSION in. (mm)	MASH 2016 ALLOWABLE INTRUSION in. (mm)
Wheel Well & Toe Pan	0.3 (7.6)	≤ 9 (229)
Floor Pan & Transmission Tunnel	0.0 (0.0)	≤ 12 (305)
A-Pillar	0.4 (10.2)	≤ 5 (127)
A-Pillar (Lateral)	0.1 (2.5)	≤ 3 (76)
B-Pillar	0.2 (5.1)	≤ 5 (127)
B-Pillar (Lateral)	0.0 (0.0)	≤ 3 (76)
Side Front Panel (in Front of A-Pillar)	0.9 (22.9)	≤ 12 (305)
Side Door (Above Seat)	0.3 (7.6)	≤ 9 (229)
Side Door (Below Seat)	0.1 (2.5)	≤ 12 (305)
Roof	0.3 (7.6)	≤ 4 (102)
Windshield	0.0 (0.0)	≤ 3 (76)
Side Window	Intact	No shattering due to contact with structural member of test article
Dash	0.4 (10.2)	N/A

N/A – Not applicable

14.5 Occupant Risk

The calculated occupant impact velocities (OIVs) and maximum 0.010-sec average occupant ridedown accelerations (ORAs) in both the longitudinal and lateral directions, as determined from the accelerometer data, are shown in Table 76 and Figure 236. The recorded data from the accelerometers and the rate transducers are shown graphically in Appendix H. Note, the SLICE-2 unit was designated as the primary unit during this test as it was mounted closer to the c.g. of the vehicle.

Table 76. Summary of OIV, ORA, THIV, PHD, and ASI Values, Test No. STBR-2

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

14.6 2,270P Peak Lateral Impact Force Calculation

The longitudinal and lateral vehicle accelerations, as measured at the vehicle's c.g., were also processed using a SAE CFC-60 filter and 50-msec moving average. The 50-msec moving average vehicle accelerations were then combined with the uncoupled yaw angle versus time data in order to estimate the vehicular loading applied to the barrier system. From the data analysis, the perpendicular impact force was determined for the bridge rail, as shown in Figures 234 and 235. The maximum perpendicular (i.e., lateral) load imparted to the barrier was 72.1 kips (321 kN) and 82.0 kips (365 kN), as determined by the SLICE-1 and SLICE-2, respectively. Note that SLICE-2 was the primary accelerometer unit.

14.7 Discussion

The analysis of the results for test no. STBR-2 showed that the system adequately contained and redirected the 2270P vehicle with controlled lateral displacements of the barrier. A summary of the test results and sequential photographs are shown in Figure 236. Detached elements, fragments, or other debris from the test article did not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or work-zone personnel. Deformations of, or intrusions into, the occupant compartment that could have caused serious injury did not occur. The test vehicle did not penetrate nor ride over the barrier and remained upright during and after the collision. Vehicle roll, pitch, and yaw angular displacements, as shown in Appendix H, were deemed acceptable as they did not adversely influence occupant risk nor cause rollover. After impact, the vehicle exited the barrier at a trajectory angle of 6.8 degrees, and its trajectory did not violate the bounds of the exit box. Therefore, test no. STBR-2 was determined to be acceptable according to the MASH 2016 safety performance criteria for test designation no. 4-11.

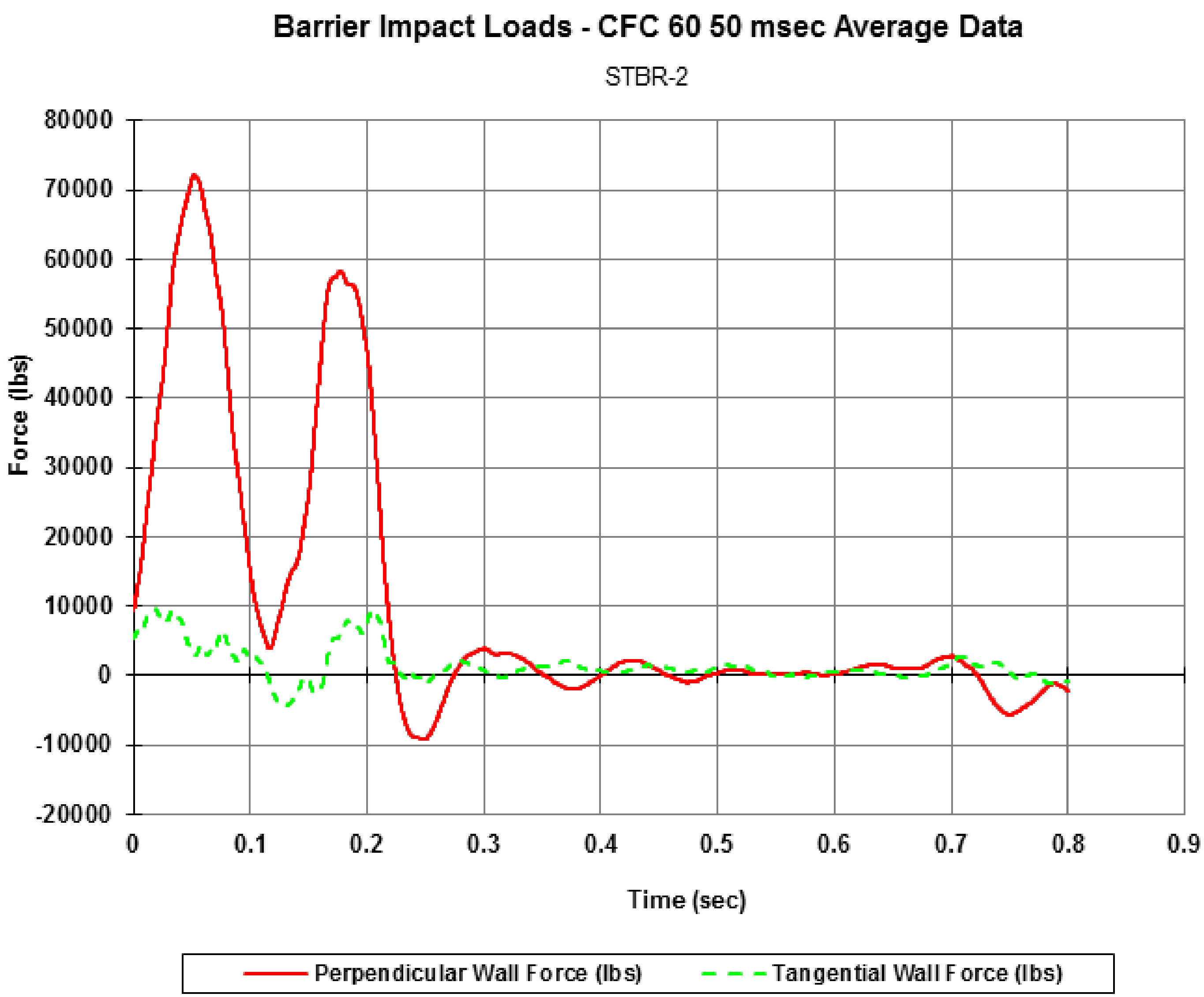


Figure 234. Perpendicular and Tangential Forces Imparted to the Barrier System (SLICE-1), Test No. STBR-2

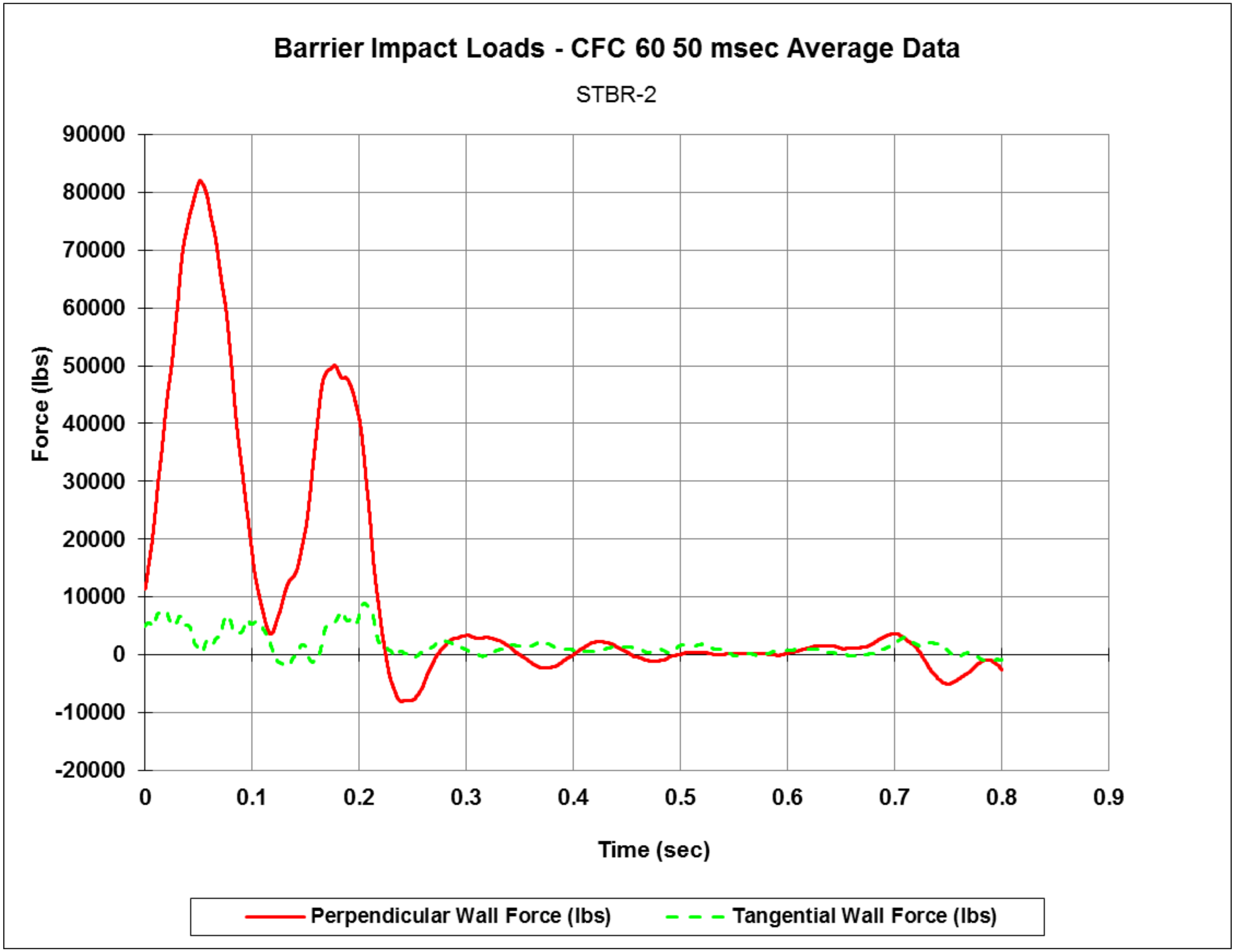
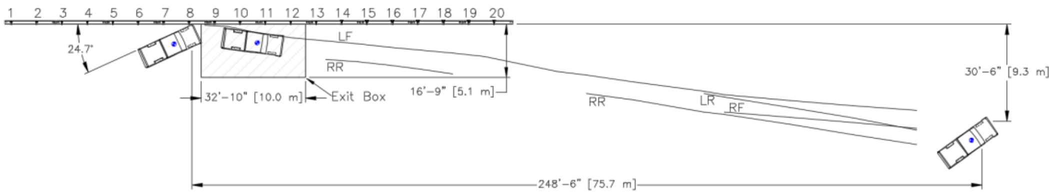
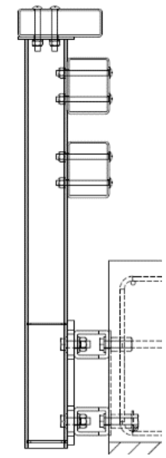


Figure 235. Perpendicular and Tangential Forces Imparted to the Barrier System (SLICE-2), Test No. STBR-2

0.000 sec | 0.100 sec | 0.200 sec | 0.300 sec | 0.400 sec





- Test Agency .... MwRSF
- Test Number STBR-2
- Date 2/22/2019
- MASH 2016 Test Designation No. 4-11
- Test Article .... Steel, Side-Mounted, Beam-and-Post, Bridge Rail
- Total Length .... 159 ft – 11½ in. (48.8 m)
- Key Component – Top Rail
  - Length .... 191¼ in. (4,858 mm)
  - Width .... 12 in. (305 mm)
  - Depth .... 4 in. (102 mm)
- Key Component - Post
  - Length .... 58½ in. (1,486 mm)
  - Width .... 6 in. (152 mm)
  - Spacing .... 8 ft (2.4 m)
- Soil Type
- Vehicle Make /Model .... 2011 Dodge Ram 1500
  - Curb .... 4,938 lb (2,240 kg)
  - Test Inertial .... 4,992 lb (2,264 kg)
  - Gross Static .... 5,157 lb (2,339 kg)
- Impact Conditions
  - Speed .... 64.5 mph (103.8 km/h)
  - Angle .... 24.7 deg.
  - Impact Location .... 6 ft – 10 in. (2.1 m) upstream from post no. 9
- Impact Severity 120.9 kip-ft (163.9 kJ) > 105.6 kip-ft (143.1 kJ) limit from MASH 2016
- Exit Conditions
  - Speed .... 53.1 mph (85.4 km/h)
  - Angle .... 6.8 deg.
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 248 ft – 6 in. (75.7 m) DS from impact
  30 ft – 6 in. laterally in front
- Vehicle Damage .... Moderate
  - VDS [56] 11-LFQ-5
  - CDC [57] 11-LFAW-5
  - Maximum Interior Deformation .... 0.9 in. (23 mm)
- Test Article Damage .... Minimal
- Maximum Test Article Deflections
  - Permanent Set .... 3.5 in. (89 mm)
  - Dynamic .... 7.0 in. (178 mm)
  - Working Width .... 19.0 in. (483 mm)
- Transducer Data

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

Figure 236. Summary of Test Results and Sequential Photographs, Test No. STBR-2

15 CONSTRUCTION DETAILS TEST NO. STBR-3

The test installation for the bridge rail system consisted of steel rails, posts assemblies, post-to-rail and rail-to-rail connections, as well as a surrogate concrete bridge deck, as shown in Figures 237 through 258. The total length of the bridge rail was 111 ft – 11¼ in. long (34.1 m). Photographs of the test installation are shown in Figures 259 through 262 . Material specifications, mill certifications, and certificates of conformity for the system materials are shown in Appendix D. After test no. STBR-2, post nos. 7, 8, 9, and 10, and the nearest two railing elements for each of the three rails, were replaced for test no. STBR-3.

The system was constructed with fourteen galvanized ASTM A992, W6x15 (W150x22.5) steel post assemblies spaced on 96 in. (2,438 mm) centers. Post assembly nos. 1 through 14 were side-mounted to the vertical side edge of the surrogate, reinforced-concrete bridge deck. For the construction of the surrogate concrete bridge deck, the threaded rod and coupling nuts were held in place to the embedded vertical plates by placing bolts through the formwork, rather than utilizing the option of welding the coupling nuts to the embedded vertical plates, as shown in Figure 254.

Post assembly nos. 1 through 14 were 58⅞ in. (1,495 mm) long. An ASTM A572 Grade 50 steel plate PL 8 in. x 8 in. x ⅜ in. (PL 203 mm x 203 mm x 10 mm) was attached to the top of each post assembly with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Similarly, an ASTM A572 Grade 50 steel, vertical plate PL 13-in. x 17¾-in. x 1-in. (PL 330 mm x 451 mm x 25 mm) was attached to bottom of the front flange of each post with all-around ¼-in. (6.4-mm) fillet welds. Four gusset plates, fabricated with ASTM A572 Grade 50 steel plate, measuring PL 6⅛ -in. x 5$^{11}/_{16}$-in. x ¼-in. (PL 156-mm x 144-mm x 6.4-mm), were welded to the top and bottom of the vertical plates, inner faces of post flanges, and web with all-around ¼-in. (6.4-mm) fillet welds. Post assembly nos. 1 through 14 were bolted to the tension and compression sides of the vertical plates with ASTM A500 Grade 50 horizontal spacer tubes, which were specified as HSS 5-in. x 4-in. x ½-in. (HSS 127-mm x 102-mm x 13-mm) sections.

Post assembly nos. 1 through 14 were bolted to the horizontal spacer tubes using ASTM F3125 Grade A325 1-in. (25-mm) diameter by 3½-in. (89-mm) long, heavy hex-head bolts with ¼-in. (6.4-mm) thick, ASTM A36 steel square washers and 1-in. (25-mm) diameter ASTM A563DH heavy hex nuts. The deck anchorage in the tension region consisted of two 1-in. (25-mm) diameter by 32¾-in. (832-mm) long, ASTM F1554 Grade 105 all-thread anchor rods with ASTM A563DH heavy hex coupling nuts, and ASTM A563DH heavy hex nuts. The deck anchorage in the compression region consisted of two 1-in. (25-mm) diameter, ASTM A449 anchor bolts with ¼-in. (6.4-mm) thick, 3-in. (76-mm) ASTM A36 steel square washers, and ASTM A563DH heavy hex coupling nuts. A $^{3}/_{16}$-in. (4.8-mm) thick, vertical embedment plate was used at every post location.

The three rail elements consisted on an ASTM A500 Grade C HSS 12-in. x 4-in. x ¼-in. (HSS 304.8-mm x 101.6-mm x 6.4-mm) section for the top rail and ASTM A500 Grade C HSS 8-in. x 6-in. x ¼-in. (HSS 203.2-mm x 152.4-mm x 6.4-mm) section for the lower two rails. Rail-to-rail connections were located 2 ft (610 mm) downstream from every other post location. The top rails were attached to the post assemblies with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long, ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom rails were attached to the front flanges of the posts with two staggered ¾-in. (19-mm) diameter by 7½-in. (191-mm) long ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

The splice tube for the top rails consisted of two horizontal PL 30 in. x 10⅝ in. x $^{5}/_{16}$ in. (PL 762 mm x 270 mm x 8 mm) and two vertical PL 30 in. x 2⅝ in. x ⅜ in. (PL 762 mm x 67 mm x 10 mm) attached with ¼-in. (6.4-mm) fillet welds. The splice tubes for the middle and bottom rails consisted on two vertical PL 30-in. x 6⅝-in. x ⅜-in. (PL 762-mm x 168-mm 10-mm) and two horizontal PL 30-in. x 4⅝-in. x $^{5}/_{16}$-in. (PL 762-mm x 117-mm x 8-mm) attached with ¼-in. (6.4-mm) fillet welds. The top splice tubes were attached to the top rail end sections with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long, ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom splice tubes were attached to the rail end sections with two ¾-in. (19-mm) diameter by 9½-in. (241-mm) long, ASTM A449 hex-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

After test no. STBR-3, post nos. 5, 6, 7, and 8, the nearest two railing elements for each of the three rails, and the three splice tubes connecting these rails were replaced for further MASH 2016 test designation no. 4-12 crash testing.

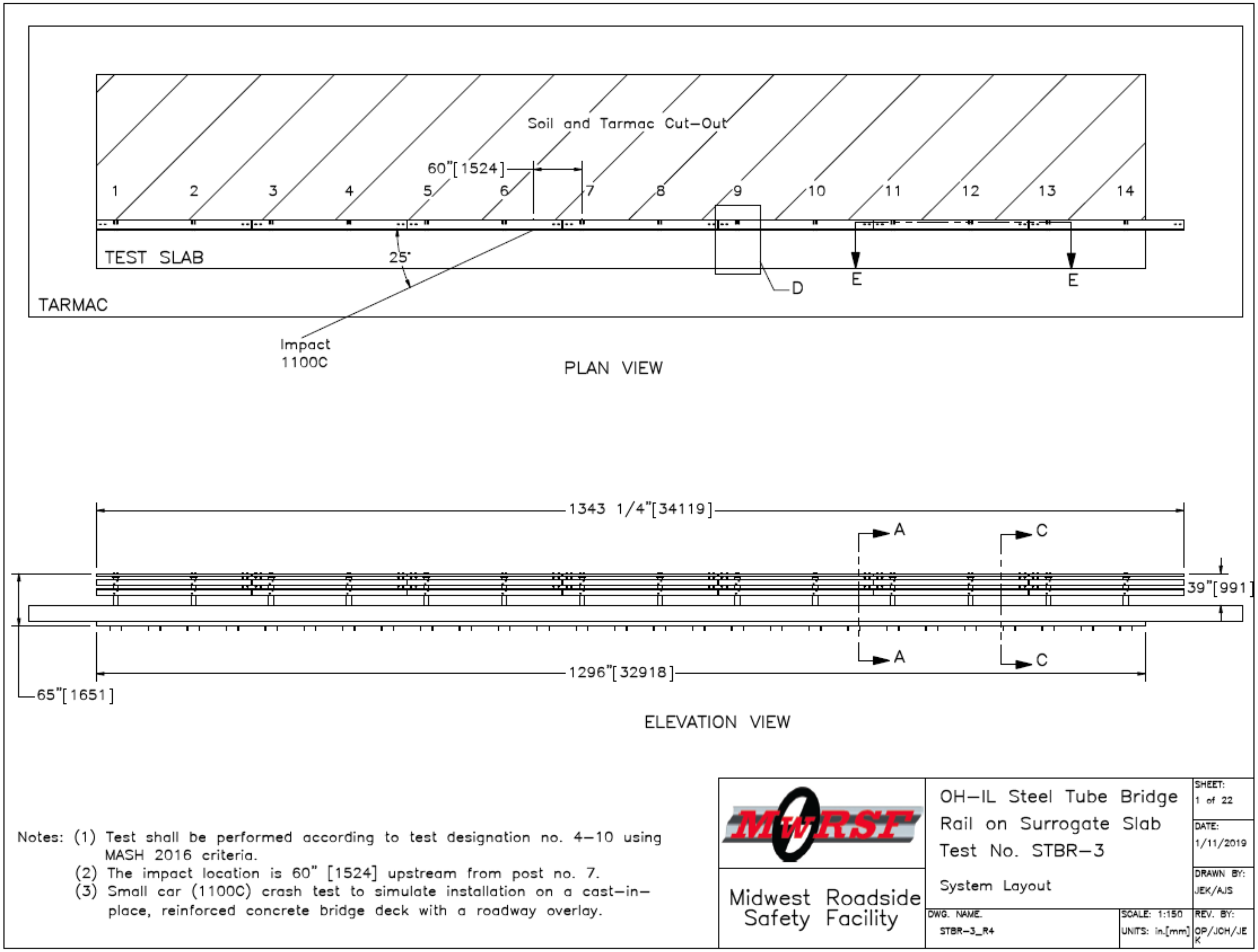


Figure 237. System Layout and Impact Location, Test No. STBR-3

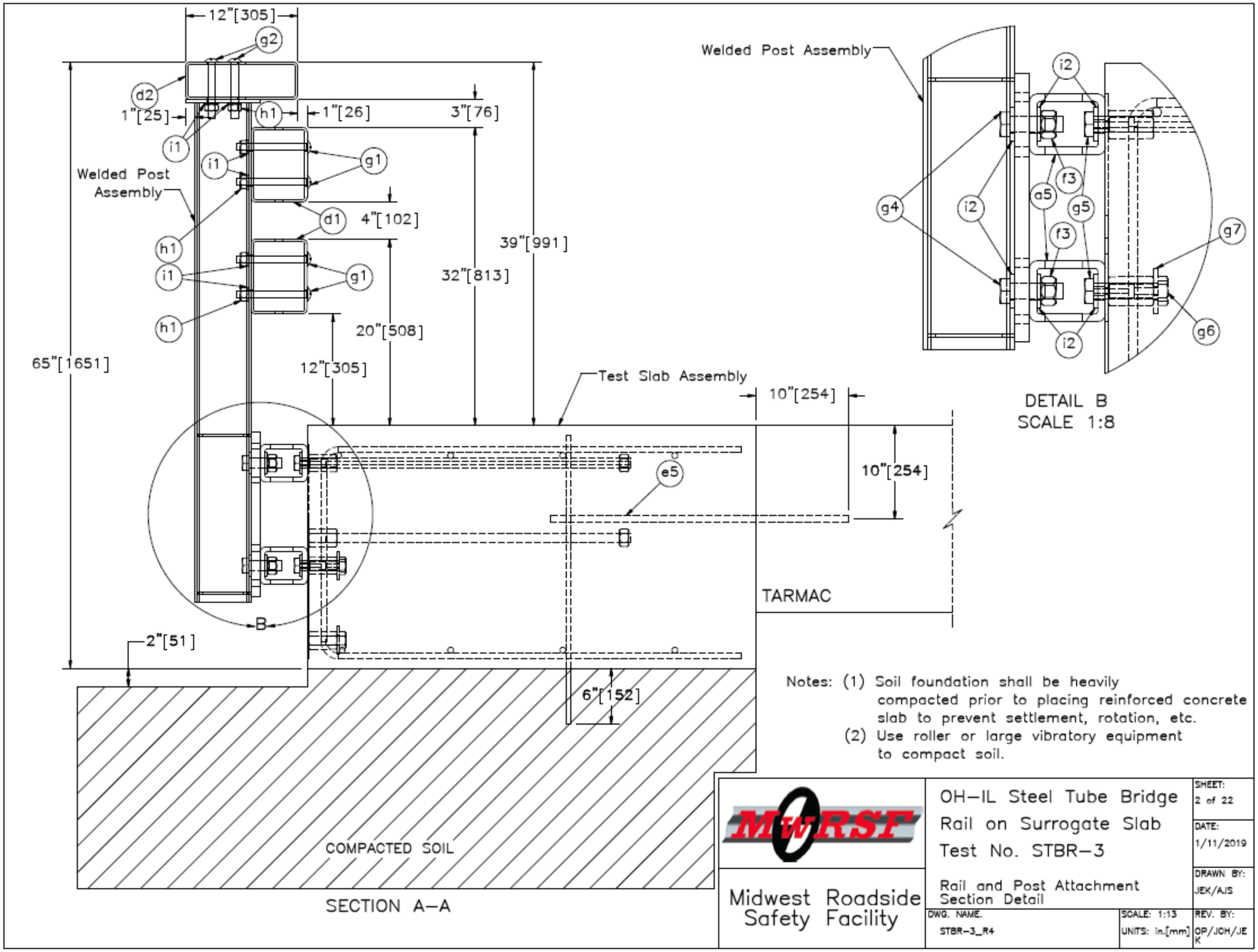


Figure 238. Rail and Post Attachment Section Detail, Test No. STBR-3

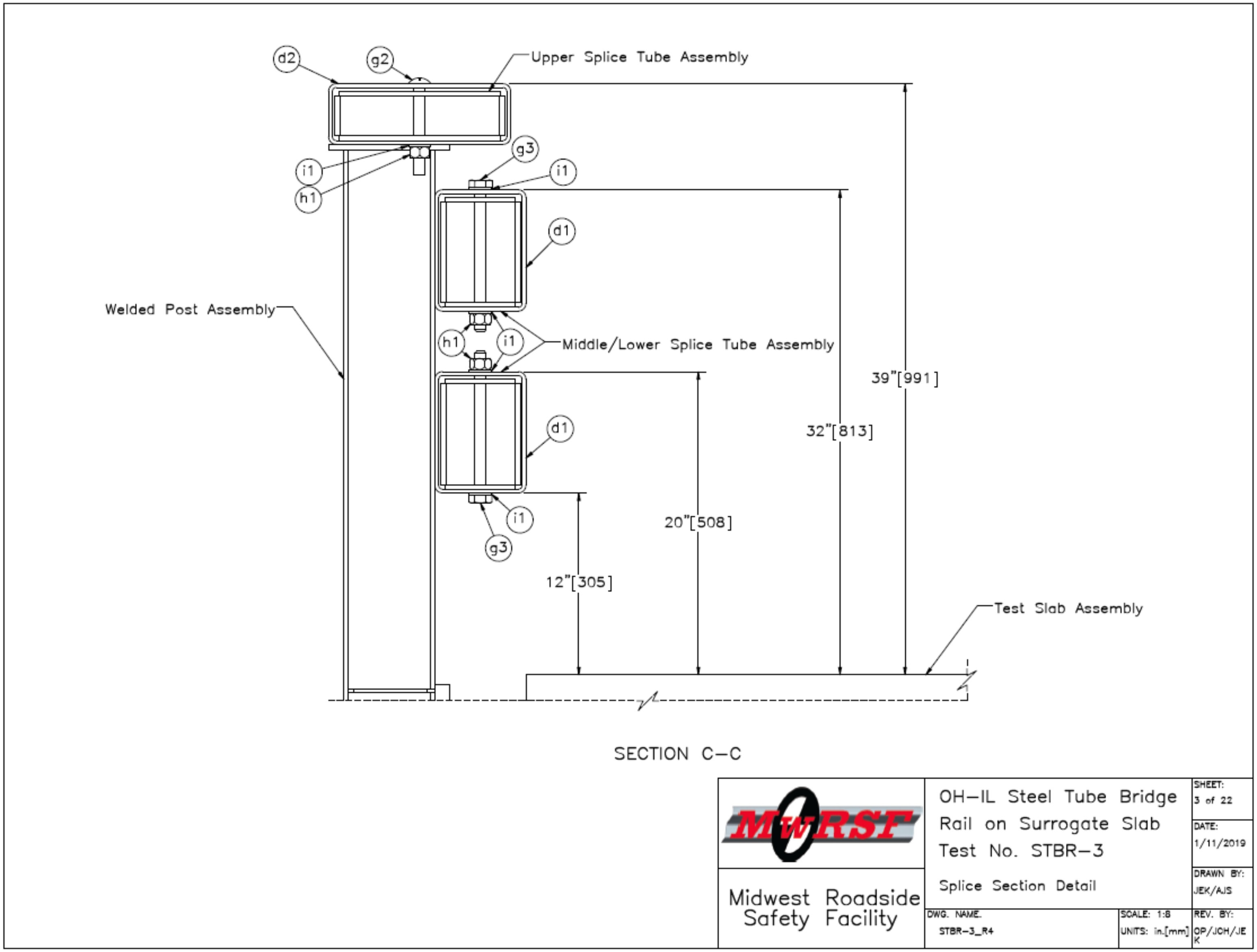


Figure 239. Splice Section Detail, Test No. STBR-3

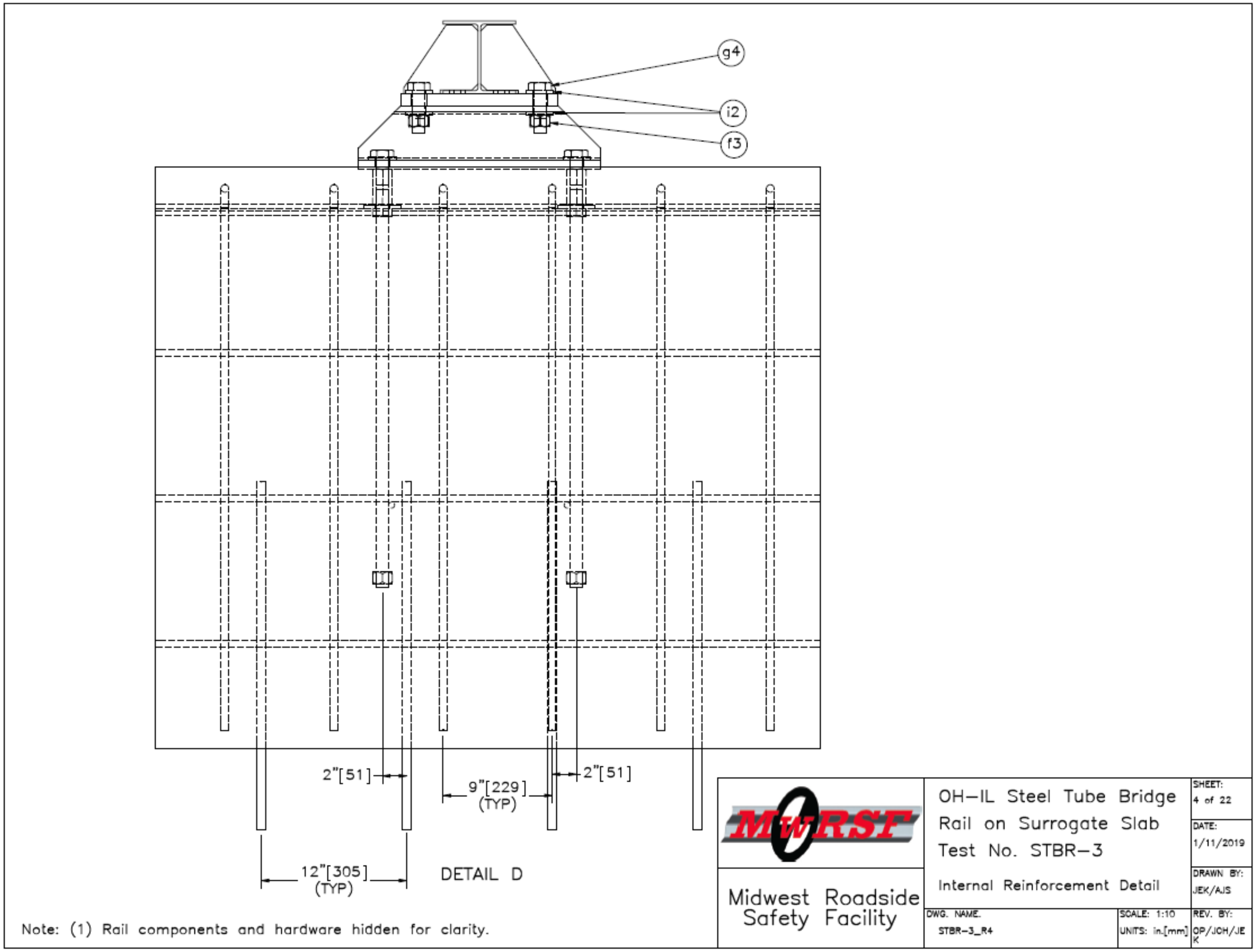


Figure 240. Lateral Reinforcement Detail, Test No. STBR-3

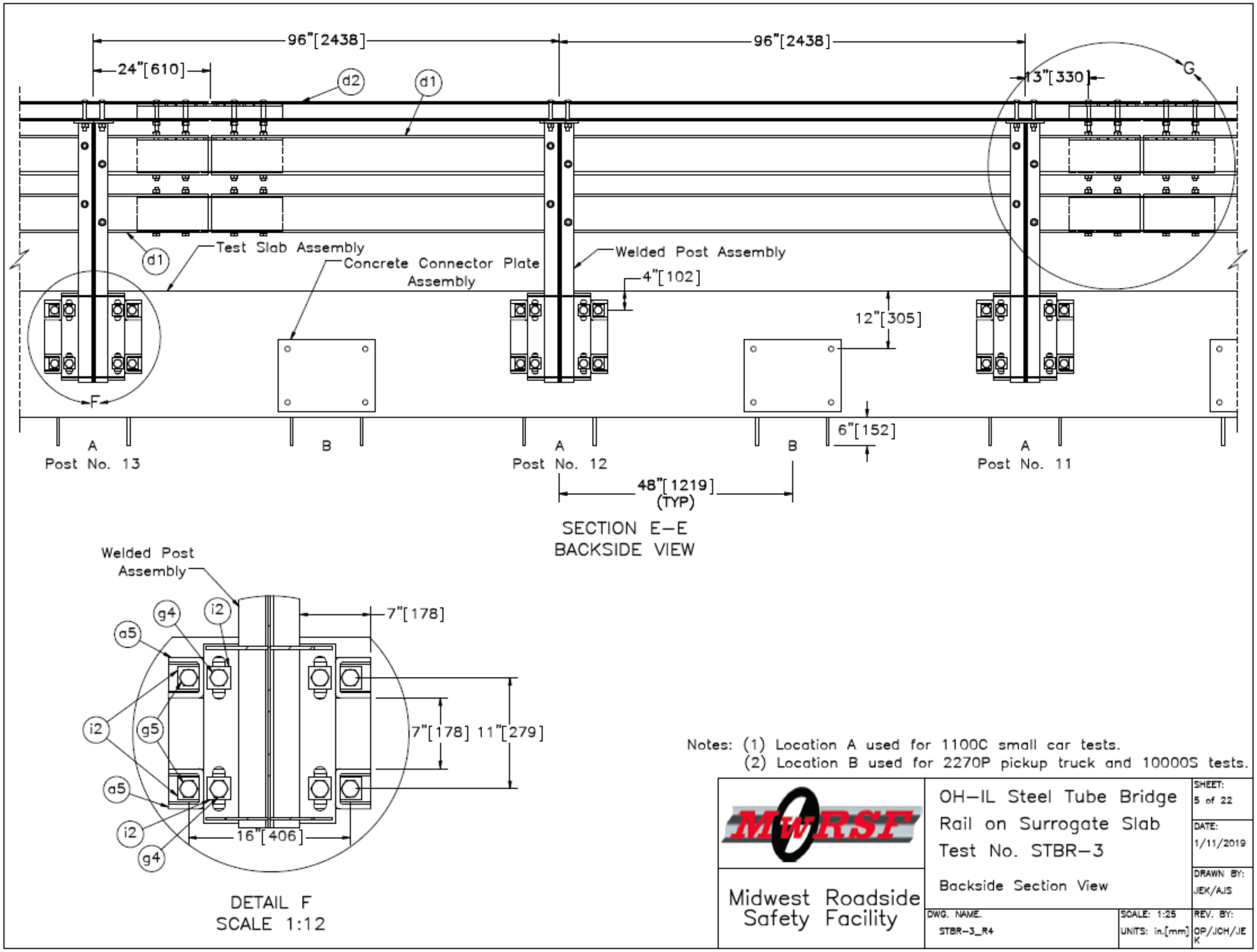


Figure 241. Backside Section View, Test No. STBR-3

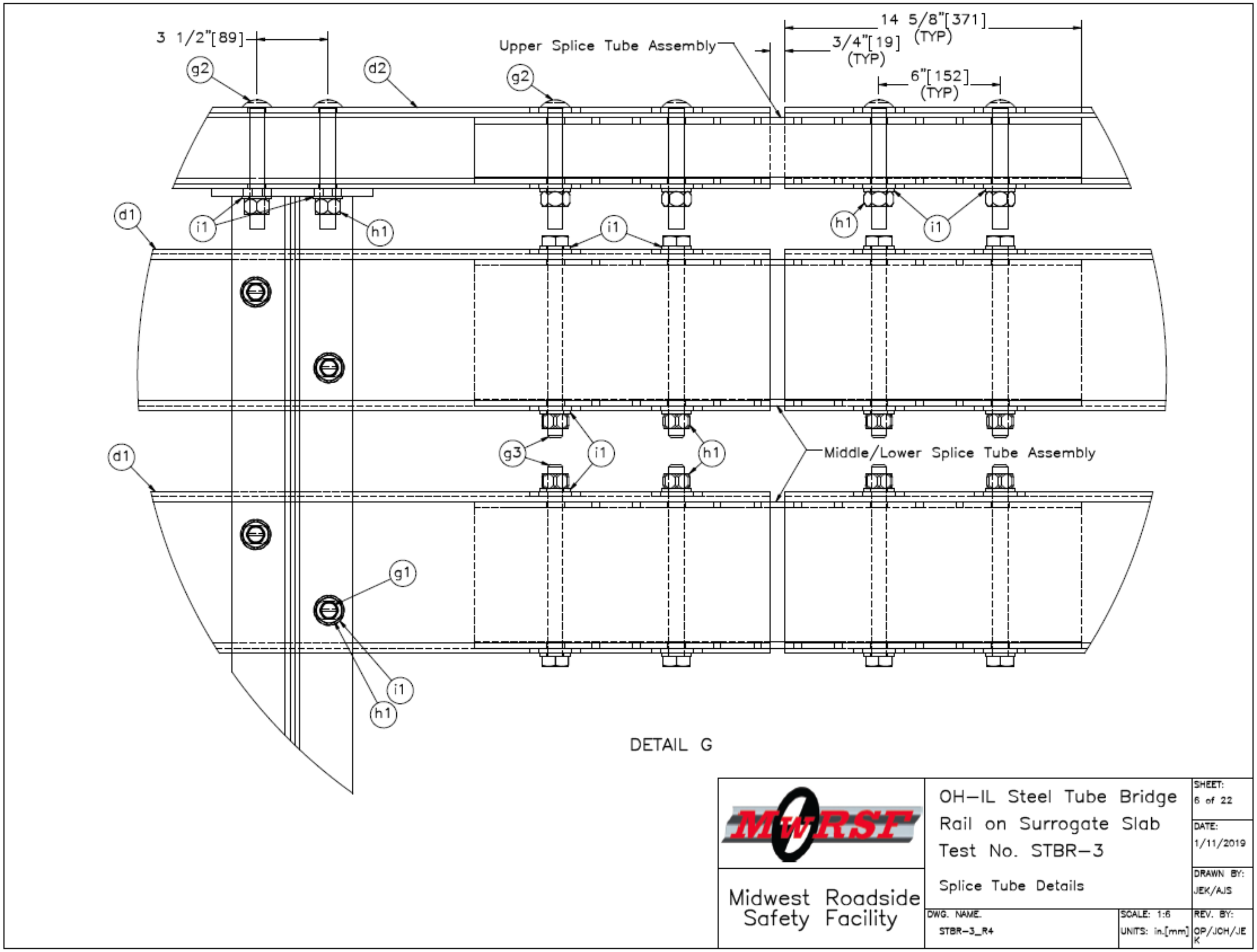
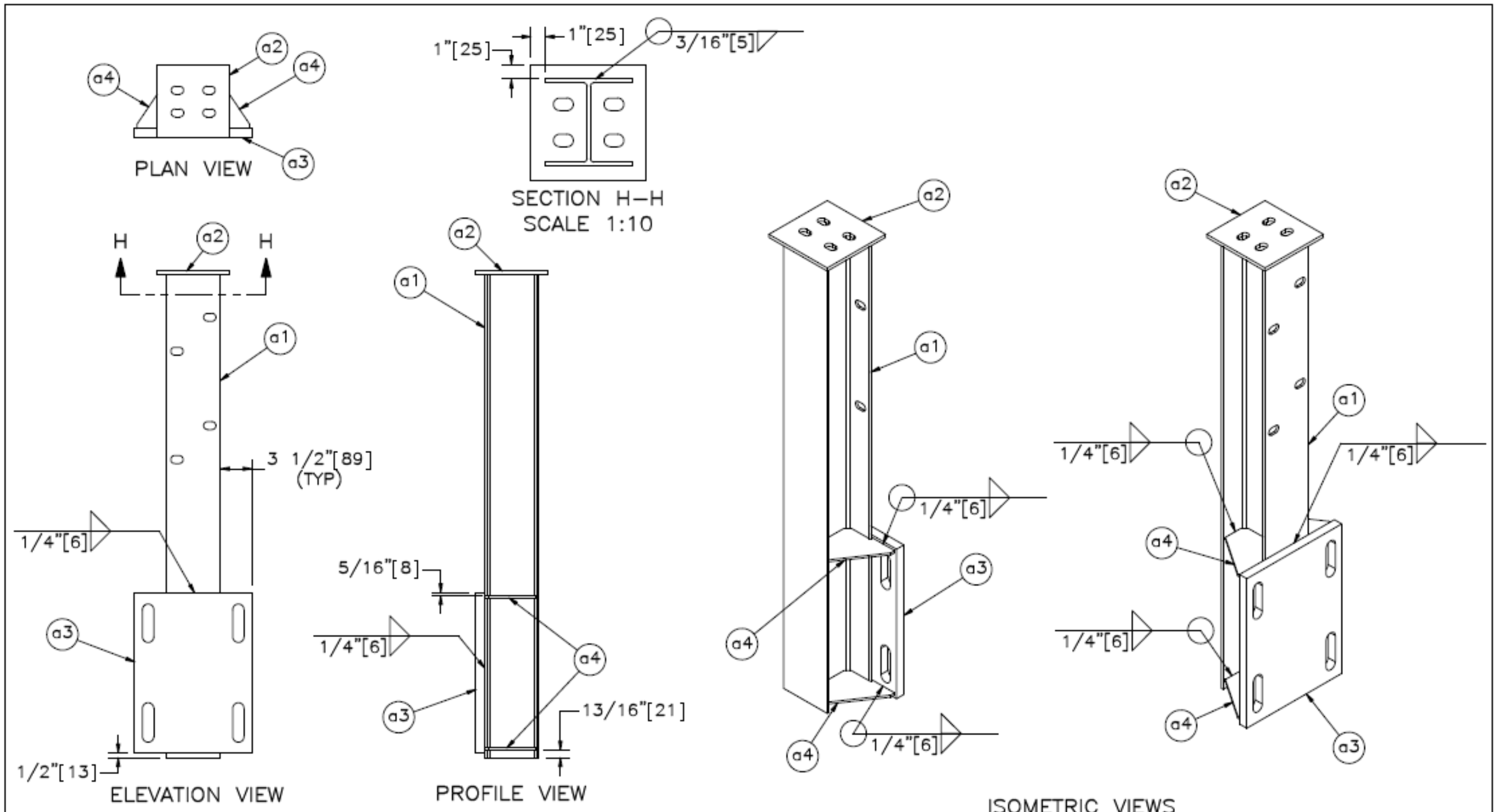


Figure 242. Splice Tube Section Details, Test No. STBR-3



Notes: (1) Gussets on both sides of post.
(2) To prevent welds to end at edges of gussets, leave 5/16" [8] of base material beyond weld to edge of welded part.
(3) Leave coped corners of gussets open for drainage. Do not fill with welds.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	14	Welded Post Assembly	–	ASTM A123
a1	1	W6x15 [W152x22], 53 1/2" [1,359] Long Post	ASTM A992	See Assembly
a2	1	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly
a3	1	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly
a4	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly

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OH–IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR–3

Welded Post Assembly Details

SHEET: 7 of 22
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DRAWN BY: JEK/AJS
DWG. NAME. STBR–3_R4
SCALE: 1:16
UNITS: in.[mm]
REV. BY: DP/JOH/JEK

Figure 243. Welded Post Assembly Details, Test No. STBR-3

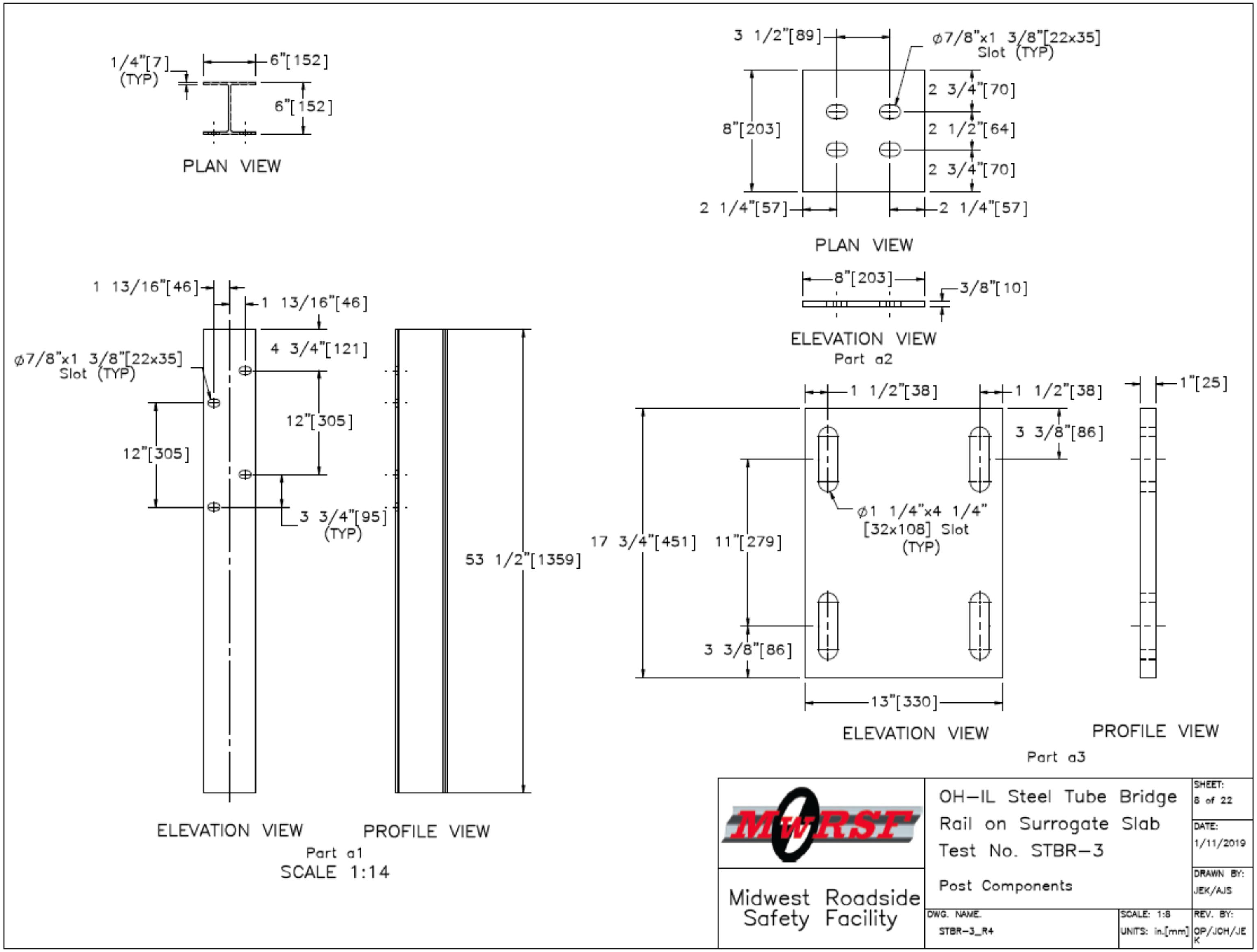


Figure 244. Post Components, Test No. STBR-3

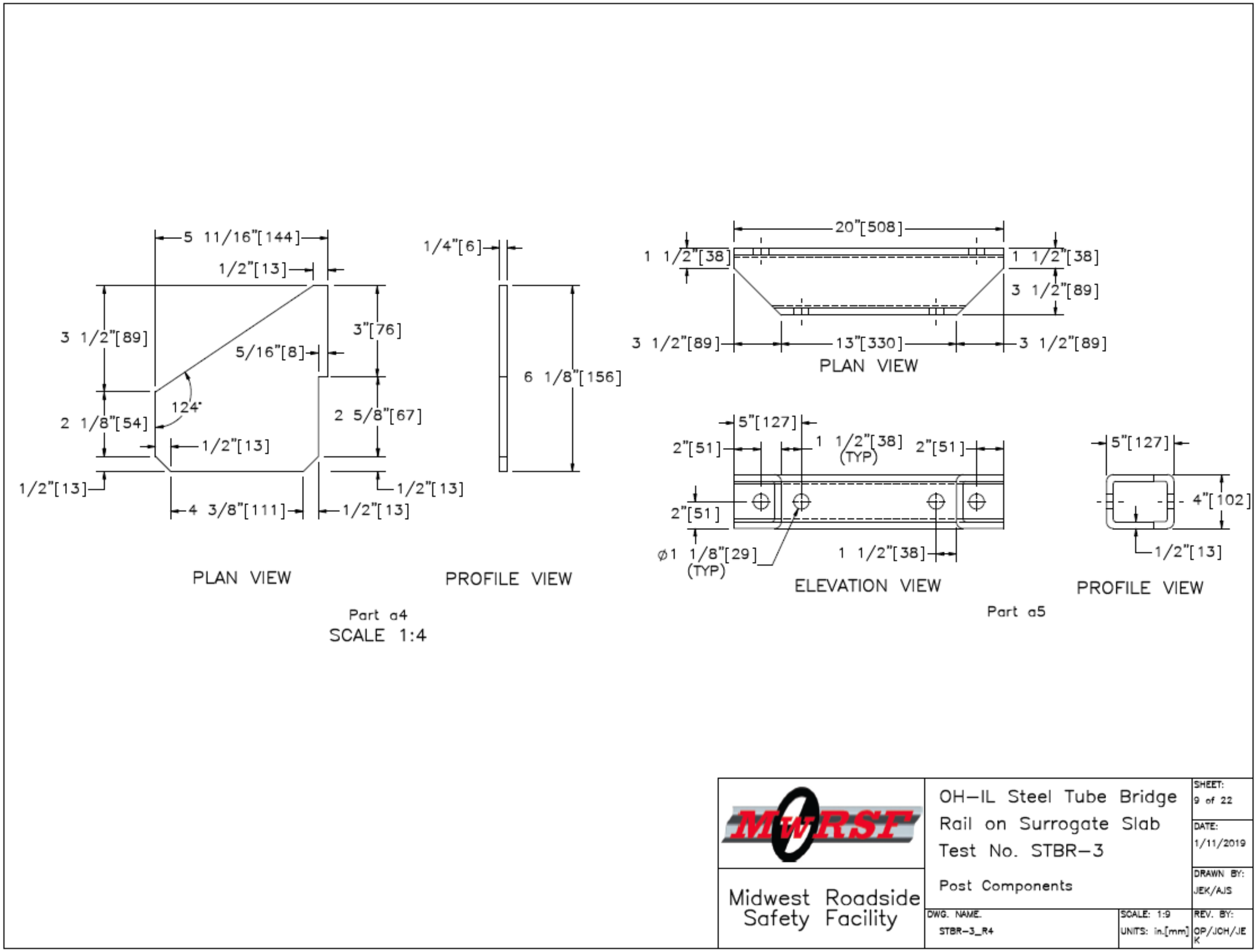
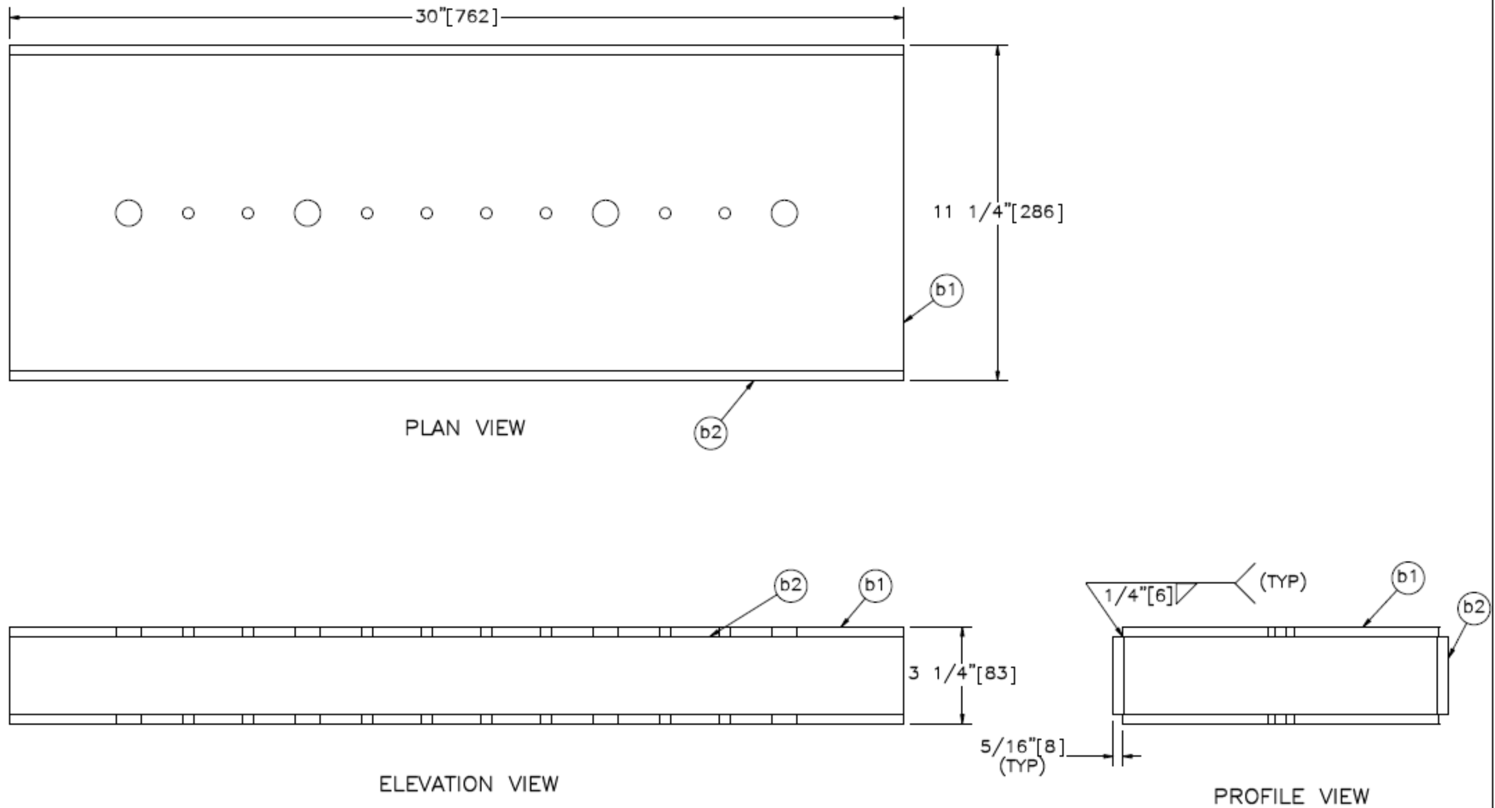


Figure 245. Post Components, Test No. STBR-3



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	6	Upper Splice Tube Assembly – Option 1	–	ASTM A123
b1	2	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly
b2	2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly

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OH-IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR-3

Upper Splice Tube Assembly

SHEET: 10 of 22
DATE: 1/11/2019
DRAWN BY: JEK/AJS
DWG. NAME. STBR-3_R4
SCALE: 1:5
UNITS: in.[mm]
REV. BY: OP/JOH/JEK

Figure 246. Upper Splice Tube Assembly, Test No. STBR-3

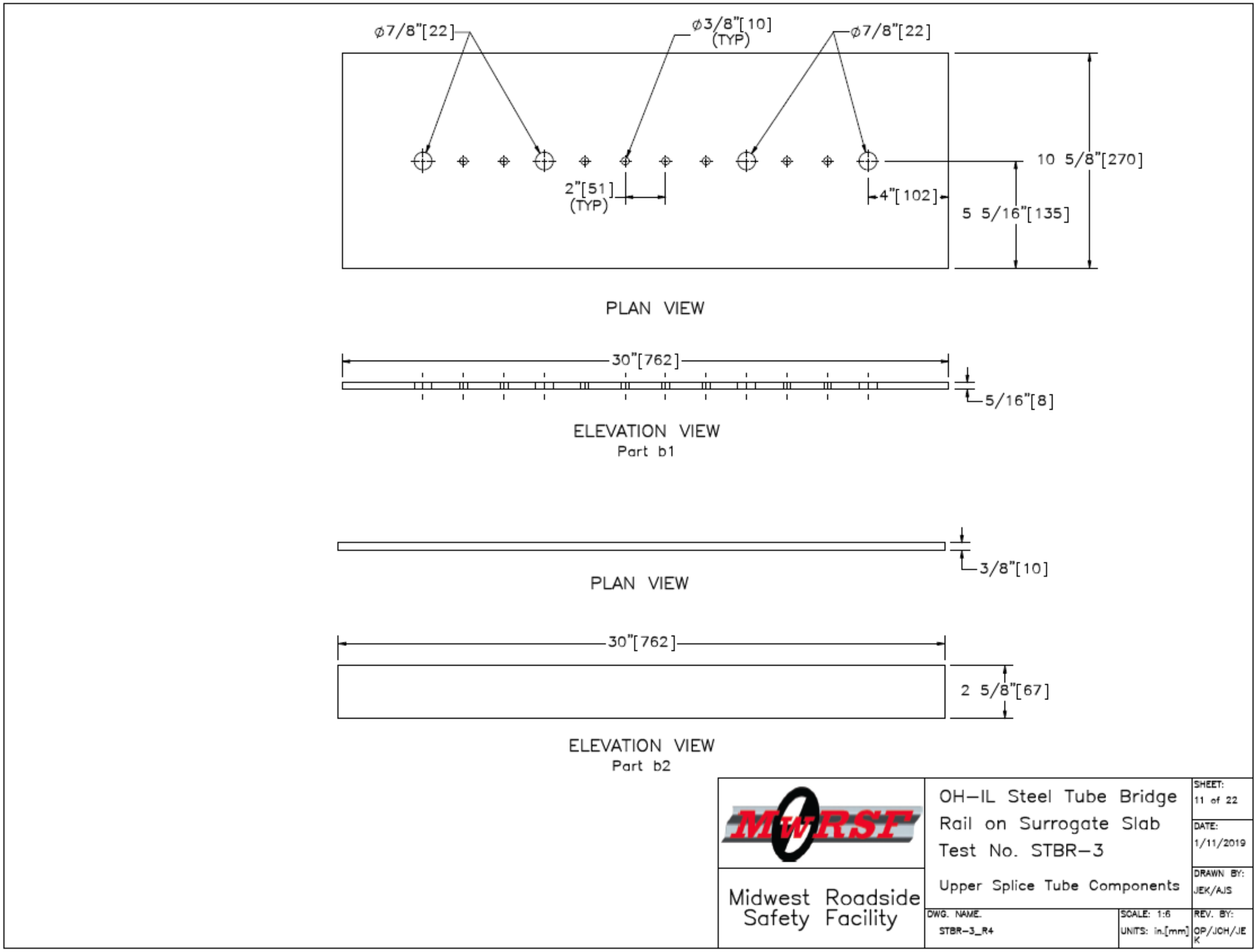
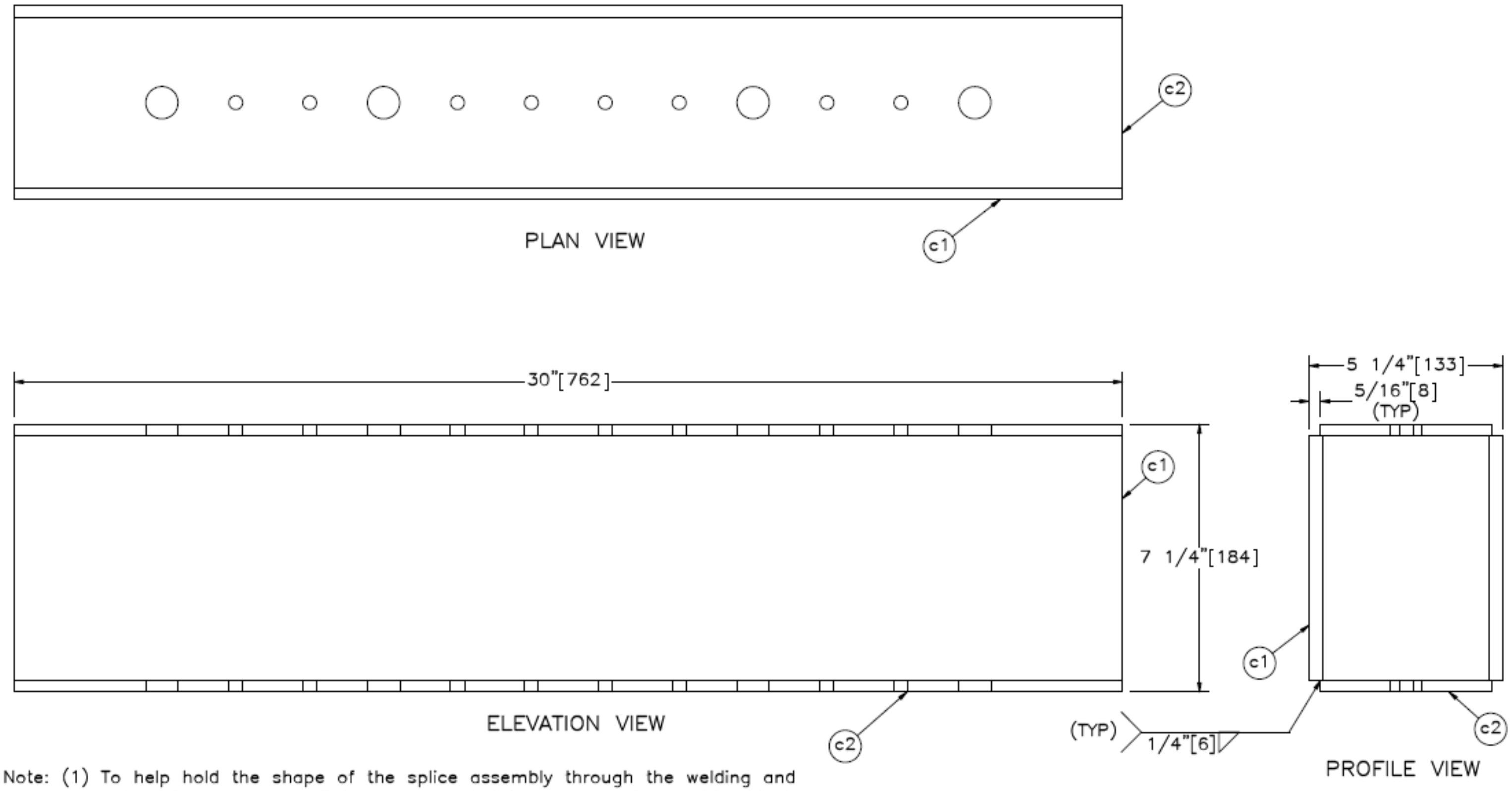


Figure 247. Upper Splice Tube Components, Test No. STBR-3



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	12	Middle/Lower Splice Tube Assembly – Option 1	–	ASTM A123
c1	2	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly
c2	2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–3

Middle/Lower Splice Tube Assembly

SHEET: 12 of 22
DATE: 1/11/2019
DRAWN BY: JEK/AJS
DWG. NAME. STBR–3_R4
SCALE: 1:4
UNITS: in.[mm]
REV. BY: OP/JCH/JEK

Figure 248. Middle/Lower Splice Tube Assembly, Test No. STBR-3

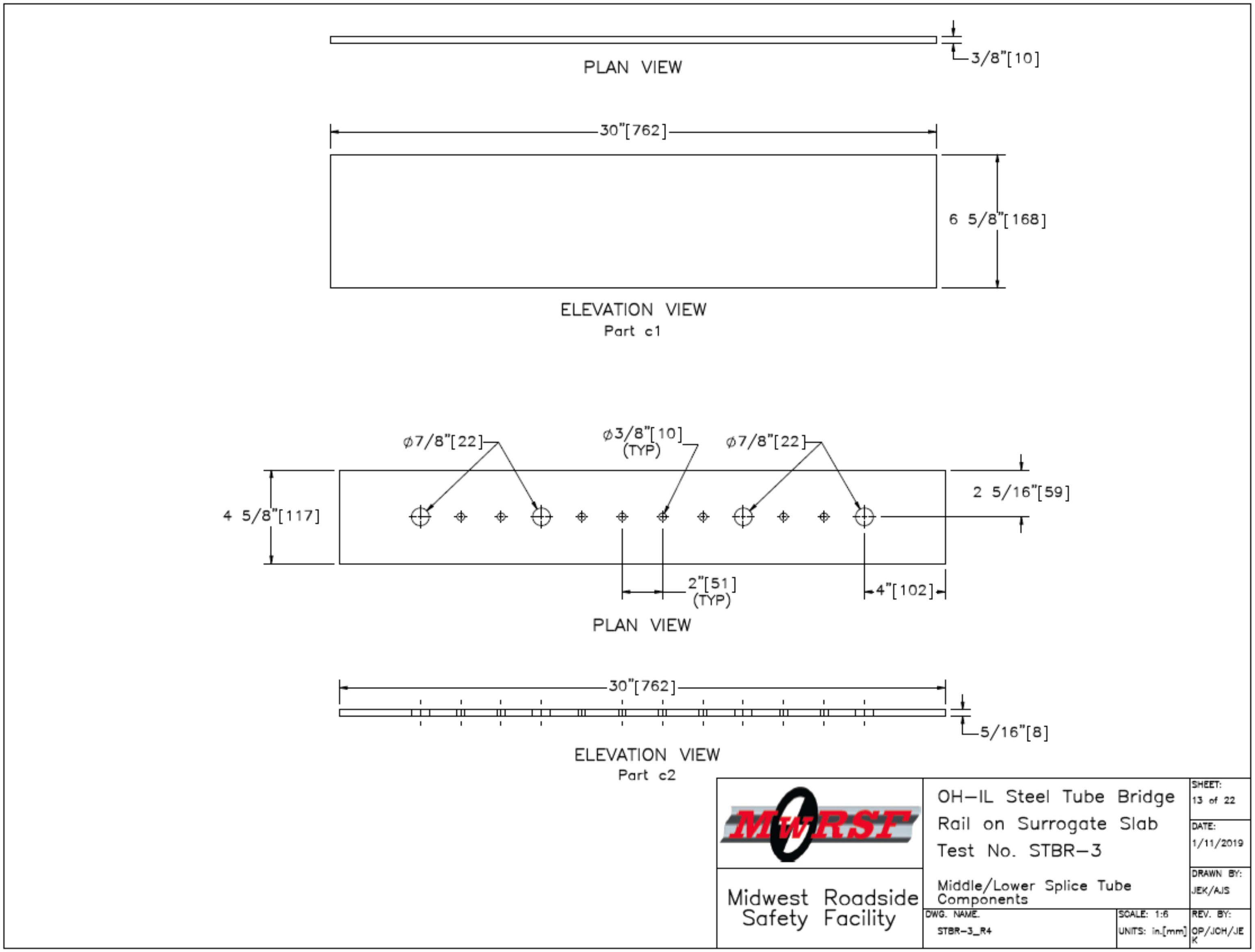


Figure 249. Middle/Lower Splice Tube Components, Test No. STBR-3

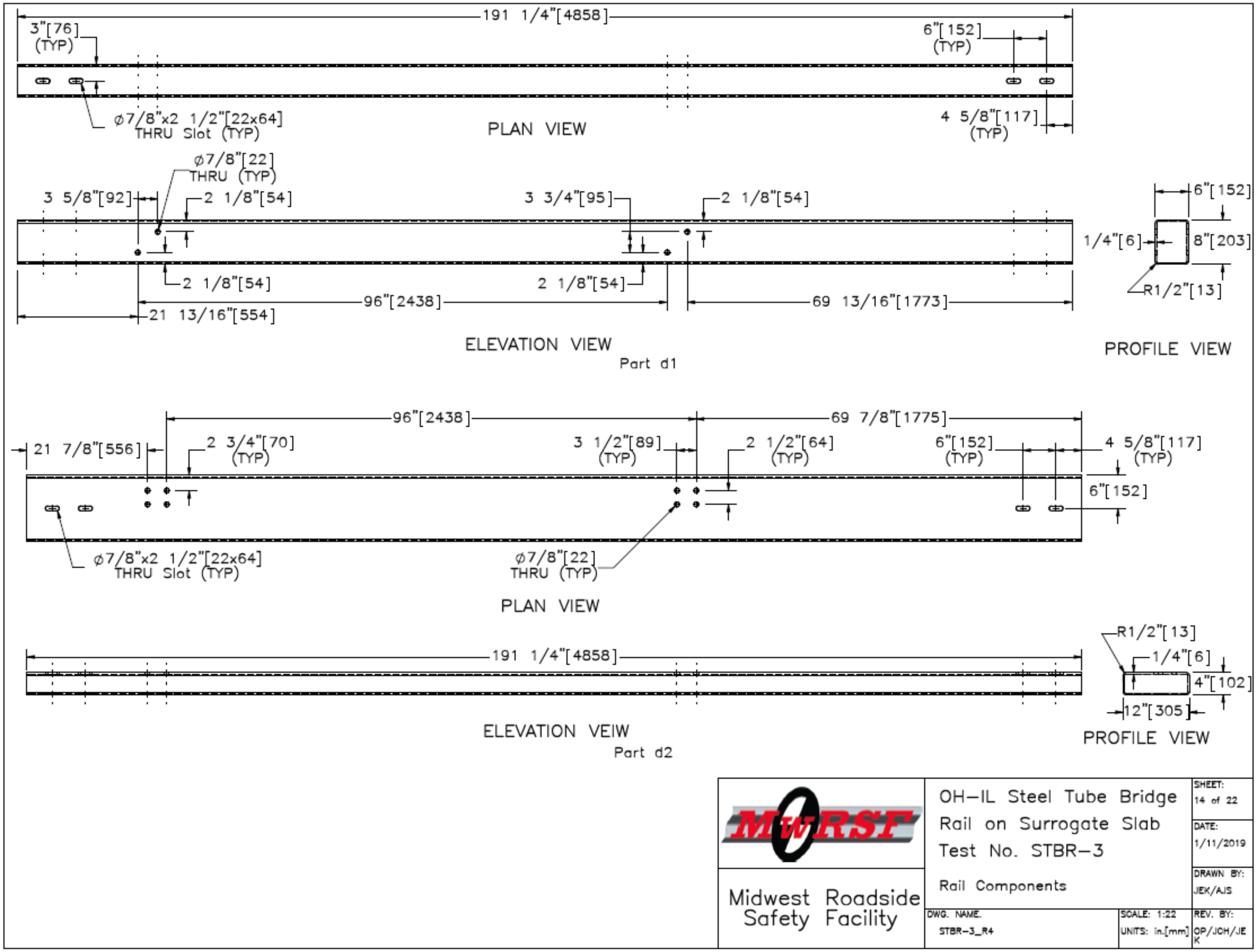


Figure 250. Rail Components, Test No. STBR-3

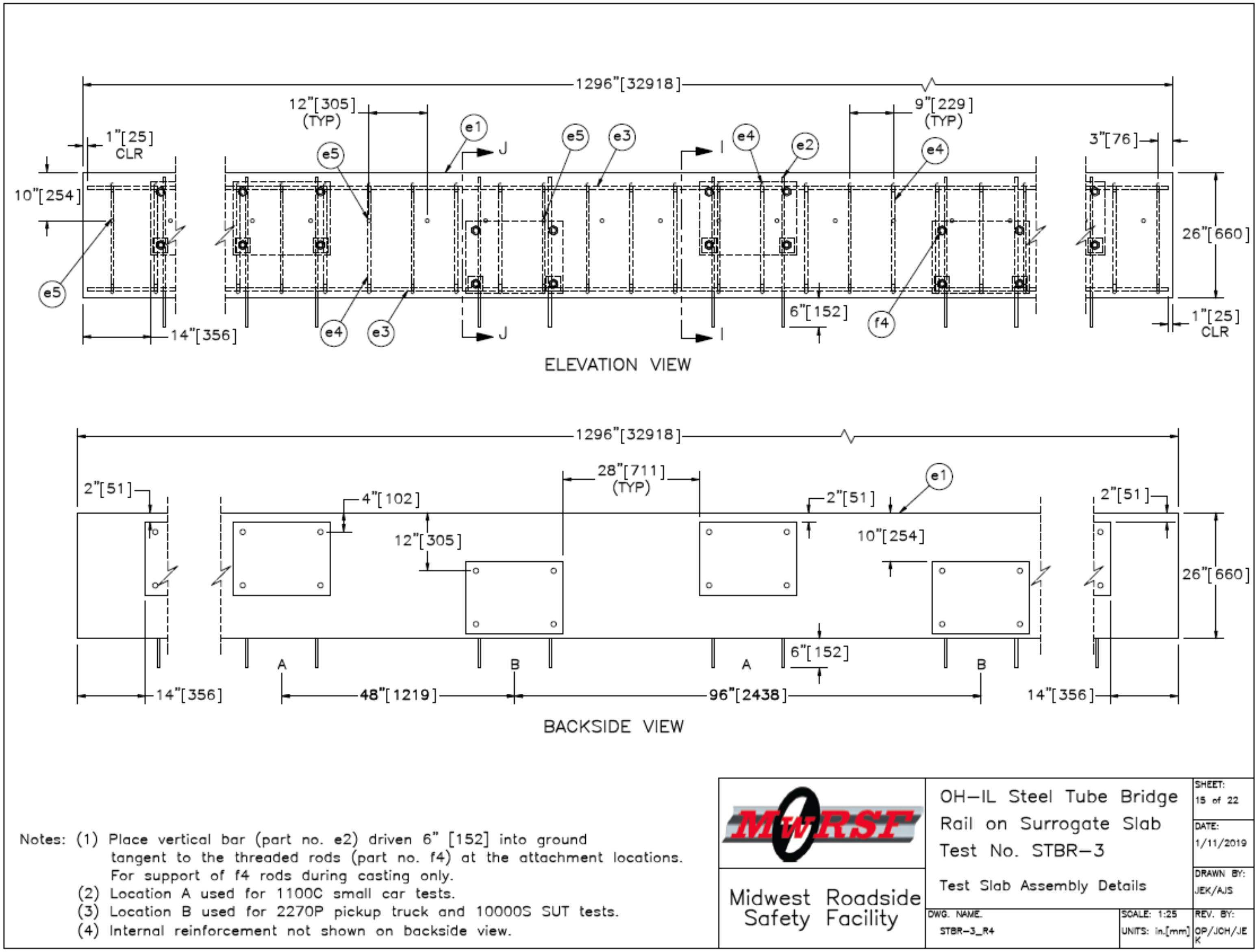
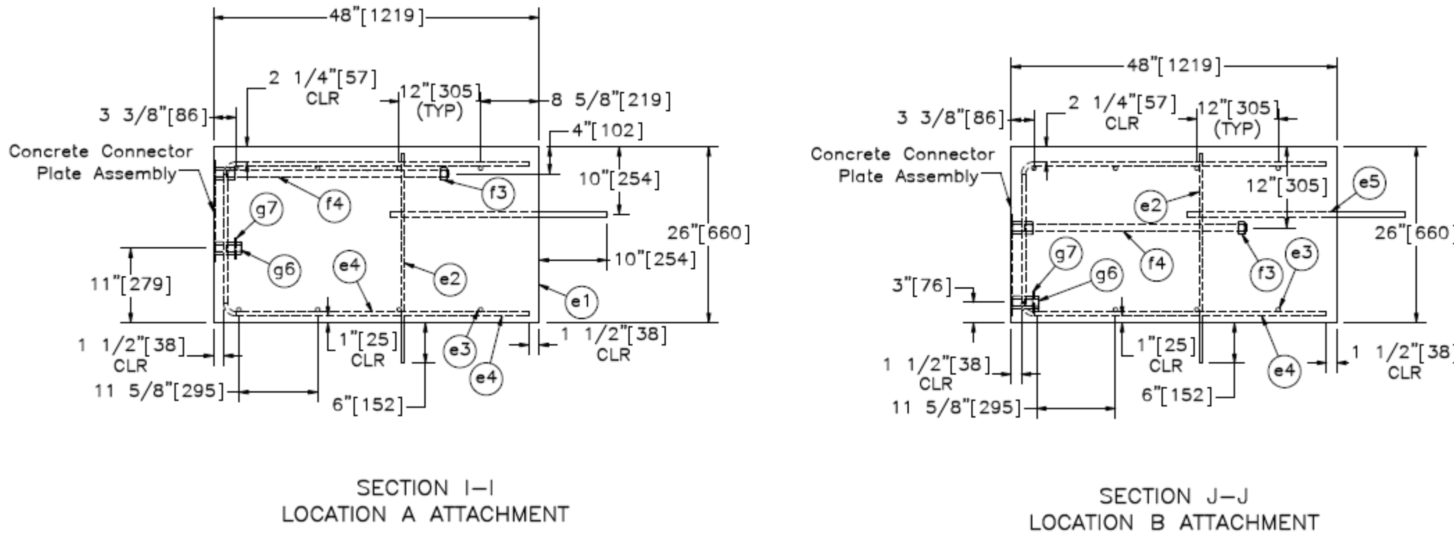


Figure 251. Test Slab Assembly Details, Test No. STBR-3

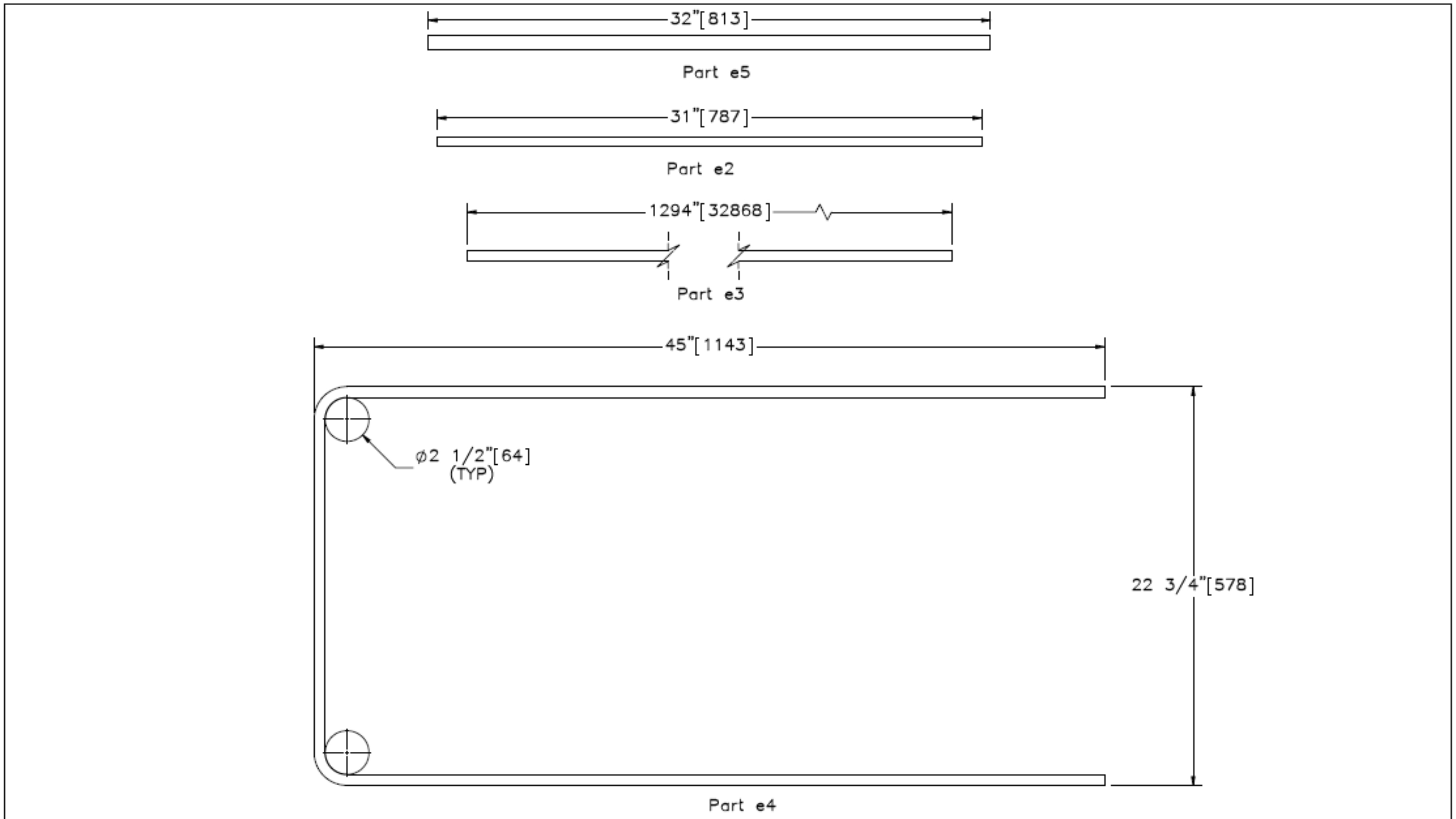


Item No.	QTY.	Description	Material Specification	Treatment Specification
–	1	Test Slab Assembly	–	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
–	27	Concrete Connector Plate Assembly	–	ASTM A123
f3	54	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
g6	54	1"–8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329
g7	54	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123

Notes: (1) Location A used for 1100C small car tests.
(2) Location B used for 2270P pickup truck and 10000 SUT tests.
(3) Place vertical bar (part no. e2) driven 6" [152] into ground tangent to the threaded rods (part no. f4) at the attachment locations. For support during casting.
(4) Part f4 is threaded 1 1/4" [32] into part f2 (coupling nut) on Concrete Connector Plate Assembly.

MwRSF Midwest Roadside Safety Facility	OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–3 — Test Slab Assembly Details	SHEET: 16 of 22; DATE: 1/11/2019; DRAWN BY: JEK/AJS
DWG. NAME. STBR–3_R4	SCALE: 1:22 UNITS: in.[mm]	REV. BY: OP/JCH/JEK

Figure 252. Test Slab Assembly Details, Test No. STBR-3



General Table					
Bill of Bars					
Bar	QTY.	Size	Total Unbent Length	Material Specification	Treatment Specification
e2	54	#5	31" [787]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e3	8	#5	1294" [32868]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e4	144	#5	110 3/16" [2799]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e5	108	#6	32" [813]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)

Note: (1) Minimum lap length for a #5 bar is 18" [457].

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–3

Reinforcement Details

SHEET: 17 of 22
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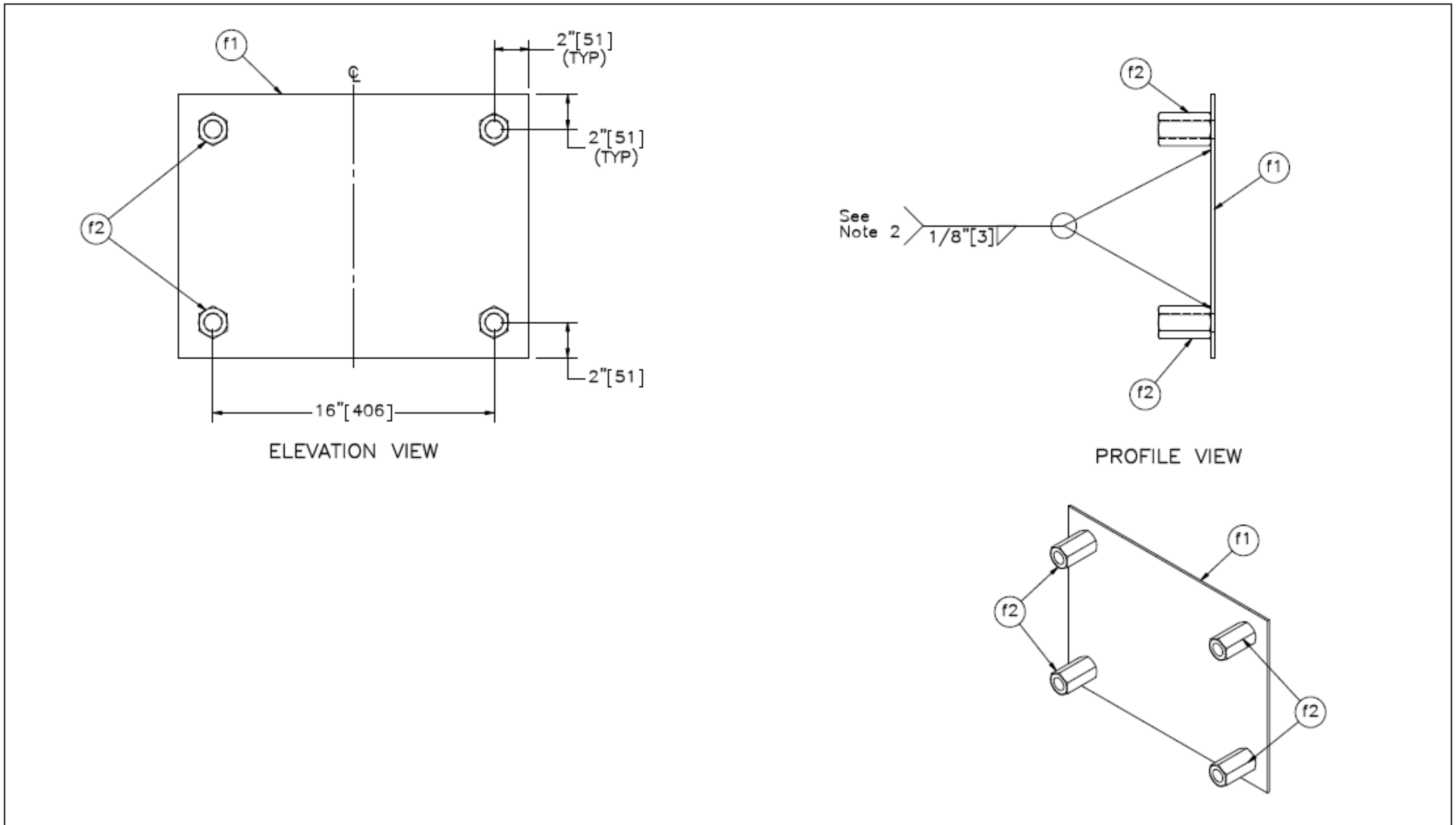
DRAWN BY: JEK/AJS
DWG. NAME. STBR–3_R4
SCALE: 1:8
UNITS: in.[mm]
REV. BY: OP/JCH/JEK

Figure 253. Reinforcement Details, Test No. STBR-3



Notes: (1) Connector plate assembly is symmetric along the centerline.
(2) During casting, the nuts may be held in place by extending bolts through the formwork into the nuts, or by tack welding nuts to inside face of the plate. The welds are not structurally important.

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
–	27	Concrete Connector Plate Assembly	–	ASTM A123	–
f1	1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	See Assembly	–
f2	4	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–3

Concrete Connector Plate Assembly Details

SHEET: 18 of 22
DATE: 1/11/2019
DRAWN BY: JEK/AJS
DWG. NAME. STBR–3_R4
SCALE: 1:10
UNITS: in.[mm]
REV. BY: OP/JCH/JEK

Figure 254. Concrete Connector Plate Assembly Details, Test No. STBR-3

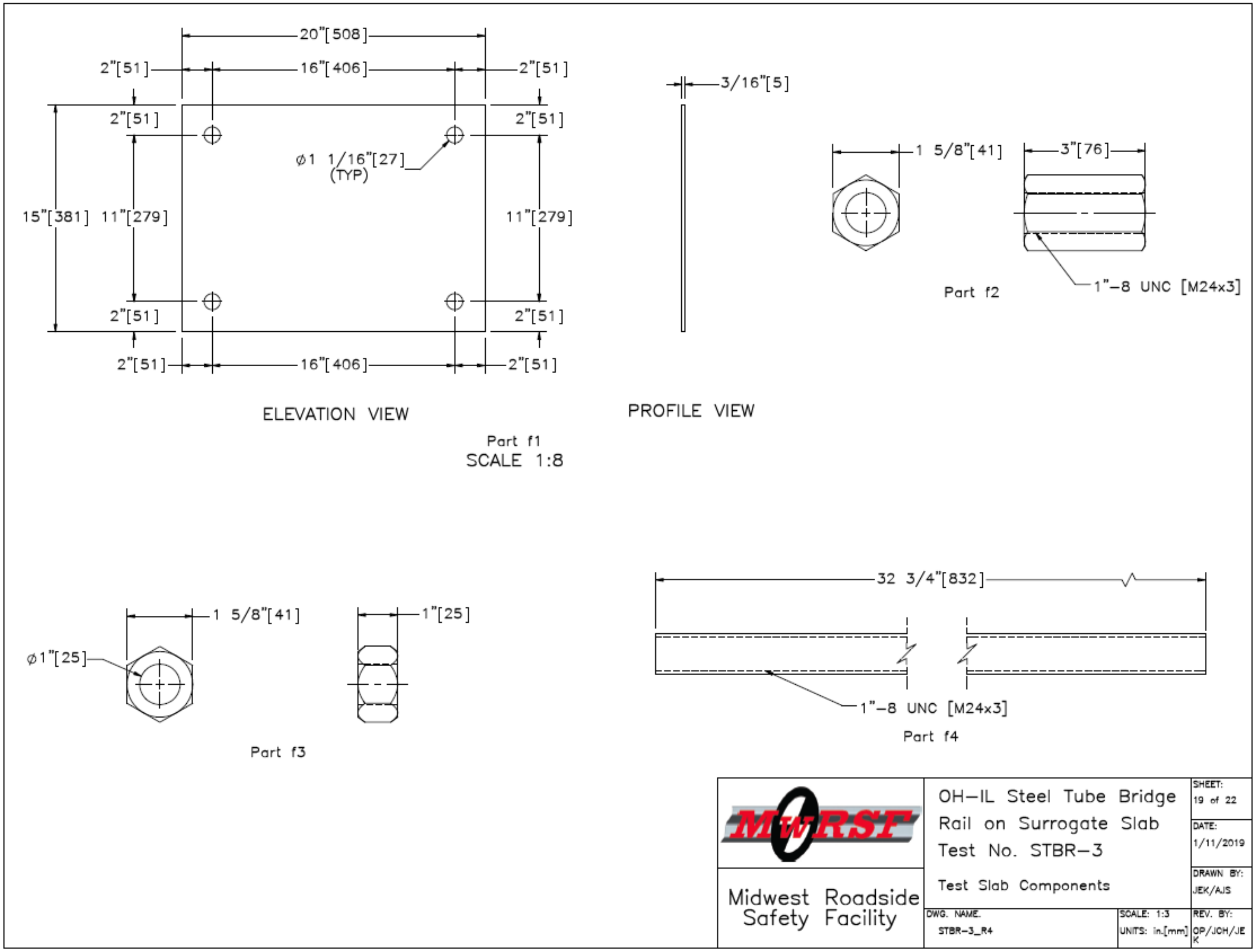

Figure 255. Test Slab Components, Test No. STBR-3

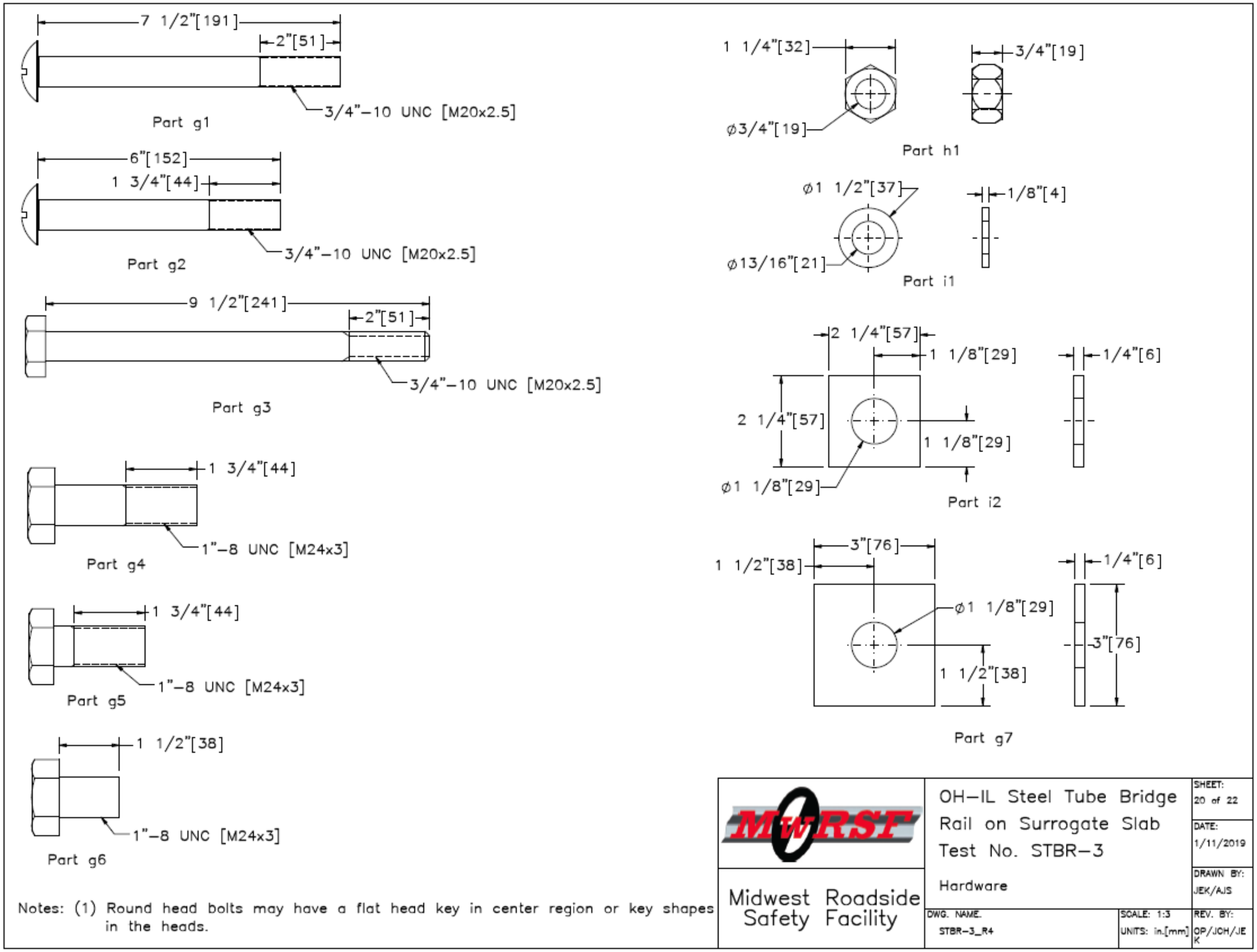

Figure 256. Hardware, Test No. STBR-3

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
a1	14	W6x15 [W152x22], 53 1/2" [1,359] Long Post	ASTM A992	See Assembly	–
a2	14	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly	–
a3	14	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly	–
a4	56	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly	–
a5	28	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	ASTM A123	–
b1	12	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly	–
b2	12	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c1	24	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c2	24	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly	–
d1	14	HSS 8"x6"x1/4" [203x152x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
d2	7	HSS 12"x4"x1/4" [305x102x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
f1	27	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	See Assembly	–
f2	108	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–
f3	110	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX24b
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329	FRR24b

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–3

Bill of Materials

SHEET: 21 of 22
DATE: 1/11/2019
DRAWN BY: JEK/AJS
DWG. NAME. STBR–3_R4
SCALE: 1:384 UNITS: in.[mm]
REV. BY: OP/JCH/JEK

Figure 257. Bill of Materials, Test No. STBR-3

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
g1	56	3/4"-10 UNC [M20x2.5], 7 1/2" [191] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g2	80	3/4"-10 UNC [M20x2.5], 6" [152] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g3	48	3/4"-10 UNC [M20x2.5], 9 1/2" [241] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F2329	FBX20b
g4	56	1"-8 UNC [M24x3], 3 1/2" [89] 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
g5	56	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
g6	54	1"-8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX24b
g7	54	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123	–
h1	184	3/4 -10 UNC [M20x2.5] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX20b
i1	232	3/4" [19] Dia. Hardened Flat Washer	ASTM F436	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329	FWC20b
i2	168	2 1/4"x2 1/4"x1/4" [57x57x6] Square Washer	ASTM A36	ASTM A123	–

Midwest Roadside Safety Facility

OH-IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR-3

Bill of Materials

SHEET: 22 of 22
DATE: 1/11/2019
DRAWN BY: JEK/AJS
DWG. NAME. STBR-3_R4
SCALE: 1:384
UNITS: in.[mm]
REV. BY: OP/JOH/JEK

Figure 258. Bill of Materials, Test No. STBR-3

Figure 259. Test Installation Photographs, Test No. STBR-3

Figure 260. Test Installation Photograph, Side-Mounted Posts and Post-to-Deck Connections, Test No. STBR-3

Figure 261. Bridge Railing End Views, Test No. STBR-3

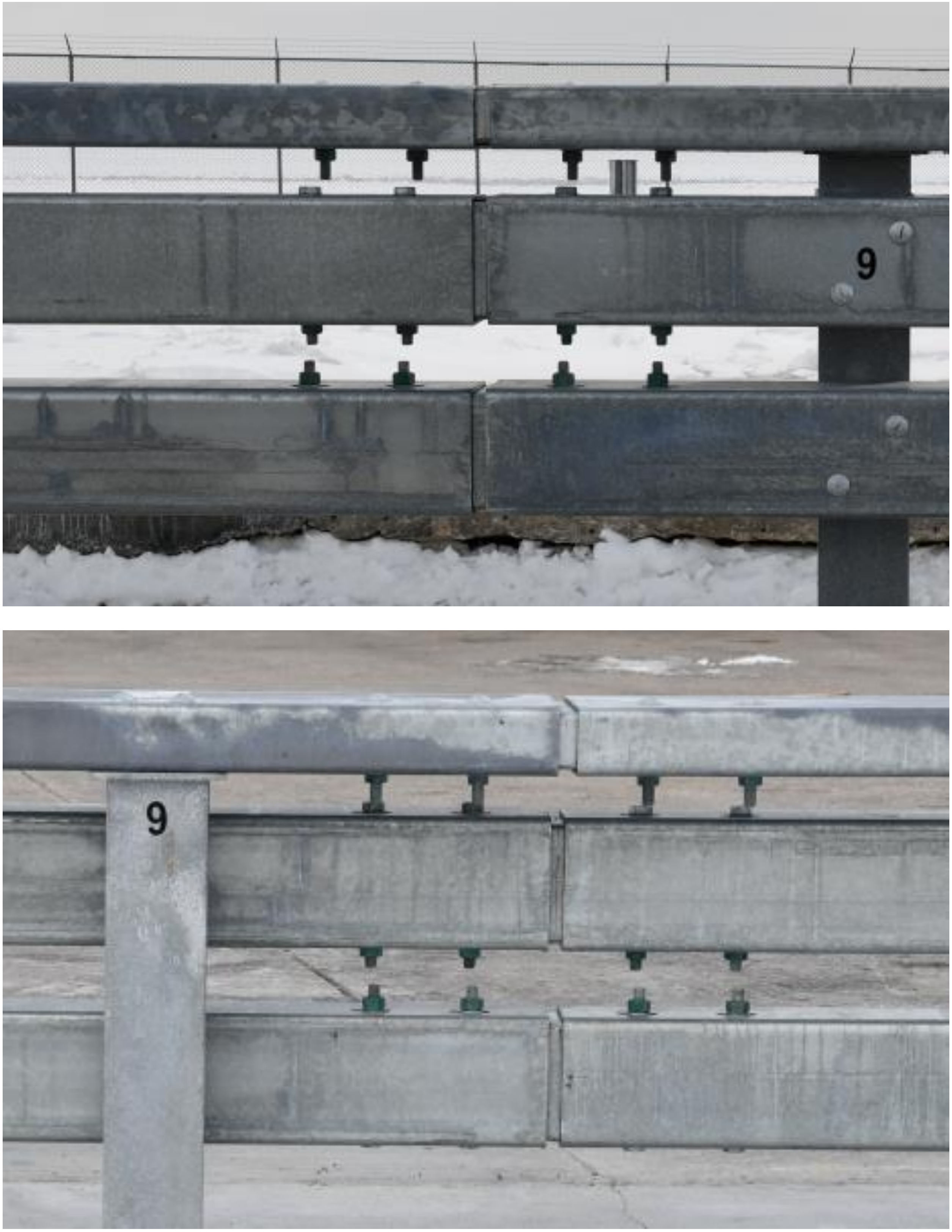


Figure 262. Splice Tubes, Test No. STBR-3

16 FULL-SCALE CRASH TEST NO. STBR-3

16.1 Weather Conditions

Test no. STBR-3 was conducted on March 1, 2019 at approximately 2:00 p.m. The weather conditions as per the National Oceanic and Atmospheric Administration (station 14939/LNK) were reported and are shown in Table 77.

Table 77. Weather Conditions, Test No. STBR-3

Temperature	18° F
Humidity	41%
Wind Speed	4 mph
Wind Direction	90° from True North
Sky Conditions	Sunny
Visibility	10 Statute Miles
Pavement Surface	Dry
Previous 3-Day Precipitation	2.26 in.
Previous 7-Day Precipitation	2.26 in.

16.2 Test Description

Initial vehicle impact was to occur 60 in. (1.5 m) upstream from post no. 7, as shown in Figure 263, which was selected as discussed in Chapter 9. In test no. STBR-3, the 2,569-lb (1,165-kg) Kia Rio small car impacted the bridge rail at a speed of 62.0 mph (99.8 km/h) and at an angle of 24.8 degrees. The actual point of impact was 61.3 in. (1.6 m) upstream from post no. 7. The vehicle came to rest 198 ft – 2 in. (60.4 m) downstream from the original impact point and laterally 34 ft – 2 in. (10.4 m) in front of the bridge rail.

The crash testing parameters are shown in Table 78. A detailed description of the sequential impact events is contained in Table 79. Sequential photographs are shown in Figures 264 and 265. Documentary photographs of the crash test are shown in Figure 266. The vehicle trajectory and final position are shown in Figure 267.

Figure 263. Impact Location, Test No. STBR-3

Table 78. Actual, Lower-Bound, and Target Crash Test Parameters, Test No. STBR-3

Test Parameter	Actual	Lower-Bound	Target
Speed	62.0 mph	59.5 mph	62.0 mph
Angle	24.8 deg.	23.5 deg.	25.0 deg.
Impact Severity	54.5 kip-ft	51.0 kip-ft	55.9 kip-ft

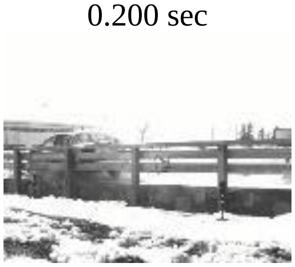
Table 79. Sequential Description of Impact Events, Test No. STBR-3

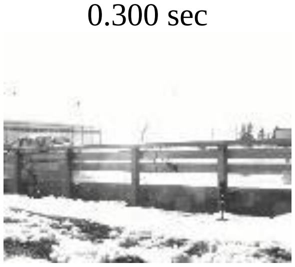
Time (sec):	**Event Description**
0.000	Vehicle's front bumper contacted rail between post nos. 6 and 7.
0.002	Vehicle's front bumper deformed.
0.006	Vehicle's left fender contacted rail, Vehicle's left headlight contacted rail.
0.008	Vehicle's left fender deformed. Vehicle's left-front tire contacted rail.
0.012	Vehicle yawed away from system.
0.016	Post no. 7 deflected backward.
0.018	Vehicle's hood contacted rail.
0.020	Vehicle's hood deformed.
0.024	Post no. 6 deflected backward.
0.028	Vehicle's left mirror contacted rail. Vehicle rolled toward system.
0.030	Vehicle's left mirror deformed.
0.034	Post no. 8 deflected backward. Vehicle's left-front door flexed away from frame.
0.038	Vehicle pitched downward.
0.064	Vehicle's right-rear tire became airborne.
0.066	Vehicle's windshield shattered.
0.068	Vehicle's left-front tire contacted post no. 7.
0.070	Vehicle's left-front tire snagged on post no. 7.
0.072	Post no. 6 deflected forward, Post no. 7 deflected forward.
0.076	Vehicle's left-front tire deflated.
0.088	Vehicle rolled away from system.
0.112	Post no. 8 deflected forward.
0.124	Vehicle's front bumper disengaged and underrode vehicle.
0.150	Vehicle's right-rear tire regained contact with ground.
0.164	Post no. 7 deflected backward. Vehicle was parallel to system.
0.170	Vehicle's left-rear tire contacted rail. Vehicle's left quarter panel contacted rail. Vehicle's rear bumper contacted rail.
0.172	Vehicle's left quarter panel deformed. Post no. 6 deflected backward.
0.176	Post no. 8 deflected backward.
0.188	Vehicle rolled toward system.
0.190	Vehicle pitched upward.
0.228	Vehicle exited system at a speed of 45.1 mph (72.6 km/h).
0.344	Vehicle rolled away from system.
0.626	Vehicle rolled toward system.
0.658	System came to a rest.


0.000 sec


0.100 sec


0.200 sec


0.300 sec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0.500 sec


0.000 sec


0.100 sec


0.200 sec


0.300 sec


0.400 sec


0.500 sec

Figure 264. Sequential Photographs, Test No. STBR-3


0.000 sec


0.000 sec


0.100 sec


0.100 sec


0.200 sec


0.200 sec


0.300 sec


0.300 sec


0.400 sec


0.400 sec


0.500 sec


0.500 sec

Figure 265. Additional Sequential Photographs, Test No. STBR-3

Figure 266. Documentary Photographs, Test No. STBR-3

Figure 267. Vehicle Final Position and Trajectory Marks, Test No. STBR-3

16.3 System Damage

Damage to the bridge rail was minimal, as shown in Figures 268 through 271. Note that shrinkage cracks were observed in the surrogate slab prior to the test and denoted with a black marker. System damage consisted of contact marks on the rails, post no. 7, and post-to-deck connection deck spacer, and minimal concrete cracks near the post-to-deck connection of post no. 7. The length of vehicle contact along the barrier was approximately 21 ft – 10 in. (6.7 m), which spanned from 3½ in. (89 mm) downstream from post no. 6 to 66½ in. (1.7 m) downstream from the centerline of post no. 8.

Contact marks were visible on the upper front-corner of the top rail starting at 3½ in. (89 mm) downstream from the centerline of post no. 6 and extending 21 ft – 10 in. (6.7 m) downstream. Contact marks were visible in the front face of the middle rail starting at 21½ in. (258 mm) downstream from the centerline of post no. 6 and extending 116½ in. (3.0 m) downstream. Contact marks were also noted in the front face of the bottom rail starting at 22 in. (559 mm) downstream from the centerline of post no. 6 and extending 113½ in. (2.9 m) downstream. Tire marks extended along the top face of the upstream, top post stiffener to its center for a total offset distance equal to 9¼ in. (235 mm). Tire marks were also visible along the top face of the top deck spacer at post no. 7 and along the top face of the 1-in. (25-mm) thick plate attachment. The upstream edge of the front flange of post no. 7 slightly buckled above the top stiffener. Plastic vehicle remnants were embedded into middle and bottom rail expansions between post nos. 6 and 7.

Concrete deck spalling was visible on the top corner starting 29¾ in. (756 mm) downstream from the centerline of post no. 6 and extending 48½ in. (1.2 m) downstream. Concrete deck spalling was also visible on the top corner and starting 15¼ in. (387 mm) upstream from post no. 7 and extending 103 in. (2.6 m) downstream. A ¼-in. (6.4-mm) thick by 4¼-in. (108-mm) long concrete crack was found on the top right corner of embedded plate of post no. 7.

Figure 268. System Damage, Test No. STBR-3

Figure 269. System Damage, Rail Span Between Posts Nos. 6 and 7, Test No. STBR-3

Figure 270. System Damage, Post No. 7, Test. No. STBR-3

Figure 271. System Damage, Concrete Damage at Post No. 7 Location, Test No. STBR-3

The maximum lateral permanent set of the barrier system was 0.6 in. (15 mm). The maximum lateral dynamic barrier deflection was 2.9 in. (74 mm) at the top rail between posts nos. 6 and 7, as determined from high-speed digital video analysis. The working width of the system was found to be 15.2 in. (386 mm), also determined from high-speed digital video analysis. A schematic of the permanent set deflection, dynamic deflection, and working width is shown in Figure 272.

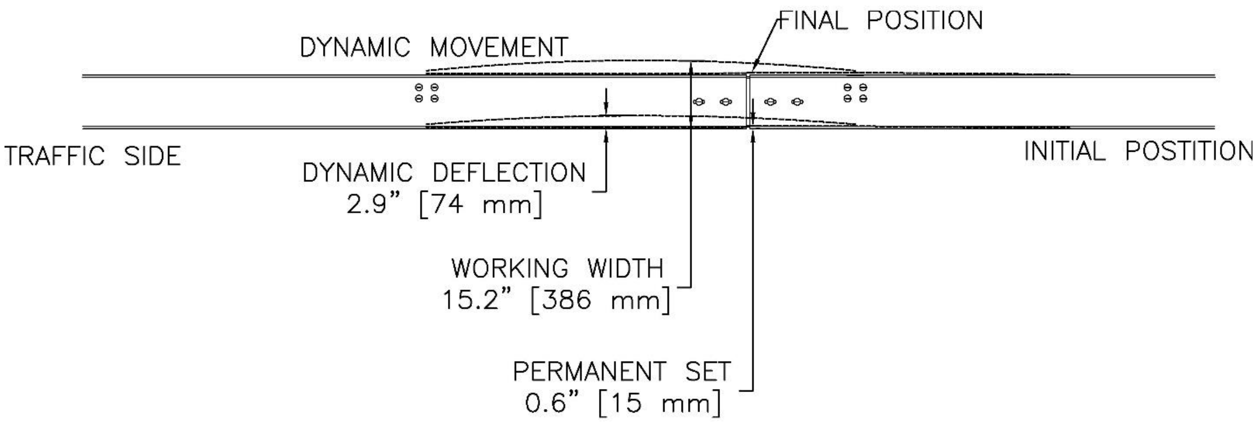


Figure 272. Permanent Set Deflection, Dynamic Deflection, and Working Width, Test No. STBR-3

16.4 Vehicle Damage

The damage to the vehicle was minimal, as shown in Figures 273 through 277. The maximum occupant compartment deformations are listed in Table 80 along with the deformation limits established in MASH 2016 for various areas of the occupant compartment. MASH 2016 defined intrusion or deformation as an occupant compartment being deformed and reduced in size with no observed penetration. Note that none of the established MASH 2016 deformation limits were violated. The entire B-pillar (lateral) and side door (above and below seat) deformed slightly outward. Outward deformations are not considered crush toward the occupant, are denoted as negative numbers in Table 80, and are not evaluated by MASH 2016 criteria. Complete occupant compartment and vehicle deformations and the corresponding locations are provided in Appendix F.

The majority of damage was concentrated on the left front-corner of the vehicle where the impact had occurred. The front bumper cover crushed and was nearly disengaged away from the body. The front bumper crushed 6.5 in. (165 mm) inward and bent forward. The left-front side of the hood was crushed inward 8 in. (203 mm). The left-front fender crushed inward 8 in. (203 mm). The left-front door and the left-back door crushed inward 0.5 in. (13 mm). The left-rear fender crushed inward 0.5 in. (13 mm). The left-front shocks and springs bent inward due to the tire being crushed inward. The left-rear shocks slightly bent due to small inward tire crush. The tie rod of the steering control arm bent toward the rear of the car. The left-front corner of the frame was bent inward and upward.

Figure 273. Vehicle Damage, Front and Rear Views, Test No. STBR-3

Figure 274. Vehicle Damage, Right and Left Views, Test No. STBR-3

Figure 275. Vehicle Damage, Right Corner Views, Test No. STBR-3

Figure 276. Vehicle Damage, Left Side, Test No. STBR-3

Figure 277. Vehicle Damage, Floor Pan and Undercarriage, Test No. STBR-3

Table 80. Maximum Occupant Compartment Intrusions by Location, Test No. STBR-3

LOCATION	MAXIMUM INTRUSION in. (mm)	MASH 2016 ALLOWABLE INTRUSION in. (mm)
Wheel Well & Toe Pan	0.2 (5.1)	≤ 9 (229)
Floor Pan & Transmission Tunnel	0.6 (15.2)	≤ 12 (305)
A-Pillar	0.7 (17.8)	≤ 5 (127)
A-Pillar (Lateral)	0.6 (15.2)	≤ 3 (76)
B-Pillar	0.3 (7.6)	≤ 5 (127)
B-Pillar (Lateral)	-0.4 (-10.2)	N/A[2]
Side Front Panel (in Front of A-Pillar)	0.6 (15.2)	≤ 12 (305)
Side Door (Above Seat)	0.4 (10.2)	≤ 9 (229)
Side Door (Below Seat)	-0.9 (-22.9)	N/A[2]
Roof	0.8 (20.3)	≤ 4 (102)
Windshield	0.9 (22.9)	≤ 3 (76)
Side Window	Intact	No shattering resulting from contact with structural member of test article
Dash	0.6 (15.2)	N/A[1]

Note: Negative values denote outward deformation
N/A[1] – No MASH 2016 criteria exist for this location
N/A[2] – MASH 2016 criteria are not applicable when deformation is outward

16.5 Occupant Risk

The calculated occupant impact velocities (OIVs) and maximum 0.010-sec average occupant ridedown accelerations (ORAs) in both the longitudinal and lateral directions, as determined from the accelerometer data, are shown in Table 81. Note that the OIVs and ORAs were within suggested limits, as provided in MASH 2016. The calculated THIV, PHD, and ASI values are also shown in Table 81. The recorded data from the accelerometers and the rate transducers are shown graphically in Appendix I. Note, the SLICE-1 unit was designated as the primary unit during this test as it was mounted closer to the c.g. of the vehicle.

