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ATTACHMENT OF A COMBINATION BRIDGE RAIL TO CONCRETE PARAPET UTILIZING EPOXY ADHESIVE ANCHORS

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ATTACHMENT OF A COMBINATION BRIDGE RAIL TO CONCRETE PARAPET UTILIZING EPOXY ADHESIVE ANCHORS

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16. Abstract <p>The Iowa Department of Transportation (IaDOT) was interested in investigating the use of epoxy adhesive anchorages for the attachment of posts used in the BR27C combination bridge rail system. Alternative anchorage concepts were developed using a modified version of the ACI 318-11 procedures for embedded anchor design. Four design concepts were developed for review by IaDOT, including: (1) a four-bolt square anchorage, (2) a four-bolt spread anchorage, (3) a two-bolt centered anchorage, and (4) a two-bolt offset anchorage. IaDOT representatives selected the four-bolt spread anchorage and the two-bolt offset anchorage as the preferred designs for evaluation. In addition to these two proposed configurations, IaDOT also requested that the researchers evaluate a third option that had been previously installed on the US-20 bridge near Hardin, IA.</p> <p>The proposed alternative anchorages and the original cast-in-place anchorage for the BR27C combination bridge rail were evaluated through dynamic component testing. The test of the original cast-in-place anchorage was used as a baseline for comparison with the alternative designs. Test no. IBP-1 of the original cast-in-place anchorage developed a peak load of 22.9 kips (101.9 kN) at a deflection of 1.5 in. (38 mm). All three of the tested alternative anchorages provided greater load capacity than the original cast-in-place design and were deemed acceptable surrogates. Of the three alternative designs, the two-bolt offset design was deemed the best option.</p>			
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UNCERTAINTY OF MEASUREMENT STATEMENT

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

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1 INTRODUCTION

1.1 Background

Combination bridge rails are commonly used by many state departments of transportation and often consist of a concrete parapet with an upper steel railing system. In the past, these types of bridge rails have typically been designed with the steel posts attached to the concrete parapet using a cast-in-place anchorage system. While cast-in-place anchors have performed well, they have several disadvantages, including added complexity and construction costs, as well as issues with dimensional tolerances regarding their placement in the parapet.

The Iowa Department of Transportation (IaDOT) was interested in investigating the use of epoxy adhesive anchorages for the attachment of posts used in combination bridge rails. IaDOT desired an alternative anchorage method for the attachment of the steel beam-and-post system to a concrete parapet on the BR27C combination bridge rail system. An alternative epoxy adhesive connection detail was proposed, as shown in Figure 1. The Midwest Roadside Safety Facility (MwRSF) performed initial calculations to evaluate the capacity of the epoxy anchorage based on a previous MwRSF research study involving the dynamic component testing of anchors [1] and applying the methodologies found in ACI 318-11 [2]. From this preliminary analysis, it was found that the capacity of the proposed anchorage was potentially insufficient. However, the methodology provides conservative results and may underestimate anchorage capacity. As such, it was noted that the best evaluation of this proposed alternative anchorage system may be to perform dynamic component testing of the epoxy adhesive system.

IaDOT indicated that they desired an alternative epoxy adhesive anchorage system for the BR27C combination bridge railing, as well as evaluation of an epoxy adhesive anchorage system for the BR27C previously used on an existing bridge on US-20 in Iowa.

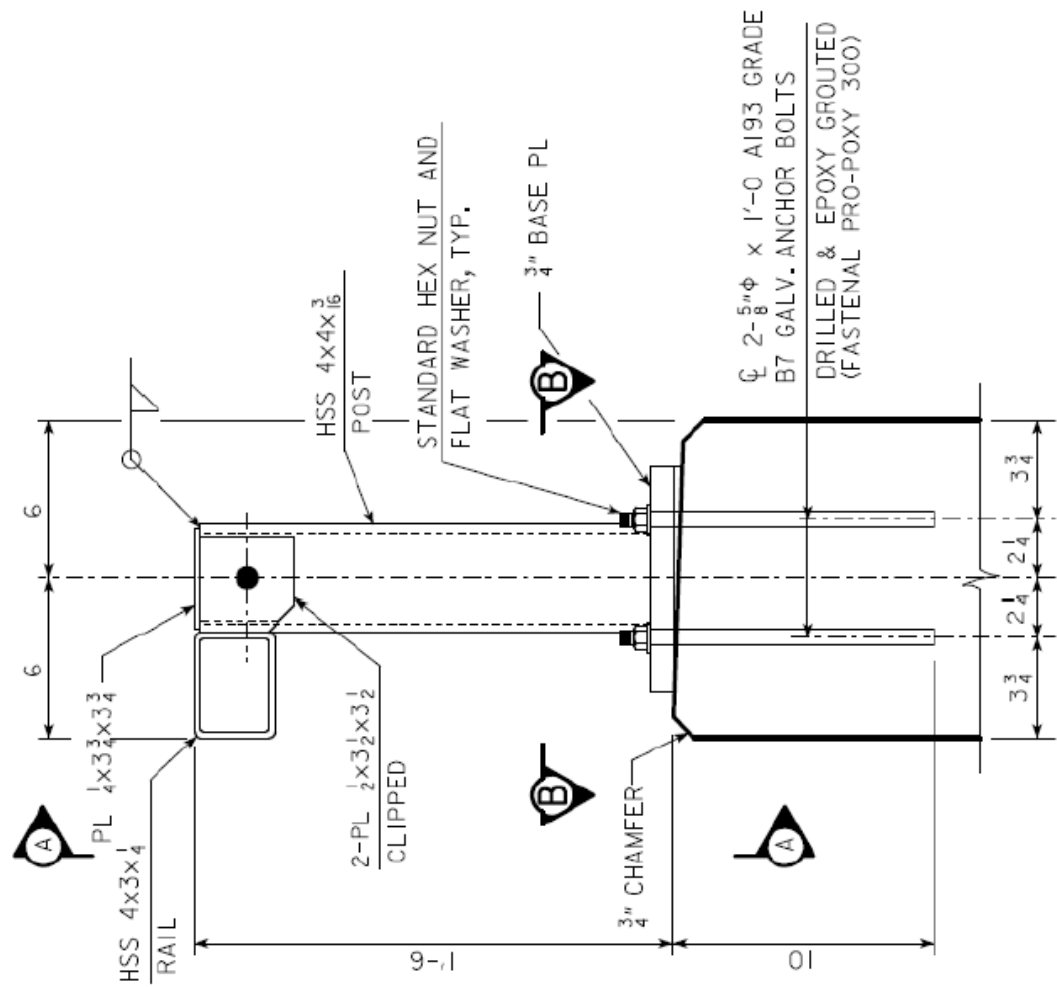


Figure 1. Proposed BR27C Combination Rail Attachment

1.2 Objective

The research objective was to design and evaluate alternative epoxy adhesive anchorages for use in the IaDOT BR27C combination bridge rail system. The alternative epoxy adhesive anchorages were to have equal or greater capacity than the current cast-in-place anchorage, so that they can be used in new construction or as a retrofit to modify existing bridge railings. The proposed epoxy attachment designs were to be evaluated through dynamic component testing to verify their capacity.

1.3 Scope

The research effort consisted of design, testing, and evaluation of alternative epoxy adhesive anchorages for attaching the beam and post system of the BR27C combination bridge railing to a concrete parapet. MwRSF researchers reviewed the current cast-in-place anchorage design and developed alternative epoxy adhesive anchorage configurations, including inline anchor systems and a four-anchor system similar to the cast in place configuration but with spacing more compatible with the epoxy adhesive. The alternative epoxy adhesive anchorage systems were submitted to IaDOT for review and selection of preferred systems to be tested and evaluated.

Dynamic component testing was used to evaluate the selected epoxy adhesive anchorages and to demonstrate that the capacities of the proposed epoxy anchorages were equal to or greater than the existing cast-in-place anchorage system. The capacity of the current cast-in-place anchorage had not been fully quantified with testing. Thus, one dynamic component test was performed on a bridge rail post using the current cast-in-place anchorage configuration. Additional dynamic component tests were performed on the proposed alternative epoxy adhesive anchorage systems. The target impact conditions for all tests would be identical, and the tests were configured so that the applied impact load occurred at a height on the post that produced a

bending moment and combined loading on the anchorage system similar to that provided during vehicle crash events. The force versus deflection, energy dissipated versus deflection, and failure modes were documented for each test and compared to one another. These comparisons were used to verify that the proposed anchorages provided equal or greater capacities than the current anchorage, and that the alternative anchorages did not display undesirable failure modes.

IaDOT also proposed an additional test to evaluate a currently installed epoxy adhesive anchorage for the BR27C bridge rail used on the US-20 bridge near Hardin, IA. This setup was tested and analyzed using the procedures described above for the cast-in-place design and the newly designed epoxy anchorages.

2 DESIGN OF ALTERNATIVE EPOXY ADHESIVE ANCHORAGE

2.1 Design Methodology

Limited prior research has been conducted related to the use of epoxy adhesive anchors for attachment of a beam-and-post railing system to the top of concrete parapets. In 2010, Texas A&M Transportation Institute (TTI) researchers conducted a study to develop two new retrofit combination steel and concrete bridge rail designs [3]. This effort included the design of a retrofit epoxy anchorage design and pendulum testing of the anchorage system on a short section of concrete parapet in order to verify the capacity of the connection. Thus, the methodology of evaluating the alternative epoxy anchorage systems through dynamic component testing has been previously accepted.

MwRSF researchers also conducted a related study for the Wisconsin Department of Transportation involving epoxy adhesive anchors for attachment of concrete barriers to bridge decks [1]. The objective of this research was to determine if epoxy adhesive anchors could be utilized to attach concrete barriers to bridge decks and to develop design procedures for implementing epoxy adhesive anchorages into concrete bridge railings. A series of 16 dynamic bogie tests and one static test were conducted to investigate the behavior of epoxy adhesive anchors under dynamic load. Additional dynamic tests were conducted on 1½-in. (29-mm) diameter ASTM A307 threaded rods.

Comparisons were made between the results from the component tests and analytical models for epoxy adhesive anchors. The cone or full uniform bond model [4-5] and ACI 318-11 [2] procedures were both compared with the component tests in order to verify their effectiveness. Review of the comparisons between the analytical models and the tensile component tests found that both the cone and full uniform bond model and ACI 318-11 provided reasonable predictions for the failure mode of the epoxy adhesive anchors, but both methods

were conservative for the prediction of capacities (i.e., underestimated strength). The shear testing results and predicted capacities were compared, but findings were limited due to the observed failure modes in the component tests. However, it was found that ACI 318-11 provided reasonable yet conservative estimates for shear capacity of the epoxy adhesive anchors. It was also found that the proposed dynamic increase factors for concrete breakout, steel fracture, and bond strength improved the prediction of the anchor failure modes and capacities. It was recommended that the ACI 318-11 procedures be combined with the proposed dynamic increase factors for designing epoxy adhesive anchors. Recommendations for future research were made to fill gaps in the existing research effort and to evaluate the conservative nature of the proposed design methodology.

Based on the previous research on epoxy adhesive anchorages, it was proposed to design several potential alternatives for the BR27C combination rail anchorage using the analytical procedures developed during the Wisconsin study. Then IaDOT could select the alternative anchorage designs they found most desirable, and dynamic component testing would be performed to verify their capacity.

2.2 IaDOT BR27C Combination Bridge Rail

The BR27C combination bridge rail design was originally developed and tested at the Texas A&M Transportation Institute in 1993 [6]. The bridge rail design consisted of a 24-in. (610-mm) tall by 10-in. (254-mm) thick vertical concrete parapet, with the combination rail mounted on top of the parapet, as shown in Figure 2. Both the sidewalk- and bridge deck-mounted versions of the combination bridge rail were subjected to three full-scale crash tests according to Performance Level 2 (PL-2) of the *AASHTO Guide Specifications for Bridge Railings* [7]. The three full-scale crash tests included:

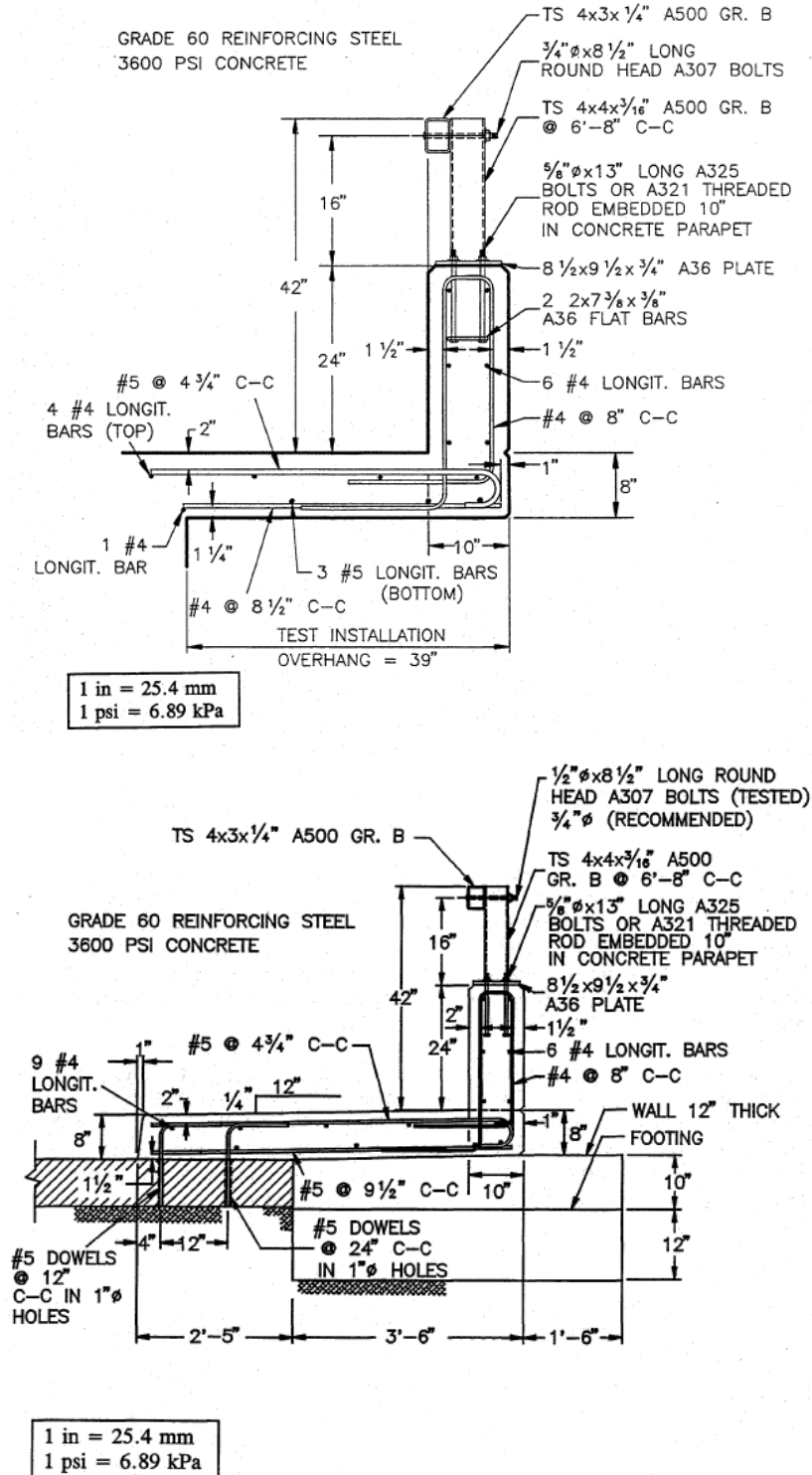


Figure 2. BR27C Design on Concrete Bridge Deck and Sidewalk

1. Impact of an 1,800-lb (817-kg) small car at 60 mph (96.6 km/h) and an angle of 20 degrees.
2. Impact of a 5,400-lb (2,452-kg) pickup truck at 60 mph (96.6 km/h) and an angle of 20 degrees.
3. Impact of an 18,000-lb (8,172-kg) single unit truck at 50 mph (80.5 km/h) and an angle of 15 degrees.

All six crash tests of the BR27C combination rail were successful and met the AASHTO PL-2 criteria. Damage to the combination rail and parapet was limited in the majority of the tests. One of the single-unit truck tests did show detachment of the rail from the support posts, but most of the bridge rail damage was minor, and the combination rail posts remained attached to the parapet in all of the tests.

Subsequent to the design and testing of the original BR27C combination bridge rail, the Federal Highway Administration (FHWA) released a memo regarding listings of bridge railing designs that were considered acceptable for use on federal aid projects by virtue of their previous crash test performance [8]. FHWA officials reviewed these listings and assigned each a rating that was relative to one of the six test levels suggested in NCHRP Report No. 350 [9]. In this memo, the BR27C design was listed as equivalent to NCHRP Report No. 350 Test Level 4 (TL-4).