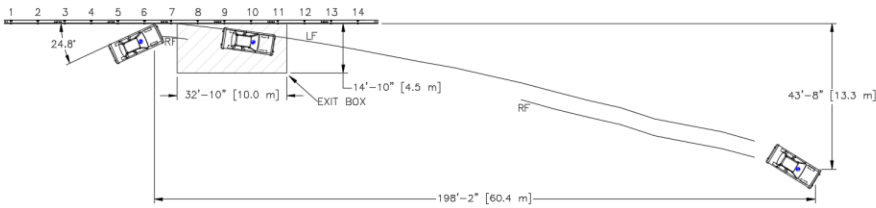
Table 81. Summary of OIV, ORA, THIV, PHD, and ASI Values, Test No. STBR-3

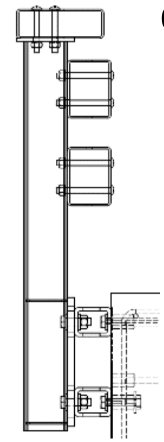
Evaluation Criteria		Transducer		MASH 2016 Limit
		SLICE-1 (primary)	**SLICE-2**	
OIV ft/s (m/s)	Longitudinal	-18.46 (-5.63)	-18.70 (-5.63)	±40 (12.2)
	Lateral	33.19 (10.12)	31.48 (9.59)	±40 (12.2)
ORA g's	Longitudinal	-16.82	-15.76	±20.49
	Lateral	-14.77	-13.31	±20.49
MAX ANGULAR DISP. deg.	Roll	-7.9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

16.6 Discussion

The analysis of the test results for test no. STBR-3 showed that the system adequately contained and redirected the 1100C vehicle with controlled lateral displacements of the barrier. A summary of the test results and sequential photographs are shown in Figure 278. Detached elements, fragments, or other debris from the test article did not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or work-zone personnel. Deformations of, or intrusions into, the occupant compartment that could have caused serious injury did not occur. The test vehicle did not penetrate nor ride over the barrier and remained upright during and after the collision. Vehicle roll, pitch, and yaw angular displacements, as shown in Appendix I, were deemed acceptable because they did not adversely influence occupant risk nor cause rollover. After impact, the vehicle exited the barrier at a trajectory angle of 4.6 degrees, and its trajectory did not violate the bounds of the exit box. Therefore, test no. STBR-3 was determined to be acceptable according to the MASH 2016 safety performance criteria for test designation no. 4-10.

0.000 sec 0.100 sec 0.200 sec 0.300 sec 0.400 sec





- Test Agency .... MwRSF
- Test Number STBR-3
- Date 3/1/2019
- MASH 2016 Test Designation No. 4-10
- Test Article .... Steel, Side-Mounted, Beam-and-Post, Bridge Rail
- Total Length .... 111 ft – 11¼ in. (34.1 m)
- Key Component – Top Rail
  - Length .... 191¼ in. (4,858 mm)
  - Width .... 12 in. (305 mm)
  - Depth .... 4 in. (102 mm)
- Key Component - Post
  - Length .... 58½ in. (1,486 mm)
  - Width .... 6 in. (152 mm)
  - Spacing .... 8 ft (2.4 m)
- Soil Type
- Vehicle Make /Model .... 2009 Kia Rio
  - Curb .... 2,456 lb (1,114 kg)
  - Test Inertial .... 2,408 lb (1,092 kg)
  - Gross Static .... 2,569 lb (1,120 kg)
- Impact Conditions
  - Speed .... 62.0 mph (99.8 km/h)
  - Angle .... 24.8 deg.
  - Impact Location .... 61.3 in (1.6 m) upstream from post no. 7
- Impact Severity ....54.5 kip-ft (73.9 kJ) > 51.0 kip-ft (69.7 kJ) limit from MASH 2016
- Exit Conditions
  - Speed .... 45.1 mph (72.6 km/h)
  - Angle .... 4.6 deg.
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance ....198 ft – 2 in. (60.4 m) downstream from of impact
  43 ft – 8 in. (13.3 m) laterally in front
- Vehicle Damage .... Moderate
  - VDS [56] 11-LFQ-6
  - CDC [57] 11-LFAW-6
  - Maximum Interior Deformation .... 0.9 in. (23 mm)

- Test Article Damage .... Minimal
- Maximum Test Article Deflections
  - Permanent Set .... 0.6 in. (15 mm)
  - Dynamic .... 2.9 in. (74 mm)
  - Working Width .... 15.2 in. (386 mm)
- Transducer Data

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

Figure 278. Summary of Test Results and Sequential Photographs, Test No. STBR-3

17 CONSTRUCTION DETAILS - TEST NO. STBR-4

The test installation for the bridge rail system consisted of steel rails, posts assemblies, post-to-rail and rail-to-rail connections, as well as a surrogate concrete bridge deck, as shown in Figures 279 through 305. The total length of the bridge rail was 159 ft – 11¼ in. (48.7 m). Photographs of the test installation are shown in Figures 306 through 310. Material specifications, mill certifications, and certificates of conformity for the system materials are shown in Appendix D.

The system was constructed with twenty galvanized ASTM A992, W6x15 (W150x22.5) steel post assemblies spaced on 96-in. (2,438-mm) centers. Post assembly nos. 1 through 13 were side-mounted to the vertical side edge of the surrogate, reinforced-concrete bridge deck. For the construction of the surrogate concrete bridge deck, the threaded rod and coupling nuts were held in place to the embedded vertical plates by placing bolts through the formwork, rather than utilizing the option of welding the coupling nuts to the embedded vertical plates, as shown in Figure 301. For the concrete slab that was repaired at post no. 9, four studs were welded to the embedded vertical plate and the coupling nuts were not welded to the embedded vertical plate, as shown in Figure 282. Post assembly nos. 14 through 20 were surface-mounted to the top of existing concrete tarmac, which provided the necessary system length for vehicle redirection and were used for testing-purposes only.

Post assembly nos. 1 through 13 were 58⅞ in. (1,495 mm) long. An ASTM A572 Grade 50 steel plate PL 8 in. x 8 in. x ⅜ in. (PL 203 mm x 203 mm x 10 mm) was attached to the top of each post assembly with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Similarly, an ASTM A572 Grade 50 steel, vertical plate PL 13 in. x 17¾ in. x 1 in. (PL 330 mm x 451 mm x 25 mm) was attached to bottom of the front flange of each post with all-around ¼-in. (6.4-mm) fillet welds. Four gusset plates, fabricated with ASTM A572 Grade 50 steel plate, measuring PL 6⅛ in. x 5$^{11}/_{16}$ in. x ¼ in. (PL 156 mm x 144 mm x 6.4 mm), were welded to the top and bottom of the vertical plates, inner faces of post flanges, and web with all-around ¼-in. (6.4-mm) fillet welds. Post assembly nos. 1 through 13 were bolted to the tension and compression sides of the vertical plates with ASTM A500 Grade 50 horizontal spacer tubes, which were specified as HSS 5-in. x 4-in. x ½-in. (HSS 127-mm x 102-mm x 13-mm) sections.

Post assembly nos. 1 through 13 were bolted to the horizontal spacer tubes using ASTM F3125 Grade A325 1-in. (25-mm) diameter by 3½-in. (89-mm) long, heavy hex-head bolts with ¼-in. (6.4-mm) thick, ASTM A36 steel square washers and 1-in. (25-mm) diameter ASTM A563DH heavy hex nuts. The deck anchorage in the tension region consisted of two 1-in. (25-mm) diameter by 32¾-in. (832-mm) long, ASTM F1554 Grade 105 all-thread anchor rods with ASTM A563DH heavy hex coupling nuts and ASTM A563DH heavy hex nuts. The deck anchorage in the compression region consisted of two 1-in. (25-mm) diameter, ASTM A449 anchor bolts with ¼-in. (6.4-mm) thick, 3-in. (76-mm) ASTM A36 steel square washers, and ASTM A563DH heavy hex coupling nuts. A $^{3}/_{16}$-in. (4.8-mm) thick, vertical embedment plate was used at every post location.

Post assembly nos. 14 through 20 were 32 in. (813 mm) long. Post assembly nos. 14 through 20 consisted of three parts – a base plate, a top plate, and a vertical post. The top plate consisted of an ASTM A572 Grade 50 steel plate measuring PL 8 in. x 8 in. x ⅜ in. (PL 203 mm x 203 mm x 10 mm) with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Similarly, the bottom plate consisted of an ASTM A572 Grade 50 steel plate measuring PL 12 in. x 12 in. x ¾ in. (PL 305 mm x 305 mm x 19 mm) with all-around $^{3}/_{16}$-in. (4.8-mm) fillet welds. Finally, the post was fabricated with ASTM A992 W6x15 (W150x22.5) sections measuring 30⅞ in. (784 mm) long. Post assembly nos. 14 through 20 were anchored to the existing tarmac with four ¾-in. (19-mm) diameter by 12-in. (305-mm) long ASTM F1554 Grade 36 all-thread anchor rods with ¼-in. (6.4-mm) thick, ASTM A36 steel square washers, and ASTM A563DH heavy hex nuts.

The three rail elements consisted on an ASTM A500 Grade C HSS 12 in. x 4 in. x ¼ in. (HSS 304.8 mm x 101.6 mm x 6.4 mm) section for the top rail and ASTM A500 Grade C HSS 8 in. x 6 in. x ¼ in. (HSS 203.2 mm x 152.4 mm x 6.4 mm) section for the lower two rails. Rail-to-rail connections were located 2 ft (610 mm) downstream from every other post location. The top rails were attached to the post assemblies with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long, ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom rails were attached to the front flanges of the posts with two staggered ¾-in. (19-mm) diameter by 7½-in. (191-mm) long ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

The splice tube for the top rails consisted of two horizontal PL 30 in. x 10⅝ in. x $^{5}/_{16}$ in. (PL 762 mm x 270 mm x 8 mm) and two vertical PL 30 in. x 2⅝ in. x ⅜ in. (PL 762 mm x 67 mm x 10 mm) attached with ¼-in. (6.4-mm) fillet welds. The splice tubes for the middle and bottom rails consisted on two vertical PL 30 in. x 6⅝ in. x ⅜ in. (PL 762 mm x 168 mm 10 mm) and two horizontal PL 30 in. x 4⅝ in. x $^{5}/_{16}$ in. (PL 762 mm x 117 mm x 8 mm) attached with ¼-in. (6.4-mm) fillet welds. The top splice tubes were attached to the top rail end sections with four ¾-in. (19-mm) diameter by 6-in. (152-mm) long ASTM A449 round-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts. The middle and bottom splice tubes were attached to the rail end sections with two ¾-in. (19-mm) diameter by 9½-in. (241-mm) long, ASTM A449 hex-head bolts with ASTM F436 flat washers and ASTM A563DH heavy hex nuts.

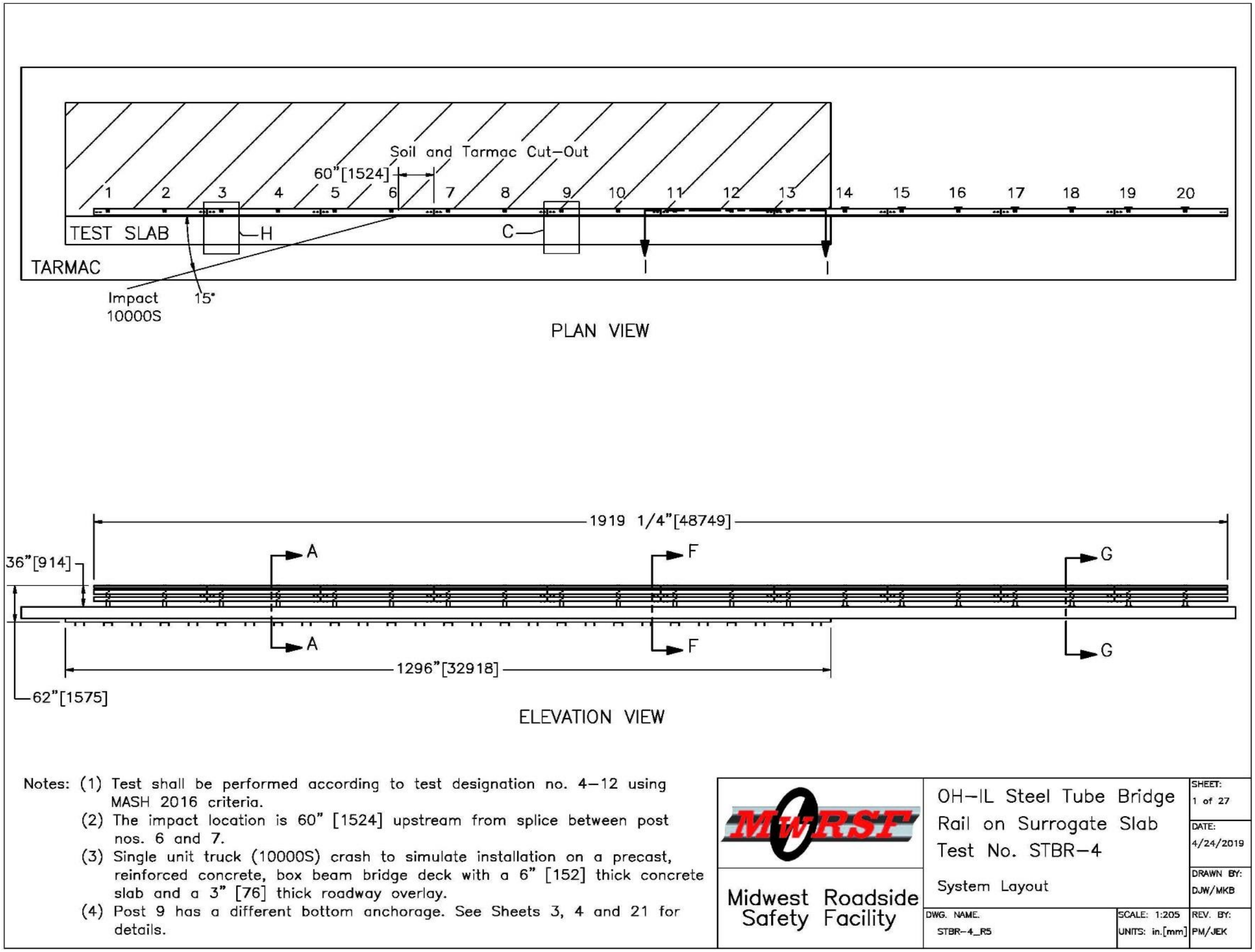


Figure 279. System Layout and Impact Location, Test No. STBR-4

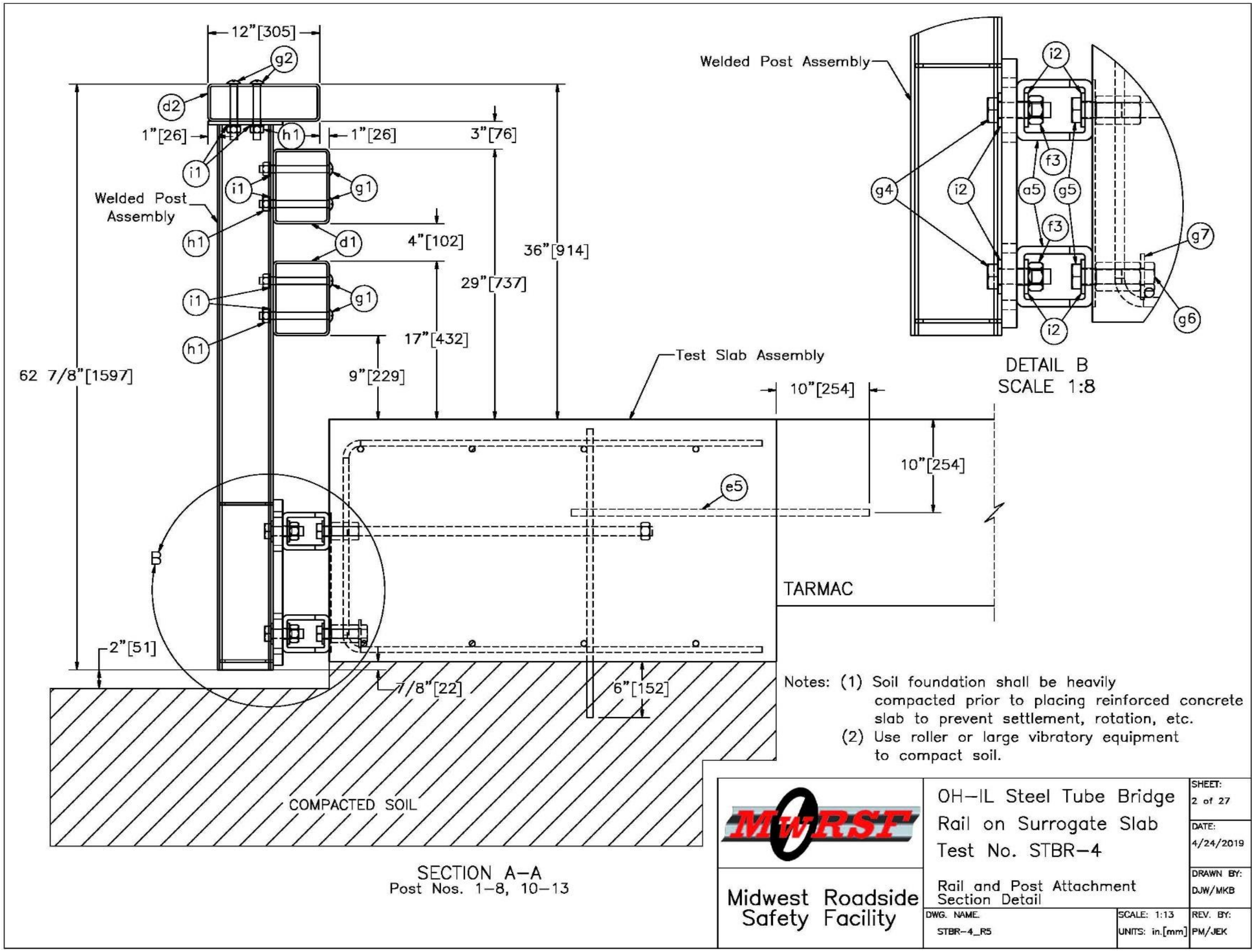


Figure 280. Rail and Post Attachment Section Detail, Test No. STBR-4

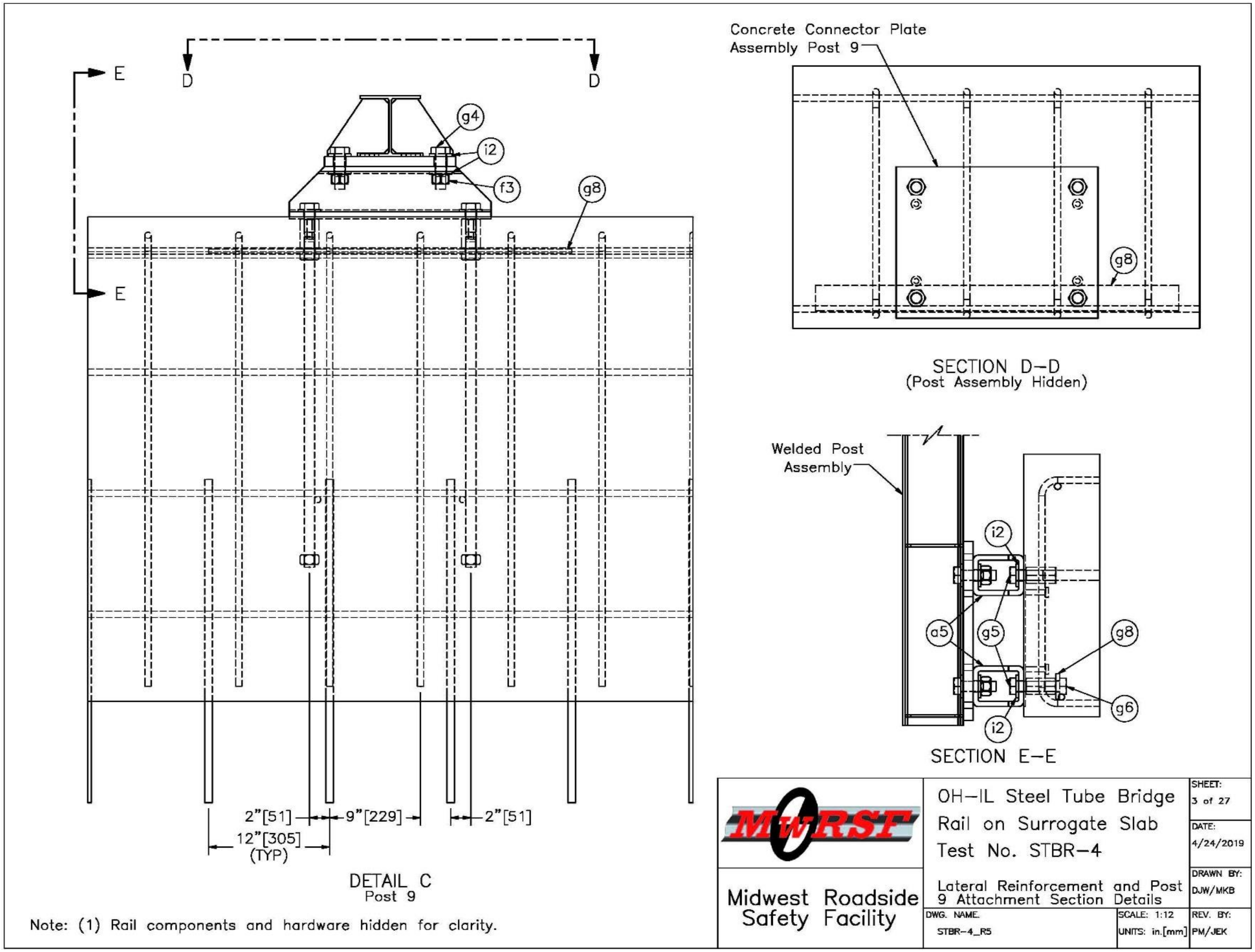


Figure 281. Post-to-Deck Connection Details, Test No. STBR-4

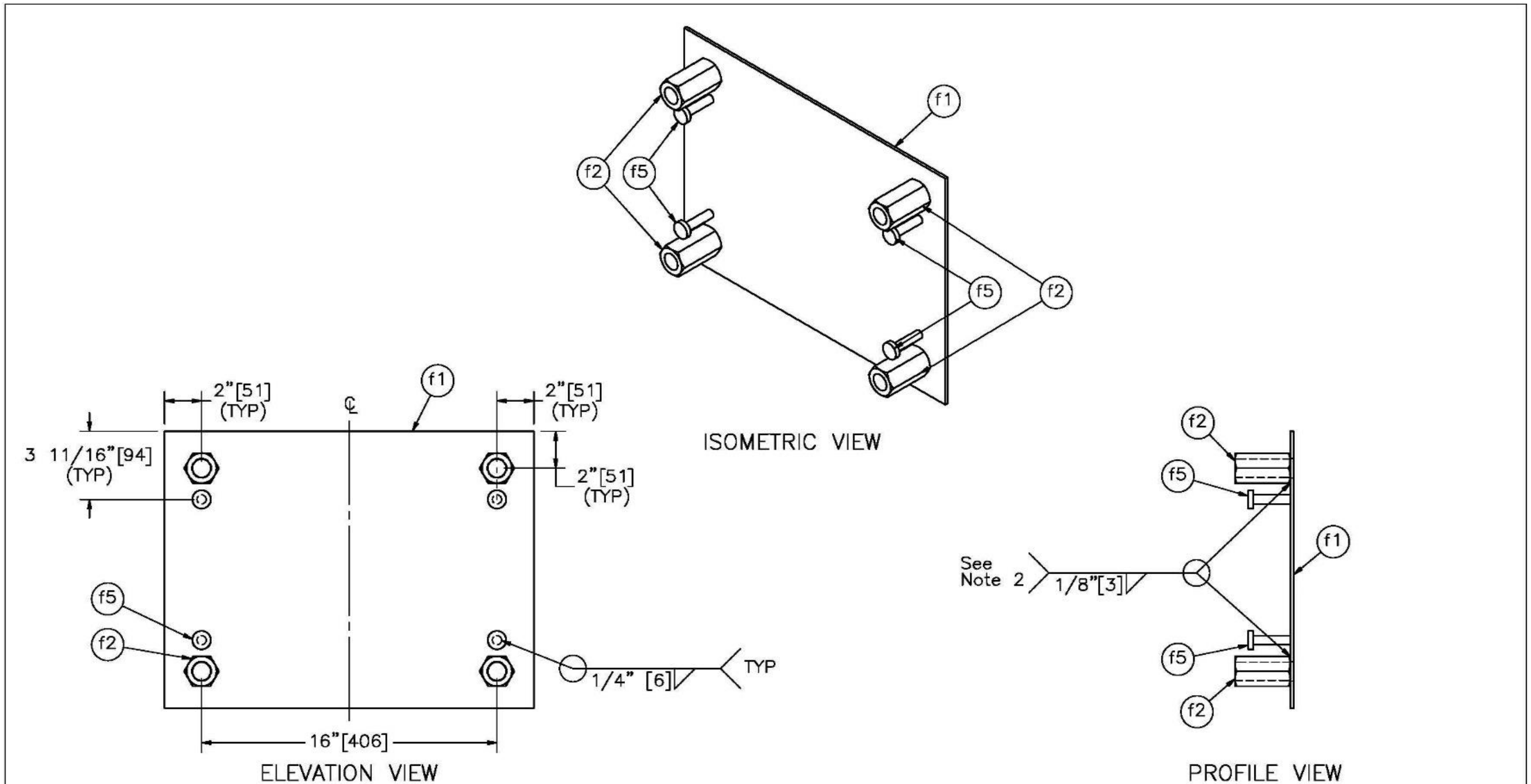


Plate Assembly
Location (Post No. 9)

Notes: (1) Connector plate assembly is symmetric along the centerline.
(2) During casting, the nuts may be held in place by extending bolts through the formwork into the nuts, or by tack welding nuts to inside face of the plate. The welds are not structurally important.

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
–	1	Post 9 Concrete Connector Plate Assembly	–	ASTM A123	–
f1	1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	ASTM A123	–
f2	4	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–
f5	4	1/2" [13] Dia. Shear Stud, 2" [51] Long	ASTM A108	–	–

Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Concrete Connector Plate Assembly Post 9 Details

SHEET: 4 of 27
DATE: 4/24/2019
DRAWN BY: DJW/MKB
DWG. NAME. STBR–4_R5
SCALE: 1:8
UNITS: in.[mm]
REV. BY: PM/JEK

Figure 282. Concrete Connector Plate Assembly Details, Test No. STBR-4

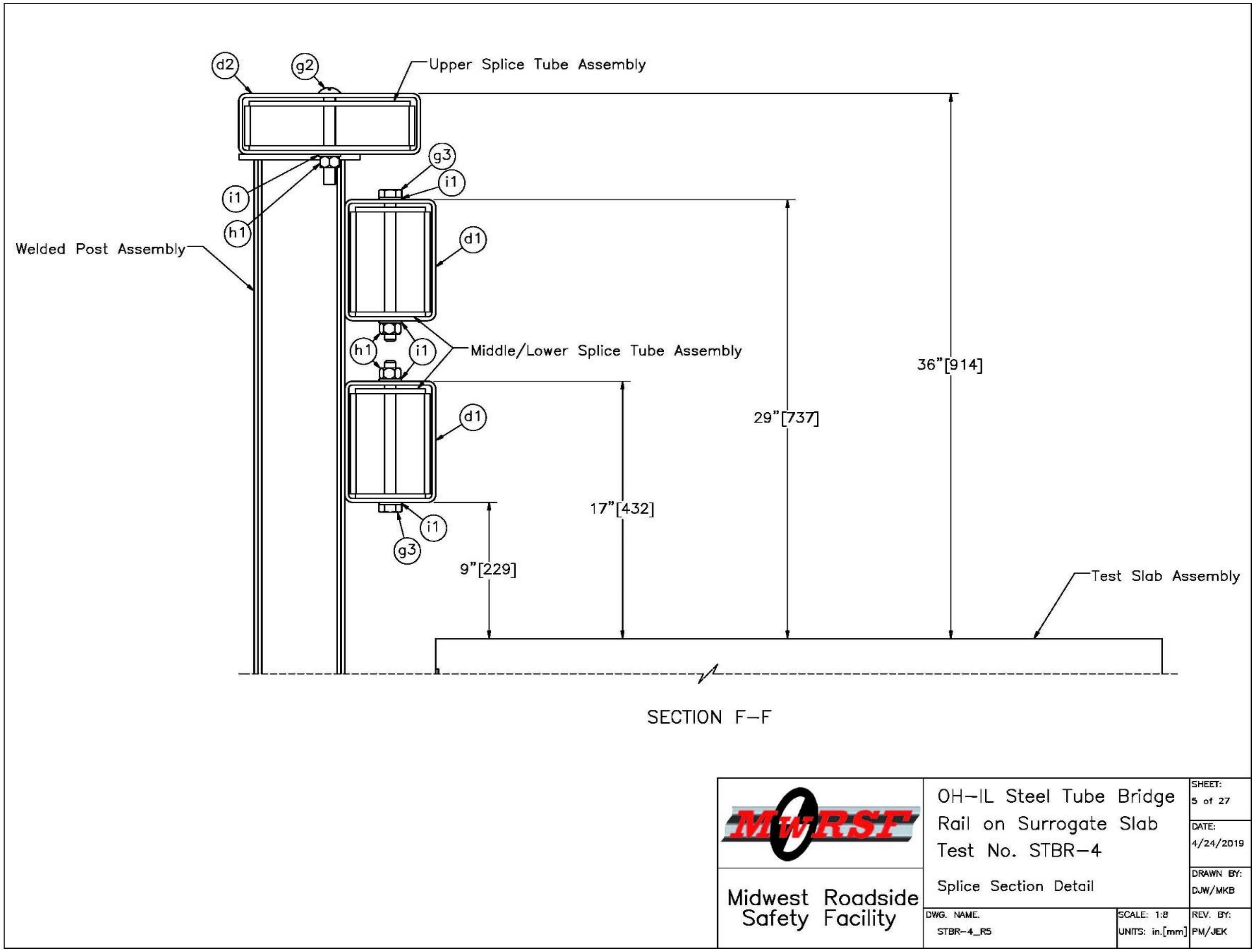


Figure 283. Splice Section Detail, Test No. STBR-4

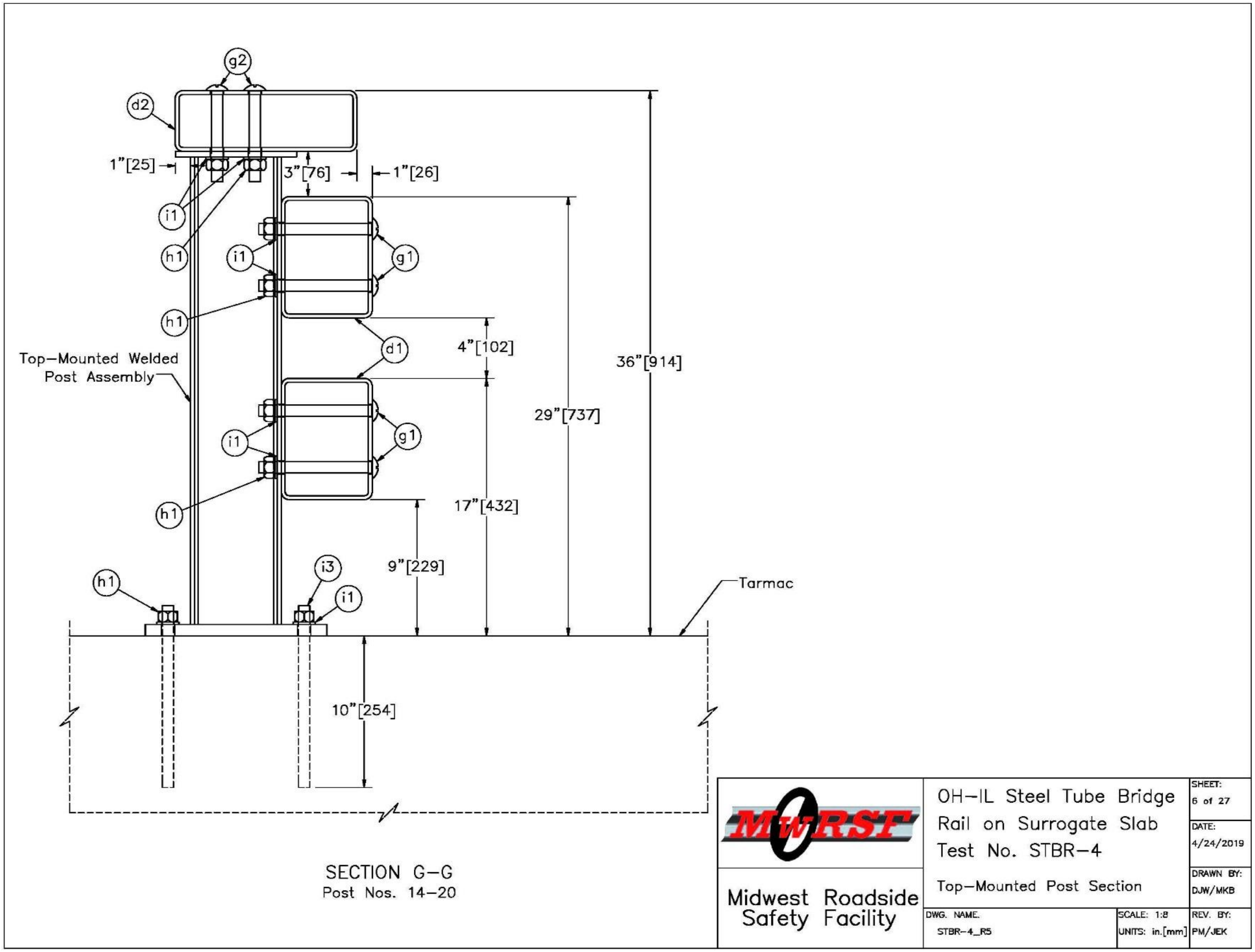


Figure 284. Top-Mounted Post Section, Test No. STBR-4

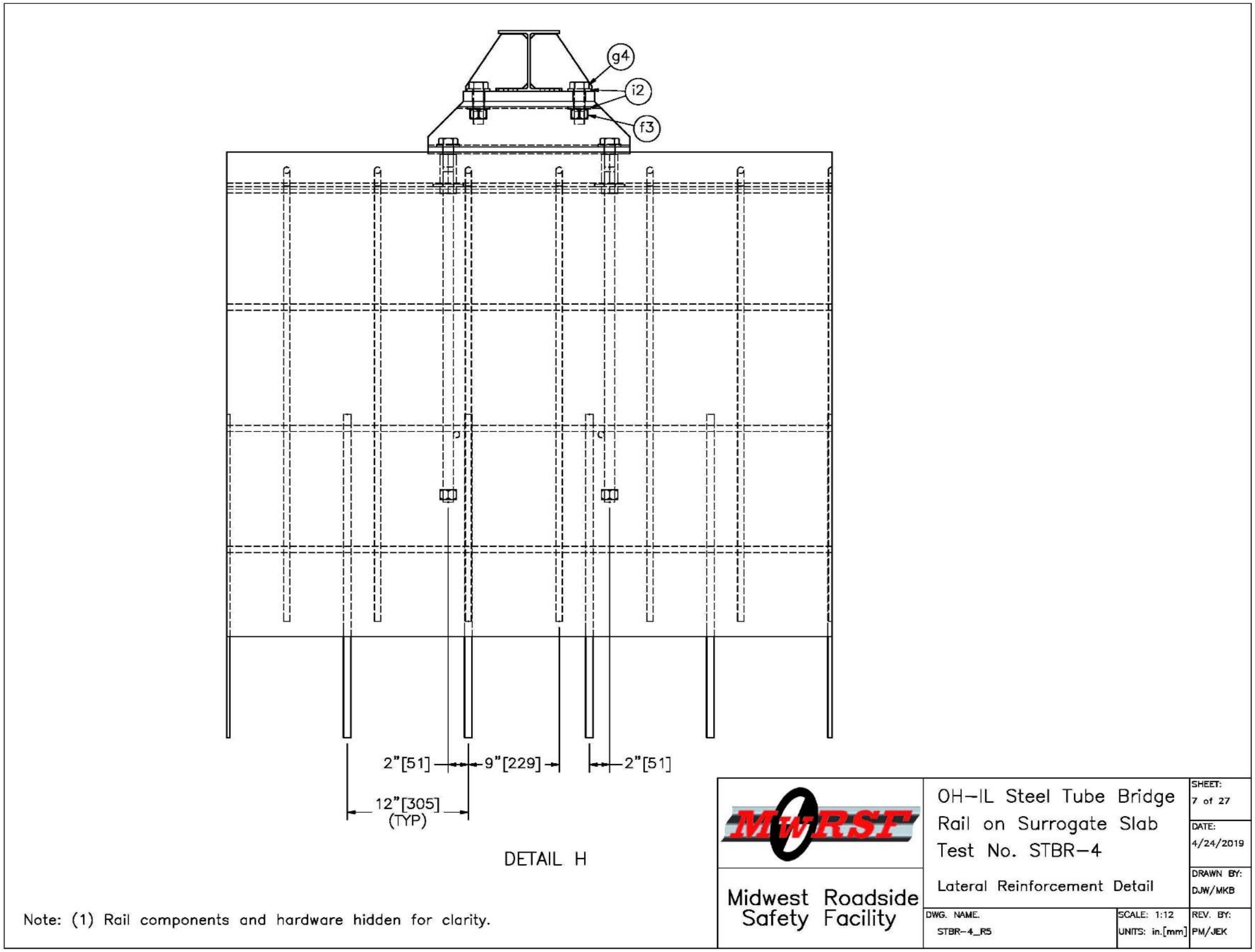


Figure 285. Lateral Reinforcement Detail, Test No. STBR-4

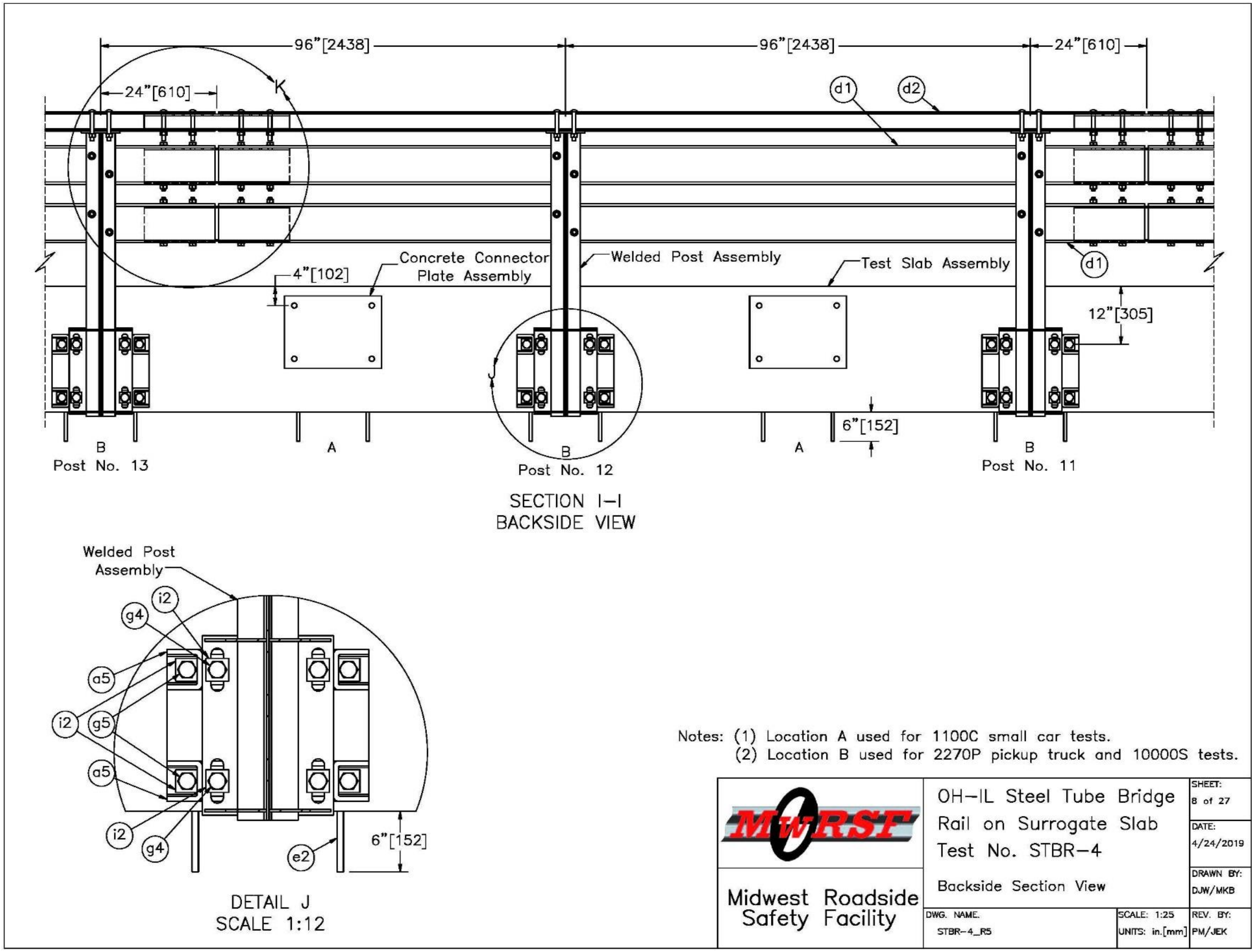


Figure 286. Backside Section View, Test No. STBR-4

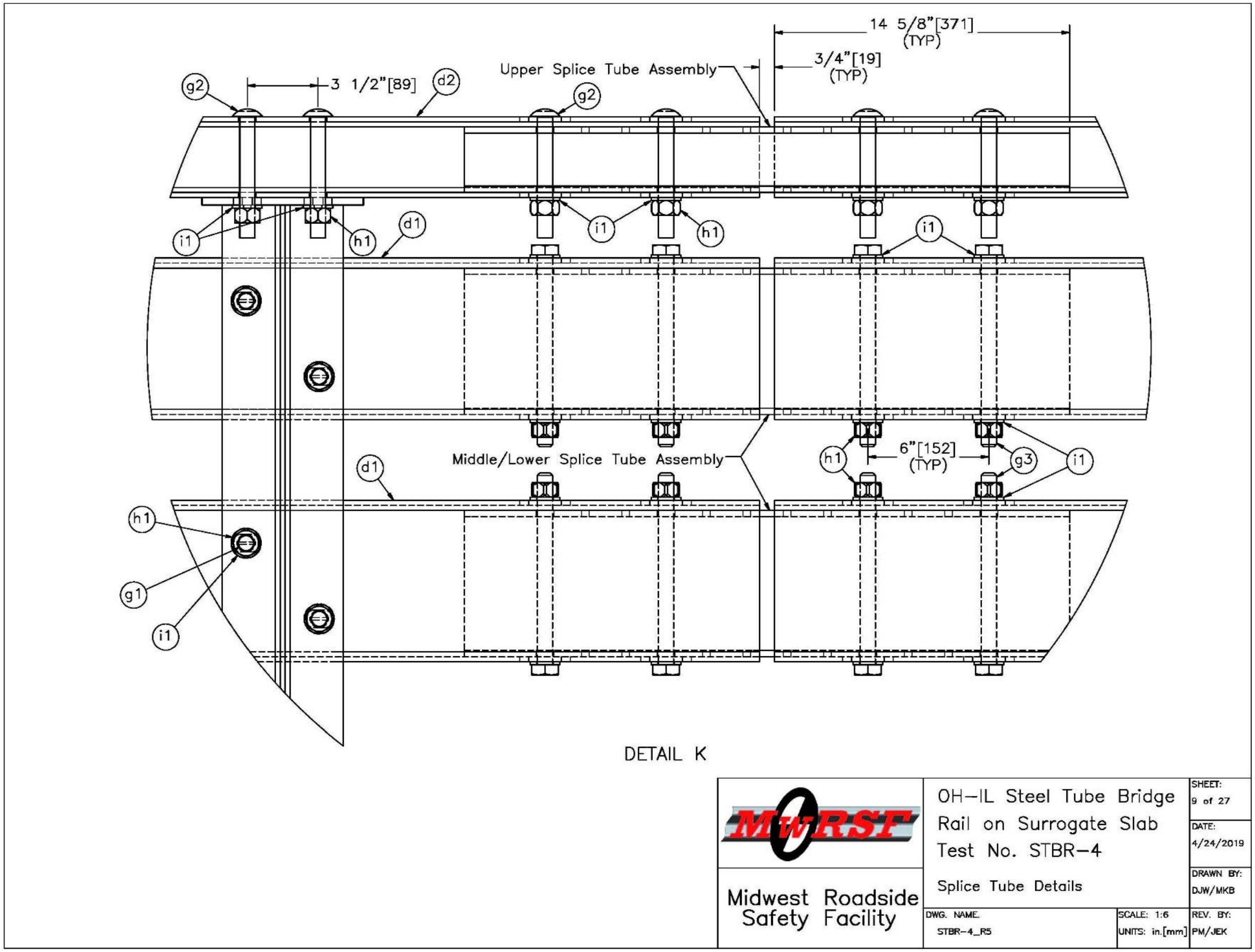
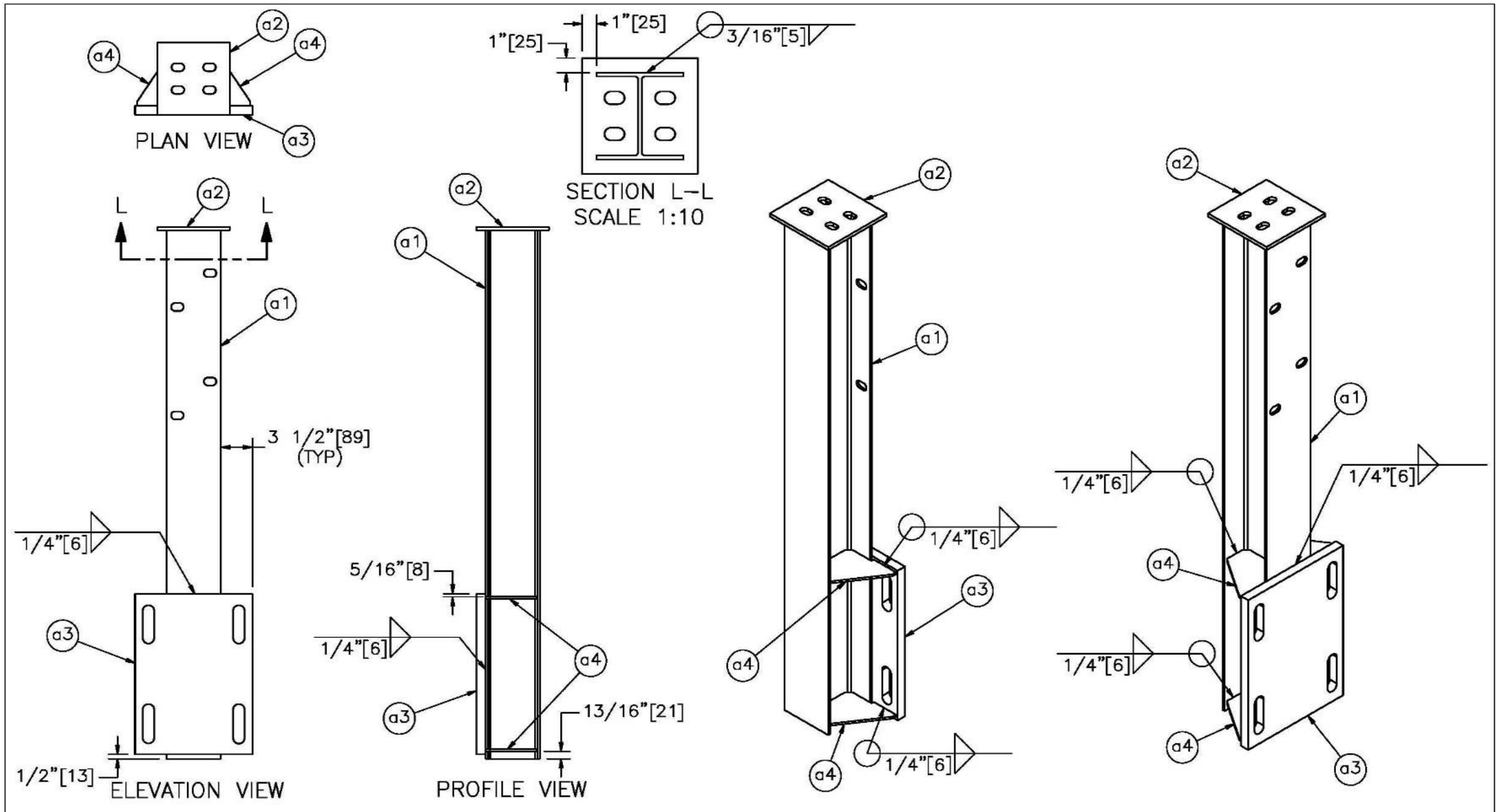


Figure 287. Splice Tube Section Details, Test No. STBR-4



Notes: (1) Gussets on both sides of post.
(2) To prevent welds to end at edges of gussets, leave 5/16" [8] of base material beyond weld to edge of welded part.
(3) Leave coped corners of gussets open for drainage. Do not fill with welds.

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	13	Welded Post Assembly	–	ASTM A123
a1	1	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	See Assembly
a2	1	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly
a3	1	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly
a4	4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly

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OH-IL Steel Tube Bridge Rail on Surrogate Slab
Test No. STBR-4

Welded Post Assembly Details

SHEET: 10 of 27
DATE: 4/24/2019
DRAWN BY: DJW/MKB
DWG. NAME. STBR-4_R5
SCALE: 1:16
UNITS: in.[mm]
REV. BY: PM/JEK

Figure 288. Welded Post Assembly Details, Test No. STBR-4

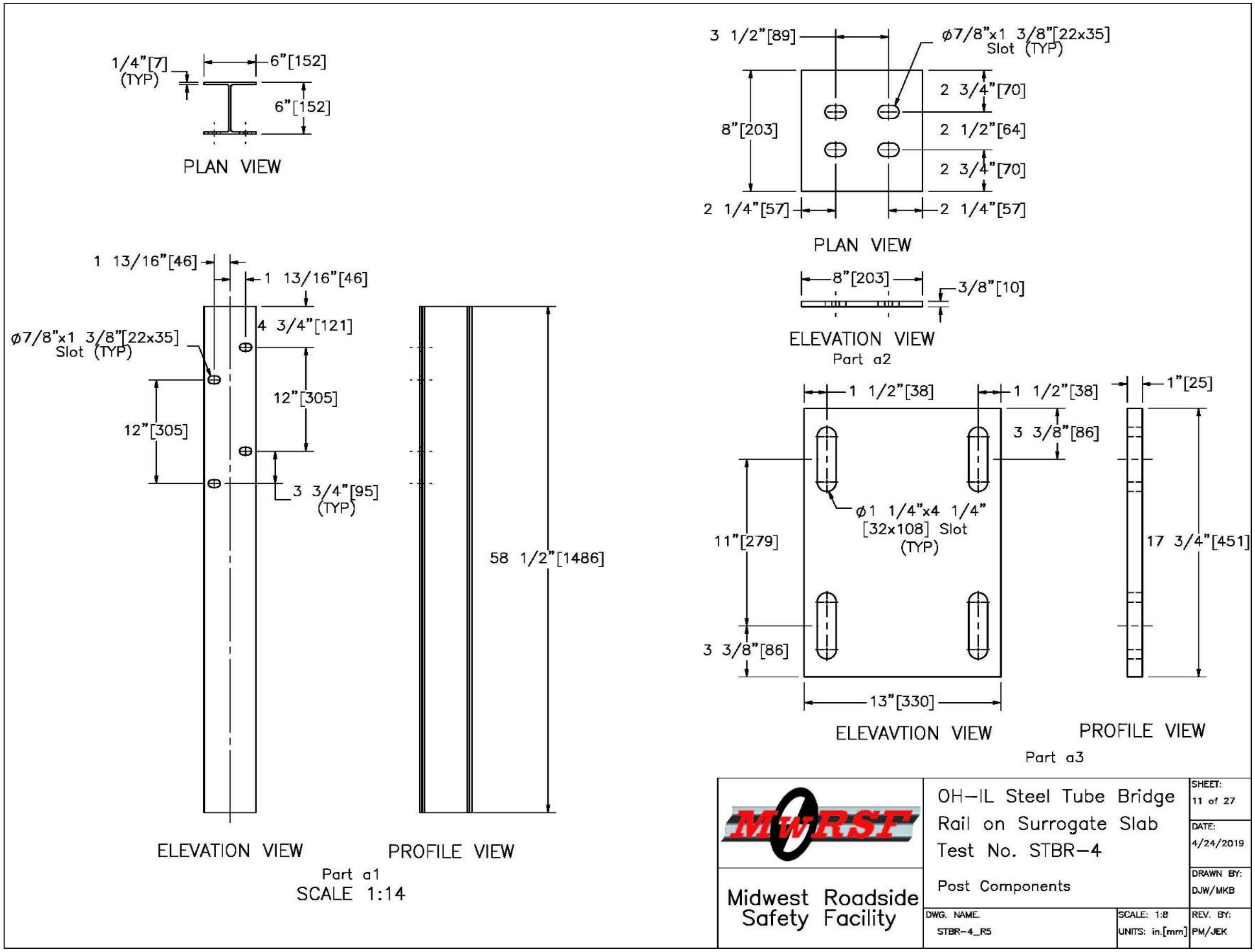


Figure 289. Post Components, Test No. STBR-4

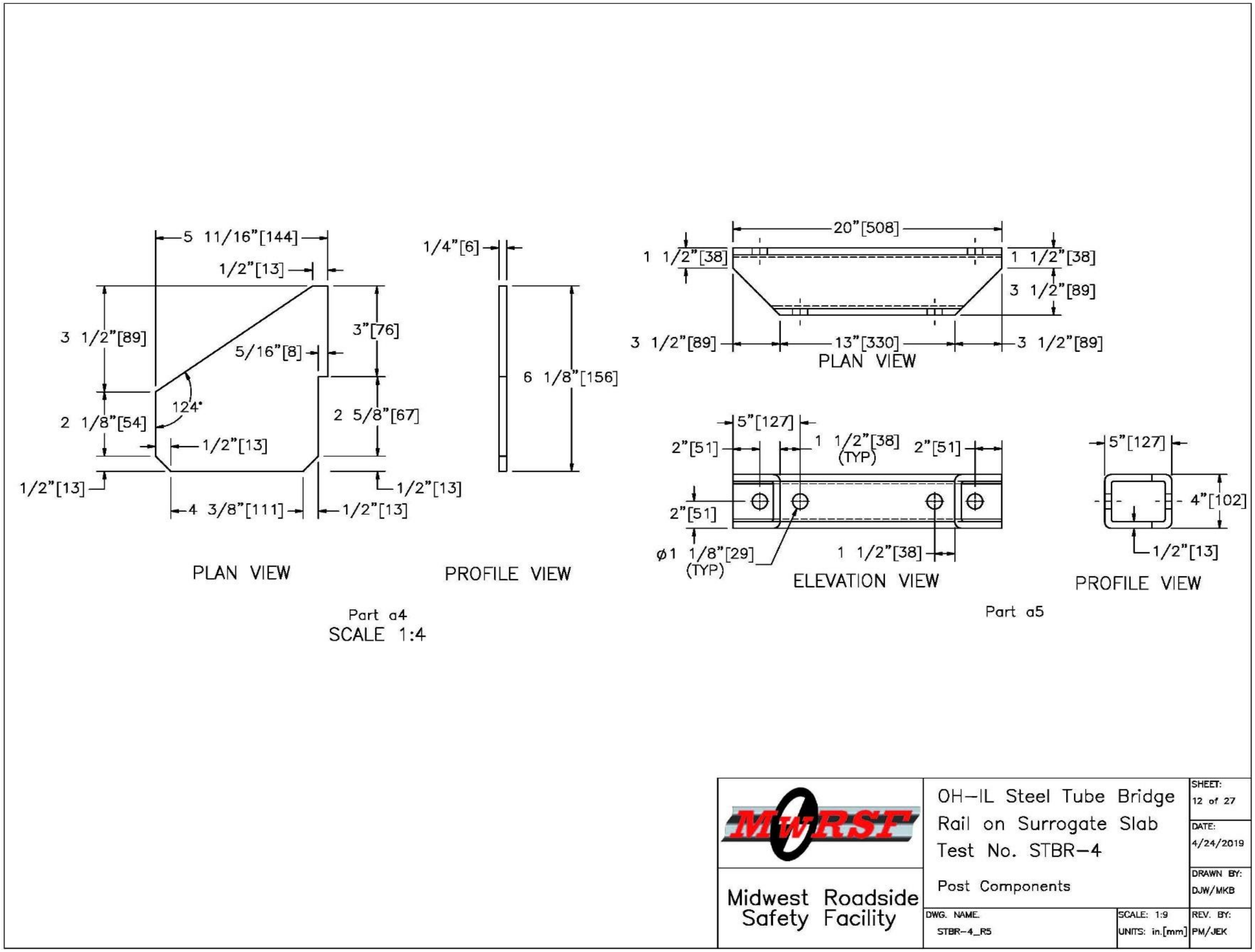
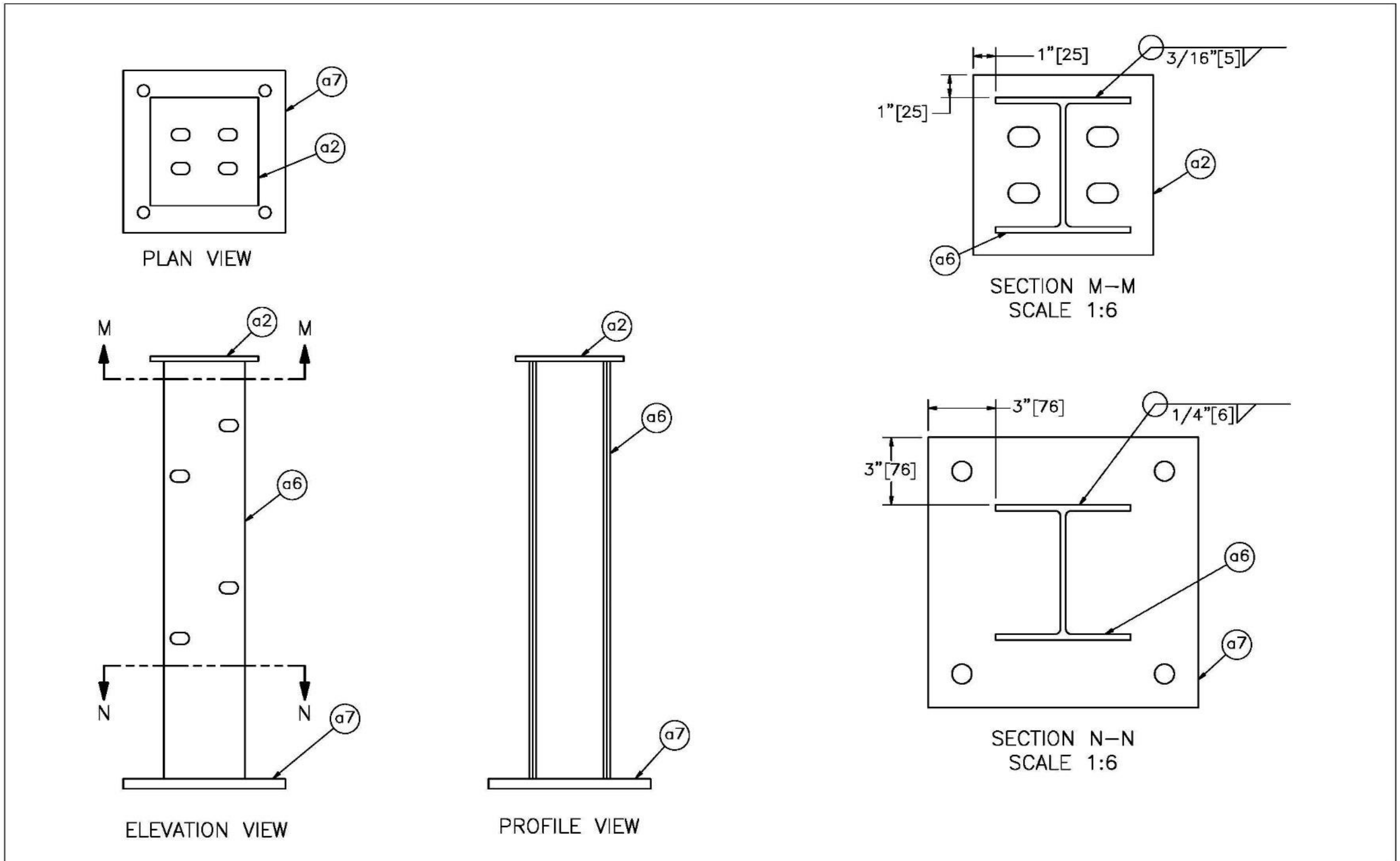


Figure 290. Post Components, Test No. STBR-4



Item No.	QTY.	Description	Material Specification	Treatment Specification
–	7	Top–Mounted Welded Post Assembly	–	ASTM A123
a2	1	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly
a6	1	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	See Assembly
a7	1	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	See Assembly

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Top–Mounted Welded Post Assembly

SHEET: 13 of 27

DATE: 4/24/2019

DRAWN BY: DJW/MKB

DWG. NAME. STBR–4_R5

SCALE: 1:10

UNITS: in.[mm]

REV. BY: PM/JEK

Figure 291. Top-Mounted Welded Post Assembly, Test No. STBR-4

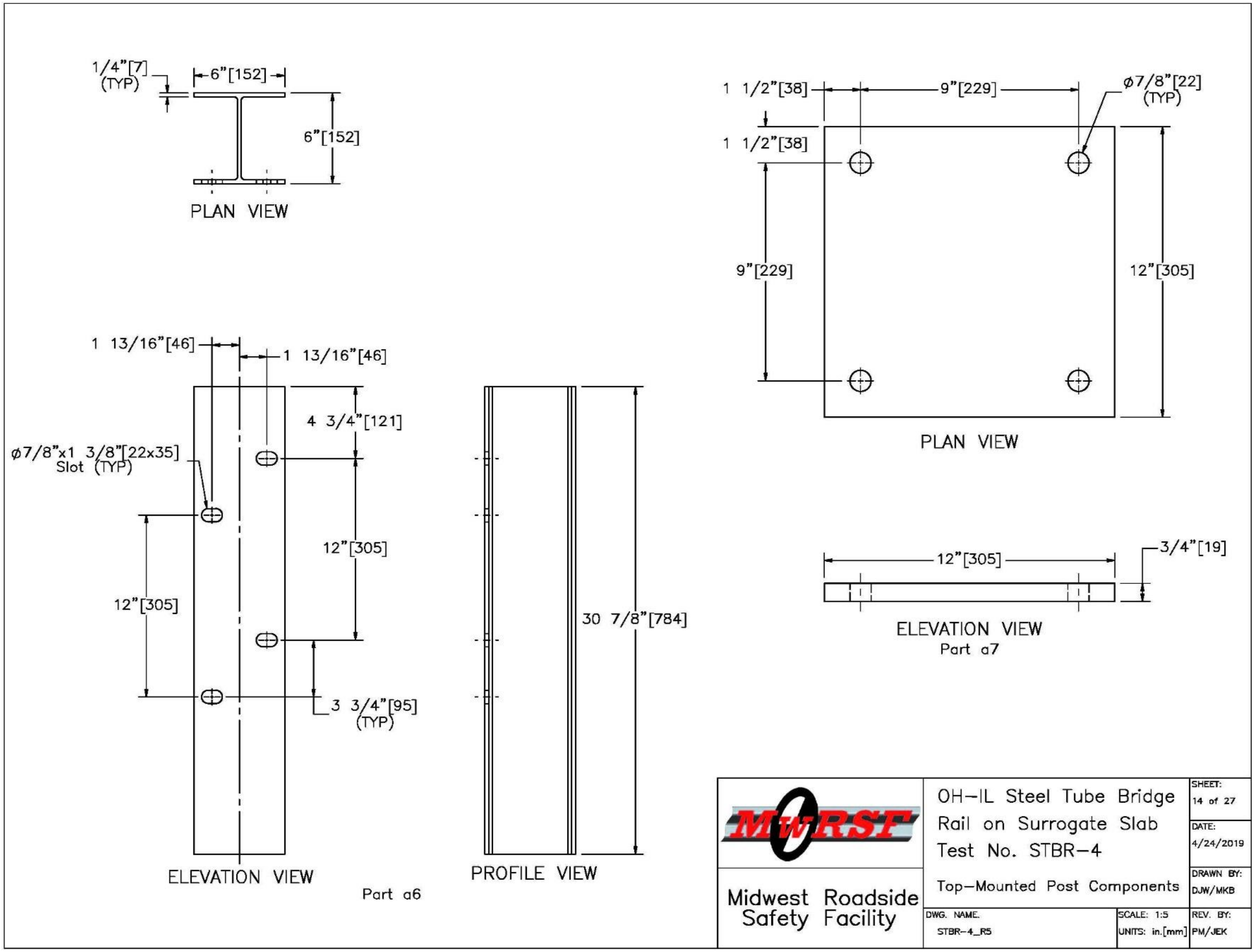
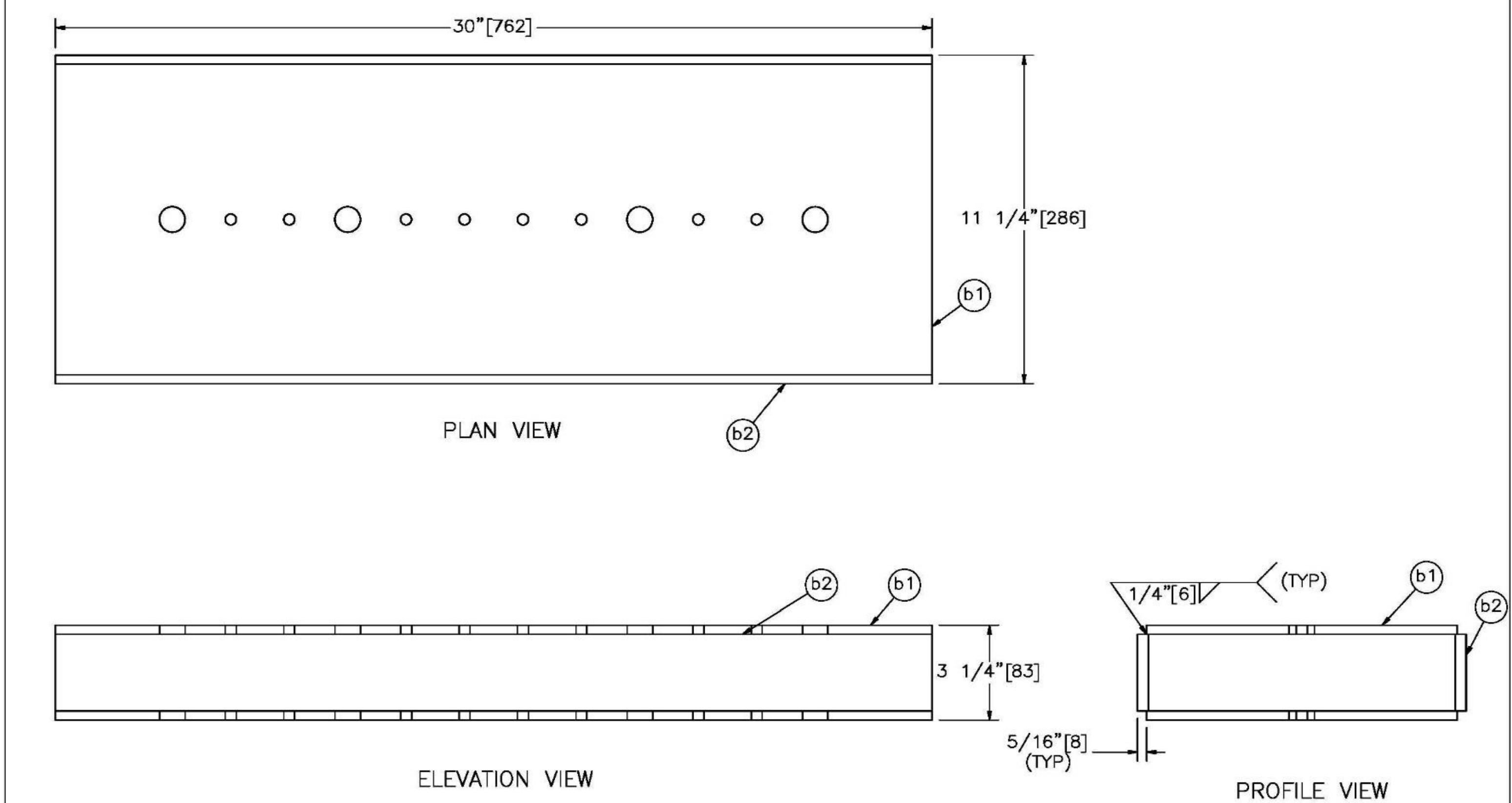


Figure 292. Top-Mounted Post Components, Test No. STBR-4



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	9	Upper Splice Tube Assembly – Option 1	–	ASTM A123
b1	2	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly
b2	2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Upper Splice Tube Assembly

SHEET: 15 of 27
DATE: 4/24/2019
DRAWN BY: DJW/MKB
DWG. NAME. STBR–4_R5
SCALE: 1:5
UNITS: in.[mm]
REV. BY: PM/JEK

Figure 293. Upper Splice Tube Assembly, Test No. STBR-4

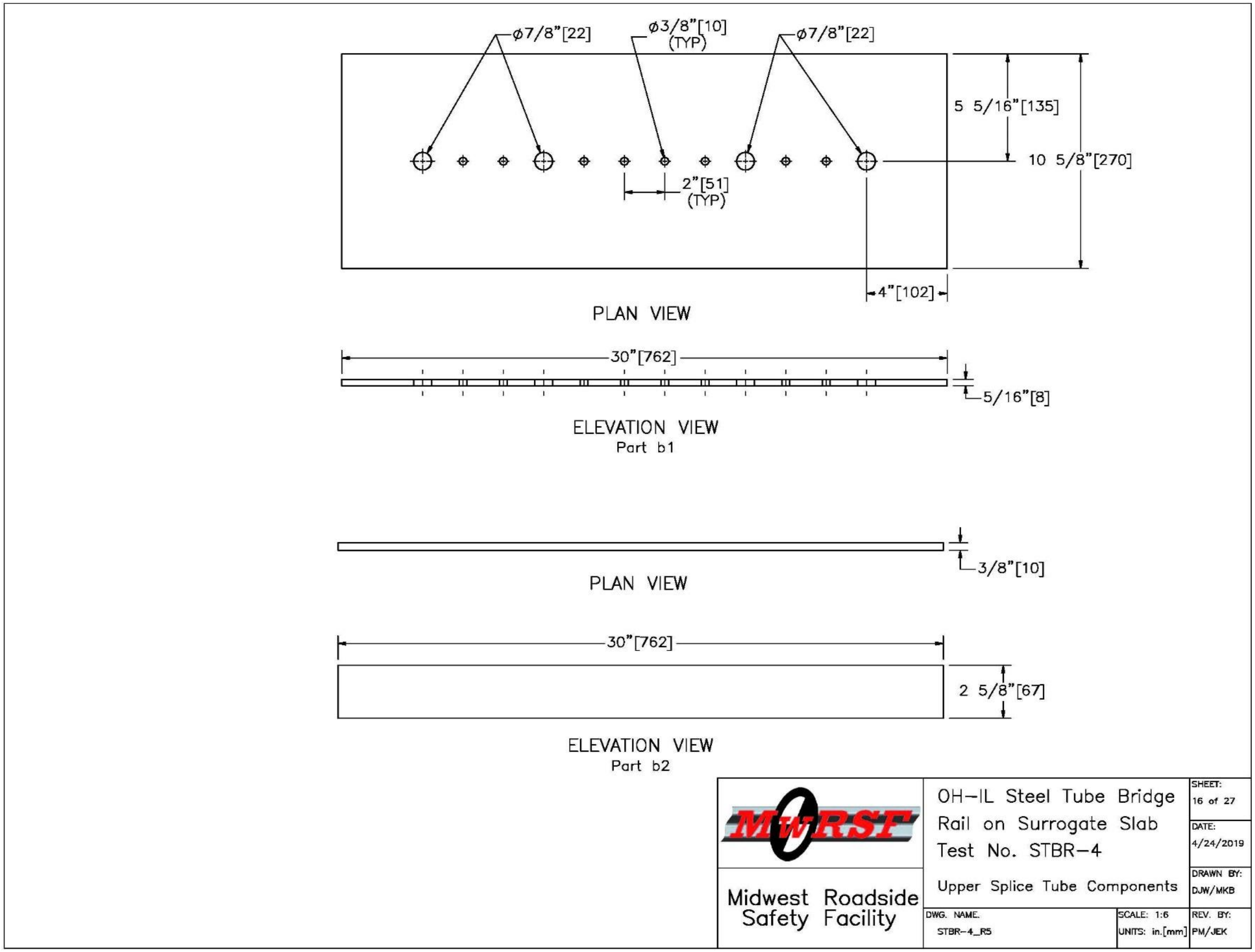
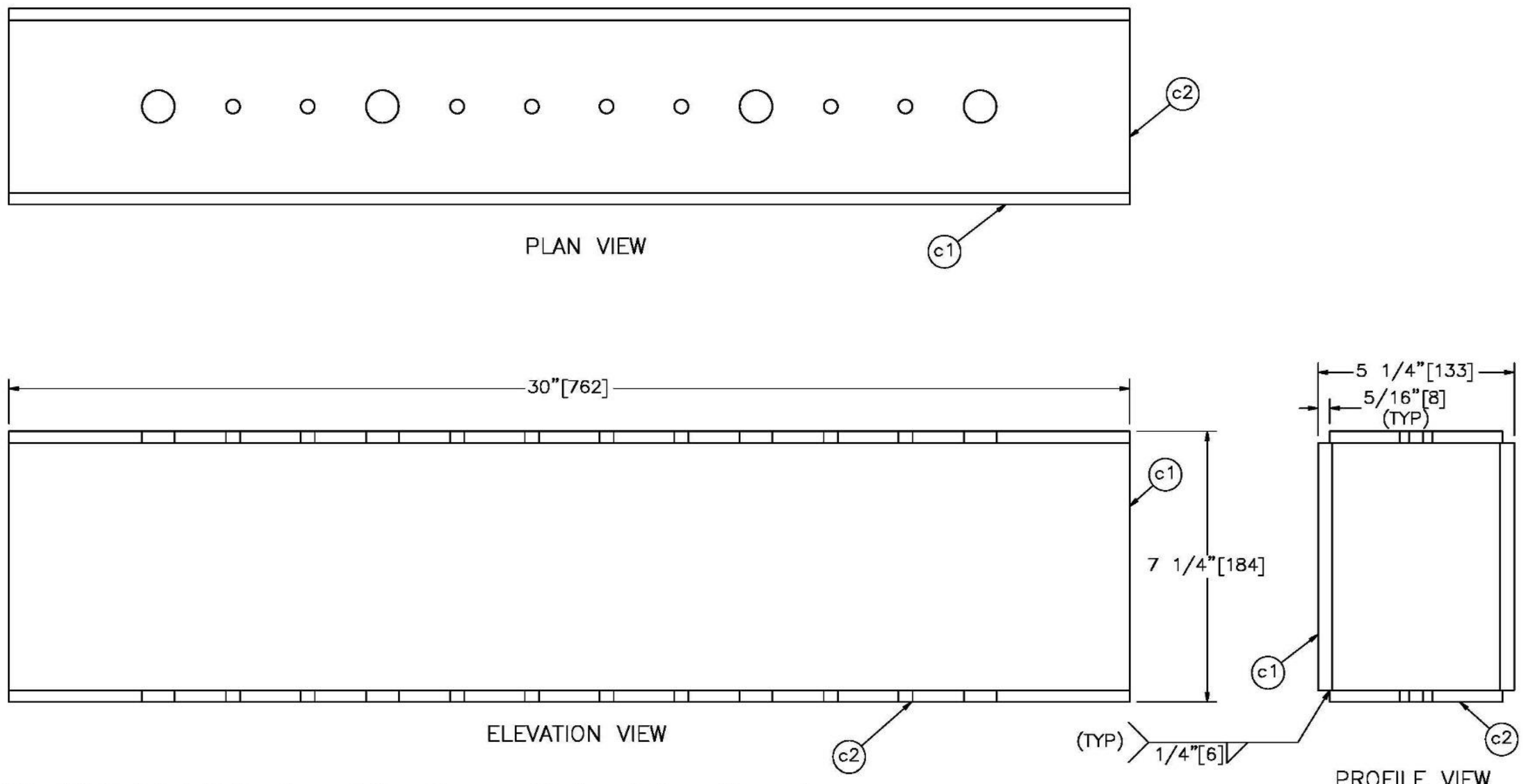


Figure 294. Upper Splice Tube Components, Test No. STBR-4



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

Item No.	QTY.	Description	Material Specification	Treatment Specification
–	18	Middle/Lower Splice Tube Assembly – Option 1	–	ASTM A123
c1	2	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly
c2	2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Middle/Lower Splice Tube Assembly

SHEET: 17 of 27	DATE: 4/24/2019	DRAWN BY: DJW/MKB
DWG. NAME. STBR–4_R5	SCALE: 1:4 UNITS: in.[mm]	REV. BY: PM/JEK

Figure 295. Middle/Lower Splice Tube Assembly, Test No. STBR-4

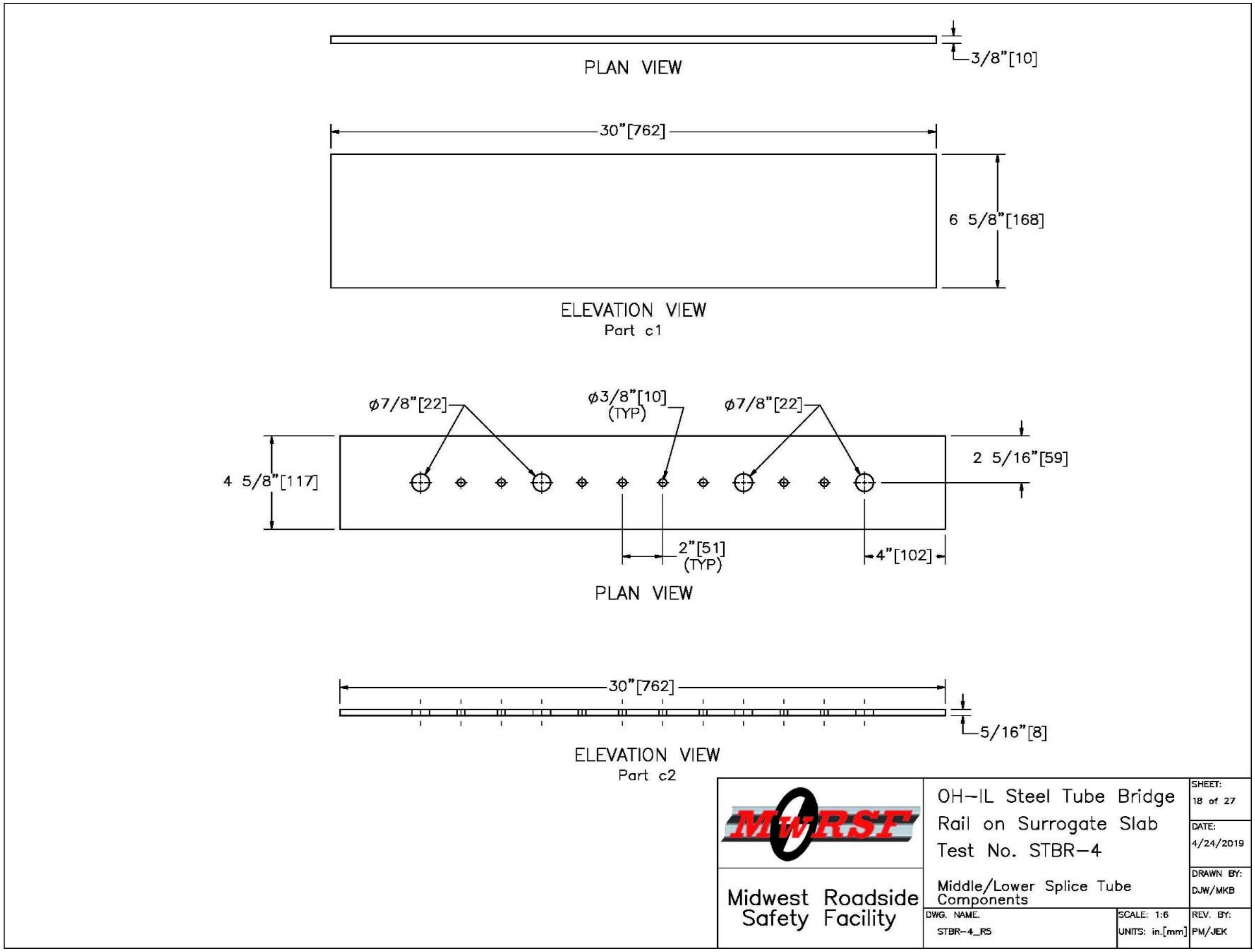


Figure 296. Middle/Lower Splice Tube Components, Test No. STBR-4

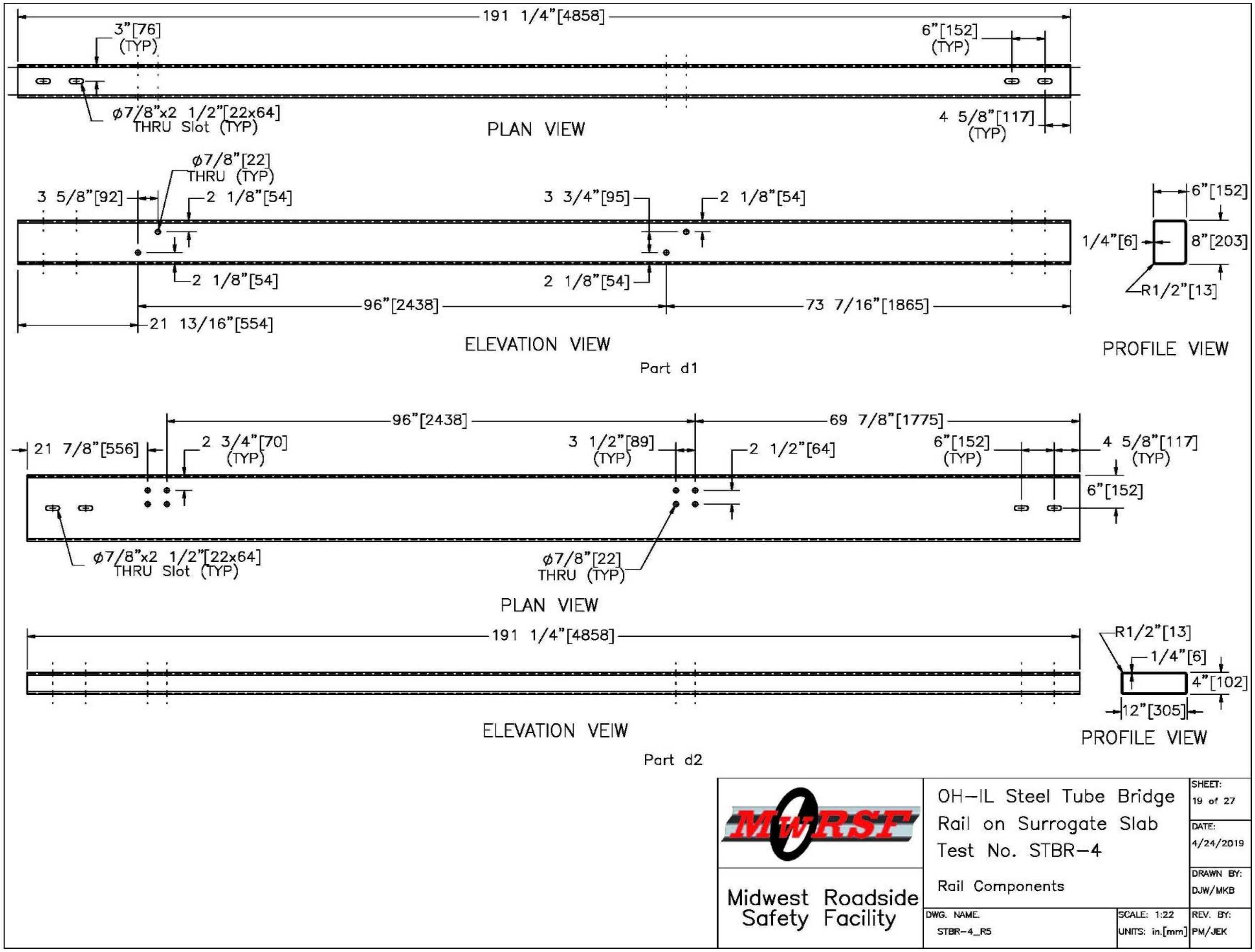
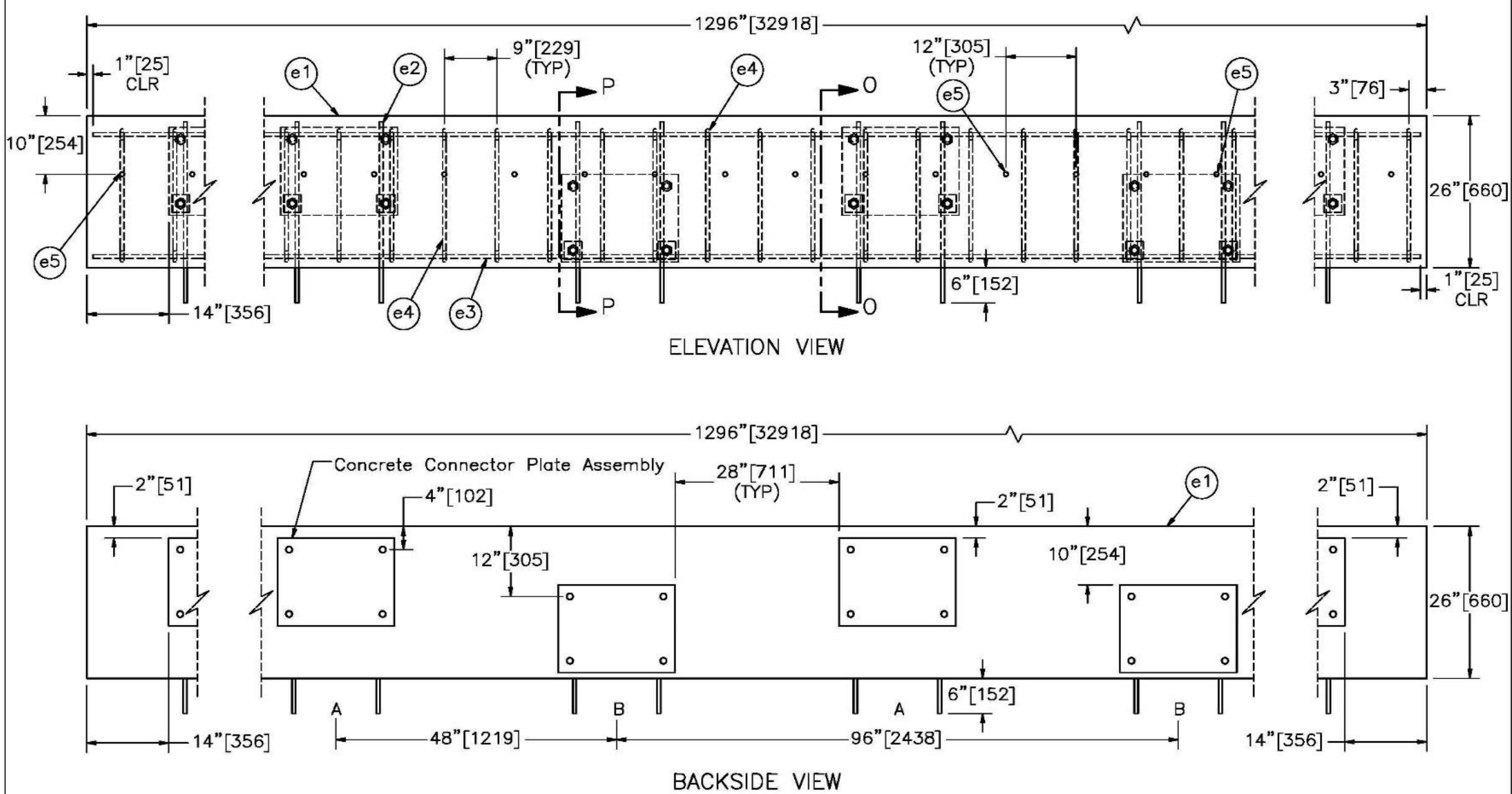


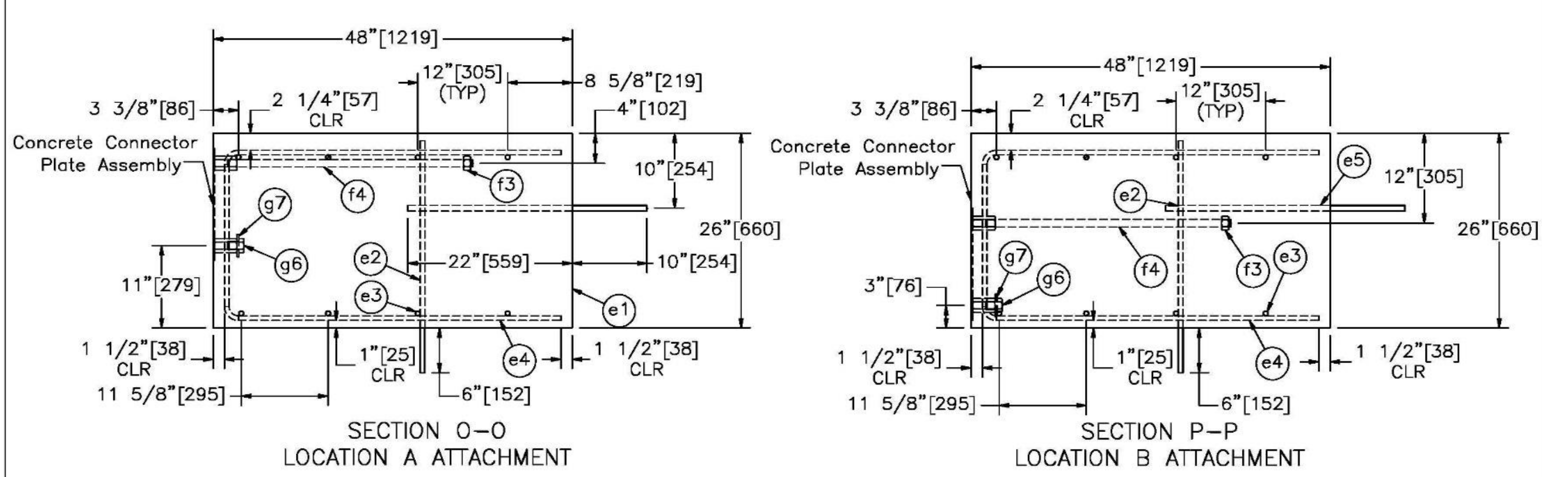
Figure 297. Rail Components, Test No. STBR-4



Notes: (1) Place vertical bar (part no. e2) driven 6" [152] into ground tangent to the threaded rods (part no. f4) at the attachment locations. For support of f4 rods during casting only.
(2) Location A used for 1100C small car tests.
(3) Location B used for 2270P pickup truck and 10000S SUT tests.
(4) Internal reinforcement not shown on backside view.

MwRSF Midwest Roadside Safety Facility	OH-IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR-4	SHEET: 20 of 27
		DATE: 4/24/2019
	Test Slab Assembly Details	DRAWN BY: DJW/MKB
	DWG. NAME. STBR-4_R5	SCALE: 1:25 UNITS: in.[mm]	REV. BY: PM/JEK

Figure 298. Test Slab Assembly Details, Test No. STBR-4

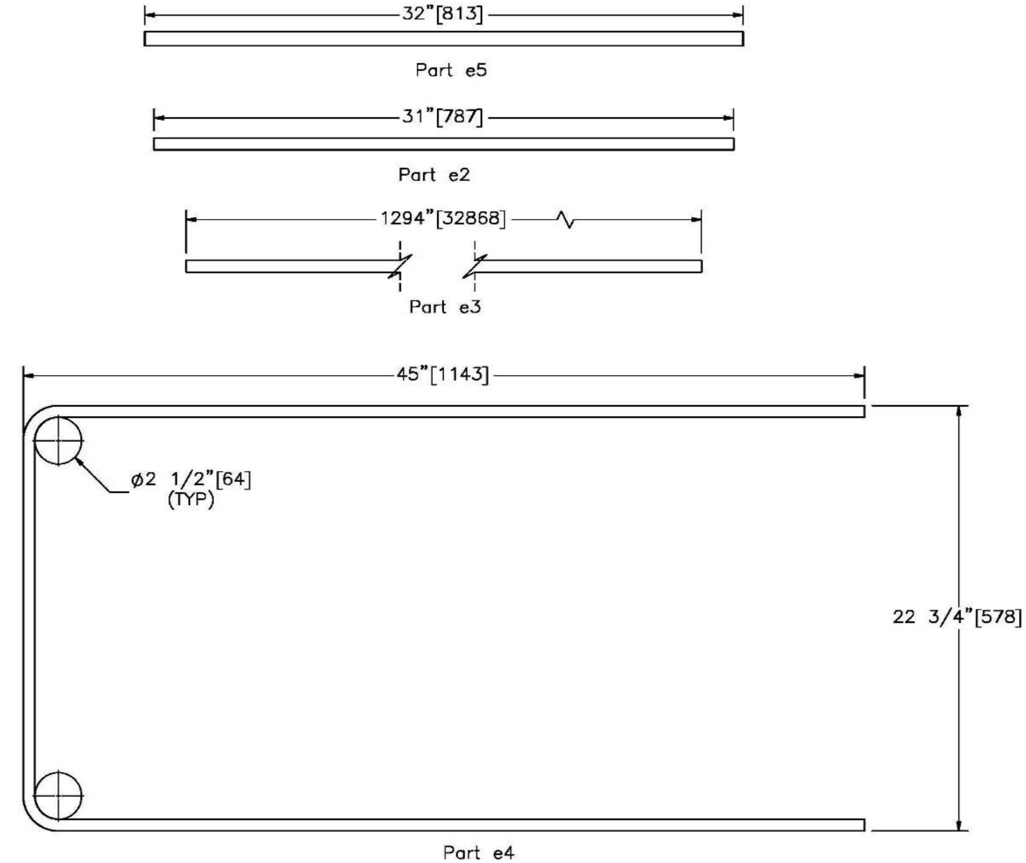


Item No.	QTY.	Description	Material Specification	Treatment Specification
–		Test Slab Assembly	–	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
–	26	Concrete Connector Plate Assembly	–	ASTM A123
–	1	Post 9 Concrete Connector Plate Assembly	–	ASTM A123
f3	54	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329
g6	54	1"–8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329
g7	52	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123
g8	1	36"x2 1/2"x3/8" [914x64x10] Plate	ASTM A572 Gr. 50	ASTM A123

Notes: (1) Location A used for 1100C small car tests.
(2) Location B used for 2270P pickup truck and 10000 SUT tests.
(3) For Post 9 in Location B, part g8 is used instead of g7. Post 9 also has a different Concrete Connector Plate Assembly, as detailed on Sheet 4.
(4) Place vertical bar (part no. e2) driven 6" [152] into ground tangent to the threaded rods (part no. f4) at the attachment locations. For support during casting.
(5) Part f4 is threaded 1 1/4" [32] into part f2 (coupling nut) on Concrete Connector Plate Assemblies.

MwRSF Midwest Roadside Safety Facility	OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4		SHEET: 21 of 27
			DATE: 4/24/2019
	Test Slab Assembly Details		DRAWN BY: DJW/MKB
	DWG. NAME. STBR–4_R5	SCALE: 1:20 UNITS: in.[mm]	REV. BY: PM/JEK

Figure 299. Test Slab Assembly Details, Test No. STBR-4



Note: (1) Minimum lap length for a #5 bar is 18" [457].

Bill of Bars					
Bar	QTY.	Size	Total Unbent Length	Material Specification	Treatment Specification
e2	54	#5	31" [787]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e3	8	#5	1294" [32868]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e4	144	#5	110 3/16" [2799]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)
e5	108	#6	32" [813]	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)

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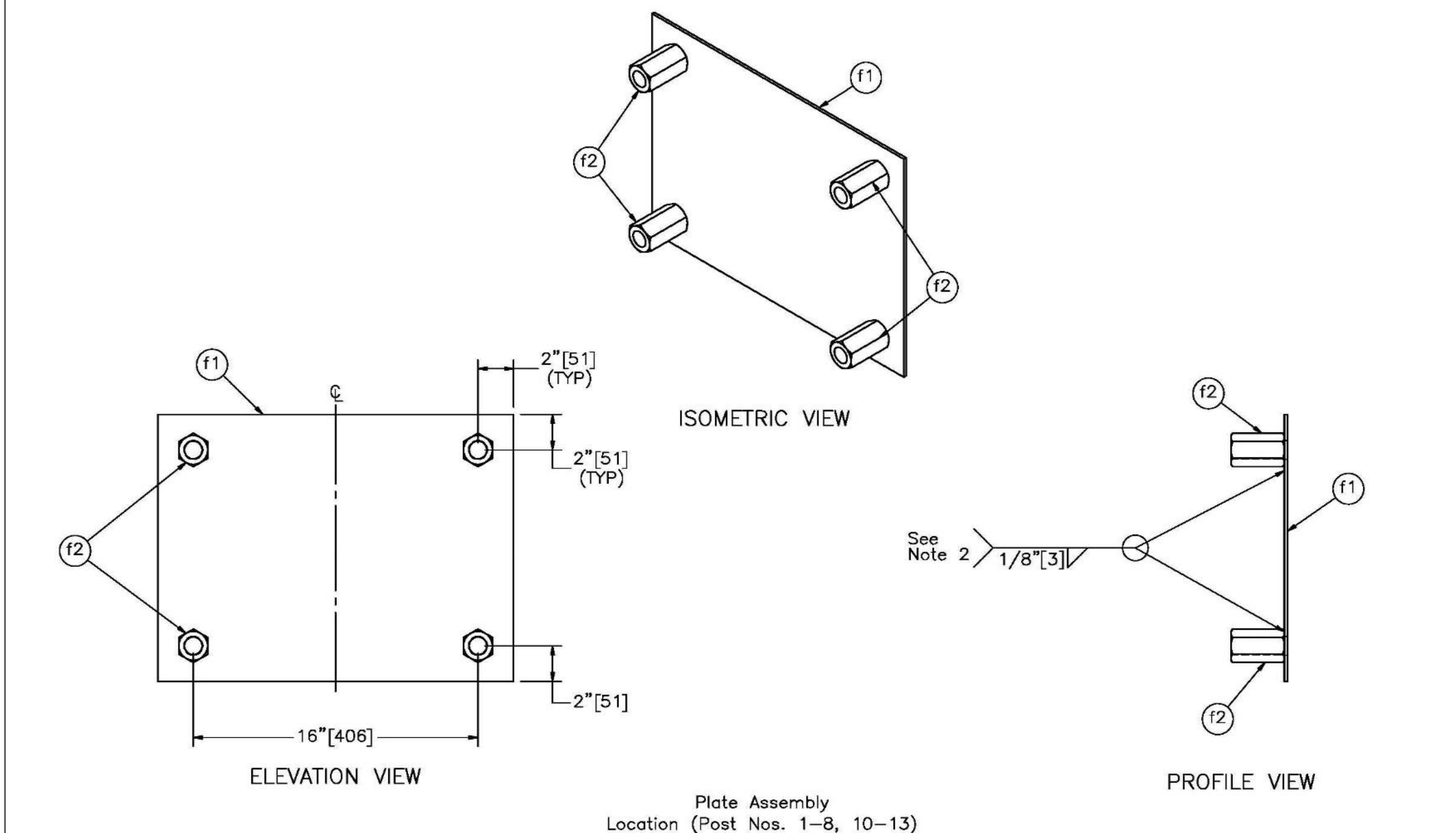
Midwest Roadside Safety Facility

OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Reinforcement Details

SHEET: 22 of 27
DATE: 4/24/2019
DRAWN BY: DJW/MKB
DWG. NAME. STBR–4_R5
SCALE: 1:8 UNITS: in.[mm]
REV. BY: PM/JEK

Figure 300. Reinforcement Details, Test No. STBR-4



Notes: (1) Connector plate assembly is symmetric along the centerline.
(2) During casting, the nuts may be held in place by extending bolts through the formwork into the nuts, or by tack welding nuts to inside face of the plate. The welds are not structurally important.

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
–	26	Concrete Connector Plate Assembly	–	ASTM A123	–
f1	1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	ASTM A123	–
f2	4	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Concrete Connector Plate Assembly Details

SHEET: 23 of 27
DATE: 4/24/2019
DRAWN BY: DJW/MKB
REV. BY: PM/JEK
DWG. NAME. STBR–4_R5
SCALE: 1:8
UNITS: in.[mm]

Figure 301. Concrete Connector Plate Assembly Details, Test No. STBR-4

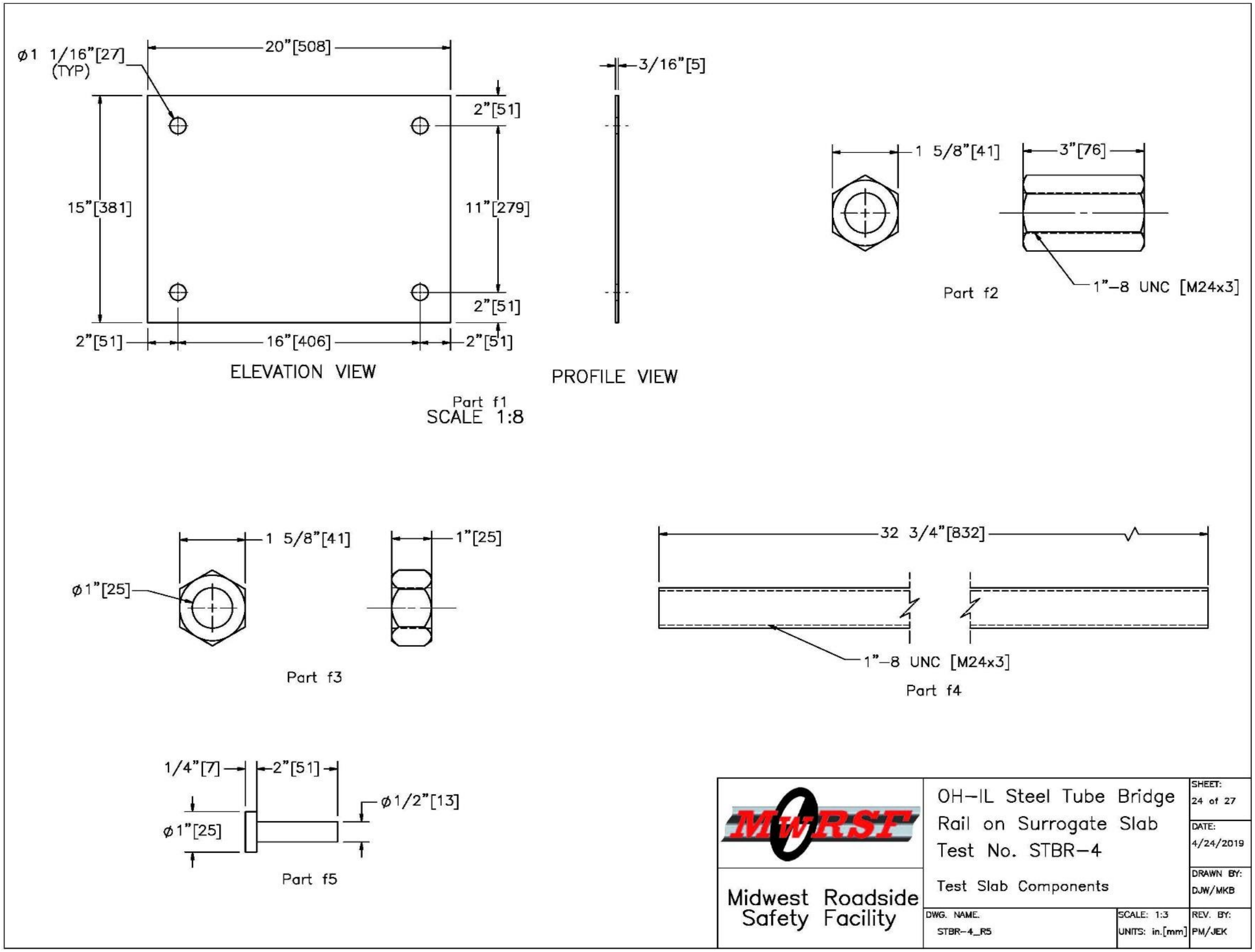

Figure 302. Concrete Connector Plate Components, Test No. STBR-4

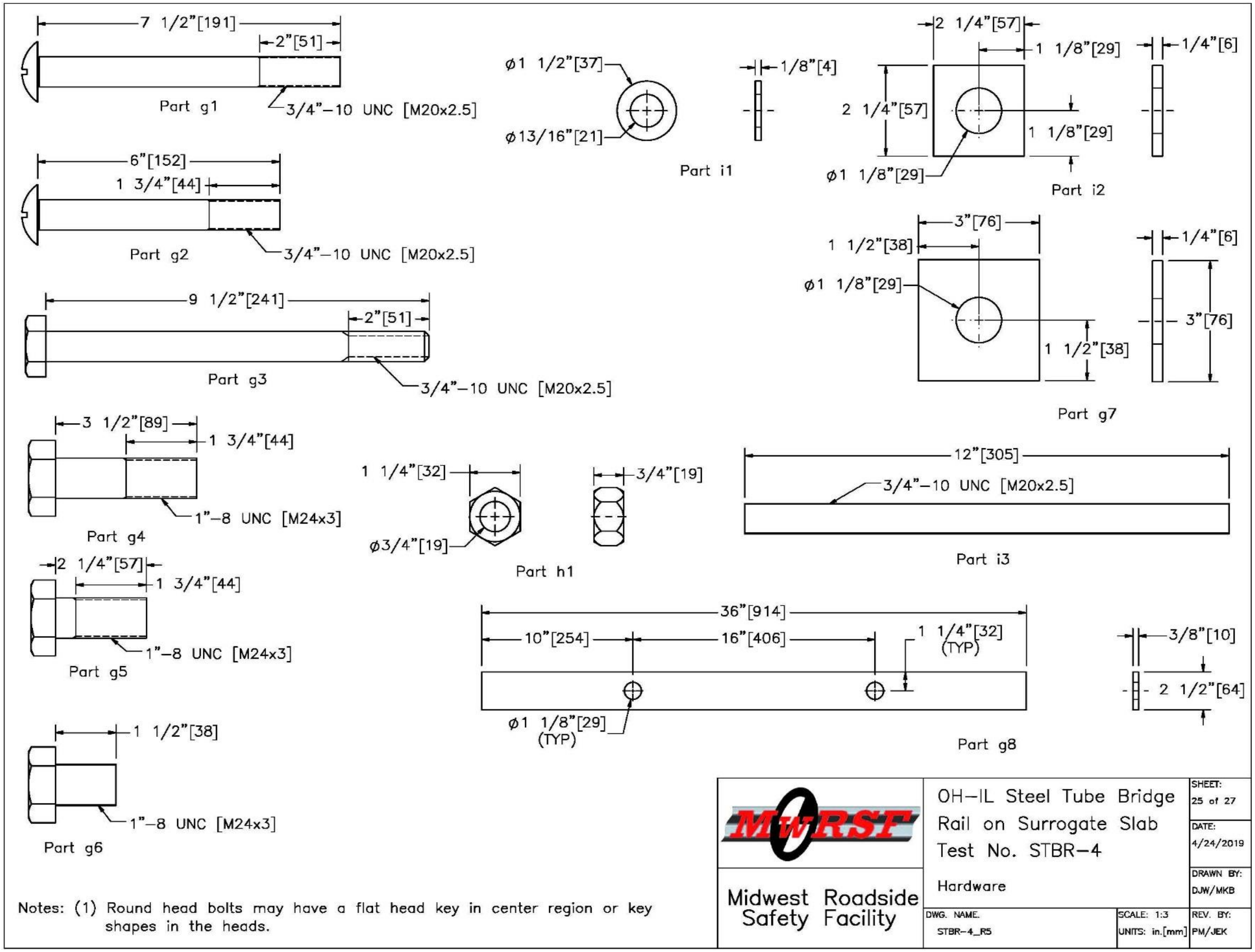

Figure 303. Hardware, Test No. STBR-4

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
a1	13	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	See Assembly	–
a2	20	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	See Assembly	–
a3	13	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	See Assembly	–
a4	52	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	See Assembly	–
a5	26	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	ASTM A123	–
a6	7	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	See Assembly	–
a7	7	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	See Assembly	–
b1	18	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	See Assembly	–
b2	18	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c1	36	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	See Assembly	–
c2	36	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	See Assembly	–
d1	20	HSS 8"x6"x1/4" [203x152x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
d2	10	HSS 12"x4"x1/4" [305x102x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	ASTM A123	–
e1	1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	–	–
e2	54	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e3	8	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e4	144	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
e5	108	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	Epoxy–Coated (ASTM A775 or ASTM A934)	–
f1	27	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	ASTM A123	–
f2	108	1"–8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	See Assembly	–
f3	106	1"–8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX24b
f4	54	1"–8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	ASTM A153 or B695 Class 55 or F2329	FRR24b
f5	4	1/2" [13] Dia. Shear Stud, 2" [51] Long	ASTM A108	–	–

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Bill of Materials

SHEET:	26 of 27
DATE:	4/24/2019
DRAWN BY:	DJW/MKB
DWG. NAME.	STBR–4_R5
SCALE:	None
UNITS:	in.[mm]
REV. BY:	PM/JEK

Figure 304. Bill of Materials, Test No. STBR-4

Item No.	QTY.	Description	Material Specification	Treatment Specification	Hardware Guide
g1	84	3/4"–10 UNC [M20x2.5], 7 1/2" [191] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g2	120	3/4"–10 UNC [M20x2.5], 6" [152] Long Round Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX20b
g3	72	3/4"–10 UNC [M20x2.5], 9 1/2" [241] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	ASTM A153 or B695 Class 55 or F2329	FBX20b
g4	52	1"–8 UNC [M24x3], 3 1/2" [89] 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
g5	52	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
g6	54	1"–8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	ASTM A153 or B695 Class 55 or F2329	FBX24b
g7	52	3"x3"x1/4" [76x76x6] Plate	ASTM A36	ASTM A123	–
g8	1	36"x2 1/2"x3/8" [914x64x10] Plate	ASTM A572 Gr. 50	ASTM A123	–
h1	304	3/4 –10 UNC [M20x2.5] Heavy Hex Nut	ASTM A563DH	ASTM A153 or B695 Class 55 or F2329	FNX20b
i1	376	3/4" [19] Dia. Hardened Flat Washer	ASTM F436	ASTM A153 or B695 Class 55 or F1136 Gr. 3 or F2329	FWC20b
i2	156	2 1/4"x2 1/4"x1/4" [57x57x6] Square Washer	ASTM A36	ASTM A123	–
i3	28	3/4"–10 UNC [M20x2.5], 12" [305] Long Threaded Rod	ASTM F1554 Gr. 36	ASTM A153 or B695 Class 55 or F2329	FRR20a

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OH–IL Steel Tube Bridge Rail on Surrogate Slab Test No. STBR–4

Bill of Materials

SHEET: 27 of 27

DATE: 4/24/2019

DRAWN BY: DJW/MKB

DWG. NAME. STBR–4_R5

SCALE: None

UNITS: in.[mm]

REV. BY: PM/JEK

Figure 305. Bill of Materials, Test No. STBR-4

Figure 306. Test Installation Photographs, Test No. STBR-4

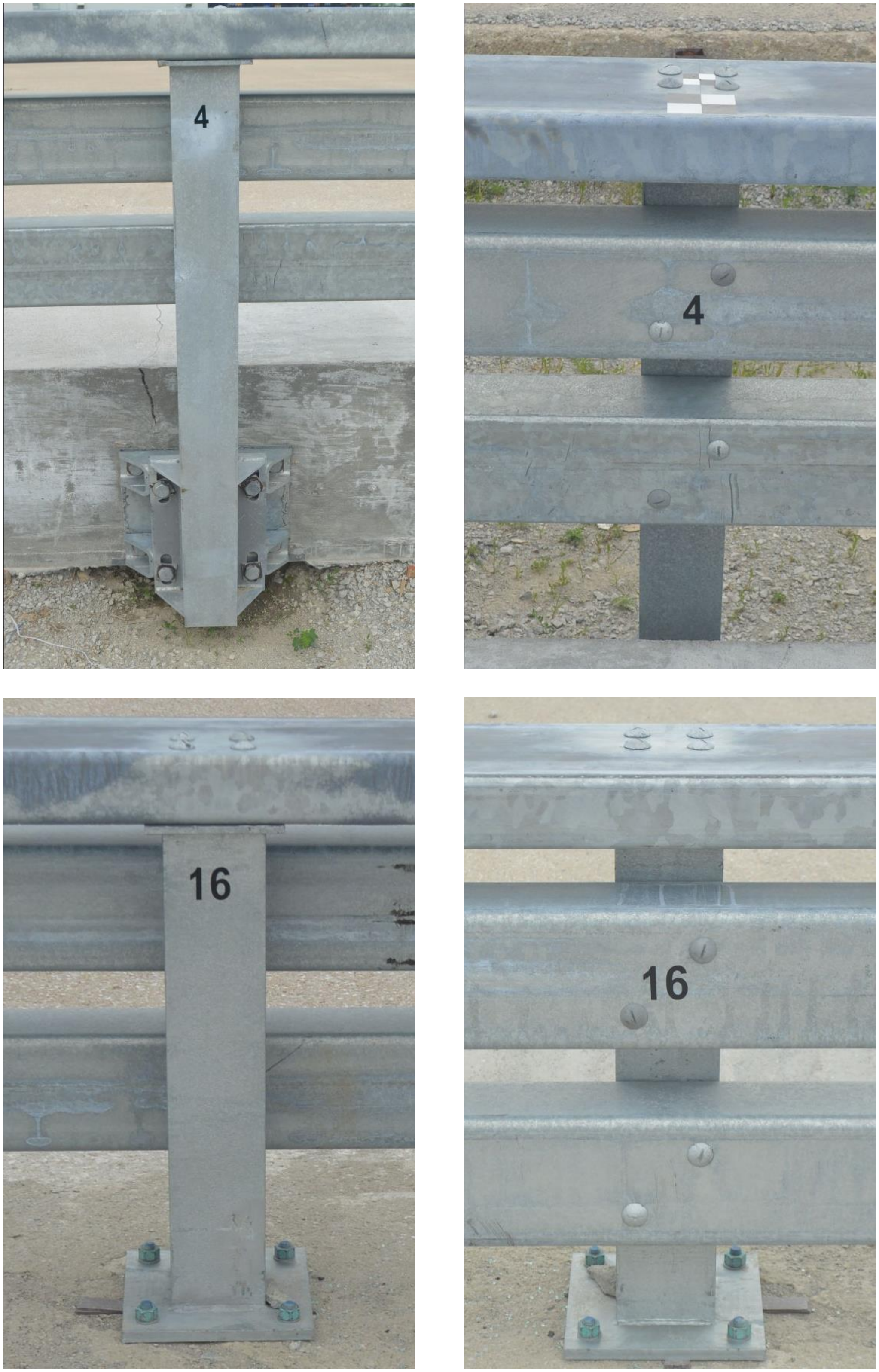


Figure 307. Test Installation Photographs, Side-Mounted and Top-Mounted Posts, Test No. STBR-4

Figure 308. Test Installation Photographs, Bridge Railing End Views, Test No. STBR-4

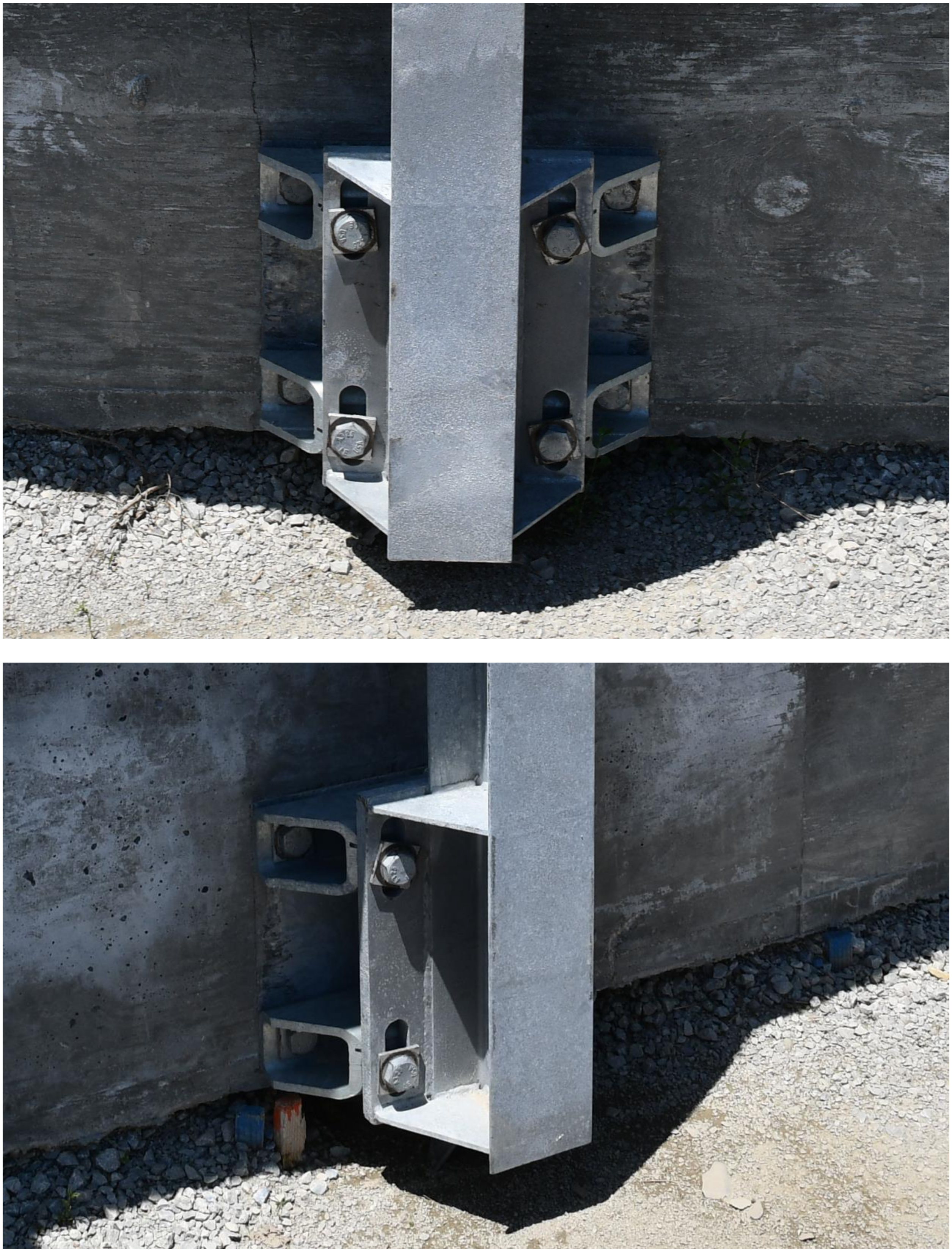


Figure 309. Side-Mounted Post-to-Deck Connections, Test No. STBR-4

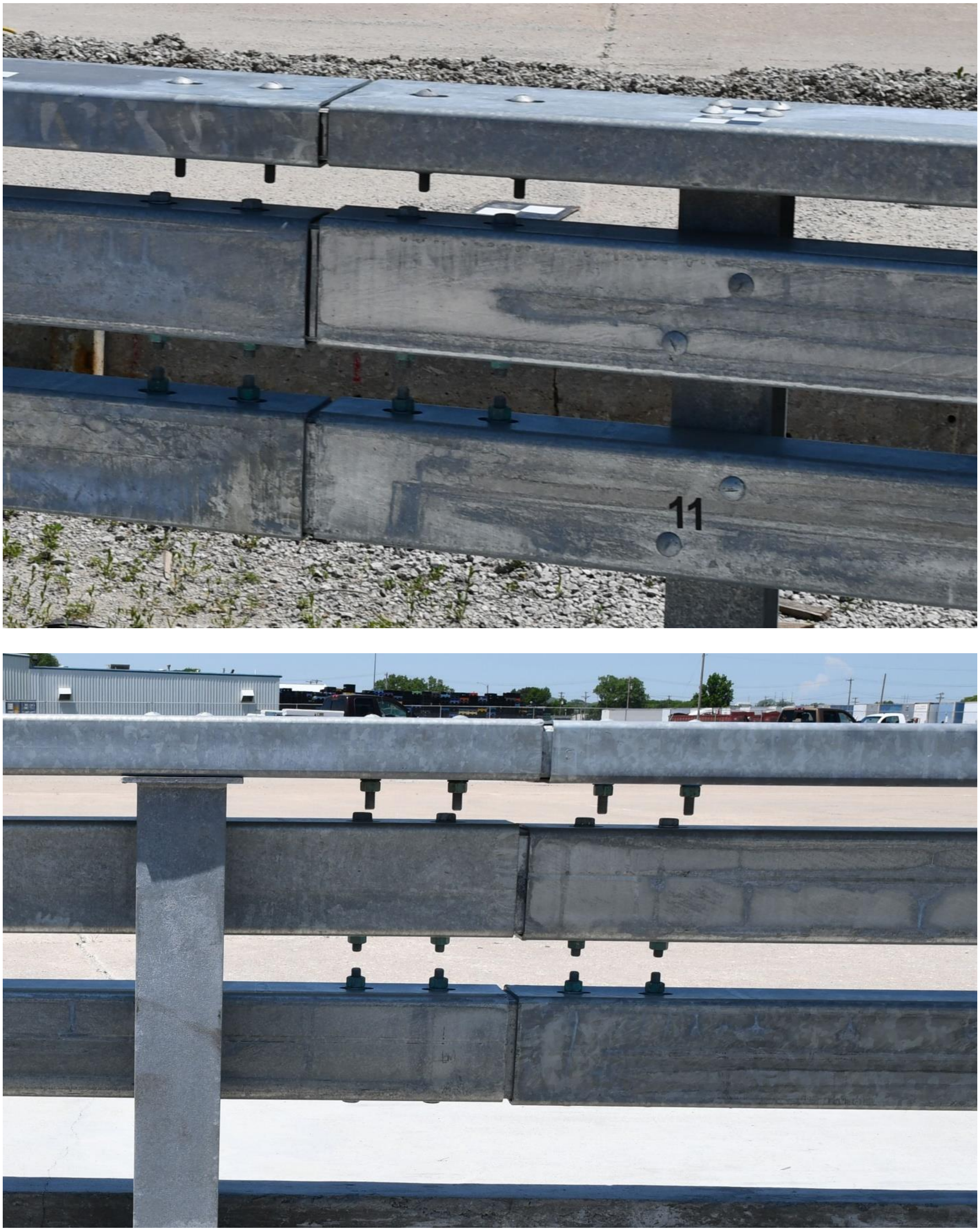


Figure 310. Splice Tubes, Test No. STBR-4

18 FULL-SCALE CRASH TEST NO. STBR-4

18.1 Weather Conditions

Test no. STBR-4 was conducted on June 6, 2019 at approximately 3:00 p.m. The weather conditions as per the National Oceanic and Atmospheric Administration (station 14939/LNK) were reported and are shown in Table 82.

Table 82. Weather Conditions, Test No. STBR-4

Temperature	82° F
Humidity	58%
Wind Speed	8 mph
Wind Direction	150° from True North
Sky Conditions	Sunny
Visibility	10 Statute Miles
Pavement Surface	Dry
Previous 3-Day Precipitation	1.37 in.
Previous 7-Day Precipitation	1.37 in.

18.2 Test Description

Initial vehicle impact was to occur 60 in (1.5 m) upstream from the splice between post nos. 6 and 7, as shown in Figure 311, which was selected as discussed in Chapter 9. In test no. STBR-4, the 22,152-lb (10,048-kg) 2007 Freightliner M2 106 SUT impacted the bridge rail at a speed of 56.4 mph (90.8 km/h) and at an angle of 14.7 degrees. The actual point of impact was 6.8 in. (173 mm) downstream from the target impact location. The vehicle came to rest 242 ft – 10 in. (74.0 m) downstream from the original impact point and laterally 22 ft – 6 in. (6.8 m) in front of the bridge rail.

The crash testing parameters are shown in Table 83. A detailed description of the sequential impact events is contained in Table 84. Sequential photographs are shown in Figures 312 and 313. Documentary photographs of the crash test are shown in Figures 314 through 318. The vehicle trajectory and final position are shown in Figure 319.

Figure 311. Impact Location, Test No. STBR-4

Table 83. Actual, Lower-Bound, and Target Crash Test Parameters, Test No. STBR-4

Test Parameter	Actual	Lower-Bound	Target
Speed	56.4 mph	53.5 mph	56.0 mph
Angle	14.7 deg.	13.5 deg.	15.0 deg.
Impact Severity	151.7 kip-ft	142.0 kip-ft	154.4 kip-ft

Table 84. Sequential Description of Impact Events, Test No. STBR-4

Time (sec):	**Event Description**
0.000	Vehicle's front bumper contacted rail between post nos. 6 and 7.
0.004	Vehicle's front bumper deformed.
0.008	Vehicle's left-front tire and left fender contacted rail.
0.010	Post no. 6 deflected backward and vehicle's left fender deformed.
0.014	Post no. 7 deflected backward.
0.018	Post no. 5 deflected backward.
0.024	Vehicle's left-front tire became airborne.
0.028	Post no. 8 deflected backward and vehicle rolled toward system.
0.030	Vehicle cab yawed away from system.
0.034	Post no. 7 bent backward.
0.050	Post no. 6 bent backward.
0.066	Vehicle's left-front door deformed.
0.080	Vehicle's left-front tire deflated and vehicle trailer yawed away from system.
0.092	Vehicle pitched upward
0.094	Vehicle's left fuel tank deformed.
0.112	Vehicle's right-front tire became airborne.
0.118	Vehicle's left window shattered.
0.148	Post no. 9 deflected backward.
0.274	Vehicle's right-rear tire became airborne.
0.282	Vehicle's left cargo box contacted rail.
0.292	Vehicle's left-rear tire contacted rail.
0.296	Vehicle was parallel to the system at a speed of 50.3 mph (80.9 km/h).
0.308	Vehicle's left-front tire regained contact with ground.
0.320	Vehicle pitched downward.
0.604	Vehicle's left-rear tire became airborne.
0.716	Vehicle's front bumper contacted ground.
0.882	Vehicle's left cargo box side contacted rail.
1.020	Vehicle's left cargo box side deformed.
1.048	Vehicle's box placard became disengaged.
1.094	Vehicle's left fender contacted ground.
2.000	Vehicle exited system at a speed of 36.0 mph (58.0 km/h).
2.356	Vehicle's left-rear tire regained contact with ground.
2.372	Vehicle's left cargo box contacted ground.

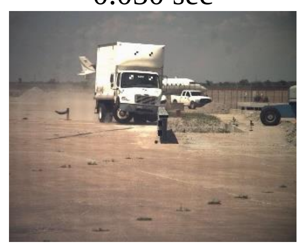
0.000 sec

0.000 sec

0.050 sec

0.050 sec

0.200 sec



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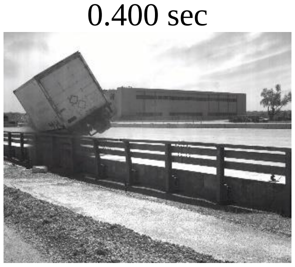

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Figure 312. Sequential Photographs, Test No. STBR-4


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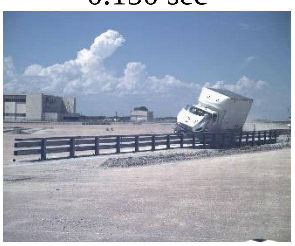
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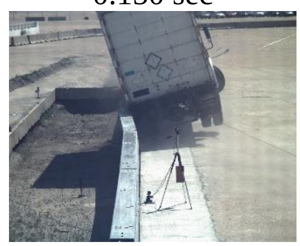
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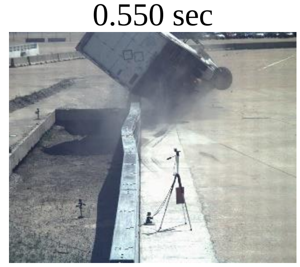
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1.100 sec

Figure 313. Additional Sequential Photographs, Test No. STBR-4

Figure 314. Documentary Photographs, Test No. STBR-4

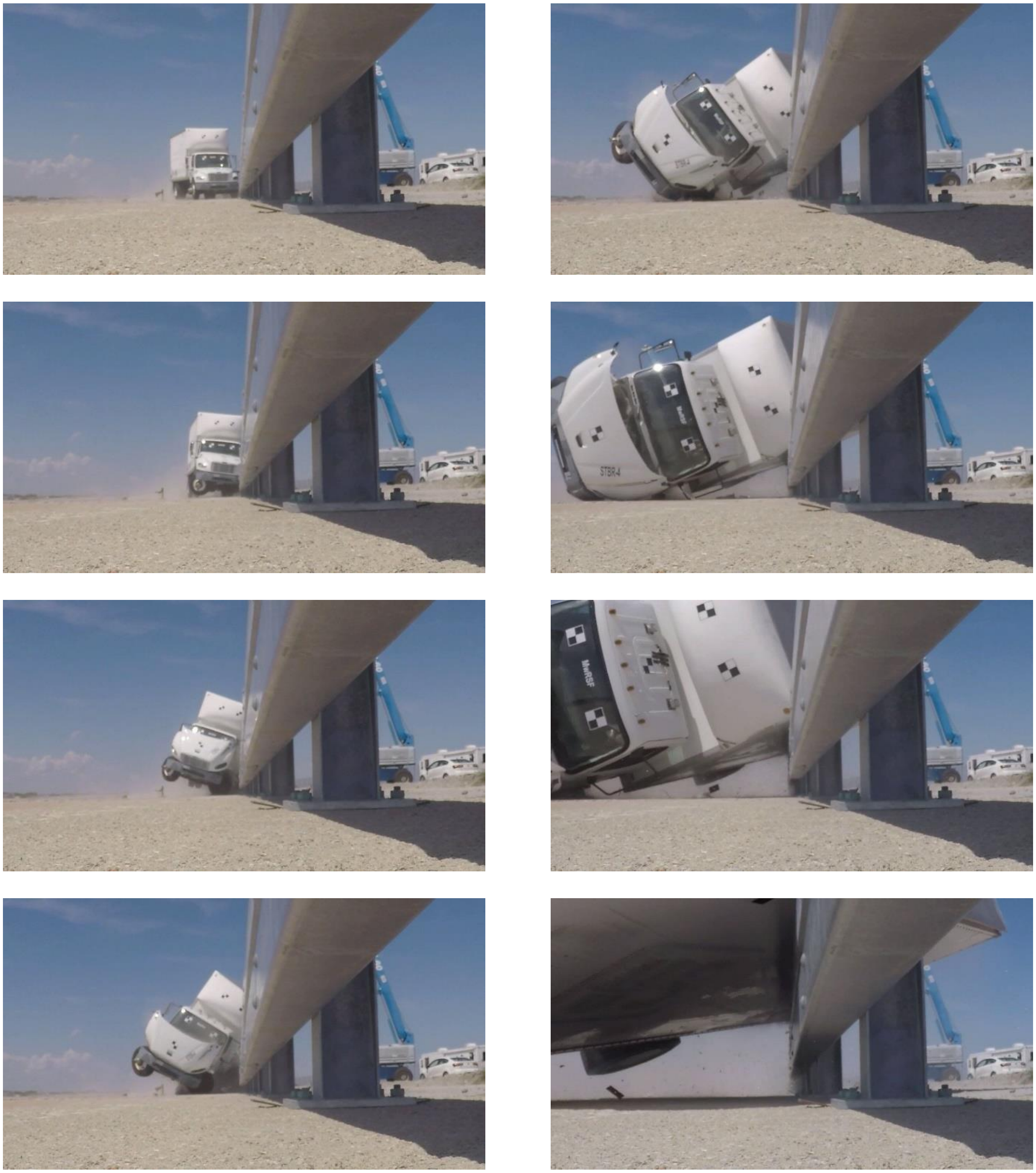

Figure 315. Documentary Photographs, Test No. STBR-4

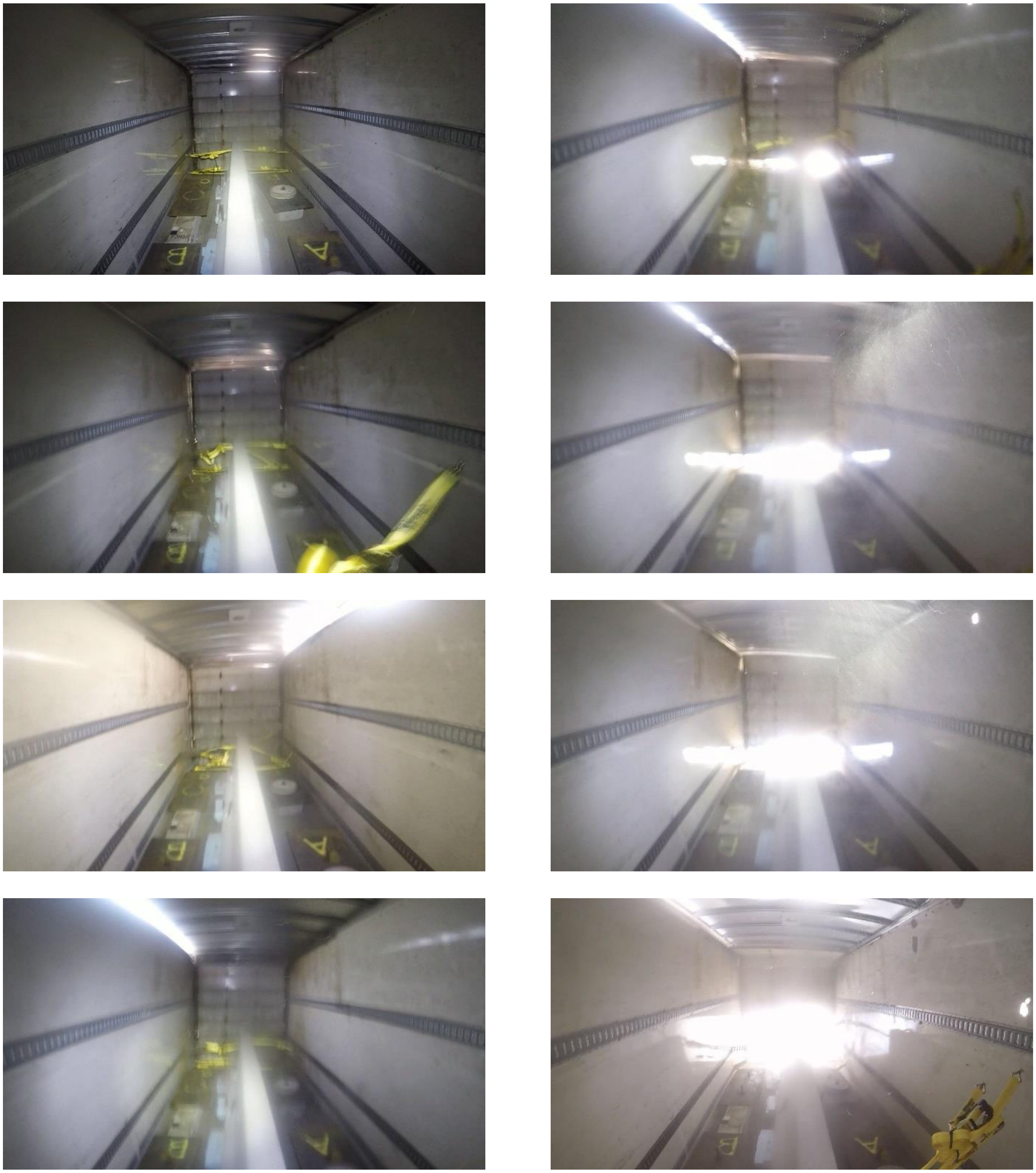

Figure 316. Documentary Photographs, Test No. STBR-4

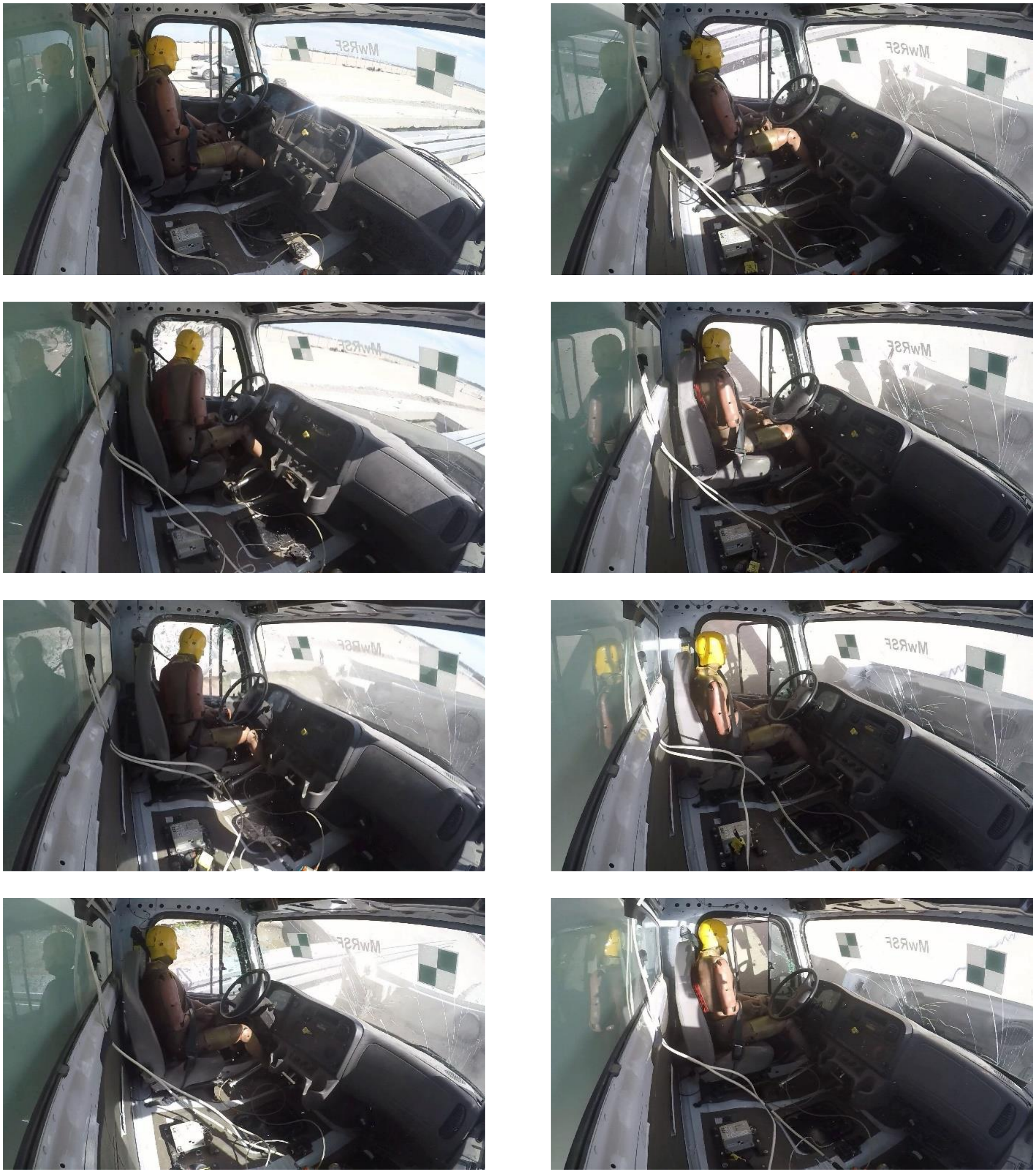

Figure 317. Documentary Photographs, Test No. STBR-4

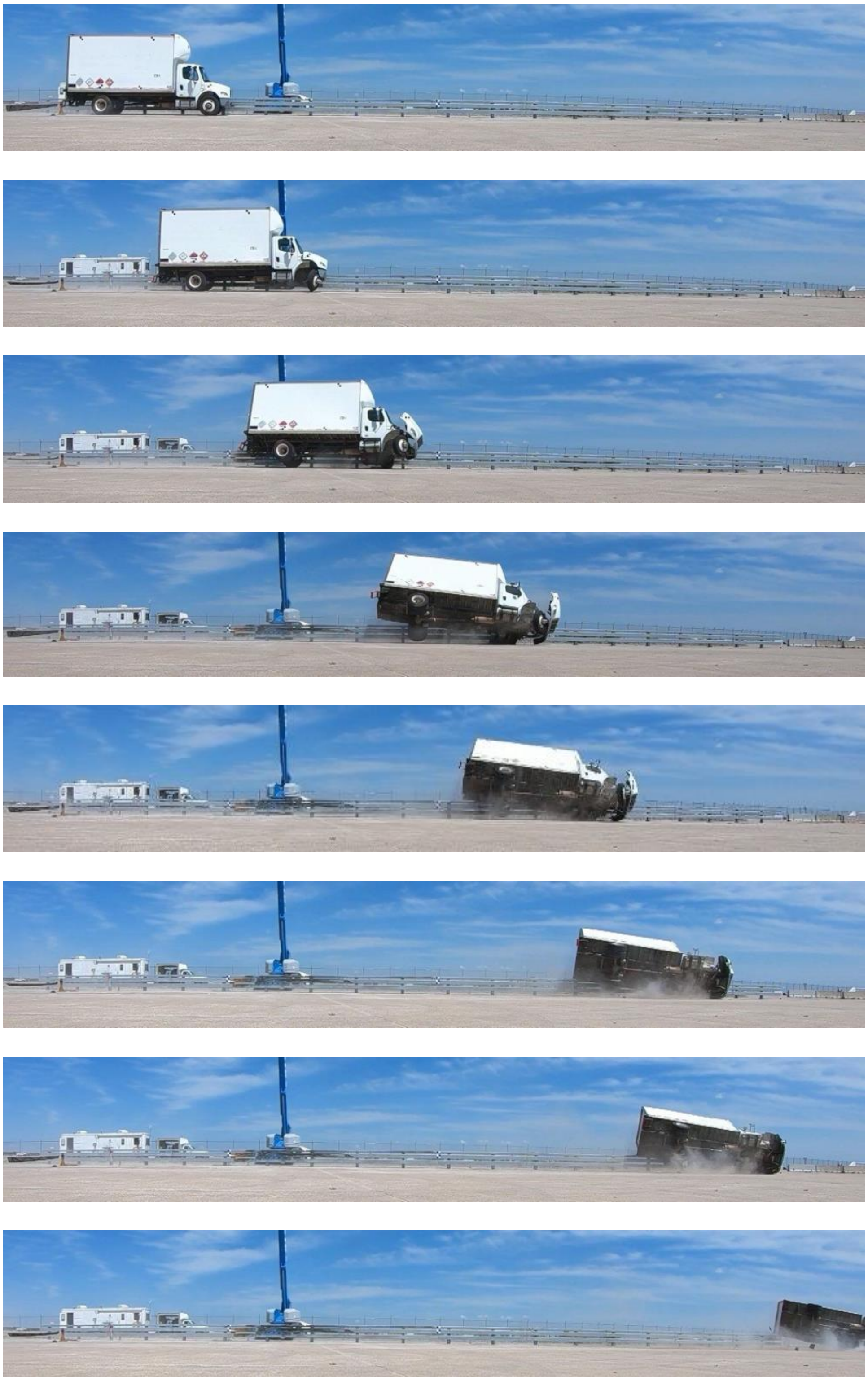

Figure 318. Documentary Photographs, Test No. STBR-4

Figure 319. Vehicle Final Position and Trajectory Marks, Test No. STBR-4

18.3 System Damage

Damage to the bridge rail was moderate, as shown in Figures 320 through 326. Note that shrinkage cracks were observed in the surrogate slab prior to the test and denoted with a black marker. System damage consisted of contact marks, scraping, denting, and gouging on the rails, plastic hinges at post nos. 6, 7, and 8 for a four-span collapse, and minimal concrete spalling between post nos. 6 and 7, as well as near the post-to-deck connection of post no. 7. The length of vehicle contact along the barrier was approximately 118 ft – 11⅝ in. (36.3 m), which spanned from 12 in. (305 mm) upstream from post no. 6 to the end of the barrier system.

Plastic hinges were found 3 in. (76 mm) above the location of the tension anchor rods for post nos. 6, 7, and 8. Contact marks were visible on the front of the top rail starting at 12 in. (305 mm) upstream from the centerline of post no. 6 and extending 64 in. (1,626 mm). Contact marks were visible on the front face of the middle rail starting at 23½ in. (597 mm) downstream from the centerline of post no. 6 and extending 259 in. (6,579 mm) downstream. Contact marks were also noted in the front face of the bottom rail starting 23½ in. (597 mm) downstream from the centerline of post no. 6 and extending 239½ in. (6,083 mm). Contact marks were again visible on the top rail beginning 51 in. (1,295 mm) upstream from the centerline of post no. 7 and extending 24 in. (610 mm) downstream, and 16½ in. (419 mm) upstream from the centerline of post no. 7 extending 193½ in. (4,915 mm) downstream. The box of the single unit truck came into contact with the top face of the top rail further downstream at 32 in. (813 mm) upstream from the centerline of post no. 12 extending for approximately 72 ft – 7 in. (22.1 m) to the end of the rail system.

A gouge was found at the center of the middle rail beginning 13½ in. (342 mm) downstream from the centerline of post no. 6 extending 22½ in. (572 mm) farther downstream, and on the bottom rail beginning 11 in. (279 mm) downstream from the centerline of post no. 6 extending for 11 in. (279 mm) downstream. Beginning 24 in. (610 mm) downstream from the centerline of post no. 6, a dent was found measuring roughly 6 in. (152 mm) in height and ¼ in. (6.4 mm) deep that extended 21 in. (533 mm) downstream. Another dent was visible beginning 43½ in. (1,105 mm) upstream from the centerline of post no. 7, measuring 7¾ in. (197 mm) in height, ⅛ in. (3 mm) deep, and extending 13 in. (330 mm) downstream. On the top face of the top rail, scraping was observed beginning 27½ in. (699 mm) downstream from the centerline of post no. 7, extending 175 in. (4.4 m) downstream.

Post no. 5 bent backward beginning 9 in. (229 mm) below the top of the post. Approximately 40½ in. (1,029 mm) below the top of post no. 6, the post bent backward and began to twist in the counterclockwise direction, and galvanization flaking was visible. The front flanges of post no. 6 bent outward at the middle and lower rail-to-post connections, and all bolts in the top rail-to-post connection were loosened. Post no. 7 bent backward and began to rotate clockwise with galvanization flaking visible, all located approximately 40½ in. (1,029 mm) below the top of the post. Again, the front flanges of post no. 7 bent outward at the middle and lower rail-to-post connections, and all bolts in the top rail-to-post connection were loosened. The upstream bolt in the lower rail-to-post connection was also loosened on post no. 7. Backward bending and galvanization flaking were also evident approximately 40½ in. (1,029 mm) below the top of post no. 8.

Figure 320. System Damage, Rail Span Between Posts Nos. 6 and 7, Test No. STBR-4

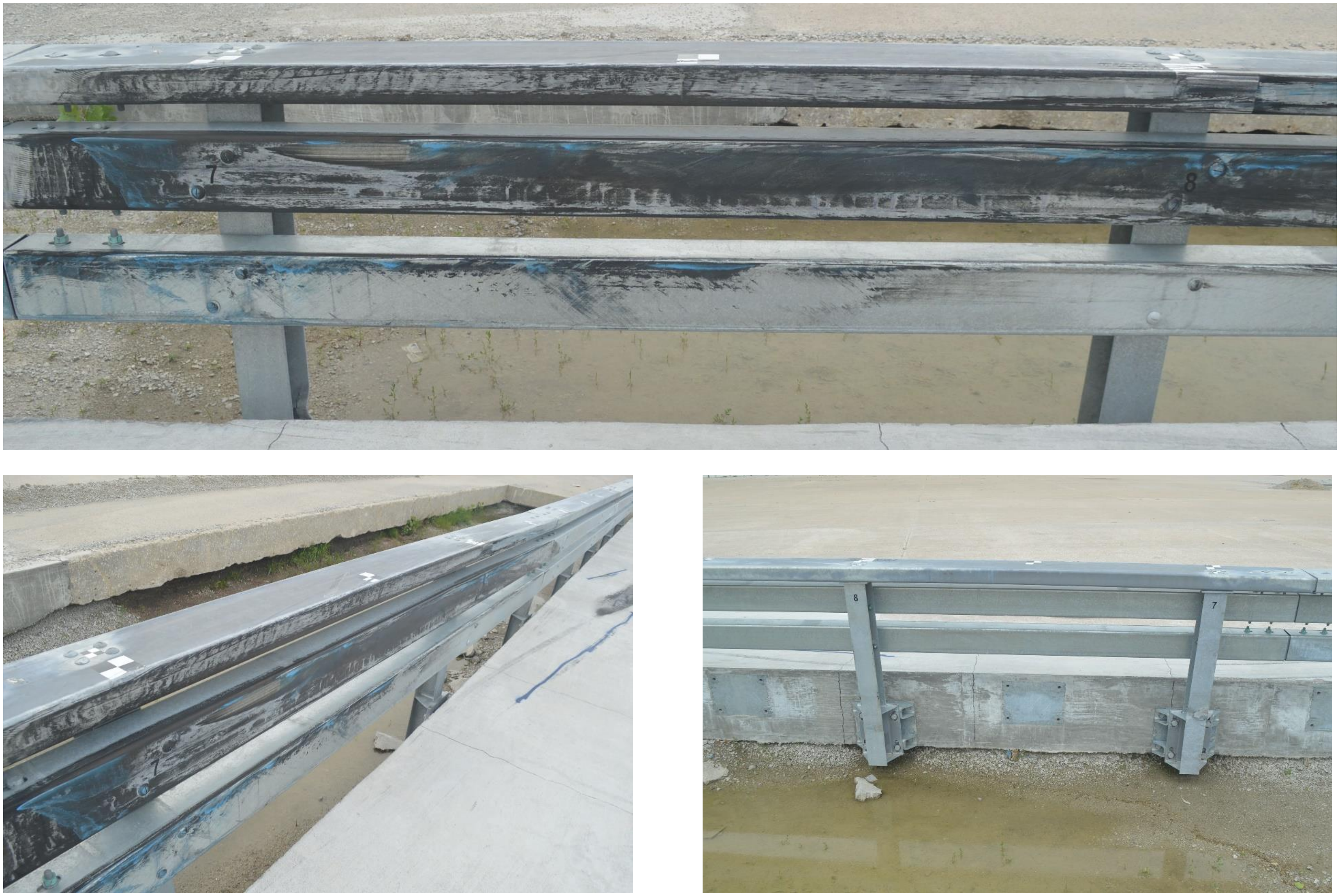

Figure 321. System Damage, Rail Span Between Posts Nos. 7 and 8, Test No. STBR-4

Figure 322. System Damage, Rail Span Between Posts Nos. 8 and 9, Test No. STBR-4

Figure 323. System Damage, Post No. 5, Test No. STBR-4

Figure 324. System Damage, Post No. 6, Test No. STBR-4

Figure 325. System Damage, Post No. 7, Test No. STBR-4

Figure 326. System Damage, Post No. 8, Test No. STBR-4

Concrete deck spalling was visible on the top corner starting 37 in. (940 mm) downstream from the centerline of post no. 6 and extending 59 in. (1,499 mm) downstream to the centerline of post no.7. Concrete spalling was also visible on the downstream top corner of the post-to-deck connection of post no. 7. This spall measured 2½ in. (64 mm) across diagonally.

The maximum lateral permanent set of the barrier system was 7.3 in. (185 mm). The maximum lateral dynamic barrier deflection was 7.9 in. (201 mm) at the top rail between post nos. 6 and 7, as determined from high-speed digital video analysis. The working width of the system was found to be 87.7 in. (2,228 mm), also determined from high-speed digital video analysis. A schematic of the permanent set deflection, dynamic deflection, and working width is shown in Figure 327.

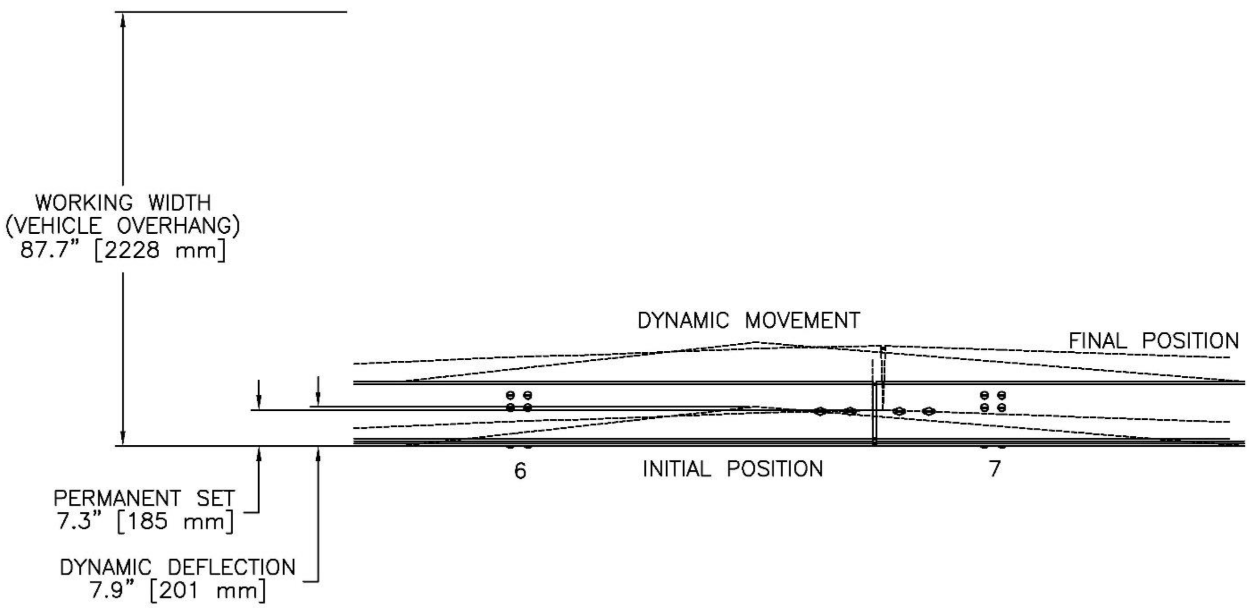

Figure 327. Permanent Set Deflection, Dynamic Deflection, and Working Width, Test No. STBR-4

18.4 Vehicle Damage

The damage to the vehicle was moderate, as shown in Figures 328 through 333. The maximum occupant compartment deformations are listed in Table 85 along with the deformation limits established in MASH 2016 for various areas of the occupant compartment. MASH 2016 defined intrusion or deformation as an occupant compartment being deformed and reduced in size with no observed penetration. Note that none of the established MASH 2016 deformation limits were violated. The A-pillar (lateral), B-pillar (lateral), side front panel (in front of the A-pillar), and side door (above and below seat) all deformed slightly outward. Outward deformations are not considered crush toward the occupant, are denoted as negative numbers in Table 85, and are not evaluated by MASH 2016 criteria. Complete occupant compartment and vehicle deformations and the corresponding locations are provided in Appendix F.

Majority of the damage was concentrated on the left front-corner of the vehicle where the impact had occurred. The front bumper cover bent inward toward the engine compartment. The hood was scraped and cracked on the left side. The crack was located under the left headlight, and the scrape started behind the left headlight and traveled to the back of the wheel well. The left headlight was also disengaged from its compartment. The left-front inner wheel well was disengaged and shredded. The bottom of the left door was bent into itself. The stairs and fuel tank on the left side of the vehicle were bent and crushed in toward the middle of the truck. The front bubble on the box of the truck was scraped and cracked on the left side. The left side of the box was also scraped and cracked, with the scrape stretching from the top front corner of the box to the lower back corner. The ballast in the box of the truck shifted toward the left side, bending the thread bolts holding each component of the ballast in place. The windshield was also disengaged, and the left side window was shattered due to contact with the head of the simulated occupant. In general, the front axle and frame were twisted and bent on the left side of the truck. The front-left lower leaf in the leaf spring pack was disengaged from the rest of the pack on the undercarriage of the truck. The back-left leaf spring keeper was also bent, causing it to open. The lower control arm that connects the wheels together was bent, as well as the front control arm on the left side. The front axle U-bolts that were part of the steering control arm were broken. The shear plates and U-bolts that connected the box to the frame were all slightly bent to the left. The left-side floor pan was crushed inward into the cab.

Figure 328. Vehicle Damage, Front and Rear Views, Test No. STBR-4

Figure 329. Vehicle Damage, Right and Left Views, Test No. STBR-4

Figure 330. Vehicle Damage, Right Side, Test No. STBR-4

Figure 331. Vehicle Damage, Left Side, Test No. STBR-4

Figure 332. Vehicle Damage, Left Side Interior and Vehicle Undercarriage, Test No. STBR-4

Right Front

Left Front

Figure 333. Vehicle Damage, Left-Front and Shear Plate Damage Views, Test No. STBR-4

Table 85. Maximum Occupant Compartment Intrusions by Location

LOCATION	MAXIMUM INTRUSION in. (mm)	MASH 2016 ALLOWABLE INTRUSION in. (mm)
Wheel Well & Toe Pan	4.5 (114)	≤ 9 (229)
Floor Pan & Transmission Tunnel	1.2 (30)	≤ 12 (305)
A-Pillar	0.4 (10)	≤ 5 (127)
A-Pillar (Lateral)	-0.4 (-10)	≤ 3 (76)
B-Pillar	0.4 (10)	≤ 5 (127)
B-Pillar (Lateral)	-0.3 (-8)	N/A[2]
Side Front Panel (in Front of A-Pillar)	-1.3 (-33)	≤ 12 (305)
Side Door (Above Seat)	-0.7 (-18)	≤ 9 (229)
Side Door (Below Seat)	-0.6 (-15)	N/A[2]
Roof	0.2 (5)	≤ 4 (102)
Windshield	0.0 (0)	≤ 3 (76)
Side Window	Shattered due to contact with head of simulated occupant	No shattering resulting from contact with structural member of test article
Dash	0.7 (18)	N/A[1]

Note: Negative values denote outward deformation
N/A[1] – No MASH 2016 criteria exist for this location
N/A[2] – MASH 2016 criteria are not applicable when deformation is outward

18.5 Occupant Risk

Occupant risk values are not required evaluation criteria for test designation no. 4-12. However, the occupant risk values were calculated with the same procedure as used for the 1100C and 2270P vehicles in order to make comparisons. The calculated occupant impact velocities (OIVs) and maximum 0.010-sec average occupant ridedown accelerations (ORAs) in both the longitudinal and lateral directions, as determined from the accelerometer data, are shown in Table 86. The calculated THIV, PHD, and ASI values are also shown in Table 86. The recorded data from the accelerometers and the rate transducers are shown graphically in Appendix J. Note, the SLICE-1 unit was designated as the primary unit during this test as it was mounted closer to the c.g. of the vehicle. The SLICE-2 unit was mounted in the vehicle's cab. The data from the DTS unit was not used in the occupant risk calculations due to the unit's distance from the vehicle's c.g.

Table 86. Summary of OIV, ORA, THIV, PHD, and ASI Values, Test No. STBR-4

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

18.6 10,000S Peak Lateral Impact Force Calculation

The longitudinal and lateral vehicle accelerations, as measured at the vehicle's c.g., were also processed using a SAE CFC-60 filter and a 50-msec moving average. The 50-msec moving average vehicle accelerations were then combined with the uncoupled yaw angle versus time data in order to estimate the vehicular loading applied to the barrier system. From the data analysis, the perpendicular impact forces were determined for the bridge rail, as shown in Figures 334 and 335. The maximum perpendicular (i.e., lateral) load imparted to the barrier was 106.4 kips (473 kN) and 110.3 kips (491 kN), as determined by the SLICE-1 (primary) unit and DTS, respectively.

18.7 Discussion

The analysis of the test results for test no. STBR-4 showed that the system adequately contained and redirected the 10000S vehicle with controlled lateral displacements of the barrier. A summary of the test results and sequential photographs are shown in Figure 336. Detached elements, fragments, or other debris from the test article did not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or work-zone personnel. Deformations of, or intrusions into, the occupant compartment that could have caused serious injury did not occur. The test vehicle contained and redirected with the box riding along the top rail of the system, and although the vehicle rolled onto its left side, it did so of the traffic side of the bridge rail, which is acceptable. Vehicle roll, pitch, and yaw angular displacements, as shown in Appendix J, were deemed acceptable as they did not adversely influence occupant risk. Therefore, test no. STBR-4 was determined to be acceptable according to the MASH 2016 safety performance criteria for test designation no. 4-12.

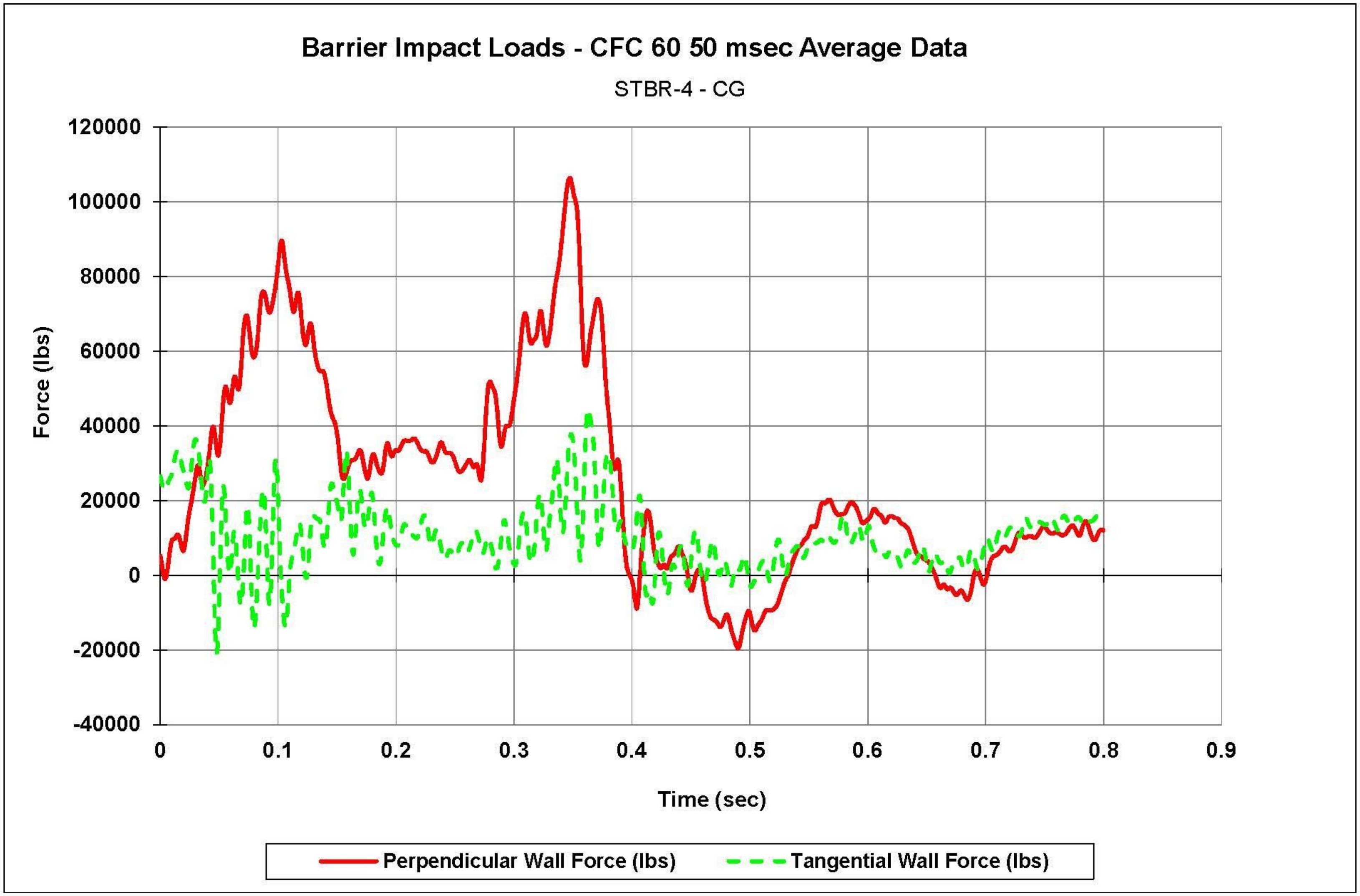

Figure 334. Perpendicular and Tangential Forces Imparted to the Barrier System (SLICE-1), Test No. STBR-4

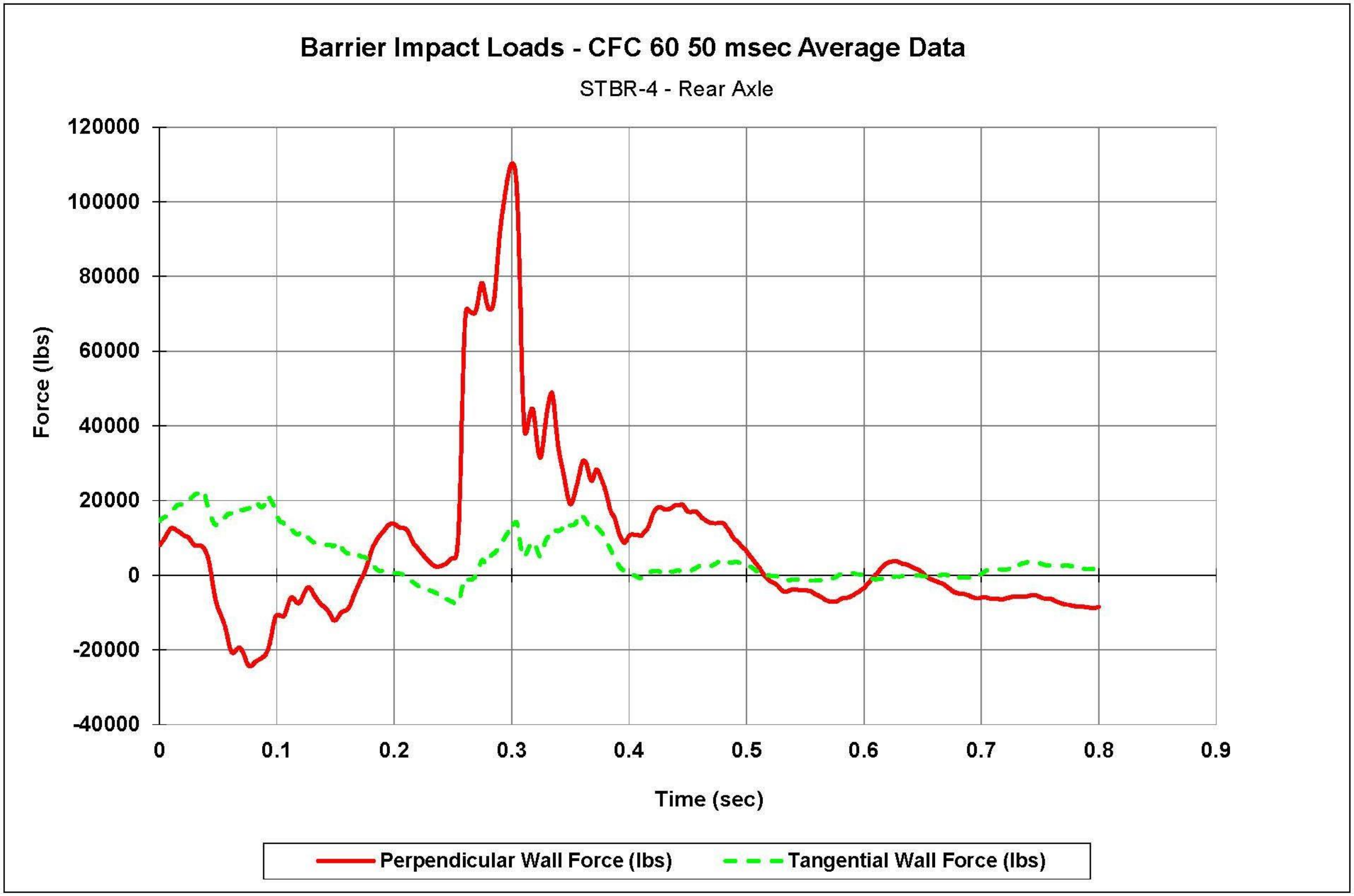

Figure 335. Perpendicular and Tangential Forces Imparted to the Barrier System (DTS), Test No. STBR-4

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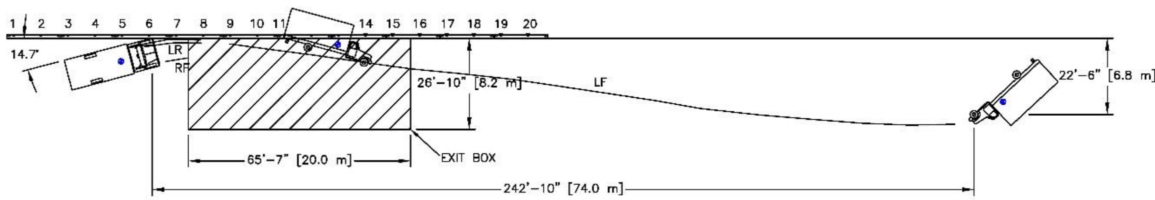
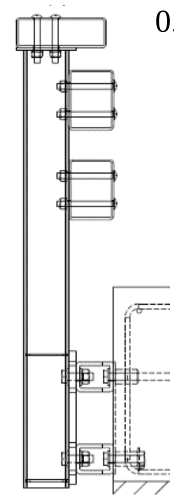




- Test Agency .... MwRSF
- Test Number STBR-4
- Date 6/6/2019
- MASH 2016 Test Designation No. 4-12
- Test Article .... Steel, Side-Mounted, Beam-and-Post, Bridge Rail
- Total Length .... 159 ft – 11¼ in. (48.7 m)
- Key Component – Top Rail
  - Length .... 191¼ in. (4,858 mm)
  - Width .... 12 in. (305 mm)
  - Depth .... 4 in. (102 mm)
- Key Component - Post
  - Length .... 58½ in. (1,486 mm)
  - Width .... 6 in. (152 mm)
  - Spacing .... 8 ft (2.4 m)
- Soil Type .... N/A
- Vehicle Make /Model .... 2007 Freightliner M2 106
  - Curb .... 13,884 lb (6,298 kg)
  - Test Inertial .... 22,152 lb (10,048 kg)
  - Gross Static .... 22,314 lb (10,121 kg)
- Impact Conditions
  - Speed .... 56.4 mph (90.8 km/h)
  - Angle .... 14.7 deg.
  - Impact Location .... 53.2 in (1.4 m) US from the splice between post nos. 6 and 7
- Impact Severity 151.7 kip-ft (205.7 kJ) > 142.0 kip-ft (192.5 kJ) limit from MASH 2016
- Exit Conditions
  - Speed .... 36.0 mph (58.0 km/h)
  - Angle .... N/A
- Exit Box Criterion .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 242 ft – 10 in. (74.0 m) DS from impact
  22 ft – 6 in. (6.8 m) laterally in front
- Vehicle Damage .... Moderate
  - VDS [56] 11-LFQ-6
  - CDC [57] 11-LFAW-6
  - Maximum Interior Deformation .... 4.5 in. (114 mm)
- Test Article Damage .... Moderate
- Maximum Test Article Deflections
  - Permanent Set .... 7.3 in. (185 mm)
  - Dynamic .... 7.9 in. (201 mm)
  - Working Width .... 87.7 in. (2,228 mm)
- Transducer Data

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

Figure 336. Summary of Test Results and Sequential Photographs, Test No. STBR-4

19 FULL-SCALE CRASH TESTING DISCUSSION

For test no. STBR-1 (MASH 2016 test designation no. 4-12), the SUT impacted the system with an impact severity of 133.2 kip-ft (180.6 kJ), which was below the allowable limit of 142.0 kip-ft (192.5 kJ) according to MASH 2016. Although test no. STBR-1 was not acceptable for determining MASH 2016 crashworthiness, the existing crash test was used to access progress toward compliance while preparations were underway to rerun the 10,000S crash test. For an impact severity of 93.8% of the lower-bound, MASH 2016 TL-4 impact condition, the bridge rail performed successfully.

The primary concerns associated with MASH 2016 test designation no. 4-12 were vehicular containment, stability, and override, as well as peak lateral impact loading to the bridge rail and deck. Note that peak lateral impact loading to the structural deck systems was evaluated in the dynamic bogie testing program. As such, vehicle containment, stability, and override were evaluated with test no. STBR-1. A minimum barrier height of 36 in. (914 mm) was utilized for the new bridge rail based on research performed on a successfully crash-tested, single-slope, concrete barrier with a height equal to 36 in. (914 mm) using a MASH 2016 SUT [38].

At the time of this research, only one steel, side-mounted, beam-and-post, bridge rail was successfully crash-tested under the MASH TL-4 safety performance criteria, which consisted of the California ST-70, Side-Mounted, Bridge Rail [20]. The overall height of the bridge rail was 42 in. (1,067 mm), as measured from the concrete deck surface to the top of upper rail. The California ST-70, Side-Mounted, Bridge Rail utilized four rails that were supported by vertical posts. The California ST-70 weighed 152.9 lb/ft (21.1 kg/m), while the new bridge rail developed in this project only weighed approximately 107.4 lb/ft (14.8 kg/m).

During testing with passenger vehicles, no bridge rail elements contacted and shattered the side windows of the small car and pickup truck vehicles, which was largely attributed to the 1-in. (25-mm) top rail setback behind the front faces of the middle and bottom rails. For test designation no. 4-10, the bridge rail had a vertical clear opening of 12 in. (305 mm) below the bottom rail. During test no. STBR-3, the front wheel of the small car snagged on the upstream front flange of post no. 7. However, the vehicle was contained and redirected, and the occupant ridedown accelerations met the MASH 2016 limits. The post-to-rail connections and rail-to-rail connections performed in an acceptable manner without bolt tear-out during all four full-scale crash tests.

The maximum lateral impact force imparted to the system in test no. STBR-1, based on the primary accelerometer system, was determined to be 96.1 kips (427.5 kN) at a time of 0.085 seconds. Later, the vehicle started to yaw toward the bridge rail, and a second impact occurred when the rear of the vehicle contacted the bridge rail, as shown in Figure 337.

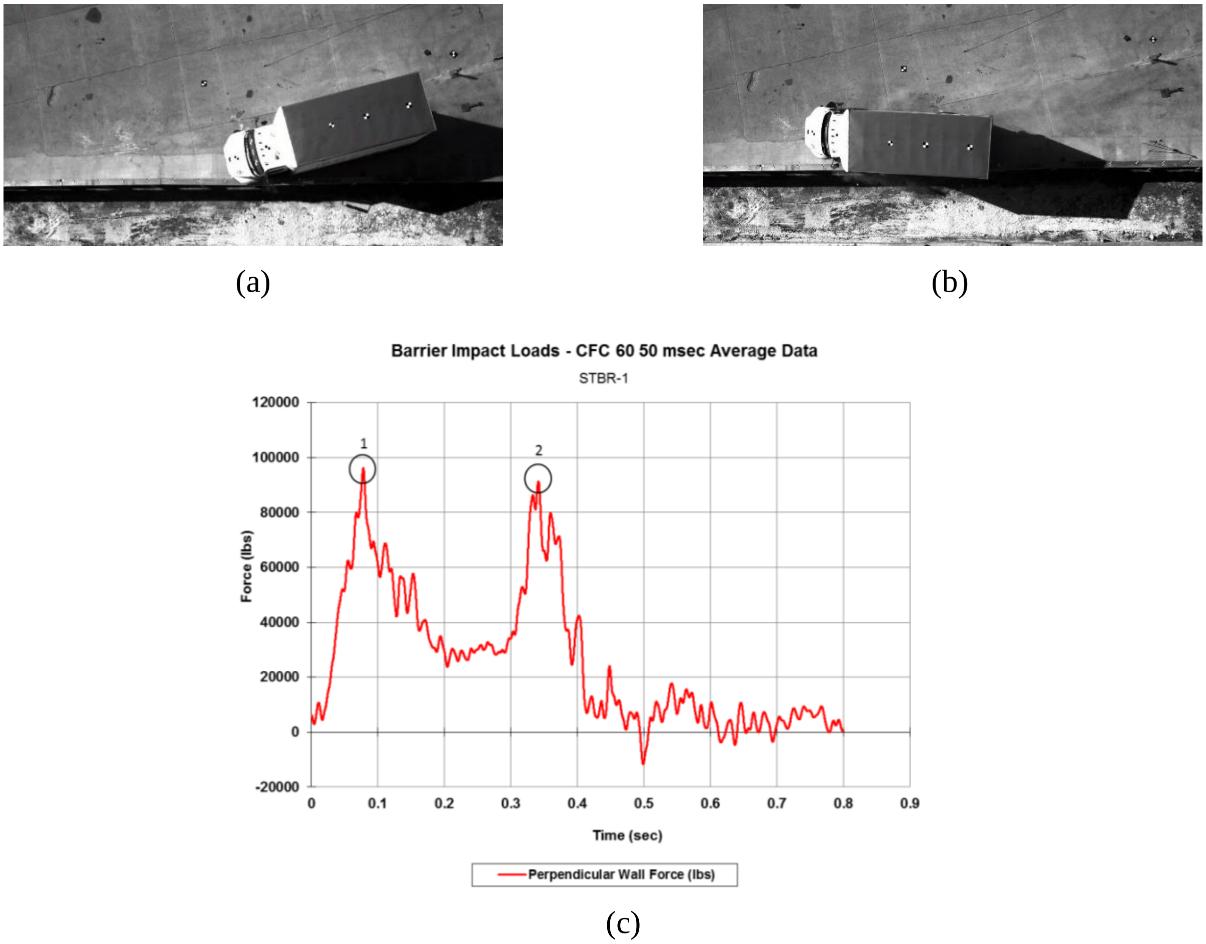


Figure 337. (a) Front-End Impact, (b) Rear-End Impact, and (c) Perpendicular Wall Impact Forces, Test No. STBR-1

For SUT impact scenarios, the lateral barrier resistance when considering all three of the rails was calculated as 81.5 kips (362.5 kN) for a five-span collapse when considering a DMF equal to 1.5 applied on the posts. However, after considering a DMF equal to 1.0 based on component testing results and after reducing the plastic section modulus of the three rails to consider the final post-to-deck connection attachments, the lateral barrier capacity of the bridge rail decreased to 65.8 kips (292.7 kN) for a five-span collapse. In test no. STBR-1, only one post plastically deformed, which occurred 5 in. (127 mm) above the location of the tension anchor rods.

Permanent deformation data was obtained from surveying with GPS equipment and high-speed digital video analysis of the SUT crash test to determine the actual number of spans deflected. The permanent set of the side-mounted posts and midspans of the bridge rail is shown in Figure 338. For test no. STBR-1, the permanent set of the bridge rail was 2.7 in. (68.6 mm), as stated in Chapter 12 and based on GPS data. Also as stated in Chapter 12, a plastic hinge at post no. 7 was visually observed after the crash test, forming a two-span collapse mechanism. However, based on Figure 338, both GPS and high-speed digital video analysis curves indicated that four spans (i.e., from post no. 5 to post no. 9) plastically deformed.

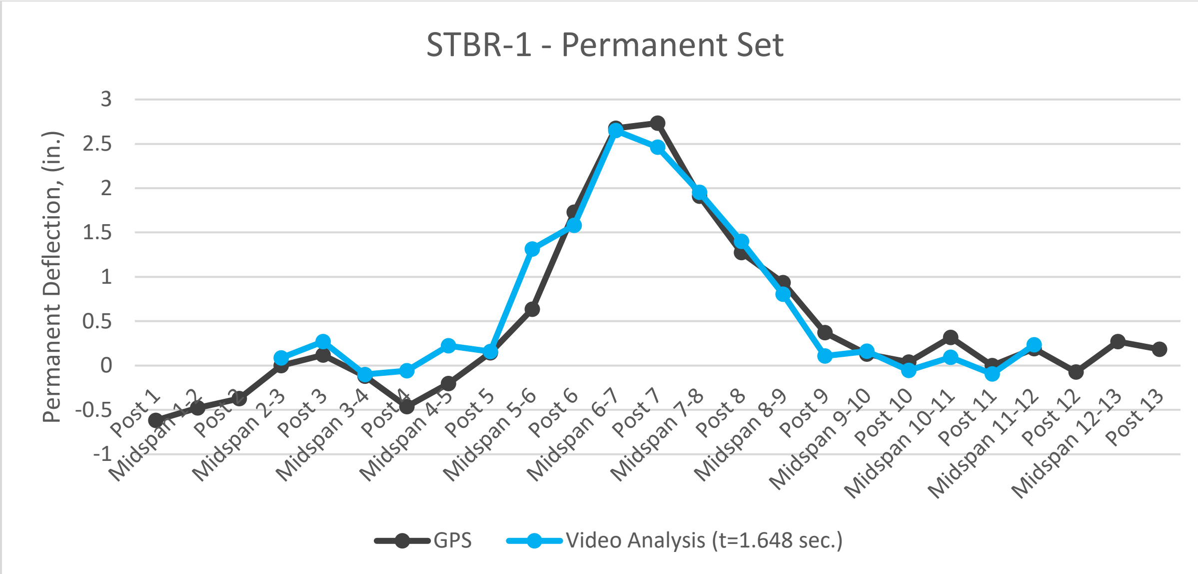


Figure 338. Permanent Set, Test No. STBR-1

For test no. STBR-1, the dynamic deflections for the side-mounted posts and midspans in between were obtained from high-speed digital video analysis. Two time-steps were considered, specifically, the time when peak loading was observed based on Figure 337 and when the maximum dynamic deflection occurred. As shown in Figure 339, the maximum visible dynamic deflection was approximately 4.3 in. (109 mm). With gaps in the data due to the box blocking the deflecting rail, there may have been greater dynamic deflection than observed. Moreover, both curves indicated that four spans (i.e., from post no. 5 to post no. 9) deformed. Again, some data points were not visible as the SUT rolled and leaned on top of the upper rail of the system.

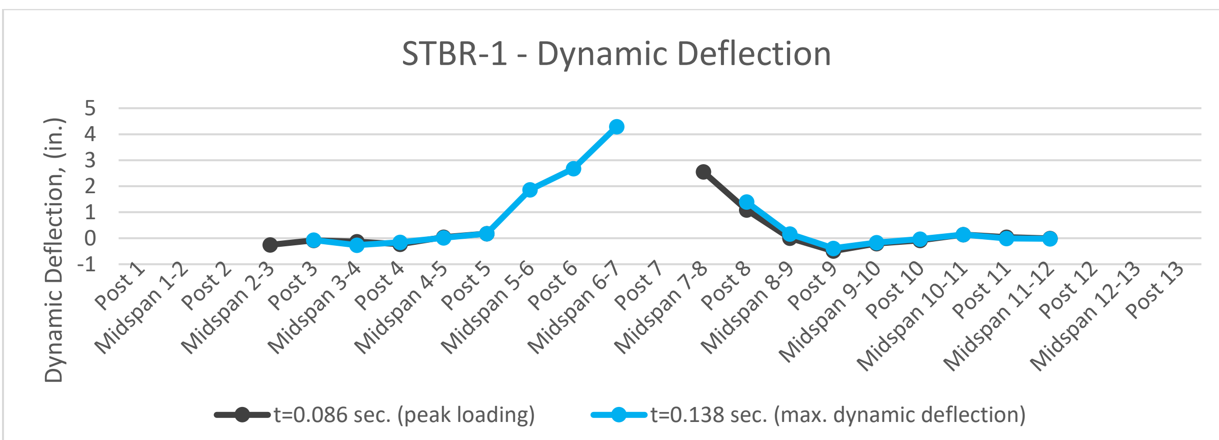


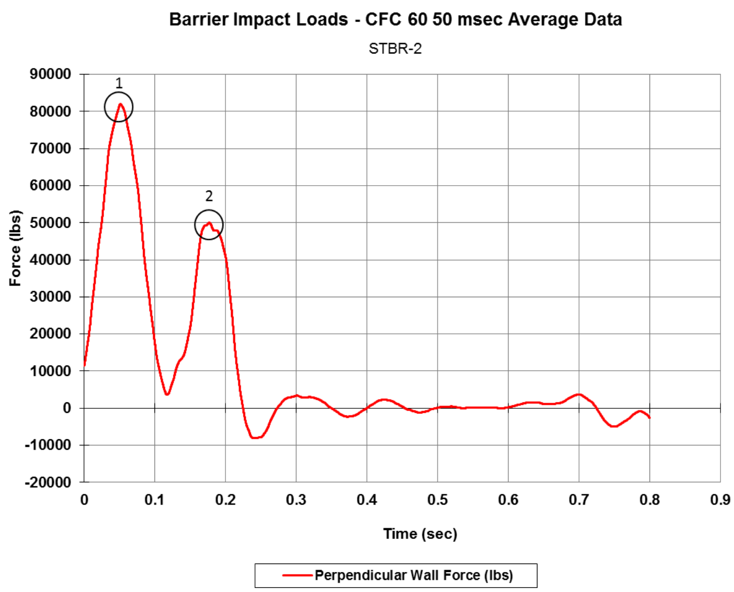
Figure 339. Visible Dynamic Deflections, Test No. STBR-1

Inelastic analysis calculations were made while considering a DMF equal to 1.0, a four-span collapse based on permanent and dynamic deflections, and having plastic hinges 5 in. (127 mm) above the tension anchor rods. This analysis revealed a modified lateral barrier resistance equal to 73.9 kips (328.7 kN).

For test no. STBR-2, the maximum lateral impact force imparted to the bridge rail was determined to be 82.0 kips (364.8 kN) at a time of 0.05 seconds. Later, the vehicle started to yaw toward the bridge rail and a second impact occurred when the rear of the vehicle contacted the bridge rail, as shown in Figure 340.

(a)

(b)



(c)

Figure 340. (a) Front-End Impact, (b) Rear-End Impact, and (c) Perpendicular Wall Impact Forces, Test No. STBR-2

For pickup truck impact scenarios, the lateral barrier resistances considering a DMF equal to 1.5 were 67.1 kips (298.5 kN) for a three-span collapse when only considering the lower two rails and 107.2 kips (476.9 kN) for a five-span collapse when considering all three rails. After considering a DMF equal to 1.0 based on component testing results and after reducing the plastic section modulus of the three rails to consider the final post-to-deck connection attachments, the lateral barrier capacities decreased to 48.9 kips (217.5 kN) with a three-span collapse when only considering the lower two rails and 75.8 kips (337 kN) with a five-span collapse when considering all three rails. In test no. STBR-2, two posts developed plastic hinges 4 in. (102 mm) above the location of the tension anchor rods.

For the pickup truck crash test (test no. STBR-2), the permanent set of the side-mounted posts and midspans were also obtained from surveying with GPS equipment and high-speed digital video analysis, as shown in Figure 341. The permanent set of the bridge rail was 3.5 in. (89 mm), as stated in Chapter 14 and based on GPS data. As stated in Chapter 14, plastic hinges at post nos. 8 and 9 for a three-span collapse were observed after the crash test. Based on Figure 341, both GPS and high-speed digital video analysis curves indicated that five spans (i.e., from post no. 6 to post no. 11) plastically deformed.

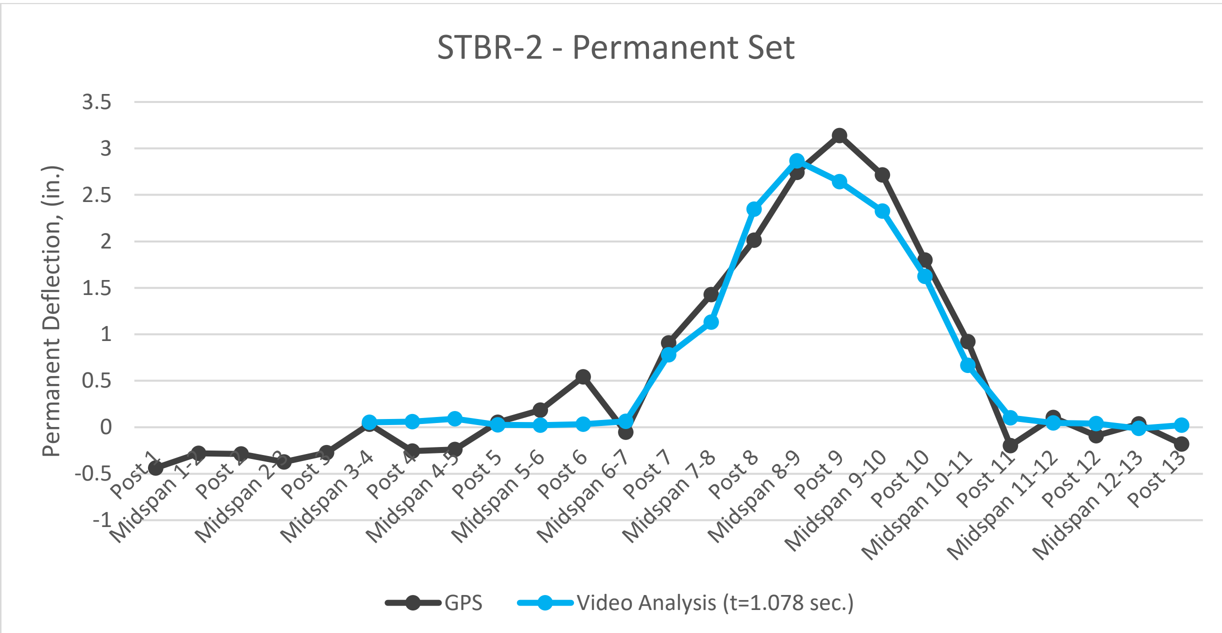


Figure 341. Permanent Set, Test No. STBR-2

For test no. STBR-2, the dynamic deflections for the side-mounted posts and midspans were also obtained for the pickup truck crash test from high-speed digital video analysis. Two time-steps were considered, specifically, the time when peak loading was observed based on Figure 340 and when the maximum dynamic deflection occurred. As shown in Figure 342, the maximum dynamic deflection was approximately 7.0 in. (178 mm). Moreover, both curves indicated that five spans (i.e., from post no. 6 to post no. 11) deformed. It should be noted that some data points were not visible as the pickup truck rolled and leaned on top of the upper rail of the system.

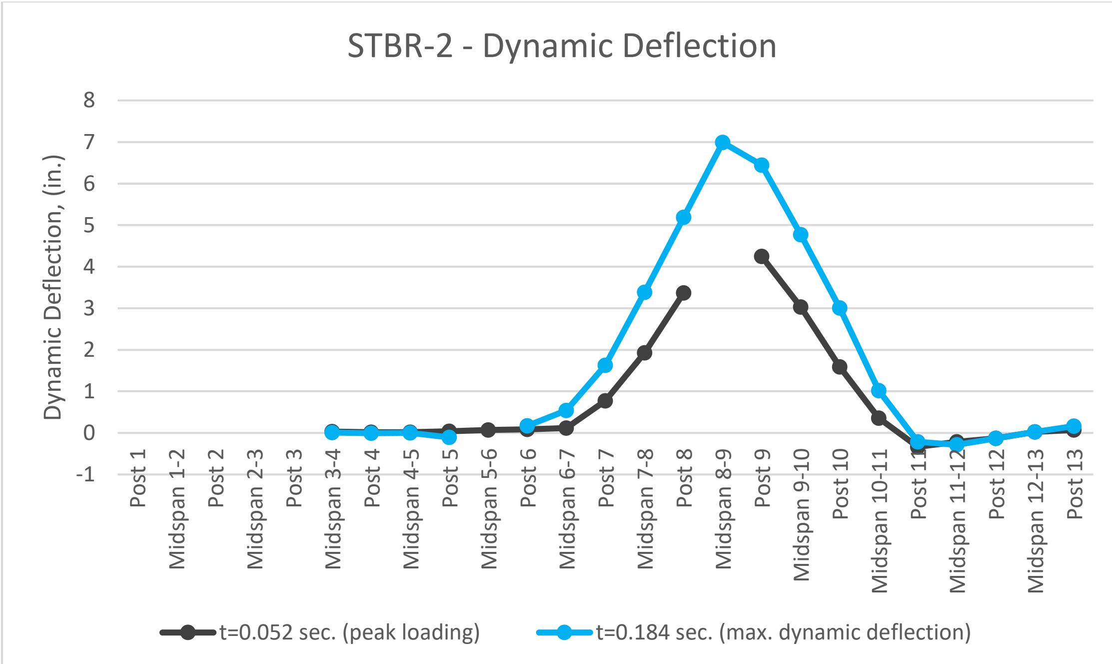


Figure 342. Dynamic Deflection, Test No. STBR-2

Inelastic analysis calculations were made while considering a five-span collapse and having plastic hinges 4 in. (102 mm) above the tension anchor rods. This analysis revealed a modified lateral barrier resistance equal to 55.0 kips (244.7 kN) with the contribution of only the lower two rails and 80.4 kips (357.6 kN) when considering all three of the rails.

As noted with the details of the pickup truck test, significant concrete damage was observed along the lower edge of the surrogate bridge deck found at post no. 9. In this crash test, at 7 ft (2.1 m) downstream from impact point, post no. 9 was the first post downstream from the where the truck impacted the system. Several factors may have contributed to the observed concrete damage near the compression zone of the vertical mounting plate. In the post-test investigation, the rails were observed to largely deform elastically with limited to no permanent deformations within the actual rail segments (splice joints rotated), while the posts deformed plastically. As the system unloaded, the former compression anchors were subjected to tension, and along with being in close proximity to the bottom of the surrogate bridge deck corner, the impact resulted in concrete breakout. Efforts were made to reconfigure the bottom anchorage. In place of each square washer at the end of the compression anchor bolts, which were originally cast into the deck, a 36-in. (914-mm) long by ⅜-in. (10-mm) thick steel plate was to be bolted to the coupling nuts, which would extend behind the no. 5 stirrups adjacent to the anchorages. The steel plate placed behind the stirrups further reinforced the bottom anchorage and reduced the risk of pryout and concrete failure. This new bottom anchorage was installed at post no. 9 for the full-scale crash test (retest) with MASH 2016 test designation no. 4-12.

For the small car crash test (test no. STBR-3), the permanent set of the side-mounted posts and midspans were also obtained from surveying with GPS equipment and high-speed digital video analysis, as shown in Figure 343. The permanent set of the bridge rail was 0.6 in. (15 mm), as stated in Chapter 16, which was based on GPS data. As stated in Chapter 16, no plastic hinges were observed at posts or rails after the crash test. However, based on both the GPS and the high-speed digital video analysis curves, it is indicated that three spans (i.e., from post no. 5 to post no. 8) plastically deformed.

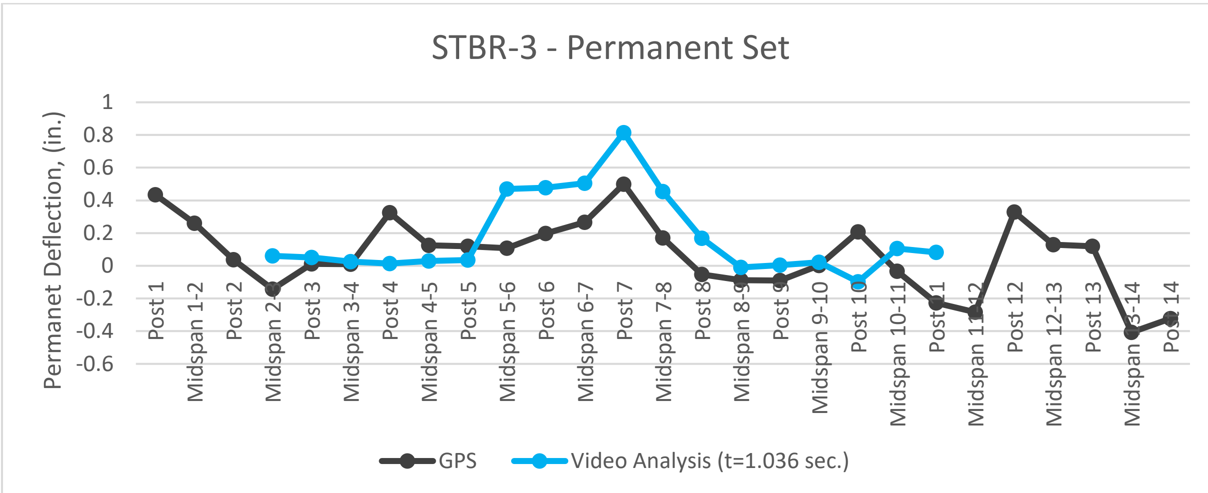


Figure 343. Permanent Set, Test No. STBR-3

For test no. STBR-3, the dynamic deflections for the side-mounted posts and midspans were also obtained for the small car crash test from high-speed digital video analysis. Two time-steps were considered. One time-step included the time when the maximum dynamic deflection occurred, while the second included small car impact with the bridge rail at the rear end. As shown in Figure 344, the maximum dynamic deflection was approximately 2.9 in (73.7 mm). Moreover, both curves indicated that three spans (i.e., from post no. 5 to post no. 8) deformed.

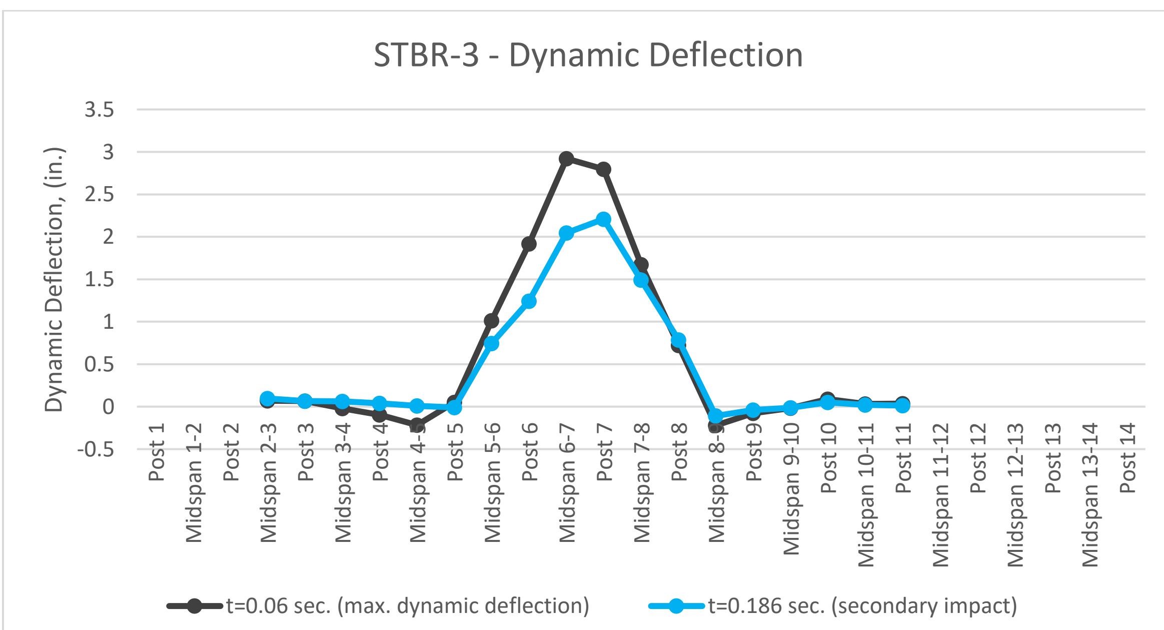


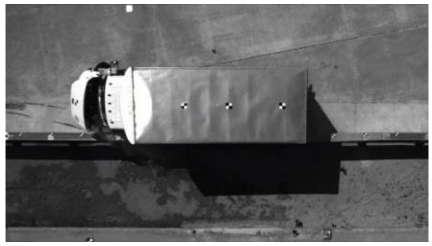
Figure 344. Dynamic Deflections, Test No. STBR-3

For test no. STBR-4, a re-run of MASH 2016 test designation no. 4-12, the SUT impacted the system with an impact severity of 151.7 kip-ft (205.7 kJ), which was above the minimum value set by MASH 2016, therefore validating test designation no. 4-12 on this bridge rail system. Similar to test no STBR-1, the primary concerns associated with MASH 2016 test designation no. 4-12 were vehicular containment, stability, and override, as well as peak lateral impact loading to the bridge rail and deck. Note that peak lateral impact loading to the structural deck system was evaluated in the dynamic bogie testing program. As such, vehicle containment, stability, and override were again evaluated with test no. STBR-4 in the same manner as in test no. STBR-1.

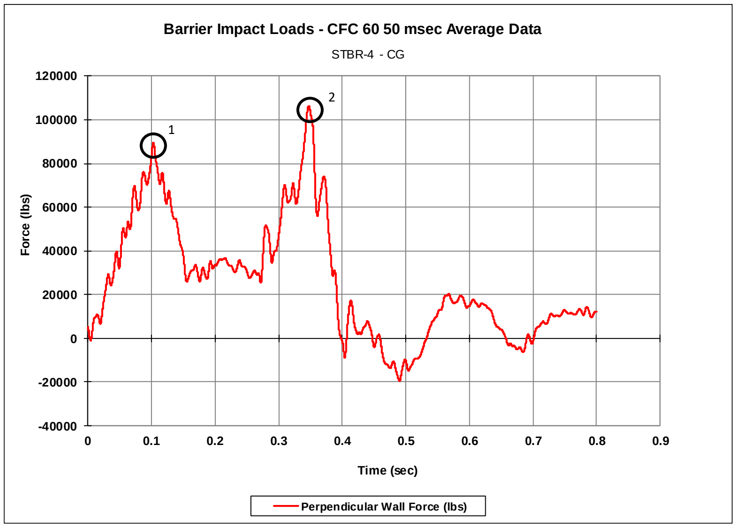
The maximum lateral impact force imparted to the system in test no. STBR-4, based on the primary accelerometer system, was determined to be 106.4 kips (473.3 kN) at a time of 0.347 seconds. This impact force occurred when the rear of the vehicle contacted the bridge rail after it had yawed, as shown in Figure 345.

For SUT impact scenarios, the lateral barrier resistance when considering all three of the rails was calculated as 81.5 kips (362.5 kN) for a five-span collapse when considering a DMF equal to 1.5 applied on the posts. However, after considering a DMF equal to 1.0 based on component testing results and after reducing the plastic section modulus of the three rails to consider the final post-to-deck connection attachments, the lateral barrier capacity of the bridge rail decreased to 65.8 kips (292.7 kN) for a five-span collapse. In test no. STBR-4, three posts plastically deformed, which occurred roughly 5 in. (127 mm) above the location of the tension anchor rods.

(a)



(b)



(c)

Figure 345. (a) Front-End Impact, (b) Rear-End Impact, and (c) Perpendicular Wall Impact Forces, Test No. STBR-4

Permanent deformation data was obtained from surveying with GPS equipment and high-speed digital video analysis of the SUT crash test to determine the actual number of spans that deflected. The permanent set of the side-mounted posts and midspans of the bridge rail is shown in Figure 346. For test no. STBR-4, the permanent set of the bridge rail was 7.3 in. (185 mm), as stated in Chapter 18 and based on GPS data. Also as stated in Chapter 18, plastic hinges were observed at post nos. 6, 7, and 8 after the crash test, forming a four-span collapse mechanism. However, based on Figure 346, both GPS and high-speed digital video analysis curves indicated that five spans (i.e., from midspan between post nos. 4 and 5 to midspan between post nos. 9 and 10) plastically deformed.

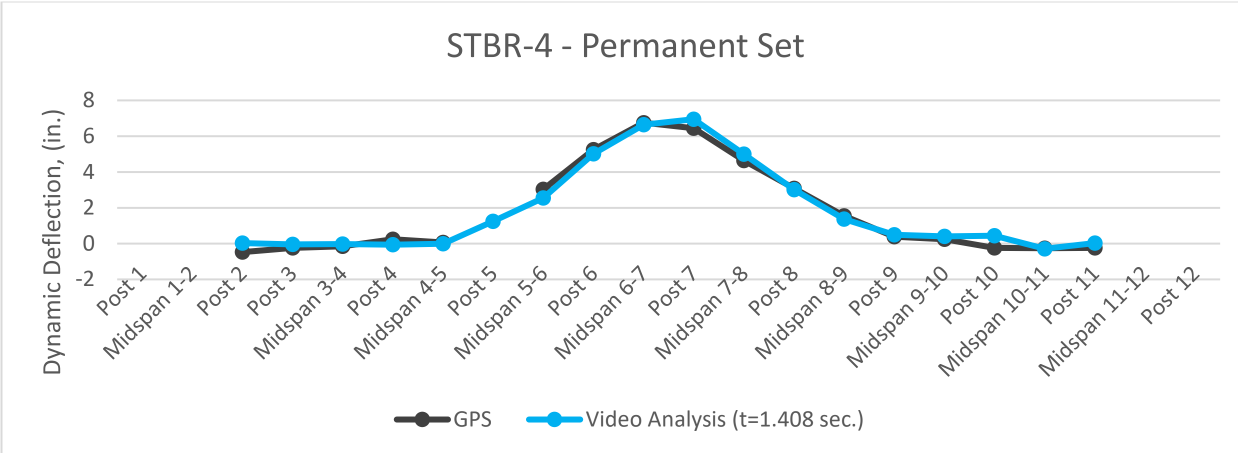


Figure 346. Permanent Set, Test No. STBR-4

For test no. STBR-4, the dynamic deflections for the side-mounted posts and midspans in between were obtained from high-speed digital video analysis. Two time-steps were considered, specifically, the time when peak loading was observed based on Figure 345 and when the maximum dynamic deflection occurred. As shown in Figure 347, the maximum visible dynamic deflection was approximately 7.9 in. (201 mm). With gaps in the data due to the box blocking the deflecting rail, there may have been greater dynamic deflection than observed. Moreover, both curves roughly indicated that five spans (i.e., from midspan between post nos. 4 and 5 to midspan between post nos. 9 and 10) deformed. Again, some data points were not visible as the SUT rolled and leaned on top of the upper rail of the system.

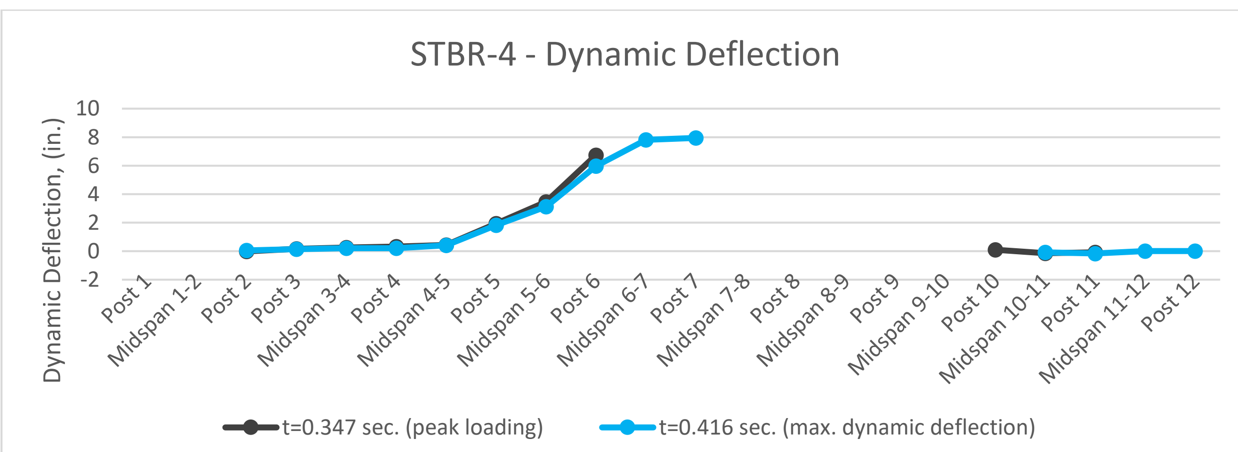


Figure 347. Visible Dynamic Deflections, Test No. STBR-4

Inelastic analysis calculations were made while considering a DMF equal to 1.0, a five-span collapse based on permanent and dynamic deflections, and assumed plastic hinges located 3 in. (76 mm) above the tension anchor rods. This analysis revealed a modified lateral barrier resistance equal to 72.7 kips (323.4 kN).

20 SUMMARY

The objective of this study was to develop and evaluate a new steel, side-mounted, beam-and-post, bridge rail according to the MASH 2016 TL-4 safety performance criteria for the Illinois and Ohio DOTs. The new bridge rail was designed to be adaptable to three bridge deck configurations utilized by the States of Illinois and Ohio and with or without future 3-in. (76-mm) thick roadway overlays. Moreover, the new bridge rail was side-mounted to the exterior, vertical edge of the bridge deck and with the front faces of the middle and bottom steel rails positioned vertically flush with the exterior edge of deck. The configuration increased the traversable deck width, and thus, reduced the overall width of the bridge deck. Finally, the new bridge rail was designed without a lower curb to allow water to drain off the outer vertical edges of the bridge deck. Finally, the MASH 2016 TL-4 system was configured with a minimum 36-in. (914-mm) height after a 3-in. (76-mm) thick future roadway overlay was applied.

First, a literature search was performed to review (1) historical and current crash testing criteria, (2) relevant steel, side-mounted and top-mounted, beam-and-post, bridge rails, (3) prior NCHRP Report 350 TL-4 and current MASH TL-4 lateral design loading for barriers, and (4) prior and current NCHRP Report 350 TL-4 and MASH TL-4 minimum barrier heights, which can be found in Chapter 2.

In Chapter 3, design criteria were established for the development of the new bridge rail. The critical bridge deck configurations were determined after evaluating the primary risks associated with the three MASH 2016 TL-4 full-scale vehicle crash tests. The critical deck configuration for MASH 2016 test designation no. 4-10 was determined to be bridge deck configuration #1 without a roadway overlay. For MASH 2016 test designation nos. 4-11 and 4-12, the critical deck configuration was bridge deck configuration #3 with a 3-in. (76-mm) roadway overlay. A top rail setback of 1 in. (25 mm) was selected to reduce the propensity for side window contact with the upper horizontal rail. Note that any bridge rail contact with side window and subsequent side window fracture would result in a test failure. A minimum bridge rail height of 36 in. (914 mm) was determined for the new bridge rail configuration based on a successful full-scale crash test of a 36 in. (914 mm) tall, single-slope, concrete barrier using a MASH SUT [38]. The minimum rail height for the MASH pickup truck was determined to be 29 in. (737 mm) based on finite element simulations impacting a 29-in. (737-mm) tall rigid barrier [36]. Initially, the research team disregarded the contribution of the top rail in providing containment and stability for pickup truck. Therefore, the middle rail was required to have a minimum top height of 29 in. (737 mm). Lateral and vertical design impact loadings for MASH TL-4 vehicles were identified and then used to design the new bridge rail. Moreover, the bridge rail was configured to mitigate vehicle snag into posts through identifying appropriate vertical clear openings, rail heights, and rail offsets away from the front face of posts. Geometric relationships provided by AASHTO LRFD *Bridge Design Specifications* as well as the geometry of the front bumper's structural components for each of the three MASH TL-4 test vehicles were analyzed and used to configure the bridge rail's geometry. Finally, personnel from the Illinois and Ohio DOTs provided additional design criteria to ease the fabrication and installation of the new bridge rail.

In Chapter 4, the two most common analysis methods for the design of steel, beam-and-post, bridge rails were reviewed – (1) nonlinear, finite element simulations of vehicle models impacting barrier systems and (2) inelastic or plastic analysis of a bridge rail under design impact loading. The inelastic or plastic analysis was selected for the design of the new steel, side-mounted, beam-and-post, bridge rail.

In Chapter 5, the inelastic or plastic analysis was demonstrated and used to estimate the lateral barrier resistance of the IL/OH Prototype Bridge Rail under MASH 2016 TL-4 SUT and pickup truck design loadings. Dynamic magnification factors of 1.0 and 1.5 were considered for the posts. A DMF equal to 1.5 was initially believed to account for strain rate effects as well as an elevated yield strength of a ASTM A36, 36-ksi (248 MPa) minimum, W6x9 (W150x13.5) steel post subjected to cantilevered loading. Guidance plots were created to identify the required plastic moment capacity for a combined number of rails at the height of the selected design impact loading in order to resist both pickup truck and SUT impact events. These plots provided guidance to design the preliminary bridge rail for the four bridge deck types utilized by the Illinois and Ohio DOTs with DMFs equal to 1.0 and 1.5. Improved bridge rail configurations were designed, while considering the critical bridge deck for SUT impact events and reducing the overall weight per foot of the system.

For the development of final bridge rail configurations, an analysis of the lateral bending resistance of the two lower rails within a single span was performed for MASH pickup truck impact events prior to post yielding and no assumed impact loading imparted to the top rail. The results from this analysis identified and ruled out lower rail sections that were unable to resist pickup truck design lateral loading within a single span. Moreover, the plastic section moduli of the three horizontal rails were reduced in order to include post-to-rail connection bolt holes before configuring final bridge rail prototypes. After the final bridge rail prototypes were designed, the weight per foot and preferences from the Illinois and Ohio DOTs were again considered. Then, the most efficient bridge rail was identified for subsequent full-scale vehicle crash testing and evaluation.

In Chapter 6, post-to-rail and rail-to-rail connections were designed for the new bridge rail configuration. For the post-to-rail connections, two concepts were produced to attach the top rail to the post. The first concept consisted of a double-angle bracket bolted between the top rail and each post’s web. The second concept consisted of a ⅜-in. (10-mm) thick, fully-welded, horizontal steel plate anchored to the top of each post. After discussion with the Illinois and Ohio DOTs, the fully-welded plate with ¾-in. (19-mm) diameter bolts was selected. A pair of staggered, ASTM A449 ¾-in. (19-mm) diameter, round-head bolts were used to attach the middle and bottom rails to the front flanges of the posts. Horizontal slotted bolt holes at the front flange of the posts and at the mounting brackets were used to provide a ⅝-in. (16-mm) horizontal construction tolerance for the installation and removal of the rails. For the rail-to-rail connections, both rectangular HSS steel section tubes with external shim plates as well as welded, built-up steel tubes were designed to properly connect the ends of the three rails, while providing continuity across the joints. After discussion with representatives from the Illinois and Ohio DOTs, built-up tubes were preferred for the new bridge rail. The installation and removal processes of the splice tubes and splice tube bolts were analyzed. Procedures for these processes were presented and explained.

In Chapter 7 and simultaneous to the design of the bridge railing, Mauricio, et al. was conducting research to develop the post-to-deck attachment hardware, which included performing seven dynamic bogie tests [13,14]. 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. Two 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. The average forces resisted by the posts in seven dynamic bogie tests were analyzed in order to evaluate the suitability for using a DMF equal to 1.5, which potentially would be applied to the posts and used to recalculate the lateral barrier capacity of the new bridge rail. This analysis showed that a DMF equal to 1.0 rather than 1.5 was appropriate for use in the development of the new bridge rail. Therefore, further review of successfully crash-tested, beam-and-post, bridge rails was made. After this additional analysis, the research team and representatives of the Illinois and Ohio DOTs decided to not strengthen the new bridge rail even though a DMF equal to 1.0 resulted in a lateral barrier capacity of 65.8 kips (292.7 kN), which was less than the 80-kip (356 kN) design load .

In Chapter 8, a surrogate bridge deck was designed to require only one surrogate bridge slab to be constructed for critically testing and evaluating the three MASH 2016 TL-4 full-scale vehicle crash tests. This surrogate bridge deck had a depth of 26 in. (660 mm) to allow for the critical installation of both post-to-deck connections at their appropriate heights.

The bridge rail system and the surrogate concrete bridge deck were then constructed and subjected to MASH 2016 TL-4 full-scale vehicle crash testing. The system installation for test no. STBR-1 was 159 ft – 11¼ in. long (48.8 m) with a nominal height of 36 in. (914 mm), including side-mounted and top-mounted posts. Only the side-mounted bridge rail system was evaluated, and the top-mounted system was only included for testing purposes to achieve the necessary system length to ensure vehicle redirection. In test no. STBR-1, the 22,124-lb (10,035-kg) SUT impacted the system at an angle of 14.5 degrees and a speed of 53.6 mph (86.2 km/h). According to MASH 2016, the target impact speed is 56.0 mph (90 km/h) with a tolerance of ± 2.5 mph (4.0 km/h), which was met, and the target impact angle is 15 degrees with a tolerance of ± 1.5 degrees, which was met. Although the test was within the limits for individual test parameters, the combination of the impact speed and the impact angle resulted in an impact severity of 133.2 kip-ft (180.6 kJ), which was below the allowable limit of 142.0 kip-ft (192.5 kJ). Nonetheless, the bridge rail properly contained and redirected the SUT. The maximum lateral load imparted to the barrier was approximately 96.1 kips (427.5 kN), as determined by the primary accelerometer system at the c.g. The maximum lateral load at the rear axle was found to be 102.4 kips (455.5 kN). During test no. STBR-1, post no. 7 developed a plastic hinge 5 in. (127 mm) above the location of the tension anchor rods. The remainder of the posts did not visually show signs of permanent damage. The middle and bottom rails were gouged from contact with the left-front wheel near the actual impact point. Denting was found in the front face of the middle rail upstream splice tube between post nos. 6 and 7. Additionally, minimal concrete spalling was found on the top-right corner of the embedded plates at post nos. 7 and 8. The maximum dynamic deflection was determined to be 4.3 in. (109 mm), as determined from high-speed digital video analysis.

The system installation for test no. STBR-2 was 159 ft – 11¼ in. long (48.8 m) with a nominal height of 36 in. (914 mm), including side-mounted and top-mounted posts. Only the side-mounted bridge rail system was evaluated, and the top-mounted system was only included for testing purposes to achieve the necessary system length to ensure vehicle redirection. The bridge rail properly contained and redirected the pickup truck, and all occupant risk values were within

MASH 2016 limits. The maximum lateral load imparted to the barrier was approximately 82.0 kips (364.8 kN), as determined by the primary accelerometer system. After test no. STBR-2, denting was found in the front face of the bottom rail near post no. 8. Post nos. 8 and 9 had plastic hinges 4 in. (102 mm) above the location of the tension anchor rods. There were 2-in. tall (5.1-mm) tire marks at the left side of the front flange of post no. 9 at located 13 in. (330 mm) above the height of the tension anchor rods. Post no. 10 slightly rotated backward at the height of the top post stiffeners. Additionally, significant concrete spalling and cracks were found at the bottom edge of the concrete deck, extending 4 ft – ½ in. (1.2 m) longitudinally and 11 in. (279 mm) above the bottom edge of the concrete deck at post no. 9. The maximum dynamic deflection of the system was determined to be 7.0 in. (178 mm), as determined from high-speed digital video analysis.

The system installation for test no. STBR-3 was 111 ft – 11¼ in. long (34.1 m) with a nominal height of 39 in. (991 mm), including only side-mounted posts. The system contained and redirected the small car, and all occupant risk values were within MASH 2016 limits. Tire marks were visible in the front flange of the post no. 7, top stiffeners, and deck spacer due to the snagging of the left-front wheel.

The system installation for test no. STBR-4 was 159 ft – 11¼ in. long (48.8 m) with a nominal height of 36 in. (914 mm), including side-mounted and top-mounted posts. Only the side-mounted bridge rail system was evaluated, and the top-mounted system was only included for testing purposes to achieve the necessary system length to ensure vehicle redirection. In test no. STBR-4, the 22,152-lb (10,048-kg) SUT impacted the system at an angle of 14.7 degrees and a speed of 56.4 mph (90.8 km/h). These conditions met the target impact speed set by MASH 2016 of 56.0 mph (90 km/h) with a tolerance of ± 2.5 mph (4.0 km/h), and the target impact angle set by MASH 2016 of 15 degrees with a tolerance of ± 1.5 degrees. The impact severity for this test was calculated to be 151.7 kip-ft (205.7 kJ), which is above the minimum limit of 142.0 kip-ft (192.5 kJ) set in MASH 2016. The bridge rail properly contained and redirected the SUT, and the maximum lateral load imparted to the barrier was approximately 106.4 kips (473 kN), as determined by the primary accelerometer system at the c.g. The maximum lateral load at the rear axle was found to be 110.3 kips (490.6 kN). During test no. STBR-4, post nos. 6, 7, and 8 each developed a plastic hinge 3 in. (76 mm) above the location of the tension anchor rods. Post no. 5 also experienced a small degree of bending, but the remainder of the posts experience no other permanent damage. The middle and bottom rails were gouged and dented from contact with the left-front wheel near the actual impact point. There was evidence of contact between the top rail of the system and the box of the vehicle extending from near the impact point all the way to the end of the bridge rail system. Additionally, minimal concrete spalling was found on the top corner of the surrogate bridge deck between post nos. 6 and 7, as well as at the top downstream corner of the post to deck connection of post no. 7. The maximum dynamic deflection was determined to be 7.9 in. (201 mm), as determined from high-speed digital video analysis.

In test no. STBR-1, the impact severity did not meet the allowable lower limit of 142.0 kip-ft for MASH 2016 test designation no. 4-12. Thus, test designation no. 4-12 was re-run in test no. STBR-4, and after the successful completion of the test, the bridge railing system was proven to be compliant with all MASH 2016 TL-4 impact safety standards.

21 CONCLUSIONS AND RECOMMENDATIONS

21.1 Conclusions

A new MASH 2016 TL-4 steel, side-mounted, beam-and-post, bridge rail was developed, crash tested, and evaluated. The new bridge rail was configured with W6x15 (W150x22.5) steel posts which were weaker that the W6x25 (W150x37.1) posts utilized in prior steel, side-mounted, beam-and-post, bridge rails utilized by Illinois and Ohio DOTs. This change was made to reduce the impact loads transferred to the deck, and consequently, reduce the potential for bridge deck damage. The bridge railing and post-to-deck connections were designed to be adaptable to multiple concrete deck configurations utilized by the States of Illinois and Ohio. These deck configurations include a minimum 18-in. thick slab deck and a minimum 17-in. thick box beam deck with up to a 6-in. thick concrete or asphalt wearing surface. A minimum height of 36 in. (914 mm) is used for MASH 2016 TL-4 systems, which takes into consideration a future 3-in. (76-mm) roadway overlay being used with the system. The new bridge rail was configured to reduce the required deck width by using side-mounted posts with the front faces of the lower two rails vertically aligned with the exterior bridge deck edge. A 1-in. (25-mm) top rail setback was utilized for full-scale crash testing and prevented vehicle-to-rail contact and shattering of the side windows of passenger vehicles with the top rail. Each of the rail segments weighed no more than 500 lb (227 kg) in order to eliminate the need for heavy construction equipment during installation.

The new bridge rail successfully mitigated snag risks for passenger vehicles with appropriate railing configurations and heights. The left-front wheel of the small car contacted and snagged against a post without excessive risk to occupants in test designation no. 4-10.

The fully-welded plate mounting bracket for the top rail, as well as the middle and bottom post-to-rail connections, performed adequately throughout the four full-scale crash tests. The splice tubes successfully performed and provided ease of installation, maintenance, and repair. Moreover, the removal and replacement processes of the splice tubes were successfully performed with no complications. The surrogate concrete bridge deck was successfully designed to allow for only one bridge deck to be constructed for all four of the full-scale crash tests.

The new steel, side-mounted, beam-and-post, bridge rail successfully contained and redirected the three MASH 2016 TL-4 vehicles. Therefore, it was determined that the plastic collapse mechanism represented an appropriate method for the design of steel-beam-and-post, bridge rails.

The tension anchor hardware performed adequately during the four full-scale crash tests without severe concrete deck damage. However, concrete damage was observed at the bottom region of one post location of the surrogate concrete bridge deck during test designation no. 4-11. The damage revealed that the bottom, square anchor plates performed ineffectively when the post was subjected to reverse-bending, resulting in concrete breakout and anchorage pullout. Due to the concrete damage in the pickup truck crash test, modifications were made to the post-to-deck connection and the surrogate concrete bridge deck before test no. STBR-4, as stated in Chapter 19.

21.2 Recommendations

Due to concrete damage in the pickup truck crash test (test no. STBR-2), modifications of the post-to-deck connection and the surrogate concrete bridge deck as stated in Chapter 19 were implemented for the re-run of test designation no. 4-12. If it is desired to reduce the potential for concrete breakout near the bottom region of the deck, then it is recommended that either the internal washer plate that was utilized in test no. STBR-4 be used at the lower anchor location and/or that the deck thickness be increased to reduce the potential for concrete breakout near the bottom region of the deck.

For the construction of the surrogate concrete bridge deck, the anchorage hardware was embedded within the form, rather than utilizing the option of welding the coupling nuts to the embedded vertical plates. When the form was removed, some vertical plates detached from the exterior, vertical edge of the slab. The use of welded studs or welded coupling nuts on the embedded vertical plates is recommended for future implementation of this bridge rail system in the field to ease installation. The welded stud option and the welded coupling nut options are shown in Figure 282.

An adequate MASH 2016 TL-3 approach guardrail transition must be developed and evaluated to safely connect the new, steel, side-mounted, beam-and-post, bridge rail to adjacent approach guardrail systems. The lateral barrier capacity of the transition will need to be investigated and compared with design impact loading using computer simulation. Post spacing near the bridge ends can also be modified to meet MASH 2016 crashworthiness requirements. The development of the transition is documented in Rasmussen, et al. [11]. 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 [12].

22 REFERENCES

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23 APPENDICES

Appendix A. Derivation of Single-Span Plastic Collapse

The plastic collapse or inelastic analysis method relies on the principle of virtual work, which involves the balance of external work imparted by vehicular impact loading and internal work represented by the energy absorbed by the bridge rail.

Consider the partially loaded fixed beam shown in Figure A-1 with a distributed load. As distributed load, W_T, increases, the bending stresses at the support locations reach the yield strength of the material. Eventually, as the load increases, the entire cross section reaches its yield stress. This bending state is known as the plastic moment capacity, M_P, of the cross section. The cross section is not capable to resist additional moment, but it maintains this moment capacity for the rotation, θ, or plastic hinges in the beam, one at each end for a total of combined 2θ and one at midspan for 2θ.

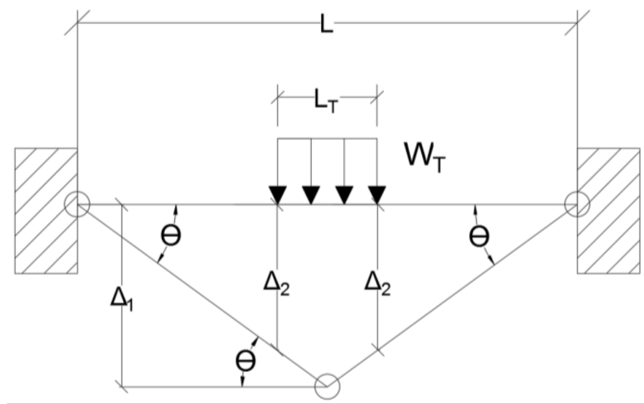


Figure A-1. Plastic Hinges at Midspan and End Sections of a Partially-Loaded, Single-Span Fixed Beam

where:

L = beam length;
L_T = length of design distributed load;
W_T =design impact distributed load;
Δ_1 = maximum deflection of beam at midspan;
Δ_2 = deflection at ends of the length of design distributed load;
θ = angle of rotation of deflected shape; and
M_P = plastic moment capacity of beam.