Based on the previous testing and the FHWA memo, IaDOT has previously used the BR27C railing on their facilities. As part of recent updates to their bridge rail designs, IaDOT has switched to a slightly wider concrete parapet design that is 24 in. (610 mm) tall by 12 in. (305 mm) thick, as shown in Figure 3. As such, the revised parapet design was used for the alternative epoxy adhesive anchor designs developed as part of this research.

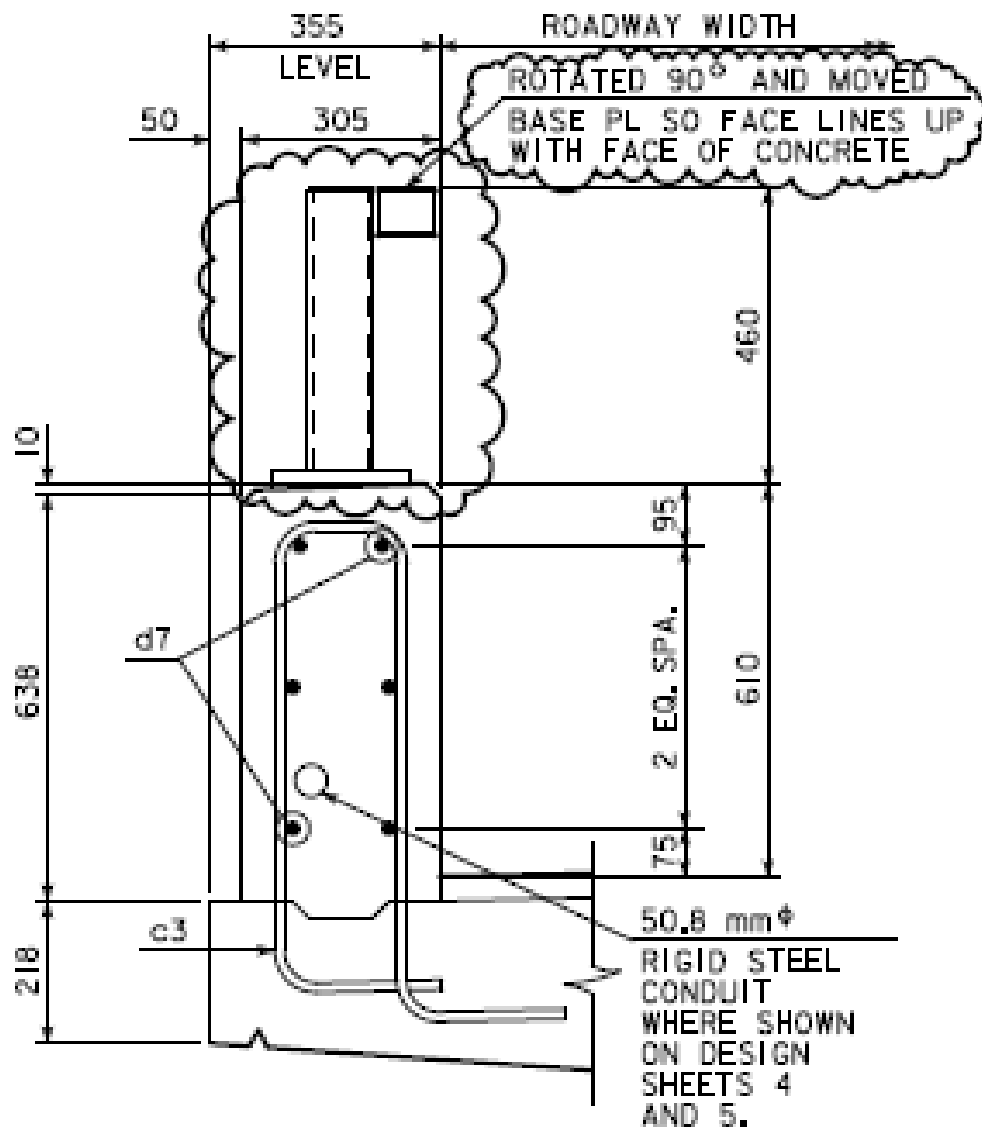


Figure 3. IaDOT Revised BR27C Parapet Design

2.3 Alternative Anchorage Design Calculations

The design of the epoxy adhesive anchorages began with determination of a design load for the post and baseplate of the BR27C combination rail. Because the exact impact loading of the BR27C rail during the original crash testing was unknown, it was assumed that the anchorage designs would need to develop the full-moment capacity of the bridge rail post. Designing the alternative anchorages to meet this load would ensure that the designs were as strong as the original cast-in-place anchorage that was tested and could develop the upper bound of the potential load imparted to the anchorage.

The BR27C railing uses a HSS 4-in. x 4-in. x $\frac{3}{16}$ -in. (102-mm x 102-mm x 5-mm) A500 Grade B steel tube for the vertical support post attached to a $\frac{3}{4}$ -in. (19-mm) thick A36 steel baseplate. The tube section has an area, section modulus, and plastic section modulus of 2.77 in² (1,787 mm²), 3.30 in³ (54,077 mm³), and 3.91 in³ (64,073 mm³), respectively. A500 Grade B steel has a minimum yield strength of 42 ksi (289.6 MPa). However, steel tube sections designed as A500 Grade B are regularly fabricated from higher-strength steel, occasionally up to the A500 Grade C minimum yield strength of 46 ksi (317.2 MPa). Assuming the potential for the higher-strength Grade C material, and using the plastic section modulus of the tube, gives a moment capacity of the post of 179.9 kip-in. (20.33 kN-m). This moment capacity was rounded to an even 180 kip-in. (20.34 kN-m) and used for the design calculations of the alternative epoxy adhesive anchorages.

As noted previously, the design of alternative epoxy adhesive anchorages for the BR27C combination bridge rail was developed using ACI 318-11 procedures for design of epoxy anchorages with modifications of dynamic increase factors for concrete breakout, steel fracture, and bond strength. Details of the design calculations for the final designs are provided in Appendix A, but some comments on the basic design procedures should be noted. First, for

concepts incorporating two rows of anchors, it was assumed the tensile loads to develop moment capacity would be supplied by the front anchors while the rear anchors would develop the shear loads. Anchorage concepts that used only a single row of bolts had to account for both tensile and shear loads in all anchors. The design calculations evaluated steel fracture, concrete breakout, and adhesive bond failure in tension. Shear calculations evaluated steel fracture, concrete breakout, and concrete pryout.

The calculations also accounted for reduction in anchor capacity due to the distance to the edge of the parapet and anchor spacing based on the area of influence for the concrete and bond failures. Anchorage area of influence defines a region of the concrete where the anchorage forces are distributed in order to develop load for both concrete breakout and bond strength. If these areas exceed the edge of the parapet or overlap the area of influence of other anchors, then the capacity of the anchor is reduced by the ratio of the unavailable area divided by the original assumed influence area. A simple example of area of influence for two anchors that exceed the concrete edge and interfere with adjacent anchors is shown in Figure 4. Note that for the simple two-anchor example, the purple area denotes where the area of influence exceeds beyond the parapet edges. The orange area indicates where the area of influence for anchors “A” and “B” overlap. In this area, only half of the overlapping area can be utilized by each anchor, so the anchor capacity must be reduced accordingly.

A final note should be made regarding an additional modification that was made to the ACI 318-11 calculations for this project. Initial calculations for tensile concrete breakout capacity indicated that extremely large embedment depths would be required to provide the desired anchorage, due to the edge distance of the anchors to the side of the parapet. These calculations assume a concrete cone failure of the parapet that extends diagonally from the base of the anchor to the edges of the area of influence.

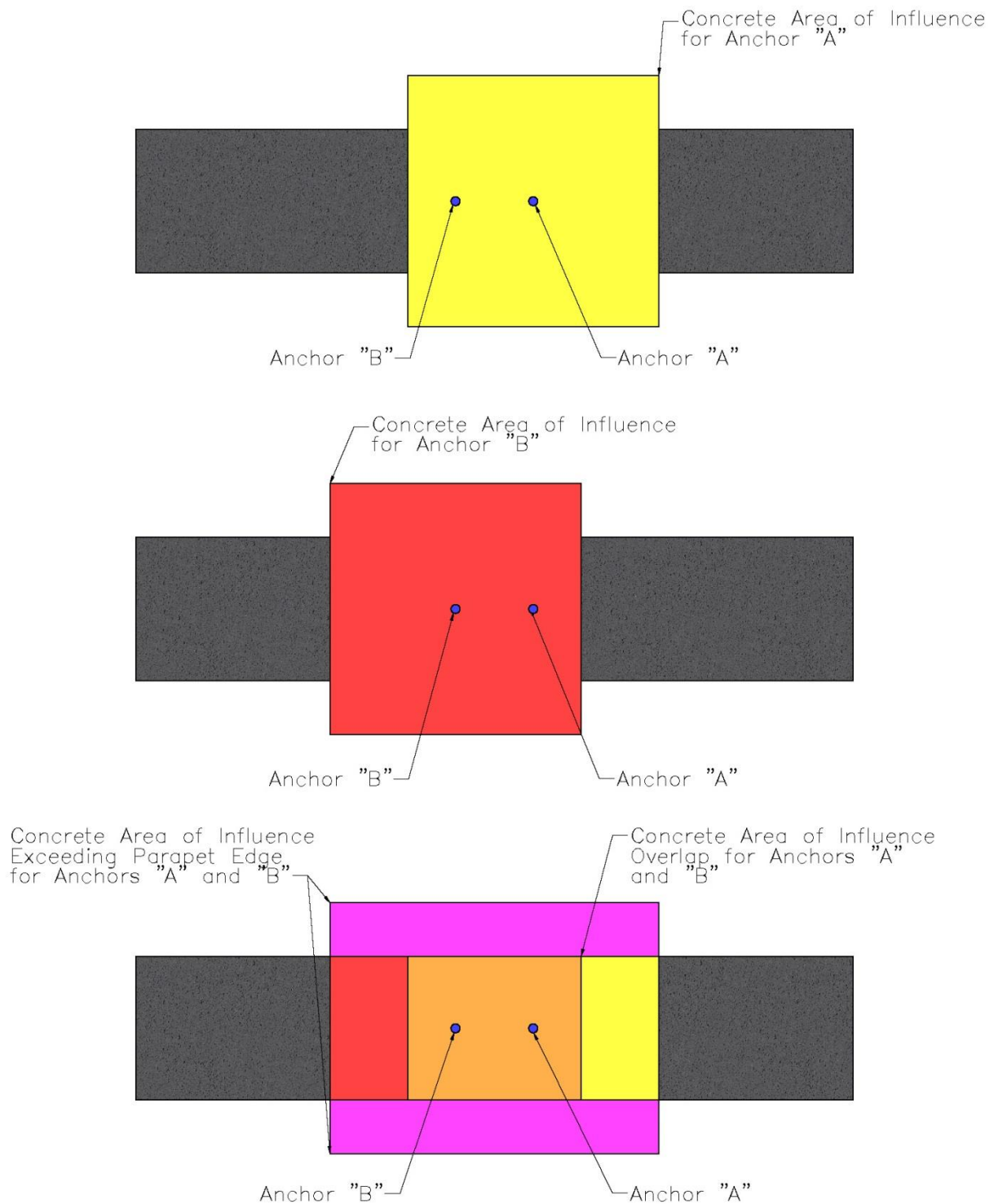
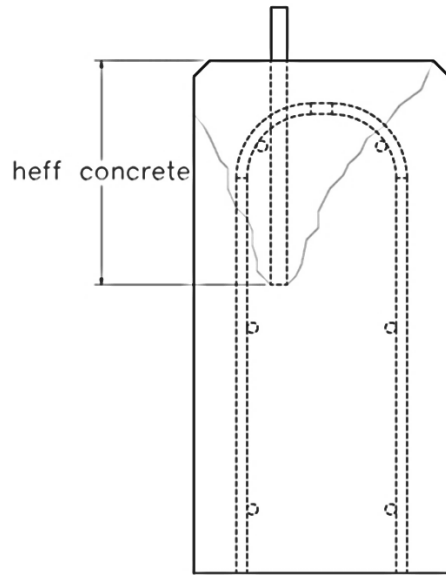


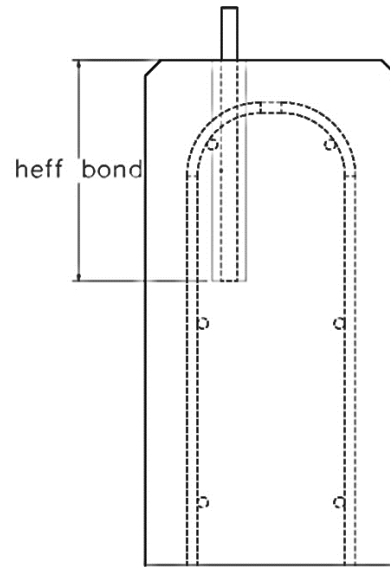
Figure 4. Concrete Area of Influence for Two Adjacent Anchors on Concrete Parapet

While this assumption may be true of large-area, unreinforced slabs, it was not believed to be accurate for the reinforced concrete parapet in this research. A more reasonable form of the failure mode was believed to be a hybrid concrete cone and adhesive bond failure, as shown in Figure 5. In this type of failure mode, the concrete cone failure is prevented from extending to the base of the anchor by the longitudinal rebar. The hybrid failure assumption was extended to the ACI 318-11 calculations by assuming that the upper half of the anchor embedment contributed to the concrete breakout and the lower half of the embedment contributed to a bond failure. Thus, the calculations for the concrete breakout and bond strength were performed with one-half of the actual anchor embedment and then summed to determine the tensile anchor capacity.

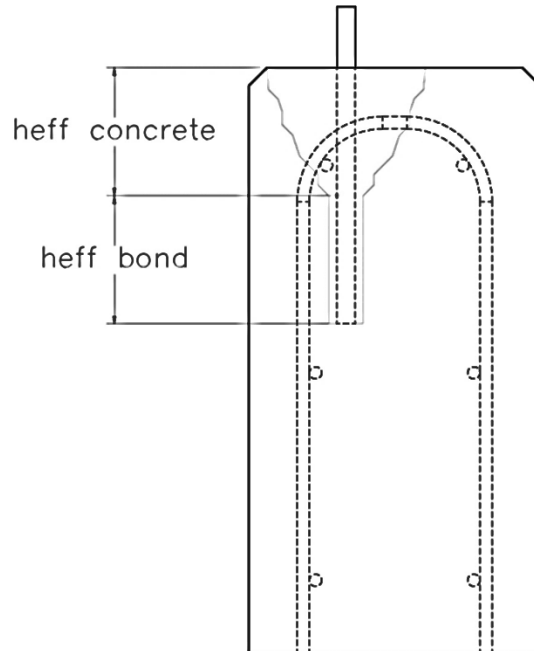
All calculations for the alternative adhesive anchorages were performed assuming the use of Hilti RE-500 epoxy adhesive, which has a bond strength of 1,800 psi (12.4MPa). It was assumed that other epoxy adhesives could also be used with the alternative anchorages, as long as the bond strength of the adhesive was equal to or greater than 1,800 psi (12.4MPa). The concrete compressive strength for the design calculations was assumed to be 4,000 psi (27.6 MPa).



ACI 318–11 Concrete
Breakout Failure Assumption



ACI 318–11 Bond
Failure Assumption



Hybrid Concrete and Bond
Failure Assumption

Figure 5. Comparison of ACI 318-11 Concrete Breakout and Hybrid Failure Assumptions

2.4 Alternative Anchorage Concepts

Multiple concepts were developed and evaluated as part of the design effort, but only four concepts were submitted to IaDOT for review. The four concepts varied the number, placement, and size of the anchors. It was believed that all of the designs would meet the design tensile and shear loads determined from the moment capacity of the post. Each of the concepts is reviewed in the subsequent sections. Details of the design calculations for the final designs are provided in Appendix A.

2.4.1 Four-Bolt Square Anchorage

The four-bolt square anchorage concept used a rectangular bolt pattern of four bolts on a square plate, as shown in Figure 6. The four bolts allowed for a design where the front bolts develop the tensile loads and the back anchors accounted for the shear loads.

This concept was also similar in layout to the current cast-in-place design. The anchor bolts were $\frac{5}{8}$ in. (16 mm) in diameter and embedded 10 in. (254 mm) into the parapet. All of the anchorage concepts were designed to have between $\frac{3}{4}$ in. (19 mm) and 1 in. (25 mm) of clearance from the longitudinal parapet reinforcement to ensure that they were not impacted during installation of the epoxy anchors. This constrained the design somewhat, but the concept did meet the tension and shear load requirements as determined from the moment capacity of the vertical post. The main drawback of this concept was that the anchors were only 2.75 in. (70 mm) apart across the width of the parapet, which could make it difficult to install.

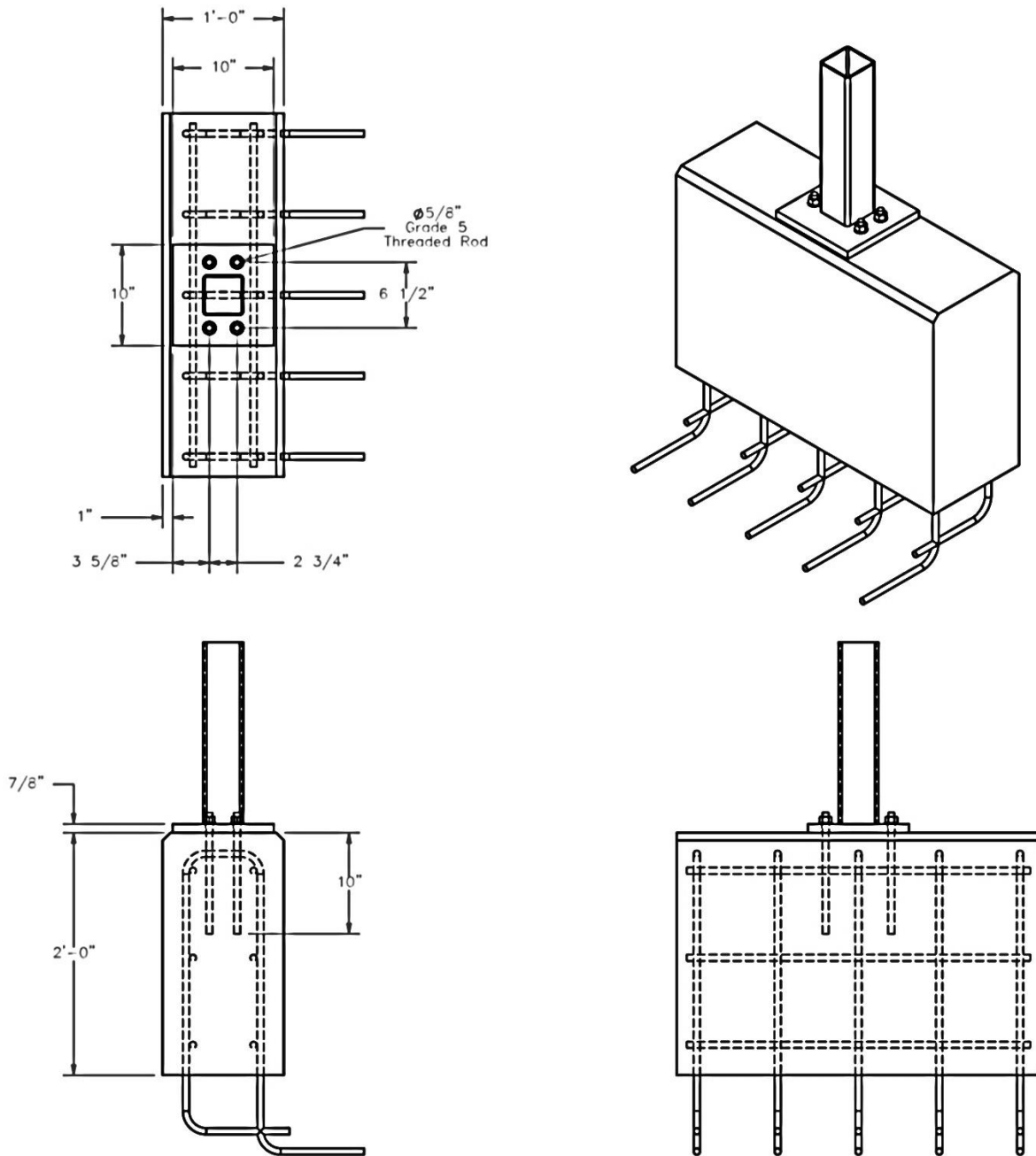


Figure 6. Four-Bolt Square Alternative Anchorage Concept

2.4.2 Four-Bolt Spread Anchorage

The four-bolt spread anchorage concept used the same anchor size and embedment depth, but it spread out the backside anchors to improve the anchor spacing for a four-bolt pattern, as shown in Figure 7. Design calculations indicated that the increased spacing of the anchors not only satisfied the design loads, but led to this configuration having a higher capacity than the four-bolt square anchorage concept.

2.4.3 Two-Bolt Centered Anchorage

The two-bolt centered anchorage concept used a linear bolt pattern of two bolts centered on a square baseplate, as shown in Figure 8. This concept reduced the number of anchors but required increased anchor diameter and embedment depth due to combined shear and tension loading of the anchors. The concept used $\frac{3}{4}$ -in. (19-mm) diameter bolts with an embedment of 12 in. (305 mm). Design calculations for this concept showed that the anchorage can develop both the shear and the tensile loads when determined individually. However, the ACI code recommends a reduction for combined loading, where the sum of the applied design load divided by the total capacity in both shear and tension must be less than 1.2. For this concept, that sum was calculated to be 1.44. However, neither the general anchor calculations nor the combined loading calculation in ACI 318-11 account for the reinforcing steel and its contributions to the anchorage capacity. As such, this design would potentially work under combined loads when including these other factors.

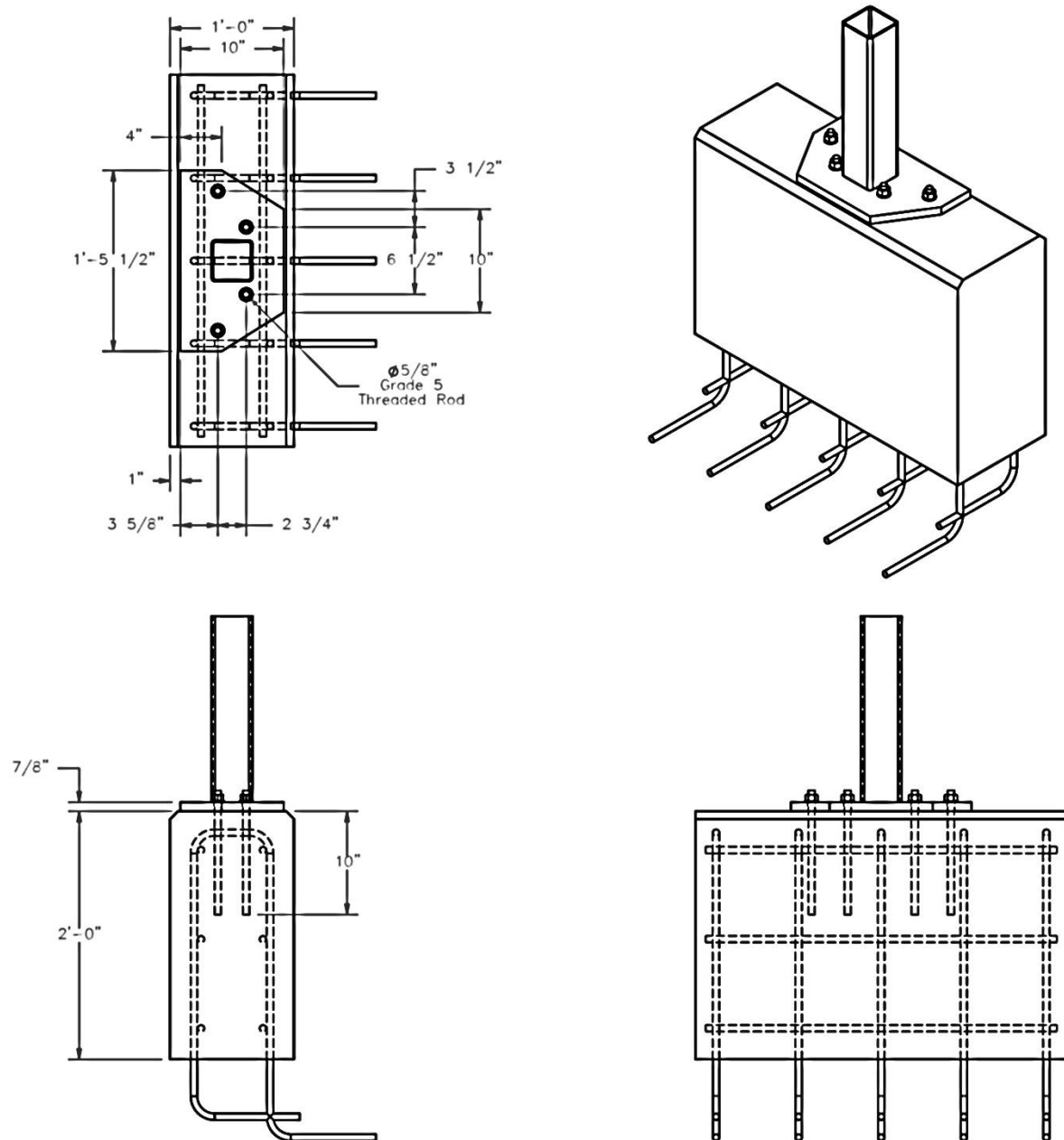


Figure 7. Four-Bolt Spread Alternative Anchorage Concept

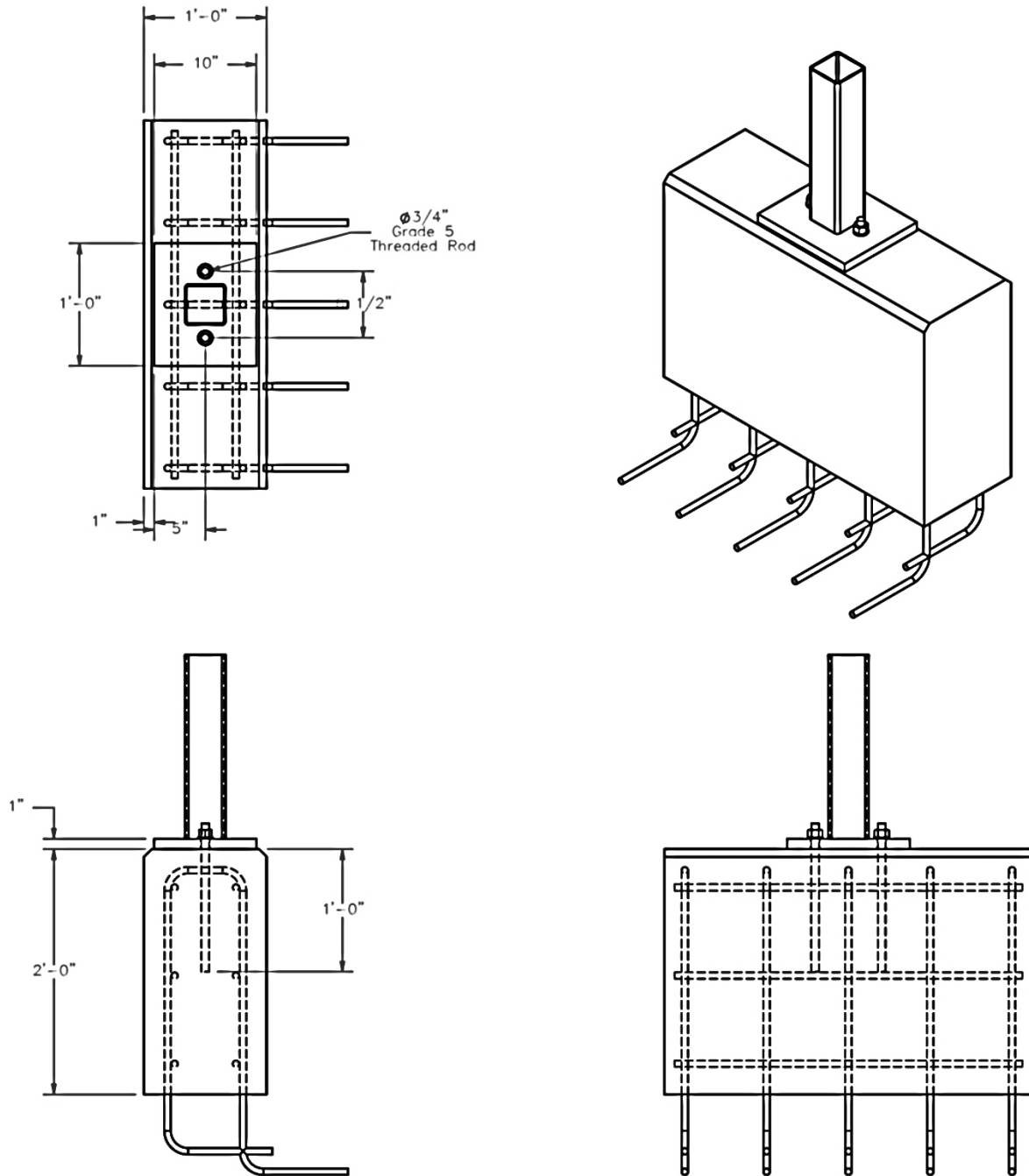


Figure 8. Two-Bolt Centered Alternative Anchorage Concept

2.4.4 Two-Bolt Offset Anchorage

The two-bolt offset anchorage concept used two bolts offset towards the front of the square baseplate, as shown in Figure 9. The design was identical to the centered concept, except that the bolts were offset towards the front of the parapet to increase the shear capacity sufficiently to meet the combined loading requirement in the ACI code. Thus, it was a more conservative design. Drawbacks to this design were the offset of the anchors and the potential for reverse bending loads. The researchers believed that the centered concept may be easier to install due to the bolts being centered on the rail rather than offset. Additionally, if the potential exists for significant reverse bending loads, then this concept would have reduced capacity in that regard. However, it was believed that the reverse bending loads on the BR27C combination rail were lower than the primary impact loads. Thus, the concern with respect to reverse bending overloading the anchorage was limited. In order to alleviate that concern, a smaller anchor could be placed on the backside of the post.

2.5 Selection of Preferred Alternative Anchorage Concepts for Evaluation

IaDOT representatives reviewed the four proposed alternative anchorage concepts and selected the four-bolt spread anchorage and the two-bolt offset anchorage as the preferred designs for evaluation through dynamic component testing. In addition to these two proposed configurations, IaDOT also requested that the researchers conduct dynamic testing on a third option that had been previously installed on the US-20 bridge near Hardin, IA, as shown in Figures 10 through 13. IaDOT was interested in evaluating whether this specific configuration meets/exceeds the capacity of the FHWA-approved cast-in-place BR27C combination bridge rail, and they wished to verify its performance as constructed.