The internal and external work of the beam are expressed by Equation 9:

$$W_T * L_T * \Delta_1 = 4\theta * M_P \tag{9}$$

The angle of rotation of the deflected shape, θ, can be simplified using a small-angle approximation for a relatively-small deflection, Δ, as shown in Equation 10.

$$\theta \approx \tan\theta \tag{10}$$

Using a small-angle approximation with Equation 10 along with a substitution for tan θ equal to $\frac{\Delta_1}{L/2}$, leads to Equation 11 for the angle of rotation, θ, at the midspan location:

$$\theta = \tan\theta = \frac{\Delta_1}{L/2} \tag{11}$$

The equation for the midspan deflection can be expressed as Equation 12:

$$\Delta_1 = \frac{\theta L}{2} \tag{12}$$

Similarly, a small-angle approximation with Equation 9 can be sued to obtain beam deflections at the end of the distributed load, thus resulting in Equation 13:

$$\theta = \tan\theta = \frac{\Delta_2}{\frac{L}{2} - \frac{L_T}{2}} \tag{13}$$

The equation for the deflection, Δ_2, at the ends of the distribution load is expressed as:

$$\Delta_2 = \frac{\theta L - \theta L_T}{2} \tag{14}$$

The average of deflection Δ_1 and deflection Δ_2 is expressed in Equation 15:

$$\Delta_{AVG} = \frac{\Delta_1 + \Delta_2}{2} = \frac{\frac{\theta L + \theta L - \theta L_T}{2}}{2} = \frac{\theta L}{2} - \frac{\theta L_T}{4} = \frac{\theta}{4}[2L - L_T] \tag{15}$$

The internal and external work in the beam with Δ_{AVG} can be expressed by Equation 16:

$$W_T * L_T * \frac{\theta}{4}[2L - L_T] = 4\theta * M_P \tag{16a}$$

$$W_T * L_T * \frac{2L - L_T}{4} = 4\ M_P \tag{16b}$$

The final equation for the plastic capacity of a fixed-beam is expressed by Equation 17, which corresponds to a single span with a partially-distributed load over the midspan.

$$W_T * L_T = \frac{16M_P}{2L - L_T} \tag{17}$$

Using Equation 2 from Section 4 with N=1 (single-span), the lateral beam or barrier capacity is:

$$R = \frac{16\ M_P + (1-1)(1+1)P_P L}{2(1)L - L_T}$$

$$R = \frac{16\ M_P + (0)(2)P_P L}{2L - L_T}$$

$$R = \frac{16\ M_P}{2L - L_T}$$

Appendix B. Bridge Rail Design

Table B-1. IL/OH MASH TL-4 Bridge Rail Prototype Design with DMF=1.0

SYSTEM INFORMATION		RAIL AND POST PLASTIC MOMENTS		RAIL AND POST PLASTIC MOMENTS		EFFECTIVE HEIGHT OF RAILS		POST SHEAR		BARRIER RESISTANCE NO. OF SPANS, R (kips)		MINIMUM BARRIER RESISTANCE			BARRIER RESISTANCE AT LOAD HEIGHT	
Number of Rails	3	Mp Post (kip - in.)	486	**Single-Unit Truck, three rails**												
ϕ	0.9	Mp Upper Rail (kip - in.)	1039.5	Mp Upper Rail (kip - in.)	1039.5	Y_{RAILS} (in.)	34.44	Ppost (kip)	14.11	ONE-SPAN	308.80	R(kip)	77.09	FIVE-SPAN	R_{DESIGN} (kip)	78.1
Dynamic Magnification Factor	1	Mp Middle Rail (kip - in.)	445.95	Mp Middle Rail (kip - in.)	445.95					TWO-SPAN	133.44					
Fy (ksi.)	50	Mp Lower Rail (kip - in.)	251.55	Mp Lower Rail (kip - in.)	251.55					THREE-SPAN	92.97					
L (Post Spacing)(in.)	75	Mp Σ Rails (kip - in.)	1737	Mp Σ Rails (kip - in.)	1737					FOUR-SPAN	82.82					
Asphalt Overlay (in.)	0									FIVE-SPAN	77.09					
Tension anchor Center to top of deck (in.)	4									SIX-SPAN	78.44					
Tension Anchor Center to top overlay (in.)	4									SEVEN-SPAN	79.38					
DESIGN CONSIDERATIONS										EIGHT-SPAN	83.79					
Pickup Truck, Ft Lateral Load (kips)	70			**Pickup Truck, two rails**												
Single-Unit Truck, Ft Lateral Load (kips)	80			Mp Middle Rail (kip - in.)	445.95	Y_{RAILS} (in.)	24.67	Ppost (kip)	19.70	ONE-SPAN	109.41	R(kip)	57.16	THREE-SPAN	R_{DESIGN} (kip)	50.4
				Mp Lower Rail (kip - in.)	251.55					TWO-SPAN	67.74					
Pickup Truck ,Lt Distributed Length (in.)	48			Mp Σ Rails (kip - in.)	697.5					THREE-SPAN	57.16					
Single-Unit Truck, Lt Distributed Length (in.)	60									FOUR-SPAN	63.04					
										FIVE-SPAN	66.41					
Pickup Truck (Load Height)(in.)	24									SIX-SPAN	75.52					
Single-Unit Truck (Load Height)(in.)	30									SEVEN-SPAN	81.91					
SECTION SELECTION										EIGHT-SPAN	91.76					
W6x15 Posts				**Pickup Truck, three rails**												
Z, Plastic Section Modulus (in.^{3})	10.8			Mp Upper Rail (kip - in.)	1039.5	Y_{RAILS} (in.)	34.44	Ppost (kip)	14.11	ONE-SPAN	272.47	R (kip)	75.77	FIVE-SPAN	R_{DESIGN} (kip)	93.2
				Mp Middle Rail (kip - in.)	445.95					TWO-SPAN	127.08					
Upper Rail				Mp Lower Rail (kip - in.)	251.55					THREE-SPAN	90.19					
Z, Plastic Section Modulus (in.^{3})	23.1			Mp Σ Rails (kip - in.)	1737					FOUR-SPAN	81.02					
Rail Center Height to Road Surface (in.)	37									FIVE-SPAN	75.77					
Top Anchor Center to Rail Center (in.)	41									SIX-SPAN	77.33					
										SEVEN-SPAN	78.43					
Middle Rail										EIGHT-SPAN	82.92					
Z, Plastic Section Modulus (in.^{3})	9.91															
Rail Center Height to Road Surface (in.)	25															
Top Anchor Center to Rail Center (in.)	29															
Lower Rail																
Z, Plastic Section Modulus (in.^{3})	5.59															
Rail Center Height to Road Surface (in.)	13															
Top Anchor Center to Rail Center (in.)	17															

Table B-2. IL/OH MASH TL-4 Bridge Rail Prototype Design with DMF=1.5

SYSTEM INFORMATION	
Number of Rails	3
ϕ	0.9
Dynamic Magnification Factor	1.5
Fy (ksi.)	50
L (Post Spacing)(in.)	75
Asphalt Overlay (in.)	0
Tension anchor Center to top of deck (in.)	4
Tension Anchor Center to top overlay (in.)	4
DESIGN CONSIDERATIONS	
Pickup Truck, Ft Lateral Load (kips)	70
Single-Unit Truck, Ft Lateral Load (kips)	80
Pickup Truck ,Lt Distributed Length (in.)	48
Single-Unit Truck, Lt Distributed Length (in.)	60
Pickup Truck (Load Height)(in.)	24
Single-Unit Truck (Load Height)(in.)	30
SECTION SELECTION	
W6x15 Posts	
Z, Plastic Section Modulus (in.^{3})	10.8
Upper Rail	
Z, Plastic Section Modulus (in.^{3})	23.1
Rail Center Height to Road Surface (in.)	37
Top Anchor Center to Rail Center (in.)	41
Middle Rail	
Z, Plastic Section Modulus (in.^{3})	9.91
Rail Center Height to Road Surface (in.)	25
Top Anchor Center to Rail Center (in.)	29
Lower Rail	
Z, Plastic Section Modulus (in.^{3})	5.59
Rail Center Height to Road Surface (in.)	13
Top Anchor Center to Rail Center (in.)	17

RAIL AND POST PLASTIC MOMENTS	
Mp Post (kip - in.)	729
Mp Upper Rail (kip - in.)	1039.5
Mp Middle Rail (kip - in.)	445.95
Mp Lower Rail (kip - in.)	251.55
Mp Σ Rails (kip - in.)	1737

RAIL AND POST PLASTIC MOMENTS		EFFECTIVE HEIGHT OF RAILS		POST SHEAR		BARRIER RESISTANCE NO. OF SPANS, R (kips)		MINIMUM BARRIER RESISTANCE			BARRIER RESISTANCE AT LOAD HEIGHT	
Single-Unit Truck, three rails												
Mp Upper Rail (kip - in.)	1039.5	Y_{RAILS} (in.)	34.44	Ppost (kip)	21.17	ONE-SPAN	308.80	R(kip)	95.49	FIVE-SPAN	R_{DESIGN} (kip)	96.7
Mp Middle Rail (kip - in.)	445.95					TWO-SPAN	142.26					
Mp Lower Rail (kip - in.)	251.55					THREE-SPAN	103.82					
Mp Σ Rails (kip - in.)	1737					FOUR-SPAN	98.50					
						FIVE-SPAN	**95.49**					
						SIX-SPAN	101.12					
						SEVEN-SPAN	105.04					
						EIGHT-SPAN	113.50					
Pickup Truck, two rails												
Mp Middle Rail (kip - in.)	445.95	Y_{RAILS} (in.)	24.67	Ppost (kip)	29.55	ONE-SPAN	109.41	R(kip)	71.86	THREE-SPAN	R_{DESIGN} (kip)	63.3
Mp Lower Rail (kip - in.)	251.55					TWO-SPAN	79.46					
Mp Σ Rails (kip - in.)	697.5					THREE-SPAN	**71.86**					
						FOUR-SPAN	84.45					
						FIVE-SPAN	91.66					
						SIX-SPAN	106.73					
						SEVEN-SPAN	117.30					
						EIGHT-SPAN	132.80					
Pickup Truck, three rails												
Mp Upper Rail (kip - in.)	1039.5	Y_{RAILS} (in.)	34.44	Ppost (kip)	21.17	ONE-SPAN	272.47	R (kip)	93.86	FIVE-SPAN	R_{DESIGN} (kip)	115.5
Mp Middle Rail (kip - in.)	445.95					TWO-SPAN	135.48					
Mp Lower Rail (kip - in.)	251.55					THREE-SPAN	100.72					
Mp Σ Rails (kip - in.)	1737					FOUR-SPAN	96.36					
						FIVE-SPAN	**93.86**					
						SIX-SPAN	99.69					
						SEVEN-SPAN	103.78					
						EIGHT-SPAN	112.31					

Table B-3. Additional Guidance Plots for Single-Unit Trucks for 30 in. Effective Height of Rails

SUT
DMF=1.0

Σ Mp (kips - in)							
3500	149.4	131.3	118.1	115.5	101.8	90.0	81.2
3400	147.1	129.6	116.4	113.9	100.5	88.6	80.0
3300	144.9	127.5	114.8	112.2	99.3	87.2	78.8
3200	142.6	125.5	113.1	110.6	98.0	85.8	77.7
3100	140.3	123.4	111.4	109.0	96.6	84.4	76.5
3000	138.1	121.4	109.7	107.4	94.8	83.0	75.4
2900	135.8	119.3	108.0	105.8	93.0	81.6	74.2
2800	133.6	117.3	106.3	104.2	91.2	80.2	73.0
2700	131.3	115.2	104.6	102.5	89.5	78.8	71.9
2600	129.0	113.1	102.9	100.9	87.7	77.4	70.7
2500	126.3	111.1	101.3	99.3	85.9	76.0	69.6
2400	123.7	109.0	99.6	97.7	84.1	74.6	68.4
2300	121.1	107.0	97.9	95.6	82.4	73.2	67.2
2200	118.5	104.9	95.7	93.3	80.6	71.8	66.1
2100	115.9	102.9	93.3	91.0	78.8	70.4	64.9
2000	113.3	100.8	90.9	88.6	77.0	69.0	63.0
1900	110.7	98.8	88.5	86.3	75.2	67.6	61.0
1800	108.0	96.5	86.1	84.0	73.5	66.2	59.0
1700	105.4	93.6	82.5	81.7	71.7	64.8	57.0
1600	102.8	90.6	81.2	79.4	69.9	62.4	55.1
1500	100.2	87.6	78.8	77.0	68.1	59.9	53.1
1400	97.6	84.7	76.4	74.7	66.4	57.5	51.1
1300	94.0	81.7	73.9	72.4	64.4	55.1	49.1
1200	90.1	78.8	71.5	70.1	61.3	52.7	47.1
1100	86.3	75.8	69.1	67.8	58.2	50.2	45.1
1000	82.5	72.8	66.7	65.4	55.1	47.8	43.1
900	78.7	69.9	63.8	61.8	52.0	45.4	41.1
800	74.9	66.9	59.5	57.7	48.9	43.0	39.1
700	71.1	63.3	55.2	53.6	45.8	40.5	37.1
600	67.3	57.9	50.9	49.5	42.7	38.1	35.2
500	62.4	52.6	46.6	45.4	39.6	35.7	33.2
400	55.4	47.3	42.3	41.3	36.5	33.3	28.1
300	48.3	41.9	38.0	37.2	33.4	26.7	21.1
200	41.3	36.6	33.7	33.1	24.2	17.8	14.0
100	34.3	26.7	19.0	17.8	12.1	8.9	7.0
	48	60	72	75	96	120	144
	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft
	Post Spacing						

30" Effective height of rails

SUT
DMF=1.5

Σ Mp (kips - in)							
2300	151.6	132.9	119.4	116.7	103.1	91.1	80.6
2200	149.0	130.0	117.0	114.4	101.3	88.7	78.6
2100	146.4	127.0	114.5	112.1	99.5	86.3	76.6
2000	142.8	124.1	112.1	109.8	97.8	83.8	74.6
1900	139.0	121.1	109.7	107.4	95.1	81.4	72.6
1800	135.2	118.1	107.3	105.1	92.0	79.0	70.6
1700	131.4	115.2	103.7	102.8	88.9	76.6	68.6
1600	127.6	112.2	102.4	100.5	85.8	74.1	66.7
1500	123.8	109.2	100.0	98.2	82.7	71.7	64.7
1400	120.0	106.3	97.6	94.8	79.6	69.3	62.7
1300	116.2	103.3	93.5	90.7	76.5	66.9	60.7
1200	112.4	100.4	89.2	86.6	73.4	64.4	58.7
1100	108.6	97.4	84.9	82.5	70.3	62.0	56.7
1000	104.7	92.2	80.6	78.4	67.2	59.6	54.7
900	100.9	86.9	76.3	74.3	64.1	57.2	52.7
800	97.1	81.5	72.0	70.2	61.0	54.7	50.7
700	90.0	76.2	67.7	66.1	57.9	52.3	48.7
600	83.0	70.9	63.4	62.0	54.8	49.9	42.1
500	76.0	65.5	59.1	57.9	51.7	44.4	35.1
400	69.0	60.2	54.8	53.8	48.5	35.6	28.1
300	62.0	54.9	50.5	49.7	36.4	26.7	21.1
200	55.0	49.5	38.1	35.6	24.2	17.8	14.0
100	44.4	26.7	19.0	17.8	12.1	8.9	7.0
	48	60	72	75	96	120	144
	4 ft	5 ft	6 ft	6 ft - 3 in	8 ft	10 ft	12 ft
	Post Spacing						

30" Effective height of rails

Table B-4. Additional Guidance Plots for Single-Unit Trucks for 31 in. Effective Height of Rails

SUT
DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
3600	149.4	130.9	117.9	115.2	101.1	90.1	81.0
3500	147.1	129.1	116.2	113.6	99.9	88.7	79.8
3400	144.9	127.3	114.5	112.0	98.6	87.3	78.7
3300	142.6	125.6	112.8	110.3	97.4	85.9	77.5
3200	140.3	123.5	111.2	108.7	96.1	84.5	76.4
3100	138.1	121.5	109.5	107.1	94.9	83.1	75.2
3000	135.8	119.4	107.8	105.5	93.5	81.7	74.0
2900	133.6	117.4	106.1	103.9	91.7	80.3	72.9
2800	131.3	115.3	104.4	102.3	89.9	78.9	71.7
2700	129.0	113.3	102.7	100.6	88.1	77.5	70.6
2600	126.8	111.2	101.0	99.0	86.4	76.1	69.4
2500	124.4	109.2	99.3	97.4	84.6	74.7	68.2
2400	121.8	107.1	97.7	95.8	82.8	73.3	67.1
2300	119.2	105.1	96.0	94.2	81.0	71.9	65.9
2200	116.5	103.0	94.3	91.9	79.2	70.5	64.8
2100	113.9	101.0	92.0	89.6	77.5	69.1	63.6
2000	111.3	98.9	89.5	87.3	75.7	67.7	62.3
1900	108.7	96.9	87.1	85.0	73.9	66.3	60.3
1800	106.1	94.8	84.7	82.6	72.1	64.9	58.3
1700	103.5	92.2	81.2	80.3	70.4	63.5	56.3
1600	100.9	89.2	79.8	78.0	68.6	61.6	54.3
1500	98.2	86.3	77.4	75.7	66.8	59.2	52.3
1400	95.6	83.3	75.0	73.4	65.0	56.7	50.3
1300	92.5	80.3	72.6	71.0	63.2	54.3	48.3
1200	88.7	77.4	70.1	68.7	60.5	51.9	46.3
1100	84.9	74.4	67.7	66.4	57.4	49.5	44.4
1000	81.1	71.4	65.3	64.1	54.3	47.0	42.4
900	77.3	68.5	62.9	61.0	51.2	44.6	40.4
800	73.5	65.5	58.7	56.9	48.1	42.2	38.4
700	69.7	62.4	54.4	52.8	45.0	39.8	36.4
600	65.9	57.1	50.1	48.7	41.9	37.3	34.4
500	61.5	51.8	45.8	44.6	38.8	34.9	32.4
400	54.5	46.4	41.5	40.5	35.7	32.5	28.1
300	47.5	41.1	37.2	36.4	32.6	26.7	21.1
200	40.4	35.8	32.9	32.3	24.2	17.8	14.0
100	33.4	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

31" Effective height of rails

SUT
DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2400	151.3	133.8	119.8	117.0	102.9	92.4	81.5
2300	148.7	130.9	117.3	114.7	101.1	90.0	79.5
2200	146.0	127.9	114.9	112.4	99.3	87.5	77.5
2100	143.4	124.9	112.5	110.0	97.5	85.1	75.5
2000	140.7	122.0	110.1	107.7	95.8	82.7	73.5
1900	136.9	119.0	107.6	105.4	93.9	80.3	71.5
1800	133.1	116.0	105.2	103.1	90.8	77.8	69.5
1700	129.3	113.1	101.6	100.8	87.7	75.4	67.5
1600	125.5	110.1	100.4	98.4	84.6	73.0	65.5
1500	121.6	107.2	97.9	96.1	81.5	70.6	63.5
1400	117.8	104.2	95.5	93.6	78.4	68.1	61.6
1300	114.0	101.2	92.3	89.5	75.3	65.7	59.6
1200	110.2	98.3	88.0	85.4	72.2	63.3	57.6
1100	106.4	95.3	83.7	81.3	69.1	60.9	55.6
1000	102.6	91.0	79.4	77.2	66.0	58.4	53.6
900	98.8	85.6	75.1	73.1	62.9	56.0	51.6
800	95.0	80.3	70.8	69.0	59.8	53.6	49.6
700	88.7	75.0	66.5	64.9	56.7	51.2	47.6
600	81.7	69.6	62.2	60.8	53.6	48.8	42.1
500	74.7	64.3	57.9	56.7	50.5	44.4	35.1
400	67.7	59.0	53.6	52.6	47.4	35.6	28.1
300	60.7	53.6	49.3	48.5	36.4	26.7	21.1
200	53.6	48.3	38.1	35.6	24.2	17.8	14.0
100	44.4	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

31" Effective height of rails

Table B-5. Additional Guidance Plots for Single-Unit Trucks for 32 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
3700	149.5	130.6	117.8	115.0	100.6	90.3	80.9
3600	147.3	128.8	116.1	113.4	99.4	88.9	79.8
3500	145.0	127.0	114.4	111.8	98.1	87.5	78.6
3400	142.7	125.2	112.8	110.2	96.9	86.1	77.5
3300	140.5	123.5	111.1	108.6	95.6	84.7	76.3
3200	138.2	121.7	109.4	106.9	94.4	83.3	75.1
3100	136.0	119.7	107.7	105.3	93.1	81.9	74.0
3000	133.7	117.6	106.0	103.7	91.9	80.5	72.8
2900	131.4	115.6	104.3	102.1	90.4	79.1	71.7
2800	129.2	113.5	102.6	100.5	88.7	77.7	70.5
2700	126.9	111.5	100.9	98.9	86.9	76.3	69.3
2600	124.7	109.4	99.2	97.2	85.1	74.9	68.2
2500	122.4	107.4	97.6	95.6	83.3	73.5	67.0
2400	119.9	105.3	95.9	94.0	81.5	72.1	65.9
2300	117.3	103.3	94.2	92.4	79.8	70.6	64.7
2200	114.7	101.2	92.5	90.6	78.0	69.2	63.5
2100	112.1	99.2	90.7	88.3	76.2	67.8	62.4
2000	109.5	97.1	88.2	86.0	74.4	66.4	61.2
1900	106.8	95.1	85.8	83.7	72.7	65.0	59.6
1800	104.2	93.0	83.4	81.4	70.9	63.6	57.6
1700	101.6	90.9	79.9	79.0	69.1	62.2	55.6
1600	99.0	87.9	78.6	76.7	67.3	60.8	53.6
1500	96.4	84.9	76.1	74.4	65.5	58.5	51.6
1400	93.8	82.0	73.7	72.1	63.8	56.0	49.6
1300	91.2	79.0	71.3	69.8	62.0	53.6	47.6
1200	87.4	76.1	68.9	67.4	59.8	51.2	45.6
1100	83.6	73.1	66.4	65.1	56.7	48.8	43.7
1000	79.8	70.1	64.0	62.8	53.6	46.3	41.7
900	75.9	67.2	61.6	60.3	50.5	43.9	39.7
800	72.1	64.2	57.9	56.2	47.4	41.5	37.7
700	68.3	61.2	53.6	52.1	44.3	39.1	35.7
600	64.5	56.3	49.3	48.0	41.2	36.6	33.7
500	60.7	51.0	45.0	43.9	38.1	34.2	31.7
400	53.6	45.6	40.7	39.8	35.0	31.8	28.1
300	46.6	40.3	36.4	35.7	31.9	26.7	21.1
200	39.6	35.0	32.1	31.6	24.2	17.8	14.0
100	32.6	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

32" Effective height of rails

SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2400	148.5	131.9	117.8	115.1	101.0	91.2	80.4
2300	145.9	128.9	115.4	112.8	99.2	88.9	78.4
2200	143.3	125.9	113.0	110.4	97.4	86.5	76.4
2100	140.7	123.0	110.6	108.1	95.7	84.0	74.4
2000	138.1	120.0	108.1	105.8	93.9	81.6	72.4
1900	134.9	117.0	105.7	103.5	92.1	79.2	70.5
1800	131.1	114.1	103.3	101.2	89.7	76.8	68.5
1700	127.2	111.1	99.7	98.8	86.6	74.3	66.5
1600	123.4	108.2	98.4	96.5	83.5	71.9	64.5
1500	119.6	105.2	96.0	94.2	80.4	69.5	62.5
1400	115.8	102.2	93.6	91.9	77.3	67.1	60.5
1300	112.0	99.3	91.2	88.4	74.2	64.7	58.5
1200	108.2	96.3	86.9	84.3	71.1	62.2	56.5
1100	104.4	93.3	82.6	80.2	68.0	59.8	54.5
1000	100.6	89.8	78.3	76.1	64.9	57.4	52.5
900	96.8	84.5	74.0	72.0	61.8	55.0	50.6
800	93.0	79.1	69.7	67.9	58.7	52.5	48.6
700	87.5	73.8	65.4	63.8	55.6	50.1	46.6
600	80.5	68.5	61.1	59.7	52.5	47.7	42.1
500	73.5	63.1	56.8	55.6	49.4	44.4	35.1
400	66.4	57.8	52.5	51.5	46.3	35.6	28.1
300	59.4	52.5	48.2	47.4	36.4	26.7	21.1
200	52.4	47.1	38.1	35.6	24.2	17.8	14.0
100	44.4	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

32" Effective height of rails

Table B-6. Additional Guidance Plots for Single-Unit Trucks for 33 in. Effective Height of Rails

SUT

DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
3800	149.8	130.4	117.8	115.0	100.2	89.9	80.9
3700	147.5	128.6	116.1	113.4	99.0	88.9	79.8
3600	145.3	126.8	114.4	111.7	97.7	87.7	78.6
3500	143.0	125.1	112.8	110.1	96.5	86.3	77.5
3400	140.7	123.3	111.1	108.5	95.2	84.9	76.3
3300	138.5	121.5	109.4	106.9	94.0	83.5	75.1
3200	136.2	119.7	107.7	105.3	92.7	82.1	74.0
3100	134.0	117.9	106.0	103.7	91.5	80.7	72.8
3000	131.7	115.9	104.3	102.0	90.2	79.3	71.7
2900	129.4	113.9	102.6	100.4	89.0	77.9	70.5
2800	127.2	111.8	100.9	98.8	87.5	76.5	69.3
2700	124.9	109.8	99.3	97.2	85.7	75.1	68.2
2600	122.7	107.7	97.6	95.6	83.9	73.7	67.0
2500	120.4	105.7	95.9	94.0	82.1	72.3	65.9
2400	118.1	103.6	94.2	92.3	80.4	70.9	64.7
2300	115.6	101.6	92.5	90.7	78.6	69.5	63.5
2200	113.0	99.5	90.8	89.1	76.8	68.1	62.4
2100	110.3	97.5	89.1	87.1	75.0	66.7	61.2
2000	107.7	95.4	87.0	84.8	73.3	65.3	60.1
1900	105.1	93.4	84.6	82.5	71.5	63.9	58.9
1800	102.5	91.3	82.2	80.2	69.7	62.5	56.9
1700	99.9	89.2	78.7	77.8	67.9	61.1	54.9
1600	97.3	86.7	77.3	75.5	66.1	59.7	52.9
1500	94.7	83.7	74.9	73.2	64.4	57.8	51.0
1400	92.0	80.8	72.5	70.9	62.6	55.4	49.0
1300	89.4	77.8	70.1	68.6	60.8	52.9	47.0
1200	86.1	74.8	67.6	66.2	59.0	50.5	45.0
1100	82.3	71.9	65.2	63.9	56.0	48.1	43.0
1000	78.5	68.9	62.8	61.6	52.9	45.7	41.0
900	74.7	65.9	60.4	59.3	49.8	43.2	39.0
800	70.9	63.0	57.2	55.5	46.7	40.8	37.0
700	67.1	60.0	52.9	51.4	43.6	38.4	35.0
600	63.3	55.6	48.6	47.3	40.5	36.0	33.0
500	59.4	50.2	44.3	43.2	37.4	33.5	31.1
400	52.9	44.9	40.0	39.1	34.3	31.1	28.1
300	45.9	39.6	35.7	35.0	31.2	26.7	21.1
200	38.8	34.2	31.4	30.9	24.2	17.8	14.0
100	31.8	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

33" Effective height of rails

SUT

DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2500	148.5	132.8	118.4	115.6	101.0	90.9	81.4
2400	145.9	130.0	116.0	113.3	99.2	89.5	79.4
2300	143.3	127.1	113.6	111.0	97.4	87.9	77.4
2200	140.7	124.1	111.2	108.6	95.7	85.5	75.4
2100	138.1	121.1	108.7	106.3	93.9	83.0	73.4
2000	135.5	118.2	106.3	104.0	92.1	80.6	71.5
1900	132.8	115.2	103.9	101.7	90.3	78.2	69.5
1800	129.2	112.2	101.5	99.4	88.6	75.8	67.5
1700	125.4	109.3	97.9	97.0	85.6	73.3	65.5
1600	121.5	106.3	96.6	94.7	82.5	70.9	63.5
1500	117.7	103.4	94.2	92.4	79.4	68.5	61.5
1400	113.9	100.4	91.8	90.1	76.3	66.1	59.5
1300	110.1	97.4	89.4	87.3	73.2	63.6	57.5
1200	106.3	94.5	85.8	83.2	70.1	61.2	55.5
1100	102.5	91.5	81.5	79.1	67.0	58.8	53.5
1000	98.7	88.5	77.2	75.0	63.9	56.4	51.6
900	94.9	83.3	72.9	70.9	60.8	54.0	49.6
800	91.1	78.0	68.6	66.8	57.7	51.5	47.6
700	86.3	72.7	64.3	62.7	54.6	49.1	45.6
600	79.3	67.3	60.0	58.6	51.5	46.7	42.1
500	72.3	62.0	55.7	54.5	48.4	44.3	35.1
400	65.3	56.7	51.4	50.4	45.3	35.6	28.1
300	58.3	51.3	47.1	46.3	36.4	26.7	21.1
200	51.2	46.0	38.1	35.6	24.2	17.8	14.0
100	44.2	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

33" Effective height of rails

Table B-7. Additional Guidance Plots for Single-Unit Trucks for 34 in. Effective Height of Rails

SUT

DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
3900	150.2	130.3	117.5	114.9	99.9	89.3	81.0
3800	147.9	128.5	116.0	113.4	98.7	88.4	79.9
3700	145.6	126.8	114.5	111.8	97.4	87.4	78.7
3600	143.4	125.0	112.9	110.2	96.2	86.4	77.5
3500	141.1	123.2	111.2	108.5	94.9	85.2	76.4
3400	138.9	121.4	109.5	106.9	93.7	83.8	75.2
3300	136.6	119.7	107.8	105.3	92.4	82.4	74.1
3200	134.3	117.9	106.1	103.7	91.2	81.0	72.9
3100	132.1	116.1	104.4	102.1	89.9	79.6	71.7
3000	129.8	114.3	102.7	100.5	88.7	78.2	70.6
2900	127.6	112.3	101.1	98.8	87.4	76.8	69.4
2800	125.3	110.2	99.4	97.2	86.2	75.4	68.3
2700	123.0	108.2	97.7	95.6	84.6	74.0	67.1
2600	120.8	106.1	96.0	94.0	82.8	72.6	65.9
2500	118.5	104.1	94.3	92.4	81.0	71.2	64.8
2400	116.3	102.0	92.6	90.8	79.3	69.8	63.6
2300	113.9	100.0	90.9	89.2	77.5	68.4	62.5
2200	111.3	97.9	89.2	87.5	75.7	67.0	61.3
2100	108.7	95.9	87.6	85.9	73.9	65.6	60.1
2000	106.1	93.8	85.9	83.7	72.1	64.2	59.0
1900	103.5	91.8	83.5	81.3	70.4	62.8	57.8
1800	100.9	89.7	81.1	79.0	68.6	61.4	56.3
1700	98.3	87.7	77.6	76.7	66.8	60.0	54.3
1600	95.6	85.5	76.2	74.4	65.0	58.6	52.3
1500	93.0	82.6	73.8	72.1	63.3	57.2	50.3
1400	90.4	79.6	71.4	69.8	61.5	54.7	48.3
1300	87.8	76.6	68.9	67.4	59.7	52.3	46.4
1200	84.9	73.7	66.5	65.1	57.9	49.9	44.4
1100	81.1	70.7	64.1	62.8	55.4	47.5	42.4
1000	77.3	67.7	61.7	60.5	52.3	45.0	40.4
900	73.5	64.8	59.2	58.2	49.2	42.6	38.4
800	69.7	61.8	56.5	54.8	46.1	40.2	36.4
700	65.9	58.9	52.2	50.7	43.0	37.8	34.4
600	62.1	54.9	47.9	46.6	39.9	35.3	32.4
500	58.3	49.5	43.6	42.5	36.8	32.9	30.4
400	52.1	44.2	39.3	38.4	33.7	30.5	28.1
300	45.1	38.9	35.0	34.3	30.6	26.7	21.1
200	38.1	33.5	30.7	30.2	24.2	17.8	14.0
100	31.1	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

34" Effective height of rails

SUT

DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2500	146.1	130.4	116.7	113.9	99.3	89.3	80.5
2400	143.5	128.3	114.3	111.6	97.6	87.9	78.5
2300	140.9	125.3	111.9	109.3	95.8	86.4	76.5
2200	138.2	122.4	109.5	106.9	94.0	84.5	74.5
2100	135.6	119.4	107.0	104.6	92.2	82.1	72.5
2000	133.0	116.4	104.6	102.3	90.4	79.7	70.5
1900	130.4	113.5	102.2	100.0	88.7	77.2	68.5
1800	127.4	110.5	99.8	97.7	86.9	74.8	66.5
1700	123.6	107.5	96.2	95.4	84.6	72.4	64.6
1600	119.8	104.6	94.9	93.0	81.5	70.0	62.6
1500	116.0	101.6	92.5	90.7	78.4	67.6	60.6
1400	112.1	98.7	90.1	88.4	75.3	65.1	58.6
1300	108.3	95.7	87.7	86.1	72.2	62.7	56.6
1200	104.5	92.7	84.8	82.2	69.1	60.3	54.6
1100	100.7	89.8	80.5	78.1	66.0	57.9	52.6
1000	96.9	86.8	76.2	74.0	62.9	55.4	50.6
900	93.1	82.3	71.9	69.9	59.8	53.0	48.6
800	89.3	77.0	67.6	65.8	56.7	50.6	46.6
700	85.2	71.6	63.3	61.7	53.6	48.2	44.7
600	78.2	66.3	59.0	57.6	50.5	45.7	42.1
500	71.2	61.0	54.7	53.5	47.4	43.3	35.1
400	64.2	55.6	50.4	49.4	44.3	35.6	28.1
300	57.2	50.3	46.1	45.3	36.4	26.7	21.1
200	50.1	45.0	38.1	35.6	24.2	17.8	14.0
100	43.1	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

34" Effective height of rails

Table B-8. Additional Guidance Plots for Single-Unit Trucks for 35 in. Effective Height of Rails

SUT
DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4000	150.6	130.4	117.2	114.6	99.7	88.9	81.2
3900	148.4	128.6	115.7	113.2	98.4	87.9	80.0
3800	146.1	126.8	114.3	111.8	97.2	86.9	78.8
3700	143.9	125.0	112.8	110.3	95.9	85.9	77.7
3600	141.6	123.2	111.3	108.7	94.7	84.9	76.5
3500	139.3	121.5	109.7	107.1	93.4	83.9	75.4
3400	137.1	119.7	108.0	105.4	92.2	82.8	74.2
3300	134.8	117.9	106.3	103.8	91.0	81.4	73.0
3200	132.6	116.1	104.6	102.2	89.7	80.0	71.9
3100	130.3	114.4	102.9	100.6	88.5	78.6	70.7
3000	128.0	112.6	101.3	99.0	87.2	77.2	69.6
2900	125.8	110.8	99.6	97.4	86.0	75.8	68.4
2800	123.5	108.7	97.9	95.7	84.7	74.4	67.2
2700	121.3	106.7	96.2	94.1	83.5	73.0	66.1
2600	119.0	104.6	94.5	92.5	81.8	71.6	64.9
2500	116.7	102.6	92.8	90.9	80.0	70.2	63.8
2400	114.5	100.5	91.1	89.3	78.2	68.8	62.6
2300	112.2	98.4	89.4	87.7	76.4	67.4	61.4
2200	109.8	96.4	87.8	86.0	74.7	66.0	60.3
2100	107.2	94.3	86.1	84.4	72.9	64.6	59.1
2000	104.6	92.3	84.4	82.6	71.1	63.1	58.0
1900	101.9	90.2	82.4	80.3	69.3	61.7	56.8
1800	99.3	88.2	80.0	78.0	67.5	60.3	55.6
1700	96.7	86.1	76.5	75.6	65.8	58.9	53.7
1600	94.1	84.1	75.1	73.3	64.0	57.5	51.7
1500	91.5	81.5	72.7	71.0	62.2	56.1	49.7
1400	88.9	78.5	70.3	68.7	60.4	54.1	47.8
1300	86.3	75.5	67.9	66.4	58.7	51.7	45.8
1200	83.6	72.6	65.4	64.0	56.9	49.3	43.8
1100	80.0	69.6	63.0	61.7	54.8	46.9	41.8
1000	76.2	66.7	60.6	59.4	51.7	44.4	39.8
900	72.4	63.7	58.2	57.1	48.6	42.0	37.8
800	68.6	60.7	55.7	54.2	45.5	39.6	35.8
700	64.8	57.8	51.6	50.1	42.4	37.2	33.8
600	60.9	54.2	47.3	46.0	39.3	34.7	31.8
500	57.1	48.9	43.0	41.9	36.2	32.3	29.8
400	51.5	43.6	38.7	37.8	33.1	29.9	27.9
300	44.4	38.2	34.4	33.7	30.0	26.7	21.1
200	37.4	32.9	30.1	29.6	24.2	17.8	14.0
100	30.4	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

35" Effective height of rails

SUT
DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2600	146.4	130.2	117.6	114.6	99.5	89.1	81.6
2500	143.8	128.2	115.1	112.3	97.8	87.7	79.6
2400	141.2	126.1	112.7	110.0	96.0	86.3	77.6
2300	138.5	123.7	110.3	107.7	94.2	84.9	75.6
2200	135.9	120.7	107.9	105.3	92.4	83.5	73.6
2100	133.3	117.8	105.4	103.0	90.7	81.2	71.6
2000	130.7	114.8	103.0	100.7	88.9	78.8	69.6
1900	128.1	111.8	100.6	98.4	87.1	76.4	67.7
1800	125.5	108.9	98.2	96.1	85.3	73.9	65.7
1700	121.9	105.9	94.6	93.8	83.5	71.5	63.7
1600	118.1	103.0	93.3	91.4	80.6	69.1	61.7
1500	114.3	100.0	90.9	89.1	77.5	66.7	59.7
1400	110.5	97.0	88.5	86.8	74.4	64.2	57.7
1300	106.7	94.1	86.0	84.5	71.3	61.8	55.7
1200	102.8	91.1	83.6	81.3	68.2	59.4	53.7
1100	99.0	88.1	79.6	77.2	65.1	57.0	51.7
1000	95.2	85.2	75.3	73.1	62.0	54.5	49.7
900	91.4	81.3	71.0	69.0	58.9	52.1	47.8
800	87.6	76.0	66.7	64.9	55.8	49.7	45.8
700	83.8	70.7	62.4	60.8	52.7	47.3	43.8
600	77.2	65.3	58.1	56.7	49.6	44.8	41.8
500	70.2	60.0	53.8	52.6	46.5	42.4	35.1
400	63.1	54.7	49.5	48.5	43.4	35.6	28.1
300	56.1	49.3	45.2	44.4	36.4	26.7	21.1
200	49.1	44.0	38.1	35.6	24.2	17.8	14.0
100	42.1	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

35" Effective height of rails

Table B-9. Additional Guidance Plots for Single-Unit Trucks for 36 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4000	149.0	128.7	115.6	113.0	98.3	87.5	80.2
3900	146.7	126.9	114.1	111.6	97.0	86.5	79.0
3800	144.5	125.2	112.6	110.2	95.8	85.5	77.9
3700	142.2	123.4	111.2	108.8	94.6	84.5	76.7
3600	139.9	121.6	109.7	107.3	93.3	83.6	75.5
3500	137.7	119.8	108.2	105.7	92.1	82.6	74.4
3400	135.4	118.0	106.6	104.0	90.8	81.6	73.2
3300	133.2	116.3	104.9	102.4	89.6	80.4	72.1
3200	130.9	114.5	103.2	100.8	88.3	79.0	70.9
3100	128.6	112.7	101.5	99.2	87.1	77.6	69.8
3000	126.4	110.9	99.8	97.6	85.8	76.2	68.6
2900	124.1	109.2	98.2	96.0	84.6	74.8	67.4
2800	121.9	107.3	96.5	94.3	83.3	73.4	66.3
2700	119.6	105.2	94.8	92.7	82.1	72.0	65.1
2600	117.3	103.2	93.1	91.1	80.8	70.6	64.0
2500	115.1	101.1	91.4	89.5	79.0	69.2	62.8
2400	112.8	99.1	89.7	87.9	77.2	67.8	61.6
2300	110.6	97.0	88.0	86.3	75.4	66.4	60.5
2200	108.3	95.0	86.3	84.6	73.7	65.0	59.3
2100	105.7	92.9	84.7	83.0	71.9	63.6	58.2
2000	103.1	90.9	83.0	81.4	70.1	62.2	57.0
1900	100.5	88.8	81.3	79.3	68.3	60.8	55.8
1800	97.9	86.8	79.0	77.0	66.6	59.4	54.7
1700	95.3	84.7	75.5	74.6	64.8	58.0	53.2
1600	92.7	82.7	74.1	72.3	63.0	56.6	51.2
1500	90.0	80.4	71.7	70.0	61.2	55.2	49.2
1400	87.4	77.5	69.3	67.7	59.4	53.6	47.2
1300	84.8	74.5	66.9	65.4	57.7	51.2	45.2
1200	82.2	71.6	64.4	63.0	55.9	48.7	43.2
1100	78.9	68.6	62.0	60.7	54.1	46.3	41.2
1000	75.1	65.6	59.6	58.4	51.1	43.9	39.2
900	71.3	62.7	57.2	56.1	48.0	41.5	37.3
800	67.5	59.7	54.7	53.6	44.9	39.0	35.3
700	63.7	56.7	51.0	49.5	41.8	36.6	33.3
600	59.9	53.6	46.7	45.4	38.7	34.2	31.3
500	56.1	48.3	42.4	41.3	35.6	31.8	29.3
400	50.8	42.9	38.1	37.2	32.5	29.3	27.3
300	43.8	37.6	33.8	33.1	29.4	26.7	21.1
200	36.8	32.3	29.5	29.0	24.2	17.8	14.0
100	29.8	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

36" Effective height of rails

SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2600	144.2	128.1	116.0	113.1	98.1	87.6	80.8
2500	141.6	126.1	113.6	110.8	96.3	86.2	78.8
2400	139.0	124.0	111.2	108.5	94.5	84.8	76.8
2300	136.4	121.9	108.8	106.2	92.7	83.4	74.8
2200	133.8	119.2	106.4	103.8	91.0	82.0	72.8
2100	131.1	116.2	103.9	101.5	89.2	80.4	70.8
2000	128.5	113.3	101.5	99.2	87.4	77.9	68.8
1900	125.9	110.3	99.1	96.9	85.6	75.5	66.8
1800	123.3	107.3	96.7	94.6	83.8	73.1	64.8
1700	120.3	104.4	93.1	92.2	82.1	70.7	62.8
1600	116.5	101.4	91.8	89.9	79.8	68.2	60.9
1500	112.7	98.4	89.4	87.6	76.7	65.8	58.9
1400	108.9	95.5	87.0	85.3	73.6	63.4	56.9
1300	105.1	92.5	84.5	83.0	70.4	61.0	54.9
1200	101.3	89.6	82.1	80.4	67.3	58.5	52.9
1100	97.4	86.6	78.7	76.3	64.2	56.1	50.9
1000	93.6	83.6	74.4	72.2	61.1	53.7	48.9
900	89.8	80.4	70.1	68.1	58.0	51.3	46.9
800	86.0	75.1	65.8	64.0	54.9	48.8	44.9
700	82.2	69.7	61.5	59.9	51.8	46.4	42.9
600	76.2	64.4	57.2	55.8	48.7	44.0	41.0
500	69.2	59.1	52.9	51.7	45.6	41.6	35.1
400	62.2	53.7	48.6	47.6	42.5	35.6	28.1
300	55.2	48.4	44.3	43.5	36.4	26.7	21.1
200	48.1	43.1	38.1	35.6	24.2	17.8	14.0
100	41.1	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

36" Effective height of rails

Table B-10. Additional Guidance Plots for Single-Unit Trucks for 37 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4200	151.9	130.7	117.0	114.3	99.5	88.2	80.8
4100	149.6	128.9	115.5	112.8	98.2	87.2	80.0
4000	147.4	127.2	114.0	111.4	97.0	86.2	79.1
3900	145.1	125.4	112.6	110.0	95.7	85.2	78.1
3800	142.9	123.6	111.1	108.6	94.5	84.2	77.0
3700	140.6	121.8	109.6	107.2	93.2	83.2	75.8
3600	138.3	120.0	108.2	105.8	92.0	82.3	74.6
3500	136.1	118.3	106.7	104.3	90.8	81.3	73.5
3400	133.8	116.5	105.2	102.7	89.5	80.3	72.3
3300	131.6	114.7	103.6	101.1	88.3	79.3	71.2
3200	129.3	112.9	101.9	99.5	87.0	78.1	70.0
3100	127.0	111.2	100.2	97.9	85.8	76.7	68.8
3000	124.8	109.4	98.5	96.2	84.5	75.3	67.7
2900	122.5	107.6	96.8	94.6	83.3	73.9	66.5
2800	120.3	105.8	95.1	93.0	82.0	72.5	65.4
2700	118.0	103.9	93.5	91.4	80.8	71.1	64.2
2600	115.8	101.8	91.8	89.8	79.5	69.7	63.0
2500	113.5	99.8	90.1	88.2	78.1	68.3	61.9
2400	111.2	97.7	88.4	86.6	76.3	66.9	60.7
2300	109.0	95.7	86.7	84.9	74.5	65.5	59.6
2200	106.7	93.6	85.0	83.3	72.7	64.1	58.4
2100	104.4	91.6	83.3	81.7	71.0	62.7	57.2
2000	101.7	89.5	81.6	80.1	69.2	61.3	56.1
1900	99.1	87.5	80.0	78.3	67.4	59.9	54.9
1800	96.5	85.4	78.0	76.0	65.6	58.4	53.8
1700	93.9	83.4	74.5	73.7	63.8	57.0	52.6
1600	91.3	81.3	73.2	71.4	62.1	55.6	50.7
1500	88.7	79.3	70.8	69.0	60.3	54.2	48.7
1400	86.1	76.5	68.3	66.7	58.5	52.8	46.7
1300	83.4	73.5	65.9	64.4	56.7	50.6	44.7
1200	80.8	70.6	63.5	62.1	55.0	48.2	42.7
1100	77.9	67.6	61.1	59.8	53.2	45.8	40.7
1000	74.1	64.7	58.6	57.5	50.6	43.3	38.7
900	70.3	61.7	56.2	55.1	47.5	40.9	36.7
800	66.5	58.7	53.8	52.8	44.4	38.5	34.7
700	62.7	55.8	50.4	48.9	41.3	36.1	32.8
600	58.9	52.8	46.1	44.8	38.2	33.7	30.8
500	55.1	47.7	41.8	40.7	35.1	31.2	28.8
400	50.2	42.3	37.5	36.6	32.0	28.8	26.8
300	43.2	37.0	33.2	32.5	28.9	26.4	21.1
200	36.2	31.7	28.9	28.4	24.2	17.8	14.0
100	29.1	26.3	19.0	17.8	12.1	8.9	7.0

Post Spacing

37" Effective height of rails

SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2700	144.8	128.1	117.0	114.0	98.4	87.7	80.6
2600	142.1	126.1	114.6	111.7	96.7	86.3	79.5
2500	139.5	124.0	112.2	109.4	94.9	84.9	78.0
2400	136.9	122.0	109.8	107.1	93.1	83.5	76.0
2300	134.3	119.9	107.3	104.7	91.3	82.1	74.0
2200	131.7	117.7	104.9	102.4	89.6	80.7	72.0
2100	129.1	114.8	102.5	100.1	87.8	79.2	70.0
2000	126.5	111.8	100.1	97.8	86.0	77.1	68.0
1900	123.8	108.8	97.6	95.5	84.2	74.7	66.0
1800	121.2	105.9	95.2	93.1	82.4	72.3	64.1
1700	118.6	102.9	91.7	90.8	80.7	69.9	62.1
1600	115.0	99.9	90.4	88.5	78.9	67.4	60.1
1500	111.2	97.0	87.9	86.2	75.8	65.0	58.1
1400	107.4	94.0	85.5	83.9	72.7	62.6	56.1
1300	103.6	91.1	83.1	81.5	69.6	60.2	54.1
1200	99.8	88.1	80.7	79.2	66.5	57.7	52.1
1100	95.9	85.1	77.8	75.4	63.4	55.3	50.1
1000	92.1	82.2	73.5	71.3	60.3	52.9	48.1
900	88.3	79.2	69.2	67.2	57.2	50.5	46.1
800	84.5	74.2	64.9	63.1	54.1	48.1	44.2
700	80.7	68.9	60.6	59.0	51.0	45.6	42.2
600	75.3	63.5	56.3	54.9	47.9	43.2	40.2
500	68.3	58.2	52.0	50.8	44.8	40.8	35.1
400	61.3	52.9	47.7	46.7	41.7	35.6	28.1
300	54.2	47.5	43.4	42.6	36.4	26.7	21.1
200	47.2	42.2	38.1	35.6	24.2	17.8	14.0
100	40.2	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

37" Effective height of rails

Table B-11. Additional Guidance Plots for Single-Unit Trucks for 38 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4300	152.7	131.0	117.0	114.2	99.5	87.9	80.4
4200	150.4	129.2	115.5	112.8	98.2	87.0	79.6
4100	148.1	127.5	114.0	111.4	97.0	86.0	78.7
4000	145.9	125.7	112.6	110.0	95.7	85.0	77.9
3900	143.6	123.9	111.1	108.6	94.5	84.0	77.1
3800	141.4	122.1	109.6	107.2	93.3	83.0	76.1
3700	139.1	120.3	108.2	105.8	92.0	82.0	74.9
3600	136.8	118.6	106.7	104.4	90.8	81.0	73.8
3500	134.6	116.8	105.3	103.0	89.5	80.0	72.6
3400	132.3	115.0	103.8	101.5	88.3	79.1	71.4
3300	130.1	113.2	102.3	99.8	87.0	78.1	70.3
3200	127.8	111.5	100.6	98.2	85.8	77.1	69.1
3100	125.5	109.7	98.9	96.6	84.5	75.8	68.0
3000	123.3	107.9	97.3	95.0	83.3	74.4	66.8
2900	121.0	106.1	95.6	93.4	82.0	73.0	65.7
2800	118.8	104.3	93.9	91.8	80.8	71.6	64.5
2700	116.5	102.6	92.2	90.1	79.5	70.2	63.3
2600	114.3	100.6	90.5	88.5	78.3	68.8	62.2
2500	112.0	98.5	88.8	86.9	77.1	67.4	61.0
2400	109.7	96.5	87.1	85.3	75.4	66.0	59.9
2300	107.5	94.4	85.4	83.7	73.6	64.6	58.7
2200	105.2	92.4	83.8	82.1	71.9	63.2	57.5
2100	103.0	90.3	82.1	80.4	70.1	61.8	56.4
2000	100.4	88.2	80.4	78.8	68.3	60.4	55.2
1900	97.8	86.2	78.7	77.2	66.5	59.0	54.1
1800	95.2	84.1	77.0	75.1	64.7	57.6	52.9
1700	92.6	82.1	73.6	72.8	63.0	56.2	51.7
1600	90.0	80.0	72.3	70.5	61.2	54.8	50.2
1500	87.4	78.0	69.8	68.1	59.4	53.4	48.2
1400	84.7	75.6	67.4	65.8	57.6	52.0	46.2
1300	82.1	72.6	65.0	63.5	55.9	50.1	44.2
1200	79.5	69.7	62.6	61.2	54.1	47.7	42.2
1100	76.9	66.7	60.2	58.9	52.3	45.3	40.2
1000	73.2	63.7	57.7	56.6	50.0	42.8	38.2
900	69.4	60.8	55.3	54.2	46.9	40.4	36.2
800	65.6	57.8	52.9	51.9	43.8	38.0	34.2
700	61.7	54.8	49.9	48.4	40.7	35.6	32.3
600	57.9	51.9	45.6	44.3	37.6	33.1	30.3
500	54.1	47.1	41.3	40.2	34.5	30.7	28.3
400	49.6	41.8	37.0	36.1	31.4	28.3	26.3
300	42.6	36.5	32.7	32.0	28.3	25.9	21.1
200	35.6	31.1	28.4	27.9	24.2	17.8	14.0
100	28.6	25.8	19.0	17.8	12.1	8.9	7.0

Post Spacing

38" Effective height of rails

SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2800	145.4	128.3	117.2	115.0	98.9	87.8	80.5
2700	142.8	126.2	115.5	112.7	97.1	86.4	79.3
2600	140.2	124.2	113.3	110.3	95.3	85.0	78.2
2500	137.6	122.1	110.8	108.0	93.6	83.6	77.0
2400	135.0	120.1	108.4	105.7	91.8	82.1	75.2
2300	132.4	118.0	106.0	103.4	90.0	80.7	73.3
2200	129.7	116.0	103.6	101.1	88.2	79.3	71.3
2100	127.1	113.4	101.1	98.7	86.4	77.9	69.3
2000	124.5	110.4	98.7	96.4	84.7	76.4	67.3
1900	121.9	107.5	96.3	94.1	82.9	74.0	65.3
1800	119.3	104.5	93.9	91.8	81.1	71.5	63.3
1700	116.7	101.5	90.3	89.5	79.3	69.1	61.3
1600	113.6	98.6	89.0	87.1	77.6	66.7	59.3
1500	109.8	95.6	86.6	84.8	75.1	64.3	57.3
1400	106.0	92.6	84.2	82.5	72.0	61.8	55.3
1300	102.1	89.7	81.7	80.2	68.9	59.4	53.4
1200	98.3	86.7	79.3	77.9	65.8	57.0	51.4
1100	94.5	83.8	76.9	74.6	62.7	54.6	49.4
1000	90.7	80.8	72.7	70.5	59.6	52.1	47.4
900	86.9	77.8	68.4	66.4	56.5	49.7	45.4
800	83.1	73.4	64.1	62.3	53.4	47.3	43.4
700	79.3	68.0	59.8	58.2	50.3	44.9	41.4
600	74.4	62.7	55.5	54.1	47.2	42.4	39.4
500	67.4	57.4	51.2	50.0	44.1	40.0	35.1
400	60.4	52.0	46.9	45.9	41.0	35.6	28.1
300	53.4	46.7	42.6	41.8	36.4	26.7	21.1
200	46.3	41.4	38.1	35.6	24.2	17.8	14.0
100	39.3	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

38" Effective height of rails

Table B-12. Additional Guidance Plots for Single-Unit Trucks for 39 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4400	153.5	131.4	117.1	114.2	99.6	87.8	80.0
4300	151.2	129.6	115.6	112.8	98.3	86.8	79.2
4200	149.0	127.8	114.1	111.4	97.1	85.8	78.4
4100	146.7	126.1	112.7	110.0	95.8	84.8	77.6
4000	144.5	124.3	111.2	108.6	94.6	83.8	76.8
3900	142.2	122.5	109.7	107.2	93.3	82.8	75.9
3800	139.9	120.7	108.3	105.8	92.1	81.8	75.1
3700	137.7	118.9	106.8	104.4	90.8	80.9	74.1
3600	135.4	117.2	105.3	103.0	89.6	79.9	72.9
3500	133.2	115.4	103.9	101.6	88.3	78.9	71.8
3400	130.9	113.6	102.4	100.2	87.1	77.9	70.6
3300	128.6	111.8	100.9	98.6	85.8	76.9	69.5
3200	126.4	110.1	99.4	97.0	84.6	75.9	68.3
3100	124.1	108.3	97.8	95.4	83.4	74.9	67.2
3000	121.9	106.5	96.1	93.8	82.1	73.6	66.0
2900	119.6	104.7	94.4	92.2	80.9	72.2	64.8
2800	117.3	102.9	92.7	90.6	79.6	70.8	63.7
2700	115.1	101.2	91.0	89.0	78.4	69.4	62.5
2600	112.8	99.3	89.3	87.3	77.1	68.0	61.4
2500	110.6	97.3	87.6	85.7	75.9	66.6	60.2
2400	108.3	95.2	85.9	84.1	74.6	65.2	59.0
2300	106.0	93.2	84.2	82.5	72.8	63.8	57.9
2200	103.8	91.1	82.6	80.9	71.0	62.4	56.7
2100	101.5	89.1	80.9	79.3	69.2	61.0	55.6
2000	99.2	87.0	79.2	77.6	67.5	59.6	54.4
1900	96.6	85.0	77.5	76.0	65.7	58.1	53.2
1800	94.0	82.9	75.8	74.2	63.9	56.7	52.1
1700	91.4	80.9	72.8	71.9	62.1	55.3	50.9
1600	88.7	78.8	71.4	69.6	60.3	53.9	49.7
1500	86.1	76.8	69.0	67.3	58.6	52.5	47.7
1400	83.5	74.7	66.6	65.0	56.8	51.1	45.7
1300	80.9	71.7	64.1	62.7	55.0	49.6	43.7
1200	78.3	68.8	61.7	60.3	53.2	47.2	41.7
1100	75.7	65.8	59.3	58.0	51.5	44.8	39.7
1000	72.3	62.9	56.9	55.7	49.6	42.4	37.8
900	68.5	59.9	54.4	53.4	46.5	39.9	35.8
800	64.7	56.9	52.0	51.1	43.4	37.5	33.8
700	60.8	54.0	49.4	47.9	40.3	35.1	31.8
600	57.0	51.0	45.1	43.8	37.2	32.7	29.8
500	53.2	46.6	40.8	39.7	34.1	30.2	27.8
400	49.1	41.3	36.5	35.6	31.0	27.8	25.8
300	42.0	35.9	32.2	31.5	27.8	25.4	21.1
200	35.0	30.6	27.9	27.4	24.2	17.8	14.0
100	28.0	25.3	19.0	17.8	12.1	8.9	7.0

Post Spacing

39" Effective height of rails

SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
2900	146.2	128.5	117.1	114.8	99.4	87.9	80.4
2800	143.6	126.5	115.4	113.2	97.6	86.5	79.3
2700	141.0	124.4	113.7	111.4	95.9	85.1	78.1
2600	138.3	122.4	112.0	109.1	94.1	83.7	77.0
2500	135.7	120.3	109.5	106.7	92.3	82.3	75.8
2400	133.1	118.2	107.1	104.4	90.5	80.9	74.5
2300	130.5	116.2	104.7	102.1	88.7	79.5	72.6
2200	127.9	114.1	102.3	99.8	87.0	78.1	70.6
2100	125.3	112.1	99.8	97.5	85.2	76.7	68.6
2000	122.7	109.1	97.4	95.1	83.4	75.3	66.6
1900	120.0	106.1	95.0	92.8	81.6	73.2	64.6
1800	117.4	103.2	92.6	90.5	79.9	70.8	62.6
1700	114.8	100.2	89.0	88.2	78.1	68.4	60.6
1600	112.2	97.3	87.7	85.9	76.3	66.0	58.6
1500	108.4	94.3	85.3	83.5	74.3	63.6	56.6
1400	104.6	91.3	82.9	81.2	71.2	61.1	54.6
1300	100.8	88.4	80.5	78.9	68.1	58.7	52.7
1200	97.0	85.4	78.0	76.6	65.0	56.3	50.7
1100	93.2	82.4	75.6	73.9	61.9	53.9	48.7
1000	89.4	79.5	72.0	69.8	58.8	51.4	46.7
900	85.6	76.5	67.7	65.7	55.7	49.0	44.7
800	81.7	72.6	63.4	61.6	52.6	46.6	42.7
700	77.9	67.2	59.1	57.5	49.5	44.2	40.7
600	73.6	61.9	54.7	53.4	46.4	41.7	38.7
500	66.6	56.6	50.4	49.3	43.3	39.3	35.1
400	59.6	51.2	46.1	45.2	40.2	35.6	28.1
300	52.5	45.9	41.8	41.1	36.4	26.7	21.1
200	45.5	40.6	37.5	35.6	24.2	17.8	14.0
100	38.5	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

39" Effective height of rails

Table B-13. Additional Guidance Plots for Single-Unit Trucks for 40 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4600	156.7	133.6	118.7	115.7	100.4	88.6	80.6
4500	154.4	131.8	117.2	114.3	99.4	87.6	79.7
4400	152.2	130.1	115.7	112.9	98.3	86.7	78.9
4300	149.9	128.3	114.3	111.5	97.2	85.7	78.1
4200	147.6	126.5	112.8	110.1	95.9	84.7	77.3
4100	145.4	124.7	111.3	108.7	94.7	83.7	76.5
4000	143.1	123.0	109.9	107.3	93.4	82.7	75.7
3900	140.9	121.2	108.4	105.9	92.2	81.7	74.8
3800	138.6	119.4	106.9	104.5	91.0	80.7	74.0
3700	136.3	117.6	105.5	103.1	89.7	79.7	73.2
3600	134.1	115.8	104.0	101.7	88.5	78.8	72.2
3500	131.8	114.1	102.6	100.3	87.2	77.8	71.0
3400	129.6	112.3	101.1	98.9	86.0	76.8	69.8
3300	127.3	110.5	99.6	97.5	84.7	75.8	68.7
3200	125.0	108.7	98.2	95.9	83.5	74.8	67.5
3100	122.8	107.0	96.6	94.3	82.2	73.8	66.4
3000	120.5	105.2	94.9	92.7	81.0	72.8	65.2
2900	118.3	103.4	93.2	91.1	79.7	71.4	64.1
2800	116.0	101.6	91.6	89.4	78.5	70.0	62.9
2700	113.7	99.8	89.9	87.8	77.2	68.6	61.7
2600	111.5	98.1	88.2	86.2	76.0	67.2	60.6
2500	109.2	96.1	86.5	84.6	74.8	65.8	59.4
2400	107.0	94.1	84.8	83.0	73.5	64.4	58.3
2300	104.7	92.0	83.1	81.4	72.0	63.0	57.1
2200	102.4	90.0	81.4	79.7	70.2	61.6	55.9
2100	100.2	87.9	79.7	78.1	68.4	60.2	54.8
2000	97.9	85.9	78.0	76.5	66.7	58.8	53.6
1900	95.4	83.8	76.4	74.9	64.9	57.4	52.5
1800	92.8	81.8	74.7	73.3	63.1	56.0	51.3
1700	90.2	79.7	72.0	71.1	61.3	54.6	50.1
1600	87.6	77.7	70.6	68.8	59.5	53.2	49.0
1500	85.0	75.6	68.2	66.5	57.8	51.7	47.3
1400	82.3	73.6	65.8	64.2	56.0	50.3	45.3
1300	79.7	70.9	63.3	61.8	54.2	48.9	43.3
1200	77.1	68.0	60.9	59.5	52.4	46.8	41.3
1100	74.5	65.0	58.5	57.2	50.7	44.3	39.3
1000	71.4	62.0	56.1	54.9	48.9	41.9	37.3
900	67.6	59.1	53.6	52.6	46.0	39.5	35.3
800	63.8	56.1	51.2	50.2	42.9	37.1	33.3
700	60.0	53.1	48.8	47.4	39.8	34.6	31.3
600	56.2	50.2	44.6	43.3	36.7	32.2	29.3
500	52.4	46.1	40.3	39.2	33.6	29.8	27.4
400	48.5	40.8	36.0	35.1	30.5	27.4	25.4
300	41.5	35.4	31.7	31.0	27.4	24.9	21.1
200	34.5	30.1	27.4	26.9	24.2	17.8	14.0
100	27.5	24.8	19.0	17.8	12.1	8.9	7.0

Post Spacing

40" Effective height of rails

SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
3000	146.9	128.8	117.1	114.8	100.0	88.1	80.4
2900	144.4	126.8	115.4	113.1	98.2	86.7	79.3
2800	141.8	124.7	113.7	111.5	96.4	85.3	78.1
2700	139.2	122.7	112.0	109.9	94.7	83.9	76.9
2600	136.6	120.6	110.3	107.8	92.9	82.5	75.8
2500	134.0	118.6	108.3	105.5	91.1	81.1	74.6
2400	131.4	116.5	105.9	103.2	89.3	79.7	73.5
2300	128.7	114.5	103.5	100.9	87.5	78.3	71.9
2200	126.1	112.4	101.0	98.6	85.8	76.9	69.9
2100	123.5	110.4	98.6	96.2	84.0	75.5	67.9
2000	120.9	107.9	96.2	93.9	82.2	74.1	65.9
1900	118.3	104.9	93.8	91.6	80.4	72.6	63.9
1800	115.7	101.9	91.4	89.3	78.7	70.1	61.9
1700	113.1	99.0	87.8	87.0	76.9	67.7	59.9
1600	110.4	96.0	86.5	84.6	75.1	65.3	58.0
1500	107.1	93.0	84.1	82.3	73.3	62.9	56.0
1400	103.3	90.1	81.7	80.0	70.5	60.4	54.0
1300	99.5	87.1	79.2	77.7	67.4	58.0	52.0
1200	95.7	84.2	76.8	75.4	64.3	55.6	50.0
1100	91.9	81.2	74.4	73.1	61.2	53.2	48.0
1000	88.1	78.2	71.2	69.1	58.1	50.8	46.0
900	84.3	75.3	66.9	65.0	55.0	48.3	44.0
800	80.5	71.8	62.6	60.9	51.9	45.9	42.0
700	76.7	66.5	58.3	56.8	48.8	43.5	40.0
600	72.8	61.2	54.0	52.7	45.7	41.1	38.1
500	65.8	55.8	49.7	48.6	42.6	38.6	35.1
400	58.8	50.5	45.4	44.4	39.5	35.6	28.1
300	51.7	45.2	41.1	40.3	36.4	26.7	21.1
200	44.7	39.8	36.8	35.6	24.2	17.8	14.0
100	37.7	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

40" Effective height of rails

Table B-14. Additional Guidance Plots for Single-Unit Trucks for 41 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4700	157.6	134.1	118.9	115.9	100.3	88.6	80.3
4600	155.4	132.4	117.4	114.5	99.2	87.6	79.5
4500	153.1	130.6	116.0	113.1	98.1	86.6	78.7
4400	150.9	128.8	114.5	111.7	97.0	85.6	77.9
4300	148.6	127.0	113.0	110.3	96.0	84.6	77.1
4200	146.3	125.2	111.6	108.9	94.9	83.6	76.2
4100	144.1	123.5	110.1	107.5	93.6	82.6	75.4
4000	141.8	121.7	108.6	106.1	92.4	81.7	74.6
3900	139.6	119.9	107.2	104.6	91.1	80.7	73.8
3800	137.3	118.1	105.7	103.2	89.9	79.7	73.0
3700	135.0	116.4	104.2	101.8	88.6	78.7	72.2
3600	132.8	114.6	102.8	100.4	87.4	77.7	71.3
3500	130.5	112.8	101.3	99.0	86.2	76.7	70.3
3400	128.3	111.0	99.8	97.6	84.9	75.7	69.1
3300	126.0	109.2	98.4	96.2	83.7	74.7	67.9
3200	123.7	107.5	96.9	94.8	82.4	73.8	66.8
3100	121.5	105.7	95.4	93.2	81.2	72.8	65.6
3000	119.2	103.9	93.8	91.6	79.9	71.8	64.5
2900	117.0	102.1	92.2	90.0	78.7	70.6	63.3
2800	114.7	100.4	90.5	88.4	77.4	69.2	62.1
2700	112.4	98.6	88.8	86.7	76.2	67.8	61.0
2600	110.2	96.8	87.1	85.1	74.9	66.4	59.8
2500	107.9	95.0	85.4	83.5	73.7	65.0	58.7
2400	105.7	93.0	83.7	81.9	72.4	63.6	57.5
2300	103.4	90.9	82.0	80.3	71.2	62.2	56.4
2200	101.2	88.9	80.3	78.7	69.5	60.8	55.2
2100	98.9	86.8	78.7	77.0	67.7	59.4	54.0
2000	96.6	84.8	77.0	75.4	65.9	58.0	52.9
1900	94.3	82.7	75.3	73.8	64.1	56.6	51.7
1800	91.7	80.7	73.6	72.2	62.3	55.2	50.6
1700	89.1	78.6	71.2	70.3	60.6	53.8	49.4
1600	86.5	76.6	69.8	68.0	58.8	52.4	48.2
1500	83.8	74.5	67.4	65.7	57.0	51.0	46.8
1400	81.2	72.5	65.0	63.4	55.2	49.6	44.8
1300	78.6	70.1	62.6	61.1	53.5	48.2	42.9
1200	76.0	67.2	60.1	58.7	51.7	46.3	40.9
1100	73.4	64.2	57.7	56.4	49.9	43.9	38.9
1000	70.6	61.2	55.3	54.1	48.1	41.5	36.9
900	66.8	58.3	52.9	51.8	45.5	39.1	34.9
800	63.0	55.3	50.4	49.5	42.4	36.6	32.9
700	59.2	52.4	48.0	47.0	39.3	34.2	30.9
600	55.4	49.4	44.2	42.9	36.2	31.8	28.9
500	51.6	45.6	39.9	38.7	33.1	29.4	26.9
400	47.8	40.3	35.6	34.6	30.0	26.9	24.9
300	41.0	35.0	31.3	30.5	26.9	24.5	21.1
200	34.0	29.6	27.0	26.4	23.8	17.8	14.0
100	27.0	24.3	19.0	17.8	12.1	8.9	7.0

Post Spacing

41" Effective height of rails

SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
3100	147.2	129.2	117.1	114.8	100.6	88.4	80.5
3000	144.9	127.2	115.5	113.1	98.9	87.0	79.3
2900	142.7	125.1	113.8	111.5	97.1	85.6	78.2
2800	140.1	123.1	112.1	109.9	95.3	84.2	77.0
2700	137.5	121.0	110.4	108.3	93.5	82.8	75.8
2600	134.9	119.0	108.7	106.7	91.7	81.4	74.7
2500	132.3	116.9	107.0	104.4	90.0	80.0	73.5
2400	129.7	114.9	104.7	102.0	88.2	78.6	72.4
2300	127.1	112.8	102.3	99.7	86.4	77.2	71.2
2200	124.5	110.8	99.9	97.4	84.6	75.8	69.3
2100	121.8	108.7	97.5	95.1	82.9	74.4	67.3
2000	119.2	106.7	95.0	92.8	81.1	73.0	65.3
1900	116.6	103.7	92.6	90.4	79.3	71.6	63.3
1800	114.0	100.7	90.2	88.1	77.5	69.5	61.3
1700	111.4	97.8	86.7	85.8	75.7	67.1	59.3
1600	108.8	94.8	85.3	83.5	74.0	64.7	57.3
1500	105.9	91.9	82.9	81.2	72.2	62.2	55.3
1400	102.1	88.9	80.5	78.8	69.9	59.8	53.3
1300	98.3	85.9	78.1	76.5	66.8	57.4	51.3
1200	94.5	83.0	75.6	74.2	63.7	55.0	49.4
1100	90.7	80.0	73.2	71.9	60.6	52.5	47.4
1000	86.9	77.0	70.5	68.4	57.5	50.1	45.4
900	83.1	74.1	66.2	64.3	54.4	47.7	43.4
800	79.2	71.1	61.9	60.2	51.3	45.3	41.4
700	75.4	65.8	57.6	56.1	48.2	42.8	39.4
600	71.6	60.4	53.3	52.0	45.1	40.4	37.4
500	65.0	55.1	49.0	47.9	42.0	38.0	35.1
400	58.0	49.8	44.7	43.8	38.9	35.6	28.1
300	51.0	44.4	40.4	39.7	35.8	26.7	21.1
200	44.0	39.1	36.1	35.6	24.2	17.8	14.0
100	37.0	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

41" Effective height of rails

Table B-15. Additional Guidance Plots for Single-Unit Trucks for 42 in. Effective Height of Rails

SUT DMF=1.0

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
4800	158.7	134.7	119.2	116.1	100.2	88.6	80.2
4700	156.4	132.9	117.7	114.7	99.1	87.6	79.3
4600	154.2	131.1	116.2	113.3	98.0	86.6	78.5
4500	151.9	129.4	114.8	111.9	96.9	85.6	77.7
4400	149.6	127.6	113.3	110.5	95.9	84.6	76.9
4300	147.4	125.8	111.8	109.1	94.8	83.6	76.1
4200	145.1	124.0	110.4	107.7	93.7	82.6	75.2
4100	142.9	122.3	108.9	106.3	92.6	81.6	74.4
4000	140.6	120.5	107.4	104.9	91.4	80.6	73.6
3900	138.3	118.7	106.0	103.5	90.1	79.7	72.8
3800	136.1	116.9	104.5	102.1	88.9	78.7	72.0
3700	133.8	115.1	103.0	100.7	87.6	77.7	71.2
3600	131.6	113.4	101.6	99.2	86.4	76.7	70.3
3500	129.3	111.6	100.1	97.8	85.1	75.7	69.5
3400	127.0	109.8	98.6	96.4	83.9	74.7	68.4
3300	124.8	108.0	97.2	95.0	82.6	73.7	67.2
3200	122.5	106.3	95.7	93.6	81.4	72.7	66.1
3100	120.3	104.5	94.3	92.2	80.2	71.8	64.9
3000	118.0	102.7	92.8	90.6	78.9	70.8	63.8
2900	115.7	100.9	91.1	88.9	77.7	69.8	62.6
2800	113.5	99.1	89.4	87.3	76.4	68.5	61.4
2700	111.2	97.4	87.8	85.7	75.2	67.1	60.3
2600	109.0	95.6	86.1	84.1	73.9	65.7	59.1
2500	106.7	93.8	84.4	82.5	72.7	64.3	58.0
2400	104.4	92.0	82.7	80.9	71.4	62.9	56.8
2300	102.2	89.9	81.0	79.2	70.2	61.5	55.6
2200	99.9	87.9	79.3	77.6	68.7	60.1	54.5
2100	97.7	85.8	77.6	76.0	67.0	58.7	53.3
2000	95.4	83.8	75.9	74.4	65.2	57.3	52.2
1900	93.1	81.7	74.3	72.8	63.4	55.9	51.0
1800	90.6	79.6	72.6	71.2	61.6	54.5	49.8
1700	88.0	77.6	70.2	69.6	59.8	53.1	48.7
1600	85.4	75.5	69.1	67.3	58.1	51.7	47.5
1500	82.8	73.5	66.7	65.0	56.3	50.3	46.4
1400	80.2	71.4	64.2	62.7	54.5	48.9	44.4
1300	77.5	69.4	61.8	60.3	52.7	47.5	42.5
1200	74.9	66.4	59.4	58.0	51.0	45.9	40.5
1100	72.3	63.4	57.0	55.7	49.2	43.5	38.5
1000	69.7	60.5	54.5	53.4	47.4	41.1	36.5
900	66.0	57.5	52.1	51.1	45.1	38.6	34.5
800	62.2	54.6	49.7	48.7	42.0	36.2	32.5
700	58.4	51.6	47.3	46.4	38.9	33.8	30.5
600	54.6	48.6	43.7	42.4	35.8	31.4	28.5
500	50.8	45.2	39.4	38.3	32.7	29.0	26.5
400	47.0	39.8	35.1	34.2	29.6	26.5	24.5
300	40.5	34.5	30.8	30.1	26.5	24.1	21.1
200	33.5	29.2	26.5	26.0	23.4	17.8	14.0
100	26.5	23.8	19.0	17.8	12.1	8.9	7.0

Post Spacing

42" Effective height of rails

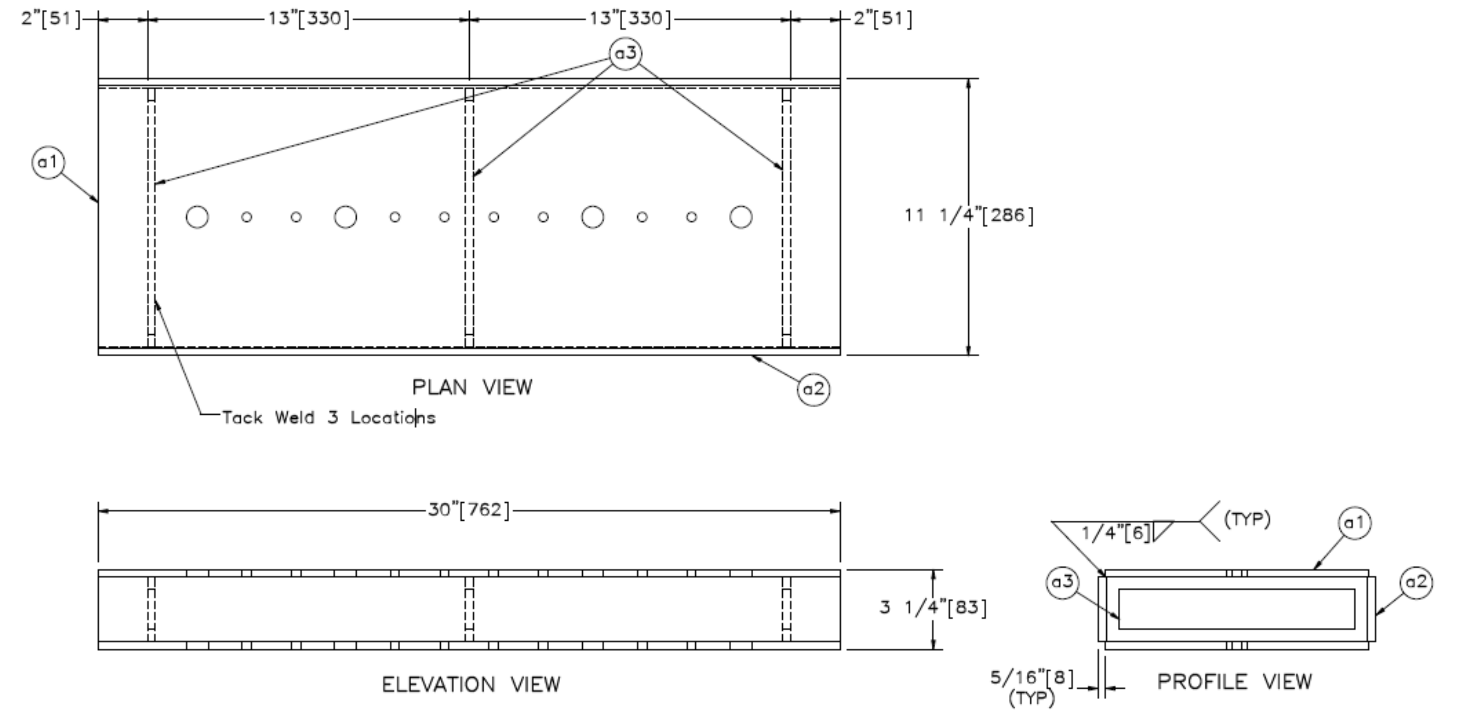
SUT DMF=1.5

Σ Mp (kips - in)	48 (4 ft)	60 (5 ft)	72 (6 ft)	75 (6 ft - 3 in)	96 (8 ft)	120 (10 ft)	144 (12 ft)
3200	147.6	129.7	117.3	114.8	101.3	88.8	80.6
3100	145.4	127.7	115.6	113.2	99.5	87.4	79.4
3000	143.1	125.6	113.9	111.6	97.8	86.0	78.3
2900	140.8	123.6	112.2	110.0	96.0	84.6	77.1
2800	138.5	121.5	110.5	108.4	94.2	83.1	75.9
2700	135.9	119.5	108.8	106.8	92.4	81.7	74.8
2600	133.3	117.4	107.2	105.1	90.7	80.3	73.6
2500	130.7	115.4	105.5	103.3	88.9	78.9	72.5
2400	128.1	113.3	103.6	100.9	87.1	77.5	71.3
2300	125.5	111.3	101.2	98.6	85.3	76.1	70.1
2200	122.9	109.2	98.8	96.3	83.5	74.7	68.7
2100	120.2	107.2	96.4	94.0	81.8	73.3	66.7
2000	117.6	105.1	93.9	91.7	80.0	71.9	64.7
1900	115.0	102.6	91.5	89.3	78.2	70.5	62.7
1800	112.4	99.6	89.1	87.0	76.4	68.9	60.7
1700	109.8	96.7	85.6	84.7	74.7	66.5	58.7
1600	107.2	93.7	84.2	82.4	72.9	64.0	56.7
1500	104.6	90.7	81.8	80.1	71.1	61.6	54.7
1400	100.9	87.8	79.4	77.7	69.2	59.2	52.7
1300	97.1	84.8	77.0	75.4	66.1	56.8	50.7
1200	93.3	81.8	74.5	73.1	63.0	54.3	48.8
1100	89.5	78.9	72.1	70.8	59.9	51.9	46.8
1000	85.7	75.9	69.7	67.7	56.8	49.5	44.8
900	81.9	73.0	65.6	63.6	53.7	47.1	42.8
800	78.1	70.0	61.3	59.5	50.6	44.6	40.8
700	74.3	65.1	57.0	55.4	47.5	42.2	38.8
600	70.5	59.8	52.7	51.3	44.4	39.8	36.8
500	64.3	54.4	48.4	47.2	41.3	37.4	34.8
400	57.3	49.1	44.1	43.1	38.2	34.9	28.1
300	50.3	43.8	39.8	39.0	35.1	26.7	21.1
200	43.3	38.4	35.5	34.9	24.2	17.8	14.0
100	36.3	26.7	19.0	17.8	12.1	8.9	7.0

Post Spacing

42" Effective height of rails

Appendix C. Midwest Steel Works Inc. Splice Fabrication Drawings



Item No.	QTY.	Description	Material Specification	Treatment Specification
a1	2	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	ASTM A123
a2	2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	ASTM A123
a3	3	10 1/2"x2 5/8"x5/16" [267x67x8] Plate	ASTM A572 Gr. 50	ASTM A123

Midwest Roadside Safety Facility

OH–IL STBR Splice Fabrication

Upper Splice Tube Assembly

SHEET: 1 of 4

DATE: 01/29/2019

DRAWN BY: DJW

DWG. NAME. OH–IL_STBR_Splices_R0

SCALE: 1:6

UNITS: in.[mm]

REV. BY: –

Figure C-1. Upper Splice Tube Assembly

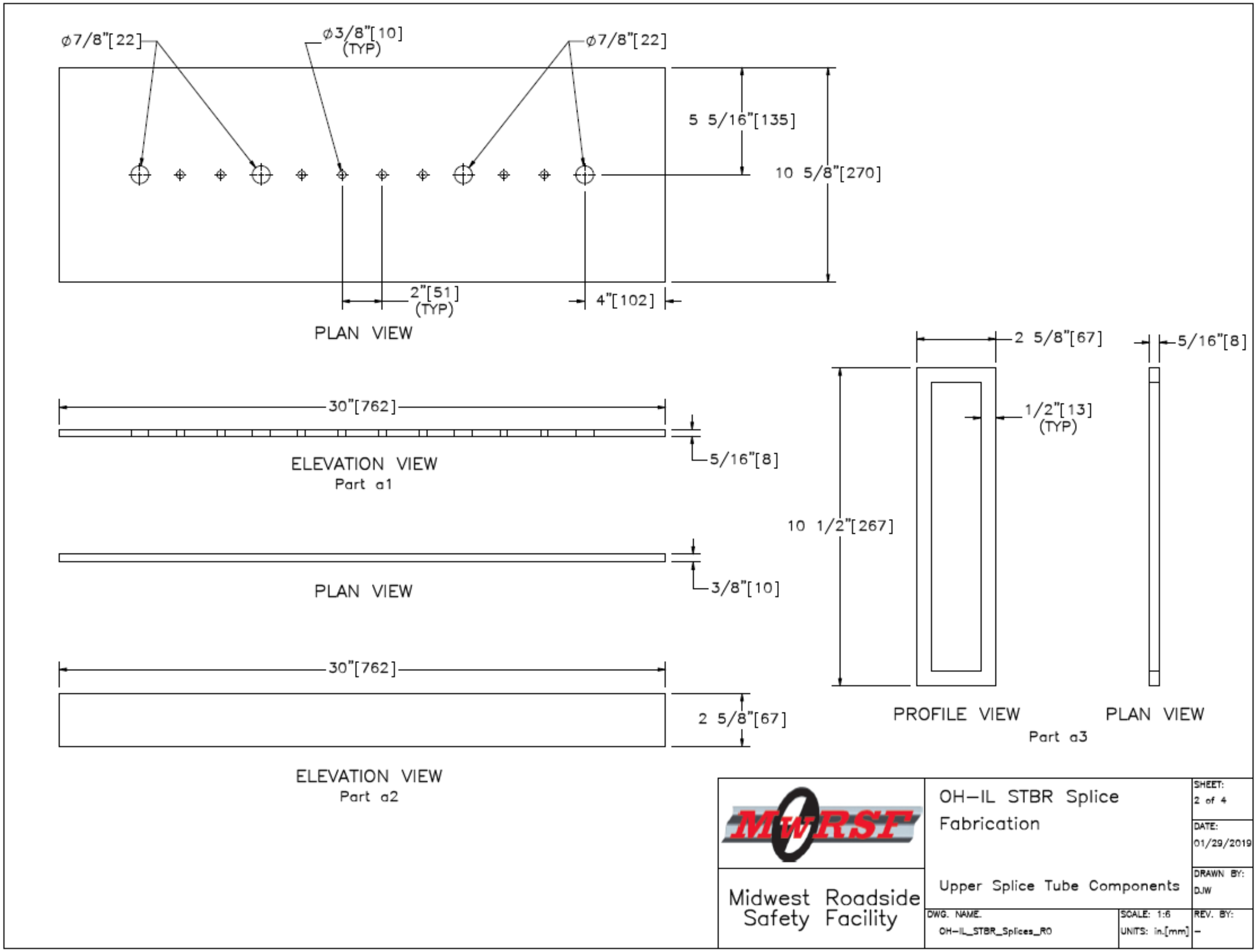
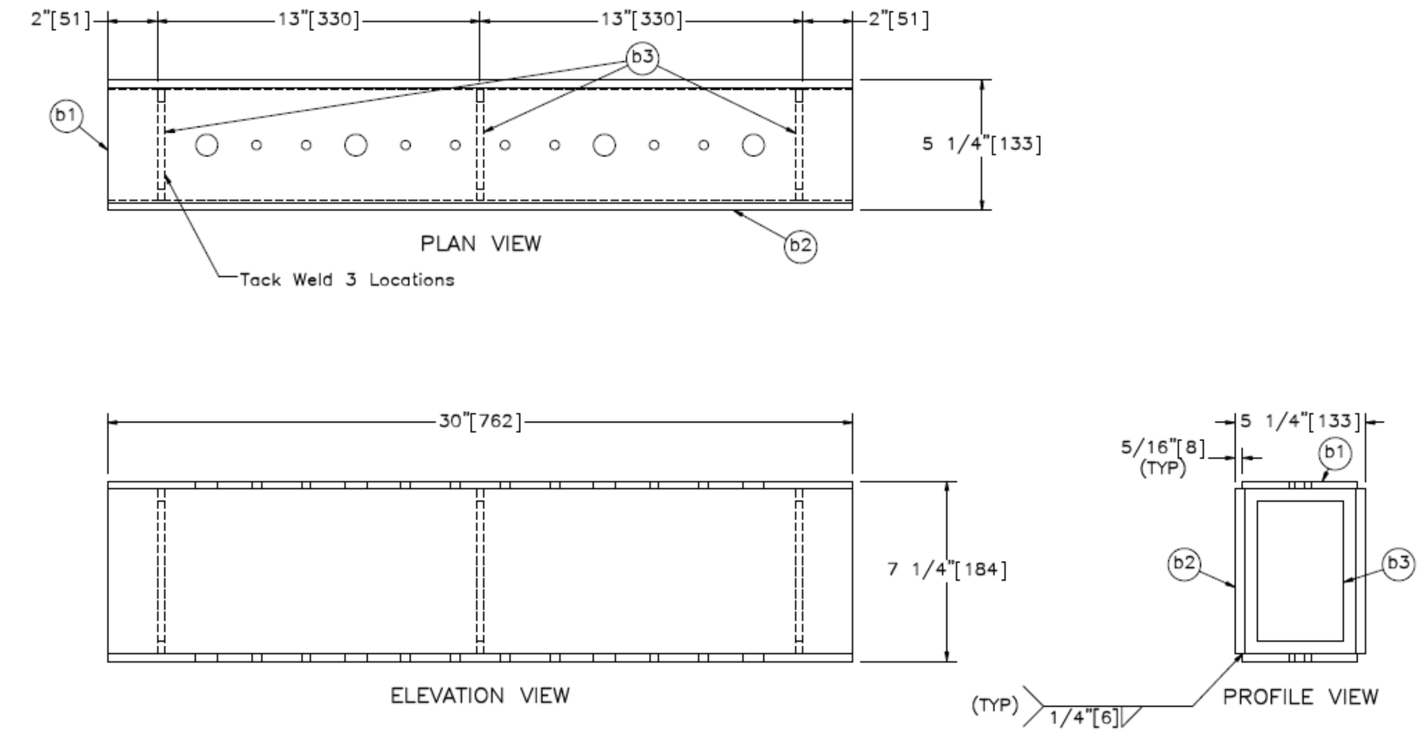


Figure C-2. Upper Splice Tube Components



Item No.	QTY.	Description	Material Specification	Treatment Specification
b1	2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	ASTM A123
b2	2	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	ASTM A123
b3	3	6 5/8"x4 1/2"x5/16" [168x114x8] Plate	ASTM A572 Gr. 50	ASTM A123

MwRSF Midwest Roadside Safety Facility	OH-IL STBR Splice Fabrication	SHEET: 3 of 4	DATE: 01/29/2019
	Middle/Lower Splice Tube Assembly	DRAWN BY: DJW	
	DWG. NAME. OH-IL_STBR_Splices_R0	SCALE: 1:6 UNITS: in.[mm]	REV. BY: –

Figure C-3. Middle/Lower Splice Tube Assembly

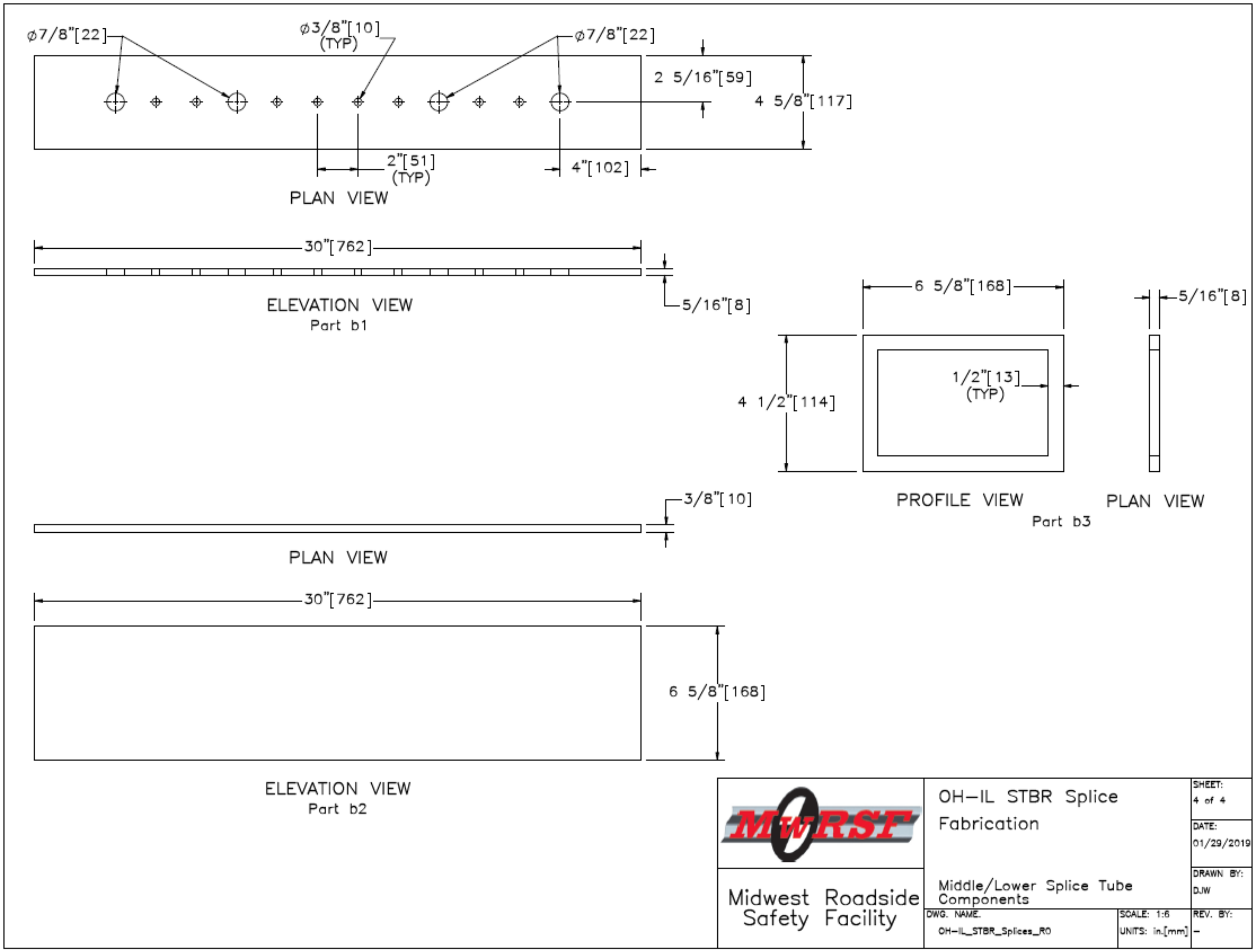


Figure C-4. Middle/Lower Splice Tube Components

Appendix D. Material Specifications

Table D-1. Bill of Materials, Test Nos. STBR-1 and STBR-2

Item No.	Description	Material Specification	Reference
a1	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	H#59082360/02
a2	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	H#E8H296
a2	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	H#W8J660
a3	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	H#W8J820
a4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	H#E8I347
a5	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	H#17111221
a6	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	H#59081160/02
a7	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	H#B8H825
b1	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	H#18170241
b2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	H#E8H296
c1	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	H#E8H296
c2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	H#18170241
d1	HSS 8"x6"x1/4" [203x152x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	H#835188
d2	HSS 12"x4"x1/4" [305x102x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	H#NH4681 "B" and H#TH4011
e1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	Ticket#1233165, 1233167, 1233170, 1233172
e2	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	H#1810025501
e3	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	H#57174895
e4	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	H#KN18100997
e5	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	H#57169293
f1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	H#B8E871
f2	1"-8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	H#NF100786021
f3	1"-8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	Fastenal COC only P#38210 C#210157128
f4	1"-8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	H#58033301/03
g1	3/4"-10 UNC [M20x2.5], 7 1/2" [191] Long Round Head Bolt	ASTM A449	H#3078659
g2	3/4"-10 UNC [M20x2.5], 6" [152] Long Round Head Bolt	ASTM A449	H#3078659
g3	3/4"-10 UNC [M20x2.5], 9 1/2" [241] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#3078659
g4	1"-8 UNC [M24x3], 3 1/2" [89] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#10552460
g5	1"-8 UNC [M24x3], 2 1/4" [57] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#10415990
g6	1"-8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449 SAE J429-2014 Gr5	P#12459 C#120271368 C#190099651 H#J11503054 H#10440680

Table D-2. Bill of Materials, Test Nos. STBR-1 and STBR-2, (Cont.)

g7	3"x3"x1/4" [76x76x6] Plate	ASTM A36	H#17126641
h1	3/4”-10 UNC [M20x2.5] Heavy Hex Nut	ASTM A563DH	H#DL18102990
i1	3/4" [19] Dia. Hardened Flat Washer	ASTM F436	H#270517 and H#281047
i2	2 1/4"x2 1/4"x1/4" [57x57x6] Square Washer	ASTM A36	H#17126641
i3	3/4"-10 UNC [M20x2.5], 12" [305] Long Threaded Rod	ASTM F1554 Gr. 36	H#145918

Table D-3. Bill of Materials, Test No. STBR-3

Item No.	Description	Material Specification	Reference No.
a1	W6x15 [W152x22], 53 1/2" [1,359] Long Post	ASTM A992	H#59082360/02
a2	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	H#E8H296
a3	13"x17 3/4"x1" [330x451x25 Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	H#W8J820
a4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	H#E8I347
a5	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	H#17111221
b1	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	H#18170241
b2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	H#E8H296
c1	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	H#E8H296
c2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	H#18170241
d1	HSS 8"x6"x1/4" [203x152x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	H#835188
d2	HSS 12"x4"x1/4" [305x102x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	H#NH4681 "B" and H#TH4011 Also See Sheet 2
e1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	Ticket#1233165, 1233167, 1233170, 1233172
e2	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	H#1810025501
e3	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	H#57174895
e4	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	H#KN18100997
e5	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	H#57169293
f1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	H#B8E871
f2	1"-8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	H#NF100786021
f3	1"-8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	Fastenal COC only P#38210 C#210157128
f4	1"-8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	H#58033301/03
g1	3/4"-10 UNC [M20x2.5], 7 1/2" [191] Long Round Head Bolt	ASTM A449	H#3078659
g2	3/4"-10 UNC [M20x2.5], 6" [152] Long Round Head Bolt	ASTM A449	H#3078659
g3	3/4"-10 UNC [M20x2.5], 9 1/2" [241] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#3078659
g4	1"-8 UNC [M24x3], 3 1/2" [89] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#10552460
g5	1"-8 UNC [M24x3], 2 1/4" [57] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#10415990
g6	1"-8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	P#12459 C#120271368 C#190099651 H#J11503054 H#10440680
g7	3"x3"x1/4" [76x76x6] Plate	ASTM A36	H#17126641
h1	3/4”-10 UNC [M20x2.5] Heavy Hex Nut	ASTM A563DH	H#DL18102990
i1	3/4" [19] Dia. Hardened Flat Washer	ASTM F436	H#270517 and H#281047
i2	2 1/4"x2 1/4"x1/4" [57x57x6] Square Washer	ASTM A36	H#17126641

Table D-4. Bill of Materials, Test No. STBR-4

Item No.	Description	Material Specification	Mill Certification No.
a1	W6x15 [W152x22], 58 1/2" [1,486] Long Post	ASTM A992	H#59082360/02
a2	8"x8"x3/8" [203x203x10] Plate	ASTM A572 Gr. 50	H#E8H296
a2	8"x8"x3/8" [203x203x10] Plate for Tarmac Posts	ASTM A572 Gr. 50	H#WJ8J660
a3	13"x17 3/4"x1" [330x451x25] Post Plate with Slots for 1" [25] Bolts	ASTM A572 Gr. 50	H#W8J820
a4	6 1/8"x5 11/16"x1/4" [156x144x6] Gusset Plate	ASTM A572 Gr. 50	H#E8I347
a5	HSS 5"x4"x1/2" [127x102x13], 20" [508] Long with 1 1/8" [29] Holes	ASTM A500 Gr. C	H#17111221
a6	W6x15 [W152x22], 30 7/8" [784] Long Post	ASTM A992	H#59081160/02
a7	12"x12"x3/4" [305x305x19] Plate	ASTM A572 Gr. 50	H#B8H825
b1	30"x10 5/8"x5/16" [762x270x8] Plate	ASTM A572 Gr. 50	H#18170241
b2	30"x2 5/8"x3/8" [762x67x10] Plate	ASTM A572 Gr. 50	H#E8H296
c1	30"x6 5/8"x3/8" [762x168x10] Plate	ASTM A572 Gr. 50	H#E8H296
c2	30"x4 5/8"x5/16" [762x117x8] Plate	ASTM A572 Gr. 50	H#18170241
d1	HSS 8"x6"x1/4" [203x152x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	H#835188
d2	HSS 12"x4"x1/4" [305x102x6], 191 1/4" [4,858] Long	ASTM A500 Gr. C	H#NH4681 "B" and H#TH4011, Also See Sheet 2
e1	Concrete	Min. f'c = 4,000 psi [27.6 MPa]	Ticket#1233165, 1233167, 1233170, 1233172
e2	#5 [16] Bar, 31" [787] Long	ASTM A615 Gr. 60	H#1810025501
e3	#5 [16] Bar, 1,294" [32,868] Long	ASTM A615 Gr. 60	H#57174895
e4	#5 [16] Bar, 110 3/16" [2,799] Long Unbent	ASTM A615 Gr. 60	H#KN18100997
e5	#6 [19] Bar, 32" [813] Long	ASTM A615 Gr. 60	H#57169293
f1	20"x15"x3/16" [508x381x5] Steel Plate	ASTM A572 Gr. 50	H#B8E871
f2	1"-8 UNC [M24x3] Heavy Hex Coupling Nut	ASTM A563DH	H#NF100786021
f3	1"-8 UNC [M24x3] Heavy Hex Nut	ASTM A563DH	Fastenal COC only P#38210 C#210157128
f4	1"-8 UNC [M24x3], 32 3/4" [832] Long Fully Threaded Anchor Rod	ASTM F1554 Gr. 105	H#58033301/03
f5	1/2" [13] Dia. Shear Stud, 2" [51] Long	ASTM A108	H#100893527
g1	3/4"-10 UNC [M20x2.5], 7 1/2" [191] Long Round Head Bolt	ASTM A449	H#3078659
g2	3/4"-10 UNC [M20x2.5], 6" [152] Long Round Head Bolt	ASTM A449	H#3078659
g3	3/4"-10 UNC [M20x2.5], 9 1/2" [241] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#3078659
g4	1"-8 UNC [M24x3], 3 1/2" [89] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#10552460
g5	1"-8 UNC [M24x3], 2 1/4" [57] Long Heavy Hex Head Bolt	ASTM F3125 Gr. A325 Type 1	H#10415990

Table D-5. Bill of Materials, Test No. STBR-4, (Cont.)

g6	1"-8 UNC [M24x3], 1 1/2" [38] Long Hex Head Bolt	ASTM A449	P#12459 C#120271368 C#190099651 H#J11503054 H#10440680
g7	3"x3"x1/4" [76x76x6] Plate	ASTM A36	H#17126641
g8	36"x2 1/2"x3/8" [914x64x10] Plate	ASTM A572 Gr. 50	H#631893983/02
h1	3/4”-10 UNC [M20x2.5] Heavy Hex Nut	ASTM A563DH	H#DL18102990
i1	3/4" [19] Dia. Hardened Flat Washer	ASTM F436	H#270517 and H#281047 (UNPAINTED) H#277411 (GREEN) H#270517 and H#281047 (BLUE)
i2	2 1/4"x2 1/4"x1/4" [57x57x6] Square Washer	ASTM A36	H#17126641
i3	3/4"-10 UNC [M20x2.5], 12" [305] Long Threaded Rod	ASTM F1554 Gr. 36	H#145918

GERDAU

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

CERTIFIED MATERIAL TEST REPORT

Page 1/1

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID:
STEEL AND PIPE SUPPLY CO INC JONESBURG INDUSTRIAL PARK JONESBURG,MO 63351 USA	STEEL AND PIPE SUPPLY CO INC MANHATTAN,KS 66505-1688 USA	A992/A572-50	Wide Flange Beam / 6 X 15# / 150 X 22.5	0000000000

LENGTH	PCS	WEIGHT	HEAT / BATCH
60'00"	0	10,800 LB	59082360/02

SALES ORDER	CUSTOMER MATERIAL N°
7177340/000010	000000000376150060

SPECIFICATION / DATE or REVISION
ASTM A6-17
ASTM A709-17
ASTM A992-11 (2015), A572-15
CSA G40.21-13 345WM

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
4500319529	1327-0000300764	11/12/2018

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	Nb %	Al %
0.09	0.81	0.015	0.036	0.20	0.37	0.12	0.14	0.024	0.011	0.002	0.016	0.003

CHEMICAL COMPOSITION

CEqyA6 %
0.29

MECHANICAL PROPERTIES

YS 0.2% PSI	UTS PSI	YS MPa	UTS MPa	Y/T rati %	G/L Inch
52633	74340	363	513	0.710	8.000
53070	74956	366	517	0.710	8.000

MECHANICAL PROPERTIES

G/L mm	Elong. %
200.0	24.20
200.0	23.60

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

Figure D-1. Side-Mounted Steel Posts, Test Nos. STBR-1 through STBR-4

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 11/01/2018
TIME 05:59:17

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
40317778-0010	721296120A2	3/8 96 X 120 A572GR50 MILL PLATE	3	3,676.800			10/31/2018

Chemical Analysis

Heat No. E8H296 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
0.1800	1.0300	0.0100	0.0020	0.0400	0.1500	0.0900	0.0300	0.0000	0.3000	0.0410	0.0010	0.0240	0.0010	0.0000	0.0000

Mechanical / Physical Properties

Mill Coil No. E8H2960947

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
80100.000	57500.000	27.50			0	NA			
82100.000	61500.000	30.30			0	NA			
84500.000	65900.000	28.00			0	NA			
79600.000	58500.000	27.00			0	NA			

Batch 0005536674 3 EA 3,676.800 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 D-2. Fully-Welded Plates for Side-Mounted Posts, Test Nos. STBR-1 through STBR-4

SSAB

Preliminary Test Certificate

12400 Highway 43 North, Axis, Alabama 36505, US **Official copy to follow**

Form TC1: Revision 3: Date 7 Feb 2018

Customer: STEEL & PIPE SUPPLY P.O. BOX 1688 MANHATTAN KS 66502	Customer P.O. No.: 4500315066	Mill Order No.: 41-551726-04	Shipping Manifest : AT275860
	Product Description: ASTM A572-50/M345(18)/A709-50/M345(17)	Ship Date: 22 Oct 18 Cert Date: 22 Oct 18	Cert No: 081690843 (Page 1 of 1)
	Size: 0.375 X 96.00 X 240.0 (IN)		

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
W8J660	B15	0.371 (DISCRT)	L	64	74		19	T							

Tested Pieces: Heat Id – Tst Loc; Tensiles: YS – Tst Dir; Charpy Impact Tests: Abs. Energy – Tst Siz

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti	ORGN
W8J660	.05	1.30	.013	<.001	.36	.032	.21	.14	.18	.04	.032	.004	.014	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:
W8J660 B15 PCES: 4, LBS: 9800

(H) Cust Part # : 721296240A2

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

SENIOR METALLURGIST - PRODUCT

Figure D-3. Fully-Welded Plates for Baseplates for Top-Mounted Posts, Test Nos. STBR-1 and STBR-2

SSAB

12400 Highway 43 North, Axis, Alabama 36505, US

Test Certificate

Form TC1: Revision 3: Date 7 Feb 2018

Customer: STEEL & PIPE SUPPLY P.O. BOX 1688 MANHATTAN KS 66502	**Customer P.O.No.:**4500317235	**Mill Order No.** 41-553979-03	**Shipping Manifest:** AT276683
	Product Description: ASTM A572-50/M345(18)/A709-50/M345(17)	**Ship Date:** 05 Nov 18 **Cert Date:** 05 Nov18	**Cert No:** 081692938 (Page 1 of 1)
	Size: 1.000 X 96.00 X 240.0 (IN)		

Tested Pieces:				Tensiles:							Charpy Impact Tests					
Heat Id	Piece Id	Piece Dimensions	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
W8J820	D19	0.999 (DISCRT)	L	58	79			24	T							

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti	ORGN
W8J820	.18	1.17	.008	<.001	.25	.031	.24	.12	.11	.03	.000	.052	.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:
W8J820 D19 PCES: 3, LBS: 19602

(P) Cust Part #: 7210096240A2

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

Justin Ward
SENIOR METALLURGIST - PRODUCT

Figure D-4. Post-to-Deck Connection, Vertical Plate, Test Nos. STBR-1 through STBR-4

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 11/30/2018
TIME 05:54:18

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
40320870-0010	72896240A2	1/4 96 X 240 A572GR50 MILL PLATE	2	3,267.200			11/29/2018

Chemical Analysis

Heat No. E8I347 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	1.0100	0.0070	0.0040	0.0300	0.1200	0.0700	0.0400	0.0000	0.2100	0.0370	0.0010	0.0210	0.0000	0.0000	0.0000

Mechanical / Physical Properties

Mill Coil No. E8I3470512

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
78500.000	59700.000	27.40			56	Longitudinal	5.0	-20 F	
75600.000	56900.000	32.40			50	Longitudinal	5.0	-20 F	
77700.000	59600.000	29.60			43	Longitudinal	5.0	-20 F	
78500.000	60400.000	25.00			0	NA			

Batch 0005571830 2 EA 3,267.200 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 D-5. Gusset Plate, Test Nos. STBR-1 through STBR-4

Atlas Tube Corporation
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: 80795210
Date: 11.30.2017
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: 20.0x8.0x500x43'0"0(1x2)NMH — Material No: 200080500 — Made in: USA — Melted in: USA

Sales order: 1216180 — Purchase Order: PO-064074 — Cust Material #: 013969

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
E44934	0.200	0.800	0.010	0.008	0.010	0.046	0.020	0.004	0.006	0.010	0.030	0.001	0.001	0.000	0.006

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M900960662	2	068209 Psi	075041 Psi	36 %	ASTM A500-13 GRADE B&C	0.34

Material Note:
Sales Or.Note:

Material: 4.0x4.0x500x40'0"0(4x2) — Material No: 400405004000 — Made in: USA — Melted in: USA

Sales order: 1236805 — Purchase Order: PO-065525 — Cust Material #: 012007

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
17117241	0.200	0.740	0.006	0.002	0.030	0.029	0.110	0.001	0.010	0.040	0.030	0.003	0.000	0.000	0.008

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M800744469	8	073296 Psi	084387 Psi	31 %	ASTM A500-13 GRADE B&C	0.35

Material Note:
Sales Or.Note:

Material: 5.0x4.0x500x40'0"0(3x3) — Material No: 500405004000 — Made in: USA — Melted in: USA

Sales order: 1236805 — Purchase Order: PO-065525 — Cust Material #: 012321

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N
17111221	0.210	0.740	0.005	0.002	0.030	0.031	0.060	0.001	0.017	0.030	0.030	0.003	0.001	0.000	0.006

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	CE
M800743105	9	066916 Psi	077416 Psi	36 %	ASTM A500-13 GRADE B&C	0.35

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.
CE calculated using the AWS D1.1 method.

Steel Tube Institute OF NORTH AMERICA

Page : 2 Of 3

Metals Service Center Institute

NOV, 18
50871
35002

Figure D-6. Post-to-Deck Connection, Spacer Tubes, Test Nos. STBR-1 through STBR-4

GERDAU US-ML-MIDLOTHIAN 300 WARD ROAD MIDLOTHIAN, TX 76065 USA	CUSTOMER SHIP TO STEEL AND PIPE SUPPLY CO INC 401 NEW CENTURY PKWY NEW CENTURY,KS 66031-1127 USA	CUSTOMER BILL TO STEEL AND PIPE SUPPLY CO INC MANHATTAN,KS 66505-1688 USA	GRADE A992/A572-50		SHAPE / SIZE Wide Flange Beam / 6 X 15# / 150 X 22.5	DOCUMENT ID: 0000259290
			LENGTH 40'00"	PCS 24	WEIGHT 14,400 LB	HEAT / BATCH 59081160/02
	SALES ORDER 6994651/000040	CUSTOMER MATERIAL Nº 000000000376150040	SPECIFICATION / DATE or REVISION ASTM A6-17 ASTM A709-17 ASTM A992-11 (2015), A572-15 CSA G40.21-13 345WM			
CUSTOMER PURCHASE ORDER NUMBER G450027351	BILL OF LADING 1327-0000296198	DATE 09/25/2018				

CERTIFIED MATERIAL TEST REPORT

Page 1/1

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	Nb %	Al %
0.08	0.82	0.009	0.030	0.17	0.25	0.06	0.10	0.015	0.005	0.001	0.013	0.003

CHEMICAL COMPOSITION

CEqvA6 %
0.26

MECHANICAL PROPERTIES

YS 0.2% PSI	UTS PSI	YS MPa	UTS MPa	Y/T rati %	G/L Inch
51652	70325	356	485	0.740	8.000
52837	71861	364	496	0.730	8.000

MECHANICAL PROPERTIES

G/L mm	Elong. %
200.0	24.80
200.0	24.80

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.

Bhaskary

BHASKAR YALAMANCHILI
QUALITY DIRECTOR

Phone: (409) 267-1071 Email: Bhaskar.Yalamanchili@gerdau.com

Wade A. L.

WADE LUMPKINS
QUALITY ASSURANCE MGR.

Phone: 972-779-3118 Email: Wade.Lumpkins@gerdau.com

Figure D-7. Top-Mounted Steel Posts, STBR-1, STBR-2, and STBR-4

SSAB

Preliminary Test Certificate

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

Form TC1: Revision 3: Date 7 Feb 2018

Customer: STEEL & PIPE SUPPLY P.O. BOX 1688 MANHATTAN KS 66502	Customer P.O. No.: 4500313371 Product Description: ASTM A572-50/M345(18)/A709-50/M345(17) Size: 0.750 X 96.00 X 240.0 (IN)	Mill Order No.: 41-548818-03 Ship Date: 31 Aug 18 Cert Date: 31 Aug 18	Shipping Manifest : MT355770 Cert No: 061729922 (Page 1 of 1)

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
B8H825	C15	0.389 (DISCRT)	L	56	76		32		T							
B8H825	C17	0.999 (DISCRT)	L	52	73			24	T							

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti	ORGN
B8H825	.16	1.10	.011	.003	.04	.024	.25	.09	.09	.02	.001	.034	.006	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:
B8H825 C16 PCES: 4, LBS: 19604

(11) 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 D-8. Baseplates for Top-Mounted Posts, Test Nos. STBR-1, STBR-2, and STBR-4

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 12/12/2018
TIME 05:49:02

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
40321447-0010	701072120A2	5/16 72 X 120 A572GR50 STP MIL PLT	1	765.600			12/11/2018

Chemical Analysis

Heat No. 18170241 Vendor BIG RIVER STEEL LLC DOMESTIC Mill BIG RIVER STEEL LLC 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
0.0500	0.8600	0.0100	0.0030	0.0300	0.0300	0.0600	0.0100	0.0001	0.0900	0.0280	0.0010	0.0040	0.0160	0.0064	0.0047

Mechanical / Physical Properties

Mill Coil No. 18170241-08

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
73700.000	65900.000	29.60			187	Longitudinal	6.7	-20 F	
72800.000	62900.000	33.20			182	Longitudinal	6.7	-20 F	
					183	Longitudinal	6.7	-20 F	

Batch 0005584036 1 EA 765.600 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 D-9. Top Splice Tubes Horizontal Plates, Test Nos. STBR-1 through STBR-4

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 11/01/2018
TIME 05:59:17

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
40317778-0010	721296120A2	3/8 96 X 120 A572GR50 MILL PLATE	3	3,676.800			10/31/2018

Chemical Analysis

Heat No. E8H296 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
0.1800	1.0300	0.0100	0.0020	0.0400	0.1500	0.0900	0.0300	0.0000	0.3000	0.0410	0.0010	0.0240	0.0010	0.0000	0.0000

Mechanical / Physical Properties

Mill Coil No. E8H2960947

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
80100.000	57500.000	27.50			0	NA			
82100.000	61500.000	30.30			0	NA			
84500.000	65900.000	28.00			0	NA			
79600.000	58500.000	27.00			0	NA			

Batch 0005536674 3 EA 3,676.800 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 D-10. Top Splice Tubes Vertical Plates and Lower Splice Tube Vertical Plates, Test Nos. STBR-1 through STBR-4

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 12/12/2018
TIME 05:49:02

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
40321447-0010	701072120A2	5/16 72 X 120 A572GR50 STP MIL PLT	1	765.600			12/11/2018

Chemical Analysis

Heat No. 18170241 Vendor BIG RIVER STEEL LLC DOMESTIC Mill BIG RIVER STEEL LLC **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
0.0500	0.8600	0.0100	0.0030	0.0300	0.0300	0.0600	0.0100	0.0001	0.0900	0.0280	0.0010	0.0040	0.0160	0.0064	0.0047

Mechanical / Physical Properties

Mill Coil No. 18170241-08

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
73700.000	65900.000	29.60			187	Longitudinal	6.7	-20 F	
72800.000	62900.000	33.20			182	Longitudinal	6.7	-20 F	
					183	Longitudinal	6.7	-20 F	

Batch 0005584036 1 EA 765.600 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 D-11. Lower Splice Tubes Horizontal Plates, Test Nos. STBR-1 through STBR-4

Atlas Tube Canada
200 Clark St.
Harrow Ontario Canada
NOR 1G0
Tel: 519-738-3541
Fax: 519-738-3537

Atlas Tube
A DIVISION OF ZEKELMAN INDUSTRIES

REF.B/L: 80852636
Date: 11/13/2018
Customer: 179

MATERIAL TEST REPORT

Sold To
Steel & Pipe Supply Company
PO Box 1688
MANHATTAN KS 66505
USA

Shipped To
Steel & Pipe Supply Company
1020 West Fort Gibson
CATOOSA OK 74015
USA

Material: 16.0x4.0x375x40'0"0(1x2). Material No: 160040375 Made in: Canada
Melted in: Canada
Sales Order: 1340914 Purchase Order: 4500318744 Cust Material#: 66160040037540

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N	Ca
835845	0.200	0.810	0.013	0.008	0.017	0.042	0.036	0.005	0.003	0.011	0.047	0.002	0.002	0.0002	0.0040	0.0002

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	
M201325206	2	060925 Psi	073628 Psi	37.8 %	ASTM A500-18 GRADE B&C	CE: 0.35

Material Note:
Sales Or. Note:

Material: 8.0x6.0x250x40'0"0(3x2). Material No: 800602504000 Made in: Canada
Melted in: Canada
Sales Order: 1337754 Purchase Order: C450007477 Cust Material#: 6680060025040

Heat No	C	Mn	P	S	Si	Al	Cu	Cb	Mo	Ni	Cr	V	Ti	B	N	Ca
835188	0.190	0.800	0.008	0.011	0.022	0.046	0.066	0.005	0.005	0.021	0.035	0.002	0.002	0.0002	0.0050	0.0000

Bundle No	PCs	Yield	Tensile	Eln.2in	Certification	
M101826832	6	058363 Psi	065756 Psi	33.9 %	ASTM A500-18 GRADE B&C	CE: 0.34

Material Note:
Sales Or. Note:

ALL 40 PIECES THIS HEAT#

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.
CE calculated using the AWS D1.1 method.

Steel Tube Institute
OF NORTH AMERICA

Metals Service Center Institute

Page: 2 of 2

Figure D-12. Lower Rails, Test Nos. STBR-1 through STBR-4

12Sep18 13:39 T E S T C E R T I F I C A T E No: DCR 865273

INDEPENDENCE TUBE CORPORATION
6226 W. 74TH STREET
CHICAGO, IL 60638
Tel: 708-496-0380 Fax: 708-563-1950

P/O No 4500315602
Rel
S/O No DCR 107708-005
B/L No DCR 73528-004 Shp 12Sep18
Inv No Inv

Sold To: (5018)
STEEL & PIPE SUPPLY
4750 W. MARSHALL AVENUE
LONGVIEW, TX 75604

Ship To: (1)
STEEL & PIPE SUPPLY
4750 W. MARSHALL AVENUE
LONGVIEW, TX 75604

Tel: 785-587-5100 Fax: 785 587-5339

CERTIFICATE of ANALYSIS and TESTS Cert. No: DCR 865273
06Sep18

Part No		Pcs	Wgt
TUBING A500 GRADE B(C) 12" X 4" X 1/4" X 40'		6	6,197
Heat Number	Tag No	Pcs	Wgt
NH4681	28422	6	6,197
	YLD=61100/TEN=70900/ELG=33		

8-PIECES MARKED "B" THIS HEAT #

Heat Number *** Chemical Analysis ***
NH4681 C=0.0600 Mn=0.5800 P=0.0050 S=0.0010 Si=0.2520 Al=0.0310
Cu=0.0700 Cr=0.0300 Mo=0.0100 V=0.0020 Ni=0.0200 Nb=0.0090
Cb=0.0090 Sn=0.0020 N=0.0048 B=0.0001 Ti=0.0010 Ca=0.0010
MELTED AND MANUFACTURED IN THE USA

WE PROUDLY MANUFACTURE ALL OUR PRODUCT IN THE USA.
INDEPENDENCE TUBE PRODUCT IS MANUFACTURED, TESTED,
AND INSPECTED IN ACCORDANCE WITH ASTM STANDARDS.
MATERIAL IDENTIFIED AS A500 GRADE B(C) MEETS BOTH
ASTM A500 GRADE B AND A500 GRADE C SPECIFICATIONS.

CURRENT STANDARDS:
A252-10
A500/A500M-18
A513/A513M-15
ASTM A53/A53M-12 | ASME SA-53/SA-53M-13
A847/A847M-14
A1085/A1085M-15

Page: 1 Last

Figure D-13. Top Rails, Test Nos. STBR-1 through STBR-4

22Aug18 22:57 T E S T C E R T I F I C A T E No: DCR 852468

INDEPENDENCE TUBE CORPORATION
6226 W. 74TH STREET
CHICAGO, IL 60638
Tel: 708-496-0380 Fax: 708-563-1950

P/O No 4500314328
Rel
S/O No DCR 107030-001
B/L No DCR 73089-002 Shp 22Aug18
Inv No Inv

Sold To: (5018)
STEEL & PIPE SUPPLY
4750 W. MARSHALL AVENUE
LONGVIEW, TX 75604

Ship To: (1)
STEEL & PIPE SUPPLY
4750 W. MARSHALL AVENUE
LONGVIEW, TX 75604

Tel: 785-587-5100 Fax: 785 587-5339

CERTIFICATE of ANALYSIS and TESTS Cert. No: DCR 852468
17Aug18

Part No		Pcs	Wgt
TUBING A500 GRADE B(C) 12" X 4" X 1/4" X 40'		6	6,197
Heat Number	Tag No	Pcs	Wgt
TH4011	21552 YLD=66200/TEN=78000/ELG=25	6	6,197

12-PIECES NOT MARKED THIS HEAT #

Heat Number *** Chemical Analysis ***
TH4011
C=0.0600 Mn=0.6200 P=0.0080 S=0.0010 Si=0.2530 Al=0.0350
Cu=0.1200 Cr=0.0500 Mo=0.0200 V=0.0030 Ni=0.0400 Nb=0.0090
Cb=0.0090 Sn=0.0070 N=0.0087 B=0.0002 Ti=0.0010 Ca=0.0016
MELTED AND MANUFACTURED IN THE USA

WE PROUDLY MANUFACTURE ALL OUR PRODUCT IN THE USA.
INDEPENDENCE TUBE PRODUCT IS MANUFACTURED, TESTED,
AND INSPECTED IN ACCORDANCE WITH ASTM STANDARDS.
MATERIAL IDENTIFIED AS A500 GRADE B(C) MEETS BOTH
ASTM A500 GRADE B AND A500 GRADE C SPECIFICATIONS.

CURRENT STANDARDS:
A252-10
A500/A500M-18
A513/A513M-15
ASTM A53/A53M-12 | ASME SA-53/SA-53M-13
A847/A847M-14
A1085/A1085M-15

Page: 1 .... Last

Figure D-14. Top Rails, Test Nos. STBR-1 through STBR-4

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

Customer's Signature:

PLANT	TRUCK	DRIVER	CUSTOMER	PROJECT	TAX	PO NUMBER	DATE	TIME	TICKET
01	124	10102	62461			STBR	12/18/18	8:28 AM	1233165

Customer	Delivery Address	Special Instructions
UNL-MIDWEST ROADSIDE SAFETY	4630 NW 36TH ST	AIRPARK / NW 36TH ST & W CUMINGSST / NORTH OF OLD GOODYEAR HANGARS

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UOM	UNIT PRICE	EXTENDED PRICE
9.25	9.25	37.00	470031PF	47BD (1PF)	yd	$115.91	$1,072.17
				WINTER SERVICE			$46.25

Water Added On Job At Customer's Request:	SLUMP	Notes:		
	4.00 in		TICKET SUBTOTAL	$1,118.42
			SALES TAX	$0.00
			TICKET TOTAL	**$1,118.42**
			PREVIOUS TOTAL	
			GRAND TOTAL	**$1,118.42**

CAUTION FRESH CONCRETE

KEEP CHILDREN AWAY

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

Terms & Conditions

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

MATERIAL	DESCRIPTION	DESIGN QTY	REQUIRED	BATCHED	% VAR	% MOISTURE	ACTUAL WATER
CEM1PF	DURACEM	658 lb	6087 lb	6075 lb	-0.19%		
G47B	47B GRAVEL	1975 lb	18431 lb	18340 lb	-0.49%	0.89% A	19 gl
L47B	47B ROCK	840 lb	7883 lb	7940 lb	0.22%	1.45% A	14 gl
LRWR	POZZ 322N LOV	20.00 oz	185.00 oz	184.00 oz	-0.54%		
AIR	MICRO AIR 200	5.00 oz	46.25 oz	46.00 oz	-0.54%		
WATER	WATER	31.5 GL	267.7 GL	266.5 GL	-0.43%		266.5 gl

Actual		Num Batches: 1			Manual	
Load: 34593 lb		Design W/C: 0.40	Water/Cement: 0.41 A	Design Water: 291.4 gl	Actual:	299.5 gl
Slump: 4.00 in #		Water in Truck: 0.0 GL	Adjust Water: 0.0 GL / Load	Trim Water: 0.0 GL / CYDS		
Actual W/C Ratio 0.41		Actual Water: 299 gl	Batched Cement: 6075 lb	Allowable Water: 0 lb	To Add:	0.0 gl

Figure D-15. Surrogate Concrete Bridge Deck Concrete Mix, Test Nos. STBR-1 through STBR-4

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

Customer's Signature: ______

PLANT	TRUCK	DRIVER	CUSTOMER	PROJECT	TAX	PO NUMBER	DATE	TIME	TICKET
01	108	7596	62461			STBR	12/18/18	9:00 AM	1233167

Customer	Delivery Address	Special Instructions
UNL-MIDWEST ROADSIDE SAFETY	4630 NW 36TH ST	AIRPARK / NW 36TH ST & W CUMINGSST / NORTH OF OLD GOODYEAR HANGARS

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UOM	UNIT PRICE	EXTENDED PRICE
9.25	18.50	37.00	470031PF	47BD (1PF)	yd	$115.91	$1,072.17
				WINTER SERVICE			$46.25

Water Added On Job At Customer's Request:	SLUMP	Notes:		
	4.00 in		TICKET SUBTOTAL	$1,118.42
			SALES TAX	$0.00
			TICKET TOTAL	**$1,118.42**
			PREVIOUS TOTAL	$1,118.42
			GRAND TOTAL	**$2,236.84**

CAUTION FRESH CONCRETE

KEEP CHILDREN AWAY

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

Terms & Conditions

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

MATERIAL	DESCRIPTION	DESIGN QTY	REQUIRED	BATCHED	% VAR	% MOISTURE	ACTUAL WATER
CEM1PF	DURACEM	658 lb	6087 lb	6080 lb	-0.11%		
G47B	47B GRAVEL	1975 lb	18400 lb	18420 lb	0.11%	0.72% E	16 gl
L47B	47B ROCK	840 lb	7889 lb	7820 lb	-0.26%	1.53% A	14 gl
LRWR	POZZ 322N LOV	20.00 oz	185.00 oz	185.00 oz	0.00%		
AIR	MICRO AIR 200	5.00 oz	46.25 oz	46.00 oz	-0.54%		
WATER	WATER	31.5 GL	270.6 GL	269.9 GL	-0.29%		269.9 gl

Actual Num Batches: 1 Manual
Load: 34586 lb Design W/C: 0.40 Water/Cement: 0.41 A Design Water: 291.4 gl Actual: 299.7 gl
Slump: 4.00 in # Water in Truck: 0.0 GL Adjust Water: 0.0 GL / Load Trim Water: 0.0 GL / CYDS
Actual W/C Ratio 0.41 Actual Water: 300 gl Batched Cement: 6080 lb Allowable Water: 0 lb To Add: 0.0 gl

Figure D-16. Surrogate Concrete Bridge Deck Concrete Mix, Test Nos. STBR-1 through STBR-4

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

Customer's Signature: ____________

PLANT	TRUCK	DRIVER	CUSTOMER	PROJECT	TAX	PO NUMBER	DATE	TIME	TICKET
01	141	8890	62461			STBR	12/18/18	9:31 AM	1233170

Customer	Delivery Address	Special Instructions
UNL-MIDWEST ROADSIDE SAFETY	4630 NW 36TH ST	AIRPARK / NW 36TH ST & W CUMINGSST / NORTH OF OLD GOODYEAR HANGARS

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UOM	UNIT PRICE	EXTENDED PRICE
9.25	27.75	37.00	470031PF	47BD (1PF)	yd	$115.91	$1,072.17
				WINTER SERVICE			$46.25

Water Added On Job At Customer's Request:	SLUMP	Notes:		
	4.00 in		TICKET SUBTOTAL	$1,118.42
			SALES TAX	$0.00
			TICKET TOTAL	**$1,118.42**
			PREVIOUS TOTAL	$2,236.84
			GRAND TOTAL	**$3,355.26**

CAUTION FRESH CONCRETE

KEEP CHILDREN AWAY

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

Terms & Conditions

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

MATERIAL	DESCRIPTION	DESIGN QTY	REQUIRED	BATCHED	% VAR	% MOISTURE	ACTUAL WATER
CEM1PF	DURACEM	658 lb	6087 lb	6075 lb	-0.19%		
G47B	47B GRAVEL	1975 lb	18420 lb	18400 lb	-0.11%	0.83% A	18 gl
L47B	47B ROCK	840 lb	7873 lb	7800 lb	-0.28%	1.33% A	12 gl
LRWR	POZZ 322N LOV	20.00 oz	185.00 oz	185.00 oz	0.00%		
AIR	MICRO AIR 200	5.00 oz	46.25 oz	47.00 oz	1.62%		
WATER	WATER	31.5 GL	270.1 GL	269.9 GL	-0.07%		269.9 gl

Actual   Num Batches: 1   Manual

Load: 34541 lb	**Design W/C: 0.40**	Water/Cement: 0.41 A	Design Water: 291.4 gl	Actual: 300.3 gl
Slump: 4.00 in #	Water in Truck: 0.0 GL	Adjust Water: 0.0 GL / Load	Trim Water: 0.0 GL / CYDS	
Actual W/C Ratio 0.41	Actual Water: 300 gl	Batched Cement: 6075 lb	Allowable Water: 0 lb	To Add: 0.0 gl

Figure D-17. Surrogate Concrete Bridge Deck Concrete Mix, Test Nos. STBR-1 through STBR-4

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

Customer's Signature: ______________________

PLANT	TRUCK	DRIVER	CUSTOMER	PROJECT	TAX	PO NUMBER	DATE	TIME	TICKET
01	251	6907	62461			STBR	12/18/18	9:49 AM	1233172

Customer	Delivery Address	Special Instructions
UNL-MIDWEST ROADSIDE SAFETY	4630 NW 36TH ST	AIRPARK / NW 36TH ST & W CUMINGSST / NORTH OF OLD GOODYEAR HANGARS

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UOM	UNIT PRICE	EXTENDED PRICE
9.25	37.00	37.00	470031PF	47BD (1PF)	yd	$115.91	$1,072.17
				WINTER SERVICE			$46.25

Water Added On Job At Customer's Request: ______

SLUMP: 4.00 in

Notes:

TICKET SUBTOTAL	$1,118.42
SALES TAX	$0.00
TICKET TOTAL	**$1,118.42**
PREVIOUS TOTAL	$3,355.26
GRAND TOTAL	**$4,473.68**

CAUTION FRESH CONCRETE
KEEP CHILDREN AWAY

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

Terms & Conditions

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

MATERIAL	DESCRIPTION	DESIGN QTY	REQUIRED	BATCHED	% VAR	% MOISTURE	ACTUAL WATER
CEM1PF	DURACEM	658 lb	6087 lb	6075 lb	-0.19%		
G47B	47B GRAVEL	1975 lb	18564 lb	18560 lb	-0.02%	1.62% A	35 gl
L47B	47B ROCK	840 lb	7864 lb	7800 lb	-0.24%	1.21% A	11 gl
LRWR	POZZ 322N LOV	20.00 oz	185.00 oz	185.00 oz	0.00%		
AIR	MICRO AIR 200	5.00 oz	46.25 oz	46.00 oz	-0.54%		
WATER	WATER	31.5 GL	254.0 GL	252.1 GL	-0.74%		252.1 gl

Actual
Load: 34553 lb | Slump: 4.00 in # | **Actual W/C Ratio 0.41**

Num Batches: 1
Design W/C: 0.40 | Water in Truck: 0.0 GL | Actual Water: 299 gl

Water/Cement: 0.41 A | Adjust Water: 0.0 GL / Load | Batched Cement: 6075 lb

Design Water: 291.4 gl | Trim Water: 0.0 GL / CYDS | Allowable Water: 0 lb

Manual
Actual: 298.6 gl | To Add: 0.0 gl

Figure D-18. Surrogate Concrete Bridge Deck Concrete Mix, Test Nos. STBR-1 through STBR-4

ABC COATING CO. OF ILLINOIS, INC.

1160 N. BOUDREAU ROAD
MANTENO, IL 60950
(708) 258-9633

AN ACUÑA CO.

DATE:	3/5/2018	CONTRACTOR:	Carroll Distributing
OUR JOB NO:	IL-8185	COUNTY:	
CUSTOMER:	Adelphia Metals, LLC	PROJECT NO:	SO# 222984
CUSTOMER PO NO:	822859/822844/5180	JOB NO:	2018 Stock

We certify that the following described bar material has been cleaned, coated with 3 M #413 or Nap-gard 7-2719, 7-2750 or Valspar 720A009 Powder inspected in accordance with and meets the specification requirements of the Iowa Department of Transportation and ASTM A775-17 AASHTO M284-06, ASTM D3963-01. Manufacturer's Certifications for the bar material and epoxy resin used are on file.

Mill	Lot/Heat	Powder	Size	Weight
Nucor GR 60	KN1810005601	3777133B	4/13	11,904
Nucor GR 60	KN1810025501	3804648	5/16	28,912
Nucor GR 60	KN1710201402	3649393	8/25	6,168
				46,984

Certifications for the listed material and resin are attached
STATE OF ILLINOIS)
COUNTY OF COOK)

Thomas Greenfield, Vice President
SUBSCRIBED AND SWORN BEFORE ME, a Notary Public in and for Cook County, Illinois, on this 5th day of March 2018

OFFICIAL SEAL
FREDDIE ROCHA
NOTARY PUBLIC, STATE OF ILLINOIS
COOK COUNTY
MY COMMISSION EXPIRES 03/31/2018

My commission expires 03-31-2018

Notary Public in and for Chicago, Illinois

EPOXY COATING PLANT CERTIFICATION TM CONCRETE REINFORCING STEEL INSTITUTE

CRSI MEMBER

Figure D-19. Vertical #5 Epoxy Rebar, Test Nos. STBR-1 through STBR-4

Page: 1

SOLD TO: ADELPHIA METALS I LLC
411 MAIN ST E
NEW PRAGUE, MN 56071-

NUCOR
NUCOR STEEL KANKAKEE, INC.

CERTIFIED MILL TEST REPORT

Ship from:
MTR #: 0000212259
Nucor Steel Kankakee, Inc.
One Nucor Way
Bourbonnais, IL 60914
815-937-3131

SHIP TO: ADELPHIA METALS LLC
ABC COATING
1160 BOUDREAU RD
MANTENO, IL 60950-

Date: 17-Jan-2018
B.L. Number: 551716
Load Number: 293888

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				CHEMICAL TESTS							
		YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% / DEF	C / Ni	Mn / Cr	P / Mo	S / V	Si / Cb	Cu / Sn	C.E.
PO# =>	822711												
KN1810025501	Nucor Steel - Kankakee Inc	67,599	103,912	15.6%	OK	-3.2%	.37	.98	.014	.051	.17	.31	
KN18100255	16/#5 Rebar	466MPa	716MPa			.039	.18	.18	.055	.008	.001		
	40' A615M GR420 (Gr60)												
	ASTM A615/A615M-16 GR 60 AASHTO M31-15												
	Melted 01/12/18 Rolled 01/16/18												
PO# =>	822711												
KN1810025602	Nucor Steel - Kankakee Inc	67,177	104,692	15.6%	OK	-3.5%	.38	1.01	.016	.058	.19	.34	
KN18100256	16/#5 Rebar	463MPa	722MPa			.039	.19	.17	.060	.009	.001		
	40' A615M GR420 (Gr60)												
	ASTM A615/A615M-16 GR 60 AASHTO M31-15												
	Melted 01/12/18 Rolled 01/16/18												

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 D-20. Vertical #5 Epoxy Rebar, Test Nos. STBR-1 through STBR-4

CERTIFIED MATERIAL TEST REPORT

Page 1/1

GERDAU

US-ML-KNOXVILLE
1919 TENNESSEE AVENUE N. W.
KNOXVILLE, TN 37921
USA

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID:
SIMCOTE INC 1645 RED ROCK SAINT PAUL,MN 55119 USA	SIMCOTE INC 1645 RED ROCK ROAD SAINT PAUL,MN 55119-6014 USA	60 (420) TMX	Rebar / #5 (16MM)	0000000000

LENGTH	WEIGHT	HEAT / BATCH
60'00"	72,092 LB	57174895/02

SALES ORDER	CUSTOMER MATERIAL N°	SPECIFICATION / DATE or REVISION
6664045/000010	BLOCKED	ASTM A615/A615M-16

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
MN-3697	1326-0000086353	07/14/2018

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	CEqyA706 %
0.32	0.56	0.015	0.060	0.20	0.28	0.11	0.13	0.022	0.007	0.003	0.44

MECHANICAL PROPERTIES

YS PSI	YS MPa	UTS PSI	UTS MPa	G/L Inch	G/L mm
80360	554	99840	688	8.000	200.0

MECHANICAL PROPERTIES

Elong. %	BendTest
12.80	OK

GEOMETRIC CHARACTERISTICS

%Light %	Def Hgt Inch	Def Gap Inch	DefSpace Inch
4.89	0.046	0.122	0.379

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) 267-1071 Email: Bhaskar.Yalamanchili@gerdau.com

Jim Hall

JIM HALL
QUALITY ASSURANCE MGR.

Phone: 865-202-5972 Email: Jim.hall@gerdau.com

Figure D-21. Horizontal #5 Epoxy Rebar, Test Nos. STBR-1 through STBR-4

Page: 1

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

NUCOR
NUCOR STEEL KANKAKEE, INC.

CERTIFIED MILL TEST REPORT

Ship from:
MTR #: 0000219520
Nucor Steel Kankakee, Inc.
One Nucor Way
Bourbonnais, IL 60914
815-937-3131

SHIP TO: ABC COATING CO - MN
2500 W COUNTY ROAD B
DOOR 16A
ROSEVILLE, MN 55113-

Date: 27-Feb-2018
B.L. Number: 553999
Load Number: 295639

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					CHEMICAL TESTS						
		YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% / DEF	C / Ni	Mn / Cr	P / Mo	S / V	Si / Cb	Cu / Sn	C.E.
PO# =>	022018-MINN												
KN1810024401	Nucor Steel - Kankakee Inc	68,727	107,604	13.8%	OK	-3.4%	.38	.99	.015	.044	.23	.32	
KN18100244	16/#5 Rebar	474MPa	742MPa			.038	.18	.15	.063	.008	.001		
	40' A615M GR420 (Gr60)												
	ASTM A615/A615M-16 GR 60 AASHT O M31-15												
	Melted 01/11/18 Rolled 01/16/18												
PO# =>	022018-MINN												
KN1810099701	Nucor Steel - Kankakee Inc	66,602	101,565	14.8%	OK	-3.2%	.38	.98	.013	.056	.22	.37	
KN18100997	16/#5 Rebar	459MPa	700MPa			.041	.20	.15	.069	.010	.001		
	40' A615M GR420 (Gr60)												
	ASTM A615/A615M-16 GR 60 AASHT O M31-15												
	Melted 02/11/18 Rolled 02/17/18												

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 D-22. Surrogate Concrete Bridge Deck, #5 Unbent Rebar, Test Nos. STBR-1 through STBR-4

GERDAU

US-ML-KNOXVILLE
1919 TENNESSEE AVENUE N. W.
KNOXVILLE, TN 37921
USA

CERTIFIED MATERIAL TEST REPORT

Page 1/1

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID:
SIMCOTE INC 1645 RED ROCK SAINT PAUL,MN 55119 USA	SIMCOTE INC 1645 RED ROCK ROAD SAINT PAUL,MN 55119-6014 USA	60 (420) TMX	Rebar / #6 (19MM)	0000000000

LENGTH	WEIGHT	HEAT / BATCH
40'00"	38,934 LB	57169293/02

SALES ORDER	CUSTOMER MATERIAL N°	SPECIFICATION / DATE or REVISION
5749568/000030		ASTM A615/A615M-15 E1

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
MN-3676	1326-0000074847	11/21/2017

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	CEqyA706
0.31	0.54	0.008	0.052	0.19	0.24	0.08	0.11	0.026	0.013	0.002	0.42

MECHANICAL PROPERTIES

YS PSI	YS MPa	UTS PSI	UTS MPa	G/L Inch	G/L mm
77630	535	95330	657	8.000	200.0

MECHANICAL PROPERTIES

Elong. %	BendTest
12.50	OK

GEOMETRIC CHARACTERISTICS

%Light %	Def Hgt Inch	Def Gap Inch	DefSpace Inch
5.33	0.056	0.117	0.414

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

Jim Hall
JIM HALL
QUALITY ASSURANCE MGR.
Phone: 865-202-5972 Email: Jim.hall@gerdau.com

Figure D-23. Surrogate Concrete Bridge Deck, #6 Epoxy Rebar, Test Nos. STBR-1 through STBR-4

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	**10/09/2018**
TIME	**08:56:11**
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
40316995-0010	72696240A2	3/16 96 X 240 A572GR50 MILL PLATE	3	3,676.800			10/08/2018

Chemical Analysis

Heat No. B8E871 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
0.1500	0.8400	0.0080	0.0020	0.0400	0.0900	0.0800	0.0300	0.0000	0.2800	0.0310	0.0080	0.0200	0.0010	0.0000	0.0000

Mechanical / Physical Properties

Mill Coil No. B8E8710925

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
72900.000	53000.000	27.60			33	Longitudinal	3.3	-20 F	
73800.000	56300.000	25.60			34	Longitudinal	3.3	-20 F	
75500.000	60200.000	27.10			33	Longitudinal	3.3	-20 F	
73900.000	56100.000	30.00			0	NA			

Batch 0005505764 3 EA 3,676.800 LB Batch 0005505757 8 EA 9,804.800 LB Batch 0005505763 8 EA 9,804.800 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
This test report shall not be reproduced, except in full, without the written approval of Steel & Pipe Supply Company, Inc.

Figure D-24. Post-to-Deck Connections, Embedded Plate, Test Nos. STBR-1 through STBR-4

NUCOR
NUCOR COLD FINISH WISCONSIN, INC.

Mill Certification
12/15/2017

MTR #: E1-139073
7200 S 6th St
OAK CREEK, WI 53154
(414) 764-0220
Fax: (414) 764-2073

Sold To: CENTRAL STEEL & WIRE CO
PO BOX 5100
CHICAGO, IL 60680-5100
(773) 471-3800
Fax: (773) 471-3845

Ship To: CENTRAL STEEL & WIRE CO
3000 W 51ST ST
CHICAGO, IL 60632
(773) 471-3800
Fax: (773) 471-3845

Customer P.O.	CHI 23821	Sales Order	656652.1
Product Group	Cold Finish Bar	Part Number	321504
Grade	4140/4142 ASTM A108 (REPLACES ASTM A331)	Lot #	E1184557
Size	Hex 1.6250 (.0060)	Heat #	NF100786021
Product	HX 1.6250" 4140DH 12-R AN.CD	B.L. Number	E1-239551
Description	CF Grade 4140DH	Load Number	E1-139073
Customer Spec	H-17	Customer Part #	43574

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.

Part Detail: HX 1.6250" 4140 12-R Cold Drawn Annealed
Process: Annealed, Cold Drawn

Melt Date: 3/28/2017

C	Mn	P	S	Si	Cu	Cr	Ni	Mo	Sn	V	Cb
0.40%	0.91%	0.010%	0.027%	0.26%	0.17%	0.92%	0.10%	0.180%	0.010%	0.0070%	0.006%

Al	Ca	Pb
0.030%	0.0010%	0.000%

DI value: 5.28 Melting Mill: Nucor Bar NE Country of Melting: USA

Grain Practice: FINE

Simulated Hardenability Band

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12	J13	J14	J15	J16	J18	J20	J22	J24	J26	J28	J30	J32
56	56	56	56	56	56	55	54	53	51	50	49	48	47	46	45	44	43	42	41	40	39	38	37

Reduction Ratio 17.3 :1 Country of Rolling: USA Rolling Mill: Nucor Bar NE

ASTM E381
Surface: 1 Mid Radius: 1 Center: 2

ASTM E45 Method A (Worst)
Sulfides: T: 2.0 H: 1.0 Alumina: T: 1.0 H: 0.0 Silicates: T: 0.5 H: 0.0 Globular: T: 0.5 H: 0.5

Nicholas Schultz

Nick Schultz
Sales Manager

NBMG-10 October 1, 2017 Page 1 of 2

Material Sold to Central Steel & Wire: Heat: NF100786021 IAC: 43574 PO: CHI23821

Figure D-25. Post-to-Deck Connections, Coupling Nuts, Test Nos. STBR-1 through STBR-4

NUCOR
NUCOR COLD FINISH WISCONSIN, INC.

Mill Certification
12/15/2017

MTR #: E1-139073
7200 S 6th St
OAK CREEK, WI 53154
(414) 764-0220
Fax: (414) 764-2073

Sold To: CENTRAL STEEL & WIRE CO
PO BOX 5100
CHICAGO, IL 60680-5100
(773) 471-3800
Fax: (773) 471-3845

Ship To: CENTRAL STEEL & WIRE CO
3000 W 51ST ST
CHICAGO, IL 60632
(773) 471-3800
Fax: (773) 471-3845

Customer P.O.	CHI 23821	Sales Order	656652.1
Product Group	Cold Finish Bar	Part Number	321504
Grade	4140/4142 ASTM A108 (REPLACES ASTM A331)	Lot #	E1184557
Size	Hex 1.6250 (.0060)	Heat #	NF100786021
Product	HX 1.6250" 4140DH 12-R AN.CD	B.L. Number	E1-239551
Description	CF Grade 4140DH	Load Number	E1-139073
Customer Spec	H-17	Customer Part #	43574

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.

Final Mechanical Date: 4/10/2017

Rockwell B Surface: 90.1 HRB   Brinell Converted Surface: 186.0 HB

Specification Comments:

1. All products produced are weld free.
2. Mercury, in any form, has not been used in the production or testing of this material.

Nicholas Schultz

Nick Schultz
Sales Manager

NBMG-10 October 1, 2017   Page 2 of 2

Figure D-26. Post-to-Deck Connections, Coupling Nuts, Test Nos. STBR-1 through STBR-4

Nov. 26. 2018 3:47PM Fastenal-NELIN No. 5947 P. 2

FASTENAL®

Certificate of Compliance

Sold To:

UNL TRANSPORTATION

Purchase Order: STBR

Job: Item# f3, h1 and i1

Invoice Date: 11/8/2018

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

80 PCS 1"-8 Hot Dipped Galvanized A563 Grade DH Heavy Hex Nut Made In USA SUPPLIED UNDER OUR TRACE NUMBER 210157128 AND UNDER PART NUMBER 38210

450 PCS 3/4"-10 Hot Dipped Galvanized A563 Grade DH Heavy Hex Nut Made In USA SUPPLIED UNDER OUR TRACE NUMBER 210169774 AND UNDER PART NUMBER 38208

80 PCS 1"-8 Hot Dipped Galvanized A563 Grade DH Heavy Hex Nut Made In USA SUPPLIED UNDER OUR TRACE NUMBER 210157128 AND UNDER PART NUMBER 38210

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

Fastenal Account Representative Signature

Ashly Stanczyk

Printed Name

11/29/18

Date

Please check current revision to avoid using obsolete copies.

This document was printed on 11/26/2018 and was current at that time.

Fastenal Store Location/Address

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

Page 1 of 1

Figure D-27. Heavy Hex Nuts, Test Nos. STBR-1 through STBR-4

Job Material Certification   Page 5 of 10

Vulcan THREADED PRODUCTS, INC.

Vulcan Threaded Products
10 Cross Creek Trail
Pelham, AL 35124
Tel (205) 620-5100
Fax (205) 620-5150

JOB MATERIAL CERTIFICATION

Job No: 577899   **Job Information**   **Certified Date:** 4/25/18

Containers: S13899080 S13899191 S13899537 S13899621 S13899814 S13899823

Customer: Conklin and Conklin   **Ship To:** 34201 Seventh Street, Union City, CA 94587

Vulcan Part No: BAR B7 .9144x144 SC

Customer Part No: RAWSTEEL-.910-B

Customer PO No: 18773   **Shipped Qty:** 18233 lbs

Order No: 349871   **Line No:** 1

Note:

Applicable Specifications

Type	Specification	Rev	Amend	Option
-	ASTM F1554 Gd 105 S4	2015		
Heat Treat	ASME SA-193/SA-193M B7	2013		
	ASTM A193 B7	2016		
Quality	EN 10204 3.1	2004		

Test Results

See following pages for tests

Certified Chemical Analysis

Heat No: 58033301/03   Origin: USA

C	Mn	P	S	Si	Cr	Mo	Ni	V	Cu
0.420	0.90	0.013	0.024	0.32	0.93	0.18	0.14	0.005	0.33
Al	Sn	Ti	B	DI	RR	G.S.	Macro S	Macro R	Macro C
0.028	0.004	0.001	0.0002	5.90	57.3:1	Fine	1	1	1
J1	J2	J3	J4	J5	J6	J7	J8	J9	J10
57	57	57	57	57	57	57	57	57	56
J12	J14	J16	J18	J20	J24	J28	J32		
53	51	50	49	49	46	45	42		

Notes

Material was manufactured, tested and inspected as required by the product standard and in accordance with Vulcan's ISO 9001:2015 Quality Management System registered June 30th, 2017. No weld repair performed on the material. No Mercury used in the production of this material. Document is in accordance with EN 10204 - 3.1B of 2004 (3.1).

Plex 4/25/18 10:14 PM vulc.mgri Page 1 of 2

PORTLAND BOLT
PO 37603
INV 76916
100 1" X 144" B7 ATR, BLK 1OF2
SEPTEMBER 25, 2018

37603-2

JUL 02 2018

https://www.plexonline.com/b6b85140-6b08-4c93-8806-4ca98cb7eced/Sales/Report_Job_Cert.asp?Mode... 6/26/2018

Figure D-28. Fully-Threaded Anchor Rods, Test Nos. STBR-1 through STBR-4

Weld Stud Certification

TSA MANUFACTURING • OMAHA

"Creative Steel Products & Services"

www.tsamfgomaha.com
Phones: 402-895-5212
800-228-2948
Fax: 402-895-3297

Customer: ________ Customer Purchase Order # ________

Work Order # ________ **Invoice#** ________

Part # 6HCA050312 **Heat #** 100893527

Weld Stud Size 1/2 X 2-5/8 HCA **QUANTITY:** ________

Certified Chemical Test Report & Chemical Analysis Information

Product meets Standard A108 requirements for all Cold Finished Carbon & Alloy Steel Bars.

AISI Grade 1018AK

- Carbon (C) .17 %
- Manganese (Mn) .70 %
- Sulfur (S) .004 %
- Silicon (Si) .08 %
- Phosphorus (P) .007 %
- Chromium (Cr) ___ %
- Aluminum (Al) ___ %
- Nickel (Ni) ___ %
- Molybdenum (Mo) ___ %
- Other/ ___ ___ %

Certified Mechanical Property Analysis Information

- Tensile Strength = 78000 PSI
- Yield Strength (.2% Offset) = 61000 PSI
- Reduction of Area = 70 Minimum %
- Elongation (In 2 Inches) = 27 %

TSA Manufacturing certifies this product was manufactured from a single Heat Code of material. All Chemical and Mechanical Analysis properties reported above are true and in accordance with **ASTM 29, ASTM A108, ASTM E8/E8M, ATM A370-97, AASHTO/AWSD1.5M /D1.5:2008ANNEX E.**

This product is free from Mercury contamination. Melted and manufactured in the U.S.A.

Signed: Dan Condon, Quality Assurance Manager **Dated:** ________
Dan Condon, Quality Assurance Manager

TSA Weld Studs are certified to AASHTO/AWS 1.5M/D1.5:1.5ANNEX E Standards.

Form:WSCrev2362011

Figure D-29. Shear Studs, Test No. STBR-4

Portland Bolt
& MANUFACTURING COMPANY

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

CERTIFICATE OF CONFORMANCE

For: MIDWEST ROADSIDE SAFETY FACIL
PB Invoice#: 115687
Cust PO#: JIM HOLLOWAY
Date: 12/10/2018
Shipped: 12/12/2018

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

Description: 3/4 X 7-1/2 GALV ASTM A449 ROUND HEAD BOLT

Heat#: 3078659 **Base Steel:** 1045 **Diam:** 3/4

Source: COMMERCIAL METALS CO **Proof Load:** 28,560 LBF

C :	.440	**Mn:**	.740	**P :**	.014	**Hardness:**	255 HBN		
S :	.024	**Si:**	.210	**Ni:**	.070	**Tensile:**	49,240 LBF	**RA:**	.00%
Cr:	.100	**Mo:**	.013	**Cu:**	.210	**Yield:**	0	**Elon:**	.00%
Pb:	.000	**V :**	.000	**Cb:**	.000	**Sample Length:**	0		
N :	.000			**CE:**	.5818	**Charpy:**		**CVN Temp:**	

LOT#19063

Description: 3/4 X 6 GALV ASTM A449 ROUND HEAD BOLT

Heat#: 3078659 **Base Steel:** 1045 **Diam:** 3/4

Source: COMMERCIAL METALS CO **Proof Load:** 28,560 LBF

C :	.440	**Mn:**	.740	**P :**	.014	**Hardness:**	255 HBN		
S :	.024	**Si:**	.210	**Ni:**	.070	**Tensile:**	49,240 LBF	**RA:**	.00%
Cr:	.100	**Mo:**	.013	**Cu:**	.210	**Yield:**	0	**Elon:**	.00%
Pb:	.000	**V :**	.000	**Cb:**	.000	**Sample Length:**	0		
N :	.000			**CE:**	.5818	**Charpy:**		**CVN Temp:**	

LOT#19063

Figure D-30. Round Head Bolts, Test Nos. STBR-1 through STBR-4

Portland Bolt & MANUFACTURING COMPANY

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

CERTIFICATE OF CONFORMANCE

For: MIDWEST ROADSIDE SAFETY FACIL
PB Invoice#: 115687
Cust PO#: JIM HOLLOWAY
Date: 12/10/2018
Shipped: 12/12/2018

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

Description: 3/4 X 9-1/2 GALV ASTM F3125 GRADE A325 HEAVY HEX BOLT

Heat#: 3078659 | **Base Steel:** 1045 | **Diam:** 3/4

Source: COMMERCIAL METALS CO | **Proof Load:** 28,560 LBF

C :	.440	**Mn:**	.740	**P :**	.014	**Hardness:**	285 HBN		
S :	.024	**Si:**	.210	**Ni:**	.070	**Tensile:**	48,220 LBF	**RA:**	.00%
Cr:	.100	**Mo:**	.013	**Cu:**	.210	**Yield:**	0	**Elon:**	.00%
Pb:	.000	**V :**	.000	**Cb:**	.000	**Sample Length:**	0		
N :	.000			**CE:**	.5818	**Charpy:**		**CVN Temp:**	

LOT#19059

Coatings:
ITEMS HOT DIP GALVANIZED PER ASTM F2329/A153C

Other:
ALL ITEMS MELTED & MANUFACTURED IN THE USA

By: ______
Certification Department Quality Assurance
Dane McKinnon

Figure D-31. Hex Head Bolts, Test Nos. STBR-1 through STBR-4

CHARTER STEEL
A Division of
Charter Manufacturing Company, Inc.

1658 Cold Springs Road
Saukville, Wisconsin 53080
(262) 268-2400
1-800-437-8789
Fax (262) 268-2570

EMAIL

CHARTER STEEL TEST REPORT

Melted in USA Manufactured in USA

Fontana Fasteners Inc
3595 West State Road 28
Frankfort,IN-46041

Cust P.O.	135820
Customer Part #	955612619A1
Charter Sales Order	10240117
Heat #	10552460
Ship Lot #	4534855
Grade	LEBA1 M SK FG RHQ 1-1/32 RNDCOIL
Process	HRCC
Finish Size	1-1/32
Ship date	11-JUN-18

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed below and that it satisfies these requirements. The recording of false, fictitious and fraudulent statements or entries on this document may be punishable as a felony under federal statute.

Test results of Heat Lot # 10552460

Lab Code: 7388

CHEM	C	MN	P	S	SI	NI	CR	MO	CU	SN	V
%Wt	.31	.92	.007	.007	.230	.05	.17	.09	.09	.006	.003
	AL	N	B	TI	NB	SB	AS				
	.027	.0070	.0028	.023	.002	.001	.003				

JOMINY(HRC)

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12
51	50	50	49	48	48	43	37	34	31	28	27
J13	J14	J15	J16	J18	J20	J22	J24	J26	J28		
25	25	25	24	23	22	21	21	20	20		

JOMINY LAB=0358-01 JOMINY SAMPLE TYPE ENGLISH=R CAT DI=3.22

Test results of Rolling Lot # 1241818

	# of Tests	Min Value	Max Value	Mean Value	
ROCKWELL B (HRBW)	3	81	83	82	RB LAB = 0358-02
ROD SIZE (Inch)	12	1.024	1.037	1.030	
ROD OUT OF ROUND (Inch)	6	.005	.008	.007	

NUM DECARB=1
REDUCTION RATIO=36:1

AVE DECARB (Inch)=.005

Specifications: Manufactured per Charter Steel Quality Manual Rev Date 05/12/17
Charter Steel certifies this product is indistinguishable from background radiation levels by having process radiation detectors in place to measure for the presence of radiation within our process & products.
Meets customer specifications with any applicable Charter Steel exceptions for the following customer documents:
Customer Document = LE 1.1 Revision = 9 Dated = 27-NOV-07

Additional Comments: GRADE 30 Cr Mn B1

Melt Source:
Charter Steel
Saukville, WI, USA

Trip: 1271794

ACCREDITED
Testing Laboratory

This MTR supersedes all previously dated MTRs for this order

Janice Barnard

Janice Barnard Division Mgr. of Quality Assurance
barnardJ@chartersteel.com
Printed Date : 06/11/2018

Page 1 of 2

Figure D-32. Vertical Plate Hex Headed Bolts, Tests Nos. STBR-1 through STBR-4

The following statements are applicable to the material described on the front of this Test Report:

1. Except as noted, the steel supplied for this order was melted, rolled, and processed in the United States meeting DFARS compliance, LEEDS compliance, REACH compliance, ROHS-WEEE compliance, and Conflict Materials Restrictions.
2. Mercury was not used during the manufacture of this product, nor was the steel contaminated with mercury during processing.
3. Unless directed by the customer, there are no welds in any of the coils produced for this order.
4. The laboratory that generated the analytical or test results can be identified by the following key:

Certificate Number	Lab Code	Laboratory		Address
0358-01	7388	**CSSM**	Charter Steel Melting Division	1658 Cold Springs Road, Saukville, WI 53080
0358-02	8171	**CSSR/ CSSP**	Charter Steel Rolling/ Processing Division	1658 Cold Springs Road, Saukville, WI 53080
0358-03	123633	**CSFP**	Charter Steel Ohio Processing Division	6255 US Highway 23, Rising Sun, OH 43457
0358-04	125544	**CSCM/ CSCR**	Charter Steel Cleveland	4300 E. 49th St., Cuyahoga Heights, OH 44125-1004
*	*	--	Subcontracted test performed by laboratory not in Charter Steel System	

5. When run by a Charter Steel laboratory, the following tests were performed according to the latest revisions of the specifications listed below, as noted in the Charter Steel Laboratory Quality Manual:

Test	Specifications	CSSM	CSSR/ CSSP	CSFP	CSCM/ CSCR
Chemistry Analysis	ASTM E415; ASTM E1019	X			X
Macroetch	ASTM E381	X			X
Hardenability (Jominy)	ASTM A255; SAE J406; JIS G0561	X			X
Grain Size	ASTM E112	X	X	X	X
Tensile Test	ASTM E8; ASTM A370		X	X	X
Rockwelll Hardness	ASTM E18; ASTM A370	X	X	X	X
Microstructure (spheroidization)	ASTM A892		X	X	
Inclusion Content (Methods A, E)	ASTM E45		X		X
Decarburization	ASTM E1077		X	X	X

Charter Steel has been accredited to perform all of the above tests by the American Association for Laboratory Accreditation (A2LA). These accreditations expire 01/31/19. All other test results associated with a Charter Steel laboratory that appear on the front of this report, if any, were performed according to documented procedures developed by Charter Steel and are not accredited by A2LA.

6. The test results on the front of this report are the true values measured on the samples taken from the production lot. They do not apply to any other sample.
7. This test report cannot be reproduced or distributed except in full without the written permission of Charter Steel. The primary customer whose name and address appear on the front of this form may reproduce this test report subject to the following restrictions:
   - It may be distributed only to their customers
   - Both sides of all pages must be reproduced in full
8. This certification is given subject to the terms and conditions of sale provided in Charter Steel's acknowledgement (designated by our Sales Order number) to the customer's purchase order. Both order numbers appear on the front page of this Report.
9. Where the customer has provided a specification, the results on the front of this test report conform to that specification unless otherwise noted on this test report.

ACCREDITED
Testing Laboratory

Page **2** of **2**

Figure D-33. Vertical Plate Hex Headed Bolts, Tests Nos. STBR-1 through STBR-4

CHARTER STEEL
A Division of
Charter Manufacturing Company, Inc.

EMAIL

1658 Cold Springs Road
Saukville, Wisconsin 53080
(262) 268-2400
1-800-437-8789
Fax (262) 268-2570

CHARTER STEEL TEST REPORT

Melted in USA Manufactured in USA

Cust P.O.	118874
Customer Part #	955610992A1
Charter Sales Order	30106122
Heat #	10415990
Ship Lot #	4385224
Grade	LEBA1 M SK FG RHQ 25/64
Process	HRCC
Finish Size	25/64
Ship date	09-JAN-16

Fontana Fasteners Inc
3595 West State Road 28
Frankfort,IN-46041

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed below and that it satisfies these requirements. The recording of false, fictitious and fraudulent statements or entries on this document may be punishable as a felony under federal statute.

Test results of Heat Lot # 10415990

Lab Code: 7388

CHEM	C	MN	P	S	SI	NI	CR	MO	CU	SN	V
%Wt	.32	.91	.007	.008	.220	.04	.17	.08	.08	.006	.004
	AL	N	B	TI	NB	SB	AS				
	.023	.0060	.0028	.022	.001	.002	.004				

JOMINY(HRC)

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12
51	51	51	50	49	48	45	39	33	30	27	25
J13	J14	J15	J16	J18	J20	J22					
24	23	23	22	21	21	20					

JOMINY LAB=0358-01 JOMINY SAMPLE TYPE ENGLISH=R CAT DI=3.18

Test results of Rolling Lot # 1174376

	# of Tests	Min Value	Max Value	Mean Value	
TENSILE (KSI)	1	91.7	91.7	91.7	TENSILE LAB = 0358-02
REDUCTION OF AREA (%)	1	65	65	65	RA LAB = 0358-02
ROCKWELL B (HRBW)	1	91	91	91	RB LAB = 0358-02
ROD SIZE (Inch)	4	.388	.393	.391	
ROD OUT OF ROUND (Inch)	1	.005	.005	.005	

NUM DECARB=1 AVE DECARB (Inch)=.003
REDUCTION RATIO=252:1

Specifications: Manufactured per Charter Steel Quality Manual Rev Date 12/12/13
Charter Steel certifies this product is indistinguishable from background radiation levels by having process radiation detectors in place to measure for the presence of radiation within our process & products.
Meets customer specifications with any applicable Charter Steel exceptions for the following customer documents:
Customer Document = LE 1.1 Revision = 9 Dated = 27-NOV-07

Additional Comments: GRADE 30 Cr Mn B1

Melt Source:
Charter Steel
Saukville, WI, USA

Rem: Load1,Fax0,Mail0

ACCREDITED
Testing Laboratory

This MTR supersedes all previously dated MTRs for this order
Janice Barnard
Janice Barnard
Manager of Quality Assurance
Printed Date : 01/09/2016

Page 1 of 2

Figure D-34. Spacer Tube Hex Headed Bolts, Tests Nos. STBR-1 through STBR-4

The following statements are applicable to the material described on the front of this Test Report:

1. Except as noted, the steel supplied for this order was melted, rolled, and processed in the United States meeting DFARS compliance, LEEDS compliance, REACH compliance, ROHS-WEEE compliance, and Conflict Materials Restrictions.
2. Mercury was not used during the manufacture of this product, nor was the steel contaminated with mercury during processing.
3. Unless directed by the customer, there are no welds in any of the coils produced for this order.
4. The laboratory that generated the analytical or test results can be identified by the following key:

Certificate Number	Lab Code	Laboratory		Address
0358-01	7388	**CSSM**	Charter Steel Melting Division	1658 Cold Springs Road, Saukville, WI 53080
0358-02	8171	**CSSR/ CSSP**	Charter Steel Rolling/ Processing Division	1658 Cold Springs Road, Saukville, WI 53080
0358-03	123633	**CSFP**	Charter Steel Ohio Processing Division	6255 US Highway 23, Rising Sun, OH 43457
0358-04	125544	**CSCM/ CSCR**	Charter Steel Cleveland	4300 E. 49th St., Cuyahoga Heights, OH 44125-1004
*	*	--	Subcontracted test performed by laboratory not in Charter Steel System	

5. When run by a Charter Steel laboratory, the following tests were performed according to the latest revisions of the specifications listed below, as noted in the Charter Steel Laboratory Quality Manual:

Test	Specifications	CSSM	CSSR/ CSSP	CSFP	CSCM/ CSCR
Chemistry Analysis	ASTM E415; ASTM E1019	X			X
Macroetch	ASTM E381	X			X
Hardenability (Jominy)	ASTM A255; SAE J406; JIS G0561	X			X
Grain Size	ASTM E112	X	X	X	X
Tensile Test	ASTM E8; ASTM A370		X	X	X
Rockwelll Hardness	ASTM E18; ASTM A370	X	X	X	X
Microstructure (spheroidization)	ASTM A892		X	X	
Inclusion Content (Methods A, E)	ASTM E45		X		X
Decarburization	ASTM E1077		X	X	X

Charter Steel has been accredited to perform all of the above tests by the American Association for Laboratory Accreditation (A2LA). These accreditations expire 03/31/17. All other test results associated with a Charter Steel laboratory that appear on the front of this report, if any, were performed according to documented procedures developed by Charter Steel and are not accredited by A2LA.

6. The test results on the front of this report are the true values measured on the samples taken from the production lot. They do not apply to any other sample.
7. This test report cannot be reproduced or distributed except in full without the written permission of Charter Steel. The primary customer whose name and address appear on the front of this form may reproduce this test report subject to the following restrictions:
   - It may be distributed only to their customers
   - Both sides of all pages must be reproduced in full
8. This certification is given subject to the terms and conditions of sale provided in Charter Steel's acknowledgement (designated by our Sales Order number) to the customer's purchase order. Both order numbers appear on the front page of this Report.
9. Where the customer has provided a specification, the results on the front of this test report conform to that specification unless otherwise noted on this test report.

ACCREDITED
Testing Laboratory

Page **2** of **2**

Figure D-35. Spacer Tube Hex Headed Bolts, Tests Nos. STBR-1 through STBR-4

FASTENAL®

Certificate of Compliance

Sold To:

UNL TRANSPORTATION

Purchase Order: STBR

Job: part g6

Invoice Date: 11/14/2018

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

11 PCS 1"-8 x 1-1/2" Grade 5 Plain Finish Hex Cap Screw SUPPLIED UNDER OUR TRACE NUMBER 120271368 AND UNDER PART NUMBER 12459

49 PCS 1"-8 x 1-1/2" Grade 5 Plain Finish Hex Cap Screw SUPPLIED UNDER OUR TRACE NUMBER 190099651 AND UNDER PART NUMBER 12459

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

Fastenal Account Representative Signature

Nathan Gemmell

Printed Name

11/14/18

Date

Please check current revision to avoid using obsolete copies.

This document was printed on 11/14/2018 and was current at that time.

Fastenal Store Location/Address

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

Page 1 of 1

Figure D-36. 1½-in. Long Hex-Headed Bolts, Tests Nos. STBR-1 through STBR-4

JINAN STAR FASTENER CO., LTD

NO.75 CUIPING STREET PINGYIN JINAN CHINA
TEL: 0086 531 87896380 FAX: 0086 531 87871032
E-mail: zhangyuhua@star-fastener.com

CERTIFICATE OF INSPECTION

HY038.1.3-12

Manufacturering Date:2017-1-7 **DATE: 2017-1-13**

Customer Part Number客户产品代号	12459							
Customer Control (PO) Number客户订单号	120271368							
Product Description产品描述	HCS 1-8X1-1/2 P5							
Surface Condition表面处理	PLAIN							
Head Marking头部标记	3radial lines and 01RL							
Lot Size (Manufactured QTY):生产数量	2043pcs							
Lot Size (QTY Shipped):装运数量	2033pcs							
Lot Number订单号	FAS16235							
Mechanical properties机械性能要求	SAE J429-2014 Gr5							
Material type:	40CR			Heat Number		J11503054		
	C%	Mn%	Si%	S%	P%	Ni%	Cr%	Cu%
Chemical composition化学成份:标准	0.25-0.55			max0.025	max0.025			
Chemical composition化学成份:实测值	0.40	0.75	0.22	0.003	0.015	0.015	1.002	0.014
Sampling Plan Used 使用的抽样方案	**Dimensional as per ASME B18.18-2011/Mechanical Property as per F1470-2012**							
Specification技术要求:	specification 檢測标准	Test method 檢測方法	Standard 标准	单位	Test value 实测值	Sampling Plan 抽样方案	ACC 合格	REJ 不合格
Width across Flat对边尺寸	ASME B18.2.1-2012		1.469-1.5	in	1.489-1.495	5/0	5	0
Width across Corners对角尺寸	ASME B18.2.1-2012		1.675-1.732	in	1.689-1.692	5/0	5	0
Height高度	ASME B18.2.1-2012		0.591-0.627	in	0.608-0.614	5/0	5	0
Length总长度	ASME B18.2.1-2012		1.40-1.50	in	1.495-1.498	15/0	15	0
Major 大径	ANSI B1.1-2003		0.983-0.998	in	0.985-0.988	15/0	15	0
Thread 螺纹	ANSI B1.1-2003		2A GO		2A GO	15/0	15	0
	ANSI B1.1-2003		2A NO GO		2A NO GO	15/0	15	0
Surface hardness表面硬度	SAE J429-2014	SAE J429-2014	max54	30N	47.7-48.7	5/0	5	0
Core Hardness芯部硬度	SAE J429-2014	ASTM F606-2014	25-34	HRC	28.9-29.6	15/0	15	0
Decarburized脱碳	SAE J429-2014	ASTM F2328-2014	max0.038	in	0	5/0	5	0
Appearance外观	ASTM F788-2013		Visual		OK	29/0	29	0

Parts are manufactured and tested according to above specification, we certify that this is a true representation of information provided by manufacturer
产品是按照上述要求进行生产和检测的，我们证明厂家提供的信息是真实的

Signature: Fu Yan Jun The requirements are fulfilled
Title: Quality Manager Inspector（终检员）:马付彬

Figure D-37. 1½-in. Long Hex-Headed Bolts, Tests Nos. STBR-1 through STBR-4

INFASCO

LOT NO.: **1701-50272**

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 2017-01-26

DESCRIPTION AND MARKING: HEX HD CAP SCREW J429-5 FNA UNC FT P
HOLLOW TRIANGLE & 3 RADIAL LINES

SIZE	GRADE	QUANTITY
1-8 X 1 1/2	1037MS	9,950

HEAT CHEMICAL ANALYSIS (provided by steel supplier)

HEAT NO.	C %	Mn %	P %	S %	Si %				
10440680	0.39	1.00	0.008	0.008	0.22				

METHOD	PROOF LOAD	WEDGE TENSILE STRENGTH	SHEAR STRENGTH	ASTM E18 SURFACE HARDNESS (HR 30N)	ASTM F606 CORE HARDNESS (ROCKWELL)	MICRO HARDNESS	COATING THICKNESS
SPEC. MIN.					HRC 25.0		
SPEC. MAX:				54.0	HRC 34.0		
SAMPLE NO.1				53.5	HRC 30.0		
NO.2				51.8	30.8		
NO.3				52.7	30.5		
NO.4				51.8	28.7		

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:
SAE J-429, ASME B18.2.1,THREADS PER ASME B1.1 CLASS 2A UNLESS OTHERWISE SPECIFIED.

* MATERIAL TOO SHORT TO BE TESTED FOR TENSILE AND PROOF LOAD.

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: 2017-01-27 Page 1 of 1

Figure D-38. 1½-in. Long Hex-Headed Bolts, Tests Nos. STBR-1 through STBR-4

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 12/19/2017
TIME 12:50:41
USER WILLIAMR

SOLD TO
66031-1127

SHIP TO
18271
SPS Warehouse 0045
401 New Century Parkway
NEW CENTURY KS

Order	Material No.	Description	Quantity	Weight	Customer Part	Customer PO	Ship Date
40299425-0020	70872178	1/4 72 X 178 A36 STP MIL PLT	6	5,452.140			12/19/2017

Chemical Analysis

Heat No. 17126641 Vendor BIG RIVER STEEL LLC DOMESTIC Mill BIG RIVER STEEL LLC 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
0.1900	0.8200	0.0070	0.0020	0.0300	0.0400	0.0500	0.0120	0.0001	0.1100	0.0260	0.0010	0.0040	0.0020	0.0077	0.0055

Mechanical / Physical Properties

Mill Coil No. 17126641-05

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
75700.000	54100.000	28.10			0	NA			
71300.000	52000.000	30.60			0	NA			
71800.000	52000.000	33.20			0	NA			
74900.000	55400.000	28.70			0	NA			

Batch 0005075120 6 EA 5,452.140 LB

Batch 0005075119 7 EA 6,360.830 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 D-39. Compression Plate Washers, Test Nos. STBR-1 through STBR-4

CERTIFIED MATERIAL TEST REPORT

Page 1/1

GERDAU

US-ML-JACKSON TN
801 GERDAU AMERISTEEL ROAD
JACKSON, TN 38305
USA

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID:
STATE STEEL SUPPLY CO INC 13433 CENTECH RD OMAHA,NE 68138-3492 USA	STATE STEEL SUPPLY CO INC SIOUX CITY,IA 51102-3224 USA	GGMULTI	Flat Bar / 3/8 X 2 1/2	0000183021

LENGTH	WEIGHT	HEAT / BATCH
20'00"	9,570 LB	63183983/02

SALES ORDER	CUSTOMER MATERIAL Nº	SPECIFICATION / DATE or REVISION
7144772/000010		ASTM A529-14, A572-15 ASTM A6-17,A36-14, ASME SA-36 ASTM A709-17, AASHTO M270-15 CSA G40.20-13/G40.21-13

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
P81030SW101	1333-0000117696	11/01/2018

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	Nb %	Al %
0.16	0.74	0.011	0.023	0.20	0.30	0.08	0.12	0.022	0.013	0.001	0.010	0.001

CHEMICAL COMPOSITION

CEqyA529 %
0.38

MECHANICAL PROPERTIES

YS PSI	UTS PSI	YS MPa	UTS MPa	G/L Inch	G/L mm
58400	77420	403	534	8.000	200.0
58450	77860	403	537	8.000	200.0

MECHANICAL PROPERTIES

Elong. %
24.00
24.00

COMMENTS / NOTES

This grade meets the requirements for the following grades:
ASTM Grades: A36; A529-50; A572-50; A709-36; A709-50
CSA Grades: 44W; 50W
AASHTO Grades: M270-36; M270-50
ASME Grades: SA36

P81030SW101

63183983

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) 267-1071 Email: Bhaskar.Yalamanchili@gerdau.com

Ben Lovell
BEN LOVELL
QUALITY ASSURANCE MGR.
Phone: (731) 423-5213 Email: benjamin.lovell@gerdau.com

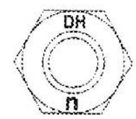
Figure D-40. Compression Anchorage Plates, Test No. STBR-4

NUCOR
FASTENER DIVISION

LOT NO.
409769D

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1600

CUSTOMER NO/NAME
8001 FASTENAL COMPANY-KS NUCOR ORDER # 89170
TEST REPORT SERIAL# FB572639 CUST PART # 38208
TEST REPORT ISSUE DATE 7/24/18
DATE SHIPPED 8/21/18 CUSTOMER P.O. # 210169774
NAME OF LAB SAMPLER: LISA EDGAR, LAB TECHNICIAN



*****************CERTIFIED MATERIAL TEST REPORT*****************

NUCOR PART NO	QUANTITY	LOT NO.	DESCRIPTION
175657	9000	409769D	3/4-10 GR DH HV H.D.G. HEX NUT HDG/GREEN LUBE

MANUFACTURE DATE 6/12/18

--CHEMISTRY MATERIAL GRADE -1045L
**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER NUCOR STEEL - SOUTH CAROL

MATERIAL NUMBER	HEAT NUMBER	C	MN	P	S	SI
RM032459	DL18102990	.45	.66	.003	.018	.20

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A563-15

SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD 50100 LBS	TENSILE STRENGTH (LBS)	DEG-WEDGE STRESS (PSI)
N/A	27.1	PASS	N/A	N/A
N/A	27.4	PASS	N/A	N/A
N/A	28.0	PASS	N/A	N/A
N/A	27.2	PASS	N/A	N/A
N/A	26.0	PASS	N/A	N/A
AVERAGE VALUES FROM TESTS	27.1			

PRODUCTION LOT SIZE 200000 PCS

--VISUAL INSPECTION IN ACCORDANCE WITH ASTM A563-15 160 PCS. SAMPLED LOT PASSED

--COATING - HOT DIP GALVANIZED TO ASTM F2329-13 - GALVANIZING PERFORMED IN THE U.S.A.
1. 0.00385 2. 0.00271 3. 0.00299 4. 0.00268 5. 0.00258 6. 0.00328 7. 0.00297
8. 0.00806 9. 0.00279 10. 0.00296 11. 0.00308 12. 0.00371 13. 0.00243 14. 0.00362
15. 0.00371
AVERAGE THICKNESS FROM 15 TESTS .00343

--HEAT TREATMENT - AUSTENITIZED, OIL QUENCHED & TEMPERED (MIN 800 DEG F)

--DIMENSIONS PER ASME B18.2.6-2010

CHARACTERISTIC	#SAMPLES TESTED	MINIMUM	MAXIMUM
Width Across Corners	8	1.404	1.420
Thickness	32	0.720	0.744

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO INTENTIONAL ADDITIONS OF BISMUTH, SELENIUM, TELLURIUM, OR LEAD WERE USED IN THE STEEL USED TO PRODUCE THIS PRODUCT.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

ACCREDITED

MECHANICAL FASTENER
CERTIFICATE NO. A2LA 0139.01
EXPIRATION DATE 12/31/19

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

Bob Haywood

BOB HAYWOOD
QUALITY ASSURANCE SUPERVISOR

Page 1 of 1

Figure D-41. Heavy Hex Nuts, Test Nos. STBR-1 through STBR-4

STAMPING THE FUTURE

WROUGHT WASHER MFG., INC.

September 10, 2018

Certification of Compliance

003280
FASTENAL COMPANY PURCHASING
P.O. BOX 978
WINONA, MN 55987

Wrought Washer Ordr/Lot Number
311174

HT ORDER 308316

Heat Number	Chemical Analysis C	Mn	P	S	Si
270517	0.340	0.840	0.011	0.000	0.220

Purchase Order Number	Part Description	Date Shipped	Quantity Shipped
110278649	3/4 F436 S MARK MECH GALV 01133551	09/10/2018	10,125

We hereby certify that the subject parts conform to the requirements of the applicable specification indicated for the subject parts and are in complete conformance to F436-11. We hereby certify that the subject parts were hardened to RC 38-45. We hereby certify that the subject parts were mechanically galvanized in accordance with specification ASTM B695 CLASS 55.

We hereby certify that all statutory requirements as to American Production and Labor Standards and all conditions of purchase applicable to the transaction have been complied with and that the subject parts were melted and manufactured in the U.S.A. No weld repairs were made to the material.

Truly yours,
Wrought Washer Mfg., Inc.

Paul J. Seggelink

Paul Seggelink
Q.A. Manager

Susan M. Daoust

Sworn and subscribed before me on September 10, 2018
My commission expires April 24, 2021

(031) SMARK, HT, MECH GALV, F436
WW INTERNAL USE : 66037502/002/017297/60937

1901 CHICORY RD. • MOUNT PLEASANT, WI 53403 • PHONE (262) 554-9550 • FAX (262) 554-9584
VISIT OUR WEBSITE: www.wroughtwasher.com

Figure D-42. Round Flat Washers, Test Nos. STBR-1 through STBR-4

STAMPING THE FUTURE

WROUGHT WASHER MFG., INC.

September 17, 2018

Certification of Compliance

003280
FASTENAL COMPANY PURCHASING
P.O. BOX 978
WINONA, MN 55987

Wrought Washer
Ordr/Lot Number
312060

HT ORDER 309012

Heat Number	**Chemical Analysis**				
	C	**Mn**	**P**	**S**	**Si**
281047	0.350	0.820	0.010	0.000	0.215

Purchase Order Number	**Part Description**	**Date Shipped**	**Quantity Shipped**
110278649	3/4 F436 S MARK MECH GALV 01133551	09/17/2018	7,875

We hereby certify that the subject parts conform to the requirements of the applicable specification indicated for the subject parts and are in complete conformance to F436-11. We hereby certify that the subject parts were hardened to RC 38-45. We hereby certify that the subject parts were mechanically galvanized in accordance with specification ASTM B695 CLASS 55.

We hereby certify that all statutory requirements as to American Production and Labor Standards and all conditions of purchase applicable to the transaction have been complied with and that the subject parts were melted and manufactured in the U.S.A. No weld repairs were made to the material.

Truly yours,
Wrought Washer Mfg., Inc.

Paul J. Seggelink

Paul Seggelink
Q.A. Manager

Susan M. Daoust

Sworn and subscribed before me on September 17, 2018
My commission expires April 24, 2021

SUSAN M. DAOUST
NOTARY
PUBLIC
STATE OF WISCONSIN

(031) SMARK, HT, MECH GALV, F436
WW INTERNAL USE : 66037503/002/017297/61189

1901 CHICORY RD. • MOUNT PLEASANT, WI 53403 • PHONE (262) 554-9550 • FAX (262) 554-9584
VISIT OUR WEBSITE: www.wroughtwasher.com

Figure D-43. Round Flat Washers, Test Nos. STBR-1 through STBR-4

STAMPING THE FUTURE
WROUGHT WASHER MFG., INC.

March 26, 2018

Certification of Compliance

003280
FASTENAL COMPANY PURCHASING
P.O. BOX 978
WINONA, MN 55987

Wrought Washer Ordr/Lot Number
307714

HT ORDER 304829

Heat Number	Chemical Analysis C	Mn	P	S	Si
277411	0.340	0.830	0.009	0.001	0.220

Purchase Order Number	Part Description	Date Shipped	Quantity Shipped
110263920	3/4 F436 S MARK MECH GALV 0133551 PO : 110263920	03/26/2018	15,750

We hereby certify that the subject parts conform to the requirements of the applicable specification indicated for the subject parts and are in complete conformance to F436-11. We hereby certify that the subject parts were hardened to RC 38-45. We hereby certify that the subject parts were mechanically galvanized in accordance with specification ASTM B695 CLASS 55.

We hereby certify that all statutory requirements as to American Production and Labor Standards and all conditions of purchase applicable to the transaction have been complied with and that the subject parts were melted and manufactured in the U.S.A. No weld repairs were made to the material.

Truly yours,
Wrought Washer Mfg., Inc.

Paul J. Seggelink

Paul Seggelink
Q.A. Manager

Susan M. Daoust

Sworn and subscribed before me on March 26, 2018
My commission expires April 24, 2021

SUSAN M. DAOUST NOTARY PUBLIC STATE OF WISCONSIN

(031) SMARK, HT, MECH GALV, F436
WW INTERNAL USE : 65494901/001/017297/59980

1901 CHICORY RD. • MOUNT PLEASANT, WI 53403 • PHONE (262) 554-9550 • FAX (262) 554-9584
VISIT OUR WEBSITE: www.wroughtwasher.com

Figure D-44. Round Flat Washers, Test No. STBR-4

METALLURGICAL TESTING CERTIFICATION

D058721

NUCOR
SHEET MILL GROUP

Nucor Steel-Crawfordsville
4537 South Nucor Road
Crawfordsville, IN 47933-0907

Certificate Number: 734212
Date Issued: 08/26/2017

Page: 2 of 3

Order Number: 291268 - 0006
Order Dimensions: 0.1230 in X 53.6250 in
HRPO,MILL,1035

ASTM A568-15
SAE J403-14 1035

Customer Name: WROUGHT WASHER MFG INC
Customer Address: 2100 S BAY ST

MILWAUKEE WI 53207

Release Order:
Cust PO Number: H3344

Coil Number: 2206517.000

Rockwell B: TAIL 84

Part Number
842129-100 TONS
Weight: 42,660 LBS

CHEMICAL ANALYSIS

Heat	Slab	C	Mn	P	S	Si	Cu	Sn	Ni	Cr	Mo	Al	N	V	Nb	Ti	B	Sb
277411	03	0.34	0.830	0.009	0.001	0.220	0.151	0.010	0.045	0.063	0.026	0.027	0.007	0.003	0.001	0.003	<0.0005	0.001

WE HEREBY CERTIFY THE ABOVE IS CORRECT AS CONTAINED IN THE RECORDS OF THE CORPORATION
MELTED AND ROLLED IN THE USA

QF-0261 11/29/2012

Eric E. Gallo
NUCOR QUALITY ASSURANCE

Figure D-45. Plate Washers, Test No. STBR-4

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 12/19/2017
TIME 12:50:41
USER WILLIAMR

SOLD TO
66031-1127

SHIP TO
18271
SPS Warehouse 0045
401 New Century Parkway
NEW CENTURY KS

Order	Material No.	Description	Quantity	Weight	Customer Part	Customer PO	Ship Date
40299425-0020	70872178	1/4 72 X 178 A36 STP MIL PLT	6	5,452.140			12/19/2017

Chemical Analysis

Heat No. 17126641 Vendor BIG RIVER STEEL LLC DOMESTIC Mill BIG RIVER STEEL LLC 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
0.1900	0.8200	0.0070	0.0020	0.0300	0.0400	0.0500	0.0120	0.0001	0.1100	0.0260	0.0010	0.0040	0.0020	0.0077	0.0055

Mechanical / Physical Properties

Mill Coil No. 17126641-05

Tensile	Yield	Elong	Rckwl	Grain	Charpy	Charpy Dr	Charpy Sz	Temperature	Olsen
75700.000	54100.000	28.10			0	NA			
71300.000	52000.000	30.60			0	NA			
71800.000	52000.000	33.20			0	NA			
74900.000	55400.000	28.70			0	NA			

Batch 0005075120 6 EA 5,452.140 LB

Batch 0005075119 7 EA 6,360.830 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 D-46. Plate Washers, Test Nos. STBR-1 through STBR-4

Portland Bolt
& MANUFACTURING COMPANY

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

For: MIDWEST ROADSIDE SAFETY
PB Invoice#: 115488
Cust PO#: SHAUN
Date: 11/09/2018
Shipped: 11/08/2018

CERTIFICATE OF CONFORMANCE

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

Product:
ASTM F1554G36 ALL THRD ROD
ASTM F1554G105 ALL THRD ROD

Coatings:
ITEMS HOT DIP GALVANIZED PER ASTM F2329/A153C

Other:
ALL ITEMS MELTED & MANUFACTURED IN THE USA

By: ______________________
Certification Department Quality Assurance
Dane McKinnon

Figure D-47. Full-Threaded Rods, Test Nos. STBR-1, STBR-2, and STBR-4

18472 3/4 ATR

CUSTOMER CONKLIN & CONKLIN INC
34201 7TH STREET
UNION CITY, CA 94587

CASCADE STEEL Rolling Mills Inc.
A Schnitzer Company

CERTIFIED MILL TEST REPORT
(CMTR)
3200 NORTH HIGHWAY 99W
McMINNVILLE, OREGON 97128
(503) 472-4181 FAX (503) 434-5739

DATE 6-06-18
BILL OF LADING 10143690
PAGE 1 OF 1

DESCRIPTION HEAT NO. / PRODUCT / GRADE	YIELD KSI	TENSILE KSI	ELONG. % 8 INCHES	REDUCTION %	Melted Rolled	Shipped Lbs/Tons	Melt Lbs Roll Lbs
*145918 17.5MM ROD MOD A36/1010 GRADE AISI 1010 for ASTM A36	42.0	64.5	24	49	04/13/18 05/07/18	47,566 23.8	224,202 166,558
Meets chemistry of A36 Intended to meet A36 mechanicals after fab	40.4	63.5	24				

PORTLAND BOLT
PO 37787
INV 76985
200 - 3/4" X 144" ATR, BLK
OCTOBER 5, 2018

CHEMICAL ANALYSIS

HEAT NO.	C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	V %	Mo %	Sn %	Cb %
145918	.16	.51	.008	.023	.14	.13	.05	.05	.003	.015	.006	.000

PO NUMBER(S): 18646

CERTIFIED BY: Jeff Kramer
Jeff Kramer
Quality Assurance Manager

* ALL MELTING AND MANUFACTURING PROCESSES FOR THE MATERIALS OCCURRED IN THE UNITED STATES.

F016-1 02

Figure D-48. Full-Threaded Rods, Test Nos. STBR-1, STBR-2, and STBR-4

Appendix E. Vehicle Center of Gravity Determination

Date:	1/25/2019	Test Name:	STBR-1	VIN:	1FVACXCS57HX61818
Year:	2007	Make:	Freightliner	Model:	M2 106

Vehicle CG Determination

	VEHICLE	Equipment	Weight (lb.)	Vertical CG (in.)	Vertical M (lb.-in.)
	+	Unballasted Truck (Curb)	13725	48.987	672347.972
	+	Hub	34	20.0	680.0
	+	Brake activation cylinder & frame	8	48.25	386.0
	+	Pneumatic tank (Nitrogen)	31	46.0	1426.0
	+	Strobe/Brake Battery	5	49.75	248.75
	+	Tow Pin Plate	9	12.75	114.75
	+	Brake Receiver/Wires	6	101.0	606.0
	+	Cab DAS Unit & Plate	11	51.5	566.5
	+	CG DAS Units & Enclosure	24	30.5	732.0
	+	Rear Axle DAS Unit and Enclosure	30	30.5	915.0
	-	Battery	-106	38.0	-4028.0
	-	Oil	-45	23.25	-1046.25
	-	Interior	-118	45.5	-5369.0
	-	Fuel	0	12.5	0
	-	Coolant	-59	56.875	-3355.625
	-	Washer fluid	-7	41.0	-287.0
	+	Onboard Supplemental Battery	14	54.25	759.5
	+	SMART Barrier Provisions	9	40.0	360.0
BALLAST	+	Portable Concrete Barrier	4868	69.25	337109.0
	+	CHIC Rail	1224	54.625	66861.0
	+	Ballast Hardware	213	47.0	10011.0
	+	Concrete Blocks	1290	55.813	71998.125
	+	1/2" Steel Plates CHIC	405	59.25	23996.25
	+	1/2" Steel Plates Concrete Blocks	405	61.625	24958.125
	+	Cargo Straps	30	69.25	2077.5
	+	"Husker Power" Plates	90	63.375	5703.75
		Note: (+) is added equipment to vehicle, (-) is removed equipment from vehicle			1207771.35

Estimated Total Weight (lb.)	22096	Total Ballast Weight (lb.)	8525
Vertical CG Location (in.)	54.66	Ballast Vertical CG Location (in.)	63.662

Vehicle Dimensions for C.G. Calculations

Wheel Base: 216.625 in. Front Track Width: 82.625 in.
Rear Track Width: 72.5 in.

Center of Gravity	10000S MASH Targets	Test Inertial	Difference
Test Inertial Weight (lb.)	22046 ± 660	22124	78.0
Longitudinal CG (in.)	NA	137.824	NA
Lateral CG (in.)	NA	0.989	NA
Vertical CG (in.)	NA	54.66	NA
Ballast Vertical CG (in.)	63 ± 2	63.662	0.66155

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

Note: Lateral CG measured from centerline - positive to vehicle right (passenger) side

CURB WEIGHT (lb.)

	Left	Right
Front	3246	3284
Rear	3691	3504

FRONT	6530	lb.
REAR	7195	lb.
TOTAL	13725	lb.

TEST INERTIAL WEIGHT (lb.)

	Left	Right
Front	3874	4174
Rear	6906	7170

FRONT	8048	lb.
REAR	14076	lb.
TOTAL	22124	lb.

Figure E-1. Vehicle Mass Distribution, Test No. STBR-1

Date:	2/14/2019	Test Name:	STBR-2	VIN:	1D7RB1GK0BS554408
Year:	2011	Make:	Dodge	Model:	Ram 1500

Vehicle CG Determination

VEHICLE	Equipment	Weight (lb.)	Vertical CG (in.)	Vertical M (lb.-in.)
+	Unballasted Truck (Curb)	4938	28.093978	138728.06
+	Hub	19	15.25	289.75
+	Brake activation cylinder & frame	8	26 1/4	210
+	Pneumatic tank (Nitrogen)	31	25 3/4	798.25
+	Strobe/Brake Battery	5	23 7/8	119.375
+	Brake Receiver/Wires	6	50 1/4	301.5
+	CG Plate including DAS	30	28 1/2	855
-	Battery	-40	38 1/4	-1530
-	Oil	-5	18 1/4	-91.25
-	Interior	-78	29	-2262
-	Fuel	-154	14 1/4	-2194.5
-	Coolant	-11	29	-319
-	Washer fluid	0		0
+	Water Ballast (In Fuel Tank)	222	20 1/4	4495.5
+	Onboard Supplemental Battery	14	24	336
+	SMART Barrier Provisions	9	22 3/4	204 3/4
				0
Note: (+) is added equipment to vehicle, (-) is removed equipment from vehicle				139941.44

Estimated Total Weight (lb.)	4994
Vertical CG Location (in.)	28.0219

Vehicle Dimensions for C.G. Calculations

Wheel Base:	140.5	in.	Front Track Width:	68.25	in.
			Rear Track Width:	67.25	in.

Center of Gravity	2270P MASH Targets	Test Inertial	Difference
Test Inertial Weight (lb.)	5000 ± 110	4992	-8.0
Longitudinal CG (in.)	63 ± 4	64.874299	1.87430
Lateral CG (in.)	NA	-0.285006	NA
Vertical CG (in.)	28 or greater	28.02	0.02191

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

Note: Lateral CG measured from centerline - positive to vehicle right (passenger) side

CURB WEIGHT (lb.)

	Left	Right
Front	1375	1333
Rear	1140	1090

FRONT	2708	lb.
REAR	2230	lb.
TOTAL	4938	lb.

TEST INERTIAL WEIGHT (lb.)

	Left	Right
Front	1363	1324
Rear	1154	1151

FRONT	2687	lb.
REAR	2305	lb.
TOTAL	4992	lb.

Figure E-2. Vehicle Mass Distribution, Test No. STBR-2

Date: 2/28/2019 **Test Name:** STBR-3 **VIN:** KNADE223996504334
Year: 2009 **Make:** Kia **Model:** Rio

Vehicle CG Determination

	Vehicle Equipment	Weight (lb.)
+	Unballasted Car (Curb)	2456
+	Hub	19
+	Brake activation cylinder & frame	8
+	Pneumatic tank (Nitrogen)	30
+	Strobe/Brake Battery	5
+	Brake Receiver/Wires	6
+	CG Plate including DAS	13
-	Battery	-37
-	Oil	-13
-	Interior	-58
-	Fuel	-20
-	Coolant	-6
-	Washer fluid	0
+	Water Ballast (In Fuel Tank)	0
+	Onboard Supplemental Battery	0
+	SMART Barrier Provisions	9

Note: (+) is added equipment to vehicle, (-) is removed equipment from vehicle

Estimated Total Weight (lb.) 2412

Vehicle Dimensions for C.G. Calculations

Wheel Base: 98.5 in. Front Track Width: 58.0 in.
Roof Height: 57.0 in. Rear Track Width: 57.375 in.

Center of Gravity	1100C MASH Targets	Test Inertial	Difference
Test Inertial Weight (lb.)	2420 ± 55	2408	-12.0
Longitudinal CG (in.)	39 ± 4	36.038	-2.962
Lateral CG (in.)	NA	-0.048	NA
Vertical CG (in.)	NA	22.271	NA

Note: Long. CG is measured from front axle of test vehicle
Note: Lateral CG measured from centerline - positive to vehicle right (passenger) side

CURB WEIGHT (lb.)

	Left	Right
Front	809	765
Rear	438	444

FRONT	1574	lb.
REAR	882	lb.
TOTAL	2456	lb.

TEST INERTIAL WEIGHT (lb.)

	Left	Right
Front	777	750
Rear	429	452

FRONT	1527	lb.
REAR	881	lb.
TOTAL	2408	lb.

Figure E-3. Vehicle Mass Distribution, Test No. STBR-3

Date:	5/22/2019	Test Name:	STBR-4	VIN:	1FVACXCS37HX61820
Year:	2007	Make:	Freightliner	Model:	M2 106

Vehicle CG Determination

		Vehicle Equipment	Weight (lb)	Vertical CG (in.)	Vertical M (lb-in.)
	+	Unballasted Truck (Curb)	13884	45.202	##########
	+	Hub	34	19.375	658.75
	+	Brake activation cylinder & frame	8	45.75	366.0
	+	Pneumatic tank (Nitrogen)	31	46.0	1426.0
	+	Strobe/Brake Battery	5	50.25	251.25
	+	Tow Pin Plate	9	12.5	112.5
	+	Brake Receiver/Wires	6	101.375	608.25
	+	Cab DAQ Unit & Mouting Plate	11	48.0	528.0
	+	CG DAQ Units & Enclosure	24	38.75	930.0
	+	Rear Axle DAS Unit and Enclosure	30	38.125	1143.75
	-	Battery	-112	36.0	-4032.0
	-	Oil	-48	23.75	-1140.0
	-	Interior	-120	55.0	-6600.0
	-	Fuel	-178	22.5	-4005.0
	-	Coolant	-58	43.5	-2523.0
	-	Washer fluid	0	0	0
	+	SMART Barrier Provisions	9	43.5	391.5
	+	Onboard Supplemental Battery	13	52.25	679.25
	-	Front Bumper Signs	-5	23.0	-115.0
BALLAST	+	Portable Concrete Barrier	4979	69.25	344795.75
	+	CHIC Rail	1224	54.625	66861.0
	+	Ballast Hardware	141	47.0	6627.0
	+	Concrete Blocks	1290	55.813	71998.125
	+	1/2" Steel Plates CHIC	405	59.25	23996.25
	+	1/2" Steel Plates Concrete Blocks	405	61.625	24958.125
	+	Cargo Straps	30	69.25	2077.5
	+	"Husker Power" Plates	180	61.25	11025.0
		Note: (+) is added equipment to vehicle, (-) is removed equipment from vehicle			1168599.4

Estimated Total Weight (lb)	22197	Total Ballast Weight (lb)	8654
Vertical CG Location (in.)	52.647	Ballast Vertical CG Location (in.)	63.825

Vehicle Dimensions for C.G. Calculations

Wheel Base: 216.25 in. Front Track Width: 82.625 in.
Rear Track Width: 72.625 in.

Center of Gravity	10000S MASH Targets	Test Inertial	Difference
Test Inertial Weight (lb)	22046 ± 660	22152	106.0
Longitudinal CG (in.)	NA	136.455	NA
Lateral CG (in.)	NA	0.918	NA
Vertical CG (in.)	NA	52.647	NA
Ballast Vertical CG (in.)	63 ± 2	63.825	0.82468

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

Note: Lateral CG measured from centerline - positive to vehicle right (passenger) side

CURB WEIGHT (lb)

	Left	Right
Front	3378	3308
Rear	3617	3581

FRONT	6686	lb
REAR	7198	lb
TOTAL	13884	lb

TEST INERTIAL WEIGHT (lb)

	Left	Right
Front	4130	4044
Rear	6684	7294

FRONT	8174	lb
REAR	13978	lb
TOTAL	22152	lb

Figure E-4. Vehicle Mass Distribution, Test No. STBR-4

Appendix F. Vehicle Deformation Records

Date: 2/8/2019 Test Name: STBR-1 VIN: 1FVACXCS57HX61818
Year: 2007 Make: Freightliner Model: M2 106

VEHICLE DEFORMATION
FLOOR PAN - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX^A (in.)	ΔY^A (in.)	ΔZ^A (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
TOE PAN - WHEEL WELL (X, Z)	1	43.3536	-9.2738	2.0549	43.3083	-9.2061	2.1282	0.0453	0.0677	-0.0733	0.1096	0.0453	X
	2	43.6220	-11.5493	1.9956	43.5536	-11.6183	2.1136	0.0684	-0.0690	-0.1180	0.1529	0.0684	X
	3	43.1699	-15.8826	1.9991	42.8147	-15.7788	1.6019	0.3552	0.1038	0.3972	0.5429	0.5329	X, Z
	4	43.7999	-19.9526	1.9046	43.0438	-19.5362	-0.1146	0.7561	0.4164	2.0192	2.1960	2.1561	X, Z
	5	43.9158	-22.7386	1.3217	43.0572	-22.0983	-0.3218	0.8586	0.6403	1.6435	1.9617	1.8543	X, Z
	6	39.6968	-9.3122	2.5415	39.6397	-9.3956	2.6456	0.0571	-0.0834	-0.1041	0.1451	0.0571	X
	7	39.7687	-12.3538	2.4792	39.6784	-12.4509	2.6114	0.0903	-0.0971	-0.1322	0.1872	0.0903	X
	8	39.5972	-15.7509	2.4188	39.4860	-15.7955	2.5306	0.1112	-0.0446	-0.1118	0.1639	0.1112	X
	9	39.1205	-19.7335	2.5114	38.5341	-19.3149	1.4384	0.5864	0.4186	1.0730	1.2924	1.2228	X, Z
	10	39.1619	-22.9299	2.6008	38.0802	-22.1894	0.3346	1.0817	0.7405	2.2662	2.6180	2.5111	X, Z
FLOOR PAN (Z)	11	34.0818	-7.3682	3.1838	34.0155	-7.3578	3.2825	0.0663	0.0104	-0.0987	0.1194	-0.0987	Z
	12	34.0356	-10.5948	3.2162	33.9389	-10.5915	3.3202	0.0967	0.0033	-0.1040	0.1420	-0.1040	Z
	13	33.9208	-14.1990	3.1840	33.8489	-14.2294	3.3056	0.0719	-0.0304	-0.1216	0.1445	-0.1216	Z
	14	33.5695	-18.0445	3.0995	33.4687	-18.0980	3.2822	0.1008	-0.0535	-0.1827	0.2154	-0.1827	Z
	15	33.5962	-22.6923	3.3538	32.9991	-22.3725	2.5426	0.5971	0.3198	0.8112	1.0568	0.8112	Z
	16	28.8979	-7.1442	3.7504	28.7970	-7.1118	3.8149	0.1009	0.0324	-0.0645	0.1241	-0.0645	Z
	17	28.9058	-10.4852	3.7846	28.8576	-10.5482	3.8557	0.0482	-0.0630	-0.0711	0.1065	-0.0711	Z
	18	28.7619	-14.2557	3.8265	28.5770	-14.2703	3.9343	0.1849	-0.0146	-0.1078	0.2145	-0.1078	Z
	19	28.4637	-18.4629	3.7269	28.3777	-18.6437	3.8856	0.0860	-0.1808	-0.1587	0.2555	-0.1587	Z
	20	28.4679	-23.8504	4.1562	28.3994	-23.8238	4.5127	0.0685	0.0266	-0.3565	0.3640	-0.3565	Z
	21	23.3087	-7.5929	3.5795	23.2433	-7.5741	3.5908	0.0654	0.0188	-0.0113	0.0690	-0.0113	Z
	22	23.5388	-10.7415	4.3703	23.4449	-10.7519	4.4271	0.0939	-0.0104	-0.0568	0.1102	-0.0568	Z
	23	23.4413	-14.0804	4.4042	23.3097	-14.1080	4.4610	0.1316	-0.0276	-0.0568	0.1460	-0.0568	Z
	24	23.3386	-18.2271	4.2892	23.2257	-18.2390	4.3896	0.1129	-0.0119	-0.1004	0.1516	-0.1004	Z
	25	23.6138	-25.3804	4.9132	23.4841	-25.4500	4.9602	0.1297	-0.0696	-0.0470	0.1545	-0.0470	Z
	26	18.0190	-8.4851	3.6353	17.9280	-8.4943	3.5991	0.0910	-0.0092	0.0362	0.0984	0.0362	Z
	27	18.5181	-13.2258	4.6854	18.4257	-13.2379	4.7282	0.0924	-0.0121	-0.0428	0.1025	-0.0428	Z
	28	18.6261	-16.6437	4.7014	18.5966	-16.6320	4.7712	0.0295	0.0117	-0.0698	0.0767	-0.0698	Z
	29	19.3119	-22.1733	4.8085	19.1644	-22.2089	4.9238	0.1475	-0.0356	-0.1153	0.1906	-0.1153	Z
	30	19.4232	-26.2596	5.0935	19.2594	-26.2965	5.2858	0.1638	-0.0369	-0.1923	0.2553	-0.1923	Z

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Pretest Floor Pan

Posttest Floor Pan

Figure F-1. Floorpan Deformation Data – Set 1, Test No. STBR-1

Date: 2/8/2019 Test Name: STBR-1 VIN: 1FVACXCS57HX61818
Year: 2007 Make: Freightliner Model: M2 106

VEHICLE DEFORMATION
FLOOR PAN - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
TOE PAN - WHEEL WELL (X, Z)	1	42.0199	-35.8622	1.6457	41.9416	-35.6805	1.5695	0.0783	0.1817	0.0762	0.2120	0.1093	X, Z
	2	42.2090	-38.1452	1.5738	42.1014	-38.0996	1.5361	0.1076	0.0456	0.0377	0.1228	0.1140	X, Z
	3	41.6054	-42.4599	1.5453	41.2188	-42.2269	0.9849	0.3866	0.2330	0.5604	0.7196	0.6808	X, Z
	4	42.0933	-46.5489	1.4294	41.3254	-45.9759	-0.7617	0.7679	0.5730	2.1911	2.3914	2.3218	X, Z
	5	42.1165	-49.3335	0.8293	41.2493	-48.5351	-0.9904	0.8672	0.7984	1.8197	2.1681	2.0158	X, Z
	6	38.3601	-35.7753	2.1023	38.2654	-35.7439	2.0633	0.0947	0.0314	0.0390	0.1071	0.1024	X, Z
	7	38.3259	-38.8171	2.0207	38.1961	-38.7982	2.0036	0.1298	0.0189	0.0171	0.1323	0.1309	X, Z
	8	38.0362	-42.2057	1.9367	37.8858	-42.1331	1.8934	0.1504	0.0726	0.0433	0.1725	0.1565	X, Z
	9	37.4195	-46.1696	1.9994	36.8167	-45.6075	0.7660	0.6028	0.5621	1.2334	1.4834	1.3728	X, Z
	10	37.3483	-49.3660	2.0683	36.2682	-48.4549	-0.3647	1.0801	0.9111	2.4330	2.8136	2.6620	X, Z
FLOOR PAN (Z)	11	32.8114	-33.6397	2.7115	32.7131	-33.5130	2.6836	0.0983	0.1267	0.0279	0.1628	0.0279	Z
	12	32.6520	-36.8628	2.7224	32.5217	-36.7422	2.6936	0.1303	0.1206	0.0288	0.1799	0.0288	Z
	13	32.4114	-40.4605	2.6657	32.3029	-40.3744	2.6479	0.1085	0.0861	0.0178	0.1397	0.0178	Z
	14	31.9263	-44.2907	2.5533	31.7861	-44.2267	2.5895	0.1402	0.0640	-0.0362	0.1583	-0.0362	Z
	15	31.7882	-48.9381	2.7775	31.1700	-48.4757	1.8112	0.6182	0.4624	0.9663	1.2368	0.9663	Z
	16	27.6339	-33.2376	3.2372	27.5033	-33.0864	3.1868	0.1306	0.1512	0.0504	0.2061	0.0504	Z
	17	27.5247	-36.5770	3.2496	27.4418	-36.5230	3.1990	0.0829	0.0540	0.0506	0.1111	0.0506	Z
	18	27.2485	-40.3403	3.2658	27.0291	-40.2333	3.2446	0.2194	0.1070	0.0212	0.2450	0.0212	Z
	19	26.8040	-44.5337	3.1363	26.6752	-44.5964	3.1578	0.1288	-0.0627	-0.0215	0.1449	-0.0215	Z
	20	26.6159	-49.9207	3.5305	26.5094	-49.7789	3.7414	0.1065	0.1418	-0.2109	0.2756	-0.2109	Z
	21	22.0341	-33.4889	3.0178	21.9382	-33.3495	2.9254	0.0959	0.1394	0.0924	0.1928	0.0924	Z
	22	22.1472	-36.6485	3.7900	22.0218	-36.5392	3.7362	0.1254	0.1093	0.0538	0.1748	0.0538	Z
	23	21.9326	-39.9822	3.8012	21.7676	-39.8886	3.7410	0.1650	0.0936	0.0602	0.1990	0.0602	Z
	24	21.6858	-44.1220	3.6583	21.5377	-44.0132	3.6344	0.1481	0.1088	0.0239	0.1853	0.0239	Z
	25	21.7051	-51.2842	4.2379	21.5369	-51.2333	4.1458	0.1682	0.0509	0.0921	0.1984	0.0921	Z
	26	16.7162	-34.1955	3.0247	16.5937	-34.0805	2.8942	0.1225	0.1150	0.1305	0.2127	0.1305	Z
	27	17.0401	-38.9572	4.0479	16.9160	-38.8478	3.9862	0.1241	0.1094	0.0617	0.1766	0.0617	Z
	28	17.0283	-42.3768	4.0425	16.9662	-42.2461	4.0017	0.0621	0.1307	0.0408	0.1503	0.0408	Z
	29	17.5192	-47.9277	4.1190	17.3351	-47.8408	4.1108	0.1841	0.0869	0.0082	0.2037	0.0082	Z
	30	17.4849	-52.0170	4.3783	17.2829	-51.9320	4.4389	0.2020	0.0850	-0.0606	0.2274	-0.0606	Z

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Pretest Floor Pan	Posttest Floor Pan
	

Figure F-2. Floorpan Deformation Data, Set 2, Test No. STBR-1

Date: 2/8/2019 Test Name: STBR-1 VIN: 1FVACXCS57HX61818
Year: 2007 Make: Freightliner Model: M2 106

VEHICLE DEFORMATION
INTERIOR CRUSH - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	38.9470	-20.7761	-28.8320	38.8049	-20.7457	-28.8811	0.1421	0.0304	-0.0491	0.1534	0.1534	X, Y, Z
	2	38.6565	-9.0076	-29.0558	38.5091	-8.9847	-29.0946	0.1474	0.0229	-0.0388	0.1541	0.1541	X, Y, Z
	3	38.0568	6.7374	-27.1932	37.9655	6.7569	-27.2082	0.0913	-0.0195	-0.0150	0.0946	0.0946	X, Y, Z
	4	33.4074	-22.1084	-14.7456	33.3671	-22.0543	-14.8659	0.0403	0.0541	-0.1203	0.1379	0.1379	X, Y, Z
	5	33.4701	-11.4549	-13.7622	33.4497	-11.4817	-13.8489	0.0204	-0.0268	-0.0867	0.0930	0.0930	X, Y, Z
	6	29.7940	5.8404	-12.8512	29.7218	5.7943	-12.8272	0.0722	0.0461	0.0240	0.0890	0.0890	X, Y, Z
SIDE PANEL (Y)	7	40.7874	-28.4569	-8.2126	40.7254	-29.1171	-8.4124	0.0620	-0.6602	-0.1998	0.6926	-0.6602	Y
	8	41.0684	-28.2177	-2.4562	40.7592	-28.6071	-2.9139	0.3092	-0.3894	-0.4577	0.6758	-0.3894	Y
	9	44.2637	-28.2167	-3.1511	43.9830	-28.5155	-3.4157	0.2807	-0.2988	-0.2646	0.4879	-0.2988	Y
IMPACT SIDE DOOR (Y)	10	8.2599	-31.0949	-21.8692	8.0852	-31.6604	-22.3055	0.1747	-0.5655	-0.4363	0.7353	-0.5655	Y
	11	19.1455	-31.7766	-20.6204	18.9758	-32.2753	-20.9829	0.1697	-0.4987	-0.3625	0.6395	-0.4987	Y
	12	29.8158	-31.1749	-19.0778	29.6777	-31.6054	-19.3738	0.1381	-0.4305	-0.2960	0.5404	-0.4305	Y
	13	17.5155	-32.4732	-4.3365	17.2690	-33.0291	-4.6778	0.2465	-0.5559	-0.3413	0.6973	-0.5559	Y
	14	27.1847	-32.1644	-2.8608	26.9682	-32.5945	-3.1129	0.2165	-0.4301	-0.2521	0.5435	-0.4301	Y
	15	22.0761	-31.7526	1.1819	21.8253	-32.2862	0.8269	0.2508	-0.5336	-0.3550	0.6882	-0.5336	Y
ROOF - (Z)	16	36.0144	-17.3678	-49.3514	36.1591	-17.3936	-49.6270	-0.1447	-0.0258	-0.2756	0.3123	-0.2756	Z
	17	36.8067	-11.7575	-49.3281	36.9722	-11.7371	-49.5512	-0.1655	0.0204	-0.2231	0.2785	-0.2231	Z
	18	37.2786	-7.5332	-49.1321	37.3913	-7.5444	-49.3991	-0.1127	-0.0112	-0.2670	0.2900	-0.2670	Z
	19	37.3527	-3.2567	-49.2128	37.4311	-3.2725	-49.4835	-0.0784	-0.0158	-0.2707	0.2823	-0.2707	Z
	20	37.4912	2.8750	-48.8145	37.5948	2.8253	-49.0147	-0.1036	0.0497	-0.2002	0.2308	-0.2002	Z
	21	12.3319	-22.0138	-53.3117	12.4663	-22.0097	-53.6342	-0.1344	0.0041	-0.3225	0.3494	-0.3225	Z
	22	12.9184	-16.0915	-53.6692	12.9339	-16.1311	-53.9549	-0.0155	-0.0396	-0.2857	0.2888	-0.2857	Z
	23	11.6852	-8.6519	-53.0979	11.6423	-8.6809	-53.3791	0.0429	-0.0290	-0.2812	0.2859	-0.2812	Z
	24	11.7511	-2.8393	-53.6884	11.8586	-2.8234	-53.9980	-0.1075	0.0159	-0.3096	0.3281	-0.3096	Z
	25	10.8386	6.2525	-53.5828	10.8969	6.2068	-53.8721	-0.0583	0.0457	-0.2893	0.2986	-0.2893	Z
	26	1.2605	-22.2411	-53.3094	1.3916	-22.3390	-53.6080	-0.1311	-0.0979	-0.2986	0.3405	-0.2986	Z
	27	3.6514	-15.4495	-53.4646	3.7959	-15.5089	-53.7424	-0.1445	-0.0594	-0.2778	0.3187	-0.2778	Z
	28	4.0706	-8.8898	-53.0734	4.2819	-8.9961	-53.3402	-0.2113	-0.1063	-0.2668	0.3566	-0.2668	Z
	29	5.3506	-1.4041	-53.6457	5.4146	-1.4338	-53.9255	-0.0640	-0.0297	-0.2798	0.2886	-0.2798	Z
	30	4.0880	6.2710	-53.5041	4.0786	6.2484	-53.7689	0.0094	0.0226	-0.2648	0.2659	-0.2648	Z
A-PILLAR Maximum (X, Y, Z)	31	40.8073	-29.2079	-27.9594	40.8724	-29.2229	-28.1364	-0.0651	-0.0150	-0.1770	0.1892	0.0000	NA
	32	39.2790	-28.9945	-31.6032	39.3740	-29.0096	-31.8531	-0.0950	-0.0151	-0.2499	0.2678	0.0000	NA
	33	37.3456	-28.5153	-35.9362	37.4725	-28.5023	-36.1816	-0.1269	0.0130	-0.2454	0.2766	0.0130	Y
	34	35.9863	-28.1470	-38.8802	36.0921	-28.1063	-39.1236	-0.1058	0.0407	-0.2434	0.2685	0.0407	Y
	35	34.0916	-27.4361	-42.6043	34.2642	-27.4596	-42.9136	-0.1726	-0.0235	-0.3093	0.3550	0.0000	NA
	36	32.8925	-26.5246	-45.9885	33.0512	-26.5325	-46.2631	-0.1587	-0.0079	-0.2746	0.3173	0.0000	NA
A-PILLAR Lateral (Y)	31	40.8073	-29.2079	-27.9594	40.8724	-29.2229	-28.1364	-0.0651	-0.0150	-0.1770	0.1892	-0.0150	Y
	32	39.2790	-28.9945	-31.6032	39.3740	-29.0096	-31.8531	-0.0950	-0.0151	-0.2499	0.2678	-0.0151	Y
	33	37.3456	-28.5153	-35.9362	37.4725	-28.5023	-36.1816	-0.1269	0.0130	-0.2454	0.2766	0.0130	Y
	34	35.9863	-28.1470	-38.8802	36.0921	-28.1063	-39.1236	-0.1058	0.0407	-0.2434	0.2685	0.0407	Y
	35	34.0916	-27.4361	-42.6043	34.2642	-27.4596	-42.9136	-0.1726	-0.0235	-0.3093	0.3550	-0.0235	Y
	36	32.8925	-26.5246	-45.9885	33.0512	-26.5325	-46.2631	-0.1587	-0.0079	-0.2746	0.3173	-0.0079	Y
B-PILLAR Maximum (X, Y, Z)	37	-0.0873	-28.9137	-43.4758	0.1350	-29.0521	-43.6457	-0.2223	-0.1384	-0.1699	0.3122	0.0000	NA
	38	-5.3319	-30.1490	-35.5725	-5.1567	-30.3362	-35.7792	-0.1752	-0.1872	-0.2067	0.3293	0.0000	NA
	39	0.4457	-30.9142	-23.8084	0.7302	-31.0746	-23.9646	-0.2845	-0.1604	-0.1562	0.3620	0.0000	NA
	40	-5.8378	-31.7360	-14.4573	-5.6316	-31.9509	-14.5989	-0.2062	-0.2149	-0.1416	0.3298	0.0000	NA
B-PILLAR Lateral (Y)	37	-0.0873	-28.9137	-43.4758	0.1350	-29.0521	-43.6457	-0.2223	-0.1384	-0.1699	0.3122	-0.1384	Y
	38	-5.3319	-30.1490	-35.5725	-5.1567	-30.3362	-35.7792	-0.1752	-0.1872	-0.2067	0.3293	-0.1872	Y
	39	0.4457	-30.9142	-23.8084	0.7302	-31.0746	-23.9646	-0.2845	-0.1604	-0.1562	0.3620	-0.1604	Y
	40	-5.8378	-31.7360	-14.4573	-5.6316	-31.9509	-14.5989	-0.2062	-0.2149	-0.1416	0.3298	-0.2149	Y

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-3. Occupant Compartment Deformation Data - Set 1, Test No. STBR-1

Date: 2/8/2019 Test Name: STBR-1 VIN: 1FVACXCS57HX61818
Year: 2007 Make: Freightliner Model: M2 106

VEHICLE DEFORMATION
INTERIOR CRUSH - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	37.3541	-46.9771	-29.5081	37.2378	-46.8250	-29.5054	0.1163	0.1521	0.0027	0.1915	0.1915	X, Y, Z
	2	37.5004	-35.2043	-29.6229	37.3614	-35.0597	-29.6322	0.1390	0.1446	-0.0093	0.2008	0.2008	X, Y, Z
	3	37.4691	-19.4656	-27.6161	37.3646	-19.3228	-27.6309	0.1045	0.1428	-0.0148	0.1776	0.1776	X, Y, Z
	4	31.6643	-48.2324	-15.4746	31.6632	-48.0415	-15.5357	0.0011	0.1909	-0.0611	0.2004	0.2004	X, Y, Z
	5	32.1133	-37.5979	-14.3903	32.1144	-37.4863	-14.4386	-0.0011	0.1116	-0.0483	0.1216	0.1216	X, Y, Z
	6	29.0721	-20.1875	-13.3424	28.9957	-20.0965	-13.3109	0.0764	0.0910	0.0315	0.1229	0.1229	X, Y, Z
SIDE PANEL (Y)	7	38.7558	-54.9093	-8.9494	38.7225	-55.4085	-9.0883	0.0333	-0.4992	-0.1389	0.5192	-0.4992	Y
	8	39.0025	-54.7334	-3.1892	38.7375	-54.9401	-3.5861	0.2650	-0.2067	-0.3969	0.5201	-0.2067	Y
	9	42.2008	-54.8444	-3.8613	41.9659	-54.9596	-4.0664	0.2349	-0.1152	-0.2051	0.3324	-0.1152	Y
IMPACT SIDE DOOR (Y)	10	6.2556	-56.2160	-22.8611	6.1065	-56.6878	-23.2100	0.1491	-0.4718	-0.3489	0.6054	-0.4718	Y
	11	17.0990	-57.3117	-21.5415	16.9592	-57.6993	-21.8220	0.1398	-0.3876	-0.2805	0.4985	-0.3876	Y
	12	27.7725	-57.1196	-19.9175	27.6671	-57.4223	-20.1390	0.1054	-0.3027	-0.2215	0.3896	-0.3027	Y
	13	15.3231	-58.0967	-5.2769	15.1175	-58.5109	-5.5344	0.2056	-0.4142	-0.2575	0.5293	-0.4142	Y
	14	24.9859	-58.1596	-3.7296	24.8153	-58.4330	-3.9039	0.1706	-0.2734	-0.1743	0.3664	-0.2734	Y
	15	19.8660	-57.5960	0.2804	19.6602	-57.9707	0.0050	0.2058	-0.3747	-0.2754	0.5085	-0.3747	Y
ROOF - (Z)	16	34.7024	-43.2746	-50.0147	34.8518	-43.2297	-50.2420	-0.1494	0.0449	-0.2273	0.2757	-0.2273	Z
	17	35.7013	-37.6979	-49.9329	35.8648	-37.6064	-50.1184	-0.1635	0.0915	-0.1855	0.2637	-0.1855	Z
	18	36.3275	-33.4960	-49.6937	36.4315	-33.4325	-49.9320	-0.1040	0.0635	-0.2383	0.2676	-0.2383	Z
	19	36.5602	-29.2246	-49.7335	36.6236	-29.1642	-49.9841	-0.0634	0.0604	-0.2506	0.2655	-0.2506	Z
	20	36.9221	-23.1061	-49.2764	37.0006	-23.0796	-49.4683	-0.0785	0.0265	-0.1919	0.2090	-0.1919	Z
	21	10.8946	-47.0042	-54.1868	11.0374	-46.9708	-54.4364	-0.1428	0.0334	-0.2496	0.2895	-0.2496	Z
	22	11.7022	-41.1047	-54.4843	11.7156	-41.1104	-54.7097	-0.0134	-0.0057	-0.2254	0.2259	-0.2254	Z
	23	10.7405	-33.6300	-53.8516	10.6857	-33.6233	-54.0862	0.0548	0.0067	-0.2346	0.2410	-0.2346	Z
	24	11.0255	-27.8187	-54.3868	11.1139	-27.7728	-54.6595	-0.0884	0.0459	-0.2727	0.2903	-0.2727	Z
	25	10.4488	-18.7007	-54.2019	10.4728	-18.7154	-54.4719	-0.0240	-0.0147	-0.2700	0.2715	-0.2700	Z
	26	-0.1774	-46.8215	-54.2653	-0.0420	-46.9061	-54.4839	-0.1354	-0.0846	-0.2186	0.2707	-0.2186	Z
	27	2.4639	-40.1219	-54.3394	2.6043	-40.1651	-54.5514	-0.1404	-0.0432	-0.2120	0.2579	-0.2120	Z
	28	3.1223	-33.5861	-53.8835	3.3186	-33.6768	-54.0970	-0.1963	-0.0907	-0.2135	0.3039	-0.2135	Z
	29	4.6823	-26.1479	-54.3760	4.7231	-26.1555	-54.6181	-0.0408	-0.0076	-0.2421	0.2456	-0.2421	Z
	30	3.7031	-18.4330	-54.1710	3.6597	-18.4320	-54.4123	0.0434	0.0010	-0.2413	0.2452	-0.2413	Z
A-PILLAR Maximum (X, Y, Z)	31	38.8950	-55.4796	-28.7018	38.9978	-55.3756	-28.8112	-0.1028	0.1040	-0.1094	0.1826	0.1040	Y
	32	37.4028	-55.1763	-32.3542	37.5329	-55.0820	-32.5357	-0.1301	0.0943	-0.1815	0.2424	0.0943	Y
	33	35.5208	-54.5862	-36.6961	35.6796	-54.4758	-36.8725	-0.1588	0.1104	-0.1764	0.2618	0.1104	Y
	34	34.1980	-54.1409	-39.6461	34.3339	-54.0095	-39.8202	-0.1359	0.1314	-0.1741	0.2570	0.1314	Y
	35	32.3586	-53.3262	-43.3767	32.5556	-53.2706	-43.6170	-0.1970	0.0556	-0.2403	0.3157	0.0556	Y
	36	31.2193	-52.3400	-46.7607	31.3988	-52.2765	-46.9671	-0.1795	0.0635	-0.2064	0.2808	0.0635	Y
A-PILLAR Lateral (Y)	31	38.8950	-55.4796	-28.7018	38.9978	-55.3756	-28.8112	-0.1028	0.1040	-0.1094	0.1826	0.1040	Y
	32	37.4028	-55.1763	-32.3542	37.5329	-55.0820	-32.5357	-0.1301	0.0943	-0.1815	0.2424	0.0943	Y
	33	35.5208	-54.5862	-36.6961	35.6796	-54.4758	-36.8725	-0.1588	0.1104	-0.1764	0.2618	0.1104	Y
	34	34.1980	-54.1409	-39.6461	34.3339	-54.0095	-39.8202	-0.1359	0.1314	-0.1741	0.2570	0.1314	Y
	35	32.3586	-53.3262	-43.3767	32.5556	-53.2706	-43.6170	-0.1970	0.0556	-0.2403	0.3157	0.0556	Y
	36	31.2193	-52.3400	-46.7607	31.3988	-52.2765	-46.9671	-0.1795	0.0635	-0.2064	0.2808	0.0635	Y
B-PILLAR Maximum (X, Y, Z)	37	-1.8441	-53.5295	-44.5049	-1.6030	-53.6428	-44.5807	-0.2411	-0.1133	-0.0758	0.2770	0.0000	NA
	38	-7.1895	-54.6421	-36.6511	-6.9895	-54.7952	-36.7583	-0.2000	-0.1531	-0.1072	0.2737	0.0000	NA
	39	-1.5319	-55.7285	-24.8540	-1.2119	-55.8287	-24.9120	-0.3200	-0.1002	-0.0580	0.3403	0.0000	NA
	40	-7.9110	-56.4027	-15.5559	-7.6634	-56.5465	-15.5943	-0.2476	-0.1438	-0.0384	0.2889	0.0000	NA
B-PILLAR Lateral (Y)	37	-1.8441	-53.5295	-44.5049	-1.6030	-53.6428	-44.5807	-0.2411	-0.1133	-0.0758	0.2770	-0.1133	Y
	38	-7.1895	-54.6421	-36.6511	-6.9895	-54.7952	-36.7583	-0.2000	-0.1531	-0.1072	0.2737	-0.1531	Y
	39	-1.5319	-55.7285	-24.8540	-1.2119	-55.8287	-24.9120	-0.3200	-0.1002	-0.0580	0.3403	-0.1002	Y
	40	-7.9110	-56.4027	-15.5559	-7.6634	-56.5465	-15.5943	-0.2476	-0.1438	-0.0384	0.2889	-0.1438	Y

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-4. Occupant Compartment Deformation Data - Set 2, Test No. STBR-1

Date: 2/22/2019 Test Name: STBR-2 VIN: 1D7RB1GK0BS554408
Year: 2011 Make: Dodge Model: Ram 1500

VEHICLE DEFORMATION
DRIVER SIDE FLOOR PAN - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
TOE PAN - WHEEL WELL (X, Z)	1	55.0544	-22.3369	1.2612	54.9993	-22.0668	1.9137	0.0551	0.2701	-0.6525	0.7083	0.0551	X
	2	54.9162	-18.1737	1.3128	54.9079	-18.1367	1.7203	0.0083	0.0370	-0.4075	0.4093	0.0083	X
	3	54.8890	-14.7157	1.2988	54.7347	-14.6890	1.6140	0.1543	0.0267	-0.3152	0.3520	0.1543	X
	4	54.7870	-10.6190	1.3537	54.8625	-10.4698	1.7809	-0.0755	0.1492	-0.4272	0.4588	0.0000	NA
	5	54.7410	-6.9377	1.2229	54.7508	-6.9879	1.5801	-0.0098	-0.0502	-0.3572	0.3608	0.0000	NA
	6	51.7301	-22.8622	3.0224	51.6788	-22.7542	3.5674	0.0513	0.1080	-0.5450	0.5580	0.0513	X
	7	51.8172	-18.8283	2.9423	51.6759	-18.6699	3.1159	0.1413	0.1584	-0.1736	0.2742	0.1413	X
	8	51.8106	-15.1280	2.9377	51.6985	-15.0256	3.2293	0.1121	0.1024	-0.2916	0.3288	0.1121	X
	9	51.7884	-11.3031	2.9555	51.7318	-11.1971	3.2814	0.0566	0.1060	-0.3259	0.3473	0.0566	X
	10	51.7069	-7.2203	3.0033	51.6884	-7.1349	3.3716	0.0185	0.0854	-0.3683	0.3785	0.0185	X
FLOOR PAN (Z)	11	48.5072	-22.8477	4.5673	48.3527	-22.7642	5.0851	0.1545	0.0835	-0.5178	0.5468	-0.5178	Z
	12	48.4712	-19.2067	4.5819	48.4017	-19.0769	4.9033	0.0695	0.1298	-0.3214	0.3535	-0.3214	Z
	13	48.4855	-15.4376	4.5819	48.4149	-15.3676	4.8624	0.0706	0.0700	-0.2805	0.2976	-0.2805	Z
	14	48.4567	-11.2070	4.6048	48.4010	-11.1060	4.9036	0.0557	0.1010	-0.2988	0.3203	-0.2988	Z
	15	48.4152	-7.4159	4.6448	48.4200	-7.3075	4.9642	-0.0048	0.1084	-0.3194	0.3373	-0.3194	Z
	16	44.8484	-23.0936	4.7147	44.7598	-23.0135	5.3403	0.0886	0.0801	-0.6256	0.6369	-0.6256	Z
	17	44.8701	-19.5763	4.7214	44.7864	-19.4912	5.1374	0.0837	0.0851	-0.4160	0.4328	-0.4160	Z
	18	44.8152	-15.5433	4.7419	44.7224	-15.4539	5.0274	0.0928	0.0894	-0.2855	0.3132	-0.2855	Z
	19	44.6056	-11.1811	4.7751	44.5593	-11.1502	5.0562	0.0463	0.0309	-0.2811	0.2866	-0.2811	Z
	20	44.6415	-7.4004	4.8062	44.5816	-7.3502	5.1329	0.0599	0.0502	-0.3267	0.3359	-0.3267	Z
	21	40.5661	-23.1937	4.9708	40.4573	-23.1418	5.6727	0.1088	0.0519	-0.7019	0.7122	-0.7019	Z
	22	40.7049	-19.6589	4.9802	40.6414	-19.5692	5.5565	0.0635	0.0897	-0.5763	0.5867	-0.5763	Z
	23	40.6000	-15.4607	5.0108	40.5245	-15.3865	5.2512	0.0755	0.0742	-0.2404	0.2627	-0.2404	Z
	24	40.8002	-10.9043	5.0445	40.6924	-10.8649	5.3425	0.1078	0.0394	-0.2980	0.3193	-0.2980	Z
	25	40.7051	-7.4414	5.0658	40.6214	-7.3461	5.3512	0.0837	0.0953	-0.2854	0.3123	-0.2854	Z
	26	35.3639	-23.6090	5.0377	35.3928	-23.4501	5.5022	-0.0289	0.1589	-0.4645	0.4918	-0.4645	Z
	27	35.5924	-19.7245	5.0309	35.5392	-19.6269	5.4735	0.0532	0.0976	-0.4426	0.4563	-0.4426	Z
	28	35.7972	-15.5618	5.0581	35.7275	-15.4769	5.3638	0.0697	0.0849	-0.3057	0.3248	-0.3057	Z
	29	35.8931	-10.9586	5.0827	35.8245	-10.8943	5.2869	0.0686	0.0643	-0.2042	0.2248	-0.2042	Z
	30	36.1295	-7.6160	5.1032	36.0800	-7.5363	5.3397	0.0495	0.0797	-0.2365	0.2544	-0.2365	Z

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Pretest Floor Pan	Posttest Floor Pan
	

Figure F-5. Floorpan Deformation Data - Set 1, Test No. STBR-2

Date: 2/22/2019 Test Name: STBR-2 VIN: 1D7RB1GK0BS554408
Year: 2011 Make: Dodge Model: Ram 1500

VEHICLE DEFORMATION
DRIVER SIDE FLOOR PAN - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
TOE PAN - WHEEL WELL (X, Z)	1	56.6698	-3.2845	-2.4688	56.4969	-2.9684	-2.1897	0.1729	0.3161	-0.2791	0.4558	0.1729	X
	2	56.5922	0.8803	-2.4222	56.4598	0.9629	-2.3758	0.1324	-0.0826	-0.0464	0.1628	0.1324	X
	3	56.6158	4.3384	-2.4399	56.3343	4.4128	-2.4757	0.2815	-0.0744	0.0358	0.2934	0.2838	X, Z
	4	56.5734	8.4361	-2.3897	56.5202	8.6296	-2.3007	0.0532	-0.1935	-0.0890	0.2195	0.0532	X
	5	56.5822	12.1176	-2.5245	56.4567	12.1131	-2.4951	0.1255	0.0045	-0.0294	0.1290	0.1255	X
	6	53.3260	-3.7591	-0.7302	53.1660	-3.6130	-0.5395	0.1600	0.1461	-0.1907	0.2886	0.1600	X
	7	53.4728	0.2730	-0.8137	53.2198	0.4718	-0.9833	0.2530	-0.1988	0.1696	0.3637	0.3046	X, Z
	8	53.5204	3.9729	-0.8220	53.2926	4.1152	-0.8630	0.2278	-0.1423	0.0410	0.2717	0.2315	X, Z
	9	53.5541	7.7978	-0.8081	53.3787	7.9428	-0.8037	0.1754	-0.1450	-0.0044	0.2276	0.1754	X
	10	53.5320	11.8814	-0.7649	53.3913	12.0049	-0.7058	0.1407	-0.1235	-0.0591	0.1963	0.1407	X
FLOOR PAN (Z)	11	50.0931	-3.6958	0.7923	49.8391	-3.5800	0.9760	0.2540	0.1158	-0.1837	0.3342	-0.1837	Z
	12	50.1104	-0.0546	0.8031	49.9391	0.1067	0.8012	0.1713	0.1613	0.0019	0.2353	0.0019	Z
	13	50.1798	3.7139	0.7995	50.0035	3.8155	0.7673	0.1763	-0.1016	0.0322	0.2060	0.0322	Z
	14	50.2129	7.9444	0.8179	50.0485	8.0768	0.8165	0.1644	-0.1324	0.0014	0.2111	0.0014	Z
	15	50.2265	11.7358	0.8539	50.1198	11.8745	0.8843	0.1067	-0.1387	-0.0304	0.1776	-0.0304	Z
	16	46.4302	-3.8879	0.9145	46.2429	-3.7801	1.2284	0.1873	0.1078	-0.3139	0.3811	-0.3139	Z
	17	46.5033	-0.3713	0.9179	46.3183	-0.2581	1.0322	0.1850	0.1132	-0.1143	0.2452	-0.1143	Z
	18	46.5073	3.6621	0.9341	46.3100	3.7799	0.9297	0.1973	-0.1178	0.0044	0.2298	0.0044	Z
	19	46.3615	8.0268	0.9615	46.2063	8.0854	0.9665	0.1552	-0.0586	-0.0050	0.1660	-0.0050	Z
	20	46.4524	11.8067	0.9891	46.2811	11.8845	1.0503	0.1713	-0.0778	-0.0612	0.1978	-0.0612	Z
	21	42.1451	-3.9251	1.1410	41.9388	-3.8496	1.5577	0.2063	0.0755	-0.4167	0.4711	-0.4167	Z
	22	42.3357	-0.3927	1.1479	42.1723	-0.2796	1.4484	0.1634	0.1131	-0.3005	0.3603	-0.3005	Z
	23	42.2921	3.8066	1.1736	42.1134	3.9048	1.1509	0.1787	-0.0982	0.0227	0.2052	0.0227	Z
	24	42.5587	8.3596	1.2042	42.3436	8.4235	1.2508	0.2151	-0.0639	-0.0466	0.2292	-0.0466	Z
	25	42.5142	11.8236	1.2214	42.3212	11.9429	1.2661	0.1930	-0.1193	-0.0447	0.2313	-0.0447	Z
	26	36.9372	-4.2641	1.1723	36.8706	-4.0877	1.3833	0.0666	0.1764	-0.2110	0.2830	-0.2110	Z
	27	37.2225	-0.3834	1.1632	37.0699	-0.2667	1.3619	0.1526	0.1167	-0.1987	0.2764	-0.1987	Z
	28	37.4881	3.7759	1.1877	37.3155	3.8804	1.2601	0.1726	-0.1045	-0.0724	0.2144	-0.0724	Z
	29	37.6512	8.3773	1.2084	37.4758	8.4614	1.1919	0.1754	-0.0841	0.0165	0.1952	0.0165	Z
	30	37.9364	11.7160	1.2273	37.7775	11.8155	1.2513	0.1589	-0.0995	-0.0240	0.1890	-0.0240	Z

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Pretest Floor Pan	Posttest Floor Pan
	

Figure F-6. Floorpan Deformation Data -Set 2, Test No. STBR-2

Date: 2/22/2019 Test Name: STBR-2 VIN: 1D7RB1GK0BS554408
Year: 2011 Make: Dodge Model: Ram 1500