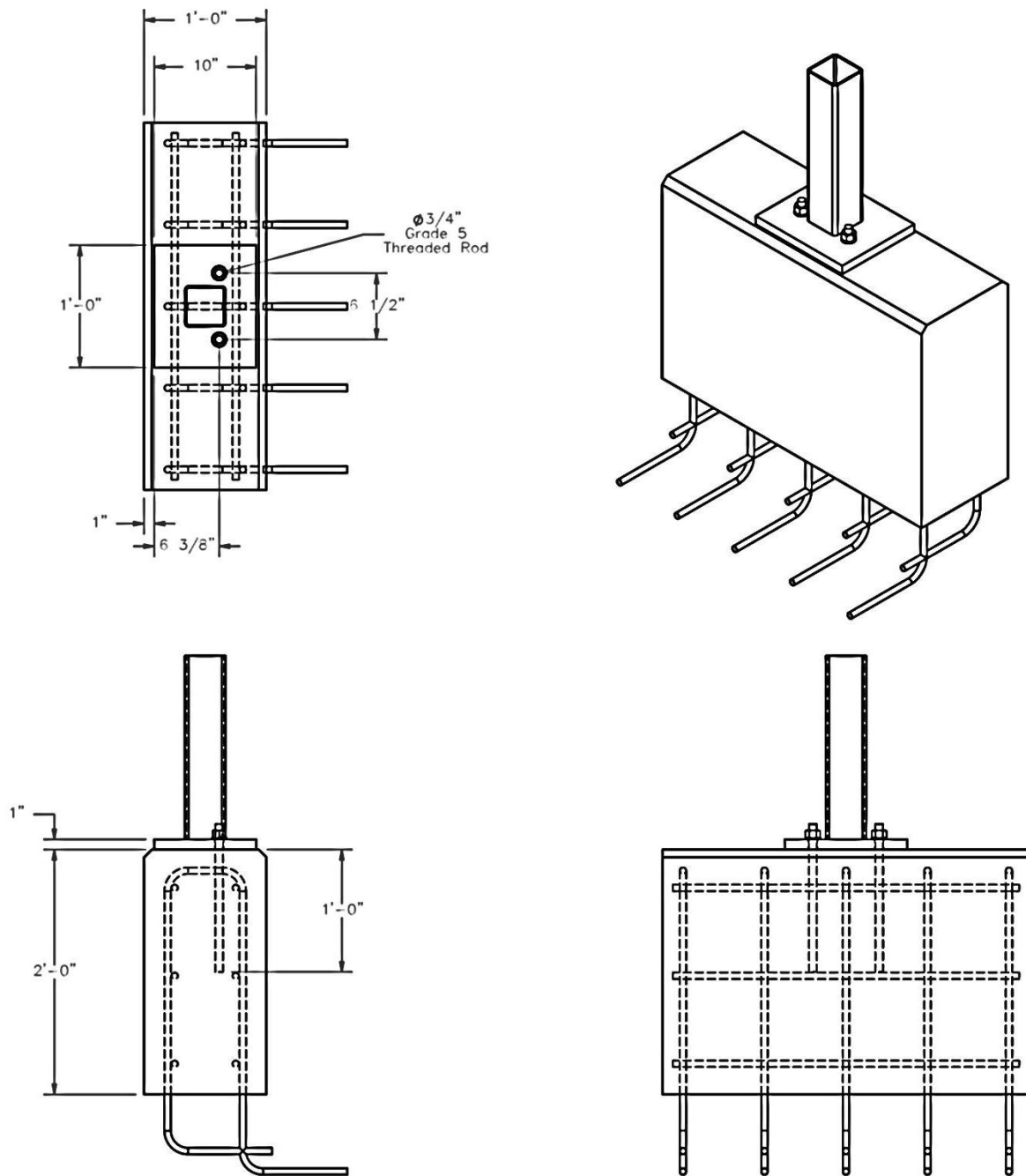


Figure 9. Two-Bolt Offset Alternative Anchorage Concept

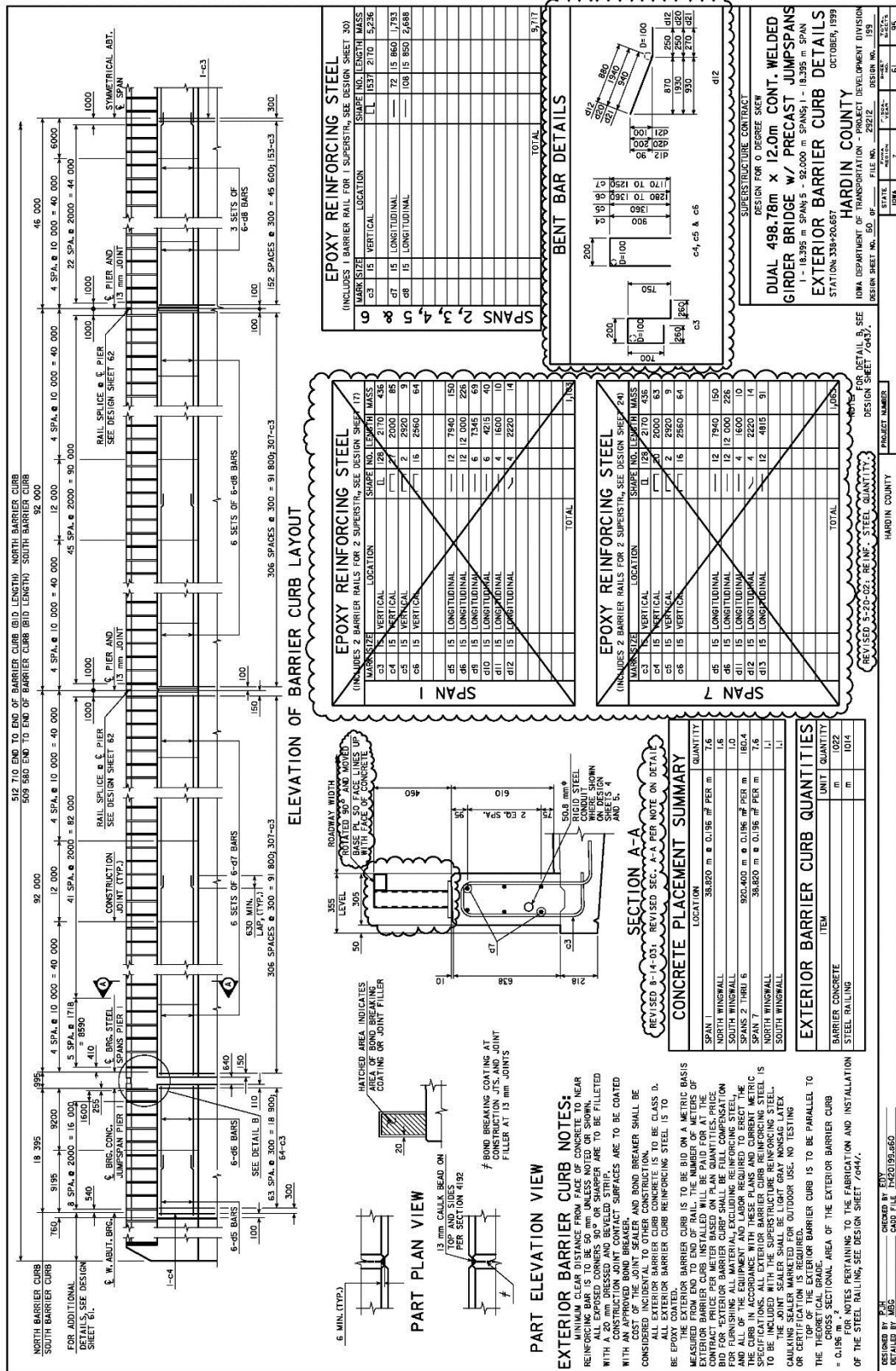
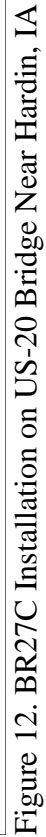


Figure 10. BR27C Installation on US-20 Bridge Near Hardin, IA



23



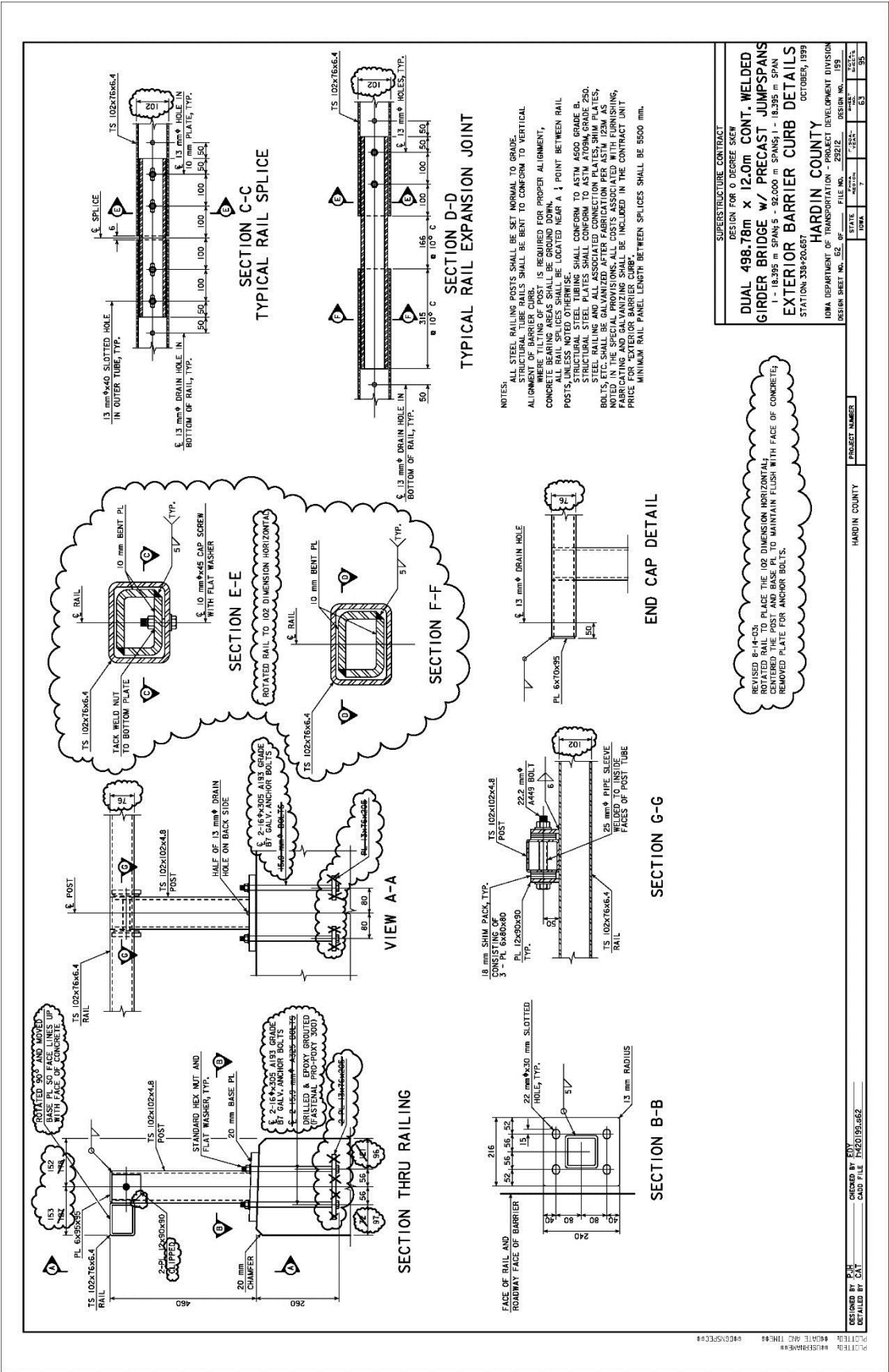


Figure 13. BR27C Installation on US-20 Bridge Near Hardin, IA

3 POST TESTING CONDITIONS

3.1 Purpose

A series of four dynamic bogie tests were conducted on the original BR27C combination bridge rail post and three alternative epoxy adhesive anchorage designs. The purposes of these tests were to establish the baseline capacity of the original BR27C cast-in-place anchorage and compare this capacity with the proposed alternative designs. The target impact conditions for all tests were identical. The tests were configured so that the applied impact load would occur at a height of 16 in. (406 mm) above the top of the parapet on the post/rail in order to produce a bending moment in the post and combined loading on the anchorage system similar to that provided during vehicle crash events. The force versus deflection, energy dissipated versus deflection, and failure modes were documented for each test and compared to one another. These comparisons were then used to verify that the proposed anchorages provided equal or greater capacity than the full-scale crash tested anchorage. The tests required construction of a short section of simulated bridge rail for attachment of the post, baseplate, and anchor hardware. All dynamic tests were conducted at the MwRSF proving grounds in Lincoln, Nebraska.

3.2 Scope

Four dynamic bogie tests were conducted on HSS 4-in. x 4-in. x $\frac{3}{16}$ -in. (102-mm x 102-mm x 5-mm) steel tubes with baseplates mounted on top of a reinforced concrete parapet. The reinforced concrete parapet was installed below grade, such that the top of the parapet was essentially level with the concrete apron at the test site. Installation of the parapet below grade allowed the researchers to impact the post assembly at the desired height to produce similar post loading to the horizontal bridge rail tube during an impact event. The concrete parapet layout was based on the parapet design used in the original full-scale crash testing of the BR27C combination bridge rail and the revised parapet design provided by IaDOT. As such, the parapet

was 10 in. (254 mm) wide on one end and was then widened to 12 in. (305 mm) for the remainder of the parapet. All parapet reinforcement was made consistent with the original and revised parapet designs that were provided. The concrete used for the parapet was selected to be a 3,600-psi (24.8-MPa) mix meeting IaDOT Class C-4 concrete specification. This mix design was consistent with the concrete strength of the parapet used in the original BR27C combination bridge rail crash testing. IaDOT typically uses a 4,000-psi (27.6-MPa) concrete mix for their concrete parapets, but the lower-strength concrete was selected for all the component tests in order to provide accurate data for the baseline test of the original cast-in-place anchorage and to provide a consistent comparison of anchorage capacity using the same concrete strength. It was believed that if the alternative anchorages provided equal or greater capacity to the original anchorage in the 3,600-psi (24.8-MPa) concrete, it would be acceptable in higher-strength concrete as well.


The posts and baseplates used in the dynamic component tests were developed based on details of the original BR27C combination bridge rail, the alternative anchorages developed in the previous chapter, and details provided by IaDOT for the US-20 bridge installation. All of the test setups used the same HSS 4-in. x 4-in. x $\frac{3}{16}$ -in. (102-mm x 102-mm x 5-mm) steel tube welded to baseplates that were anchored to the concrete parapet. Baseplates for the four-bolt spread and two-bolt offset anchorages were designed based on the anchorage system and moment capacity of the post. The two remaining designs used baseplates based on the provided details. The two alternative anchor concepts developed in the previous chapter were installed using Hilti RE-500 SD epoxy adhesive. The anchorage for the US-20 bridge was installed with Fastenal Pro-Poxy 300, per the IaDOT details.

The target impact conditions were a speed of 15 mph (24.1 km/h) and an angle of 90 degrees, creating a “head-on” or full-frontal impact and strong-axis bending. Target impact

height for the testing was 16 in. (406 mm) above the ground line. The posts were impacted 17 in. (432 mm) above the top of the parapet due to the concrete parapet being 1 in. (25 mm) lower than grade.

The test matrix is shown in Figure 14, and the test setup is shown in Figures 15 through 29. Test installation photographs are shown in Figures 30 through 34. Material specifications, mill certifications, and certificates of conformity for the combination rails attached to concrete parapets are shown in Appendix B.

<p>Notes: (1) Instrumentation –DTS SLICE 1 and 2 –High-speed cameras perpendicular, offset behind, and offset front (2) Document parapet cracking and damage before and after test. (3) Need concrete cylinders from the pour (ø6"x12" [ø152x305]) –12 from start of pour and 12 at end of pour (4) Concrete parapet and grade beam may be constructed by subcontractor.</p>	Test No.	Test Article	Impact Height	Impact Speed mph [km/h]	Angle Degrees
	1	Cast-in-Place Anchorage	16" [406]	15 [24.1]	90
	2	Four Spread Epoxy Anchors	16" [406]	15 [24.1]	90
	3	Two Offset Epoxy Anchors	16" [406]	15 [24.1]	90
	4	US-20/Hardin Bridge	16" [406]	15 [24.1]	90



Midwest Roadside Safety Facility

Iowa Bridge Rail

Test Matrix

DWG. NAME: IowaBR27C_R3

SCALE: 1:30
UNITS: in, [mm]

SHEET: 1 of 16
DATE: 4/30/2014
DRAWN BY: SDB
REV. BY: RWB/KAL

Figure 14. Bogie Testing Matrix and Setup

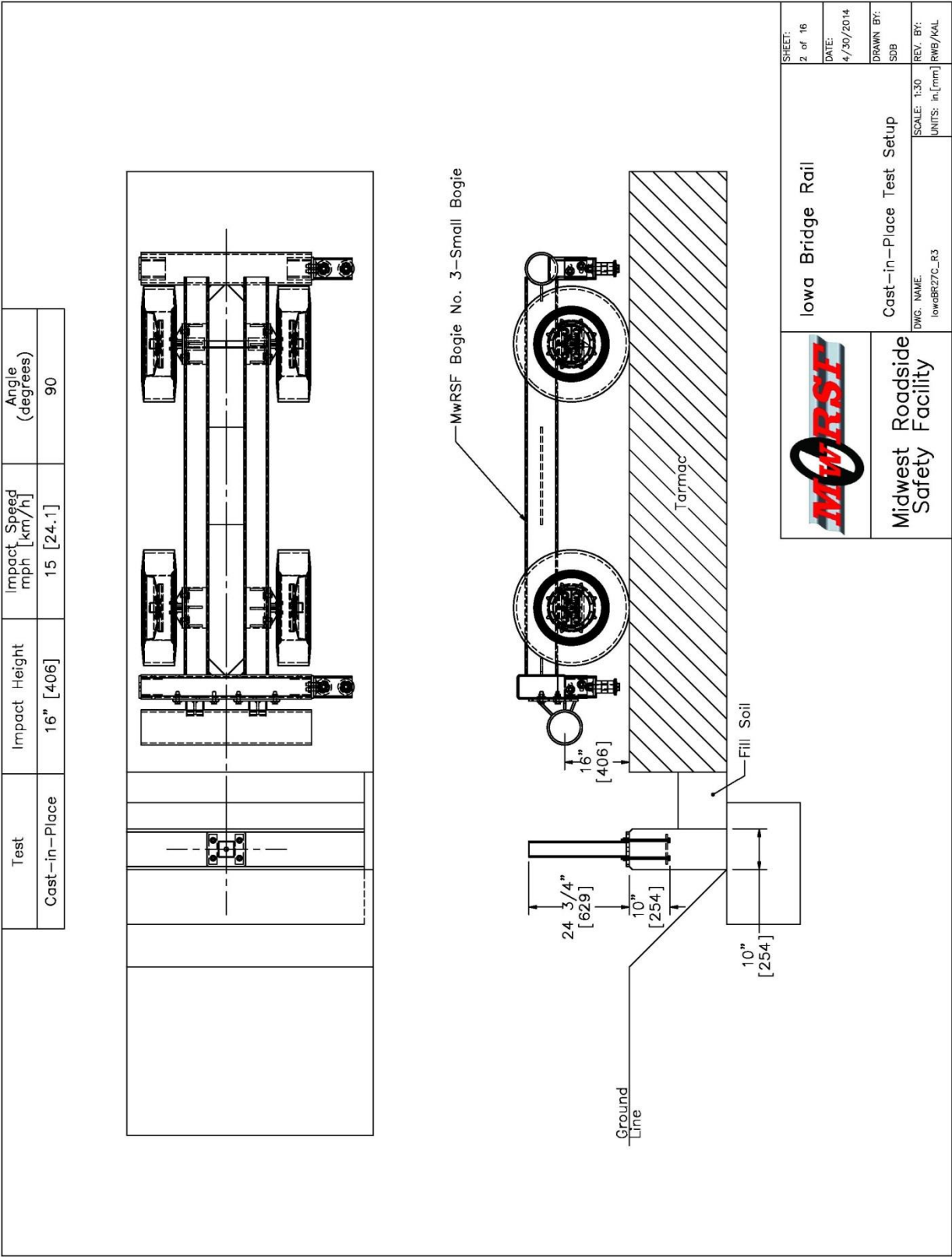


Figure 15. Cast-in-Place Test Setup, Test No. IBP-1

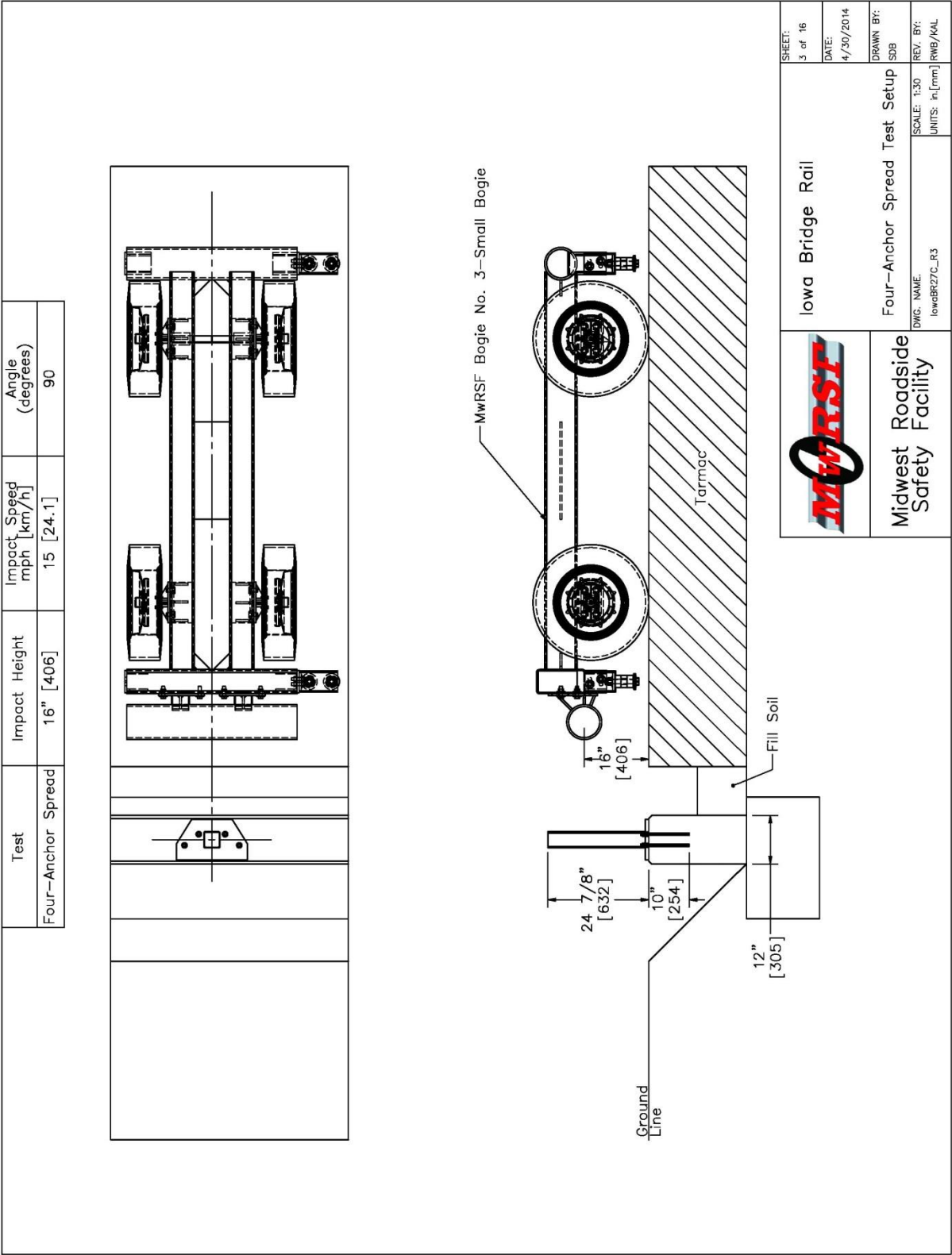


Figure 16. Four-Anchor Test Setup, Test No. IBP-2

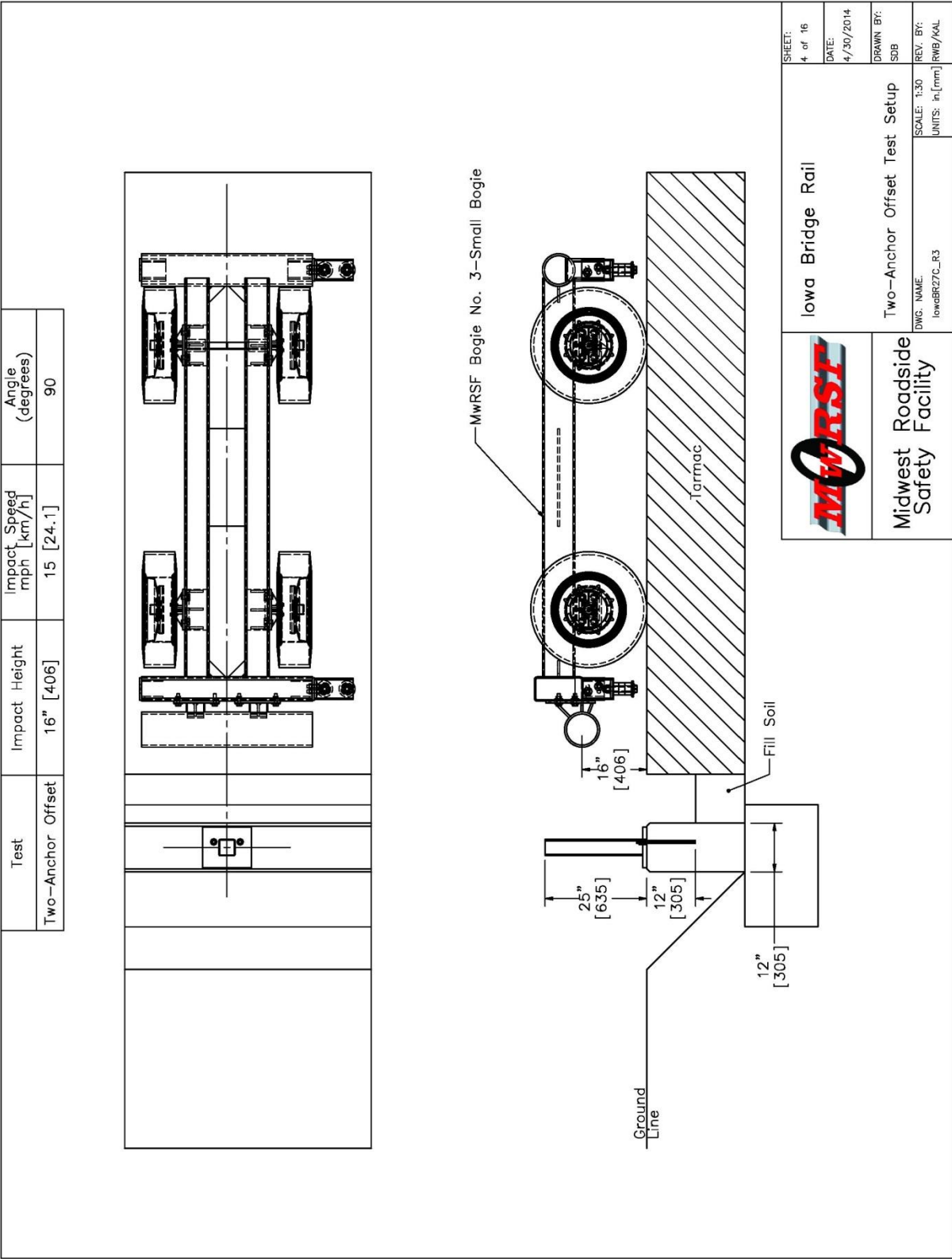


Figure 17. Two-Anchor Offset Test Setup, Test No. IBP-3

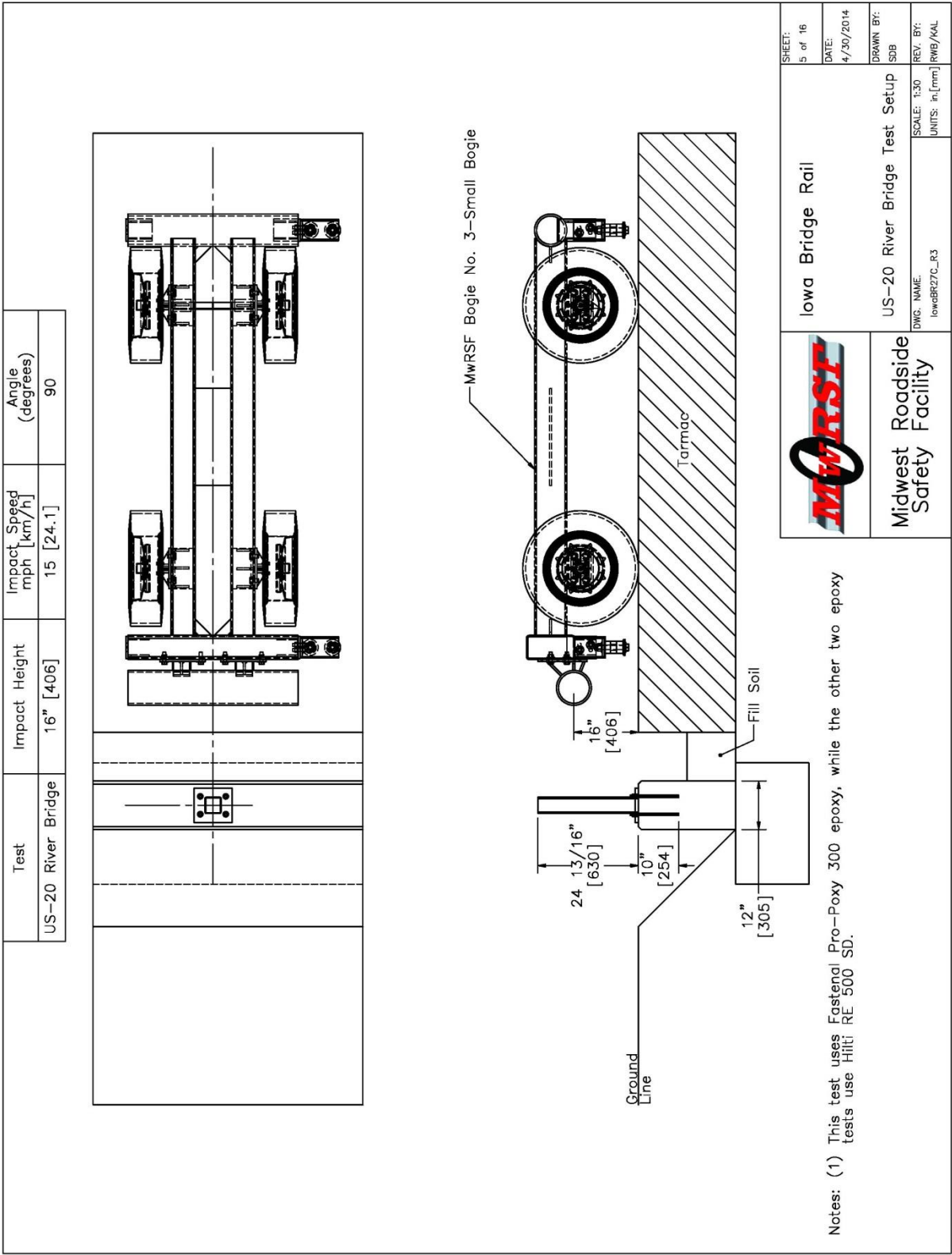


Figure 18. US-20 River Bridge Test Setup, Test No. IBP-4

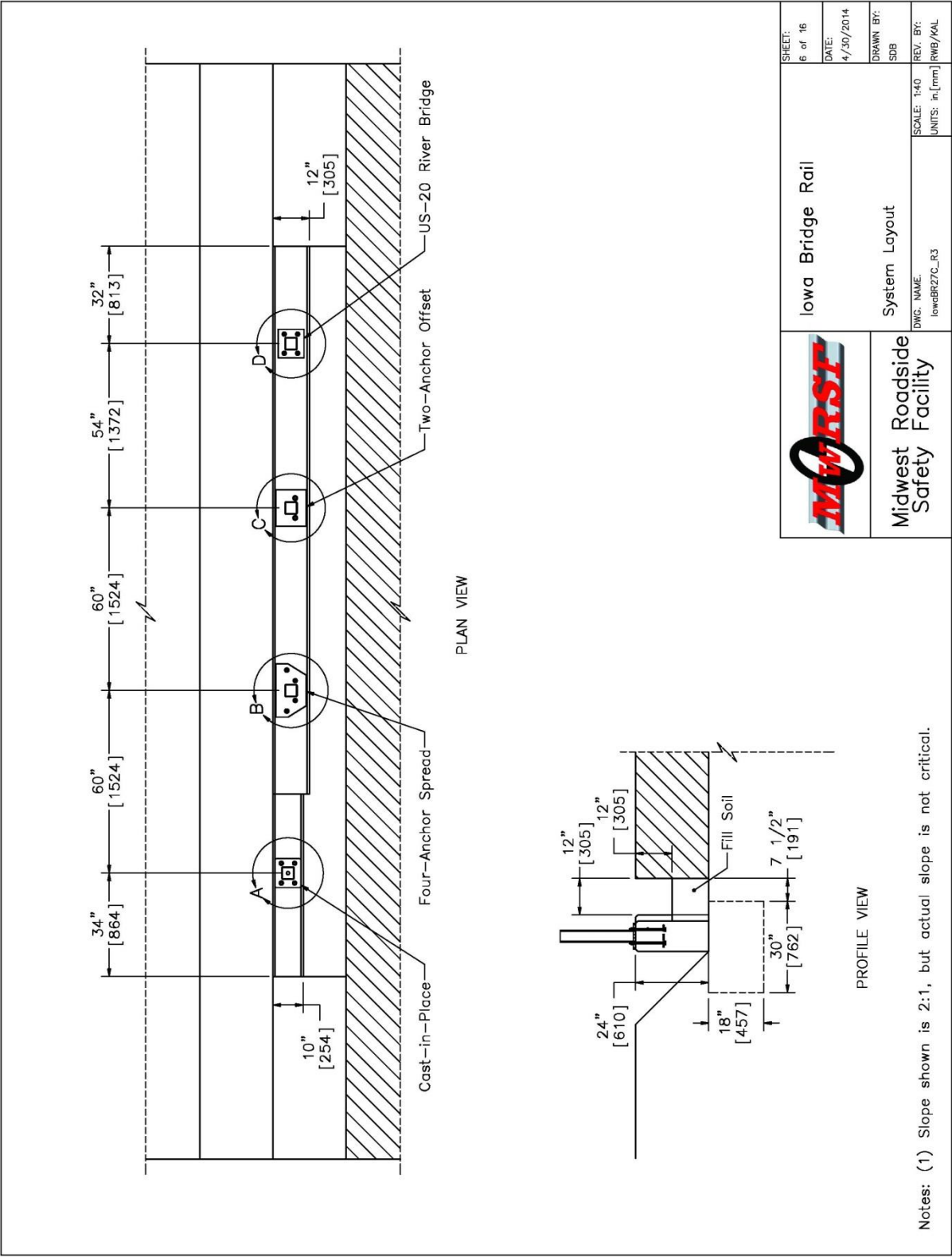
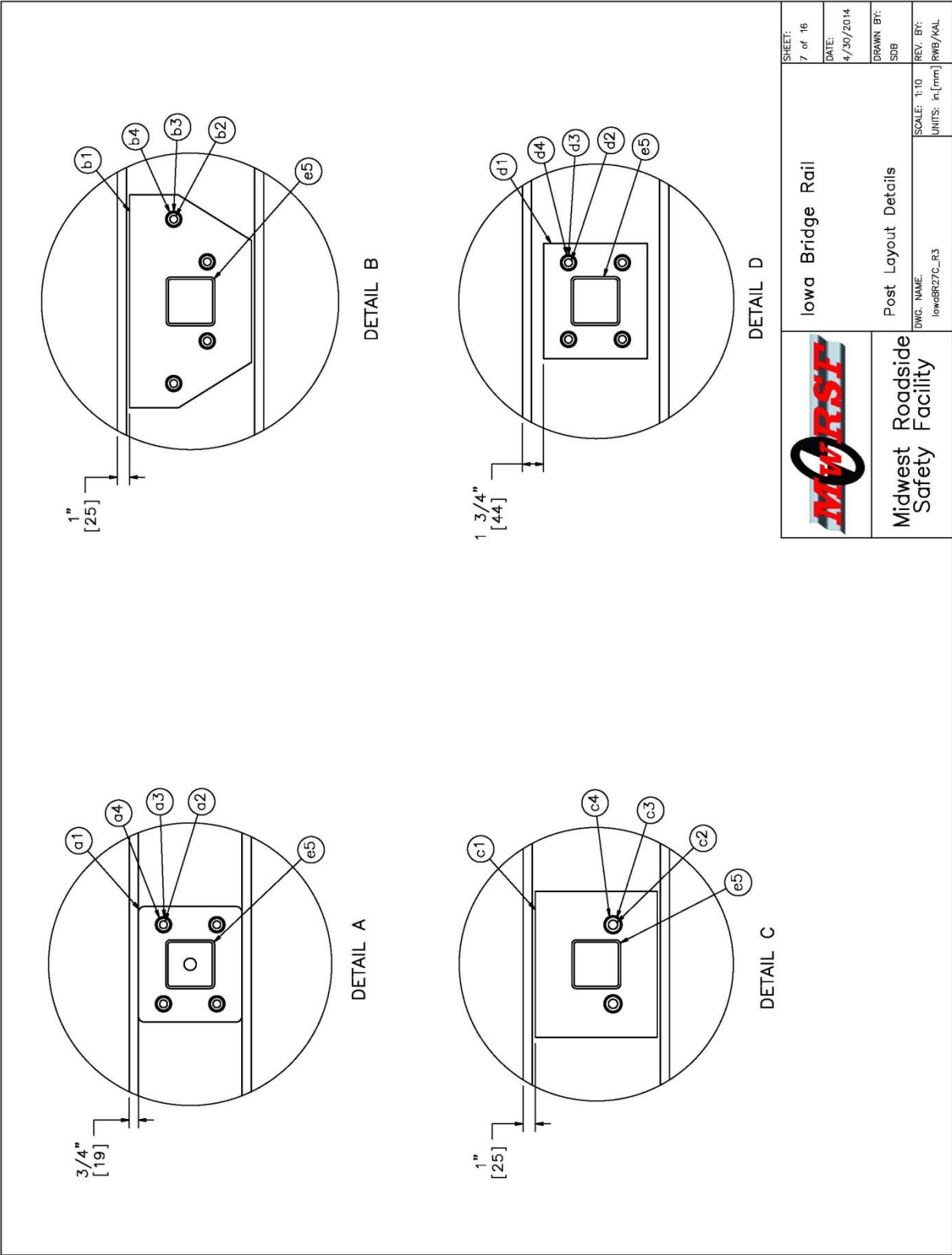