VEHICLE DEFORMATION
DRIVER SIDE INTERIOR CRUSH - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	41.9979	-19.8990	-28.8725	42.1811	-19.8738	-28.4722	-0.1832	0.0252	0.4003	0.4410	0.4410	X, Y, Z
	2	42.1286	-7.3072	-28.3963	42.3670	-7.2776	-28.0372	-0.2384	0.0296	0.3591	0.4320	0.4320	X, Y, Z
	3	43.4313	4.6005	-28.1365	43.7350	4.6571	-27.8294	-0.3037	-0.0566	0.3071	0.4356	0.4356	X, Y, Z
	4	40.0709	-24.5940	-16.2872	40.1808	-24.4730	-15.8765	-0.1099	0.1210	0.4107	0.4420	0.4420	X, Y, Z
	5	38.3403	-14.6881	-15.0702	38.4313	-14.5888	-14.7305	-0.0910	0.0993	0.3397	0.3654	0.3654	X, Y, Z
	6	36.7868	4.7561	-17.3328	36.9762	4.8011	-17.0348	-0.1894	-0.0450	0.2980	0.3560	0.3560	X, Y, Z
SIDE PANEL (Y)	7	48.8117	-27.6903	-5.1444	48.8485	-26.8892	-4.6674	-0.0368	0.8011	0.4770	0.9331	0.8011	Y
	8	49.0940	-27.6720	-1.9379	49.0062	-26.9844	-1.4532	0.0878	0.6876	0.4847	0.8458	0.6876	Y
	9	51.9364	-27.6545	-3.4010	51.9492	-27.1760	-2.9043	-0.0128	0.4785	0.4967	0.6898	0.4785	Y
IMPACT SIDE DOOR (Y)	10	38.3350	-30.1602	-16.4195	38.0654	-29.9413	-16.1244	0.2696	0.2189	0.2951	0.4557	0.2189	Y
	11	29.3686	-31.0200	-16.1872	29.1209	-31.2306	-15.9882	0.2477	-0.2106	0.1990	0.3812	-0.2106	Y
	12	18.5985	-30.8541	-17.0148	18.3399	-31.3648	-16.8024	0.2586	-0.5107	0.2124	0.6106	-0.5107	Y
	13	39.1458	-28.6813	-7.1231	38.5950	-28.6110	-6.7239	0.5508	0.0703	0.3992	0.6839	0.0703	Y
	14	30.0486	-31.1077	-3.7975	29.7302	-31.4201	-3.5820	0.3184	-0.3124	0.2155	0.4954	-0.3124	Y
	15	20.9808	-30.3559	-3.1799	20.6047	-30.6616	-2.9609	0.3761	-0.3057	0.2190	0.5319	-0.3057	Y
ROOF - (Z)	16	28.9741	-17.5851	-43.1686	29.3515	-17.7550	-42.8965	-0.3774	-0.1699	0.2721	0.4953	0.2721	Z
	17	30.5258	-12.7388	-43.1787	30.8611	-12.9302	-42.9014	-0.3353	-0.1914	0.2773	0.4753	0.2773	Z
	18	31.6727	-7.1691	-43.3235	31.9583	-7.3074	-43.0646	-0.2856	-0.1383	0.2589	0.4095	0.2589	Z
	19	32.4213	-0.5547	-43.3979	32.7579	-0.7099	-43.1320	-0.3366	-0.1552	0.2659	0.4562	0.2659	Z
	20	32.5345	5.6122	-43.4223	32.9660	5.4498	-43.1407	-0.4315	0.1624	0.2816	0.5402	0.2816	Z
	21	17.5224	-16.6155	-46.2694	17.8377	-16.7445	-46.0628	-0.3153	-0.1290	0.2066	0.3984	0.2066	Z
	22	18.5121	-11.4731	-46.5507	18.9962	-11.7104	-46.3354	-0.4841	-0.2373	0.2153	0.5805	0.2153	Z
	23	20.1147	-5.3057	-46.6582	20.4351	-5.4325	-46.4618	-0.3204	-0.1268	0.1964	0.3966	0.1964	Z
	24	21.8088	1.2329	-46.6772	22.1309	1.0602	-46.4750	-0.3221	0.1727	0.2022	0.4177	0.2022	Z
	25	22.6592	5.3309	-46.6763	23.0007	5.1416	-46.4790	-0.3415	0.1893	0.1973	0.4375	0.1973	Z
	26	9.6791	-14.6955	-46.7034	10.0364	-14.8398	-46.5597	-0.3573	-0.1443	0.1437	0.4113	0.1437	Z
	27	10.4639	-8.5532	-47.0068	10.8520	-8.6423	-46.8727	-0.3881	-0.0891	0.1341	0.4202	0.1341	Z
	28	11.5250	-2.4034	-47.1703	11.8230	-2.5409	-47.0364	-0.2980	-0.1375	0.1339	0.3545	0.1339	Z
	29	13.1998	2.4344	-47.1830	13.5334	2.2599	-47.0515	-0.3336	0.1745	0.1315	0.3988	0.1315	Z
	30	14.0678	6.5971	-47.1404	14.3460	6.4534	-47.0198	-0.2782	0.1437	0.1206	0.3355	0.1206	Z
A-PILLAR Maximum (X, Y, Z)	31	43.2899	-25.0194	-32.4373	43.4868	-25.0807	-32.0525	-0.1969	-0.0613	0.3848	0.4366	0.3848	Z
	32	41.0428	-24.3583	-34.0890	41.2554	-24.4428	-33.7304	-0.2126	-0.0845	0.3586	0.4254	0.3586	Z
	33	38.5356	-23.8156	-36.0122	38.5356	-23.8156	-36.0122	0.0000	0.0000	0.0000	0.0000	0.0000	X, Y, Z
	34	36.0021	-23.5529	-37.4739	36.3004	-23.6395	-37.1533	-0.2983	-0.0866	0.3206	0.4464	0.3206	Z
	35	33.6473	-23.0116	-38.8126	33.9395	-23.0839	-38.5171	-0.2922	-0.0723	0.2955	0.4218	0.2955	Z
	36	31.8583	-22.9679	-40.1443	32.1450	-23.0780	-39.9210	-0.2867	-0.1101	0.2233	0.3797	0.2233	Z
A-PILLAR Lateral (Y)	31	43.2899	-25.0194	-32.4373	43.4868	-25.0807	-32.0525	-0.1969	-0.0613	0.3848	0.4366	-0.0613	Y
	32	41.0428	-24.3583	-34.0890	41.2554	-24.4428	-33.7304	-0.2126	-0.0845	0.3586	0.4254	-0.0845	Y
	33	38.5356	-23.8156	-36.0122	38.5356	-23.8156	-36.0122	0.0000	0.0000	0.0000	0.0000	0.0000	Y
	34	36.0021	-23.5529	-37.4739	36.3004	-23.6395	-37.1533	-0.2983	-0.0866	0.3206	0.4464	-0.0866	Y
	35	33.6473	-23.0116	-38.8126	33.9395	-23.0839	-38.5171	-0.2922	-0.0723	0.2955	0.4218	-0.0723	Y
	36	31.8583	-22.9679	-40.1443	32.1450	-23.0780	-39.9210	-0.2867	-0.1101	0.2233	0.3797	-0.1101	Y
B-PILLAR Maximum (X, Y, Z)	37	7.0792	-23.1088	-40.9821	7.3551	-23.2538	-40.7897	-0.2759	-0.1450	0.1924	0.3663	0.1924	Z
	38	4.1056	-26.2531	-30.8976	4.3391	-26.3377	-30.7408	-0.2335	-0.0846	0.1568	0.2937	0.1568	Z
	39	8.9334	-27.6250	-24.2774	9.0949	-27.7110	-24.0473	-0.1615	-0.0860	0.2301	0.2940	0.2301	Z
	40	5.3784	-27.9366	-19.3637	5.5030	-27.9788	-19.2569	-0.1246	-0.0422	0.1068	0.1694	0.1068	Z
B-PILLAR Lateral (Y)	37	7.0792	-23.1088	-40.9821	7.3551	-23.2538	-40.7897	-0.2759	-0.1450	0.1924	0.3663	-0.1450	Y
	38	4.1056	-26.2531	-30.8976	4.3391	-26.3377	-30.7408	-0.2335	-0.0846	0.1568	0.2937	-0.0846	Y
	39	8.9334	-27.6250	-24.2774	9.0949	-27.7110	-24.0473	-0.1615	-0.0860	0.2301	0.2940	-0.0860	Y
	40	5.3784	-27.9366	-19.3637	5.5030	-27.9788	-19.2569	-0.1246	-0.0422	0.1068	0.1694	-0.0422	Y

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-7. Occupant Compartment Deformation Data - Set 1, Test No. STBR-2

Date: 2/22/2019 Test Name: STBR-2 VIN: 1D7RB1GK0BS554408
Year: 2011 Make: Dodge Model: Ram 1500

VEHICLE DEFORMATION
DRIVER SIDE INTERIOR CRUSH - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	43.8651	-0.6856	-32.6927	43.7219	-0.5362	-32.5975	0.1432	0.1494	0.0952	0.2278	0.2278	X, Y, Z
	2	44.1765	11.9034	-32.2269	44.0801	12.0554	-32.1369	0.0964	-0.1520	0.0900	0.2012	0.2012	X, Y, Z
	3	45.6512	23.7911	-31.9686	45.6114	23.9698	-31.9045	0.0398	-0.1787	0.0641	0.1940	0.1940	X, Y, Z
	4	41.7815	-5.3394	-20.1171	41.6544	-5.1330	-20.0118	0.1271	0.2064	0.1053	0.2643	0.2643	X, Y, Z
	5	40.1873	4.5919	-18.9211	40.0401	4.7719	-18.8463	0.1472	-0.1800	0.0748	0.2443	0.2443	X, Y, Z
	6	38.9342	24.0545	-21.2120	38.8516	24.1846	-21.1117	0.0826	-0.1301	0.1003	0.1839	0.1839	X, Y, Z
SIDE PANEL (Y)	7	50.3977	-8.5520	-8.9105	50.2845	-7.6903	-8.8049	0.1132	0.8617	0.1056	0.8755	0.8617	Y
	8	50.6577	-8.5347	-5.7020	50.4397	-7.7942	-5.5908	0.2180	0.7405	0.1112	0.7799	0.7405	Y
	9	53.5103	-8.5601	-7.1451	53.3803	-8.0232	-7.0414	0.1300	0.5369	0.1037	0.5621	0.5369	Y
IMPACT SIDE DOOR (Y)	10	39.9653	-10.8797	-20.2566	39.4644	-10.5712	-20.2714	0.5009	0.3085	-0.0148	0.5885	0.3085	Y
	11	30.9859	-11.6083	-20.0866	30.5030	-11.7382	-20.1406	0.4829	-0.1299	-0.0540	0.5030	-0.1299	Y
	12	20.2255	-11.2858	-20.9900	19.7215	-11.7230	-20.9584	0.5040	-0.4372	0.0316	0.6680	-0.4372	Y
	13	40.7324	-9.4036	-10.9562	40.0090	-9.2673	-10.8681	0.7234	0.1363	0.0881	0.7414	0.1363	Y
	14	31.5776	-11.6935	-7.6924	31.1055	-11.9611	-7.7346	0.4721	-0.2676	-0.0422	0.5443	-0.2676	Y
	15	22.5176	-10.8087	-7.1393	21.9910	-11.0789	-7.1148	0.5266	-0.2702	0.0245	0.5924	-0.2702	Y
ROOF - (Z)	16	30.9771	1.8042	-47.0821	30.9273	1.7874	-47.0214	0.0498	0.0168	0.0607	0.0803	0.0607	Z
	17	32.5996	6.6272	-47.0856	32.5029	6.5911	-47.0161	0.0967	0.0361	0.0695	0.1244	0.0695	Z
	18	33.8287	12.1795	-47.2273	33.6771	12.1987	-47.1675	0.1516	-0.0192	0.0598	0.1641	0.0598	Z
	19	34.6745	18.7821	-47.3024	34.5669	18.7847	-47.2213	0.1076	-0.0026	0.0811	0.1348	0.0811	Z
	20	34.8779	24.9467	-47.3314	34.8595	24.9410	-47.2175	0.0184	0.0057	0.1139	0.1155	0.1139	Z
	21	19.5628	2.9379	-50.2642	19.4295	2.9619	-50.1893	0.1333	-0.0240	0.0749	0.1548	0.0749	Z
	22	20.6296	8.0650	-50.5432	20.6569	7.9803	-50.4513	-0.0273	0.0847	0.0919	0.1279	0.0919	Z
	23	22.3229	14.2082	-50.6449	22.1818	14.2381	-50.5645	0.1411	-0.0299	0.0804	0.1651	0.0804	Z
	24	24.1125	20.7213	-50.6578	23.9663	20.7070	-50.5640	0.1462	0.0143	0.0938	0.1743	0.0938	Z
	25	25.0226	24.8065	-50.6546	24.8920	24.7760	-50.5595	0.1306	0.0305	0.0951	0.1644	0.0951	Z
	26	11.7517	4.9719	-50.7550	11.6552	4.9744	-50.6847	0.0965	-0.0025	0.0703	0.1194	0.0703	Z
	27	12.6283	11.1018	-51.0584	12.5557	11.1607	-50.9849	0.0726	-0.0589	0.0735	0.1189	0.0735	Z
	28	13.7803	17.2352	-51.2199	13.6103	17.2486	-51.1359	0.1700	-0.0134	0.0840	0.1901	0.0840	Z
	29	15.5257	22.0480	-51.2251	15.3863	22.0255	-51.1407	0.1394	0.0225	0.0844	0.1645	0.0844	Z
	30	16.4541	26.1976	-51.1801	16.2562	26.2074	-51.1003	0.1979	-0.0098	0.0798	0.2136	0.0798	Z
A-PILLAR Maximum (X, Y, Z)	31	45.1071	-5.8278	-36.2439	44.9573	-5.7532	-36.1879	0.1498	0.0746	0.0560	0.1765	0.1765	X, Y, Z
	32	42.8816	-5.1356	-37.9119	42.7354	-5.0814	-37.8652	0.1462	0.0542	0.0467	0.1628	0.1628	X, Y, Z
	33	40.3961	-4.5583	-39.8532	40.3961	-4.5583	-39.8532	0.0000	0.0000	0.0000	0.0000	0.0000	X, Y, Z
	34	37.8770	-4.2601	-41.3329	37.7930	-4.2033	-41.2880	0.0840	0.0568	0.0449	0.1109	0.1109	X, Y, Z
	35	35.5398	-3.6857	-42.6885	35.4404	-3.6127	-42.6514	0.0994	0.0730	0.0371	0.1288	0.1288	X, Y, Z
	36	33.7610	-3.6172	-44.0328	33.6466	-3.5794	-44.0559	0.1144	0.0378	-0.0231	0.1227	0.1205	X, Y
A-PILLAR Lateral (Y)	31	45.1071	-5.8278	-36.2439	44.9573	-5.7532	-36.1879	0.1498	0.0746	0.0560	0.1765	0.0746	Y
	32	42.8816	-5.1356	-37.9119	42.7354	-5.0814	-37.8652	0.1462	0.0542	0.0467	0.1628	0.0542	Y
	33	40.3961	-4.5583	-39.8532	40.3961	-4.5583	-39.8532	0.0000	0.0000	0.0000	0.0000	0.0000	Y
	34	37.8770	-4.2601	-41.3329	37.7930	-4.2033	-41.2880	0.0840	0.0568	0.0449	0.1109	0.0568	Y
	35	35.5398	-3.6857	-42.6885	35.4404	-3.6127	-42.6514	0.0994	0.0730	0.0371	0.1288	0.0730	Y
	36	33.7610	-3.6172	-44.0328	33.6466	-3.5794	-44.0559	0.1144	0.0378	-0.0231	0.1227	0.0378	Y
B-PILLAR Maximum (X, Y, Z)	37	8.9890	-3.3969	-45.0446	8.8570	-3.4137	-44.9326	0.1320	-0.0168	0.1120	0.1739	0.1731	X, Z
	38	5.8991	-6.4873	-34.9785	5.7956	-6.4764	-34.8909	0.1035	0.0109	0.0876	0.1360	0.1360	X, Y, Z
	39	10.6597	-7.9230	-28.3233	10.5298	-7.9283	-28.1987	0.1299	-0.0053	0.1246	0.1801	0.1800	X, Z
	40	7.0662	-8.1778	-23.4345	6.9331	-8.1566	-23.4100	0.1331	0.0212	0.0245	0.1370	0.1370	X, Y, Z
B-PILLAR Lateral (Y)	37	8.9890	-3.3969	-45.0446	8.8570	-3.4137	-44.9326	0.1320	-0.0168	0.1120	0.1739	-0.0168	Y
	38	5.8991	-6.4873	-34.9785	5.7956	-6.4764	-34.8909	0.1035	0.0109	0.0876	0.1360	0.0109	Y
	39	10.6597	-7.9230	-28.3233	10.5298	-7.9283	-28.1987	0.1299	-0.0053	0.1246	0.1801	-0.0053	Y
	40	7.0662	-8.1778	-23.4345	6.9331	-8.1566	-23.4100	0.1331	0.0212	0.0245	0.1370	0.0212	Y

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

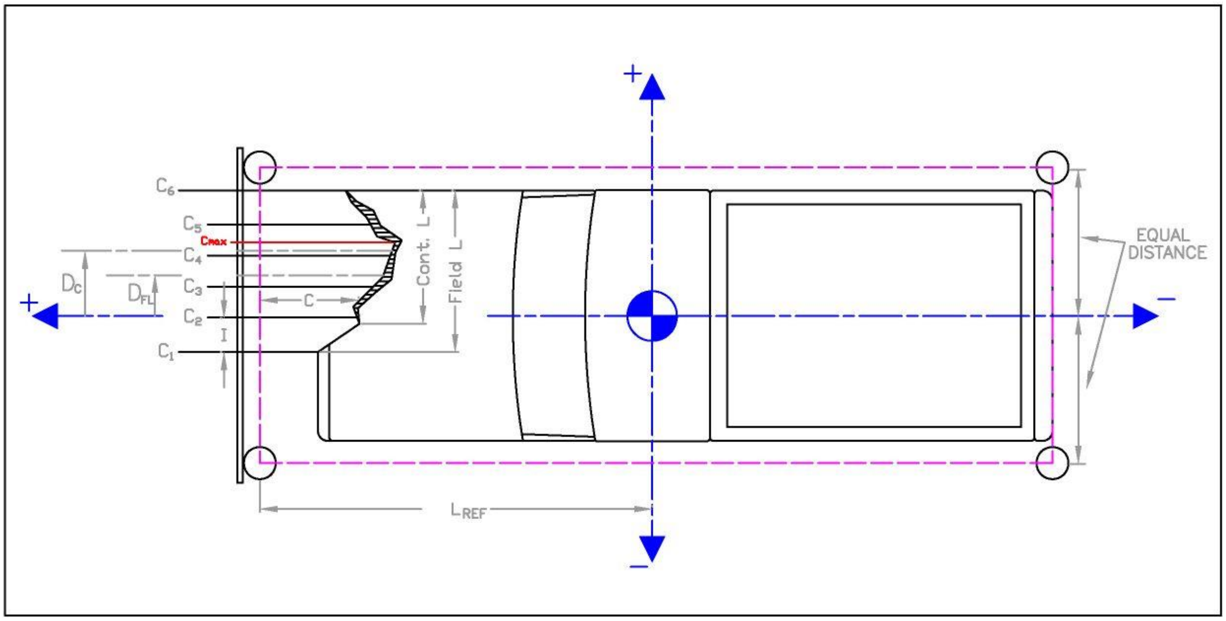
[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-8. Occupant Compartment Deformation Data - Set 2, Test No. STBR-2

Date: 2/26/2019 **Test Name:** STBR-2 **VIN:** 1D7RB1GK0BS554408

Year: 2011 **Make:** Dodge **Model:** Ram 1500



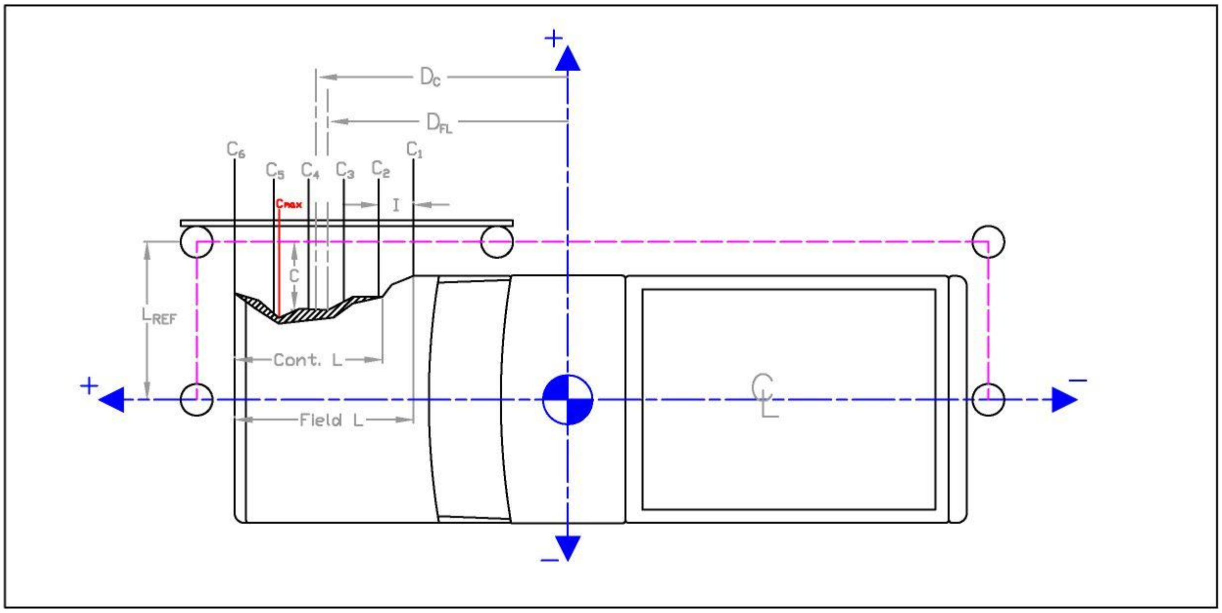
	in.	(mm)
Distance from C.G. to reference line - L_{REF}:	115 3/4	(2940)
Total Vehicle Width:	78 1/4	(1988)
Width of contact and induced crush - Field L:	26	(660)
Crush measurement spacing interval (L/5) - I:	5 1/4	(133)
Distance from center of vehicle to center of Field L - D_{FL}:	-17	-(432)
Width of Contact Damage:	13	(330)
Distance from center of vehicle to center of contact damage - D_C:	-27	-(686)

NOTE: Enter "NA" for crush measurement if distance can not be measured (i.e., side of vehicle has been pushed inward)
NOTE: All values must be filled out above before crush measurements are filled out.

	Crush Measurement		Lateral Location		Original Profile Measurement		Dist. Between Ref. Lines		Actual Crush	
	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)
C_1	N/A	#######	-30	-(762)	9 1/4	(235)	6 7/8	(175)	#######	#######
C_2	19 3/4	(502)	-24 3/4	-(629)	6 7/8	(175)			6	(152)
C_3	13 3/4	(349)	-19 1/2	-(495)	5 1/2	(140)			1 3/8	(35)
C_4	11 3/4	(298)	-14 1/4	-(362)	4 3/4	(121)			1/8	(3)
C_5	10 3/4	(273)	-9	-(229)	4 3/8	(111)			- 1/2	-(13)
C_6	10 1/4	(260)	-3 3/4	-(95)	4	(102)			- 5/8	-(16)
C_{MAX}	30 1/2	(775)	-28	-(711)	8	(203)			15 5/8	(397)

Figure F-9. Exterior Vehicle Crush (NASS) - Front, Test No. STBR-2

Date:	2/26/2019	Test Name:	STBR-2	VIN:	1D7RB1GK0BS554408
Year:	2011	Make:	Dodge	Model:	Ram 1500



	in.	(mm)
Distance from centerline to reference line - L_{REF}:	43	(1092)
Total Vehicle Length:	229 1/8	(5820)
Distance from vehicle c.g. to 1/2 of Vehicle total length:	-3 4/7	-(90)
Width of contact and induced crush - Field L:	229 1/8	(5820)
Crush measurement spacing interval (L/5) - I:	45 7/8	(1165)
Distance from vehicle c.g. to center of Field L - D_{FL}:	-10 1/3	-(262)
Width of Contact Damage:	229 1/8	(5820)
Distance from vehicle c.g. to center of contact damage - D_C:	-10 1/3	-(262)

NOTE: Enter "NA" for crush measurement if distance can not be measured (i.e., front of vehicle has been pushed inward or tire has been removed)
NOTE: All values must be filled out above before crush measurements are filled out.

	Crush Measurement		Longitudinal Location		Original Profile Measurement		Dist. Between Ref. Lines		Actual Crush	
	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)
C_1	16	(406)	-124 7/8	-(3172)	33 1/2	(851)	-1	-(25)	-16 1/2	-(419)
C_2	N/A	#######	-79	-(2007)	5 1/2	(140)			#######	#######
C_3	2	(51)	-33 1/8	-(841)	5 7/8	(149)			-2 7/8	-(73)
C_4	1 1/2	(38)	12 3/4	(324)	5	(127)			-2 1/2	-(64)
C_5	N/A	#######	58 5/8	(1489)	5 7/8	(149)			#######	#######
C_6	N/A	#######	104 1/2	(2654)	11 7/8	(302)			#######	#######
C_{MAX}	21	(533)	100 1/2	(2553)	8 1/8	(206)			13 7/8	(352)

Figure F-10. Exterior Vehicle Crush (NASS) - Side, Test No. STBR-2

Date: 2/28/2019 Test Name: STBR-3 VIN: KNADE223996504334
Year: 2009 Make: Kia Model: Rio

VEHICLE DEFORMATION
DRIVER SIDE FLOOR PAN - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
TOE PAN - WHEEL WELL (X, Z)	1	62.3397	-11.3339	5.7656	62.2156	-11.3196	5.6495	0.1241	0.0143	0.1161	0.1705	0.1699	X, Z
	2	62.6193	-16.1271	5.5682	62.5100	-16.0219	5.6282	0.1093	0.1052	-0.0600	0.1631	0.1093	X
	3	62.8779	-19.3666	5.4358	62.7407	-19.3097	5.2896	0.1372	0.0569	0.1462	0.2084	0.2005	X, Z
	4	62.9213	-21.7944	5.1305	62.7622	-21.7165	5.0103	0.1591	0.0779	0.1202	0.2141	0.1994	X, Z
	5	62.9737	-25.1874	4.6968	62.8089	-25.0855	4.6870	0.1648	0.1019	0.0098	0.1940	0.1651	X, Z
	6	59.3535	-11.2397	7.3807	59.2498	-11.1239	7.2751	0.1037	0.1158	0.1056	0.1879	0.1480	X, Z
	7	59.5227	-15.1377	7.4224	59.3952	-15.1109	7.2561	0.1275	0.0268	0.1663	0.2113	0.2096	X, Z
	8	59.7093	-19.2272	7.3450	59.5636	-19.1554	7.2491	0.1457	0.0718	0.0959	0.1886	0.1744	X, Z
	9	59.7402	-21.6551	7.0791	59.5652	-21.5934	6.9841	0.1750	0.0617	0.0950	0.2085	0.1991	X, Z
	10	59.6219	-25.0173	7.4558	59.4848	-24.9908	7.3806	0.1371	0.0265	0.0752	0.1586	0.1564	X, Z
FLOOR PAN (Z)	11	53.3513	-11.8409	8.3307	53.2620	-11.8462	8.2232	0.0893	-0.0053	0.1075	0.1399	0.1075	Z
	12	53.2047	-14.5948	8.4267	53.0446	-14.4790	7.8464	0.1601	0.1158	0.5803	0.6130	0.5803	Z
	13	53.1716	-17.7163	8.1771	52.9751	-17.6354	8.0878	0.1965	0.0809	0.0893	0.2305	0.0893	Z
	14	53.3275	-21.8437	8.2004	53.1546	-21.7426	8.1451	0.1729	0.1011	0.0553	0.2078	0.0553	Z
	15	53.6792	-27.6775	8.3972	53.5230	-27.6367	8.4153	0.1562	0.0408	-0.0181	0.1625	-0.0181	Z
	16	50.0339	-11.7337	8.5298	49.9309	-11.6881	8.4380	0.1030	0.0456	0.0918	0.1453	0.0918	Z
	17	49.7841	-16.2834	8.6423	49.6258	-16.1917	8.2866	0.1583	0.0917	0.3557	0.4000	0.3557	Z
	18	49.6639	-20.5699	8.1625	49.5255	-20.4925	8.0732	0.1384	0.0774	0.0893	0.1820	0.0893	Z
	19	49.7267	-24.4998	8.2537	49.5318	-24.3982	8.1951	0.1949	0.1016	0.0586	0.2275	0.0586	Z
	20	50.0859	-28.6584	8.7166	49.9137	-28.5619	8.6960	0.1722	0.0965	0.0206	0.1985	0.0206	Z
	21	44.8065	-12.2412	8.8167	44.7278	-12.1797	8.6811	0.0787	0.0615	0.1356	0.1684	0.1356	Z
	22	44.3714	-16.7059	8.5181	44.2135	-16.6568	8.3905	0.1579	0.0491	0.1276	0.2089	0.1276	Z
	23	44.2613	-21.1061	8.2171	44.1567	-21.0770	8.1516	0.1046	0.0291	0.0655	0.1268	0.0655	Z
	24	44.2533	-24.7807	8.3521	44.1624	-24.7101	8.2748	0.0909	0.0706	0.0773	0.1386	0.0773	Z
	25	44.1537	-29.4459	8.5950	44.0061	-29.4322	8.5554	0.1476	0.0137	0.0396	0.1534	0.0396	Z
	26	40.0713	-12.1978	8.1803	39.9856	-12.0594	8.1022	0.0857	0.1384	0.0781	0.1806	0.0781	Z
	27	39.7378	-16.7835	8.3807	39.6041	-16.7368	8.4061	0.1337	0.0467	-0.0254	0.1439	-0.0254	Z
	28	40.0031	-21.3311	8.2690	39.8348	-21.1947	8.2549	0.1683	0.1364	0.0141	0.2171	0.0141	Z
	29	40.4036	-25.1145	8.3005	40.2461	-25.0051	8.2171	0.1575	0.1094	0.0834	0.2091	0.0834	Z
	30	40.4516	-28.3763	8.3737	40.3230	-28.2954	8.3803	0.1286	0.0809	-0.0066	0.1521	-0.0066	Z

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Pretest Floor Pan

Posttest Floor Pan

Figure F-11. Floorpan Deformation Data – Set 1, Test No. STBR-3

Date: 2/28/2019 | Test Name: STBR-3 | VIN: KNADE223996504334
Year: 2009 | Make: Kia | Model: Rio

VEHICLE DEFORMATION
DRIVER SIDE FLOOR PAN - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX^A (in.)	ΔY^A (in.)	ΔZ^A (in.)	Total Δ (in.)	Crush^B (in.)	Directions for Crush^C
TOE PAN - WHEEL WELL (X, Z)	1	62.2245	8.6078	5.6156	62.1137	8.6188	5.5660	0.1108	-0.0110	0.0496	0.1219	0.1214	X, Z
	2	62.4452	3.8094	5.4745	62.3653	3.9146	5.6409	0.0799	-0.1052	-0.1664	0.2125	0.0799	X
	3	62.6640	0.5656	5.3800	62.5660	0.6186	5.3696	0.0980	-0.0530	0.0104	0.1119	0.0986	X, Z
	4	62.6769	-1.8659	5.1035	62.5656	-1.7935	5.1396	0.1113	0.0724	-0.0361	0.1376	0.1113	X
	5	62.6868	-5.2642	4.7102	62.5815	-5.1687	4.8853	0.1053	0.0955	-0.1751	0.2255	0.1053	X
	6	59.2443	8.7575	7.2376	59.1501	8.8747	7.1870	0.0942	-0.1172	0.0506	0.1587	0.1069	X, Z
	7	59.3662	4.8585	7.3252	59.2592	4.8870	7.2497	0.1070	-0.0285	0.0755	0.1340	0.1310	X, Z
	8	59.5028	0.7664	7.2960	59.3908	0.8418	7.3254	0.1120	-0.0754	-0.0294	0.1382	0.1120	X
	9	59.5034	-1.6646	7.0589	59.3701	-1.6010	7.1104	0.1333	0.0636	-0.0515	0.1564	0.1333	X
	10	59.3453	-5.0204	7.4760	59.2589	-4.9887	7.5763	0.0864	0.0317	-0.1003	0.1361	0.0864	X
FLOOR PAN (Z)	11	53.2380	8.2409	8.2110	53.1561	8.2265	8.1495	0.0819	0.0144	0.0615	0.1034	0.0615	Z
	12	53.0582	5.4904	8.3401	52.9147	5.5886	7.8267	0.1435	-0.0982	0.5134	0.5420	0.5134	Z
	13	52.9864	2.3668	8.1279	52.8165	2.4386	8.1326	0.1699	-0.0718	-0.0047	0.1845	-0.0047	Z
	14	53.0920	-1.7616	8.1999	52.9587	-1.6680	8.2740	0.1333	0.0936	-0.0741	0.1789	-0.0741	Z
	15	53.3734	-7.5966	8.4652	53.2734	-7.5585	8.6647	0.1000	0.0381	-0.1995	0.2264	-0.1995	Z
	16	49.9227	8.3910	8.4179	49.8266	8.4193	8.3610	0.0961	-0.0283	0.0569	0.1152	0.0569	Z
	17	49.6180	3.8464	8.5852	49.4805	3.9165	8.3017	0.1375	-0.0701	0.2835	0.3228	0.2835	Z
	18	49.4442	-0.4437	8.1568	49.3410	-0.3867	8.1763	0.1032	0.0570	-0.0195	0.1195	-0.0195	Z
	19	49.4594	-4.3727	8.2945	49.3118	-4.2890	8.3782	0.1476	0.0837	-0.0837	0.1892	-0.0837	Z
	20	49.7692	-8.5296	8.8060	49.6559	-8.4449	8.9642	0.1133	0.0847	-0.1582	0.2122	-0.1582	Z
	21	44.6905	7.9508	8.7250	44.6193	7.9802	8.6139	0.0712	-0.0294	0.1111	0.1352	0.1111	Z
	22	44.2001	3.4885	8.4807	44.0642	3.5029	8.4150	0.1359	-0.0144	0.0657	0.1516	0.0657	Z
	23	44.0357	-0.9133	8.2325	43.9672	-0.9205	8.2665	0.0685	-0.0072	-0.0340	0.0768	-0.0340	Z
	24	43.9832	-4.5856	8.4112	43.9398	-4.5503	8.4640	0.0434	0.0353	-0.0528	0.0769	-0.0528	Z
	25	43.8276	-9.2460	8.7099	43.7406	-9.2640	8.8412	0.0870	-0.0180	-0.1313	0.1585	-0.1313	Z
	26	39.9543	8.0445	8.1010	39.8783	8.1318	8.0325	0.0760	-0.0873	0.0685	0.1345	0.0685	Z
	27	39.5656	3.4658	8.3568	39.4542	3.4653	8.4320	0.1114	0.0005	-0.0752	0.1344	-0.0752	Z
	28	39.7752	-1.0856	8.2986	39.6444	-0.9968	8.3721	0.1308	0.0888	-0.0735	0.1743	-0.0735	Z
	29	40.1296	-4.8730	8.3740	40.0210	-4.8107	8.4122	0.1086	0.0623	-0.0382	0.1309	-0.0382	Z
	30	40.1382	-8.1341	8.4860	40.0680	-8.0975	8.6427	0.0702	0.0366	-0.1567	0.1756	-0.1567	Z

A Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

B Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

C Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Pretest Floor Pan	Posttest Floor Pan
	

Figure F-12. Floorpan Deformation Data - Set 2, Test No. STBR-3

Date: 2/28/2019 Test Name: STBR-3 VIN: KNADE223996504334
Year: 2009 Make: Kia Model: Rio

VEHICLE DEFORMATION
DRIVER SIDE INTERIOR CRUSH - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	50.3498	-22.9050	-21.8002	50.2408	-22.6220	-22.1529	0.1090	0.2830	-0.3527	0.4652	0.4652	X, Y, Z
	2	50.3631	-12.5009	-20.5589	50.2600	-12.2783	-21.0371	0.1031	0.2226	-0.4782	0.5375	0.5375	X, Y, Z
	3	49.6168	-3.8966	-20.3529	49.5161	-3.6320	-20.8452	0.1007	0.2646	-0.4923	0.5679	0.5679	X, Y, Z
	4	48.9852	-27.3425	-8.8753	48.8128	-26.9281	-9.1321	0.1724	0.4144	-0.2568	0.5171	0.5171	X, Y, Z
	5	48.6423	-18.5297	-8.0788	48.5015	-18.1232	-8.4983	0.1408	0.4065	-0.4195	0.6009	0.6009	X, Y, Z
	6	44.3030	-5.3117	-12.7726	44.2572	-5.1028	-13.1851	0.0458	0.2089	-0.4125	0.4646	0.4646	X, Y, Z
SIDE PANEL (Y)	7	54.3791	-32.0808	-0.0443	54.2815	-31.5270	-0.1777	0.0976	0.5538	-0.1334	0.5779	0.5538	Y
	8	53.9888	-32.9178	4.0510	54.0525	-32.7057	3.8738	-0.0637	0.2121	-0.1772	0.2836	0.2121	Y
	9	58.3169	-32.6574	4.1026	58.3377	-32.4091	3.8438	-0.0208	0.2483	-0.2588	0.3593	0.2483	Y
IMPACT SIDE DOOR (Y)	10	47.7103	-33.3426	-15.6747	46.9161	-32.9896	-16.0100	0.7942	0.3530	-0.3353	0.9316	0.3530	Y
	11	35.7442	-33.6217	-16.5758	35.0803	-34.3742	-16.9175	0.6639	-0.7525	-0.3417	1.0601	-0.7525	Y
	12	23.5461	-33.8711	-17.4288	22.9209	-34.9758	-17.6727	0.6252	-1.1047	-0.2439	1.2926	-1.1047	Y
	13	45.3049	-33.7275	-4.1707	44.7186	-34.6631	-4.5515	0.5863	-0.9356	-0.3808	1.1679	-0.9356	Y
	14	37.0890	-34.4212	-1.3142	36.5923	-35.7706	-1.6889	0.4967	-1.3494	-0.3747	1.4859	-1.3494	Y
	15	27.7620	-34.0548	-0.1480	27.3084	-35.9458	-0.4275	0.4536	-1.8910	-0.2795	1.9646	-1.8910	Y
ROOF - (Z)	16	28.6502	-20.6888	-37.8029	28.3215	-21.1679	-38.2362	0.3287	-0.4791	-0.4333	0.7248	-0.4333	Z
	17	29.3511	-15.6255	-37.9749	29.0822	-16.1377	-38.4057	0.2689	-0.5122	-0.4308	0.7213	-0.4308	Z
	18	29.4443	-11.3635	-38.0944	29.1722	-11.7820	-38.4787	0.2721	-0.4185	-0.3843	0.6300	-0.3843	Z
	19	29.6251	-7.9837	-38.0970	29.3085	-8.4715	-38.2896	0.3166	-0.4878	-0.1926	0.6126	-0.1926	Z
	20	29.9074	-5.2865	-38.0246	29.6425	-5.7443	-38.3363	0.2649	-0.4578	-0.3117	0.6139	-0.3117	Z
	21	22.7340	-20.4706	-38.6490	22.3497	-20.8590	-38.8631	0.3843	-0.3884	-0.2141	0.5868	-0.2141	Z
	22	23.0828	-15.6111	-38.8872	22.7077	-16.0455	-38.8238	0.3751	-0.4344	0.0634	0.5774	0.0634	Z
	23	23.0716	-11.6952	-39.0147	22.7650	-12.2341	-38.6180	0.3066	-0.5389	0.3967	0.7361	0.3967	Z
	24	23.6752	-8.1014	-38.9917	23.2969	-8.5508	-38.4006	0.3783	-0.4494	0.5911	0.8333	0.5911	Z
	25	23.9036	-4.1627	-38.9385	23.5316	-4.6773	-38.7709	0.3720	-0.5146	0.1676	0.6567	0.1676	Z
	26	15.0266	-20.9266	-39.1529	14.6719	-21.3610	-38.8443	0.3547	-0.4344	0.3086	0.6401	0.3086	Z
	27	15.3460	-18.3576	-39.3005	14.9862	-18.7958	-38.6633	0.3598	-0.4382	0.6372	0.8529	0.6372	Z
	28	16.1619	-13.2744	-39.4832	15.7640	-13.7598	-38.7121	0.3979	-0.4854	0.7711	0.9942	0.7711	Z
	29	16.7178	-8.4779	-39.5359	16.3873	-8.9350	-39.2031	0.3305	-0.4571	0.3328	0.6549	0.3328	Z
	30	17.7443	-3.4163	-39.4353	17.3485	-3.9409	-39.5447	0.3958	-0.5246	-0.1094	0.6662	-0.1094	Z
A-PILLAR Maximum (X, Y, Z)	31	53.0726	-30.4324	-23.3014	52.7999	-29.8384	-23.6483	0.2727	0.5940	-0.3469	0.7400	0.6536	X, Y
	32	49.8879	-29.8488	-25.3142	49.5861	-29.4237	-25.7122	0.3018	0.4251	-0.3980	0.6559	0.5213	X, Y
	33	47.2878	-29.3290	-26.9181	47.0488	-29.0578	-27.3835	0.2390	0.2712	-0.4654	0.5893	0.3615	X, Y
	34	43.5596	-28.5277	-29.1475	43.2766	-28.4722	-29.6534	0.2830	0.0555	-0.5059	0.5823	0.2884	X, Y
	35	40.0952	-27.8980	-30.7714	39.8859	-27.9856	-31.3187	0.2093	-0.0876	-0.5473	0.5925	0.2093	X
	36	37.7325	-27.3906	-32.0065	37.5574	-27.5937	-32.5742	0.1751	-0.2031	-0.5677	0.6278	0.1751	X
A-PILLAR Lateral (Y)	31	53.0726	-30.4324	-23.3014	52.7999	-29.8384	-23.6483	0.2727	0.5940	-0.3469	0.7400	0.5940	Y
	32	49.8879	-29.8488	-25.3142	49.5861	-29.4237	-25.7122	0.3018	0.4251	-0.3980	0.6559	0.4251	Y
	33	47.2878	-29.3290	-26.9181	47.0488	-29.0578	-27.3835	0.2390	0.2712	-0.4654	0.5893	0.2712	Y
	34	43.5596	-28.5277	-29.1475	43.2766	-28.4722	-29.6534	0.2830	0.0555	-0.5059	0.5823	0.0555	Y
	35	40.0952	-27.8980	-30.7714	39.8859	-27.9856	-31.3187	0.2093	-0.0876	-0.5473	0.5925	-0.0876	Y
	36	37.7325	-27.3906	-32.0065	37.5574	-27.5937	-32.5742	0.1751	-0.2031	-0.5677	0.6278	-0.2031	Y
B-PILLAR Maximum (X, Y, Z)	37	15.4576	-27.6379	-33.4596	15.1436	-28.2183	-33.6911	0.3140	-0.5804	-0.2315	0.6993	0.3140	X
	38	13.4135	-30.4841	-27.2950	13.2053	-31.0090	-27.4883	0.2082	-0.5249	-0.1933	0.5969	0.2082	X
	39	18.2384	-32.0546	-20.6431	18.0273	-32.5712	-20.8531	0.2111	-0.5166	-0.2100	0.5963	0.2111	X
	40	14.8774	-32.4733	-16.7450	14.7287	-32.9128	-16.9495	0.1487	-0.4395	-0.2045	0.5070	0.1487	X
B-PILLAR Lateral (Y)	37	15.4576	-27.6379	-33.4596	15.1436	-28.2183	-33.6911	0.3140	-0.5804	-0.2315	0.6993	-0.5804	Y
	38	13.4135	-30.4841	-27.2950	13.2053	-31.0090	-27.4883	0.2082	-0.5249	-0.1933	0.5969	-0.5249	Y
	39	18.2384	-32.0546	-20.6431	18.0273	-32.5712	-20.8531	0.2111	-0.5166	-0.2100	0.5963	-0.5166	Y
	40	14.8774	-32.4733	-16.7450	14.7287	-32.9128	-16.9495	0.1487	-0.4395	-0.2045	0.5070	-0.4395	Y

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-13. Occupant Compartment Deformation Data - Set 1, Test No. STBR-3

Date: 2/28/2019 Test Name: STBR-3 VIN: KNADE223996504334
Year: 2009 Make: Kia Model: Rio

VEHICLE DEFORMATION
DRIVER SIDE INTERIOR CRUSH - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	50.0127	-3.3503	-21.8870	50.0238	-3.1578	-22.0071	-0.0111	0.1925	-0.1201	0.2272	0.2272	X, Y, Z
	2	50.1313	7.0719	-20.8137	50.1385	7.2064	-21.1076	-0.0072	-0.1345	-0.2939	0.3233	0.3233	X, Y, Z
	3	49.4662	15.6849	-20.7409	49.4741	15.8612	-21.0962	-0.0079	-0.1763	-0.3553	0.3967	0.3967	X, Y, Z
	4	48.7035	-7.5669	-8.8827	48.5639	-7.1776	-8.8986	0.1396	0.3893	-0.0159	0.4139	0.4139	X, Y, Z
	5	48.4481	1.2605	-8.2258	48.3338	1.6411	-8.4488	0.1143	-0.3806	-0.2230	0.4557	0.4557	X, Y, Z
	6	44.1962	14.4415	-13.1002	44.2064	14.5992	-13.4050	-0.0102	-0.1577	-0.3048	0.3433	0.3433	X, Y, Z
SIDE PANEL (Y)	7	54.1191	-12.2131	-0.0161	53.9953	-11.6385	0.1478	0.1238	0.5746	0.1639	0.6102	0.5746	Y
	8	53.7516	-12.9806	4.0948	53.7579	-12.7301	4.2231	-0.0063	0.2505	0.1283	0.2815	0.2505	Y
	9	58.0822	-12.7600	4.1106	58.0456	-12.4736	4.1853	0.0366	0.2864	0.0747	0.2982	0.2864	Y
IMPACT SIDE DOOR (Y)	10	47.3226	-13.6630	-15.5750	46.6077	-13.3638	-15.6476	0.7149	0.2992	-0.0726	0.7784	0.2992	Y
	11	35.3480	-13.8446	-16.3842	34.7591	-14.6583	-16.5214	0.5889	-0.8137	-0.1372	1.0138	-0.8137	Y
	12	23.1421	-13.9934	-17.1438	22.5943	-15.1638	-17.2592	0.5478	-1.1704	-0.1154	1.2974	-1.1704	Y
	13	44.9995	-13.8408	-4.0491	44.4015	-14.7773	-4.1558	0.5980	-0.9365	-0.1067	1.1163	-0.9365	Y
	14	36.7991	-14.4116	-1.1218	36.2671	-15.7501	-1.2675	0.5320	-1.3385	-0.1457	1.4477	-1.3385	Y
	15	27.4848	-13.9393	0.1064	26.9827	-15.8136	0.0008	0.5021	-1.8743	-0.1056	1.9433	-1.8743	Y
ROOF - (Z)	16	28.2158	-1.1881	-37.7644	28.1095	-1.8387	-38.1088	0.1063	-0.6506	-0.3444	0.7438	-0.3444	Z
	17	28.9622	3.8650	-38.0231	28.9162	3.1796	-38.3837	0.0460	0.6854	-0.3606	0.7758	-0.3606	Z
	18	29.0938	8.1235	-38.2120	29.0462	7.5318	-38.5477	0.0476	0.5917	-0.3357	0.6820	-0.3357	Z
	19	29.3058	11.5010	-38.2703	29.2129	10.8442	-38.4279	0.0929	0.6568	-0.1576	0.6818	-0.1576	Z
	20	29.6136	14.1962	-38.2435	29.5719	13.5667	-38.5317	0.0417	0.6295	-0.2882	0.6936	-0.2882	Z
	21	22.2958	-0.9281	-38.5707	22.1404	-1.4882	-38.7397	0.1554	-0.5601	-0.1690	0.6053	-0.1690	Z
	22	22.6876	3.9235	-38.8897	22.5426	3.3217	-38.8012	0.1450	0.6018	0.0885	0.6253	0.0885	Z
	23	22.7117	7.8368	-39.0802	22.6350	7.1358	-38.6750	0.0767	0.7010	0.4052	0.8133	0.4052	Z
	24	23.3486	11.4247	-39.1195	23.2008	10.8178	-38.5349	0.1478	0.6069	0.5846	0.8555	0.5846	Z
	25	23.6138	15.3614	-39.1314	23.4707	14.6804	-38.9861	0.1431	0.6810	0.1453	0.7109	0.1453	Z
	26	14.5809	-1.3200	-39.0109	14.4584	-1.9191	-38.7075	0.1225	-0.5991	0.3034	0.6826	0.3034	Z
	27	14.9229	1.2432	-39.2022	14.7963	0.6463	-38.5802	0.1266	0.5969	0.6220	0.8713	0.6220	Z
	28	15.7844	6.3150	-39.4727	15.6202	5.6729	-38.7345	0.1642	0.6421	0.7382	0.9921	0.7382	Z
	29	16.3842	11.1046	-39.6068	16.2875	10.4803	-39.3265	0.0967	0.6243	0.2803	0.6911	0.2803	Z
	30	17.4581	16.1574	-39.5952	17.2942	15.4572	-39.7728	0.1639	0.7002	-0.1776	0.7407	-0.1776	Z
A-PILLAR Maximum (X, Y, Z)	31	52.6546	-10.9260	-23.2866	52.5158	-10.4270	-23.3524	0.1388	0.4990	-0.0658	0.5221	0.5179	X, Y
	32	49.4604	-10.3449	-25.2853	49.3047	-10.0260	-25.4232	0.1557	0.3189	-0.1379	0.3807	0.3549	X, Y
	33	46.8534	-9.8266	-26.8783	46.7699	-9.6719	-27.1008	0.0835	0.1547	-0.2225	0.2836	0.1758	X, Y
	34	43.1163	-9.0263	-29.0930	43.0019	-9.0991	-29.3810	0.1144	-0.0728	-0.2880	0.3183	0.1144	X
	35	39.6458	-8.3903	-30.7014	39.6148	-8.6164	-31.0548	0.0310	-0.2261	-0.3534	0.4207	0.0310	X
	36	37.2787	-7.8808	-31.9273	37.2893	-8.2294	-32.3173	-0.0106	-0.3486	-0.3900	0.5232	0.0000	NA
A-PILLAR Lateral (Y)	31	52.6546	-10.9260	-23.2866	52.5158	-10.4270	-23.3524	0.1388	0.4990	-0.0658	0.5221	0.4990	Y
	32	49.4604	-10.3449	-25.2853	49.3047	-10.0260	-25.4232	0.1557	0.3189	-0.1379	0.3807	0.3189	Y
	33	46.8534	-9.8266	-26.8783	46.7699	-9.6719	-27.1008	0.0835	0.1547	-0.2225	0.2836	0.1547	Y
	34	43.1163	-9.0263	-29.0930	43.0019	-9.0991	-29.3810	0.1144	-0.0728	-0.2880	0.3183	-0.0728	Y
	35	39.6458	-8.3903	-30.7014	39.6148	-8.6164	-31.0548	0.0310	-0.2261	-0.3534	0.4207	-0.2261	Y
	36	37.2787	-7.8808	-31.9273	37.2893	-8.2294	-32.3173	-0.0106	-0.3486	-0.3900	0.5232	-0.3486	Y
B-PILLAR Maximum (X, Y, Z)	37	14.9923	-7.9428	-33.2135	14.8701	-8.6713	-33.4123	0.1222	-0.7285	-0.1988	0.7650	0.1222	X
	38	12.9680	-10.6704	-26.9890	12.9098	-11.3139	-27.1518	0.0582	-0.6435	-0.1628	0.6663	0.0582	X
	39	17.8276	-12.1792	-20.3481	17.7211	-12.7813	-20.4873	0.1065	-0.6021	-0.1392	0.6271	0.1065	X
	40	14.4920	-12.5038	-16.4193	14.4218	-13.0111	-16.5761	0.0702	-0.5073	-0.1568	0.5356	0.0702	X
B-PILLAR Lateral (Y)	37	14.9923	-7.9428	-33.2135	14.8701	-8.6713	-33.4123	0.1222	-0.7285	-0.1988	0.7650	-0.7285	Y
	38	12.9680	-10.6704	-26.9890	12.9098	-11.3139	-27.1518	0.0582	-0.6435	-0.1628	0.6663	-0.6435	Y
	39	17.8276	-12.1792	-20.3481	17.7211	-12.7813	-20.4873	0.1065	-0.6021	-0.1392	0.6271	-0.6021	Y
	40	14.4920	-12.5038	-16.4193	14.4218	-13.0111	-16.5761	0.0702	-0.5073	-0.1568	0.5356	-0.5073	Y

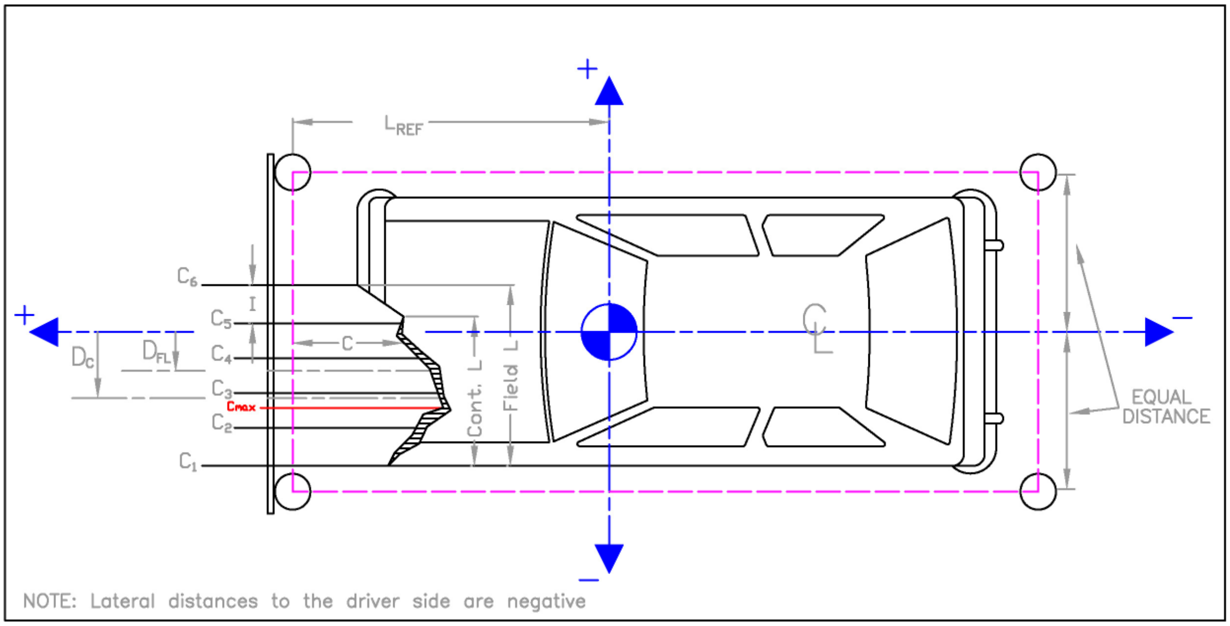
[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-14. Occupant Compartment Deformation Data - Set 2, Test No. STBR-3

Date:	3/5/2019	Test Name:	STBR-3	VIN:	KNADE223996504334
Year:	2009	Make:	Kia	Model:	Rio



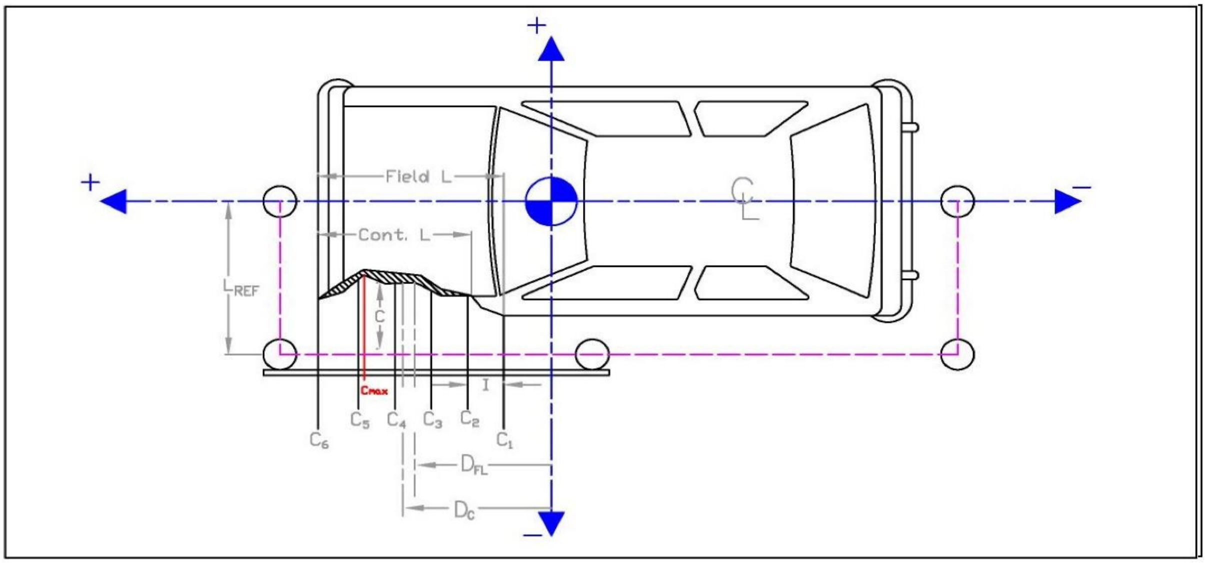
	in.	(mm)
Distance from C.G. to reference line - L_{REF}:	82 1/2	(2096)
Total Width of Vehicle:	64 3/4	(1645)
Width of contact and induced crush - Field L:	64 3/4	(1645)
Crush measurement spacing interval (L/5) - I:	13	(330)
Distance from center of vehicle to center of Field L - D_{FL}:	0	()
Width of Contact Damage:	20	(508)
Distance from center of vehicle to center of contact damage - D_C:	19	(483)

NOTE: Enter "NA" for crush measurement if distance can not be measured (i.e., side of vehicle has been pushed inward)
NOTE: All values must be filled out above before crush measurements are filled out.

	Crush Measurement		Lateral Location		Original Profile Measurement		Dist. Between Ref. Lines		Actual Crush	
	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)
C_1	N/A	NA	-32 3/8	-(822)	24	(610)	7 4/9	(189)	NA	NA
C_2	14 1/4	(362)	-19 3/8	-(492)	8 3/8	(213)			-1 4/7	-(40)
C_3	11	(279)	-6 3/8	-(162)	6 1/8	(156)			-2 4/7	-(65)
C_4	11 1/4	(286)	6 5/8	(168)	6 1/8	(156)			-2 1/3	-(59)
C_5	18 5/8	(473)	19 5/8	(498)	8 3/8	(213)			2 4/5	(71)
C_6	N/A	NA	32 5/8	(829)	24	(610)			NA	NA
C_{MAX}	35 1/2	(902)	-21 1/2	-(546)	9 1/4	(235)			18 4/5	(478)

Figure F-15. Exterior Vehicle Crush (NASS) - Front, Test No. STBR-3

Date:	3/5/2019	Test Name:	STBR-3	VIN:	KNADE223996504334
Year:	2009	Make:	Kia	Model:	Rio



	in.	(mm)
Distance from centerline to reference line - L_{REF}:	36	(914)
Total Vehicle Length:	167 3/8	(4251)
Distance from vehicle c.g. to 1/2 of Vehicle total length:	-9 5/8	-(244)
Width of contact and induced crush - Field L:	154	(3912)
Crush measurement spacing interval (L/5) - I:	30 3/4	(781)
Distance from vehicle c.g. to center of Field L - D_{FL}:	8 4/9	(214)
Width of Contact Damage:	154 3/8	(3921)
Distance from vehicle c.g. to center of contact damage - D_C:	8 4/9	(214)

NOTE: Enter "NA" for crush measurement if distance can not be measured (i.e., front of vehicle has been pushed inward or tire has been removed)
NOTE: All values must be filled out above before crush measurements are filled out.

	Crush Measurement		Longitudinal Location		Original Profile Measurement		Dist. Between Ref. Lines		Actual Crush	
	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)
C_1	5	(127)	-68 5/8	-(1743)	4	(102)	0	()	1	(25)
C_2	2 7/8	(73)	-37 7/8	-(962)	3 5/8	(92)			- 3/4	-(19)
C_3	2 5/8	(67)	-7 1/8	-(181)	3 3/4	(95)			-1 1/8	-(29)
C_4	7 1/4	(184)	23 5/8	(600)	3 3/8	(86)			3 7/8	(98)
C_5	19 3/8	(492)	54 3/8	(1381)	5	(127)			14 3/8	(365)
C_6	N/A	NA	85 1/8	(2162)	21 3/4	(552)			NA	NA
C_{MAX}	23 3/4	(603)	62 1/4	(1581)	8 3/8	(213)			15 3/8	(391)

Figure F-16. Exterior Vehicle Crush (NASS) - Side, Test No. STBR-3

Date: 5/22/2019 | Test Name: STBR-4 | VIN: 1FVACXCS37HX61820
Year: 2007 | Make: Freightliner | Model: M2 106

VEHICLE DEFORMATION
FLOOR PAN - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
TOE PAN - WHEEL WELL (X, Z)	1	37.5180	-9.2876	2.1438	37.4911	-9.6121	2.4011	0.0269	-0.3245	-0.2573	0.4150	0.0269	X
	2	38.3316	-15.2598	3.6496	38.2549	-15.5035	3.9703	0.0767	-0.2437	-0.3207	0.4100	0.0767	X
	3	38.0895	-19.8574	3.5648	37.8268	-19.8651	3.4138	0.2627	-0.0077	0.1510	0.3031	0.3030	X, Z
	4	37.8635	-25.2902	3.7820	36.9233	-24.5871	1.2176	0.9402	0.7031	2.5644	2.8204	2.7313	X, Z
	5	37.5404	-31.1958	4.4602	35.7431	-29.9205	0.5307	1.7973	1.2753	3.9295	4.5053	4.3210	X, Z
	6	32.4996	-9.2648	2.7127	32.5255	-9.5955	2.9249	-0.0259	-0.3307	-0.2122	0.3938	0.0000	NA
	7	32.6566	-14.7819	4.1798	32.5706	-14.9185	4.4878	0.0860	-0.1366	-0.3080	0.3477	0.0860	X
	8	32.9807	-19.5105	4.0695	32.8265	-19.3493	3.8383	0.1542	0.1612	0.2312	0.3213	0.2779	X, Z
	9	32.8834	-24.7628	4.3029	32.1718	-24.2328	2.8016	0.7116	0.5300	1.5013	1.7439	1.6614	X, Z
	10	33.3439	-31.7661	5.5829	32.0882	-30.6383	2.7480	1.2557	1.1278	2.8349	3.2993	3.1006	X, Z
FLOOR PAN (Z)	11	28.3059	-9.0441	3.0455	28.2869	-9.3068	3.1868	0.0190	-0.2627	-0.1413	0.2989	-0.1413	Z
	12	28.7122	-14.5103	4.5522	28.5868	-14.6412	4.8315	0.1254	-0.1309	-0.2793	0.3330	-0.2793	Z
	13	28.5594	-19.3991	4.5088	28.3523	-19.1782	4.5490	0.2071	0.2209	-0.0402	0.3055	-0.0402	Z
	14	28.1232	-24.5769	4.8135	27.9709	-24.4342	4.7882	0.1523	0.1427	0.0253	0.2102	0.0253	Z
	15	27.7580	-32.4845	6.9294	27.3346	-31.3260	5.8415	0.4234	1.1585	1.0879	1.6447	1.0879	Z
	16	23.6754	-9.0065	3.1172	23.6338	-9.2487	3.1177	0.0416	-0.2422	-0.0005	0.2457	-0.0005	Z
	17	23.7328	-14.1905	5.0157	23.6138	-14.2639	5.2576	0.1190	-0.0734	-0.2419	0.2794	-0.2419	Z
	18	23.7811	-19.1117	4.9406	23.6622	-19.0423	5.3279	0.1189	0.0694	-0.3873	0.4110	-0.3873	Z
	19	23.6261	-24.2463	5.2268	23.4925	-24.1602	5.2425	0.1336	0.0861	-0.0157	0.1597	-0.0157	Z
	20	22.7813	-31.2396	6.0026	22.3228	-30.8323	5.4222	0.4585	0.4073	0.5804	0.8444	0.5804	Z
	21	18.7732	-8.8282	3.5859	18.7524	-9.0180	3.4672	0.0208	-0.1898	0.1187	0.2248	0.1187	Z
	22	19.0102	-13.8218	5.3814	18.9402	-13.8756	5.6452	0.0700	-0.0538	-0.2638	0.2782	-0.2638	Z
	23	19.2005	-19.0059	5.3710	19.1001	-18.9715	5.8948	0.1004	0.0344	-0.5238	0.5344	-0.5238	Z
	24	19.1765	-24.2120	5.6242	19.1299	-24.2218	6.0763	0.0466	-0.0098	-0.4521	0.4546	-0.4521	Z
	25	20.6019	-26.1312	3.0741	17.9312	-30.7871	6.3875	2.6707	-4.6559	-3.3134	6.3078	-3.3134	Z
	26	14.5540	-8.3662	3.9804	14.4854	-8.6367	3.9287	0.0686	-0.2705	0.0517	0.2838	0.0517	Z
	27	14.3268	-13.6342	4.8422	14.2392	-13.6934	5.0038	0.0876	-0.0592	-0.1616	0.1931	-0.1616	Z
	28	14.3653	-19.0911	4.8493	14.3083	-19.1432	5.0962	0.0570	-0.0521	-0.2469	0.2587	-0.2469	Z
	29	14.0494	-24.1481	4.7840	13.9756	-24.1631	5.0256	0.0738	-0.0150	-0.2416	0.2531	-0.2416	Z
	30	13.0294	-30.2651	4.4409	13.1799	-30.3239	4.4506	-0.1505	-0.0588	-0.0097	0.1619	-0.0097	Z

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Pretest Floor Pan	Posttest Floor Pan
	

Figure F-17. Floorpan Deformation Data – Set 1, Test No. STBR-4

Date: 5/22/2019 Test Name: STBR-4 VIN: 1FVACXCS37HX61820
Year: 2007 Make: Freightliner Model: M2 106

VEHICLE DEFORMATION
FLOOR PAN - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
TOE PAN - WHEEL WELL (X, Z)	1	38.8362	-27.1661	0.9885	38.8165	-27.0232	0.9537	0.0197	0.1429	0.0348	0.1484	0.0400	X, Z
	2	39.7417	-33.1460	2.4088	39.6886	-32.9197	2.4457	0.0531	0.2263	-0.0369	0.2354	0.0531	X
	3	39.5344	-37.7444	2.2841	39.2742	-37.2804	1.8722	0.2602	0.4640	0.4119	0.6728	0.4872	X, Z
	4	39.3587	-43.1808	2.4522	38.3237	-41.9932	-0.3240	1.0350	1.1876	2.7762	3.1920	2.9629	X, Z
	5	39.1034	-49.0953	3.0792	37.1606	-47.3309	-1.0061	1.9428	1.7644	4.0853	4.8556	4.5237	X, Z
	6	33.8366	-27.1910	1.7036	33.8752	-27.0522	1.6712	-0.0386	0.1388	0.0324	0.1477	0.0324	Z
	7	34.0810	-32.7209	3.1091	34.0243	-32.3862	3.1894	0.0567	0.3347	-0.0803	0.3488	0.0567	X
	8	34.4398	-37.4454	2.9410	34.2904	-36.8097	2.4957	0.1494	0.6357	0.4453	0.7904	0.4697	X, Z
	9	34.3919	-42.7003	3.1234	33.6350	-41.6905	1.4471	0.7569	1.0098	1.6763	2.0982	1.8393	X, Z
	10	34.9461	-49.7119	4.3178	33.6011	-48.0959	1.3465	1.3450	1.6160	2.9713	3.6399	3.2615	X, Z
FLOOR PAN (Z)	11	29.6527	-27.0088	2.1608	29.6479	-26.8009	2.1007	0.0048	0.2079	0.0601	0.2165	0.0601	Z
	12	30.1470	-32.4861	3.5990	30.0549	-32.1448	3.6906	0.0921	0.3413	-0.0916	0.3652	-0.0916	Z
	13	30.0325	-37.3754	3.5100	29.8461	-36.6814	3.3819	0.1864	0.6940	0.1281	0.7299	0.1281	Z
	14	29.6473	-42.5595	3.7745	29.5168	-41.9420	3.5945	0.1305	0.6175	0.1800	0.6563	0.1800	Z
	15	29.4080	-50.4905	5.8191	28.9778	-48.8466	4.6179	0.4302	1.6439	1.2012	2.0810	1.2012	Z
	16	25.0262	-27.0106	2.3678	24.9954	-26.7812	2.2138	0.0308	0.2294	0.1540	0.2780	0.1540	Z
	17	25.1810	-32.2126	4.2107	25.0995	-31.8123	4.3134	0.0815	0.4003	-0.1027	0.4212	-0.1027	Z
	18	25.2668	-37.1323	4.0839	25.1892	-36.5906	4.3443	0.0776	0.5417	-0.2604	0.6060	-0.2604	Z
	19	25.1617	-42.2706	4.3220	25.0575	-41.7090	4.2255	0.1042	0.5616	0.0965	0.5793	0.0965	Z
	20	24.3964	-49.2782	5.0505	23.9496	-48.3917	4.3984	0.4468	0.8865	0.6521	1.1877	0.6521	Z
	21	20.1384	-26.8781	2.9809	20.1297	-26.5940	2.7554	0.0087	0.2841	0.2255	0.3628	0.2255	Z
	22	20.4682	-31.8872	4.7176	20.4417	-31.4661	4.8863	0.0265	0.4211	-0.1687	0.4544	-0.1687	Z
	23	20.7000	-37.0692	4.6486	20.6523	-36.5622	5.0895	0.0477	0.5070	-0.4409	0.6736	-0.4409	Z
	24	20.7255	-42.2776	4.8492	20.7315	-41.8133	5.2286	-0.0060	0.4643	-0.3794	0.5996	-0.3794	Z
	25	22.0911	-44.1592	2.2391	19.5989	-48.3906	5.5348	2.4922	-4.2314	-3.2957	5.9142	-3.2957	Z
	26	15.9290	-26.4554	3.5029	15.8811	-26.2518	3.3862	0.0479	0.2036	0.1167	0.2395	0.1167	Z
	27	15.7697	-31.7334	4.3170	15.7179	-31.3184	4.4304	0.0518	0.4150	-0.1134	0.4333	-0.1134	Z
	28	15.8524	-37.1897	4.2673	15.8344	-36.7679	4.4773	0.0180	0.4218	-0.2100	0.4715	-0.2100	Z
	29	15.5757	-42.2482	4.1595	15.5398	-41.7898	4.3805	0.0359	0.4584	-0.2210	0.5102	-0.2210	Z
	30	14.5955	-48.3698	3.7837	14.7719	-47.9526	3.7887	-0.1764	0.4172	-0.0050	0.4530	-0.0050	Z

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

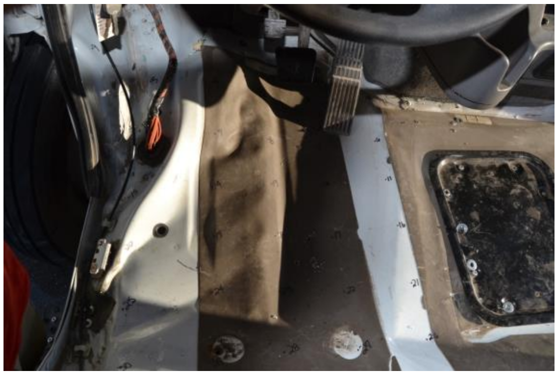
Pretest Floor Pan	Posttest Floor Pan
	

Figure F-18. Floorpan Deformation Data, Set 2, Test No. STBR-4

Date: 5/22/2019 Test Name: STBR-4 VIN: 1FVACXCS37HX61820
Year: 2007 Make: Freightliner Model: M2 106

VEHICLE DEFORMATION
INTERIOR CRUSH - SET 1

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	37.9018	-5.0239	-26.8144	37.8472	-5.3326	-26.5332	0.0546	-0.3087	0.2812	0.4211	0.4211	X, Y, Z
	2	37.5135	-18.9280	-28.4527	37.4388	-19.2497	-28.2177	0.0747	-0.3217	0.2350	0.4053	0.4053	X, Y, Z
	3	35.9941	-30.3882	-25.5505	35.8894	-30.7246	-25.3289	0.1047	-0.3364	0.2216	0.4162	0.4162	X, Y, Z
	4	31.6600	-3.6426	-15.8758	31.6429	-3.9484	-15.6345	0.0171	-0.3058	0.2413	0.3899	0.3899	X, Y, Z
	5	30.1053	-17.1026	-15.3412	29.5884	-17.3571	-15.7457	0.5169	-0.2545	-0.4045	0.7040	0.7040	X, Y, Z
	6	30.8265	-31.1781	-15.7464	30.6476	-31.5539	-15.6645	0.1789	-0.3758	0.0819	0.4242	0.4242	X, Y, Z
SIDE PANEL (Y)	7	37.8506	-33.5763	-6.0322	37.5662	-35.0541	-5.7943	0.2844	-1.4778	0.2379	1.5236	-1.4778	Y
	8	42.0105	-33.6052	-2.5200	41.6126	-34.9433	-2.3525	0.3979	-1.3381	0.1675	1.4060	-1.3381	Y
	9	37.8006	-33.1044	-0.7653	37.2551	-34.5572	-1.4151	0.5455	-1.4528	-0.6498	1.6824	-1.4528	Y
IMPACT SIDE DOOR (Y)	10	4.6656	-34.4134	-21.0775	4.4480	-35.1334	-21.3082	0.2176	-0.7200	-0.2307	0.7867	-0.7200	Y
	11	16.9465	-35.8778	-19.0566	16.7395	-36.6494	-19.1095	0.2070	-0.7716	-0.0529	0.8006	-0.7716	Y
	12	28.6972	-35.9130	-17.8170	28.4566	-36.6012	-17.7198	0.2406	-0.6882	0.0972	0.7355	-0.6882	Y
	13	5.6047	-35.2433	-3.5393	5.1603	-36.0231	-3.7795	0.4444	-0.7798	-0.2402	0.9291	-0.7798	Y
	14	16.3793	-36.2554	-2.3811	15.9803	-36.9699	-2.3994	0.3990	-0.7145	-0.0183	0.8186	-0.7145	Y
	15	25.0145	-36.4447	-2.3427	24.5668	-37.0350	-2.3326	0.4477	-0.5903	0.0101	0.7409	-0.5903	Y
ROOF - (Z)	16	33.0941	-4.9560	-47.8902	33.3837	-5.3203	-47.8452	-0.2896	-0.3643	0.0450	0.4676	0.0450	Z
	17	32.7883	-10.7653	-47.8803	33.1994	-11.1245	-47.8574	-0.4111	-0.3592	0.0229	0.5464	0.0229	Z
	18	31.8451	-17.0904	-47.8489	32.1642	-17.4357	-47.8233	-0.3191	-0.3453	0.0256	0.4709	0.0256	Z
	19	31.0979	-21.9119	-47.8325	31.3959	-22.3033	-47.8092	-0.2980	-0.3914	0.0233	0.4925	0.0233	Z
	20	30.6618	-27.6174	-47.8251	30.9279	-27.9676	-47.8130	-0.2661	-0.3502	0.0121	0.4400	0.0121	Z
	21	13.7536	-3.5218	-53.3000	14.0224	-3.9320	-53.3004	-0.2688	-0.4102	-0.0004	0.4904	-0.0004	Z
	22	13.1195	-8.7277	-53.4122	13.3860	-9.0308	-53.4307	-0.2665	-0.3031	-0.0185	0.4040	-0.0185	Z
	23	12.1783	-16.1647	-51.7076	12.4530	-16.4841	-51.7215	-0.2747	-0.3194	-0.0139	0.4215	-0.0139	Z
	24	12.6128	-21.5911	-52.9459	12.8119	-21.9173	-52.9548	-0.1991	-0.3262	-0.0089	0.3823	-0.0089	Z
	25	11.9451	-28.1818	-51.9487	12.1729	-28.5146	-51.9576	-0.2278	-0.3328	-0.0089	0.4034	-0.0089	Z
	26	-1.1827	-2.8739	-53.3167	-0.9049	-3.1999	-53.3738	-0.2778	-0.3260	-0.0571	0.4321	-0.0571	Z
	27	-1.4532	-8.5257	-53.2532	-1.2271	-8.7861	-53.3076	-0.2261	-0.2604	-0.0544	0.3491	-0.0544	Z
	28	-2.2720	-15.8214	-51.7643	-1.9865	-16.0907	-51.8069	-0.2855	-0.2693	-0.0426	0.3948	-0.0426	Z
	29	-2.1565	-21.5675	-53.1110	-1.8821	-21.8469	-53.1420	-0.2744	-0.2794	-0.0310	0.3928	-0.0310	Z
	30	-2.5574	-27.8102	-52.1423	-2.3242	-28.1168	-52.1425	-0.2332	-0.3066	-0.0002	0.3852	-0.0002	Z
A-PILLAR Maximum (X, Y, Z)	31	37.5272	-34.0673	-27.3927	37.5064	-34.4813	-27.2408	0.0208	-0.4140	0.1519	0.4415	0.1533	X, Z
	32	36.1537	-33.9353	-31.3989	36.1773	-34.3070	-31.3046	-0.0236	-0.3717	0.0943	0.3842	0.0943	Z
	33	34.2224	-33.3859	-35.0041	34.2563	-33.7506	-34.9130	-0.0339	-0.3647	0.0911	0.3774	0.0911	Z
	34	32.5164	-32.6918	-38.5513	32.6323	-33.0500	-38.4926	-0.1159	-0.3582	0.0587	0.3810	0.0587	Z
	35	30.8578	-32.1332	-42.0097	31.0327	-32.5209	-41.8885	-0.1749	-0.3877	0.1212	0.4423	0.1212	Z
	36	29.2821	-30.8458	-45.3642	29.5001	-31.2314	-45.2761	-0.2180	-0.3856	0.0881	0.4516	0.0881	Z
A-PILLAR Lateral (Y)	31	37.5272	-34.0673	-27.3927	37.5064	-34.4813	-27.2408	0.0208	-0.4140	0.1519	0.4415	-0.4140	Y
	32	36.1537	-33.9353	-31.3989	36.1773	-34.3070	-31.3046	-0.0236	-0.3717	0.0943	0.3842	-0.3717	Y
	33	34.2224	-33.3859	-35.0041	34.2563	-33.7506	-34.9130	-0.0339	-0.3647	0.0911	0.3774	-0.3647	Y
	34	32.5164	-32.6918	-38.5513	32.6323	-33.0500	-38.4926	-0.1159	-0.3582	0.0587	0.3810	-0.3582	Y
	35	30.8578	-32.1332	-42.0097	31.0327	-32.5209	-41.8885	-0.1749	-0.3877	0.1212	0.4423	-0.3877	Y
	36	29.2821	-30.8458	-45.3642	29.5001	-31.2314	-45.2761	-0.2180	-0.3856	0.0881	0.4516	-0.3856	Y
B-PILLAR Maximum (X, Y, Z)	37	-1.3302	-31.4904	-46.0059	-1.1274	-31.7822	-46.0028	-0.2028	-0.2918	0.0031	0.3554	0.0031	Z
	38	-8.3065	-31.8293	-41.7919	-8.1000	-32.1360	-41.7505	-0.2065	-0.3067	0.0414	0.3720	0.0414	Z
	39	-2.2365	-32.9676	-35.0963	-2.0664	-33.2402	-35.0813	-0.1701	-0.2726	0.0150	0.3217	0.0150	Z
	40	-8.3924	-33.2476	-28.0816	-8.2257	-33.5181	-28.1387	-0.1667	-0.2705	-0.0571	0.3228	0.0000	NA
B-PILLAR Lateral (Y)	37	-1.3302	-31.4904	-46.0059	-1.1274	-31.7822	-46.0028	-0.2028	-0.2918	0.0031	0.3554	-0.2918	Y
	38	-8.3065	-31.8293	-41.7919	-8.1000	-32.1360	-41.7505	-0.2065	-0.3067	0.0414	0.3720	-0.3067	Y
	39	-2.2365	-32.9676	-35.0963	-2.0664	-33.2402	-35.0813	-0.1701	-0.2726	0.0150	0.3217	-0.2726	Y
	40	-8.3924	-33.2476	-28.0816	-8.2257	-33.5181	-28.1387	-0.1667	-0.2705	-0.0571	0.3228	-0.2705	Y

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-19. Occupant Compartment Deformation Data - Set 1, Test No. STBR-4

Date: 5/22/2019 Test Name: STBR-4 VIN: 1FVACXCS37HX61820
Year: 2007 Make: Freightliner Model: M2 106

VEHICLE DEFORMATION
INTERIOR CRUSH - SET 2

	POINT	Pretest X (in.)	Pretest Y (in.)	Pretest Z (in.)	Posttest X (in.)	Posttest Y (in.)	Posttest Z (in.)	ΔX[A] (in.)	ΔY[A] (in.)	ΔZ[A] (in.)	Total Δ (in.)	Crush[B] (in.)	Directions for Crush[C]
DASH (X, Y, Z)	1	38.3783	-22.5925	-27.8804	38.0054	-22.5070	-27.9529	0.3729	0.0855	-0.0725	0.3894	0.3894	X, Y, Z
	2	38.0547	-36.4812	-29.6576	37.6489	-36.4140	-29.7303	0.4058	0.0672	-0.0727	0.4177	0.4177	X, Y, Z
	3	36.7072	-47.9836	-26.8385	36.3120	-47.9237	-26.8739	0.3952	0.0599	-0.0354	0.4013	0.4013	X, Y, Z
	4	32.4315	-21.3783	-16.7592	32.2242	-21.2602	-16.8078	0.2073	0.1181	-0.0486	0.2435	0.2435	X, Y, Z
	5	30.9987	-34.8556	-16.3274	30.2807	-34.6852	-16.9443	0.7180	0.1704	-0.6169	0.9618	0.9618	X, Y, Z
	6	31.8198	-48.9196	-16.9045	31.4626	-48.8723	-17.0175	0.3572	0.0473	-0.1130	0.3776	0.3776	X, Y, Z
SIDE PANEL (Y)	7	39.1293	-51.3628	-7.4146	38.7945	-52.3865	-7.4553	0.3348	-1.0237	-0.0407	1.0778	-1.0237	Y
	8	43.3851	-51.3947	-4.0193	42.9724	-52.2662	-4.1747	0.4127	-0.8715	-0.1554	0.9767	-0.8715	Y
	9	39.2217	-50.9470	-2.1436	38.6521	-51.9255	-3.0635	0.5696	-0.9785	-0.9199	1.4588	-0.9785	Y
IMPACT SIDE DOOR (Y)	10	5.5475	-52.3130	-21.5450	5.0921	-52.6390	-21.6535	0.4554	-0.3260	-0.1085	0.5705	-0.3260	Y
	11	17.8910	-53.6978	-19.8802	17.4732	-54.0635	-19.9526	0.4178	-0.3657	-0.0724	0.5599	-0.3657	Y
	12	29.6715	-53.6496	-18.9664	29.2352	-53.9230	-19.0248	0.4363	-0.2734	-0.0584	0.5182	-0.2734	Y
	13	6.9792	-53.3208	-4.0495	6.5027	-53.6554	-4.1741	0.4765	-0.3346	-0.1246	0.5954	-0.3346	Y
	14	17.7894	-54.2565	-3.2005	17.3763	-54.5175	-3.2286	0.4131	-0.2610	-0.0281	0.4895	-0.2610	Y
	15	26.4236	-54.3754	-3.4029	25.9590	-54.5077	-3.5003	0.4646	-0.1323	-0.0974	0.4928	-0.1323	Y
ROOF - (Z)	16	32.9875	-22.3411	-48.8133	32.7050	-22.3722	-49.0719	0.2825	-0.0311	-0.2586	0.3842	-0.2586	Z
	17	32.7282	-28.1524	-48.8578	32.5696	-28.1776	-49.1229	0.1586	-0.0252	-0.2651	0.3099	-0.2651	Z
	18	31.8363	-34.4850	-48.8686	31.5901	-34.4977	-49.0980	0.2462	-0.0127	-0.2294	0.3367	-0.2294	Z
	19	31.1280	-39.3124	-48.8838	30.8642	-39.3719	-49.0924	0.2638	-0.0595	-0.2086	0.3415	-0.2086	Z
	20	30.7374	-45.0210	-48.9260	30.4445	-45.0398	-49.1226	0.2929	-0.0188	-0.1966	0.3533	-0.1966	Z
	21	13.4937	-21.0085	-53.6706	13.1325	-21.1127	-53.7495	0.3612	-0.1042	-0.0789	0.3841	-0.0789	Z
	22	12.8980	-26.2179	-53.8214	12.5348	-26.2158	-53.8950	0.3632	0.0021	-0.0736	0.3706	-0.0736	Z
	23	12.0633	-33.6800	-52.1720	11.7331	-33.6898	-52.2096	0.3302	-0.0098	-0.0376	0.3325	-0.0376	Z
	24	12.5062	-39.0892	-53.4804	12.0892	-39.1101	-53.4991	0.4170	-0.0209	-0.0187	0.4179	-0.0187	Z
	25	11.9186	-45.6954	-52.5364	11.5460	-45.7202	-52.5299	0.3726	-0.0248	0.0065	0.3735	0.0065	Z
	26	-1.4420	-20.4830	-53.2674	-1.7917	-20.5113	-53.2293	0.3497	-0.0283	0.0381	0.3529	0.0381	Z
	27	-1.6659	-26.1373	-53.2575	-2.0637	-26.1005	-53.1949	0.3978	0.0368	0.0626	0.4044	0.0626	Z
	28	-2.3853	-33.4548	-51.8255	-2.7013	-33.4226	-51.7233	0.3160	0.0322	0.1022	0.3337	0.1022	Z
	29	-2.2617	-39.1852	-53.2369	-2.6008	-39.1674	-53.1072	0.3391	0.0178	0.1297	0.3635	0.1297	Z
	30	-2.5862	-45.4409	-52.3250	-2.9500	-45.4483	-52.1407	0.3638	-0.0074	0.1843	0.4079	0.1843	Z
A-PILLAR Maximum (X, Y, Z)	31	38.2176	-51.6303	-28.7621	37.8841	-51.6515	-28.8777	0.3335	-0.0212	-0.1156	0.3536	0.3335	X
	32	36.7326	-51.4672	-32.7271	36.3943	-51.4579	-32.8845	0.3383	0.0093	-0.1574	0.3732	0.3384	X, Y
	33	34.6977	-50.8955	-36.2714	34.3280	-50.8911	-36.4099	0.3697	0.0044	-0.1385	0.3948	0.3697	X, Y
	34	32.8886	-50.1780	-39.7623	32.5582	-50.1776	-39.9171	0.3304	0.0004	-0.1548	0.3649	0.3304	X, Y
	35	31.1304	-49.5964	-43.1674	30.8214	-49.6369	-43.2431	0.3090	-0.0405	-0.0757	0.3207	0.3090	X
	36	29.4521	-48.2865	-46.4629	29.1456	-48.3352	-46.5574	0.3065	-0.0487	-0.0945	0.3244	0.3065	X
A-PILLAR Lateral (Y)	31	38.2176	-51.6303	-28.7621	37.8841	-51.6515	-28.8777	0.3335	-0.0212	-0.1156	0.3536	-0.0212	Y
	32	36.7326	-51.4672	-32.7271	36.3943	-51.4579	-32.8845	0.3383	0.0093	-0.1574	0.3732	0.0093	Y
	33	34.6977	-50.8955	-36.2714	34.3280	-50.8911	-36.4099	0.3697	0.0044	-0.1385	0.3948	0.0044	Y
	34	32.8886	-50.1780	-39.7623	32.5582	-50.1776	-39.9171	0.3304	0.0004	-0.1548	0.3649	0.0004	Y
	35	31.1304	-49.5964	-43.1674	30.8214	-49.6369	-43.2431	0.3090	-0.0405	-0.0757	0.3207	-0.0405	Y
	36	29.4521	-48.2865	-46.4629	29.1456	-48.3352	-46.5574	0.3065	-0.0487	-0.0945	0.3244	-0.0487	Y
B-PILLAR Maximum (X, Y, Z)	37	-1.1601	-49.1756	-46.2650	-1.4809	-49.1496	-46.0822	0.3208	0.0260	0.1828	0.3701	0.3701	X, Y, Z
	38	-8.0141	-49.6163	-41.8637	-8.2772	-49.5968	-41.5616	0.2631	0.0195	0.3021	0.4011	0.4011	X, Y, Z
	39	-1.7519	-50.7756	-35.3511	-1.9761	-50.6986	-35.1441	0.2242	0.0770	0.2070	0.3147	0.3147	X, Y, Z
	40	-7.7084	-51.1803	-28.1724	-7.8543	-51.0832	-27.9668	0.1459	0.0971	0.2056	0.2702	0.2702	X, Y, Z
B-PILLAR Lateral (Y)	37	-1.1601	-49.1756	-46.2650	-1.4809	-49.1496	-46.0822	0.3208	0.0260	0.1828	0.3701	0.0260	Y
	38	-8.0141	-49.6163	-41.8637	-8.2772	-49.5968	-41.5616	0.2631	0.0195	0.3021	0.4011	0.0195	Y
	39	-1.7519	-50.7756	-35.3511	-1.9761	-50.6986	-35.1441	0.2242	0.0770	0.2070	0.3147	0.0770	Y
	40	-7.7084	-51.1803	-28.1724	-7.8543	-51.0832	-27.9668	0.1459	0.0971	0.2056	0.2702	0.0971	Y

[A] Positive values denote deformation as inward toward the occupant compartment, negative values denote deformations outward away from the occupant compartment.

[B] Crush calculations that use multiple directional components will disregard components that are negative and only include positive values where the component is deforming inward toward the occupant compartment.

[C] Direction for Crush column denotes which directions are included in the crush calculations. If "NA" then no intrusion is recorded, and Crush will be 0.