Figure 19. System Layout



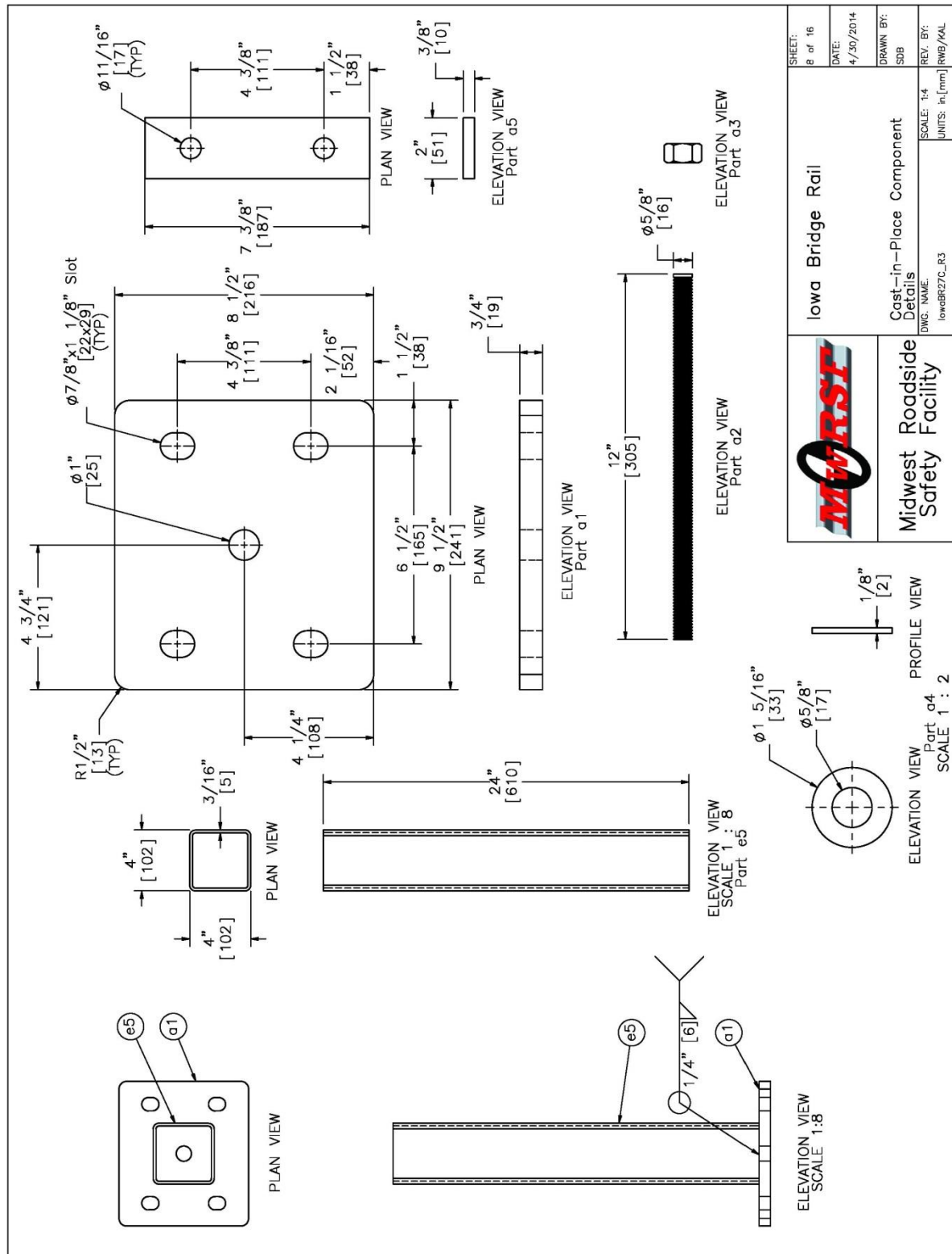


Figure 21. Cast-in-Place Component Details, Test No. IBP-1

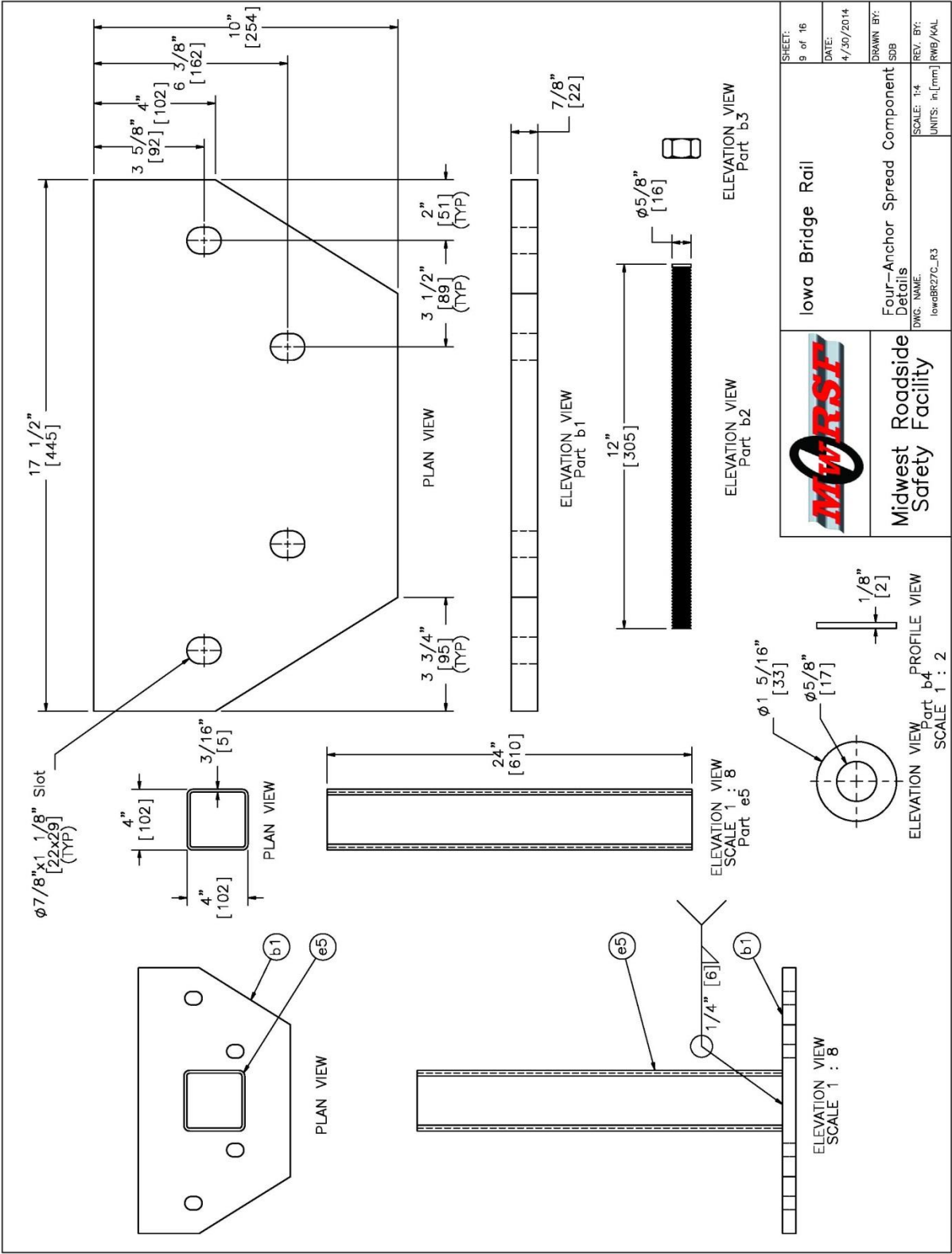


Figure 22. Four-Anchor Spread Component Details, Test No. IBP-2

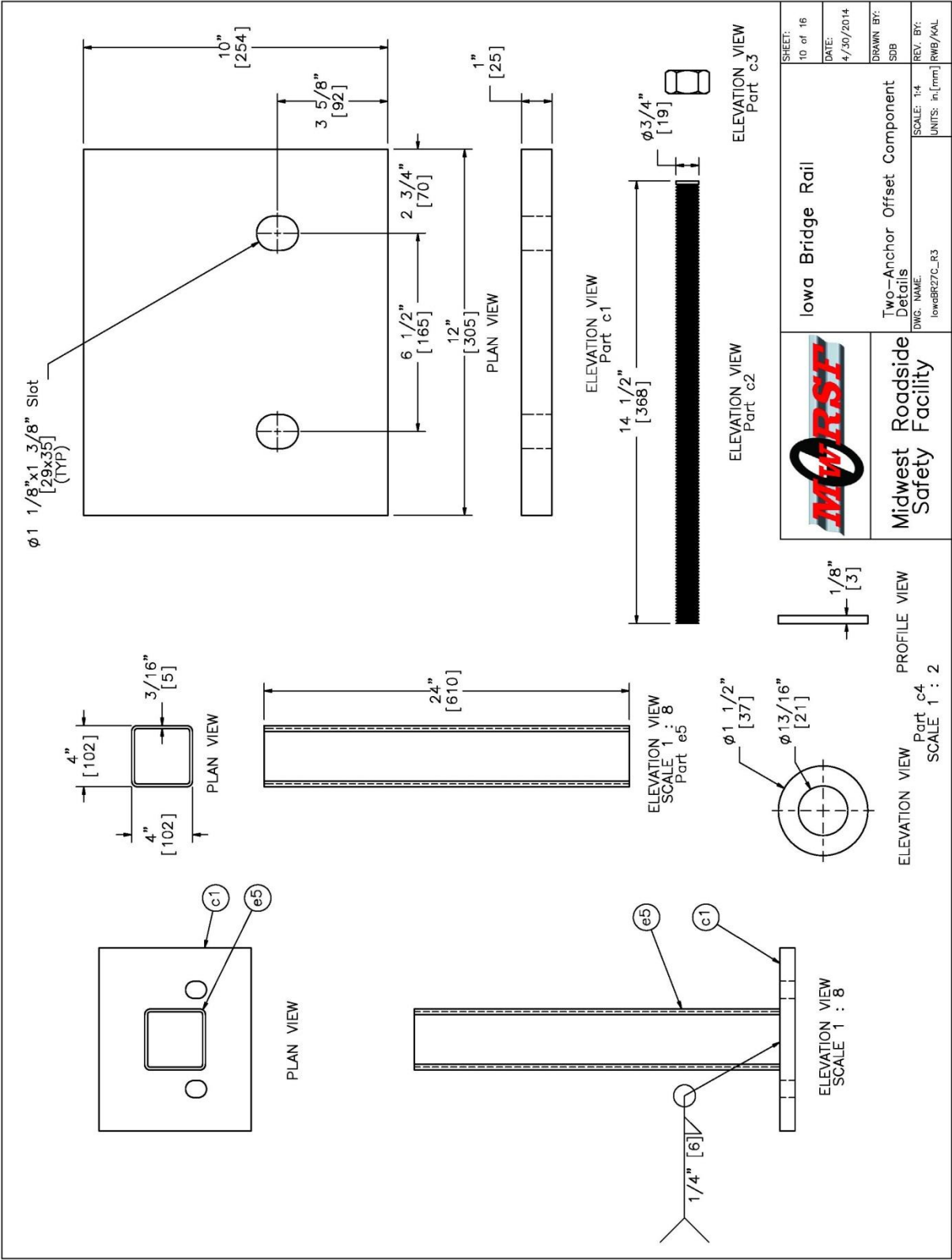


Figure 23. Two-Anchor Offset Component Details, Test No. IBP-3

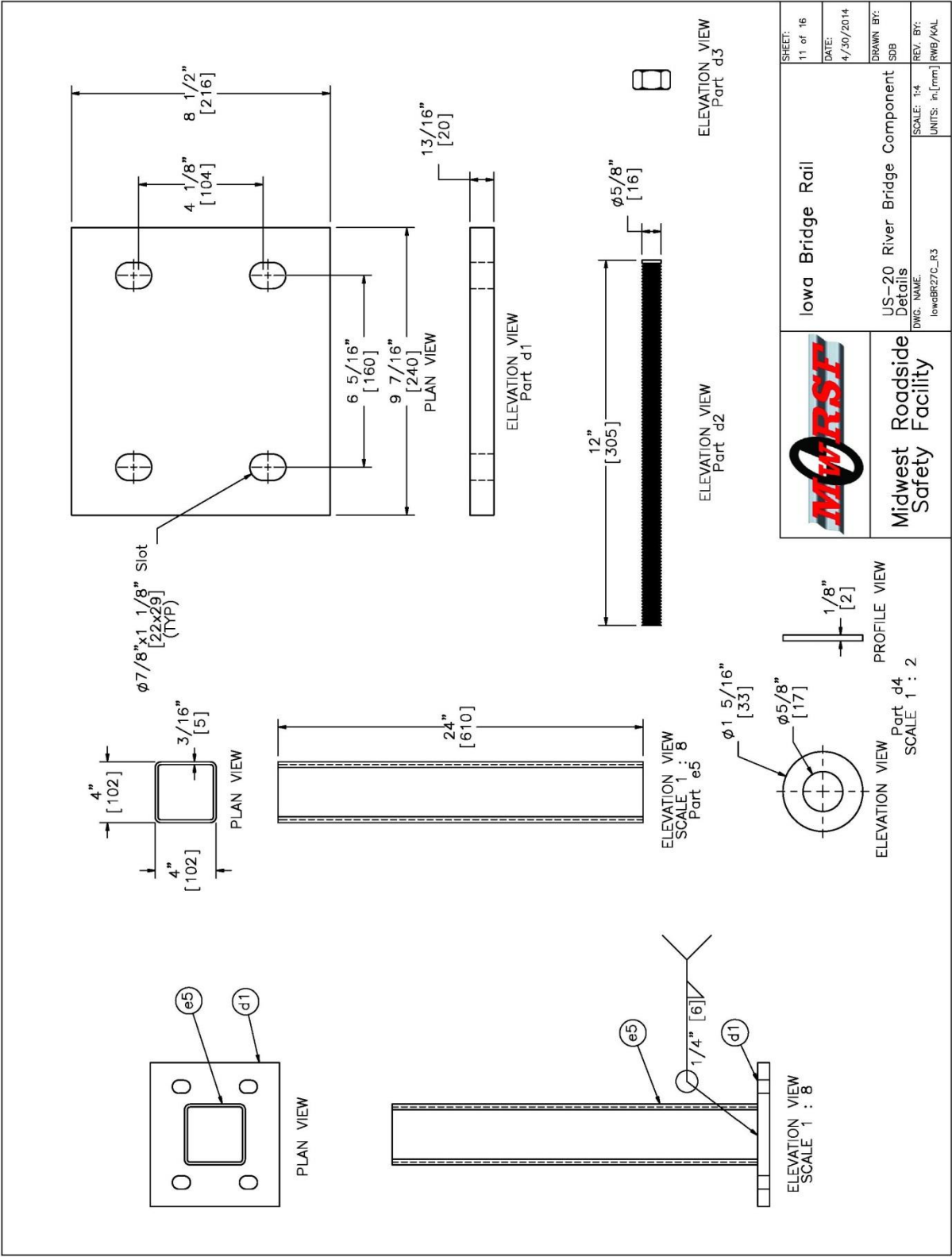


Figure 24. US-20 River Bridge Component Details, Test No. IBP-4

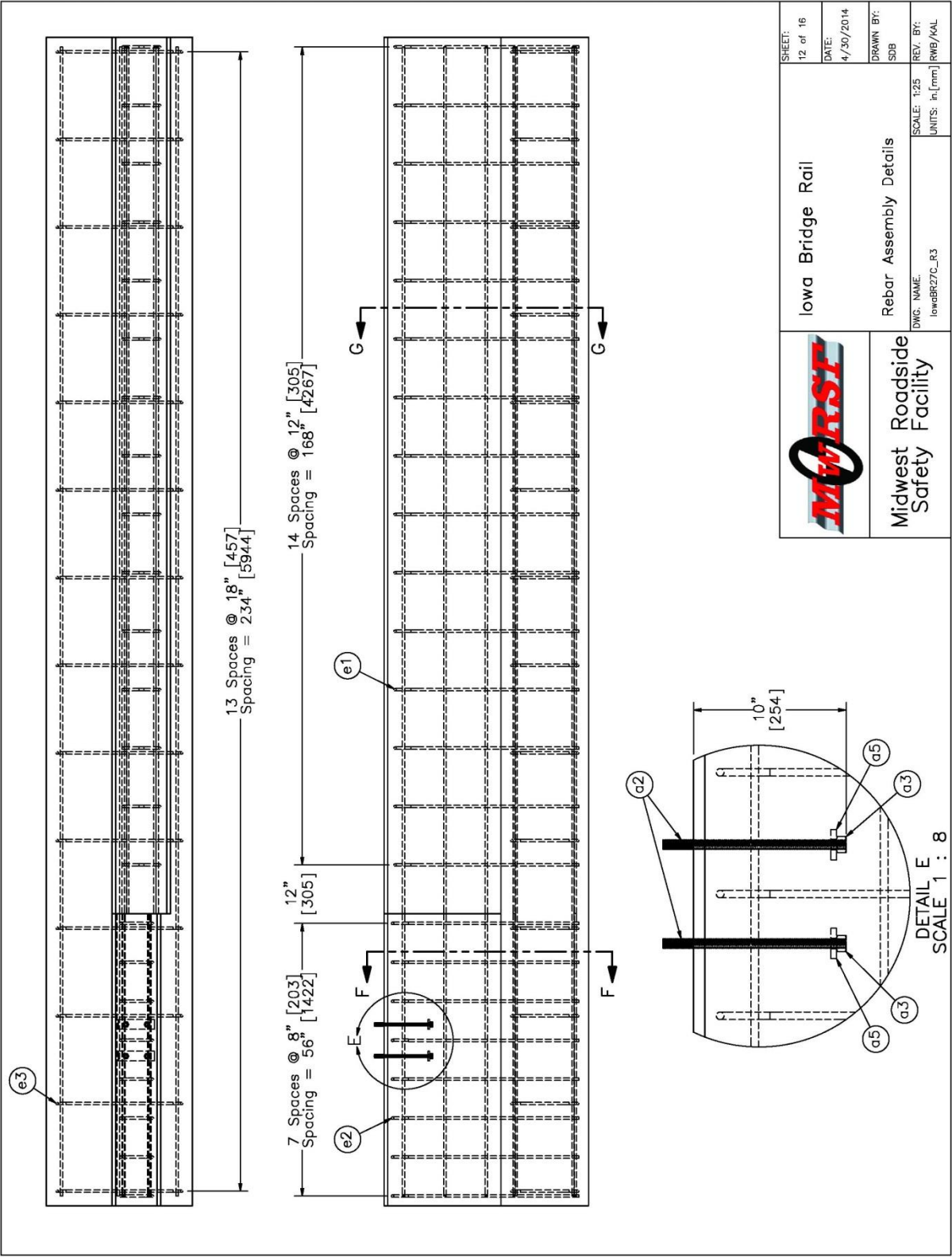


Figure 25. Rebar Assembly Details

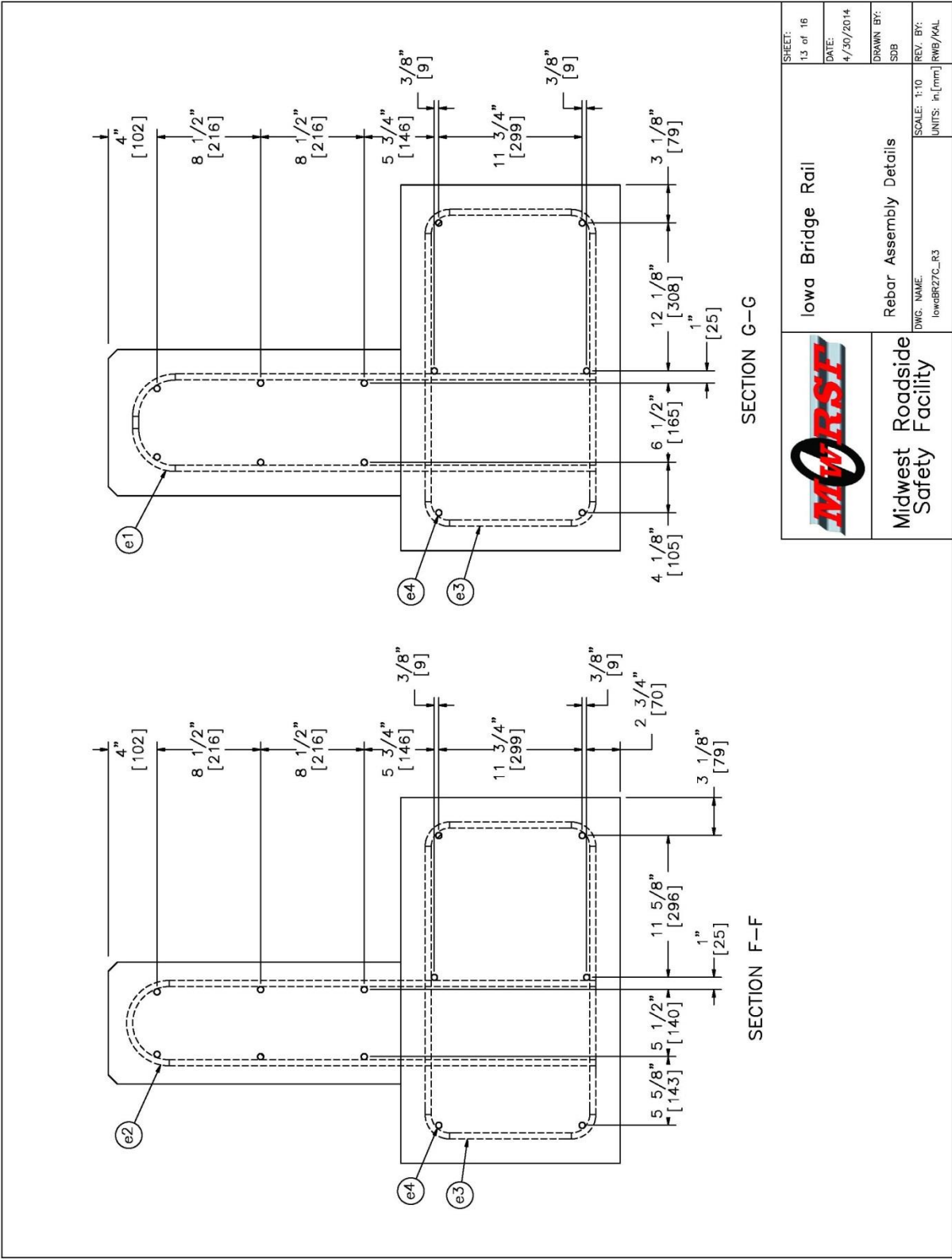


Figure 26. Additional Rebar Assembly Details

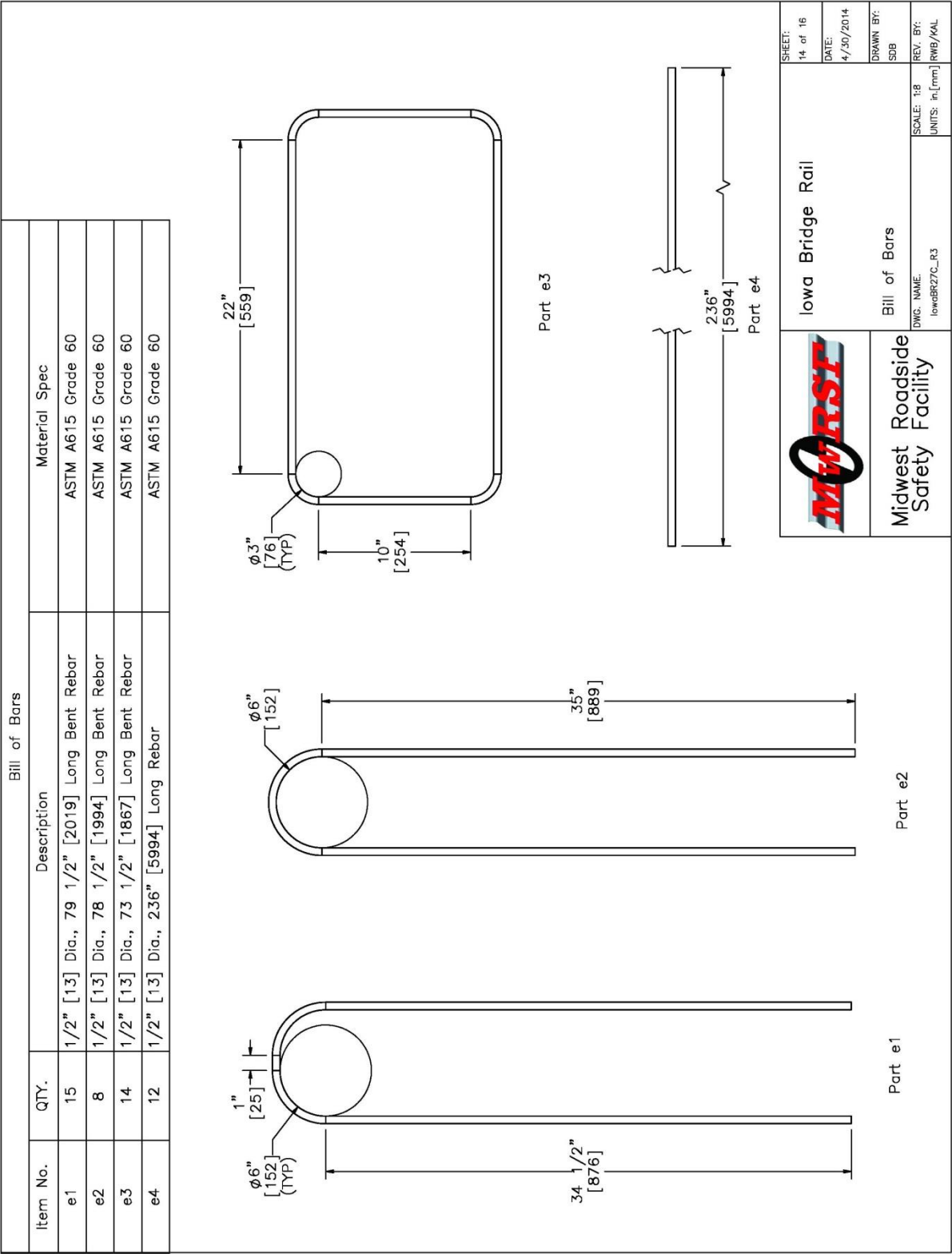


Figure 27. Bill of Bars

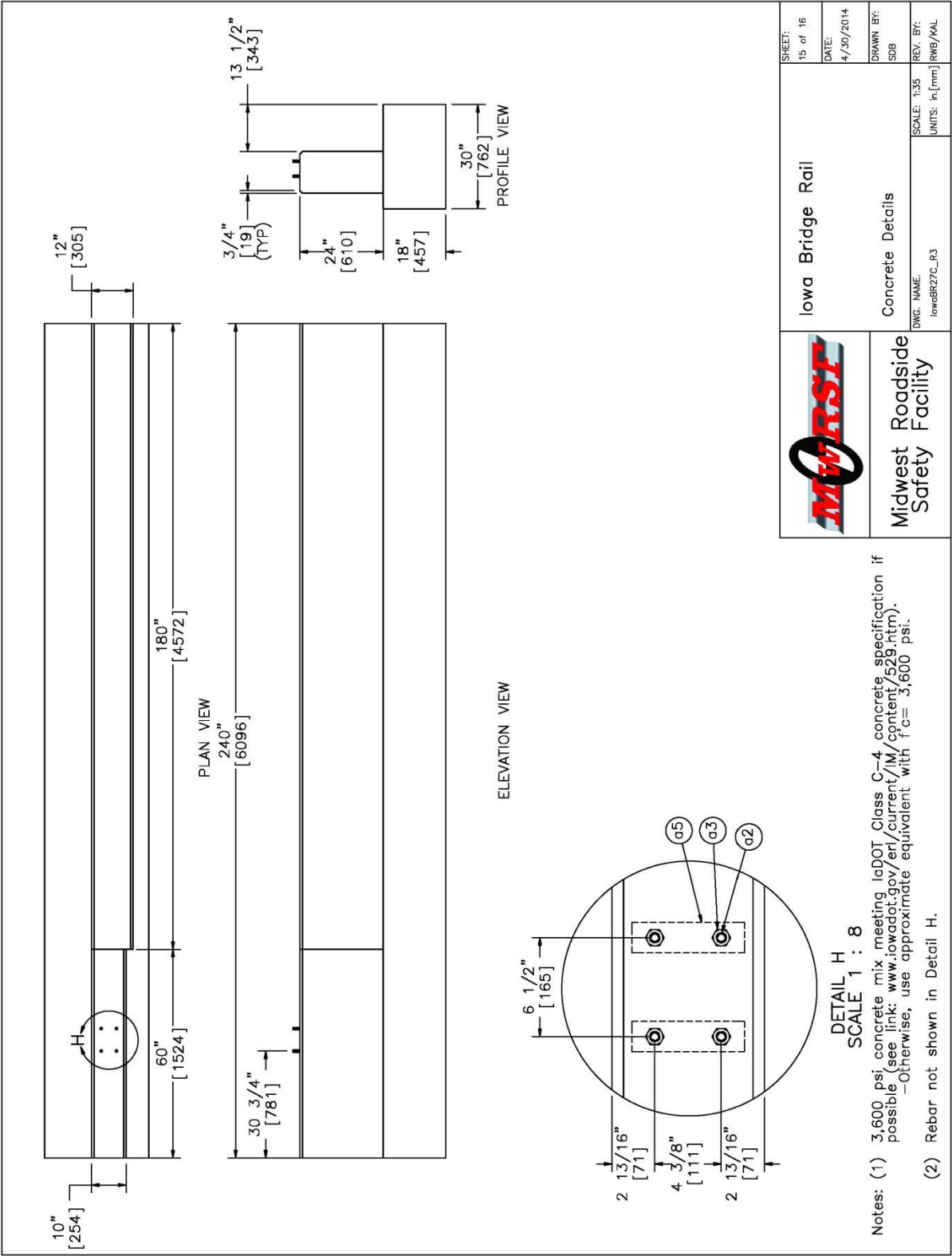


Figure 28. Concrete Details

Cast-in-Place				
Item No.	QTY.	Description	Material Spec	
a1	1	Baseplate	ASTM A36	
a2	4	5/8" [16] Dia. UNC, 12" [305] Long Threaded Rod	ASTM A193 Grade B7	
a3	8	5/8" [16] Dia. Hex Nut	ASTM A563 DH	
a4	4	5/8" [16] Dia. SAE Flat Washer	ASTM F436	
a5	2	2"x7 3/8"x3/8" [51x187x10] Washer Plate	ASTM A36	