Figure F-20. Occupant Compartment Deformation Data - Set 2, Test No. STBR-4

Appendix G. Accelerometer and Rate Transducer Data Plots, Test No. STBR-1

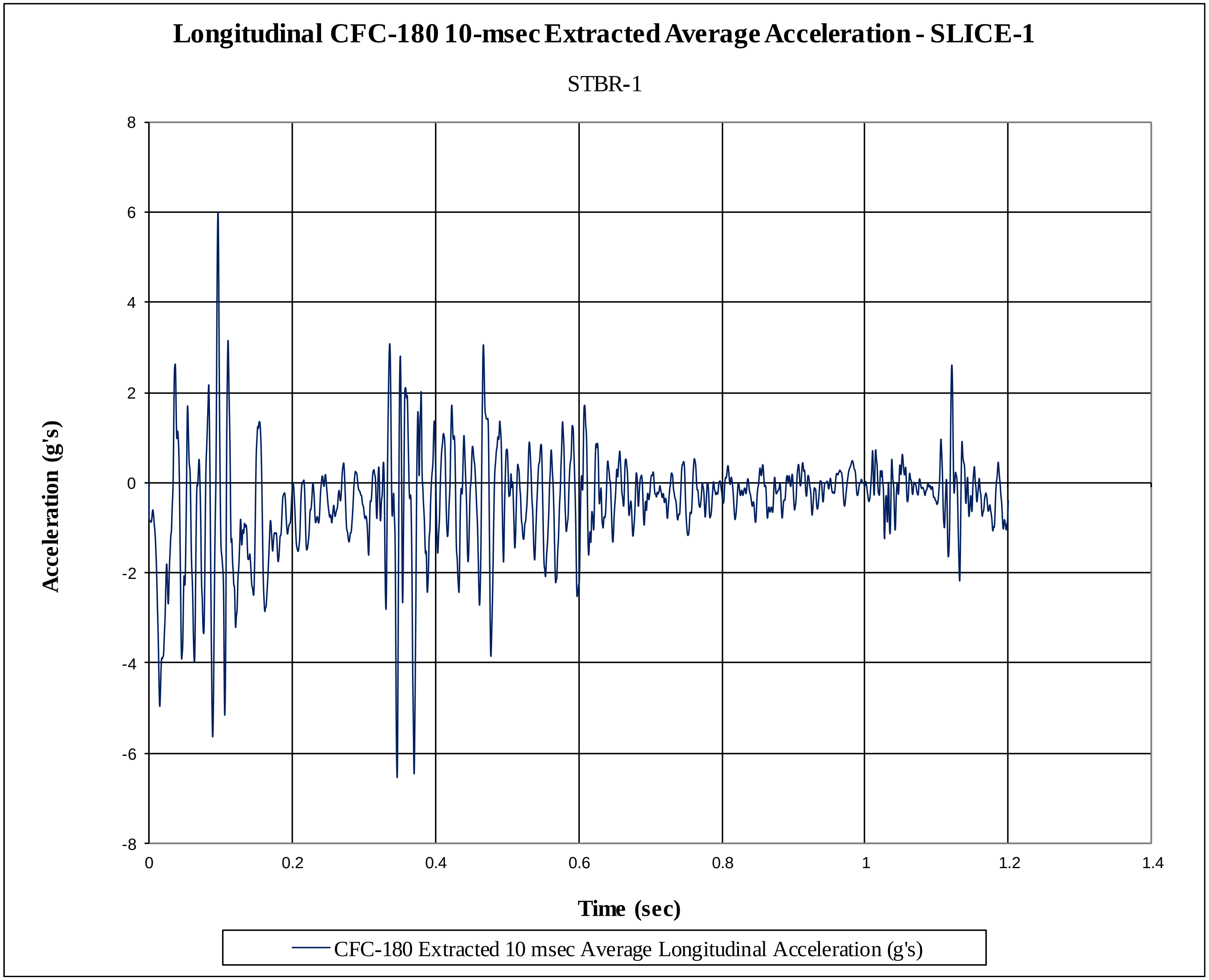


Figure G-1. 10-ms Average Longitudinal Deceleration (SLICE-1), Test No. STBR-1

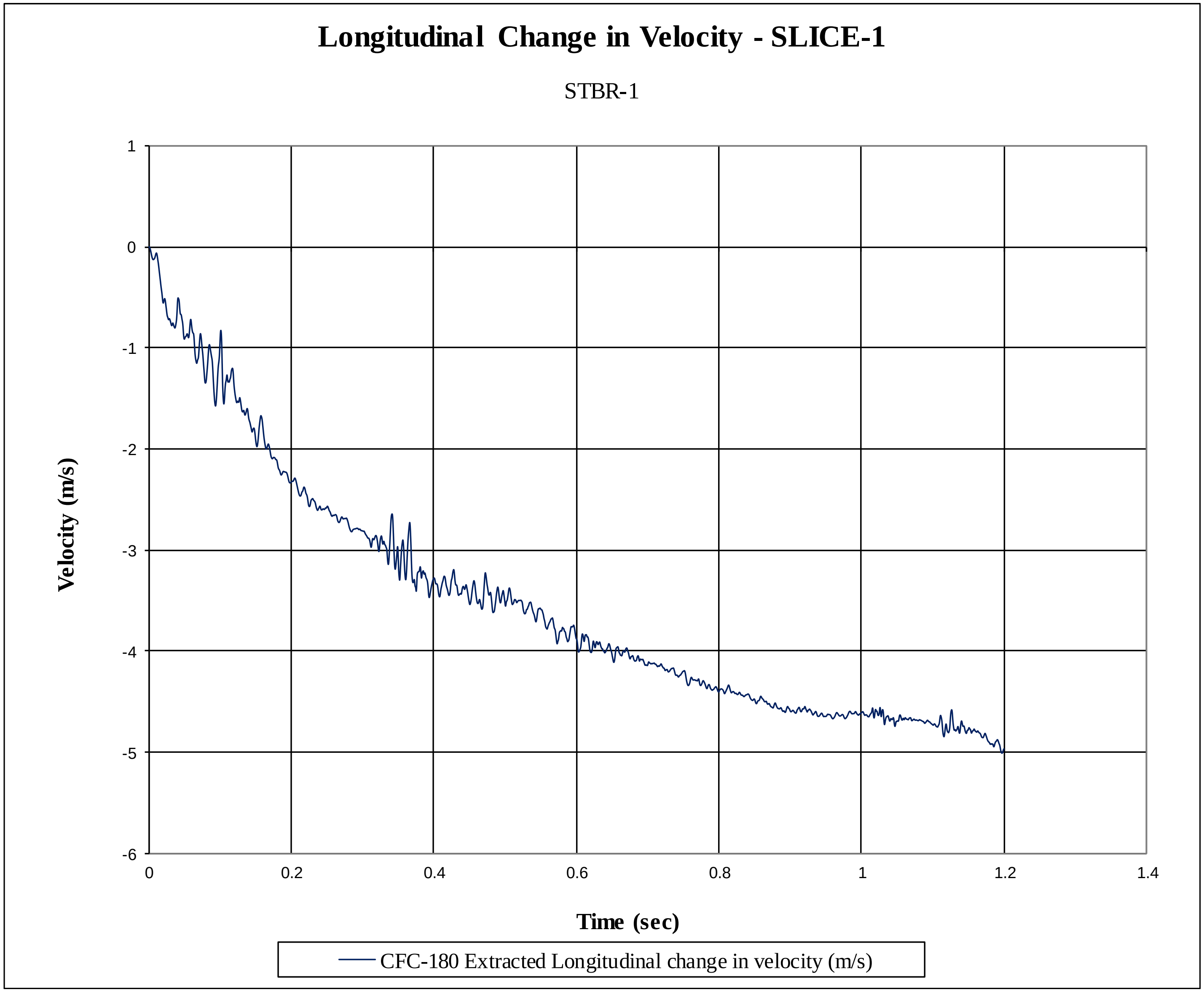


Figure G-2. Longitudinal Occupant Impact Velocity (SLICE-1), Test No. STBR-1

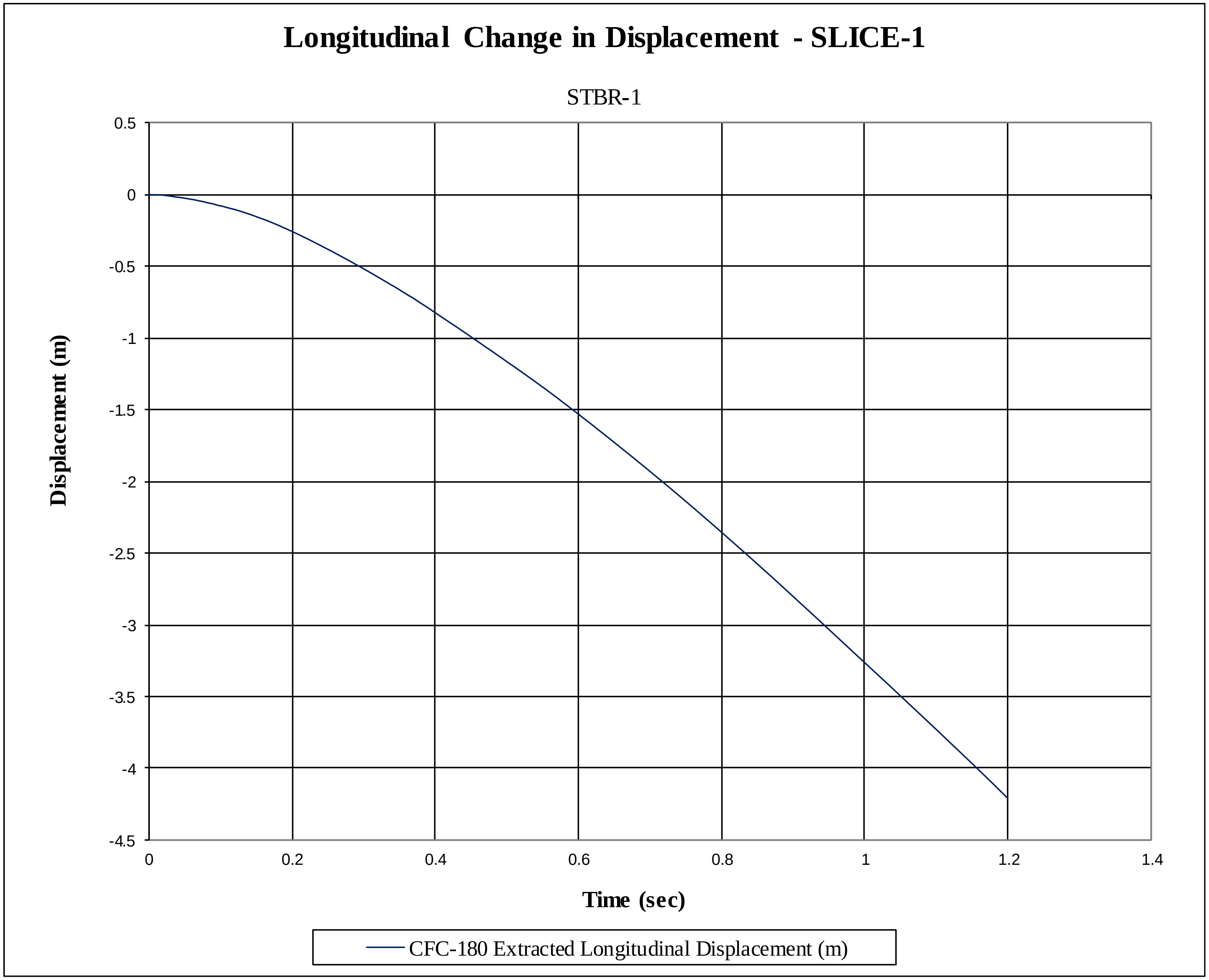


Figure G-3. Longitudinal Occupant Displacement (SLICE-1), Test No. STBR-1

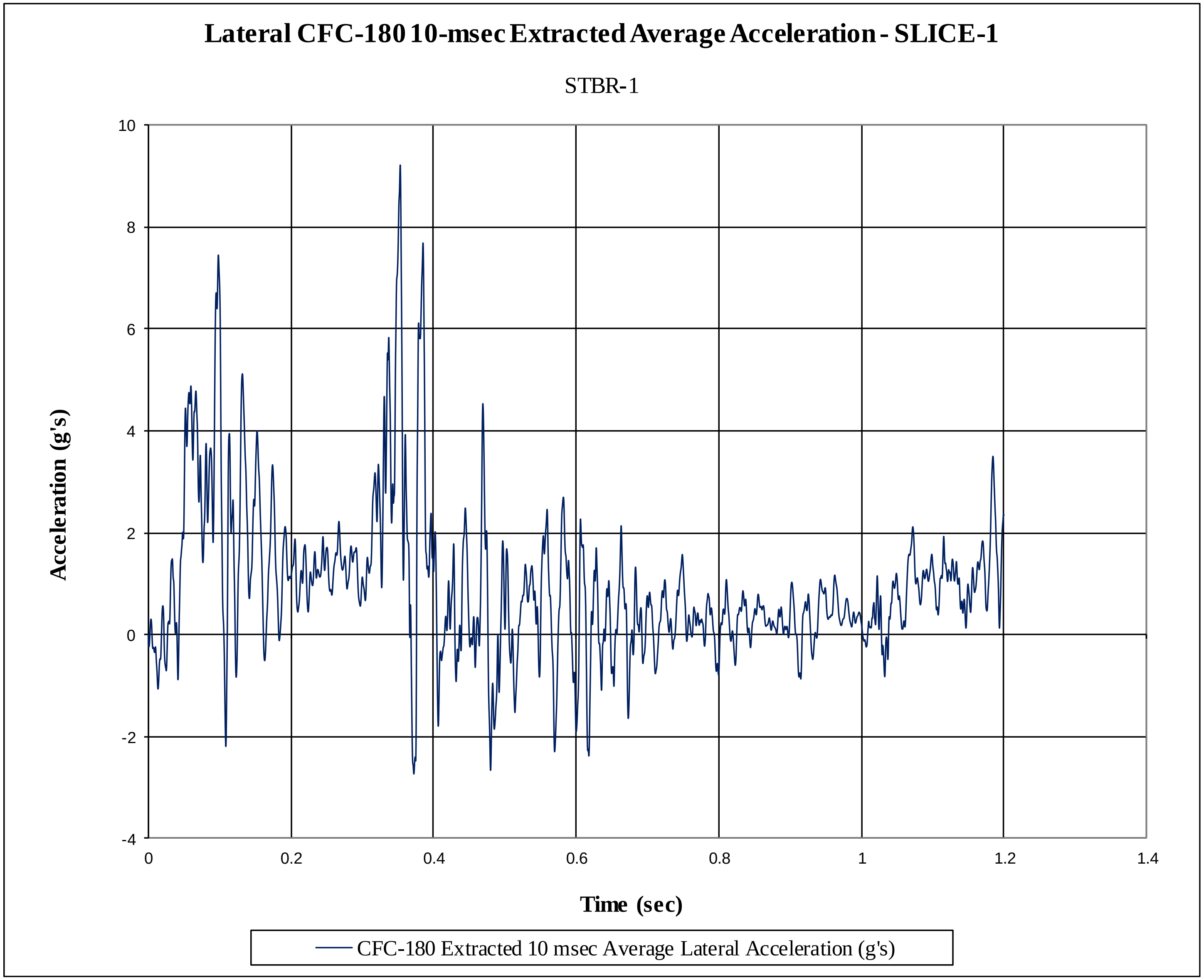


Figure G-4. 10-ms Average Lateral Deceleration (SLICE-1), Test No. STBR-1

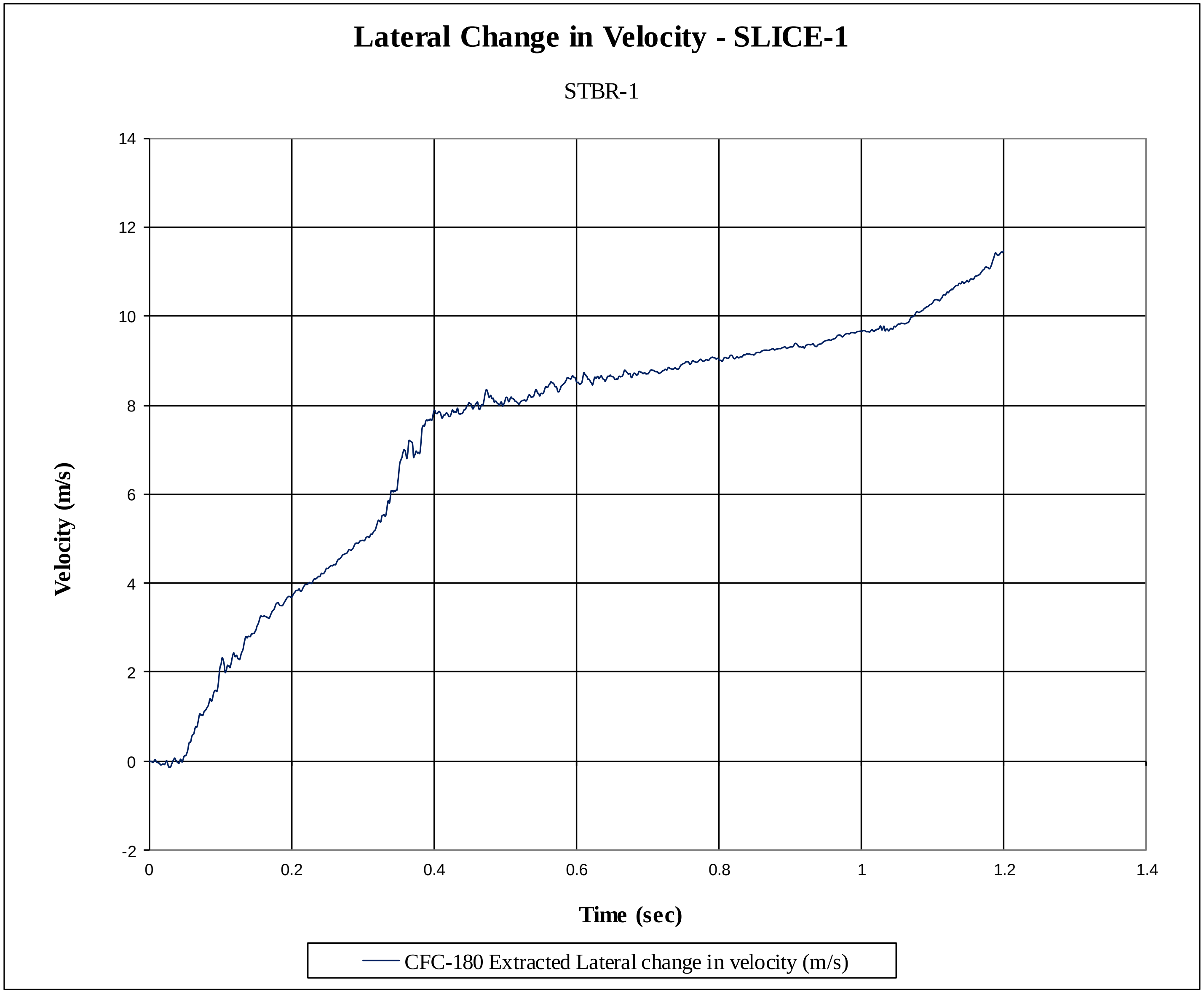


Figure G-5. Lateral Occupant Impact Velocity (SLICE-1), Test No. STBR-1

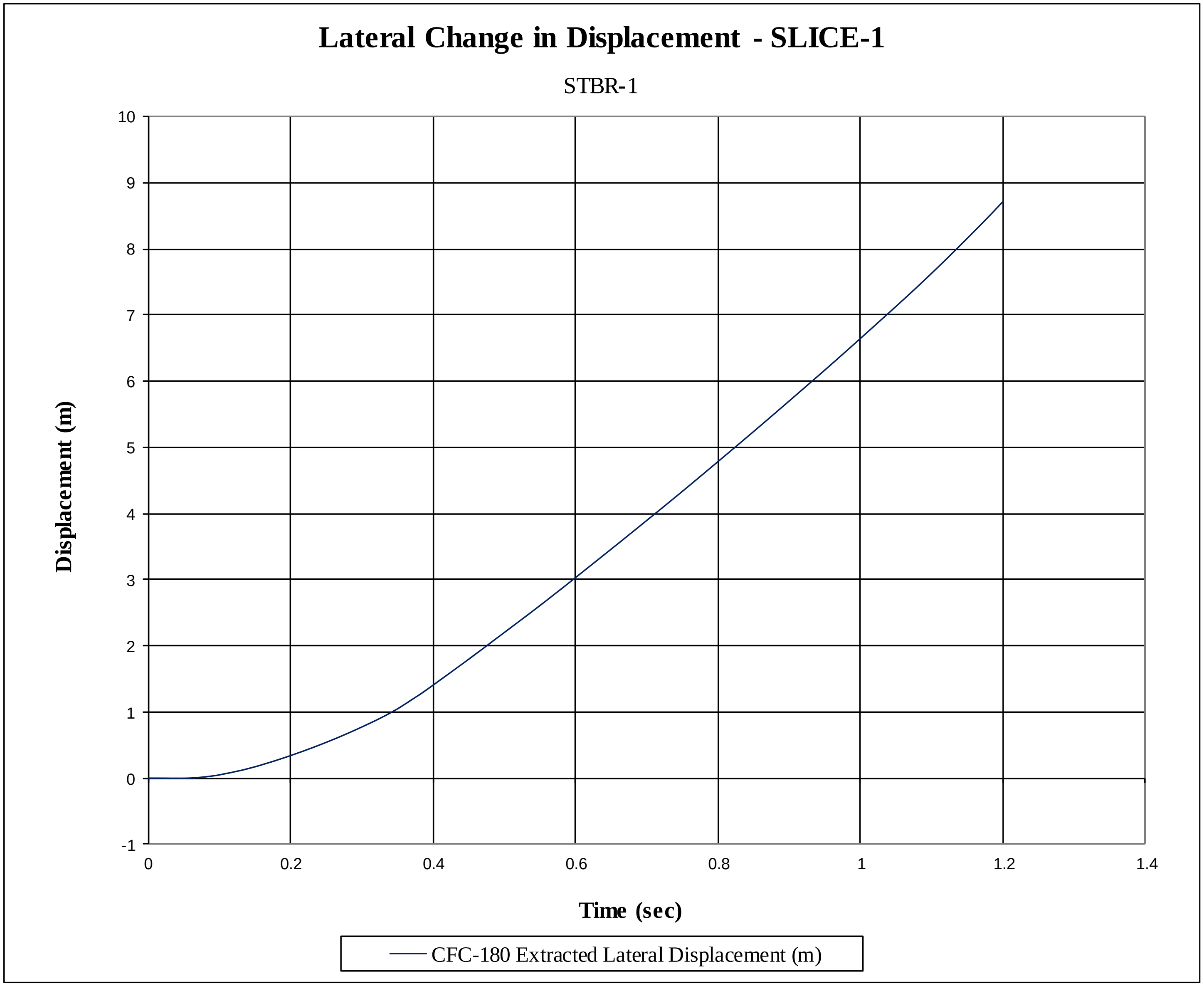


Figure G-6. Lateral Occupant Displacement (SLICE-1), Test No. STBR-1

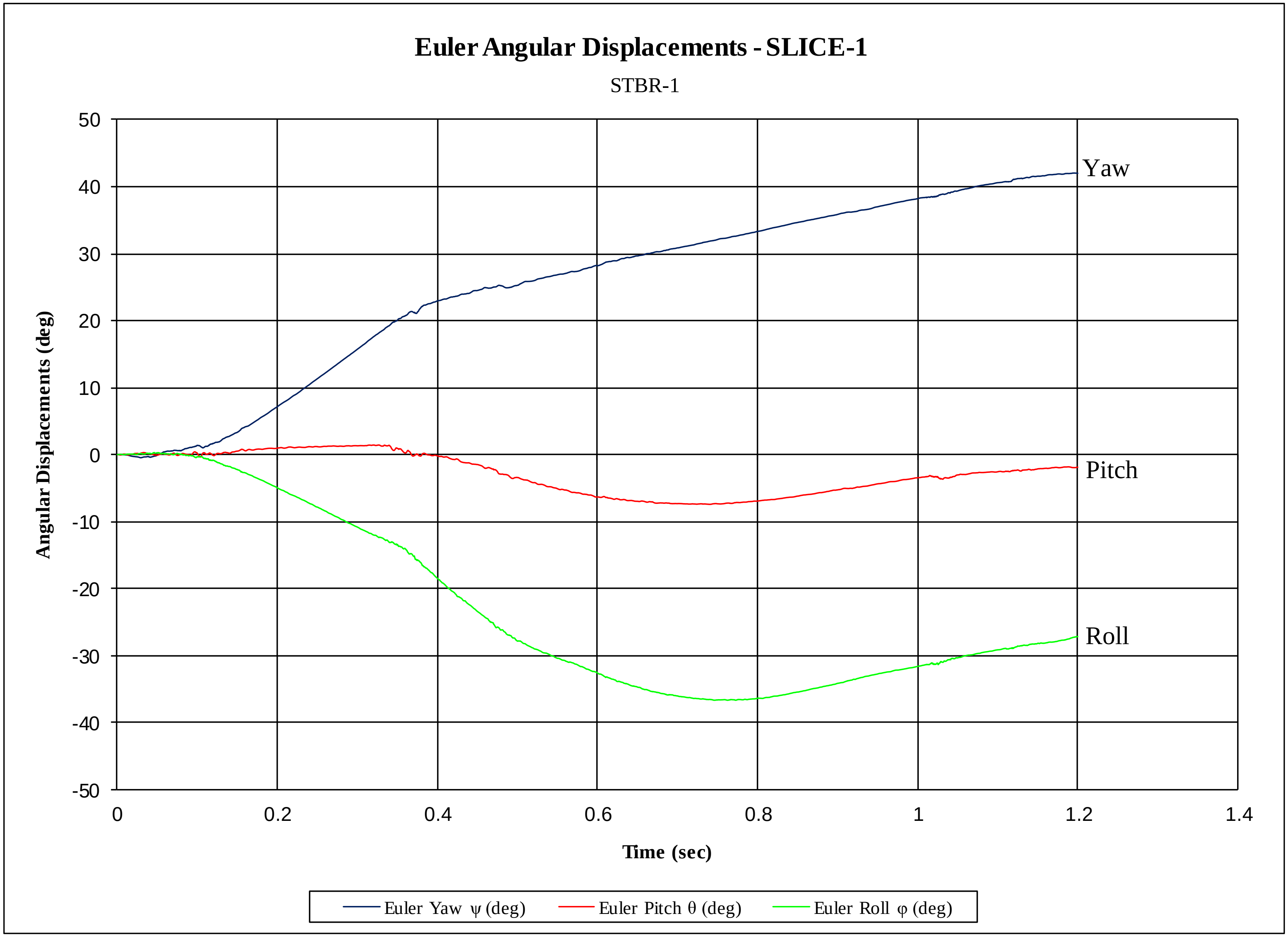


Figure G-7. Vehicle Angular Displacement (SLICE-1), Test No. STBR-1

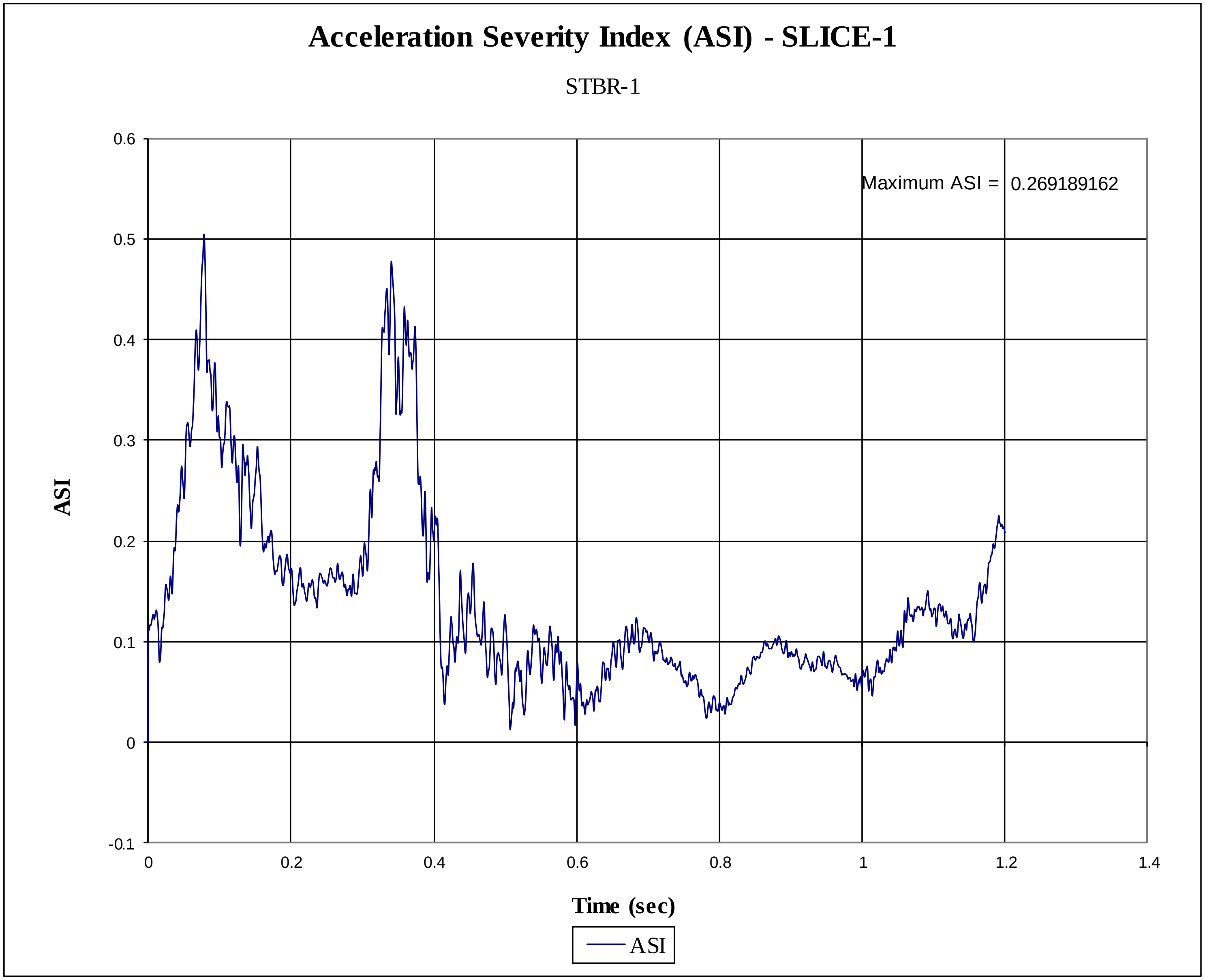


Figure G-8. Acceleration Severity Index (SLICE-1), Test No. STBR-1

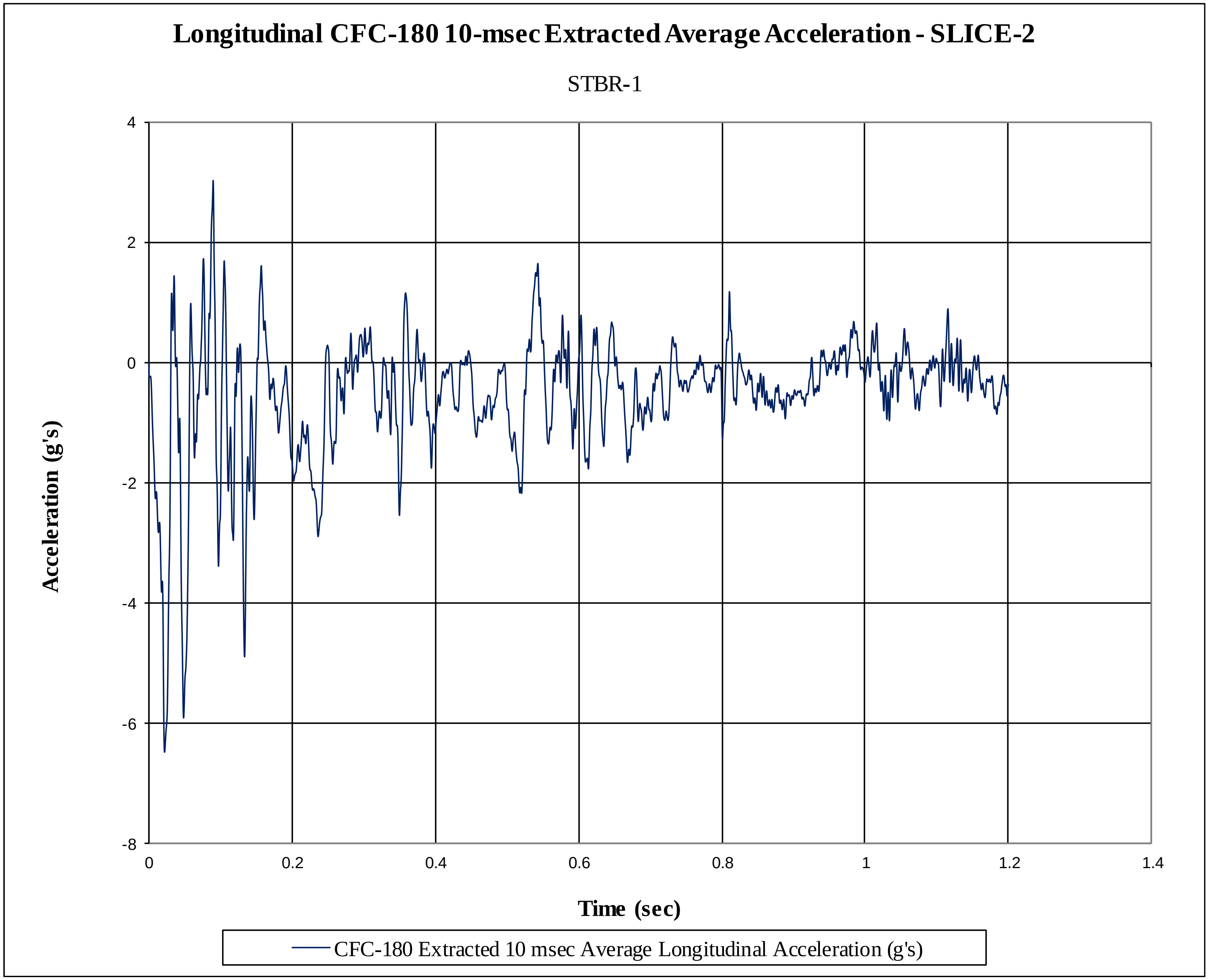


Figure G-9. 10-ms Average Longitudinal Deceleration (SLICE-2), Test No. STBR-1

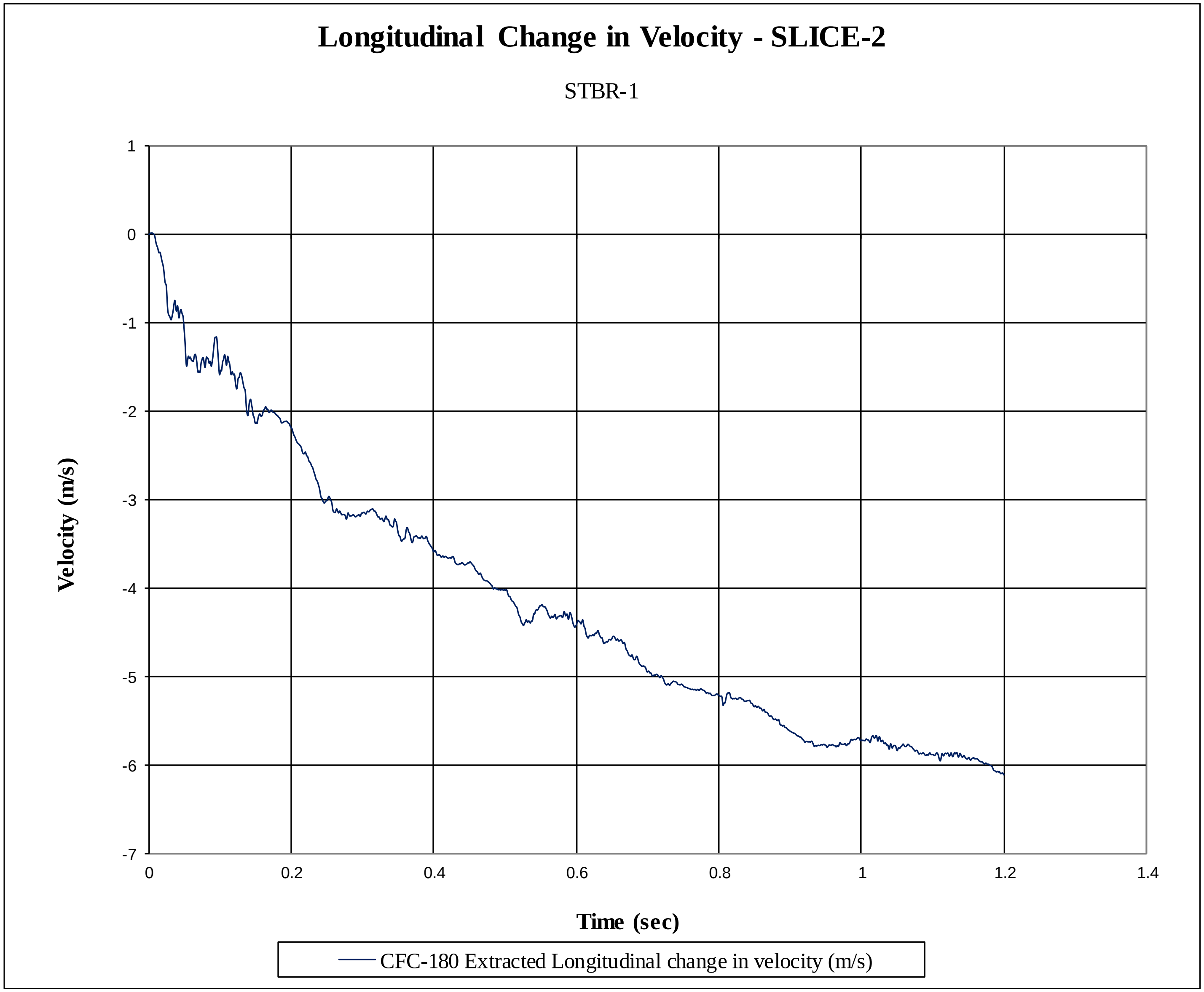


Figure G-10. Longitudinal Occupant Impact Velocity (SLICE-2), Test No. STBR-1

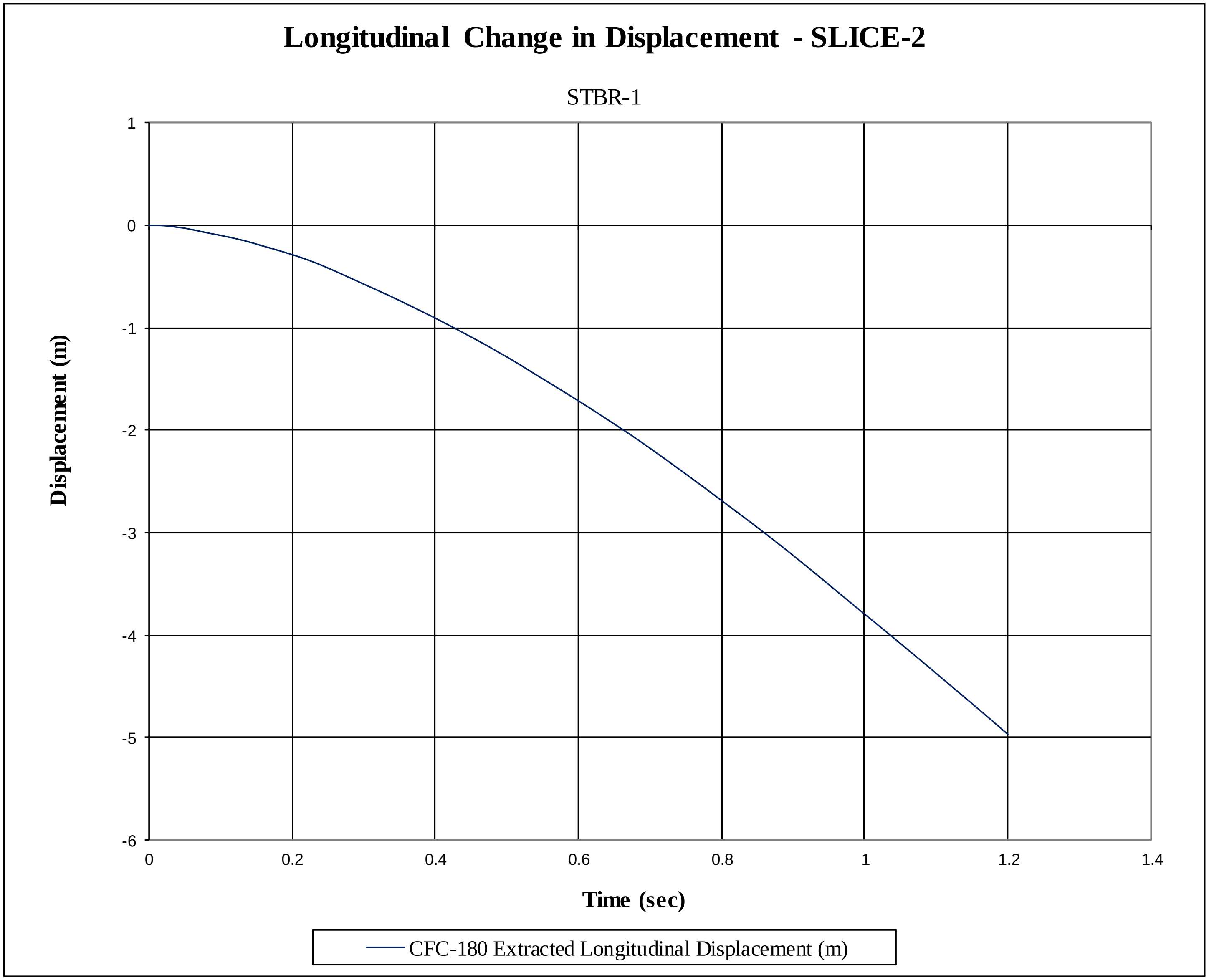


Figure G-11. Longitudinal Occupant Displacement (SLICE-2), Test No. STBR-1

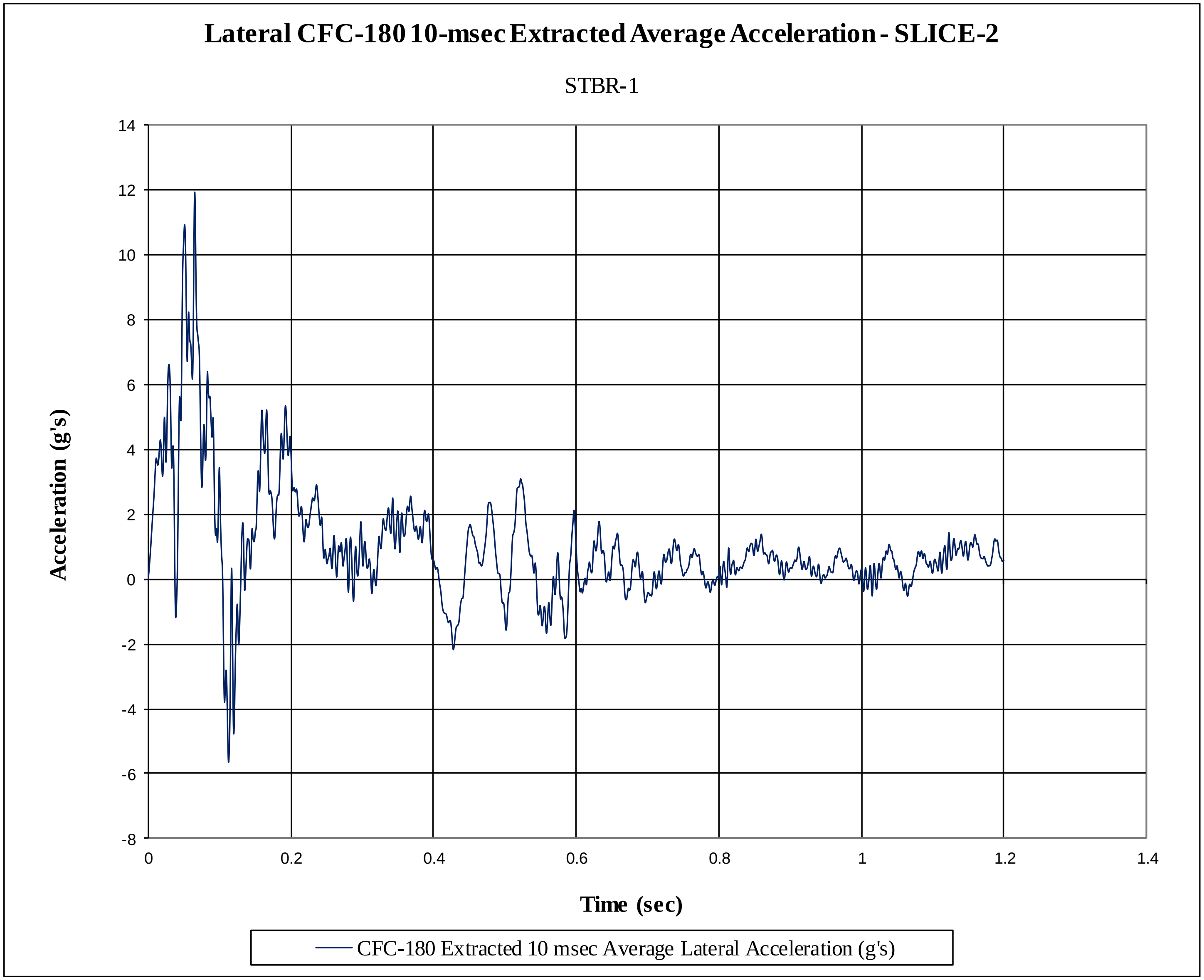


Figure G-12. 10-ms Average Lateral Deceleration (SLICE-2), Test No. STBR-1

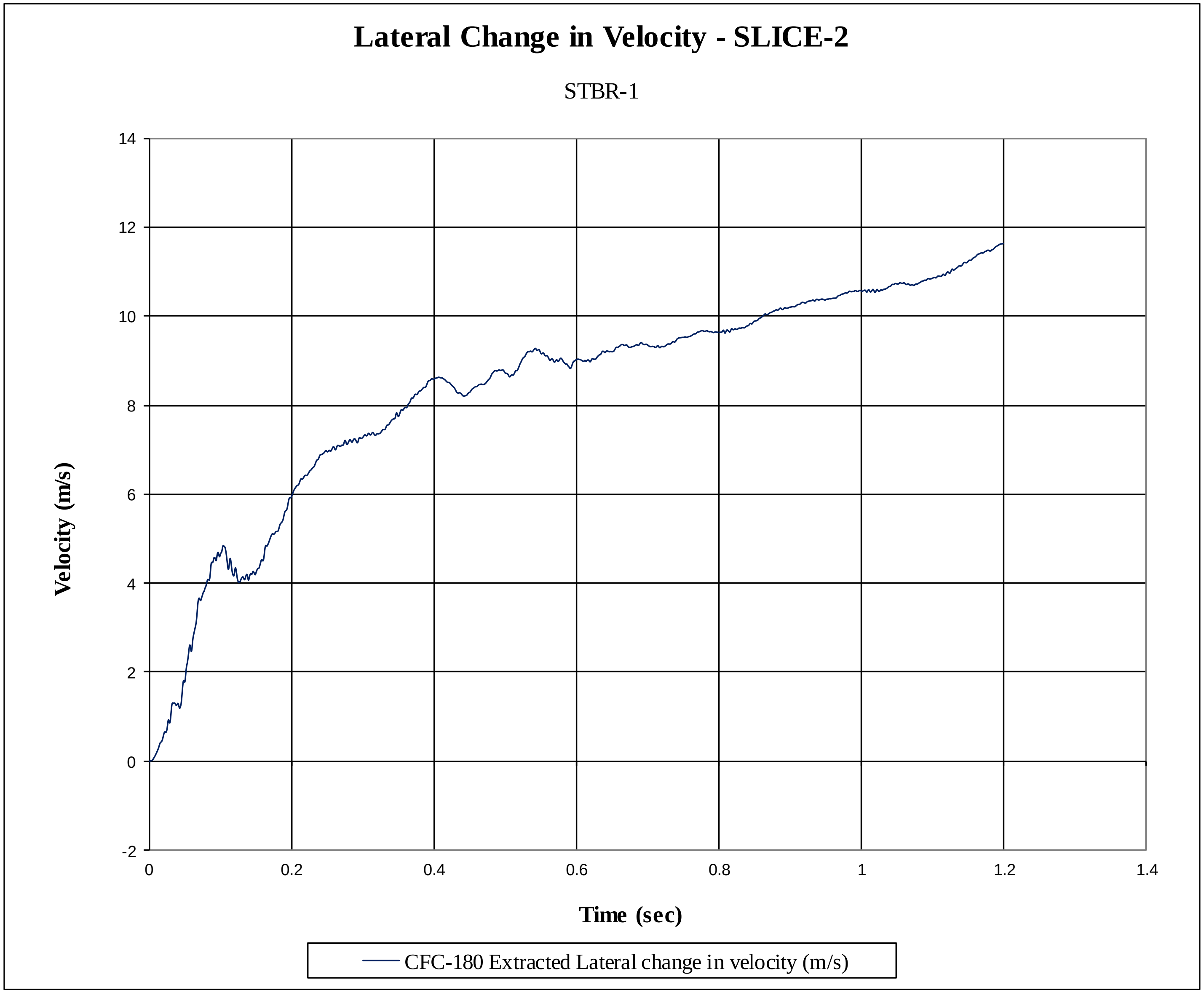


Figure G-13. Lateral Occupant Impact Velocity (SLICE-2), Test No. STBR-1

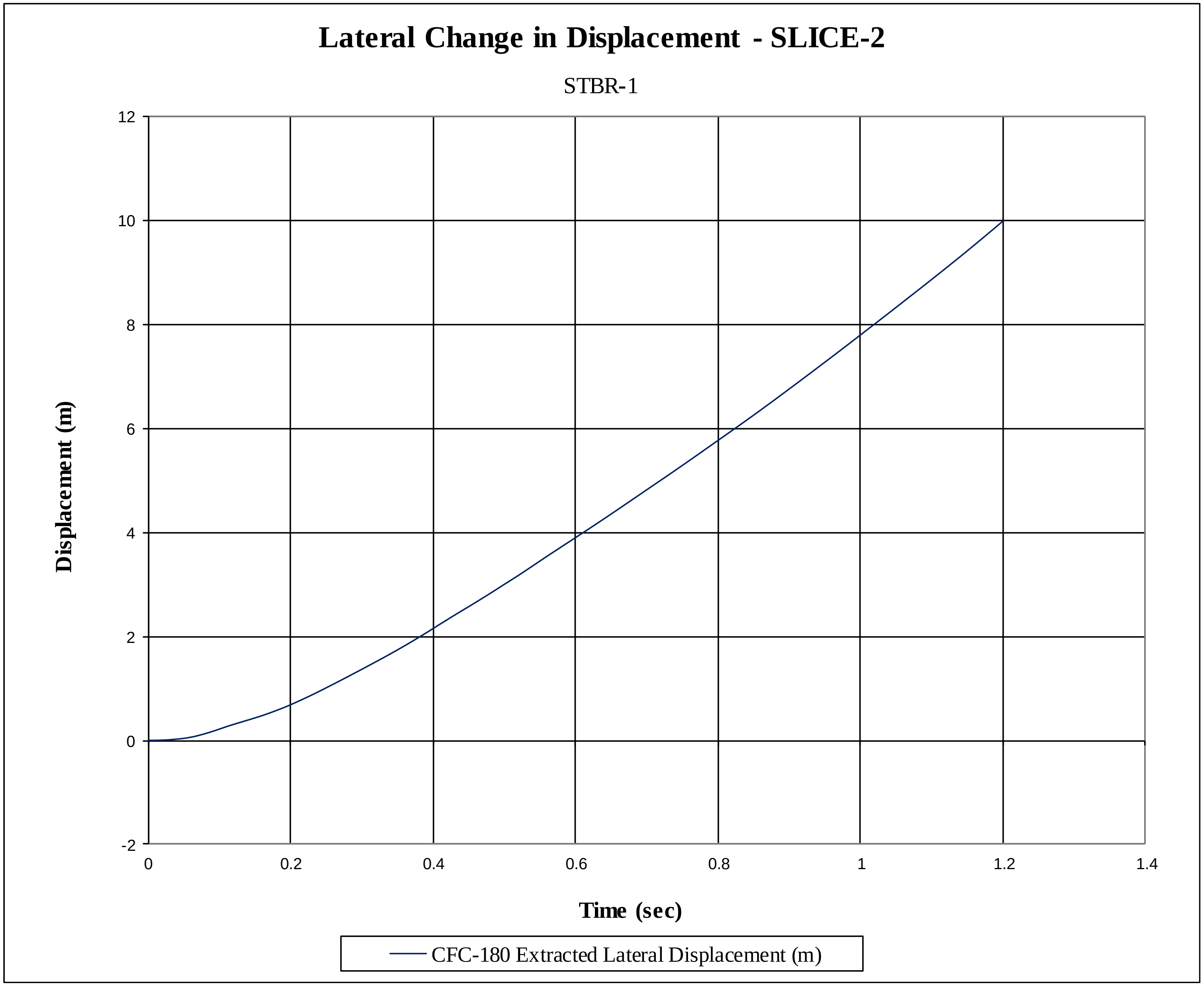


Figure G-14. Lateral Occupant Displacement (SLICE-2), Test No. STBR-1

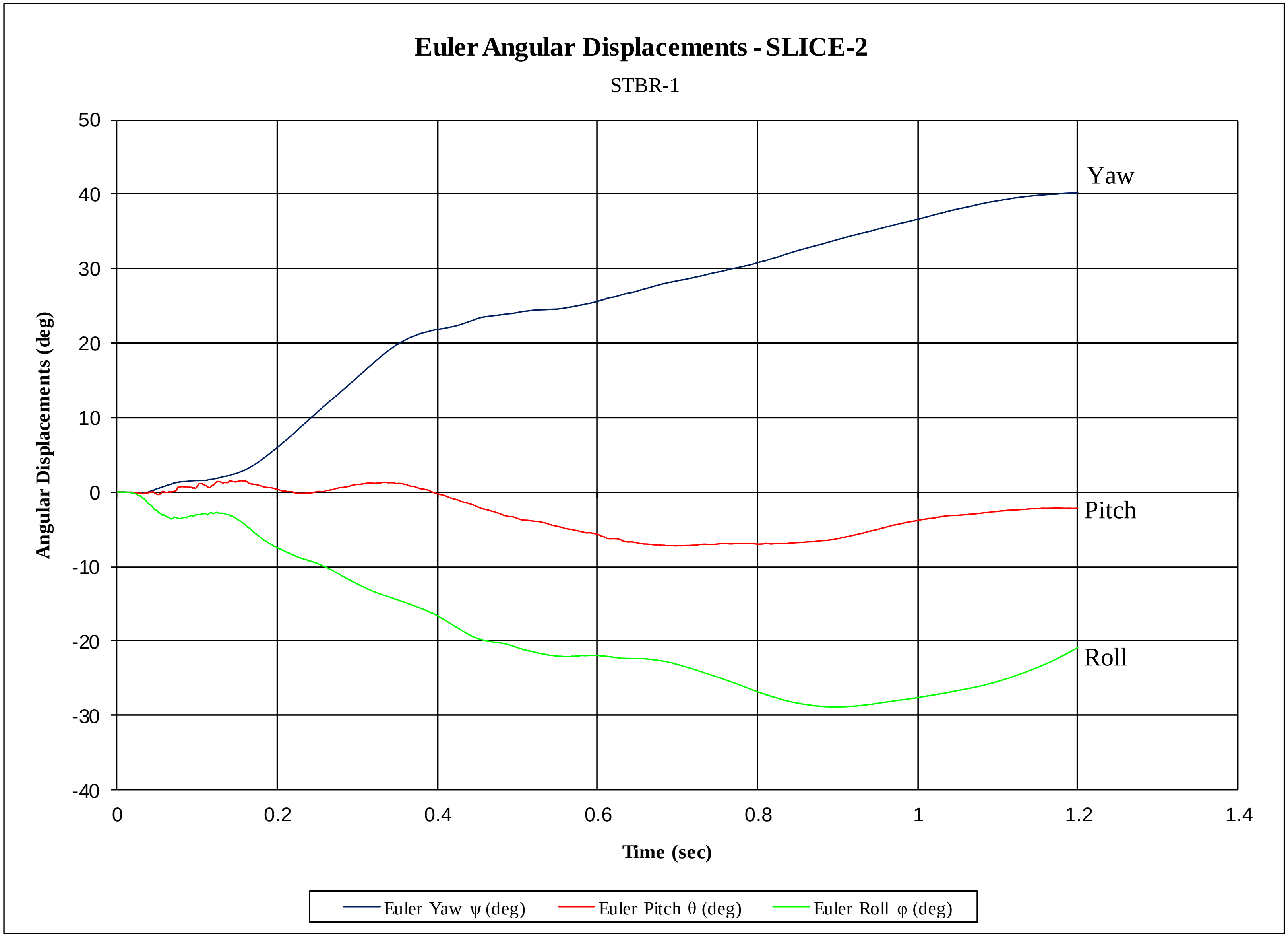


Figure G-15. Vehicle Angular Displacement (SLICE-2), Test No. STBR-1

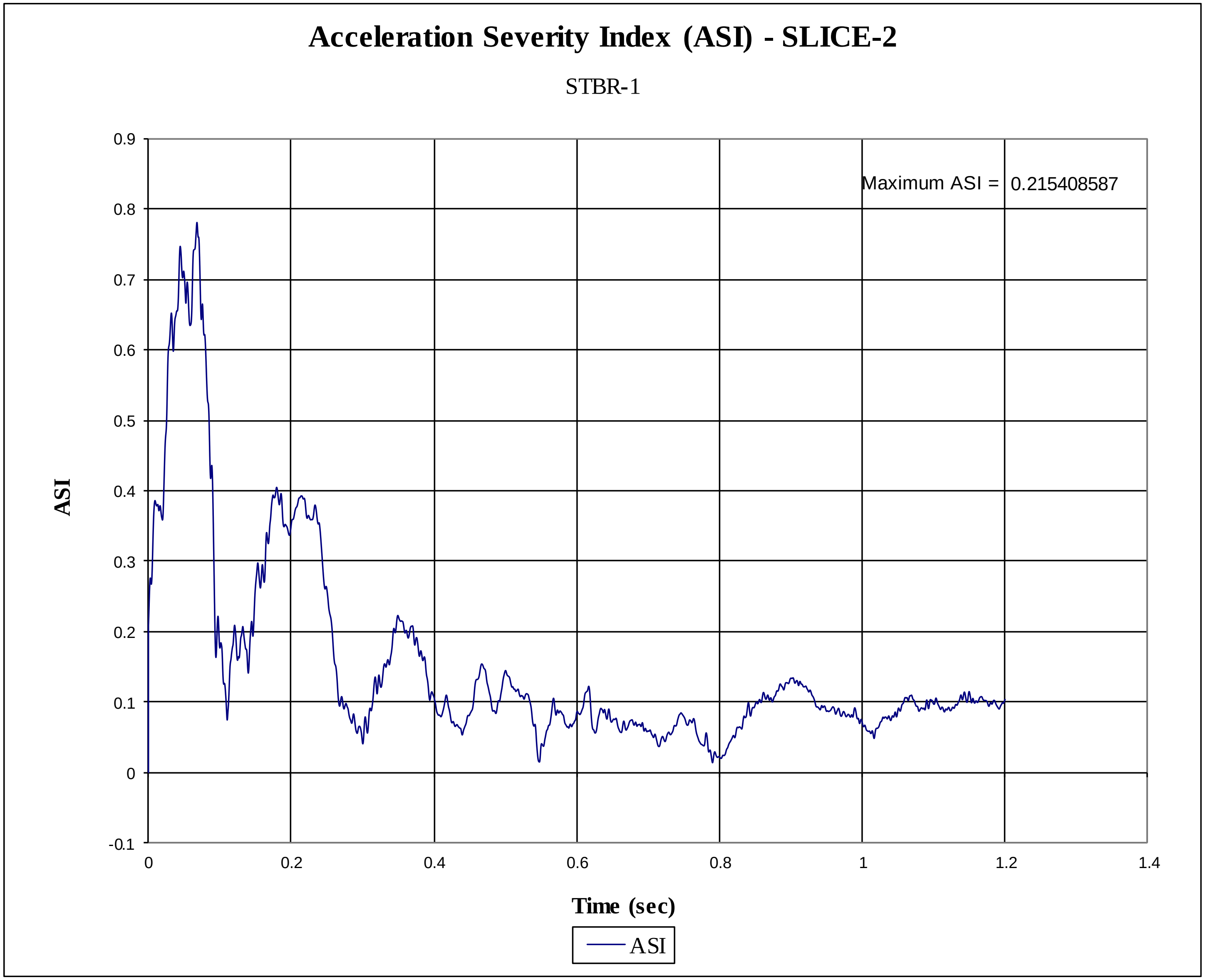


Figure G-16. Acceleration Severity Index (SLICE-2), Test No. STBR-1

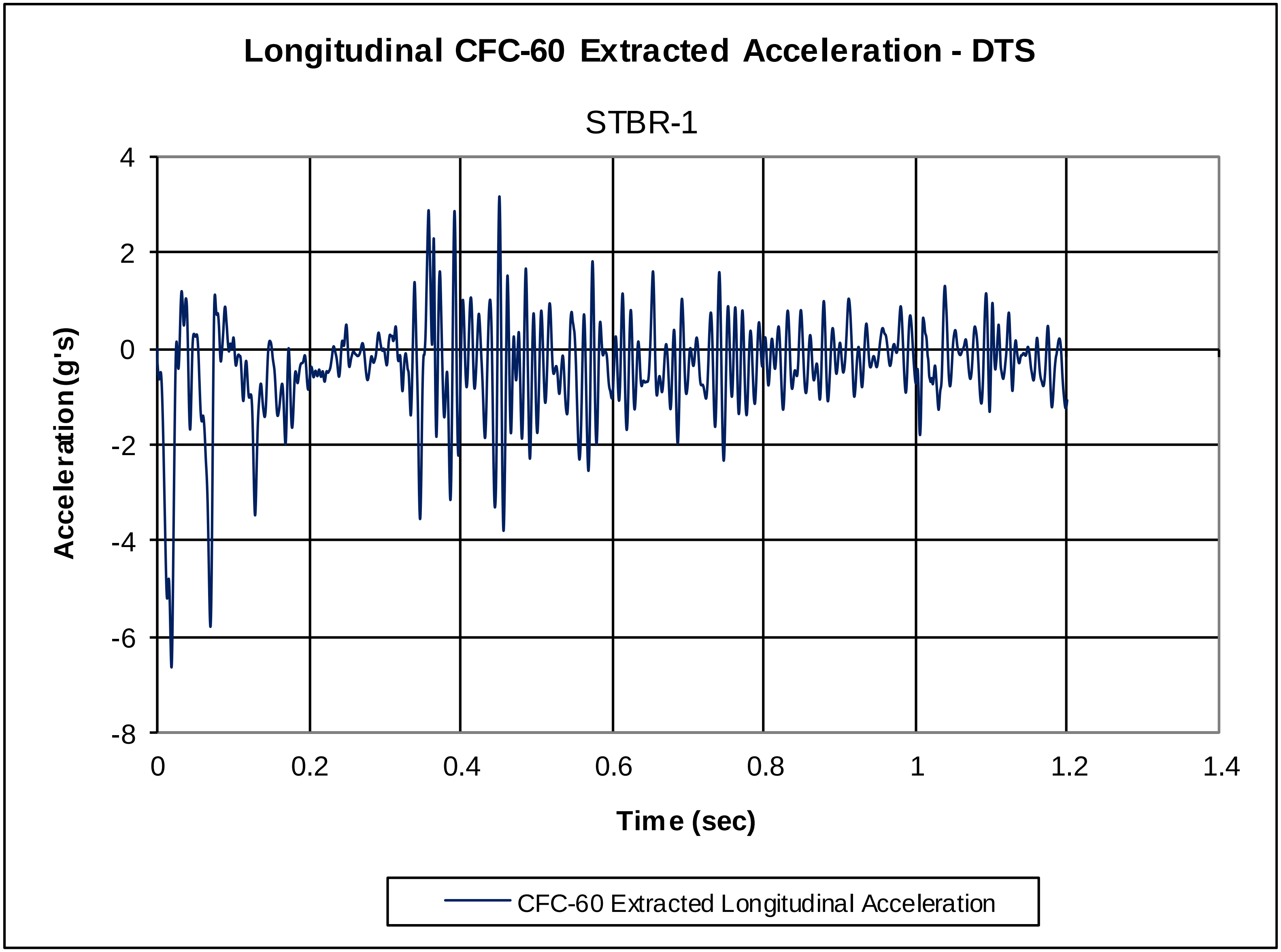


Figure G-17. Longitudinal CFC-60 Extracted Deceleration (DTS), Test No. STBR-1

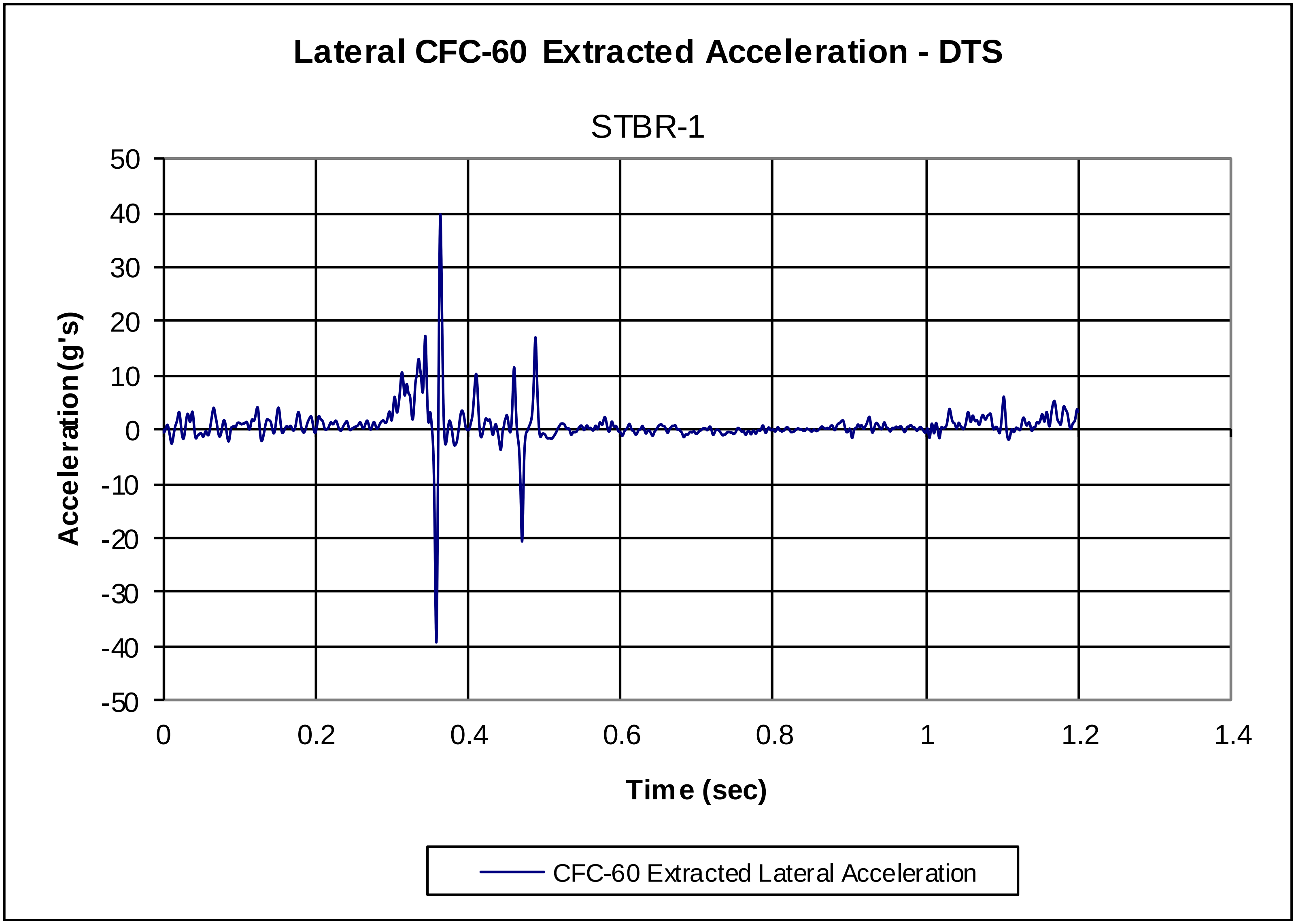


Figure G-18. Lateral CFC-60 Extracted Deceleration (DTS), Test No. STBR-1

Appendix H. Accelerometer and Rate Transducer Data Plots, Test No. STBR-2

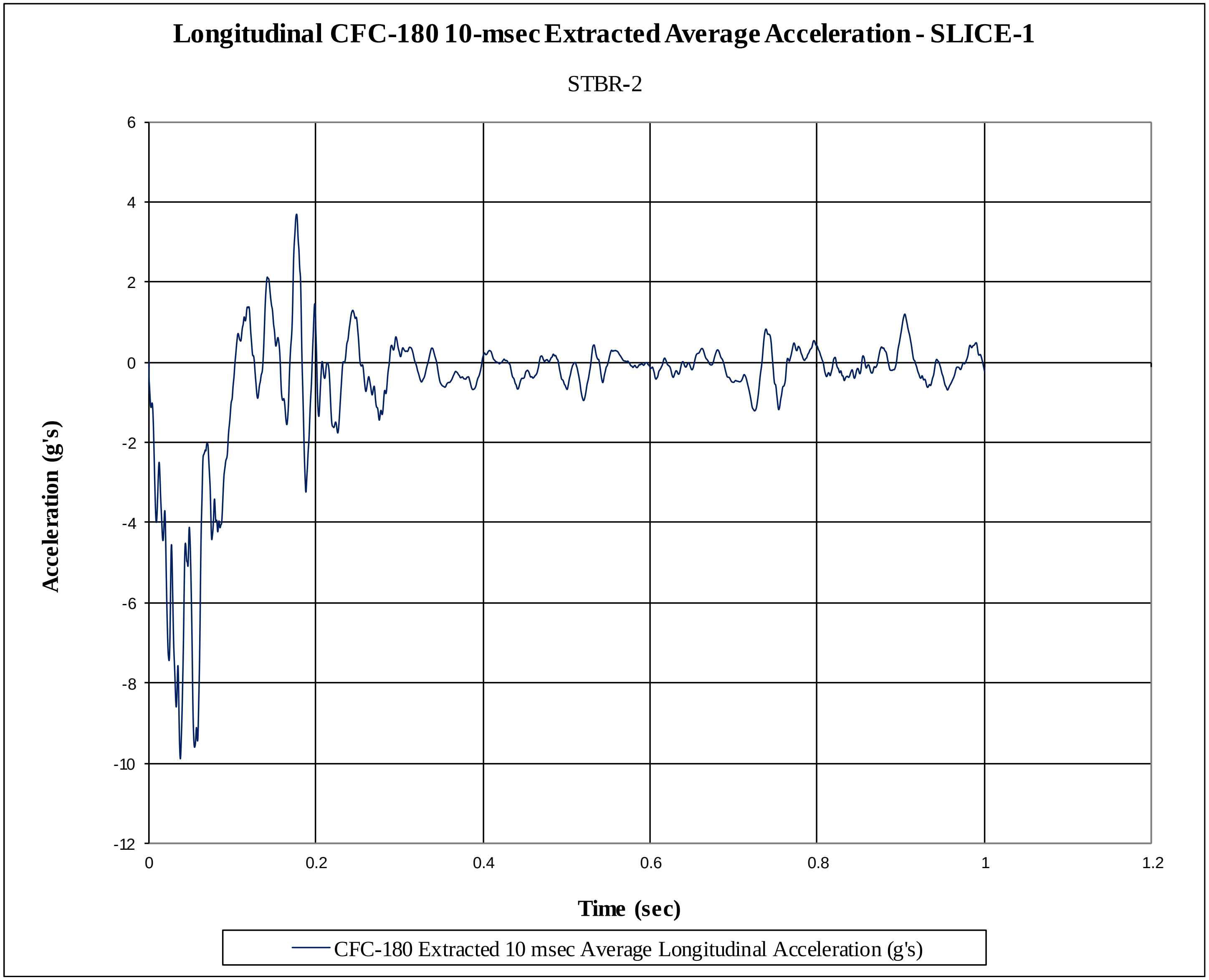


Figure H-1. 10-ms Average Longitudinal Deceleration (SLICE-1), Test No. STBR-2

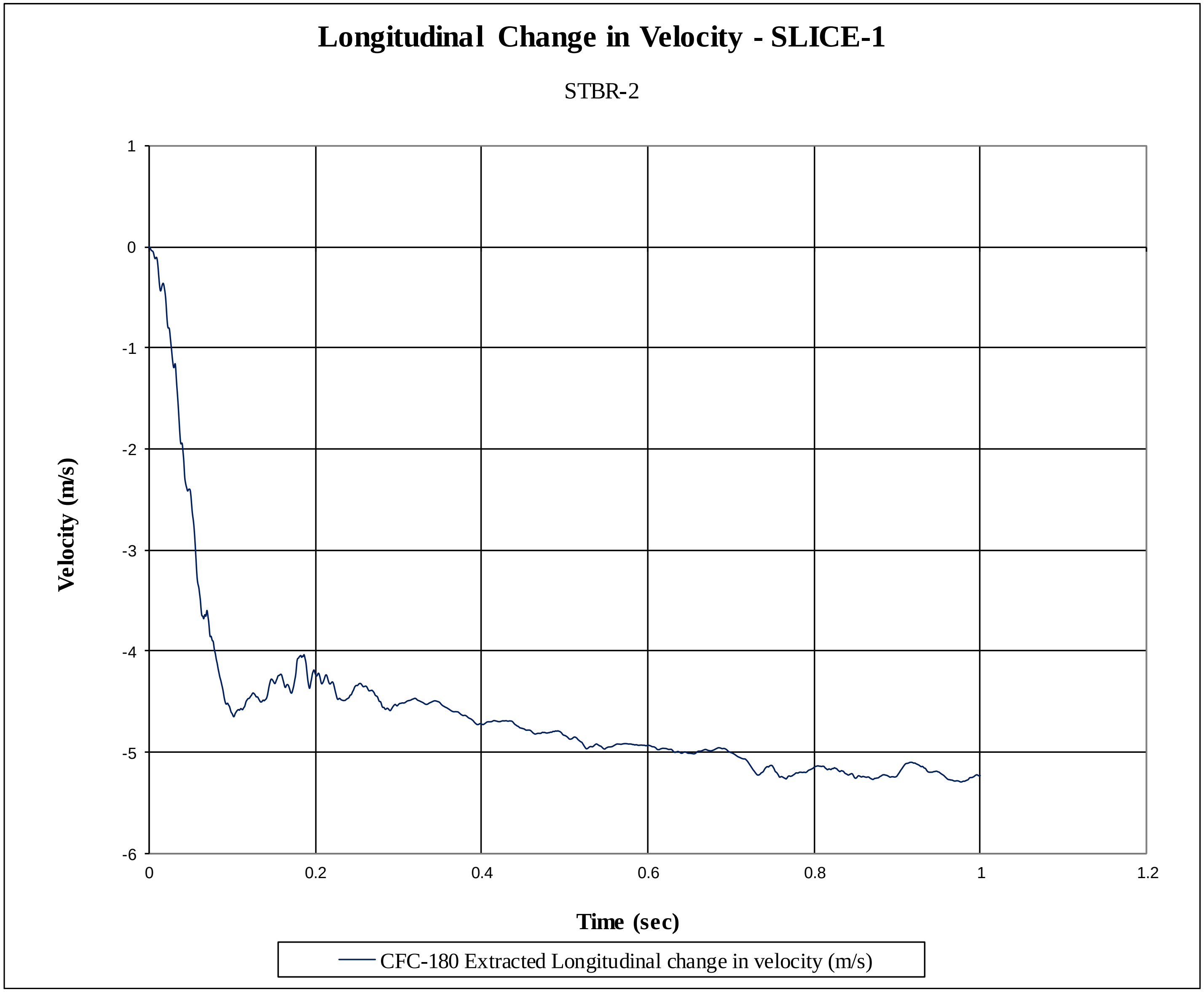


Figure H-2. Longitudinal Occupant Impact Velocity (SLICE-1), Test No. STBR-2

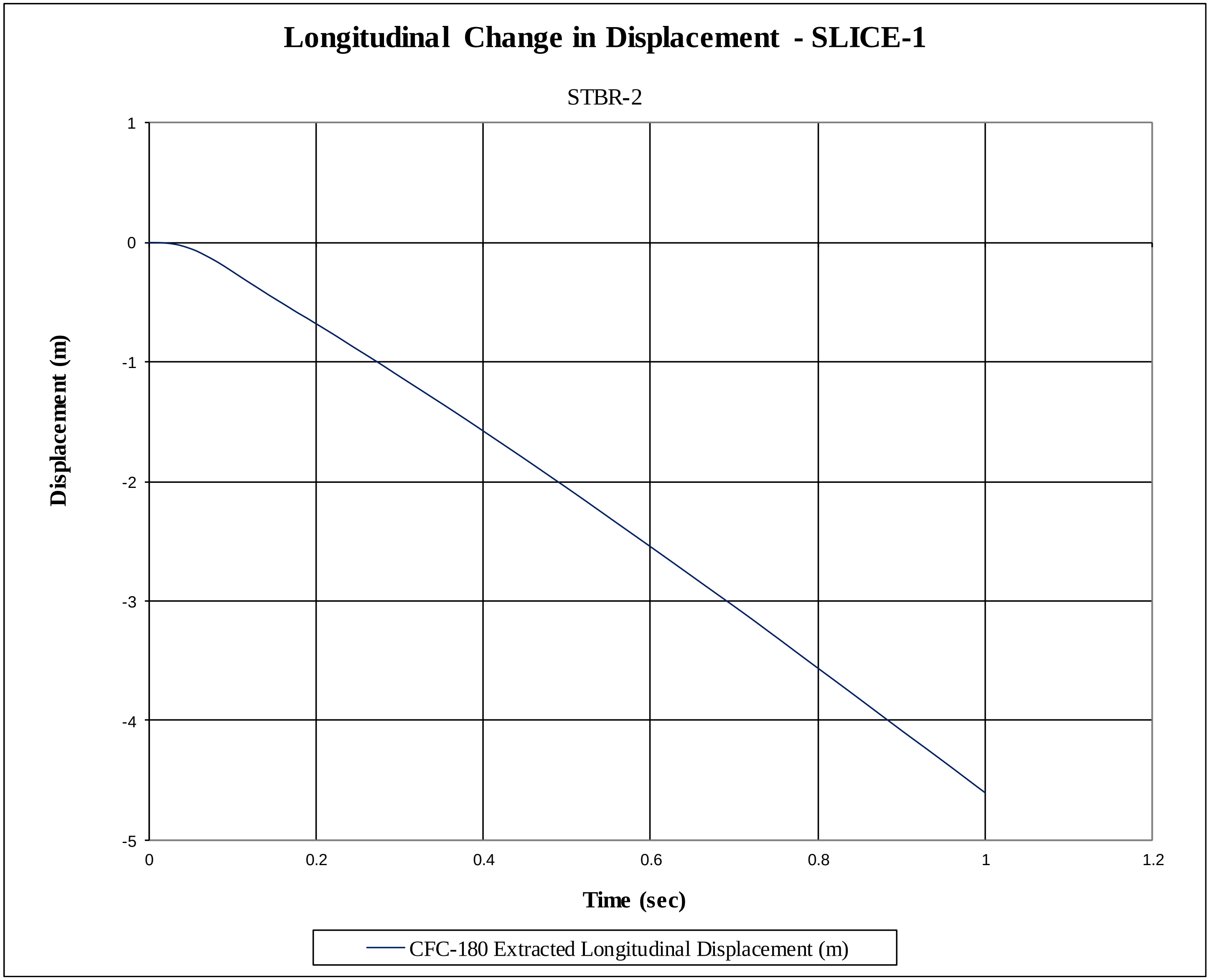


Figure H-3. Longitudinal Occupant Displacement (SLICE-1), Test No. STBR-2

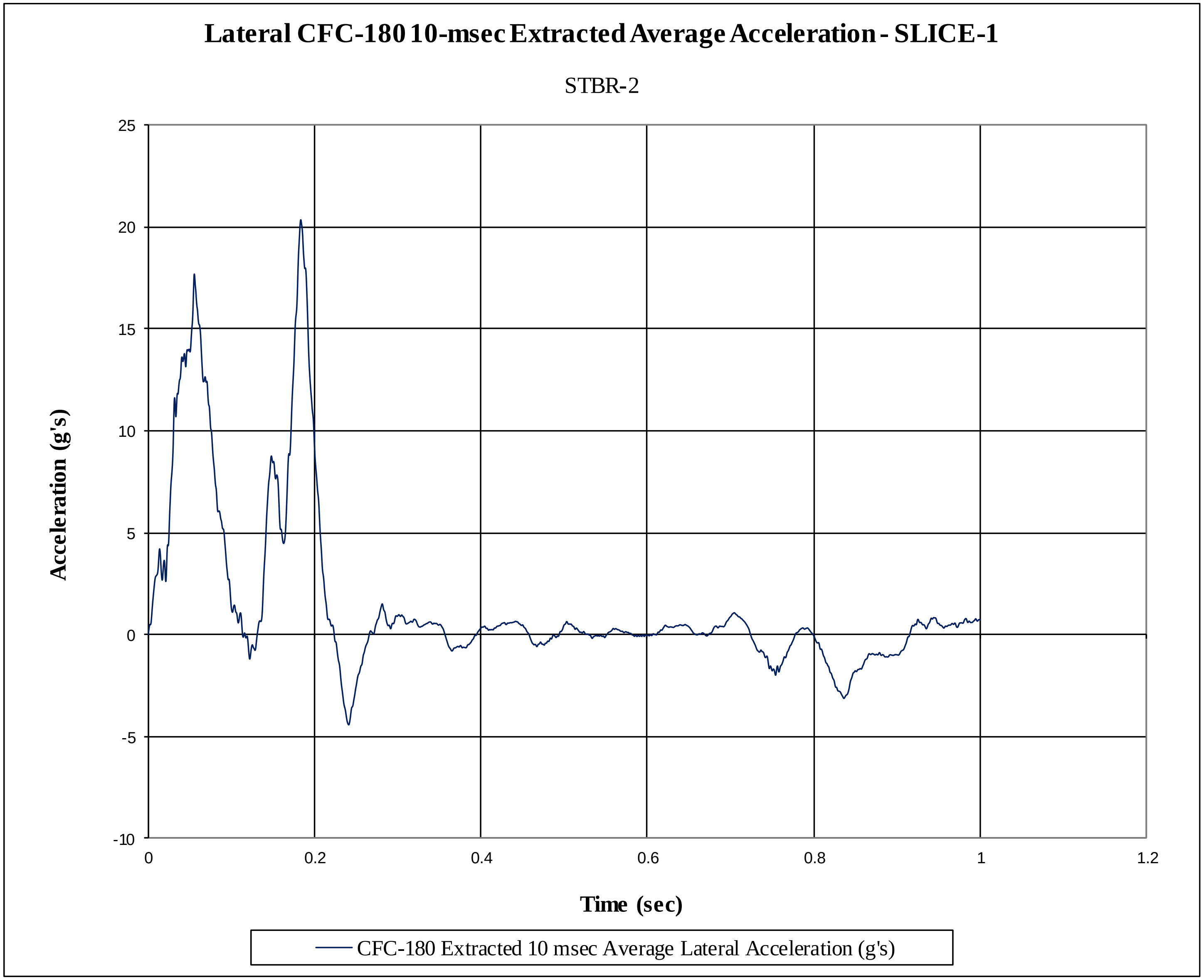


Figure H-4. 10-ms Average Lateral Deceleration (SLICE-1), Test No. STBR-2

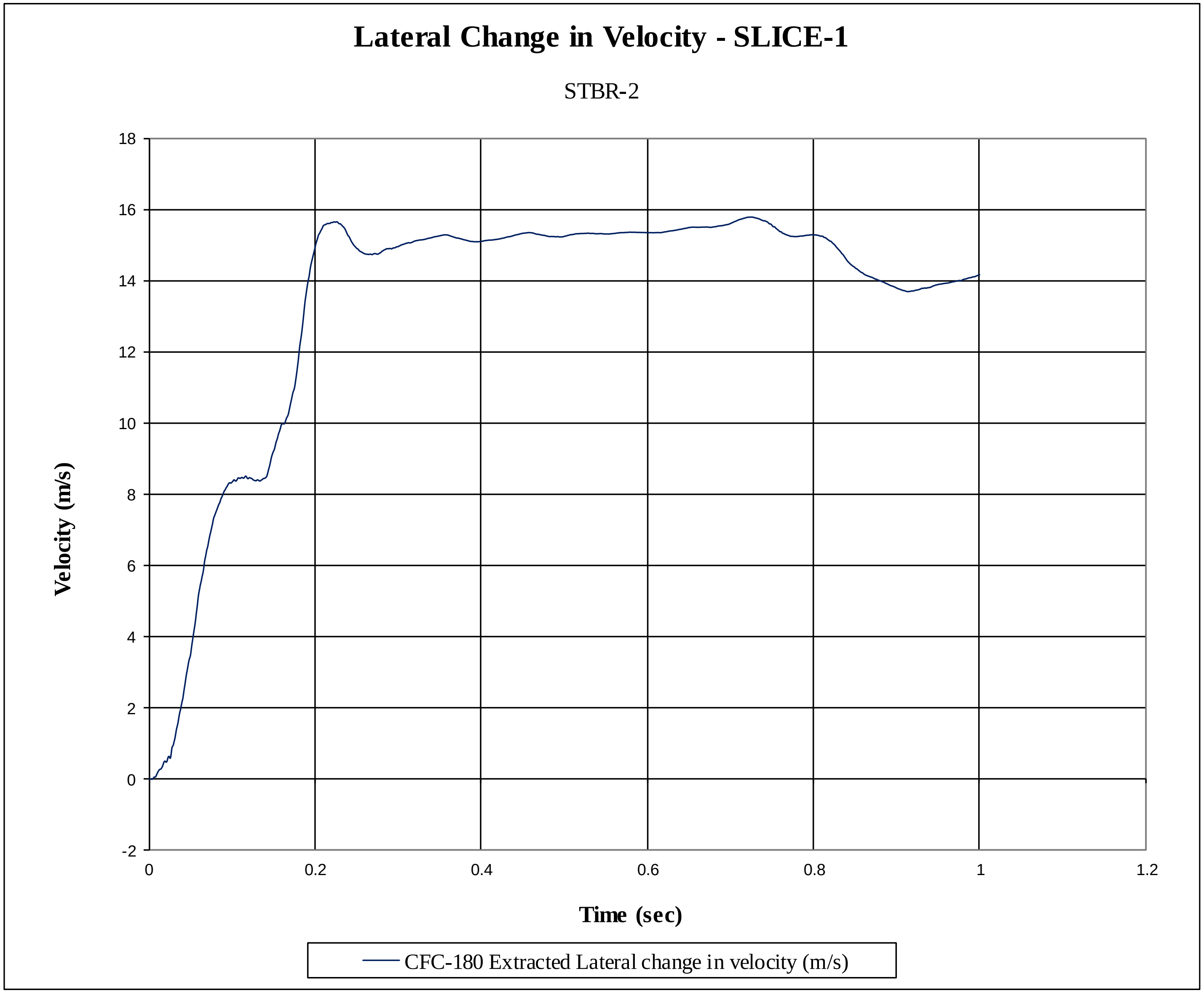


Figure H-5. Lateral Occupant Impact Velocity (SLICE-1), Test No. STBR-2

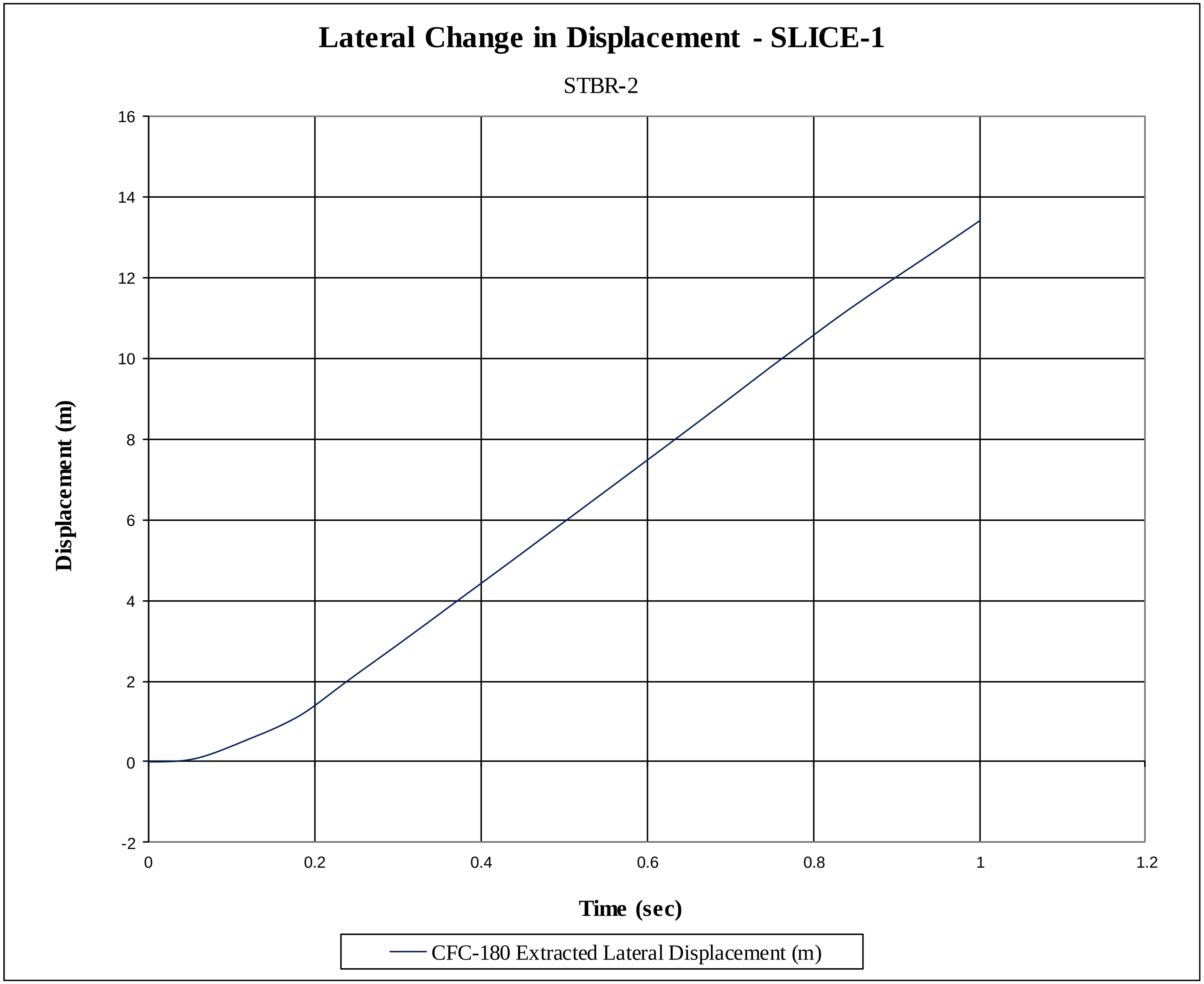


Figure H-6. Lateral Occupant Displacement (SLICE-1), Test No. STBR-2

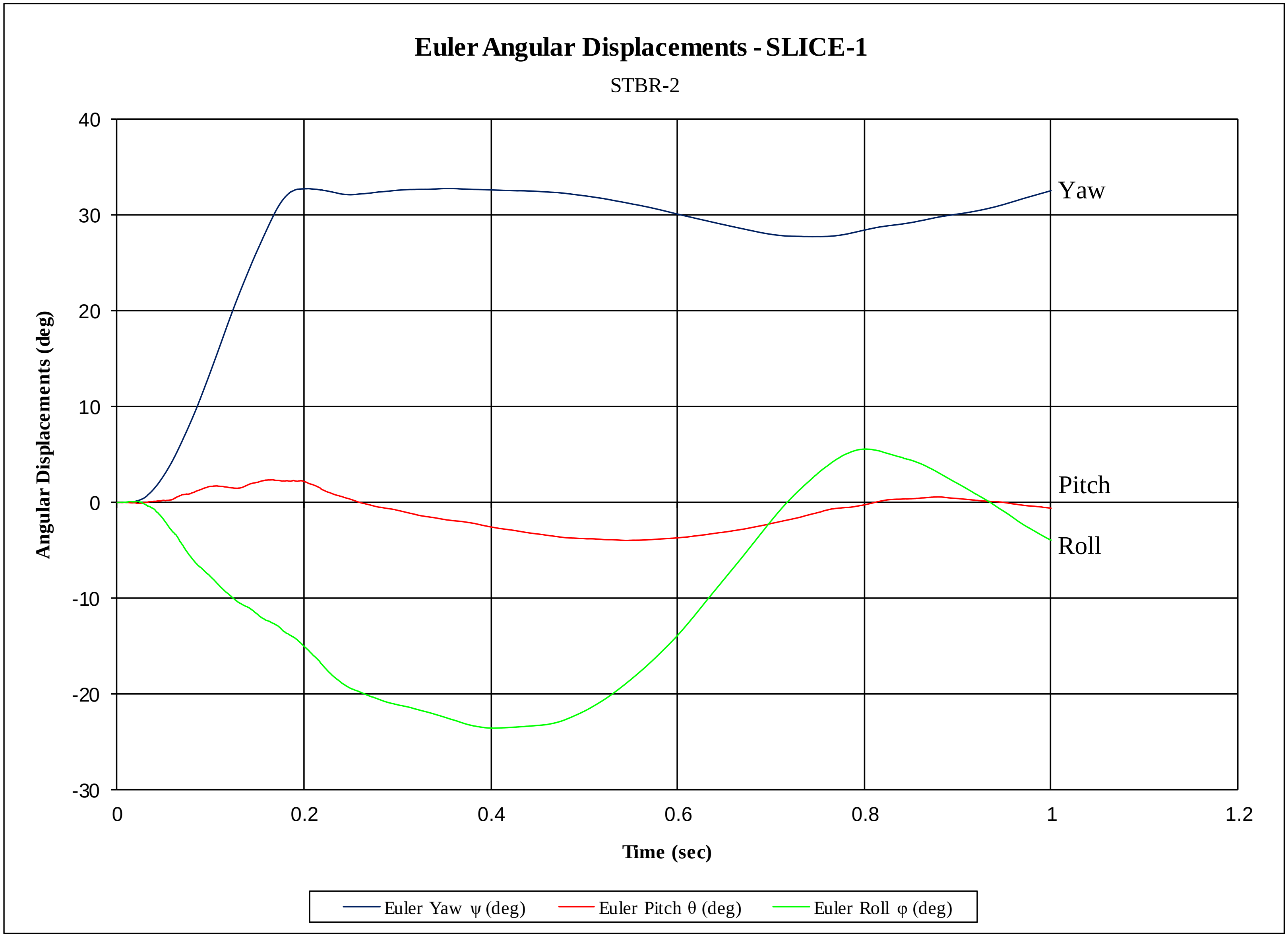


Figure H-7. Vehicle Angular Displacement (SLICE-1), Test No. STBR-2

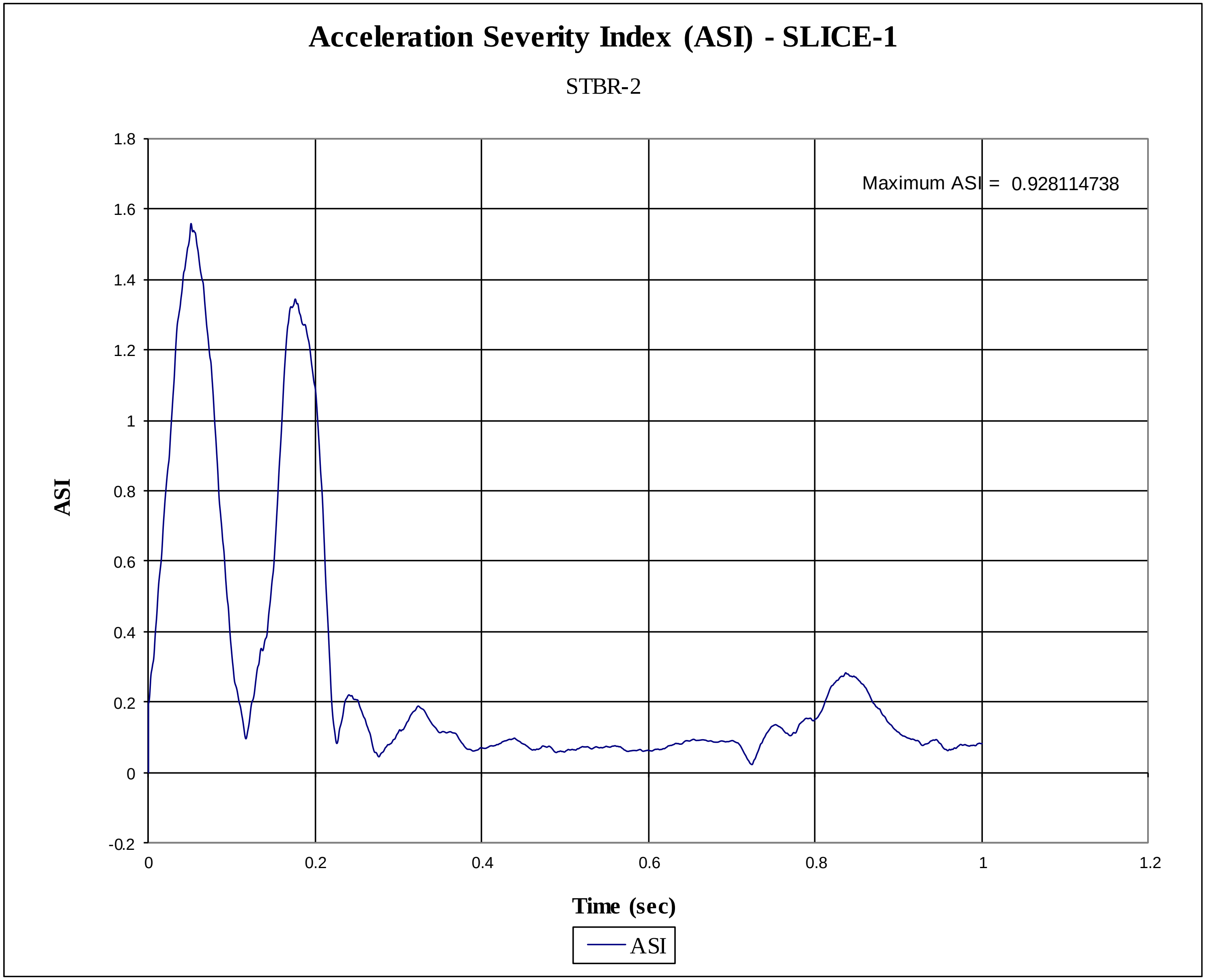


Figure H-8. Acceleration Severity Index (SLICE-1), Test No. STBR-2

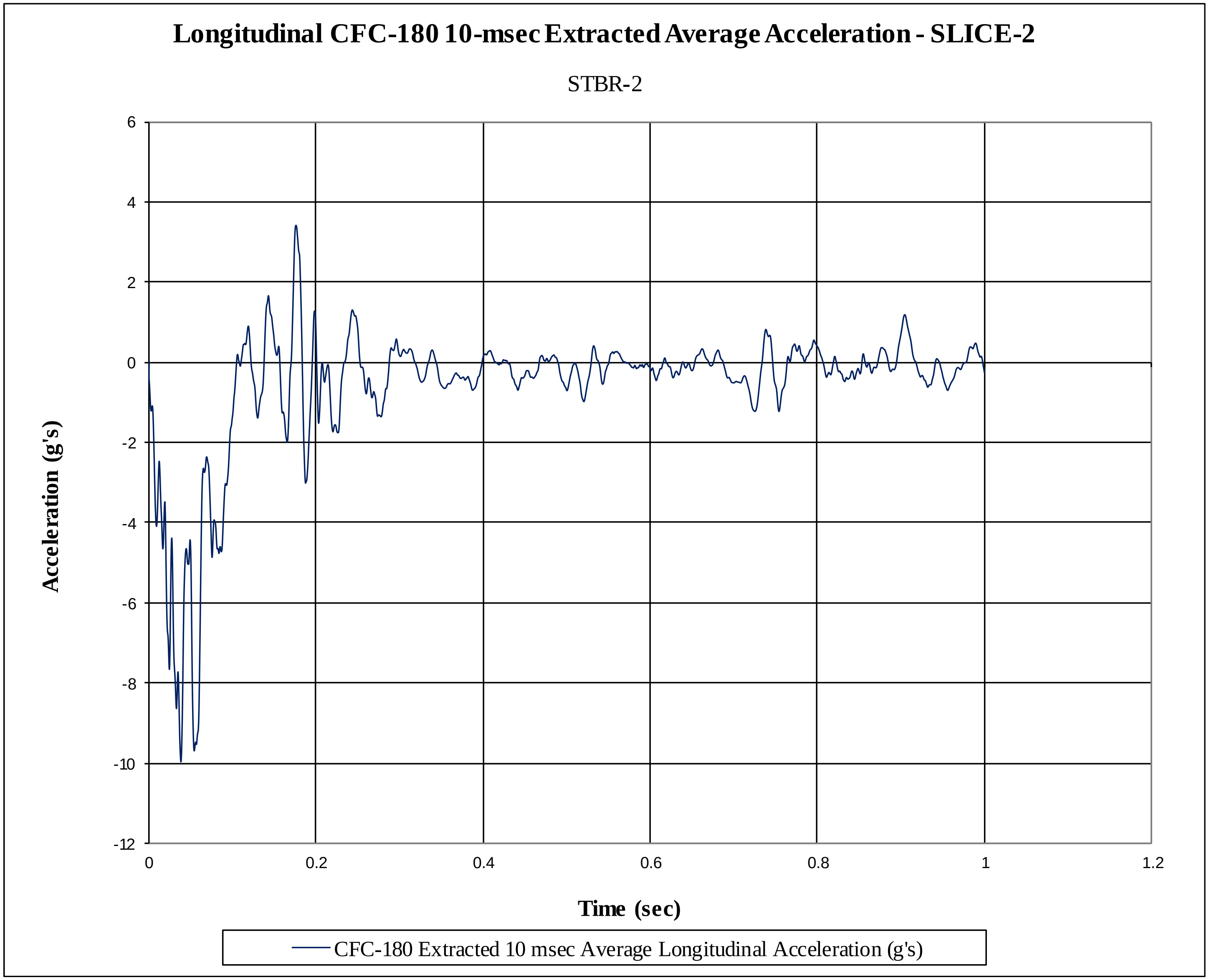


Figure H-9. 10-ms Average Longitudinal Deceleration (SLICE-2), Test No. STBR-2

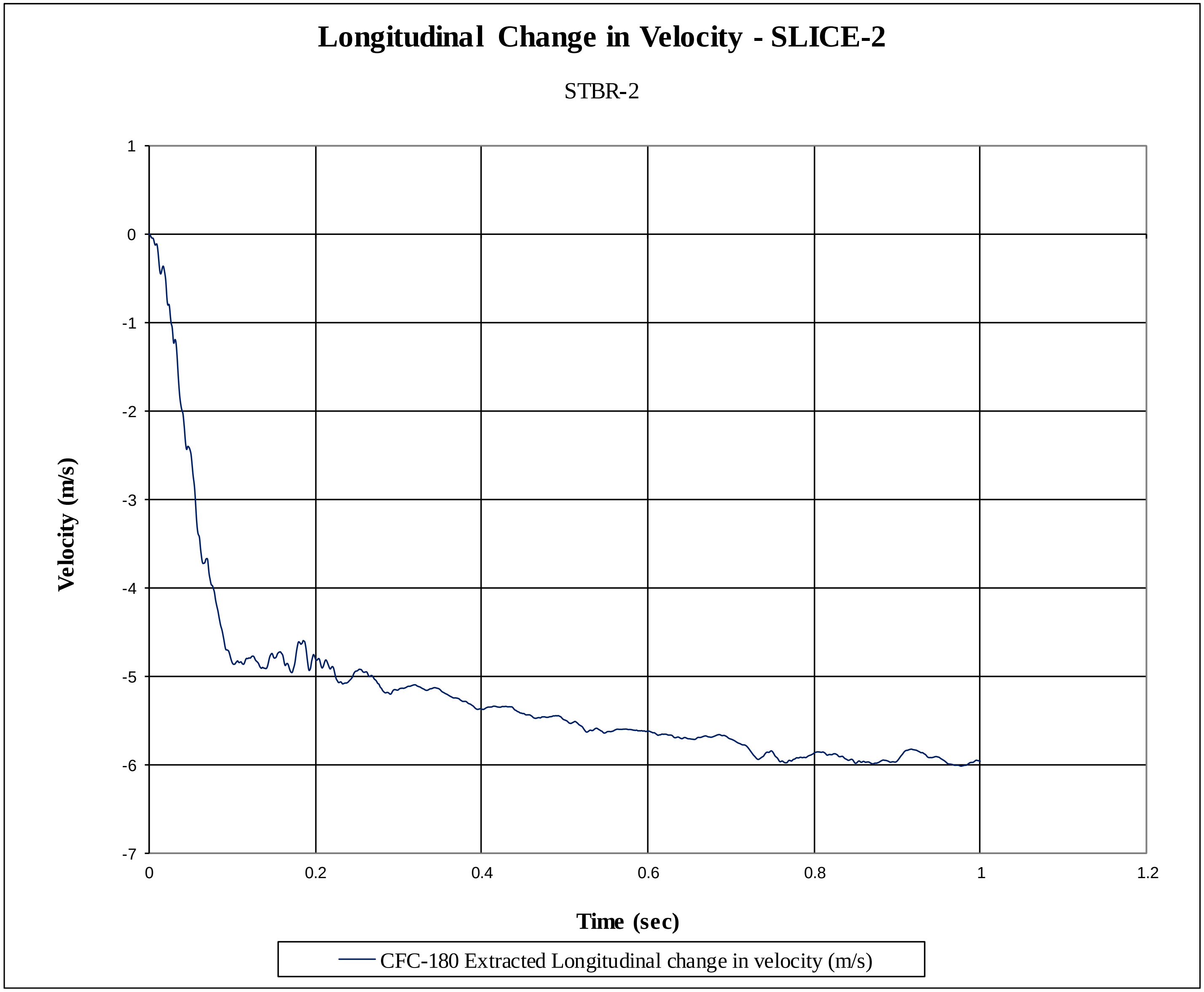


Figure H-10. Longitudinal Occupant Impact Velocity (SLICE-2), Test No. STBR-2

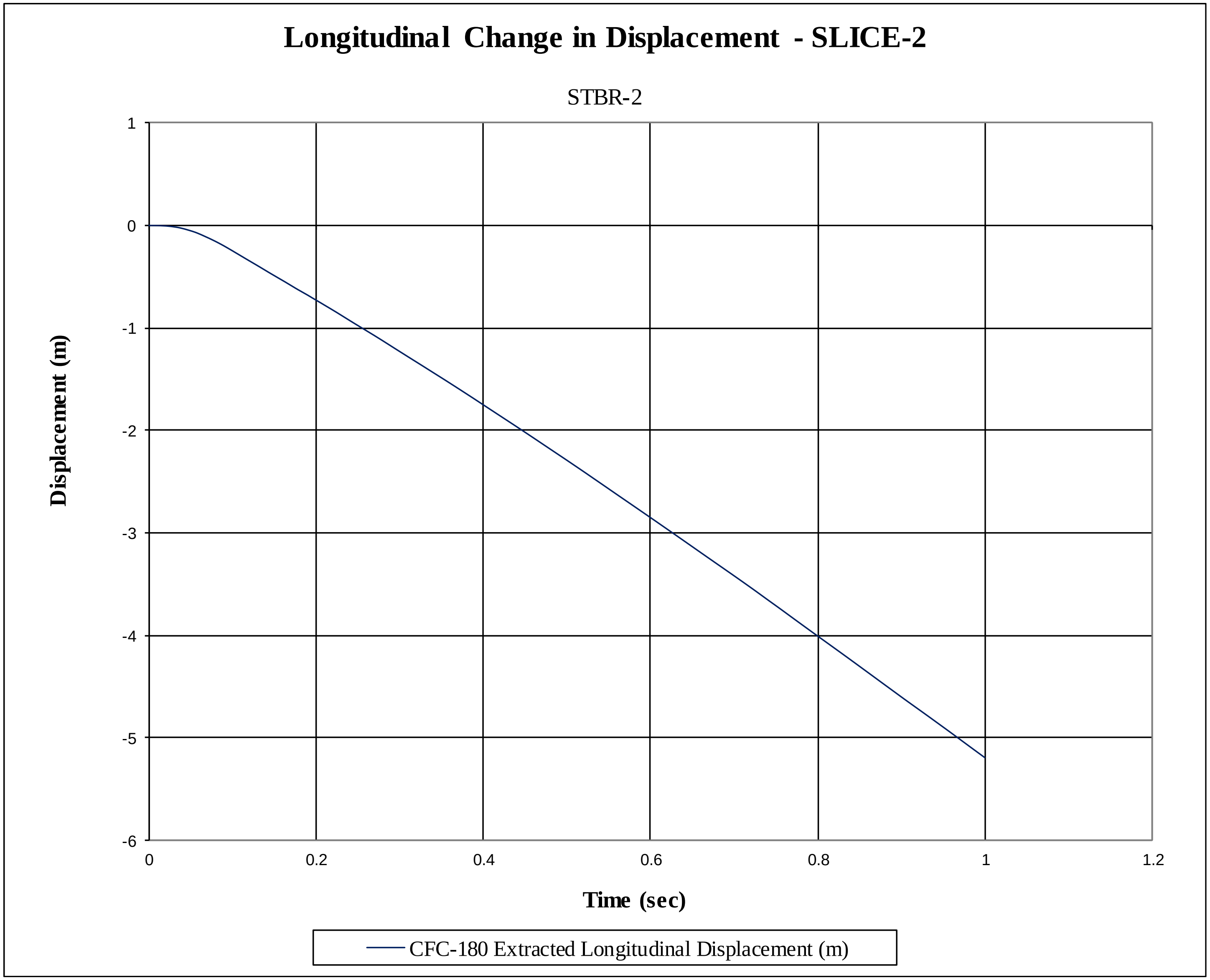


Figure H-11. Longitudinal Occupant Displacement (SLICE-2), Test No. STBR-2

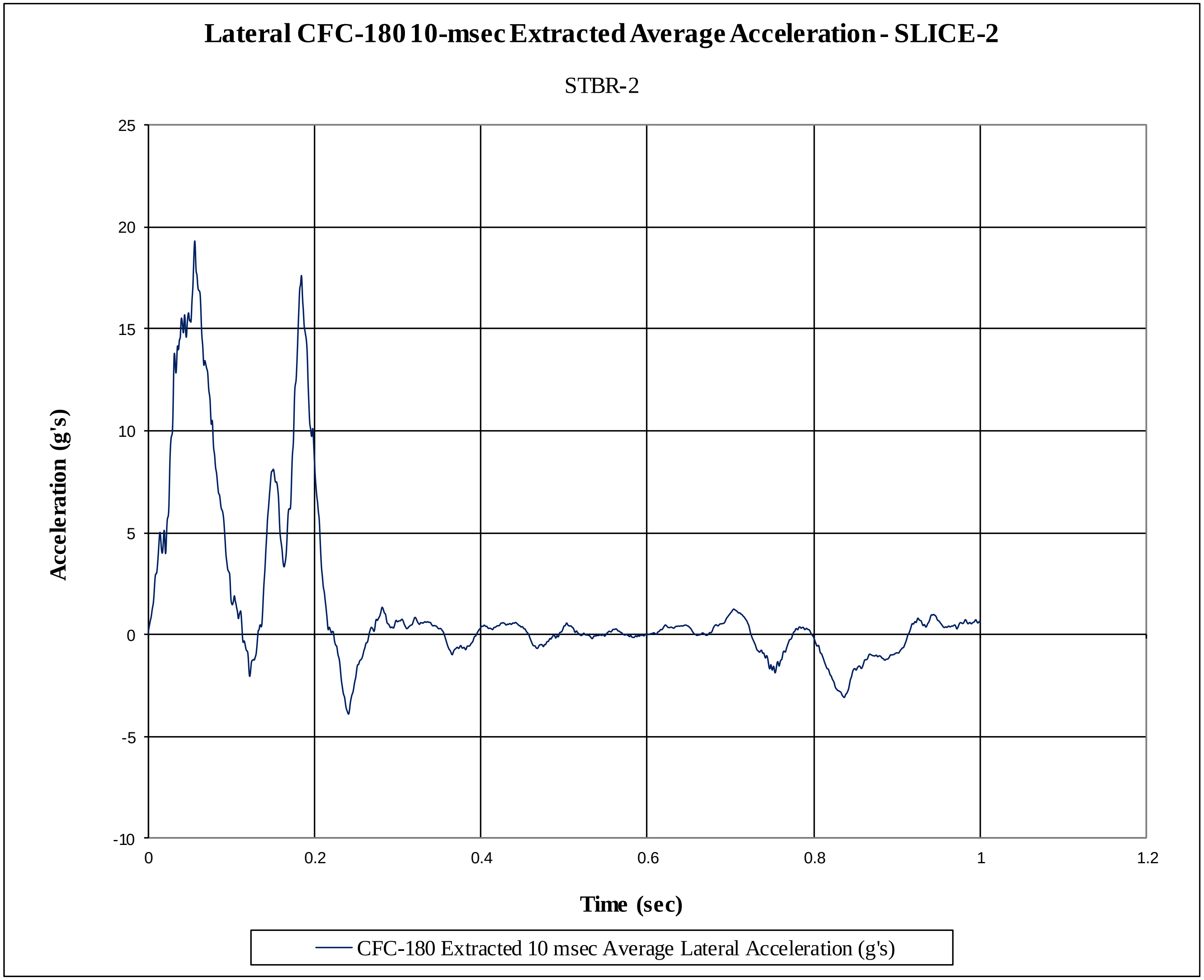


Figure H-12. 10-ms Average Lateral Deceleration (SLICE-2), Test No. STBR-2

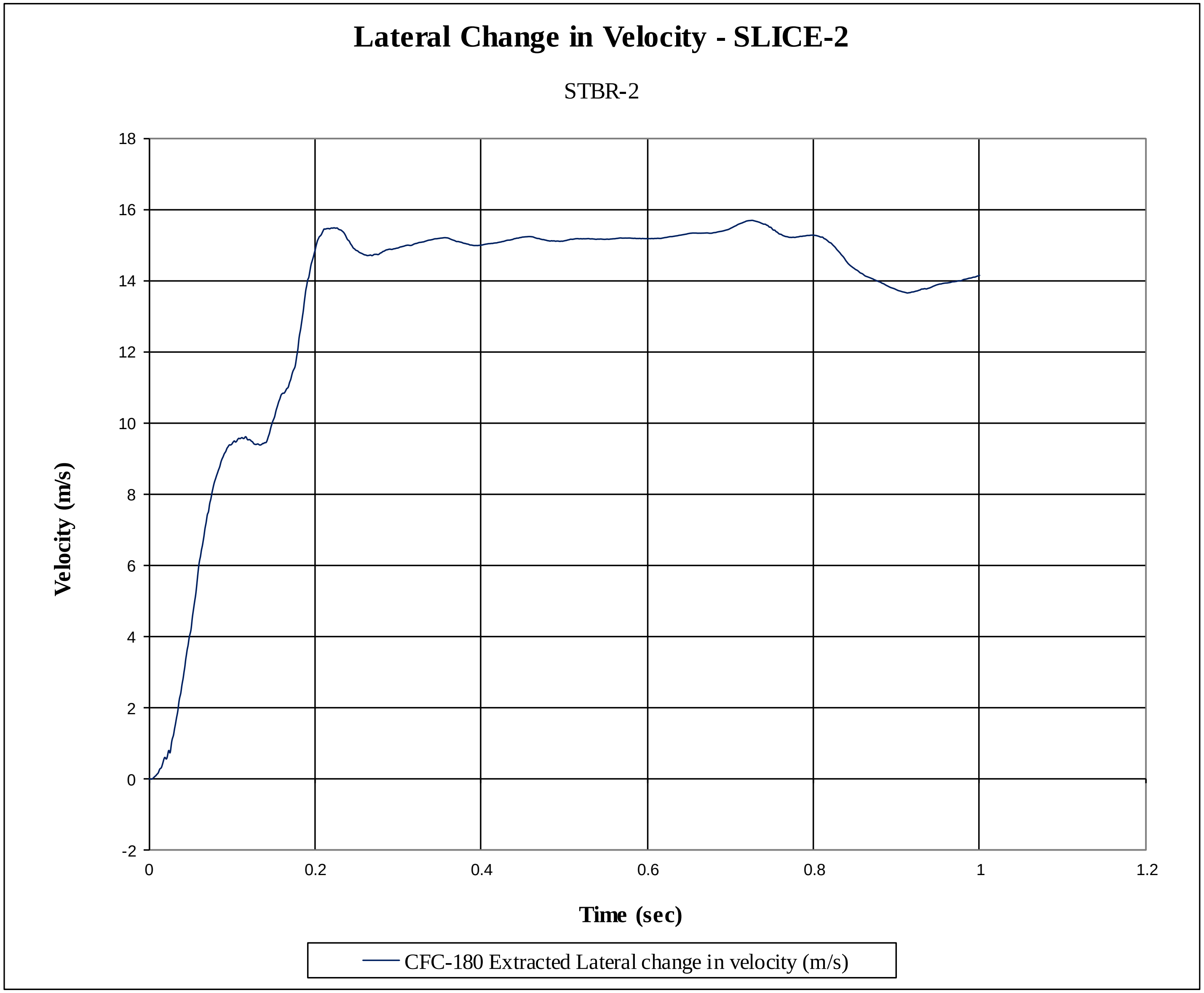


Figure H-13. Lateral Occupant Impact Velocity (SLICE-2), Test No. STBR-2

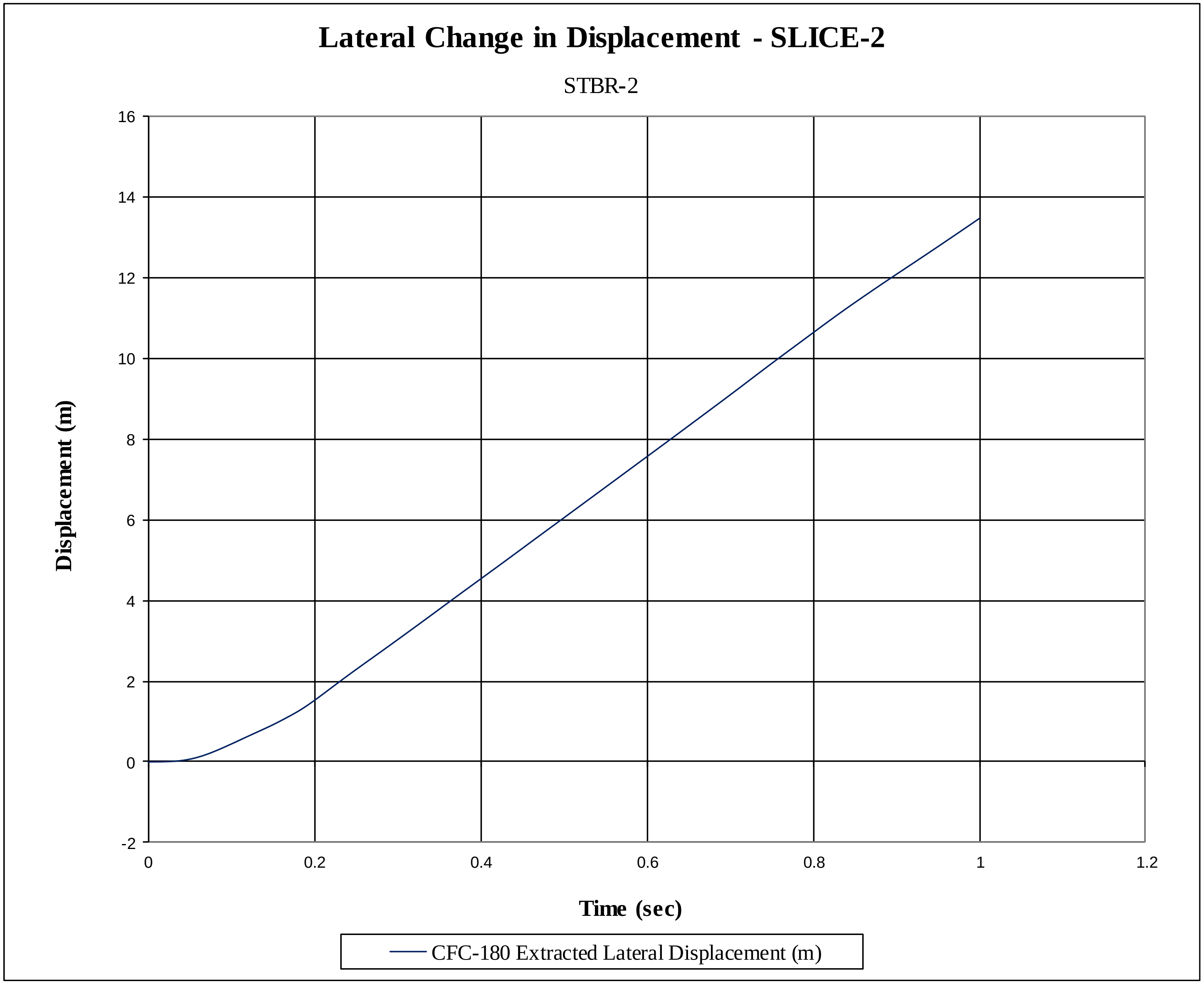


Figure H-14. Lateral Occupant Displacement (SLICE-2), Test No. STBR-2

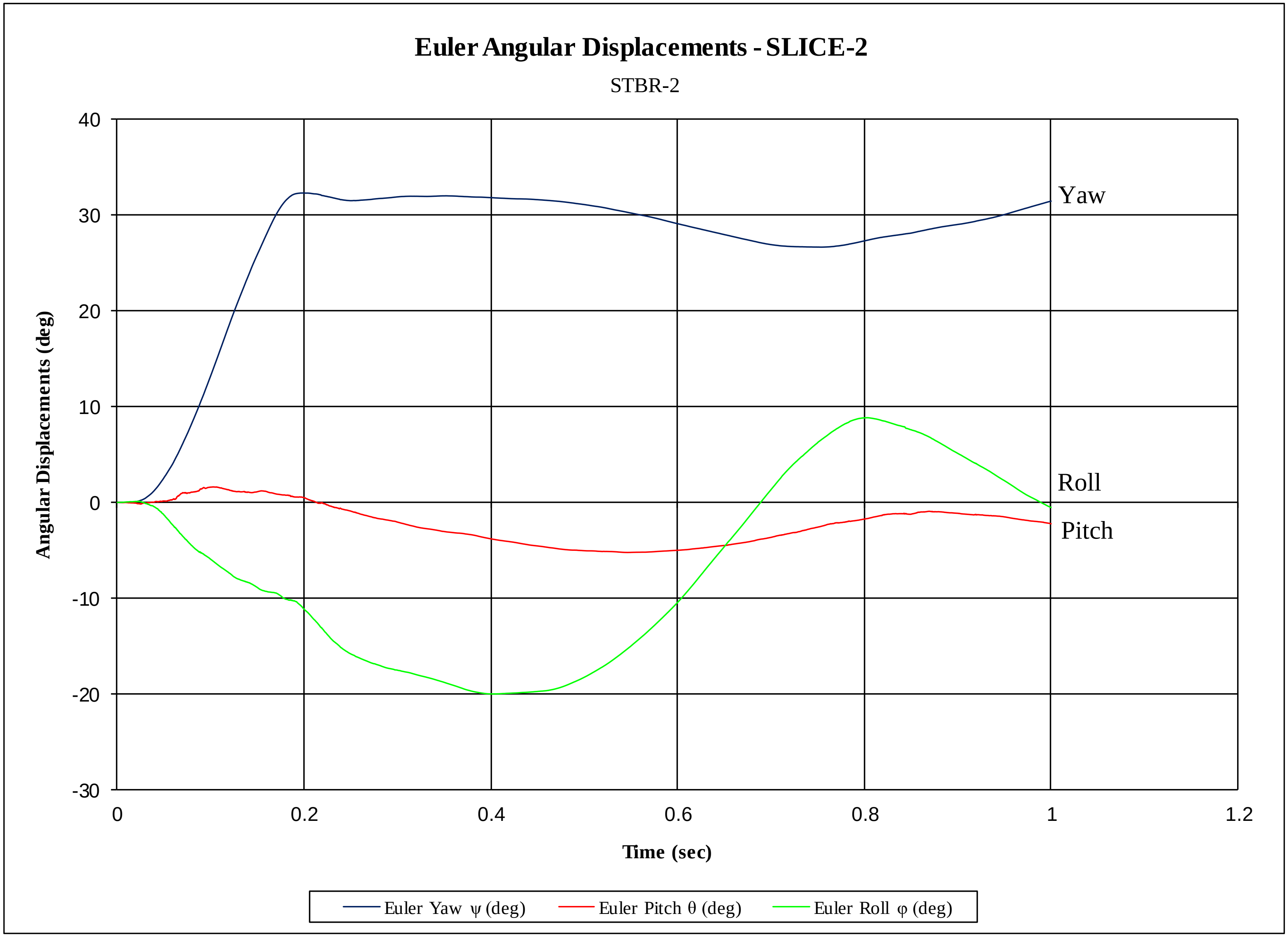


Figure H-15. Vehicle Angular Displacement (SLICE-2), Test No. STBR-2

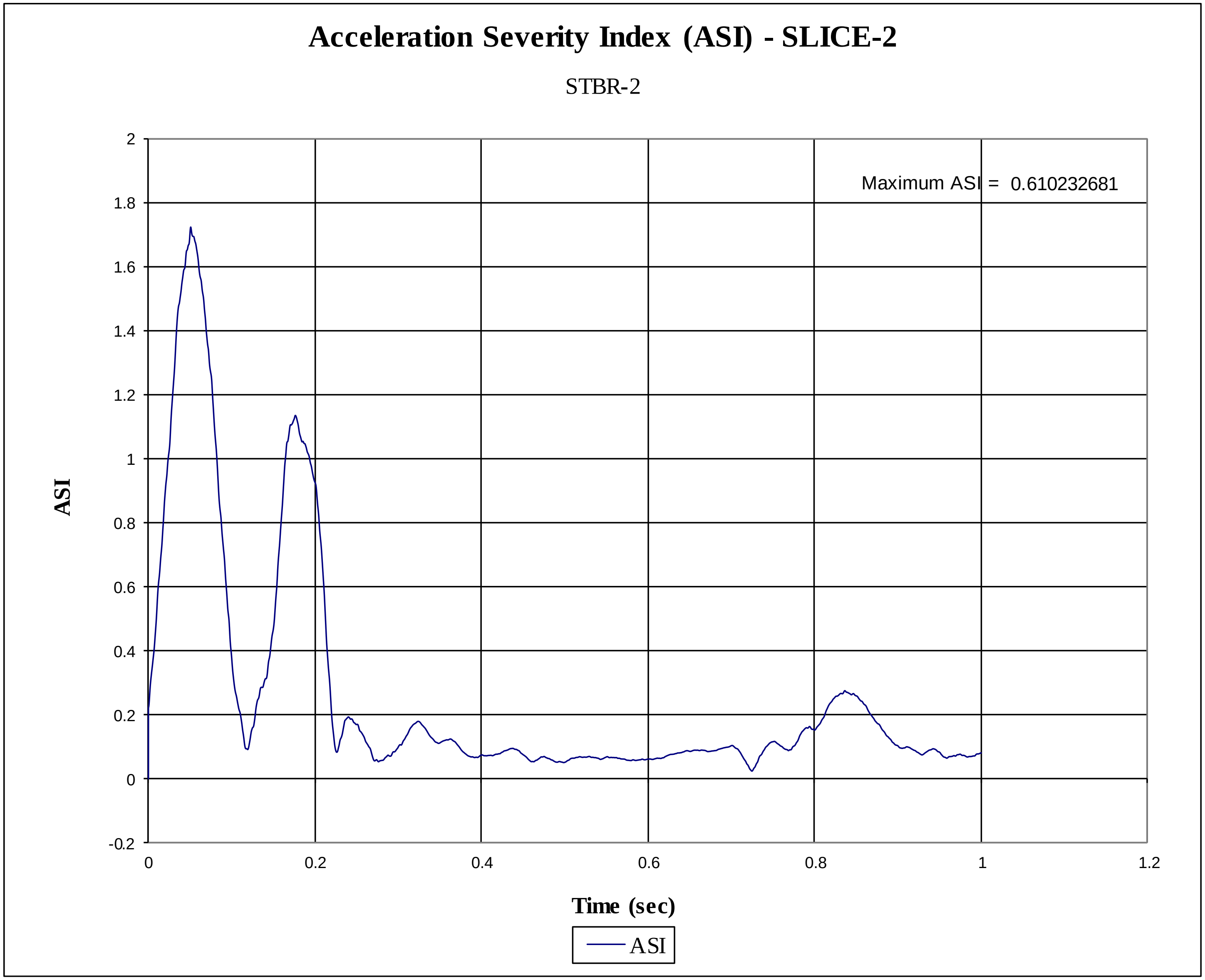


Figure H-16. Acceleration Severity Index (SLICE-2), Test No. STBR-2