e5	4	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	
Four-Anchor Spread				
Item No.	QTY.	Description	Material Spec	
b1	1	Baseplate	ASTM A36	
b2	4	5/8" [16] Dia. UNC, 12" [305] Long Threaded Rod	ASTM A193 Grade B7	
b3	4	5/8" [16] Dia. Hex Nut	ASTM A563 DH	
b4	4	5/8" [16] Dia. SAE Flat Washer	ASTM F436	
e5	1	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	
e6	–	Epoxy	Hilti RE-500 SD Epoxy	
Two-Anchor Offset				
Item No.	QTY.	Description	Material Spec	
c1	1	Baseplate	ASTM A36	
c2	2	3/4" [19] Dia. UNC, 14 1/2" [368] Long Threaded Rod	ASTM A193 Grade B7	
c3	2	3/4" [19] Dia. Hex Nut	ASTM A563 DH	
c4	2	3/4" [19] Dia. SAE Flat Washer	ASTM F436	
e5	1	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	
e6	–	Epoxy	Hilti RE-500 SD Epoxy	
US-20 River Bridge				
Item No.	QTY.	Description	Material Spec	
d1	1	Baseplate	ASTM A36	
d2	4	5/8" [16] Dia. UNC, 12" [305] Long Threaded Rod	ASTM A193 Grade B7	
d3	4	5/8" [16] Dia. Hex Nut	ASTM A563 DH	
d4	4	5/8" [16] Dia. SAE Flat Washer	ASTM F436	
e5	1	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	
e6	–	Epoxy	Fastenal Pro-Poxy 300	