Appendix I. Accelerometer and Rate Transducer Data Plots, Test No. STBR-3

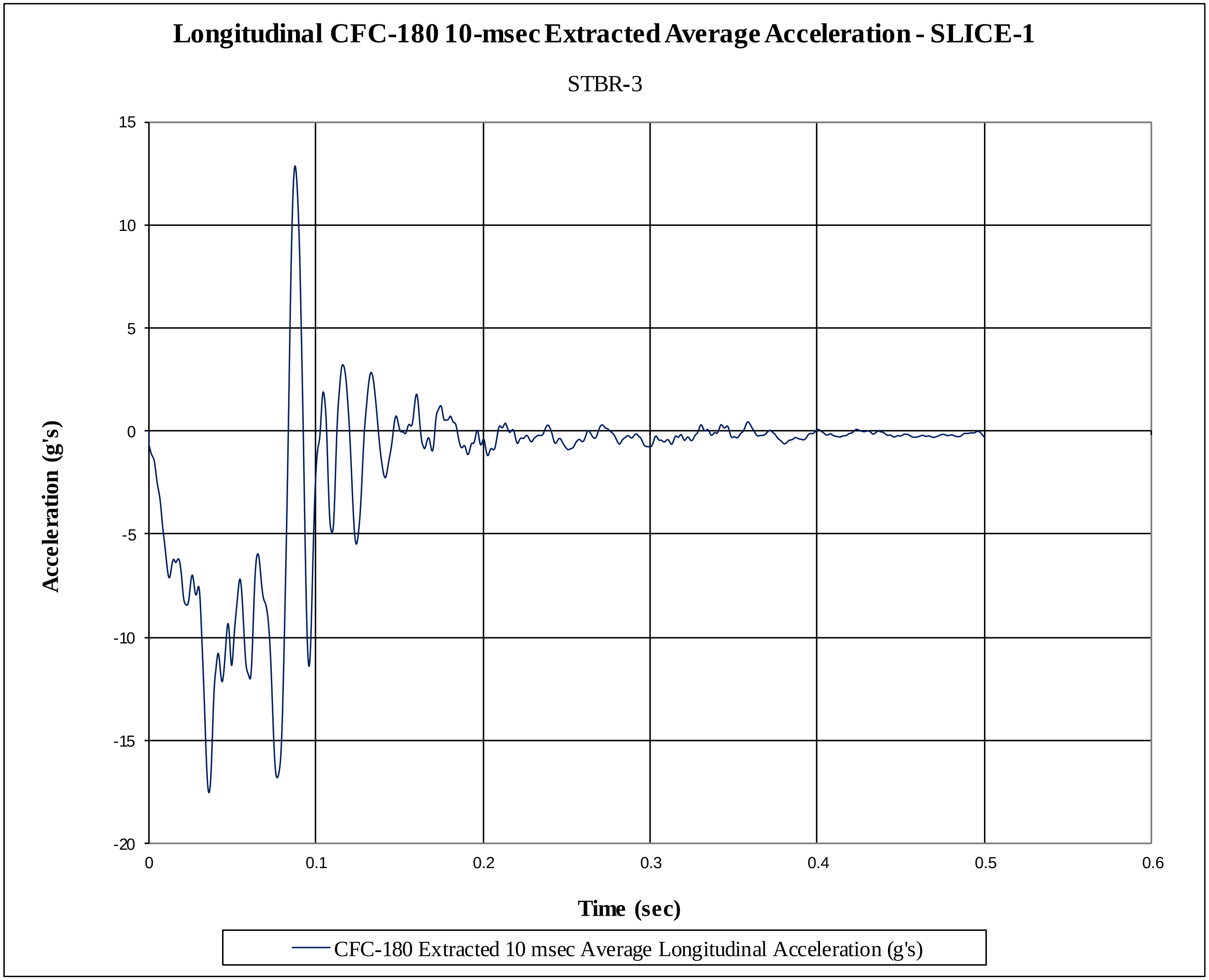


Figure I-1. 10-ms Average Longitudinal Deceleration (SLICE-1), Test No. STBR-3

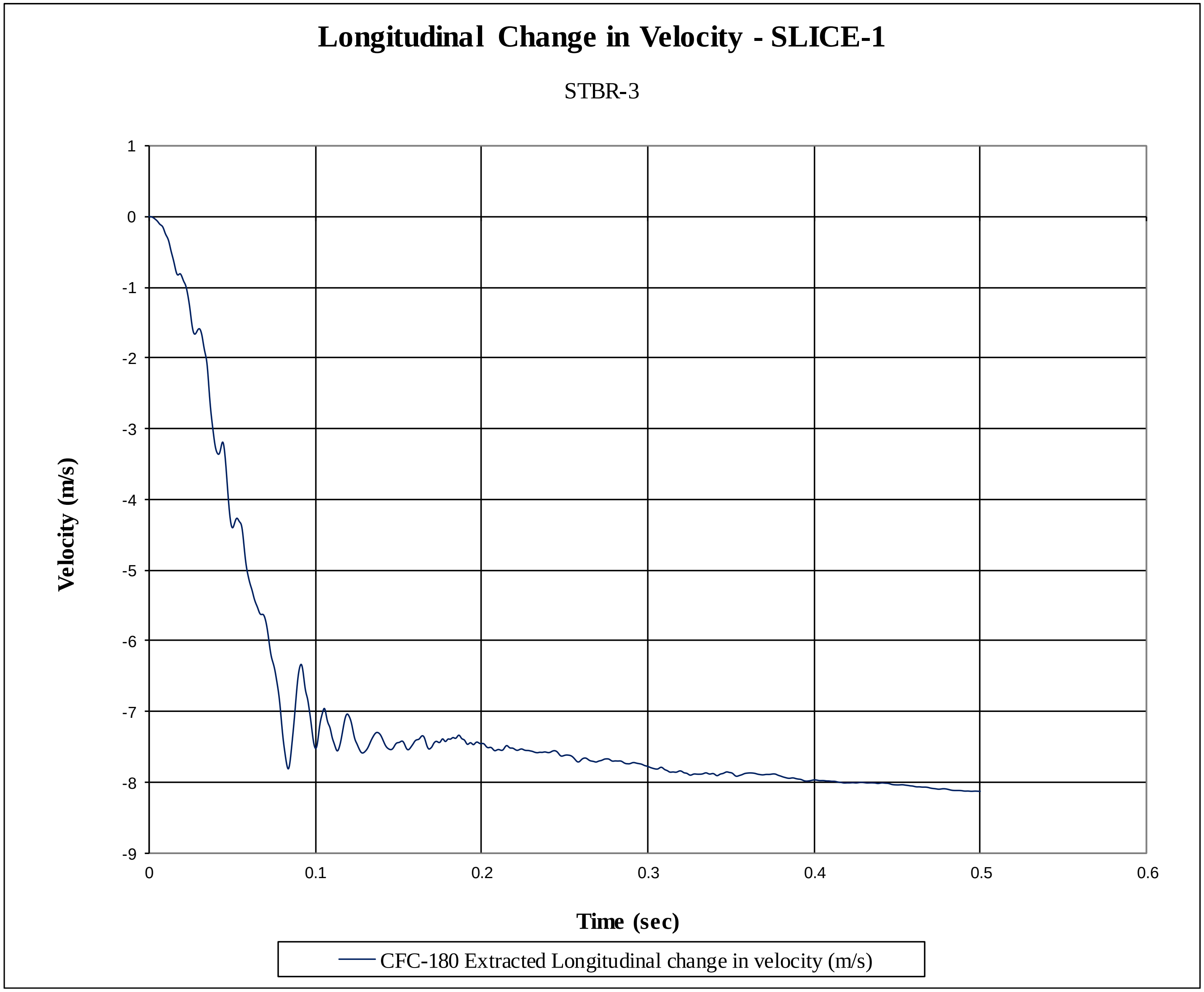


Figure I-2. Longitudinal Occupant Impact Velocity (SLICE-1), Test No. STBR-3

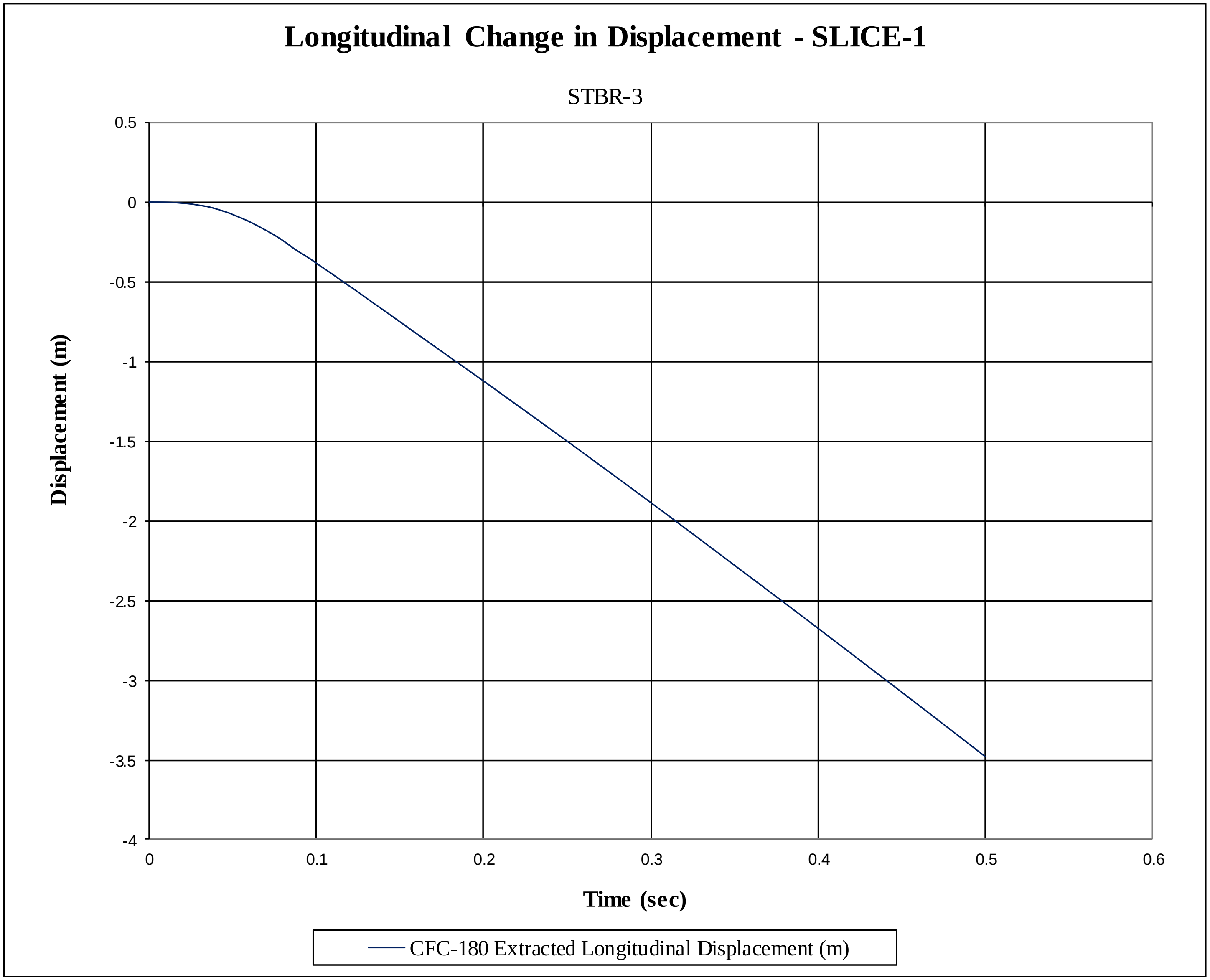


Figure I-3. Longitudinal Occupant Displacement (SLICE-1), Test No. STBR-3

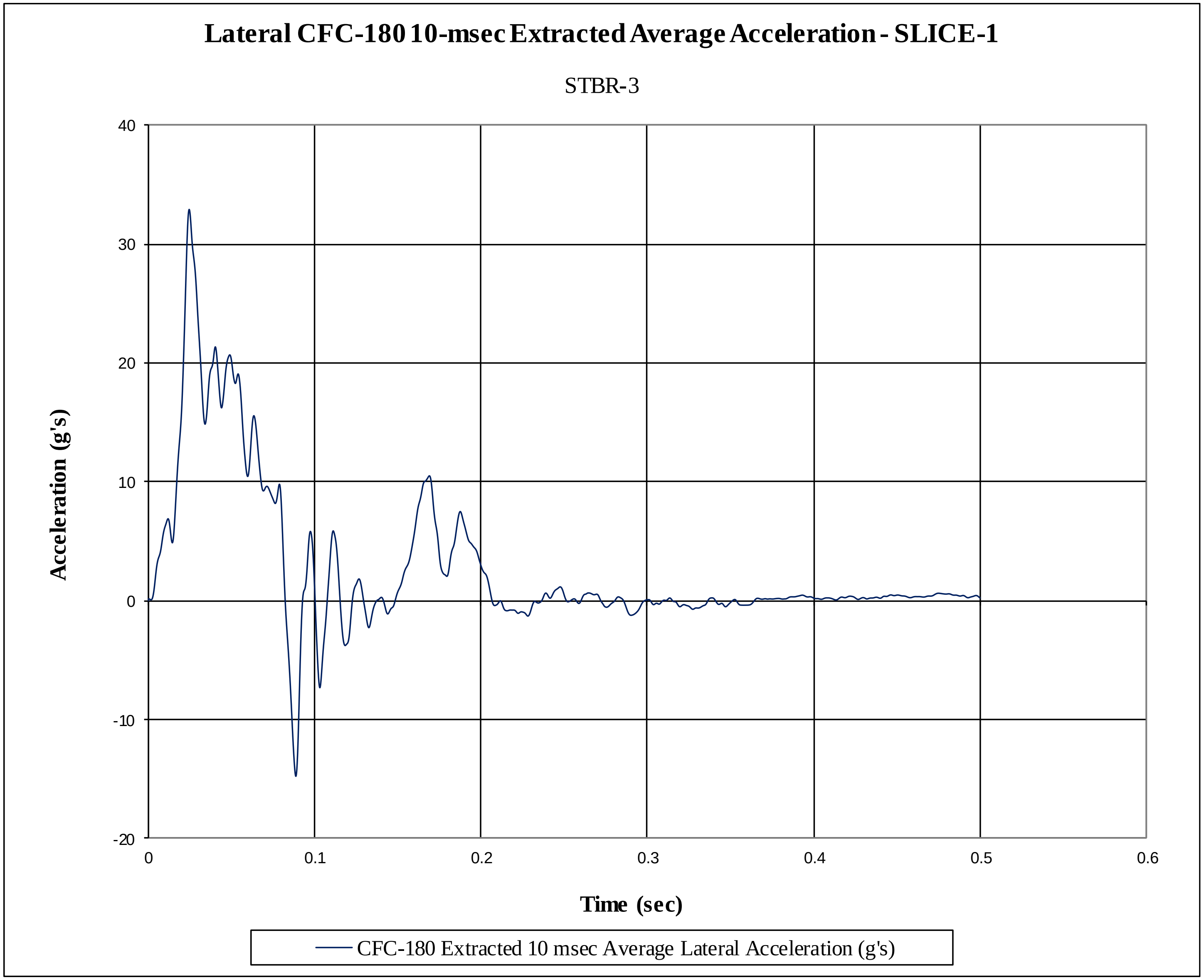


Figure I-4. 10-ms Average Lateral Deceleration (SLICE-1), Test No. STBR-3

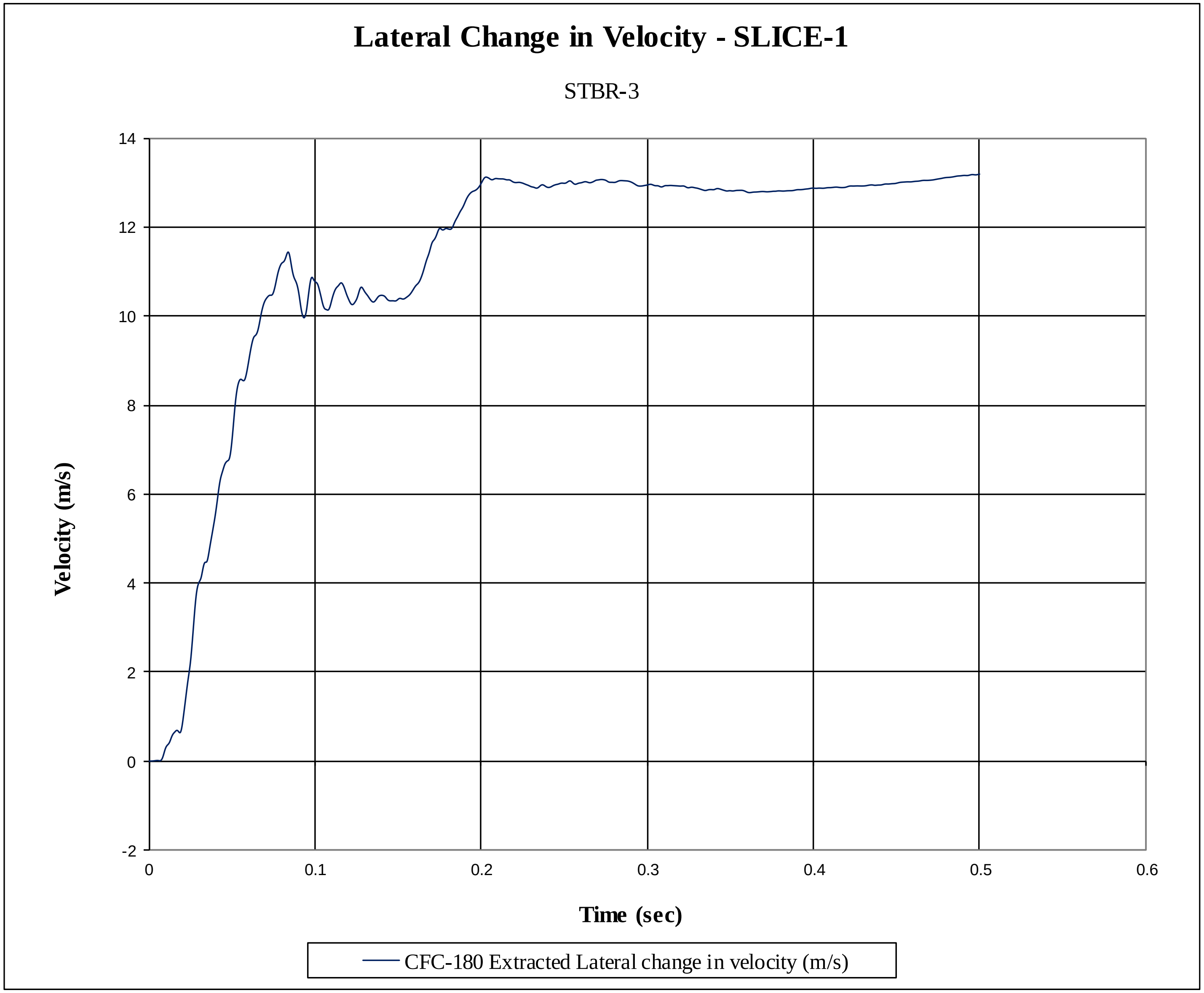


Figure I-5. Lateral Occupant Impact Velocity (SLICE-1), Test No. STBR-3

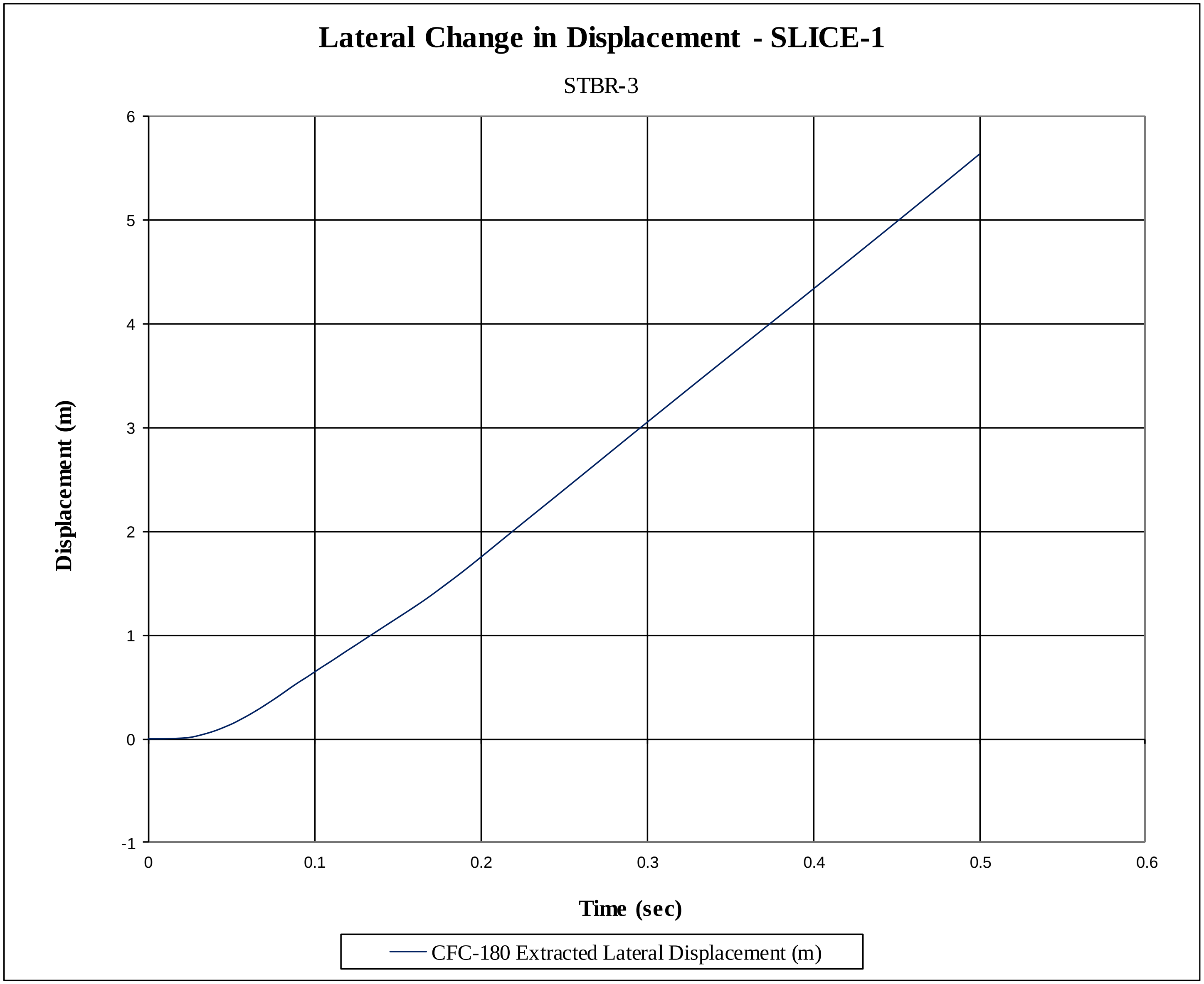


Figure I-6. Lateral Occupant Displacement (SLICE-1), Test No. STBR-3

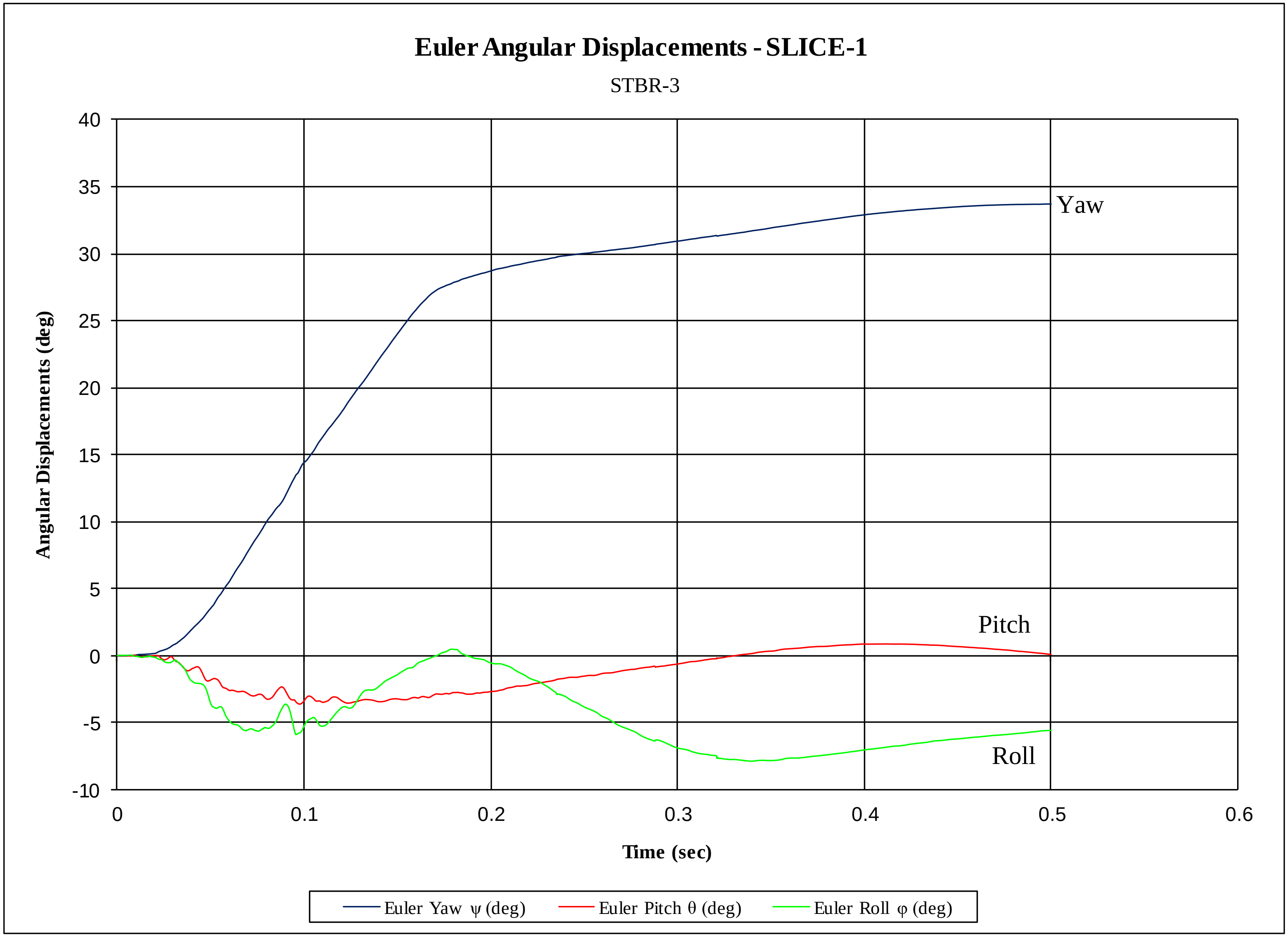


Figure I-7. Vehicle Angular Displacement (SLICE-1), Test No. STBR-3

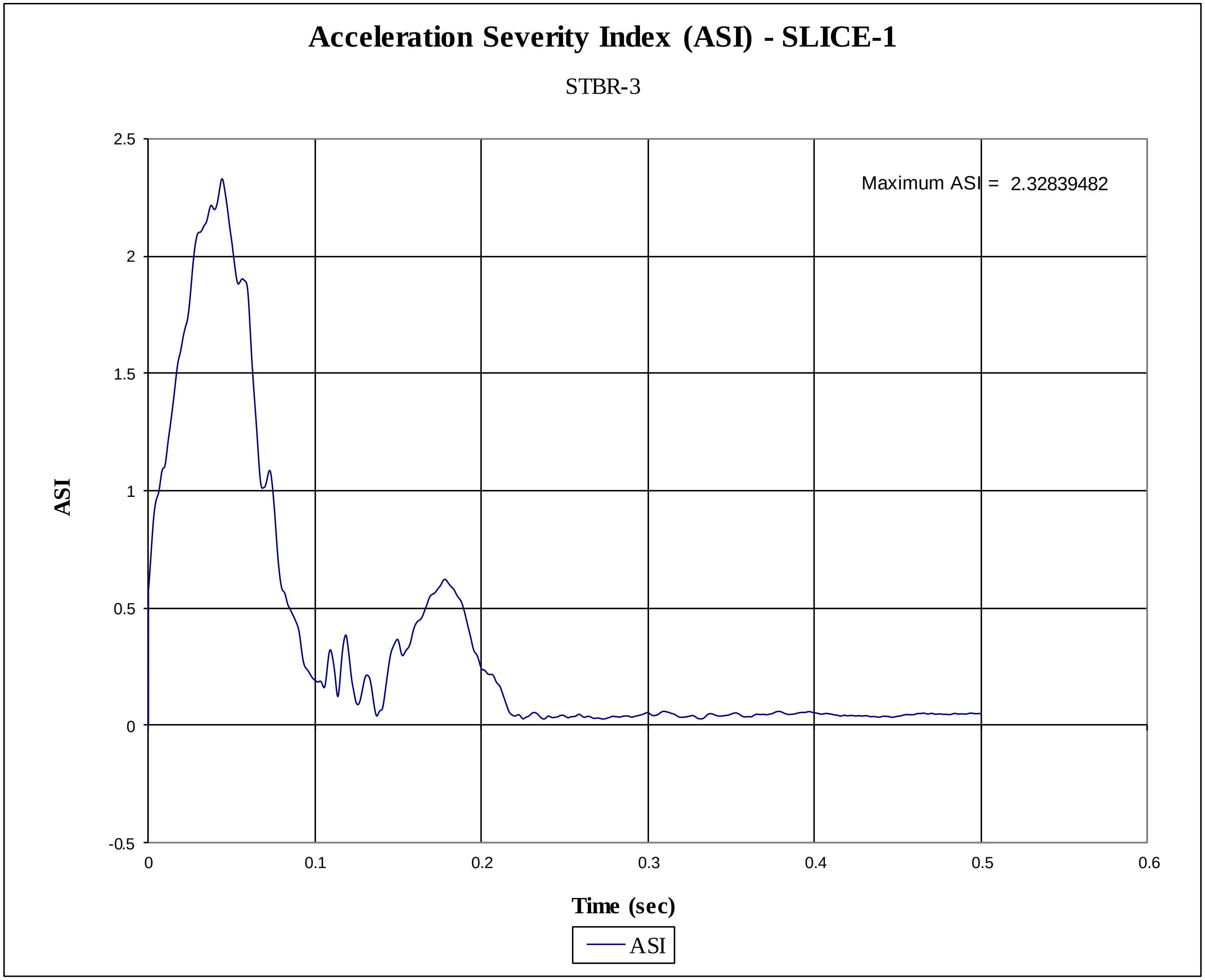


Figure I-8. Acceleration Severity Index (SLICE-1), Test No. STBR-3

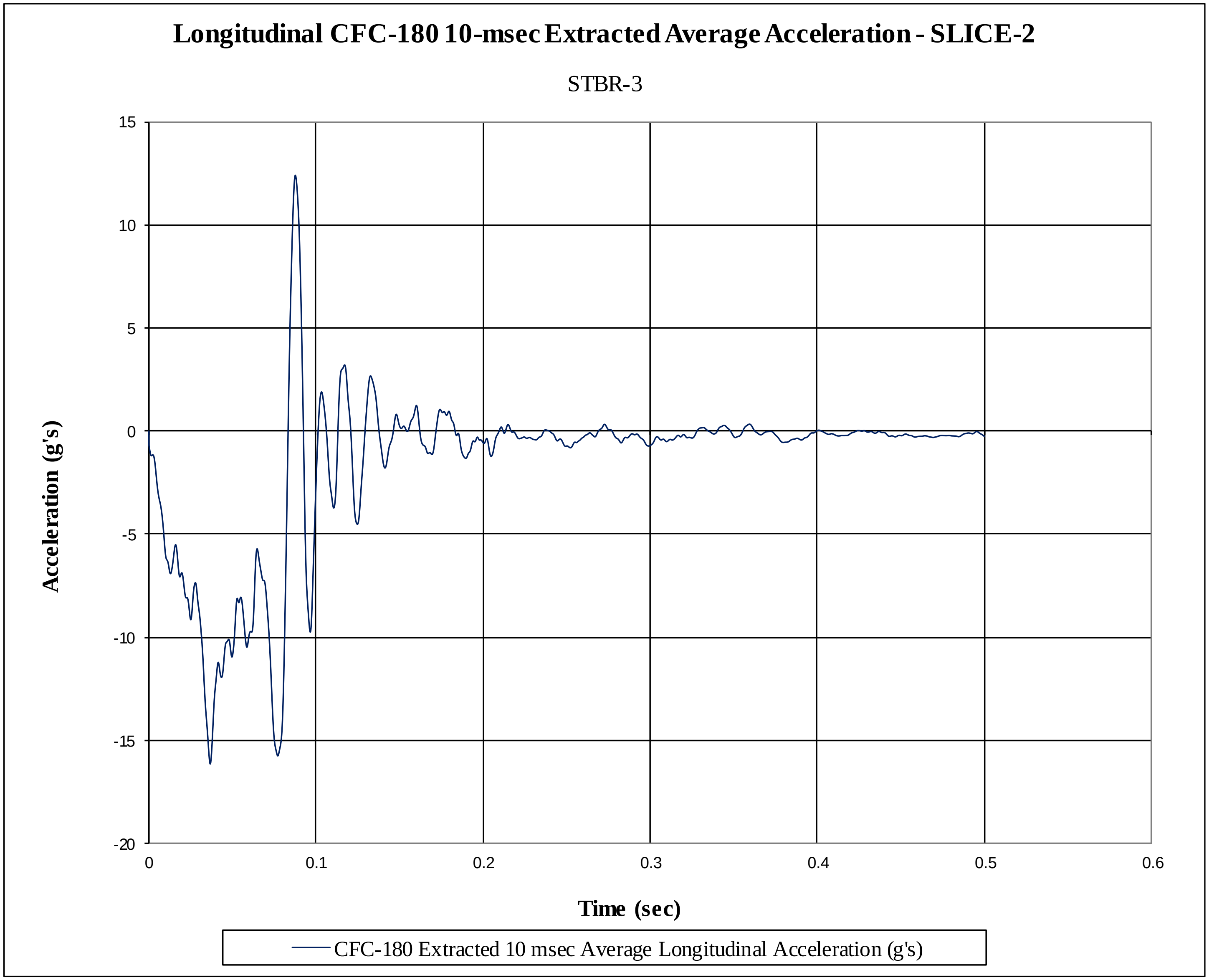


Figure I-9. 10-ms Average Longitudinal Deceleration (SLICE-2), Test No. STBR-3

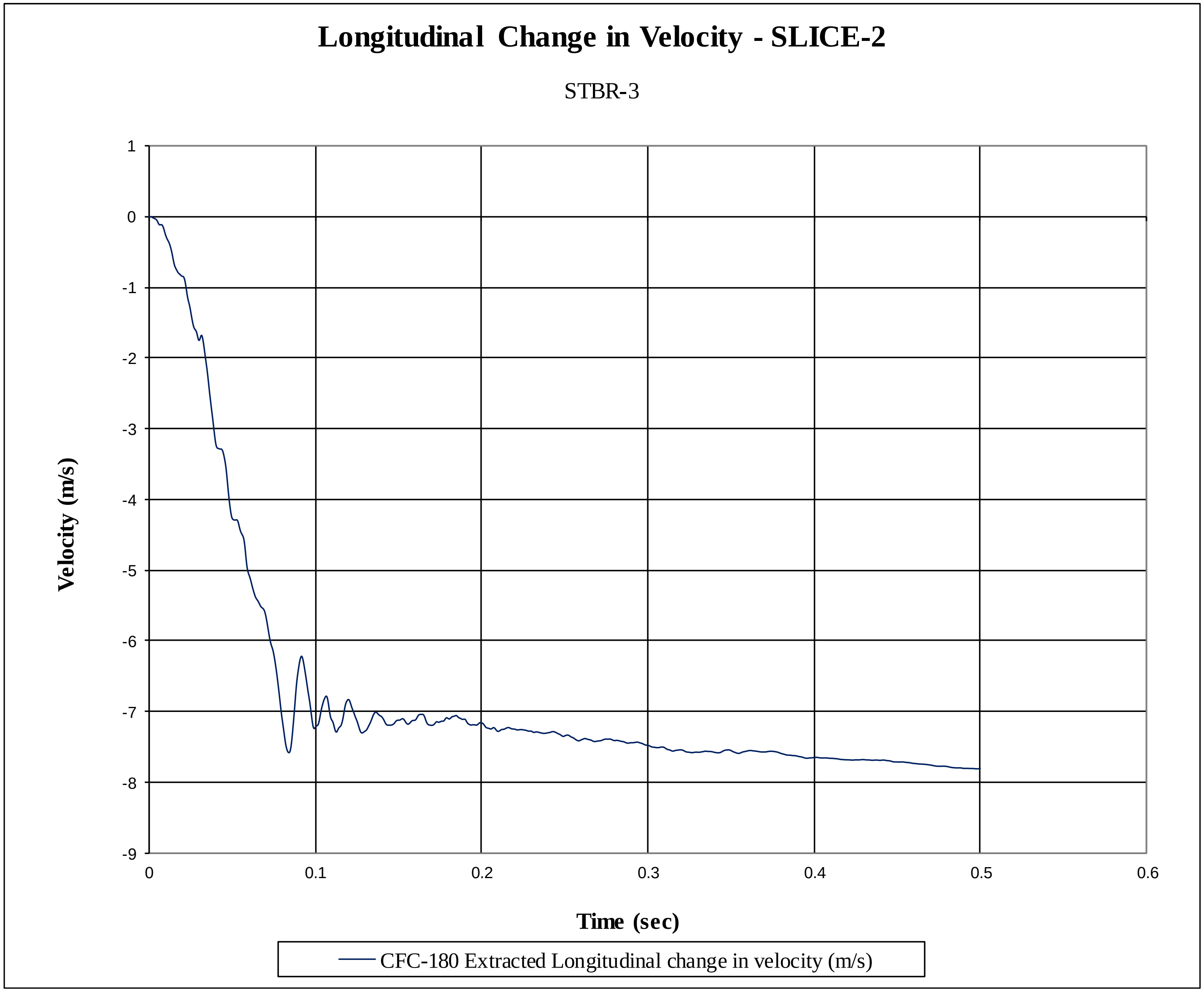


Figure I-10. Longitudinal Occupant Impact Velocity (SLICE-2), Test No. STBR-3

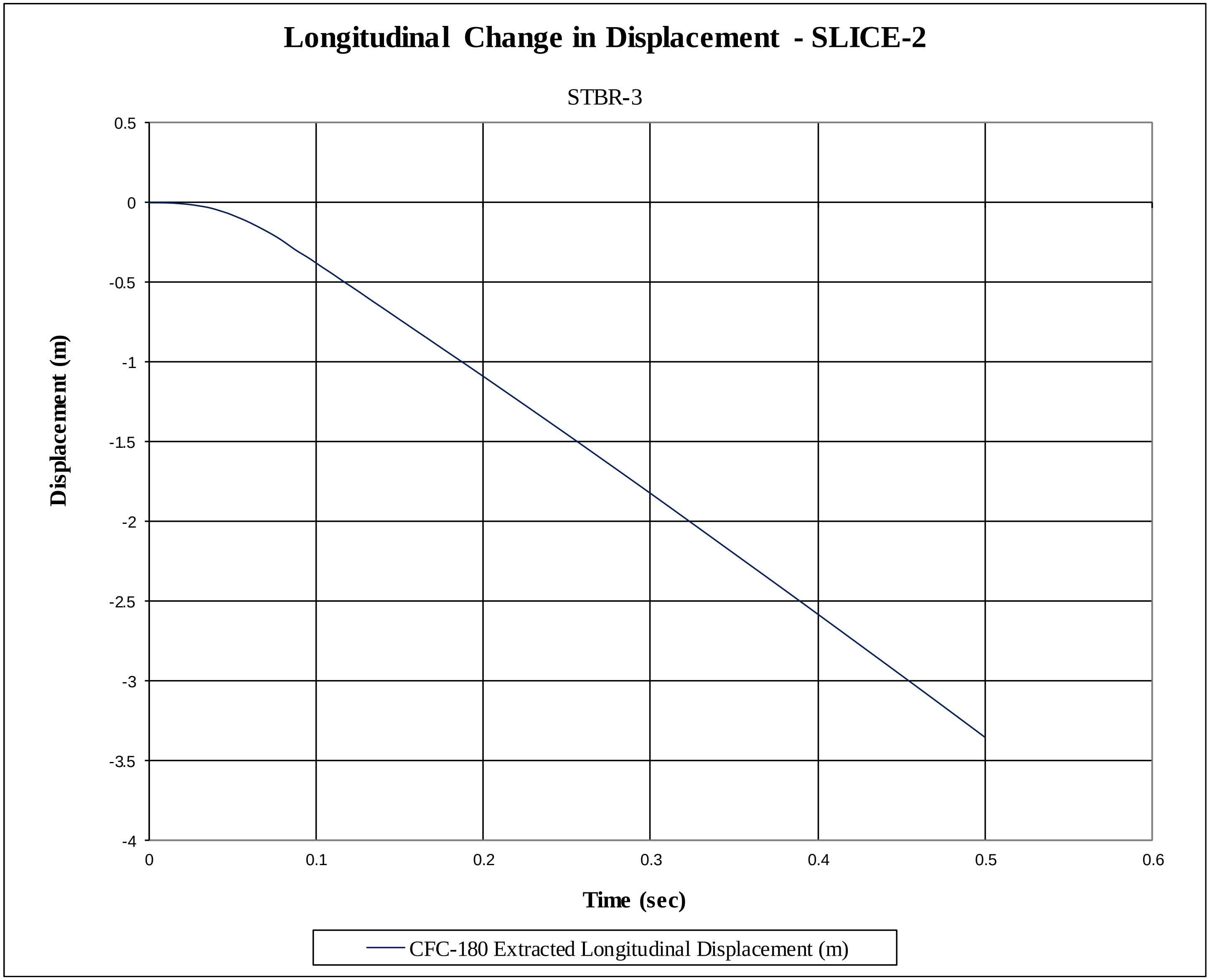


Figure I-11. Longitudinal Occupant Displacement (SLICE-2), Test No. STBR-3

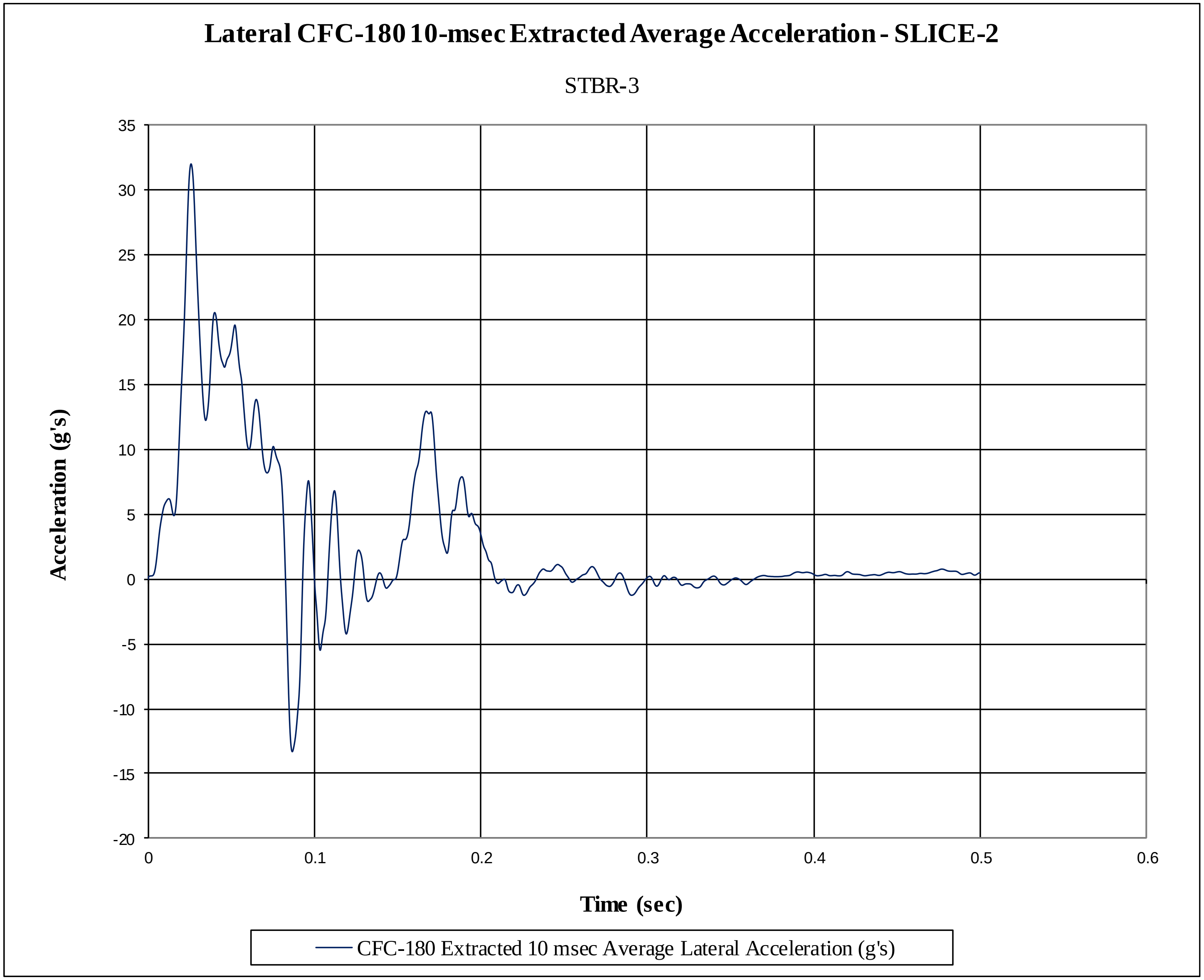


Figure I-12. 10-ms Average Lateral Deceleration (SLICE-2), Test No. STBR-3

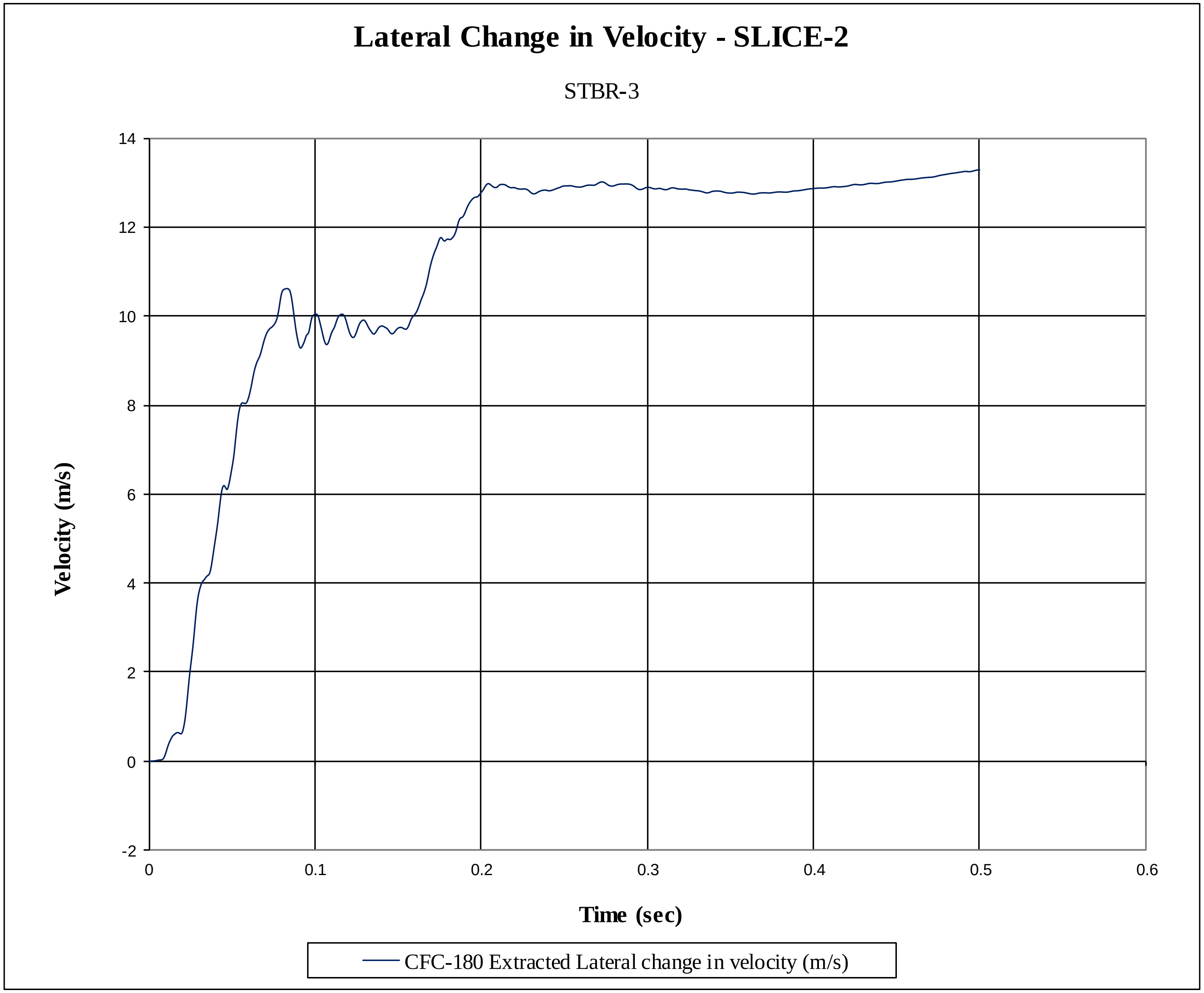


Figure I-13. Lateral Occupant Impact Velocity (SLICE-2), Test No. STBR-3

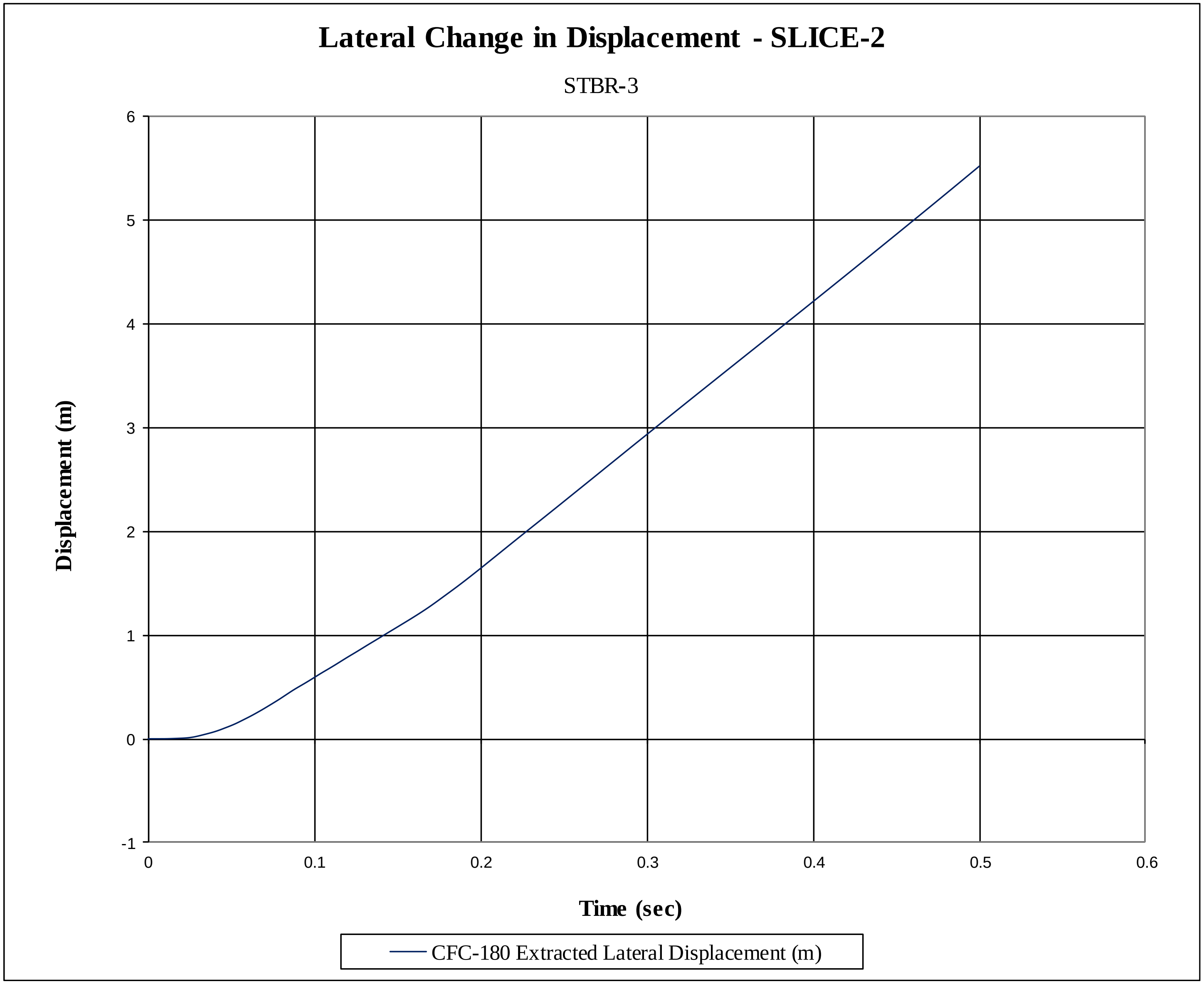


Figure I-14. Lateral Occupant Displacement (SLICE-2), Test No. STBR-3

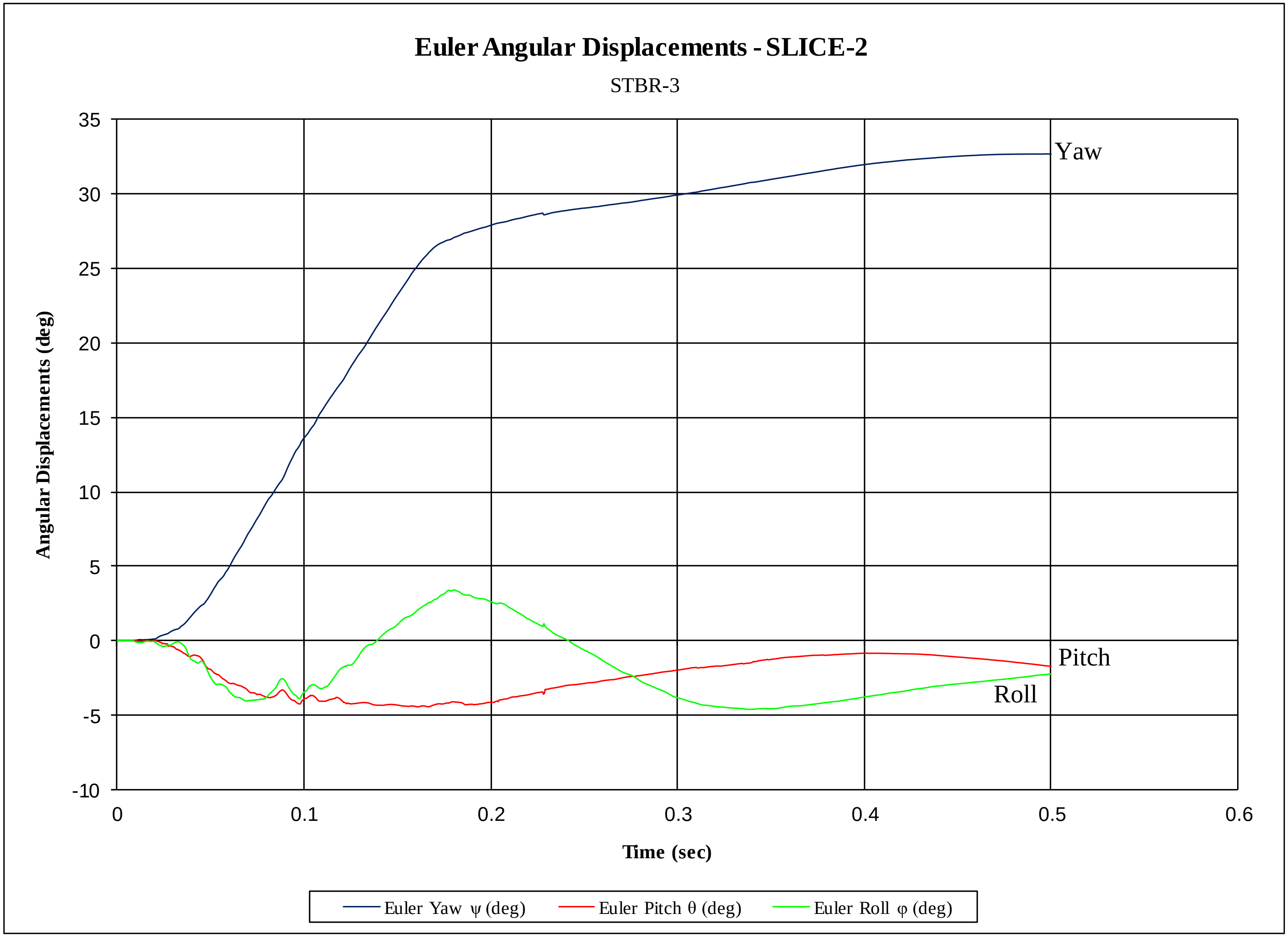


Figure I-15. Vehicle Angular Displacement (SLICE-2), Test No. STBR-3

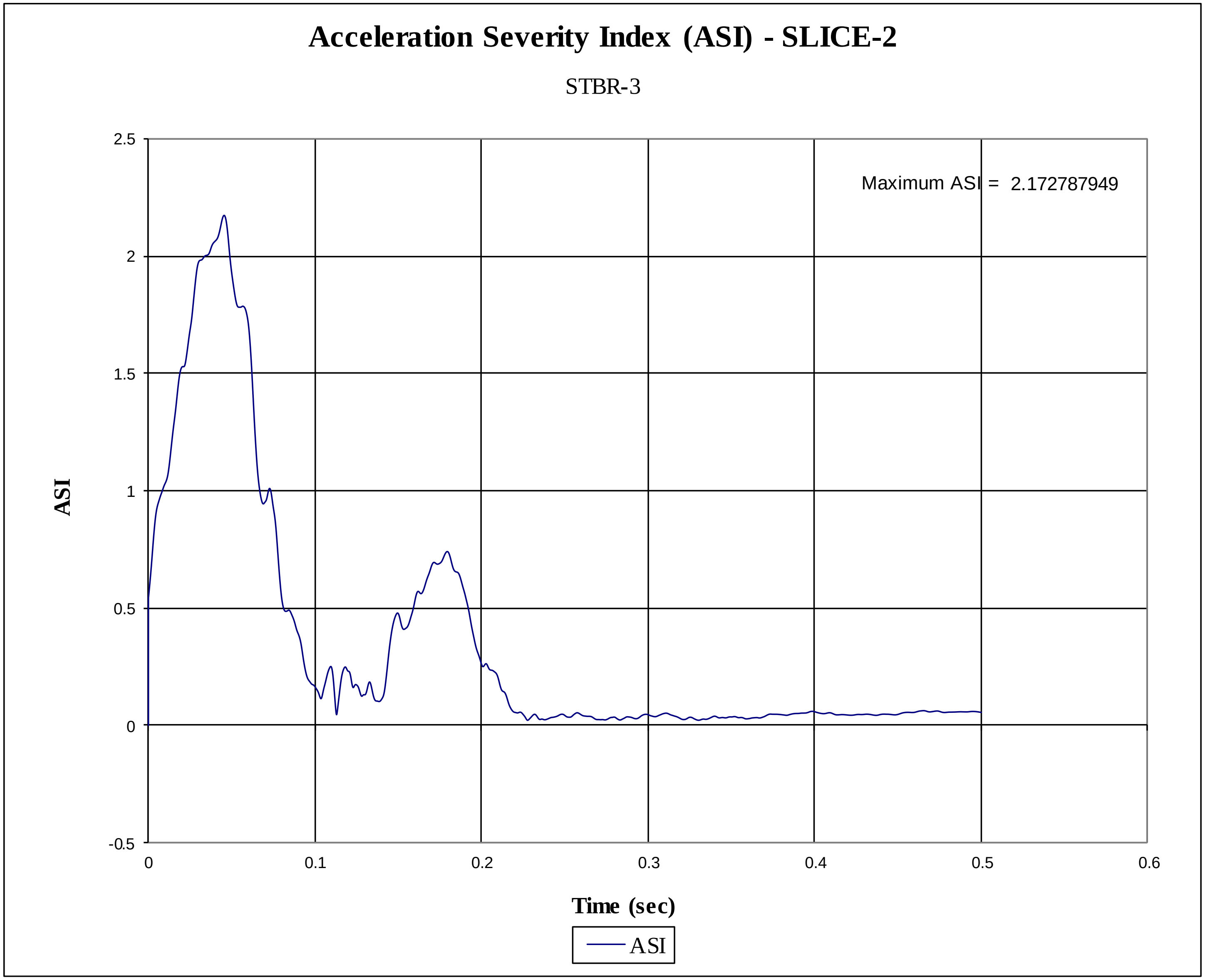


Figure I-16. Acceleration Severity Index (SLICE-2), Test No. STBR-3

Appendix J. Accelerometer and Rate Transducer Data Plots, Test No. STBR-4

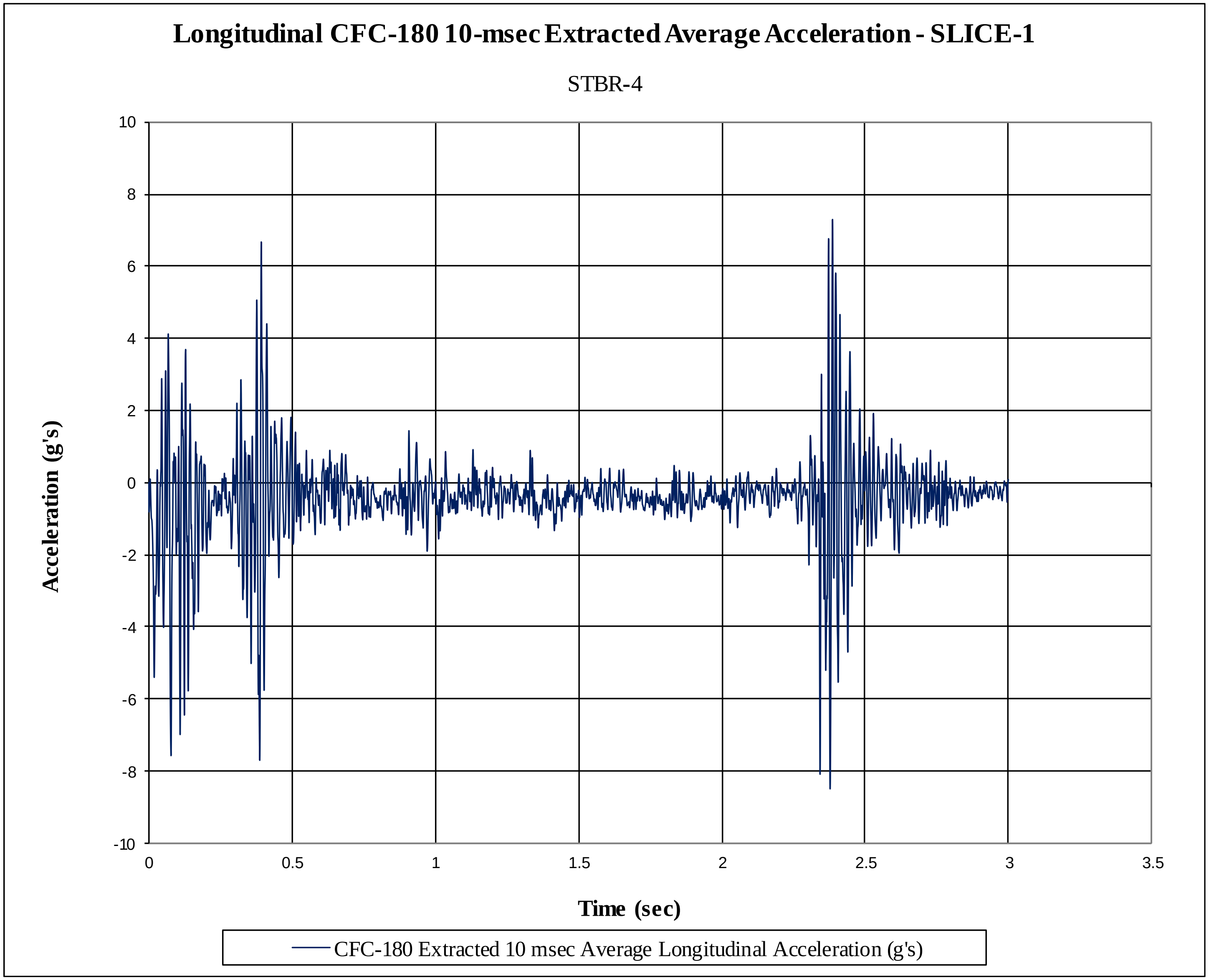


Figure J-1. 10-ms Average Longitudinal Deceleration (SLICE-1), Test No. STBR-4

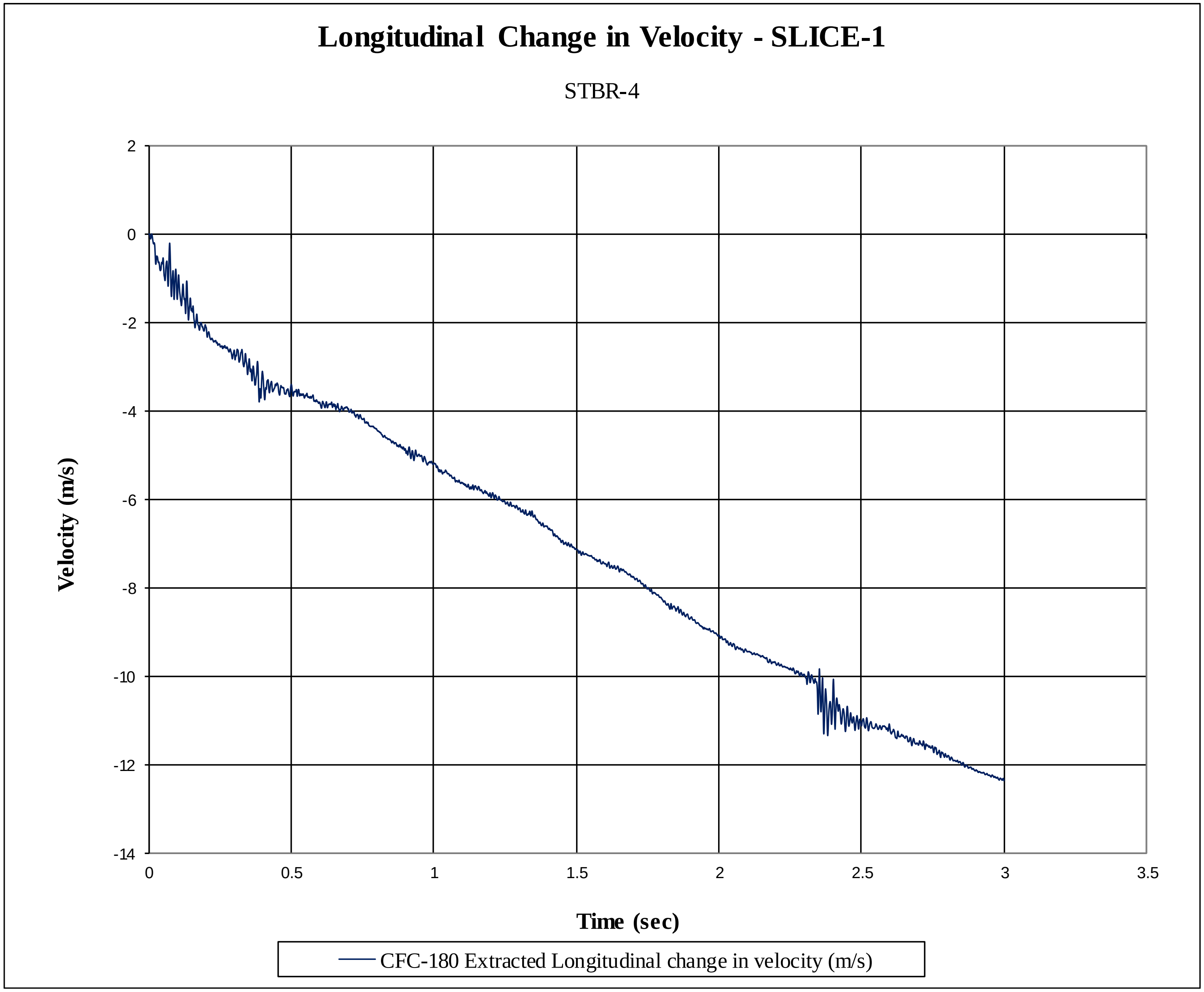


Figure J-2. Longitudinal Occupant Impact Velocity (SLICE-1), Test No. STBR-4

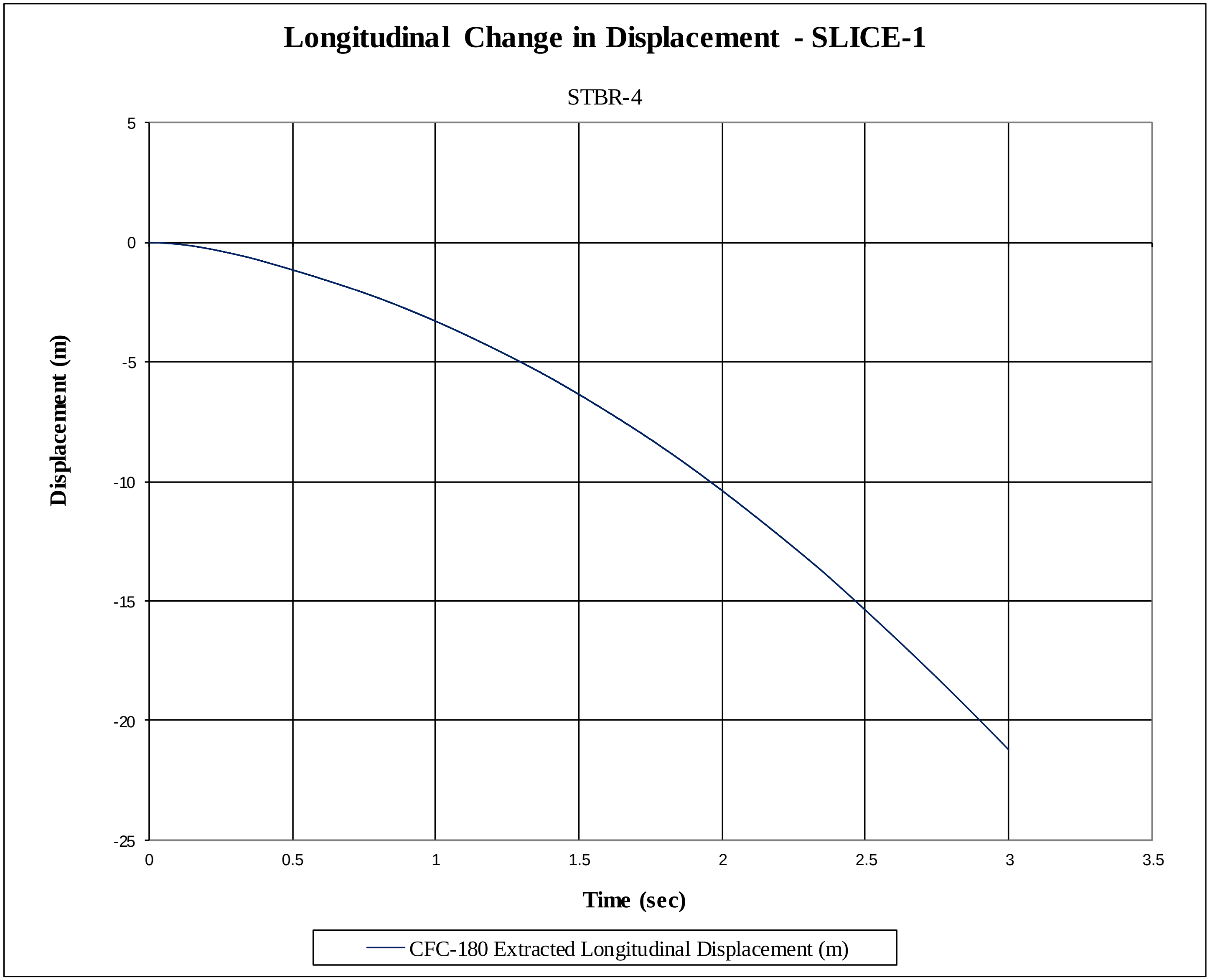


Figure J-3. Longitudinal Occupant Displacement (SLICE-1), Test No. STBR-4

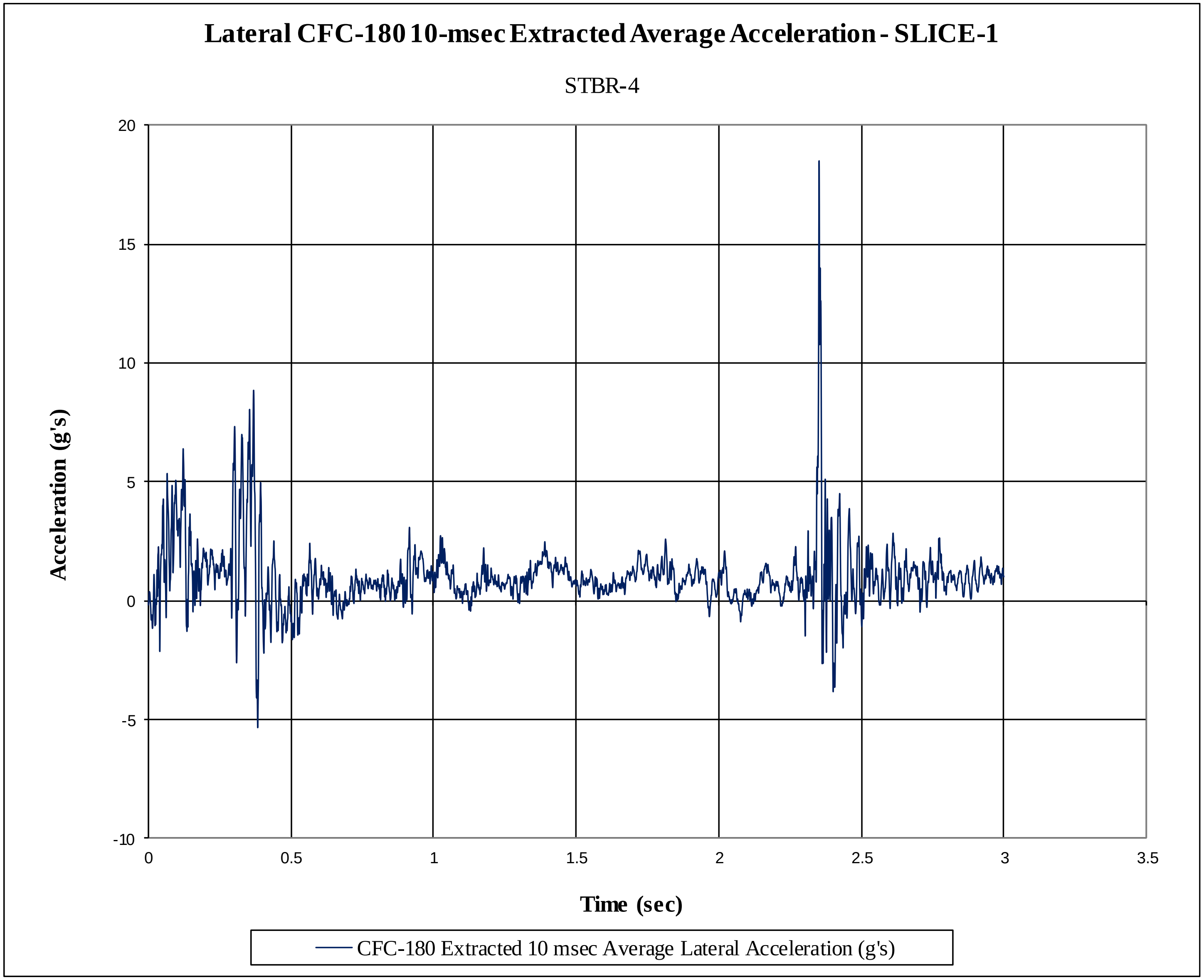


Figure J-4. 10-ms Average Lateral Deceleration (SLICE-1), Test No. STBR-4

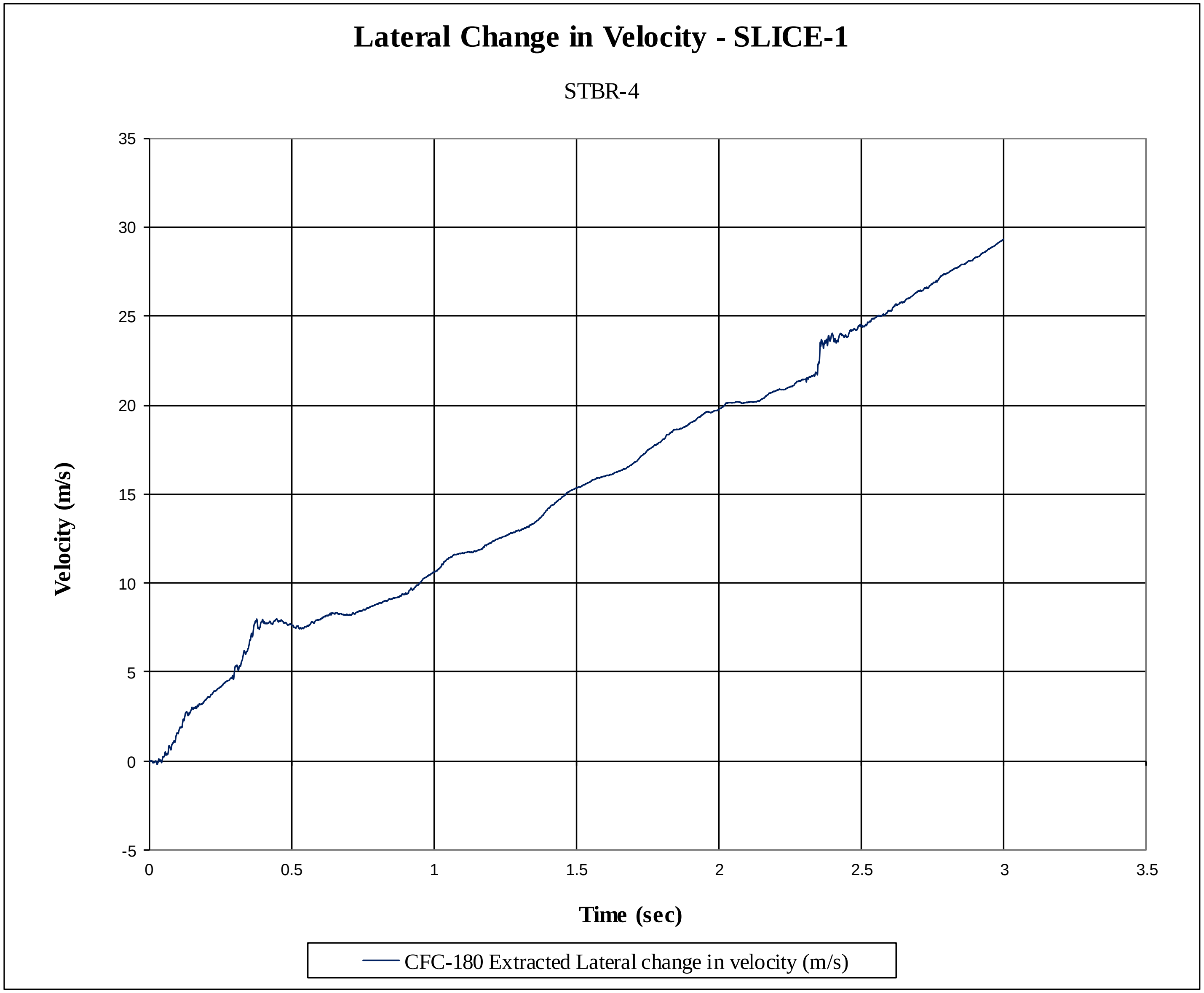


Figure J-5. Lateral Occupant Impact Velocity (SLICE-1), Test No. STBR-4

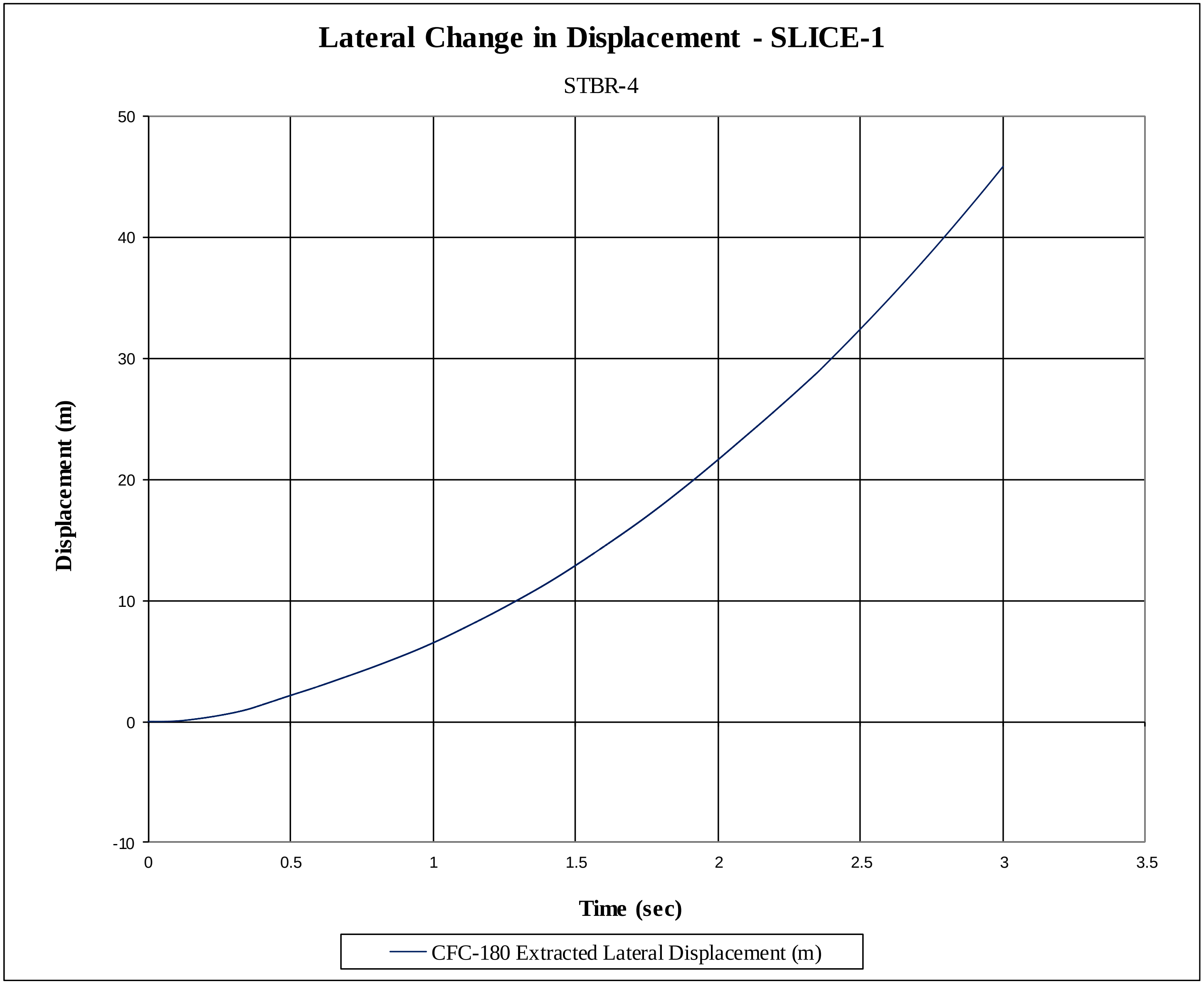


Figure J-6. Lateral Occupant Displacement (SLICE-1), Test No. STBR-4

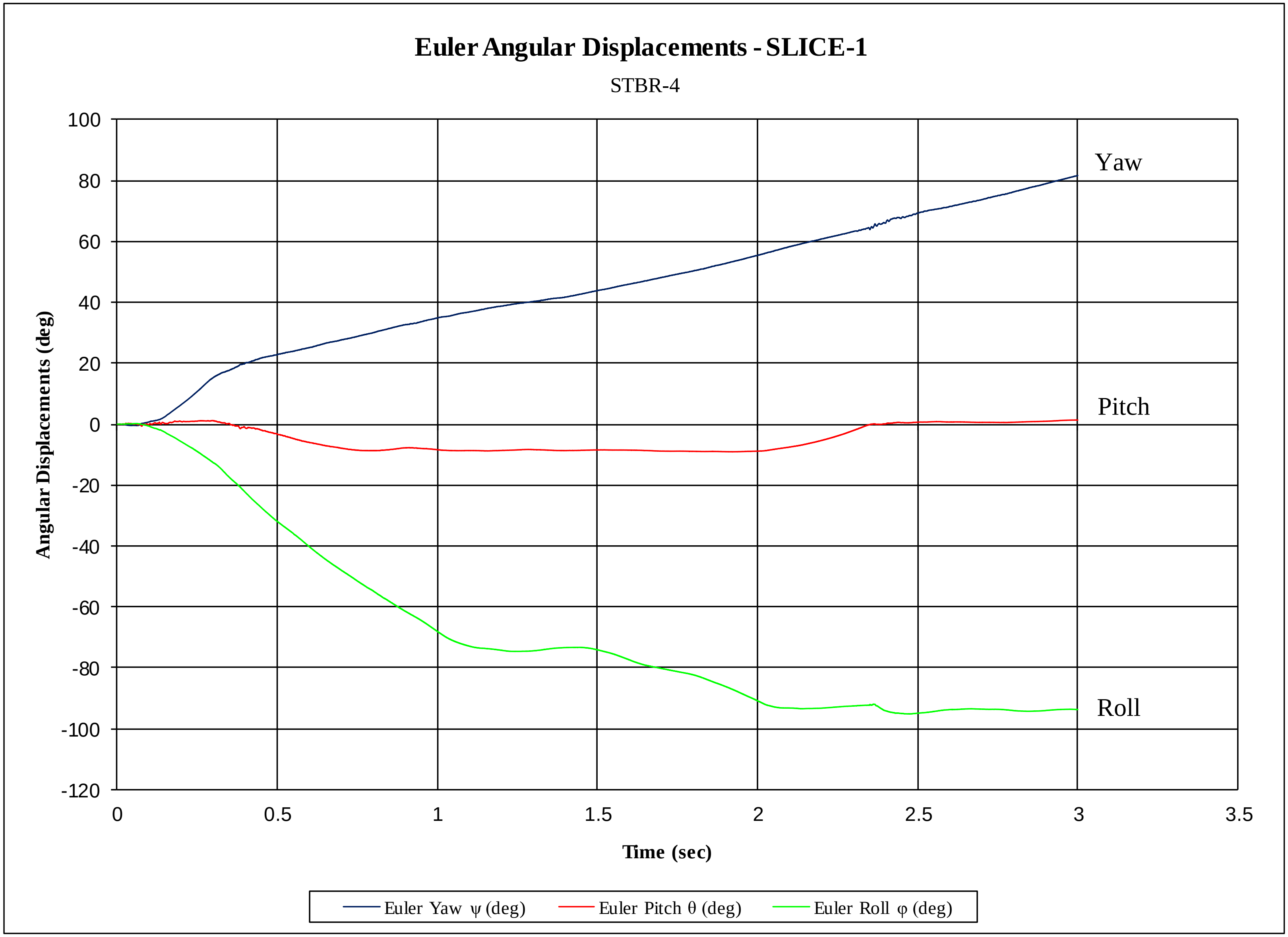


Figure J-7. Vehicle Angular Displacement (SLICE-1), Test No. STBR-4

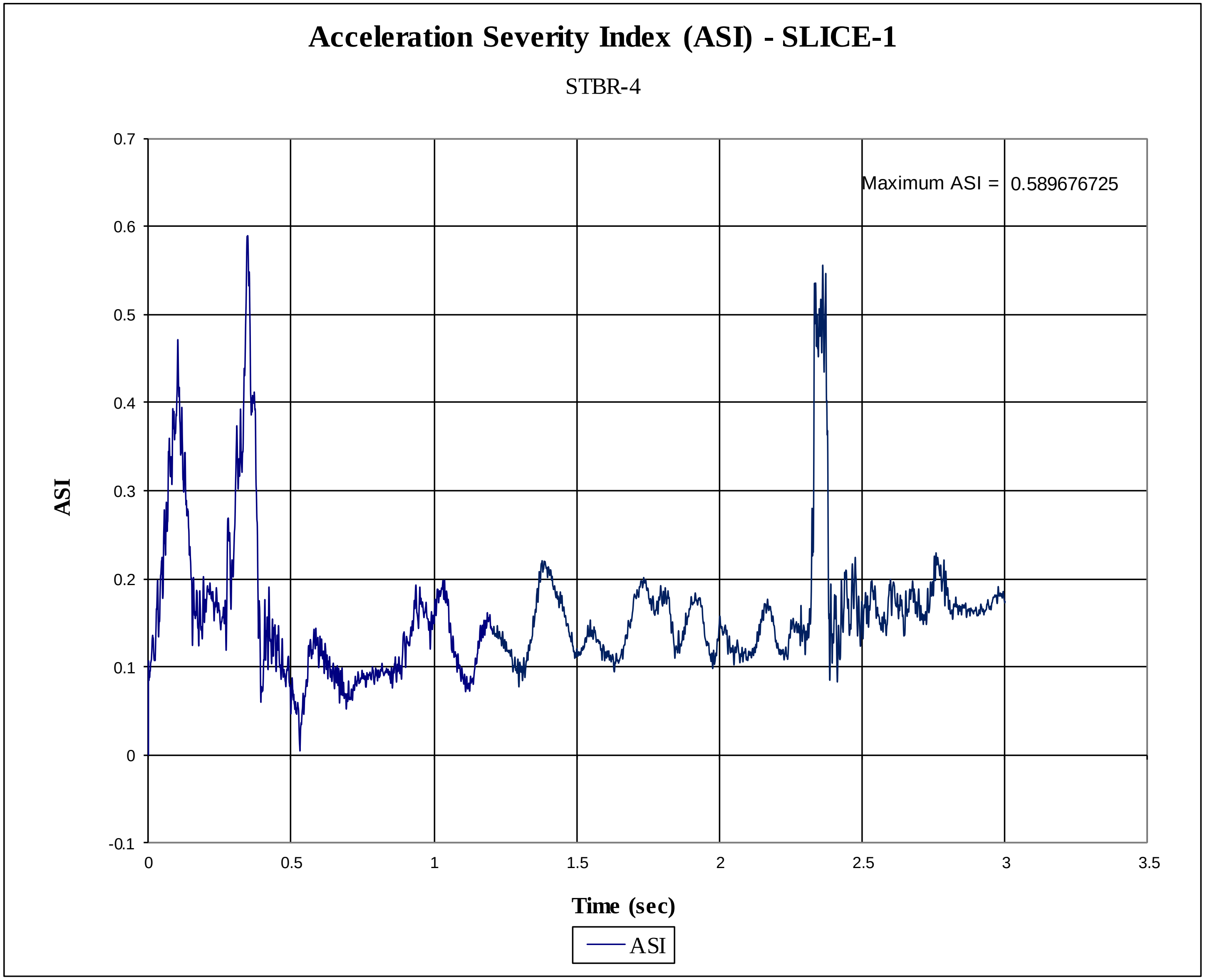


Figure J-8. Acceleration Severity Index (SLICE-1), Test No. STBR-4

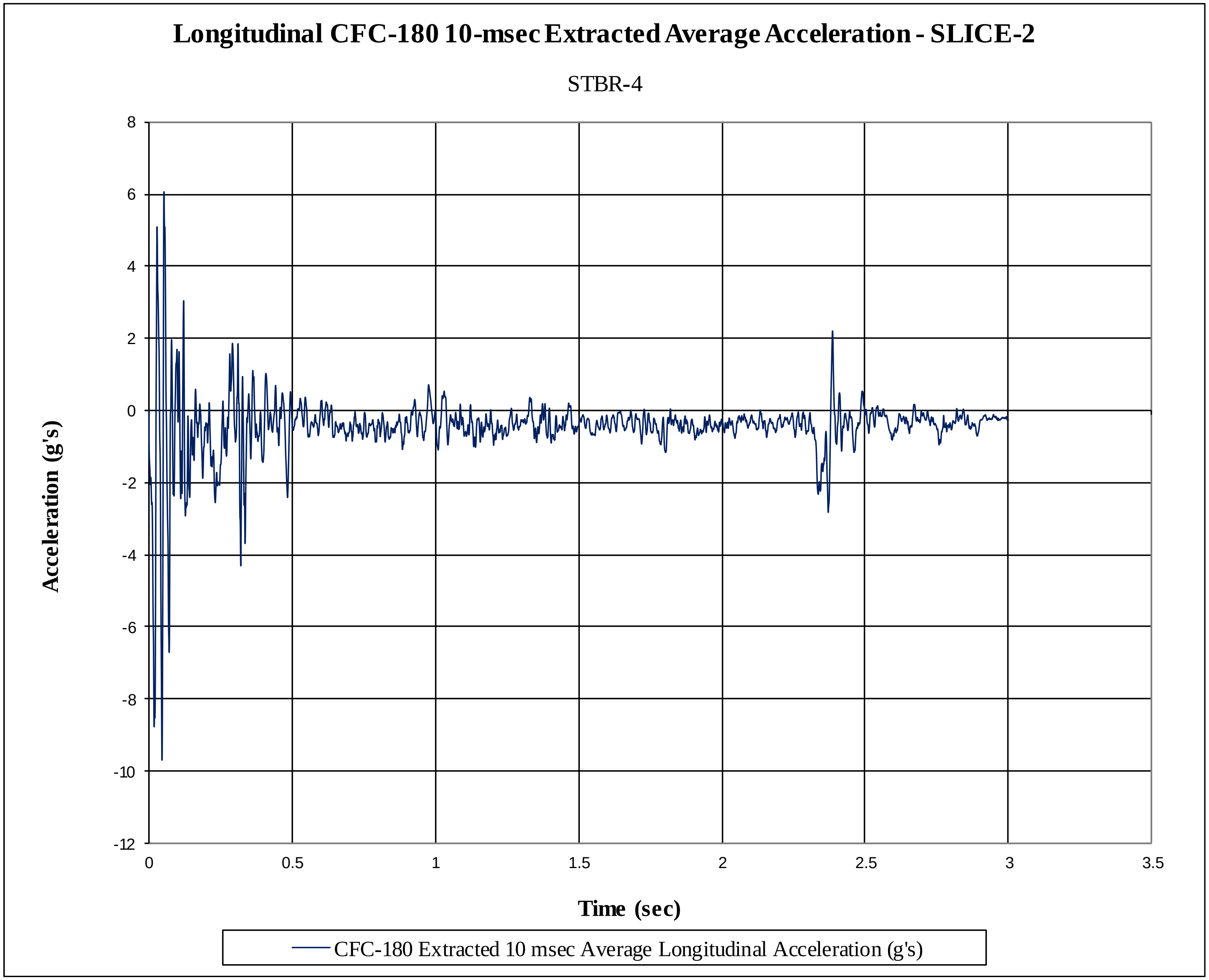


Figure J-9. 10-ms Average Longitudinal Deceleration (SLICE-2), Test No. STBR-4

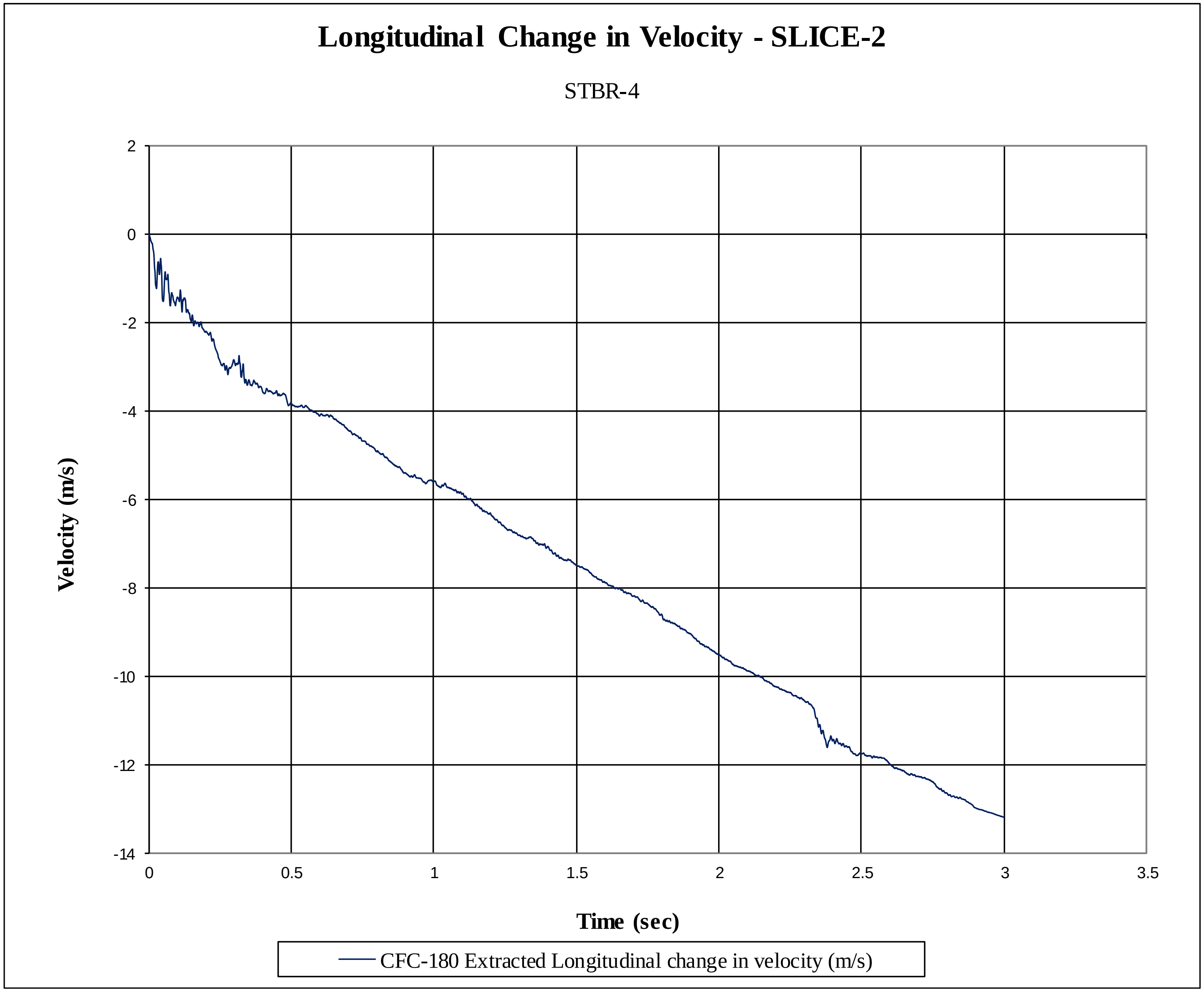


Figure J-10. Longitudinal Occupant Impact Velocity (SLICE-2), Test No. STBR-4

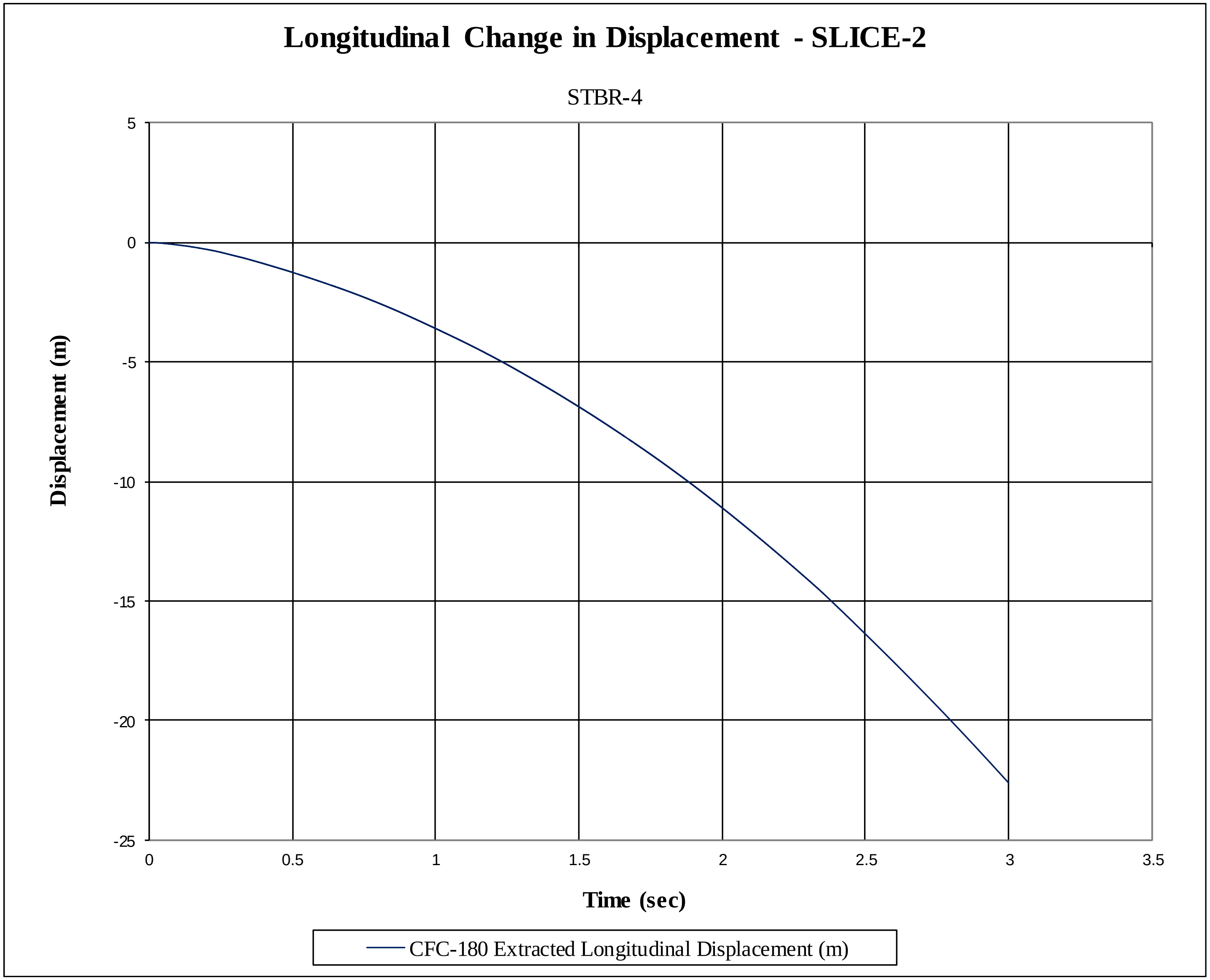


Figure J-11. Longitudinal Occupant Displacement (SLICE-2), Test No. STBR-4

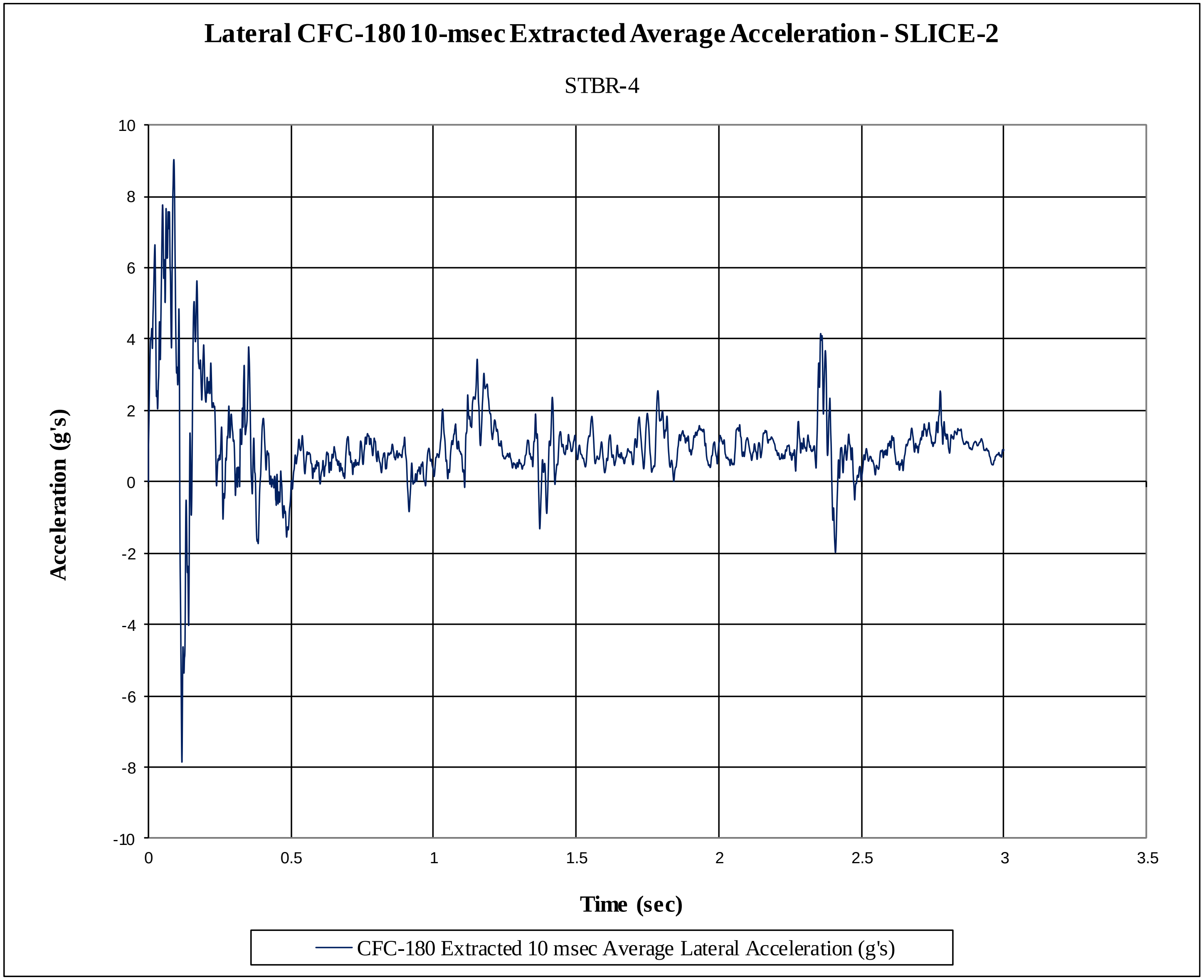


Figure J-12. 10-ms Average Lateral Deceleration (SLICE-2), Test No. STBR-4

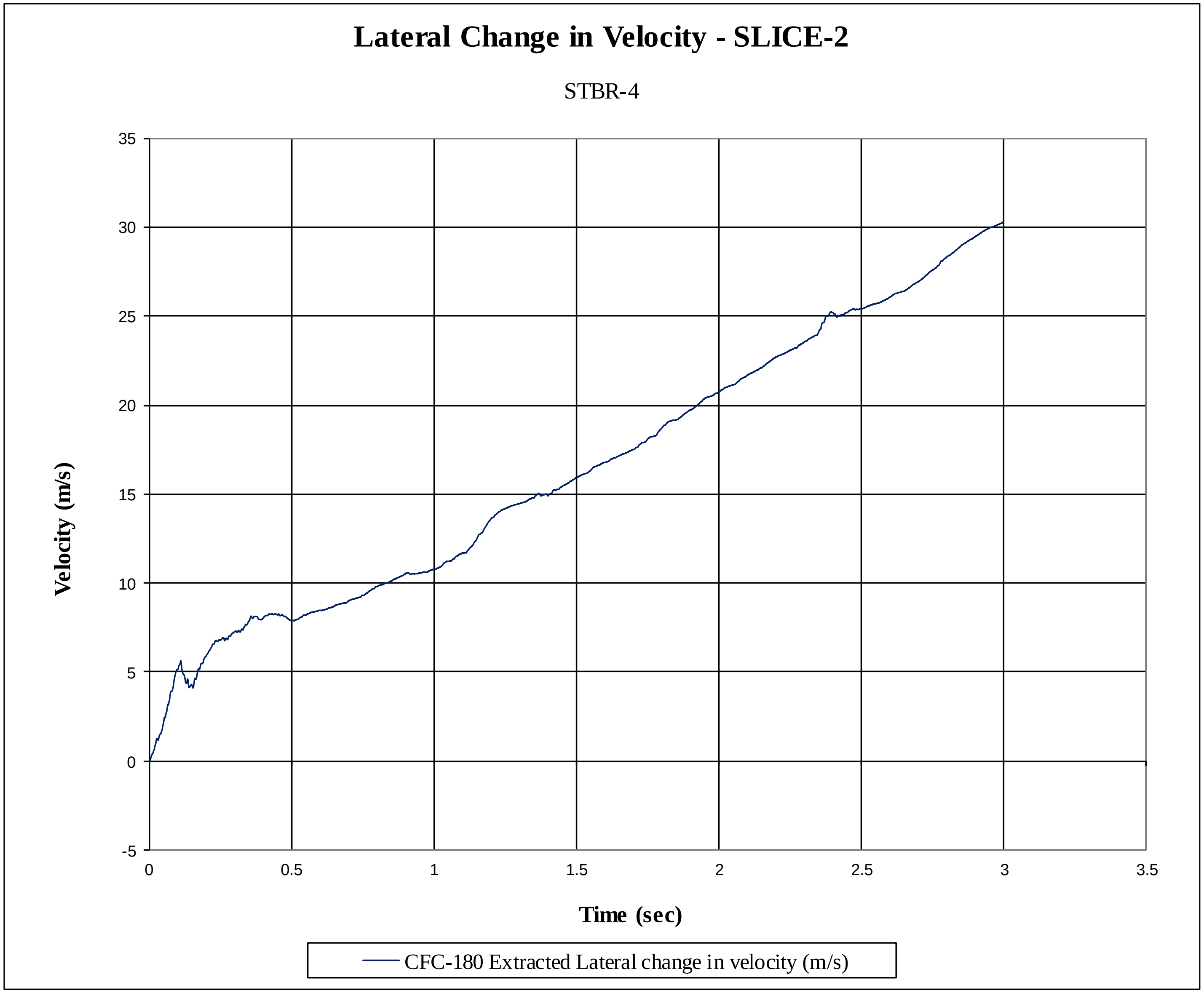


Figure J-13. Lateral Occupant Impact Velocity (SLICE-2), Test No. STBR-4

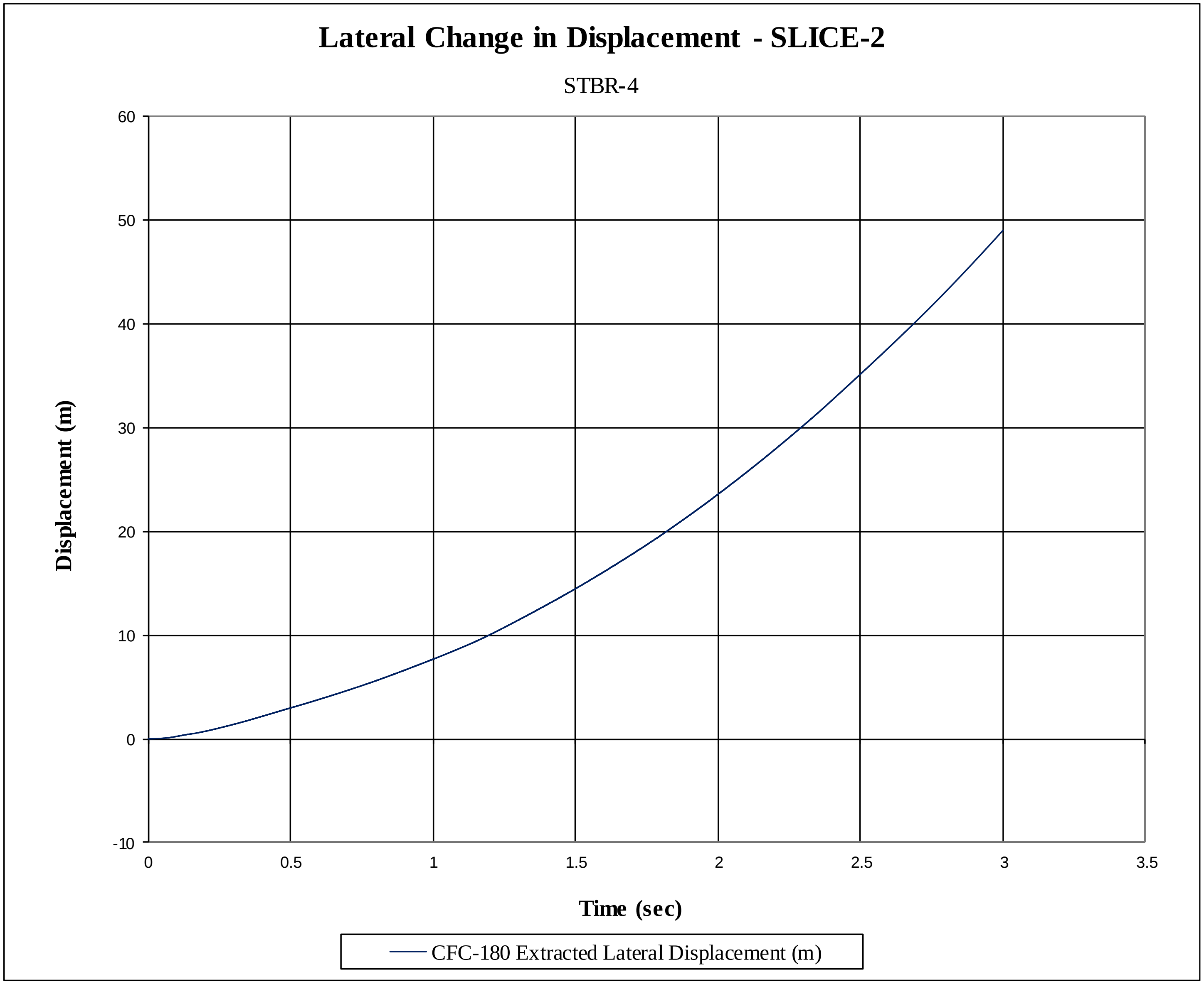


Figure J-14. Lateral Occupant Displacement (SLICE-2), Test No. STBR-4

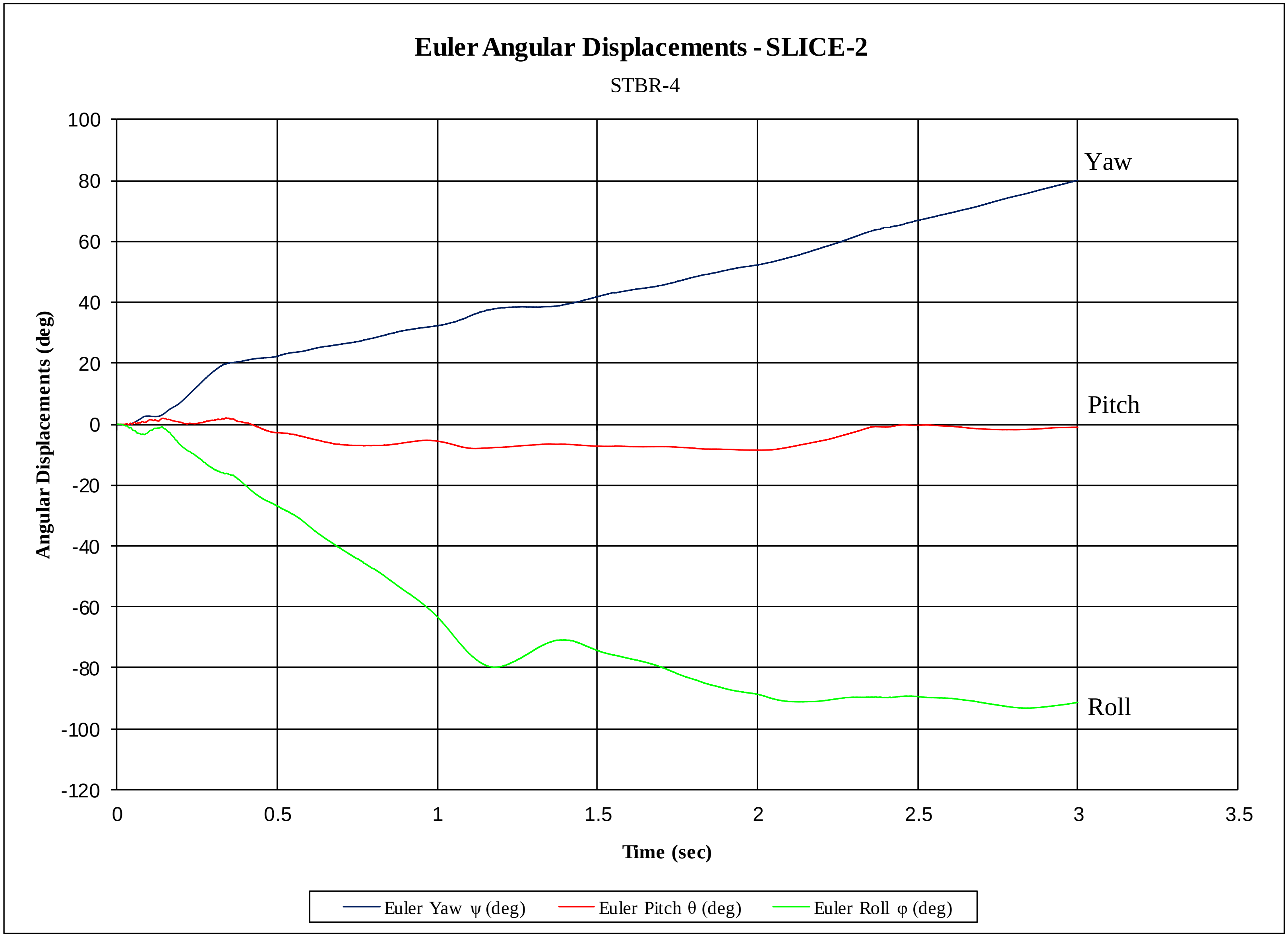


Figure J-15. Vehicle Angular Displacement (SLICE-2), Test No. STBR-4

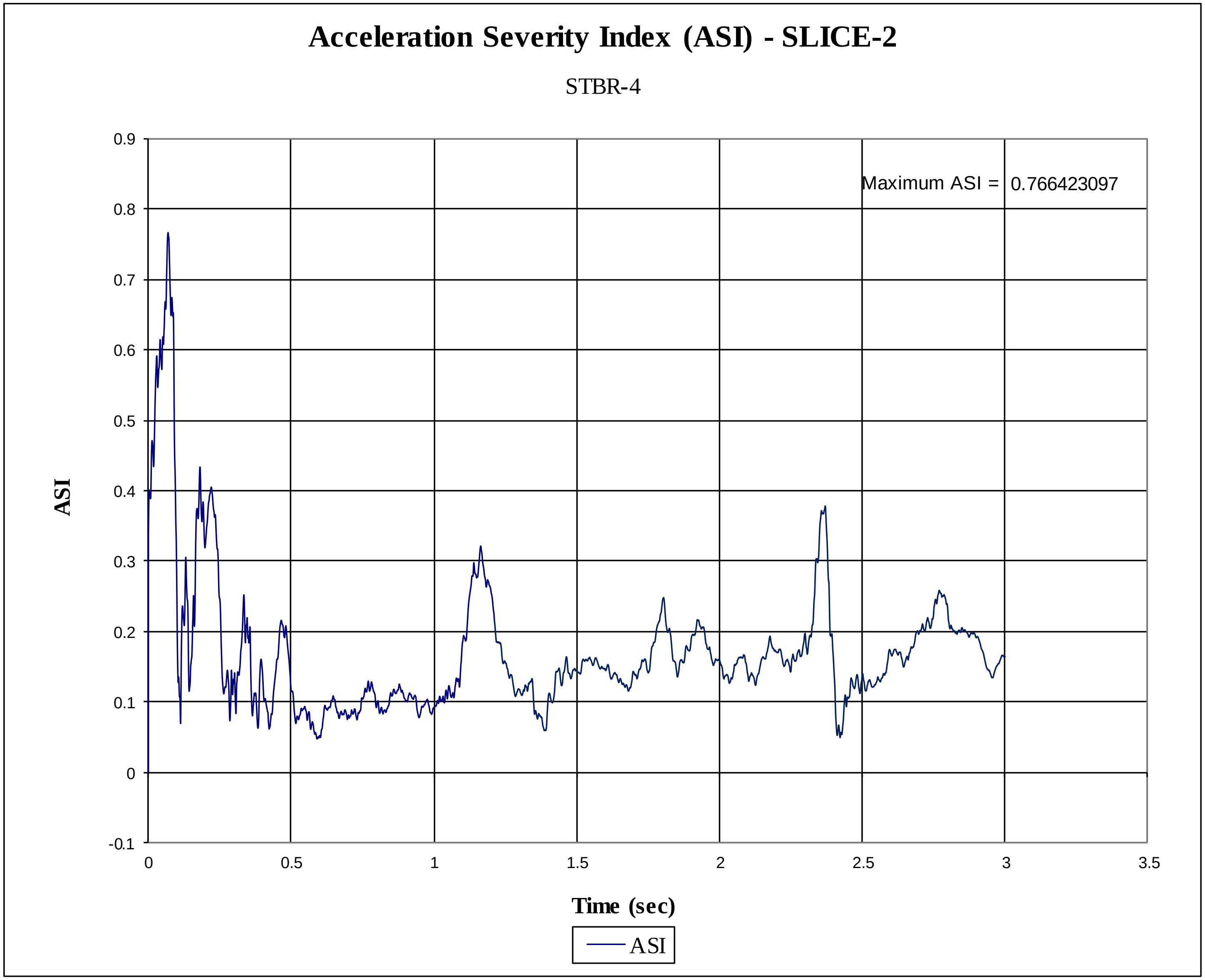


Figure J-16. Acceleration Severity Index (SLICE-2), Test No. STBR-4

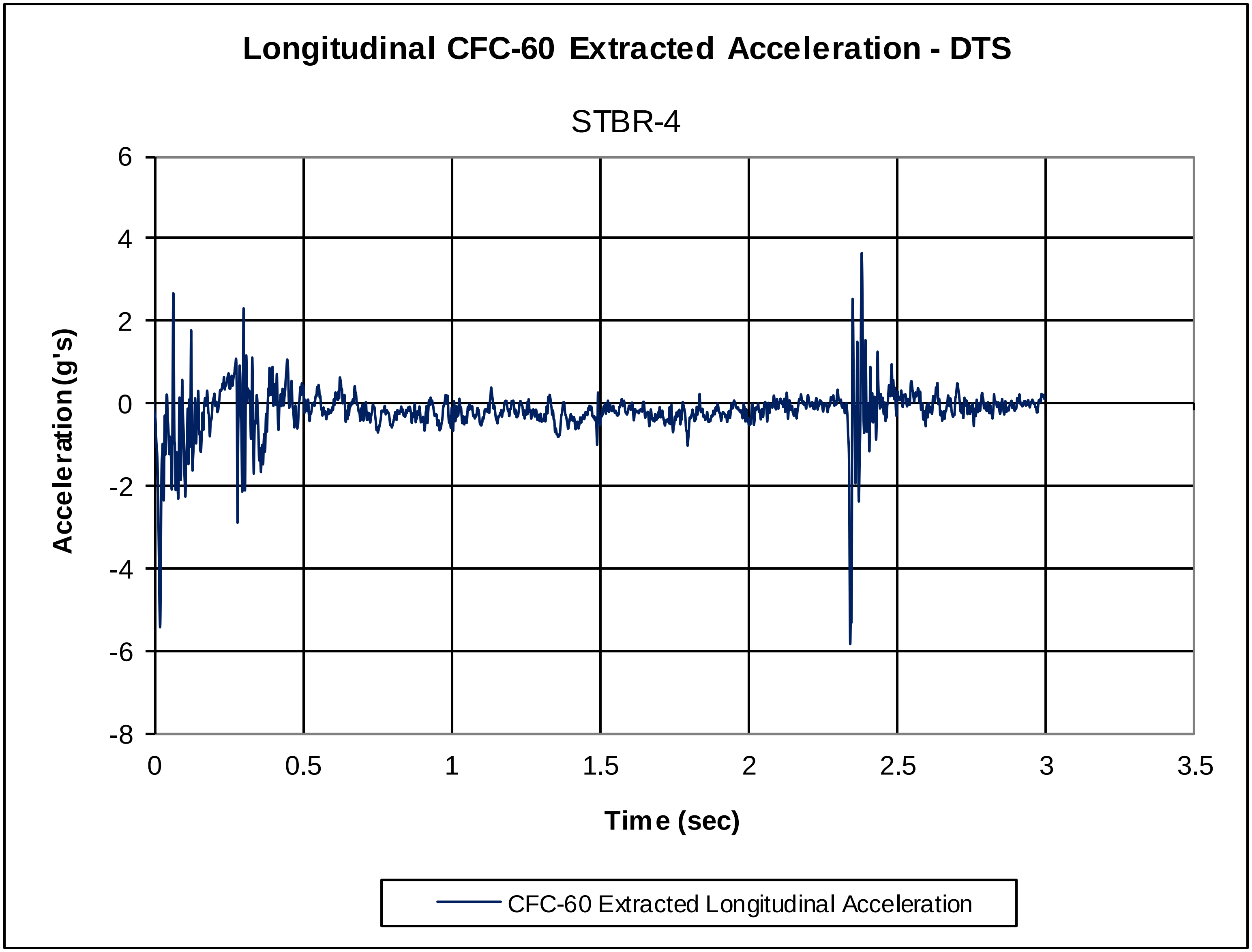


Figure J-17. Longitudinal CFC-60 Extracted Deceleration (DTS), Test No. STBR-4

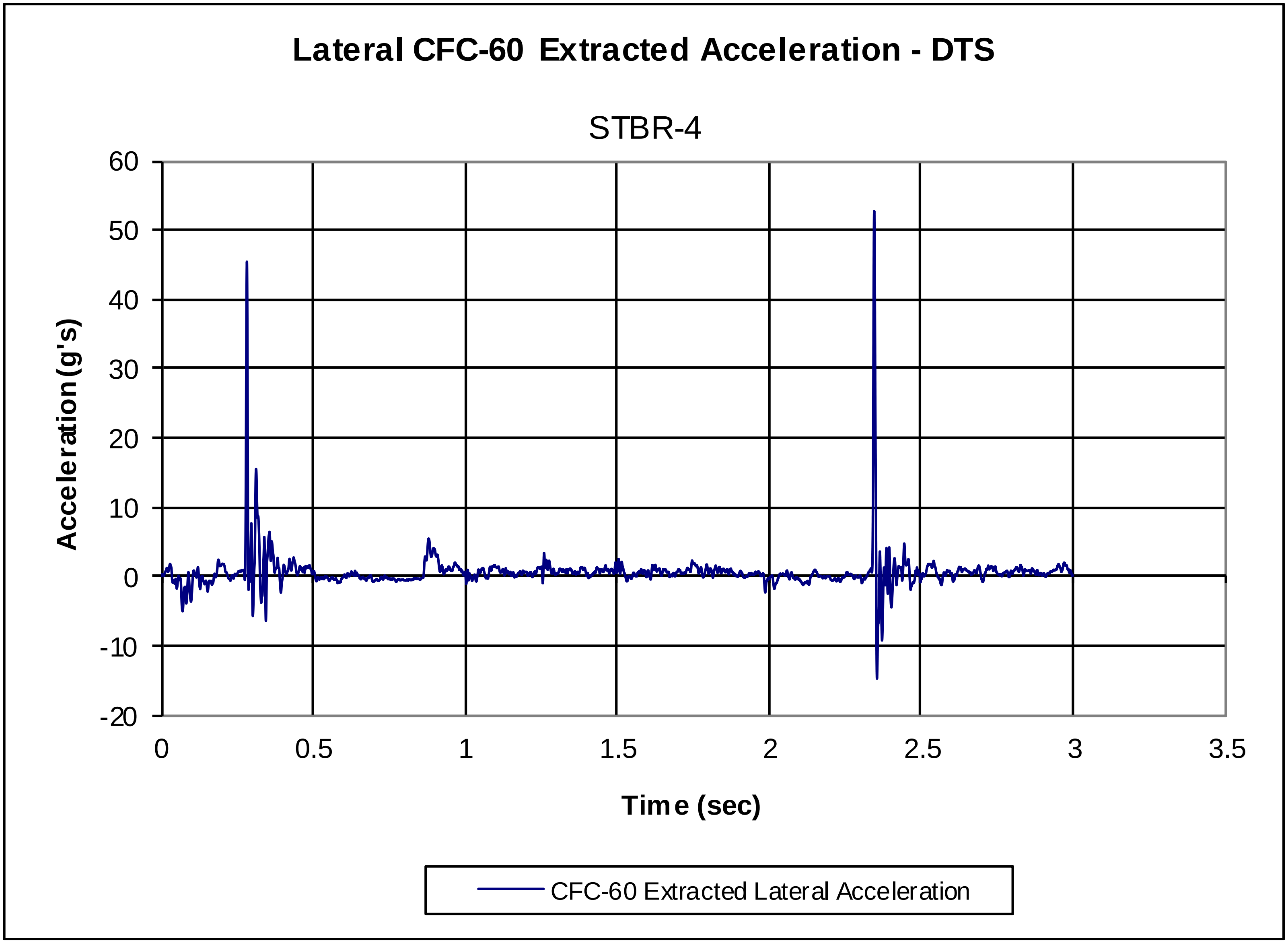


Figure J-18. Lateral CFC-60 Extracted Deceleration (DTS), Test No. STBR-4

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