 Midwest Roadside Safety Facility	Iowa Bridge Rail
	Bill of Materials
DWG. NAME: LOWGBR27C_R4	SCALE: 1/8" UNITS: in./mm
DRAWN BY: SDB/JAH	REV. BY: RWB/KAL/DPH
DATE: 7/21/2015	SHEET: 16 of 16

Figure 29. Bill of Materials



Figure 30. Test Installation Photographs



Figure 31. Pre-Test Installation Photographs, Test No. IBP-1



Figure 32. Pre-Test Installation Photographs, Test No. IBP-2



Figure 33. Pre-Test Installation Photographs, Test No. IBP-3



Figure 34. Pre-Test Installation Photographs, Test No. IBP-4

3.3 Equipment and Instrumentation

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

3.3.1 Bogie Vehicle

A rigid-frame bogie was used to impact the posts. A variable-height, detachable impact head was used in the testing. The bogie head was constructed of 8-in. (203-mm) diameter, 1/2-in. (13-mm) thick standard steel pipe, with 3/4-in. (19-mm) neoprene belting wrapped around the pipe to prevent local damage to the post from the impact. The impact head was bolted to the bogie vehicle, creating a rigid frame with an impact height of 16 in. (406 mm). The bogie with the impact head is shown in Figure 35. The total weight of the bogie with the addition of the mountable impact head and accelerometers was 1,808 lb (820 kg).



Figure 35. Rigid-Frame Bogie on Guidance Track

The tests were conducted using a steel corrugated-beam guardrail to guide the tire of the bogie vehicle. A pickup truck was used to push the bogie vehicle to the required impact velocity. After reaching the target velocity, the push vehicle braked, allowing the bogie to be free-rolling

as it came off the track. A remote braking system was installed on the bogie, allowing it to be brought safely to rest after the test.

3.3.2 Accelerometers

Two environmental shock and vibration sensor/recorder systems were used to measure the accelerations in the longitudinal, lateral, and vertical directions. However, only the longitudinal acceleration was processed and reported. All of the accelerometers were mounted near the centers of gravity 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 filters conforming to SAE J211/1 specifications [10].

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

3.3.3 Retroreflective Optic Speed Trap

Retroreflective optic speed trap was used to determine the speed of the bogie vehicle before impact. Three retroreflective targets, spaced at approximately 18-in. (457-mm) intervals, were applied to the side of the bogie vehicle. When the emitted beam of light was reflected by the targets and returned to the Emitter/Receiver, a signal was sent to the data acquisition computer, recording at 10,000 Hz, as well as the external LED box activating the LED flashes. The speed was then calculated using the spacing between the retroreflective targets and the time

between the signals. LED lights and high-speed digital video analysis are only used as a backup in the event that vehicle speeds cannot be determined from the electronic data.

3.3.4 Digital Photography

Three AOS high-speed digital video cameras and three GoPro digital video cameras were used to document each test. The AOS high-speed camera had a frame rate of 500 frames per second, and the GoPro video camera had a frame rate of 120 frames per second. The cameras were placed laterally from the post, with a view perpendicular to the bogie's direction of travel, as well as diagonally from the post. A Nikon D50 digital still camera was used to document pre- and post-test conditions for all tests.

3.4 End of Test Determination

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

3.5 Data Processing

The electronic accelerometer data obtained in dynamic testing was filtered using the SAE Class 60 Butterworth filter conforming to SAE J211/1 specifications [10]. The pertinent acceleration signal was extracted from the bulk data signals. The processed acceleration data was

then multiplied by the mass of the bogie to get the impact force using Newton's Second Law. Next, the acceleration trace was integrated to find the change in velocity versus time. Initial velocity of the bogie, calculated from the pressure tape switch data, was then used to determine the bogie velocity, and the calculated velocity trace was integrated to find the bogie's deflection. This deflection is also the deflection of the post. Combining the previous results, a force versus deflection curve was plotted for each test. Finally, integration of the force versus deflection curve provided the energy versus deflection curve for each test.

4 COMPONENT TESTING RESULTS AND DISCUSSION

4.1 Results

Results from the dynamic component testing of the four anchorage systems for the BR27C combination bridge rail are detailed in the subsequent section. In each test, acceleration data, high-speed video, and post-test documentation of the system damage were used to evaluate the anchorages. The accelerometer data for each test was processed in order to obtain acceleration, velocity, and deflection curves, as well as force versus deflection and energy versus deflection curves. Although the individual transducers produced similar results, the values described herein were calculated from the SLICE-2 data curves in order to provide common basis for comparing results from multiple tests. Test results for all transducers are provided in Appendix C. A summary of the four dynamic component tests is shown in Table 1.

Table 1. Dynamic Testing Summary

Test No.	Design Configuration	Target Impact Velocity (mph) [km/h]	Impact Height (in.) [mm]	Impact Angle (degrees)
IBP-1	Original BR27C Cast-In-Place	15.0 [24.1]	16 [406]	90
IBP-2	Four-Bolt Spread	15.0 [24.1]	16 [406]	90
IBP-3	Two-Bolt Offset	15.0 [24.1]	16 [406]	90
IBP-4	US-20 Bridge	15.0 [24.1]	16 [406]	90

4.1.1 Test No. IBP-1

During test no. IBP-1, the bogie impacted the HSS 4-in. x 4-in. x $\frac{3}{16}$ -in. (102-mm x 102-mm x 5-mm) steel post at a speed of 16.1 mph (25.9 km/h), causing the post to deflect backward. During the test, shear cracks formed starting at the front anchors that propagated to the backside of the parapet. This concrete failure caused significant damage to the parapet but did not cause the yielding of the post. The post continued to rotate backwards, causing additional fracture and disengagement of the concrete parapet behind the post. The two front anchor rods on the post fractured in tension approximately 66 msec after impact, causing the loading of the bogie vehicle to drop to zero at a deflection of 13 in. (330 mm). The bogie overrode the top of the post at approximately 224 msec, as determined from the high-speed film data. Sequential photographs of the test are shown in Figure 36.

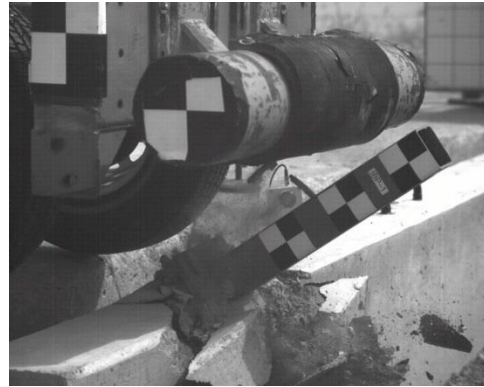
Damage to the system consisted of major damage to the concrete parapet and the cast-in-place anchorage, as shown in Figure 37. The concrete parapet displayed shear cracking along the top of the parapet and disengagement of a large section of concrete on the backside of the parapet. Lesser amounts of concrete were disengaged on the top and front sides of the parapet. The post and baseplate assembly were largely undamaged. The post and baseplate displayed minimal local deformations due to the impact, and the post did not form a plastic hinge. The threaded rod anchors on the front of the parapet fractured during the test, and the rear anchors were bent backward due to the rotation of the post.

Force versus deflection and energy versus deflection curves were created from the accelerometer data and are shown in Figure 38. A peak force of 22.9 kips (101.9 kN) was reached at a deflection of 1.5 in. (38 mm), prior to the disengagement of sections of the concrete parapet. The post continued to develop load as the post deflected until the fracture of the front

two anchor rods. At a maximum deflection of 13 in. (330 mm), the post assembly absorbed 146 kip-in. (16.5 kJ) of energy.



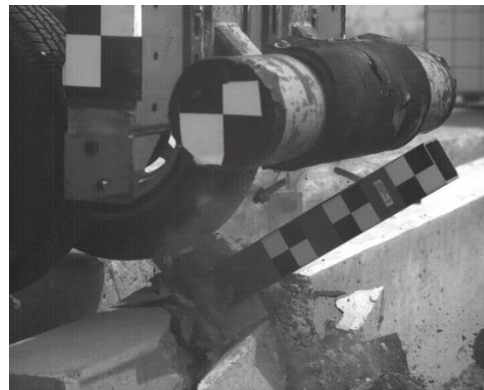
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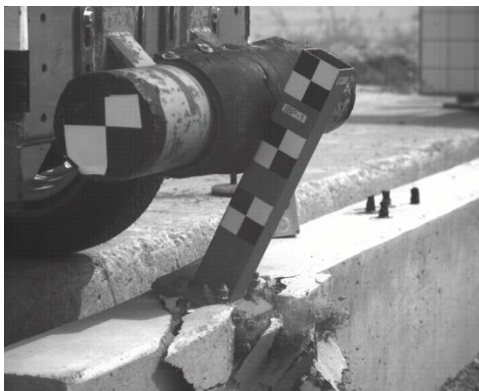
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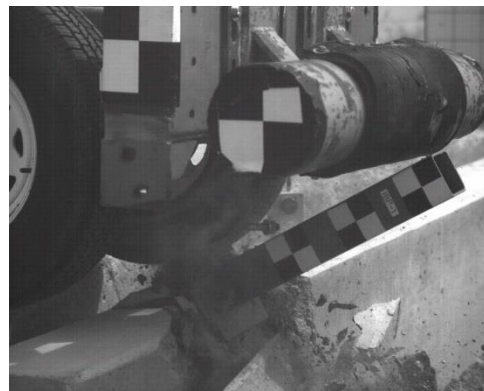
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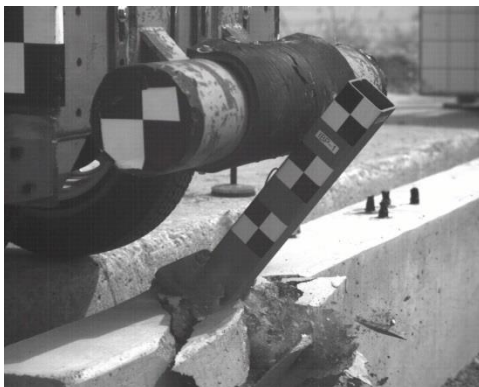
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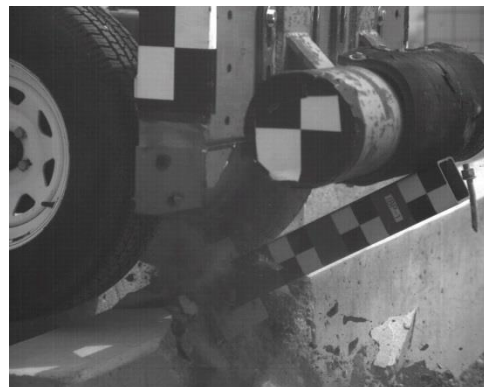
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Figure 36. Sequential Photographs, Test No. IBP-1



Figure 37. Post-Impact Photographs, Test No. IBP-1

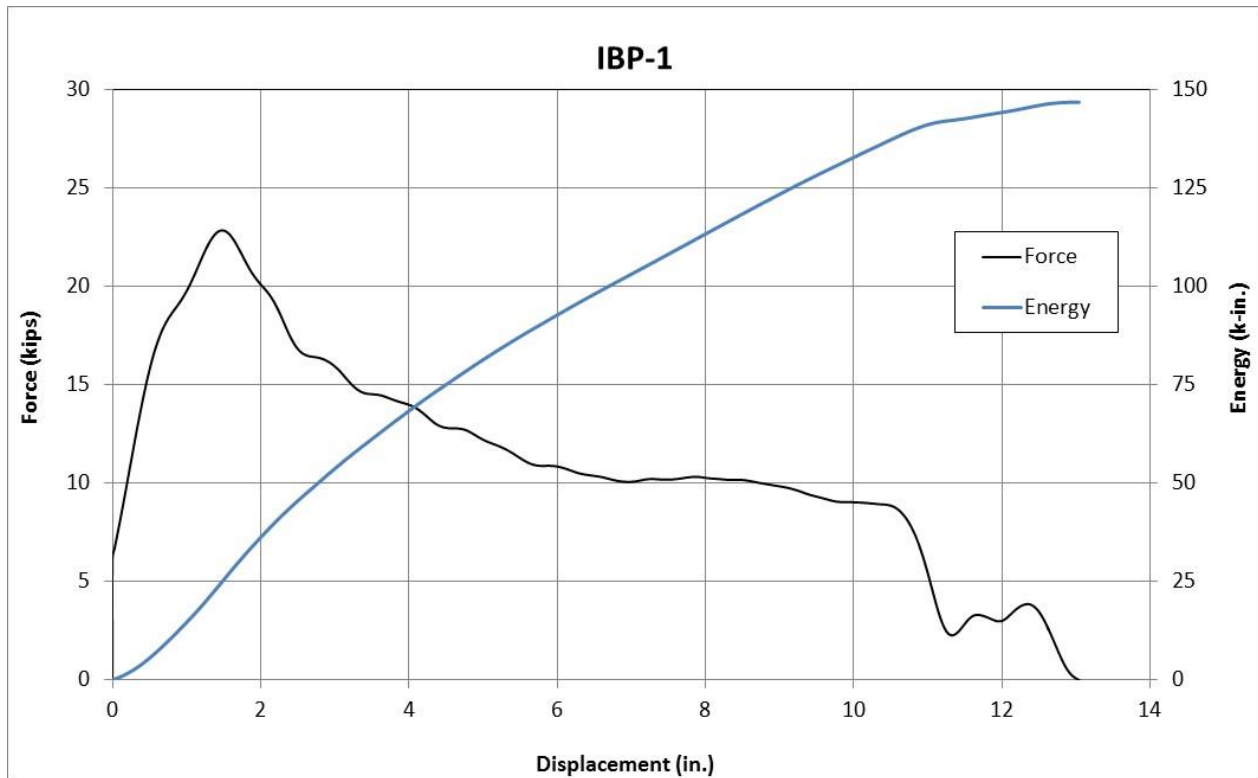


Figure 38. SLICE-2 Force vs. Deflection and Energy vs. Deflection, Test No. IBP-1

4.1.2 Test No. IBP-2

During test no. IBP-2, the bogie impacted the HSS 4-in. x 4-in. x $\frac{3}{16}$ -in. (102-mm x 102-mm x 5-mm) steel post at a speed of 16.2 mph (26.1 km/h), causing the post to deflect backward. During the test, shear cracks formed starting at the rear anchors that propagated to the backside of the parapet, and which disengaged a large section of the rear face of the parapet. At the same time, loading of the front two anchors caused cracking and concrete disengagement on the top-front of the parapet. The impact loads caused concrete failure and significant damage to the parapet but did not cause the yielding of the post. As the post continued to rotate, all four anchor rods were pried from the fracture parapet. The force on the bogie vehicle dropped to zero at a deflection of 11.9 in. (302 mm). The bogie overrode the top of the post at approximately 156 msec, as determined from the high-speed film data. Sequential photographs of the test are shown in Figure 39.

Damage to the system consisted primarily of damage to the concrete parapet, as shown in Figure 40. The concrete parapet displayed shear cracking along the top and disengagement of a large section of concrete on the backside. Lesser amounts of concrete were disengaged on the top and front sides of the parapet. The post and baseplate assembly were largely undamaged. The post and baseplate displayed minimal local deformations due to the impact, and the post did not form a plastic hinge. The four threaded rod anchors were all disengaged from the parapet due to the impact loads and fracture of the surrounding concrete.

Force versus deflection and energy versus deflection curves were created from the accelerometer data and are shown in Figure 41. A peak force of 24.9 kips (110.8 kN) was reached at a deflection of 1.4 in. (36 mm), prior to the disengagement of sections of the concrete parapet. The post continued to develop load as the post deflected until the disengagement of the

anchor rods from the parapet. At a maximum deflection of 11.9 in. (302 mm), the post assembly absorbed 69.6 kip-in. (7.9 kJ) of energy.

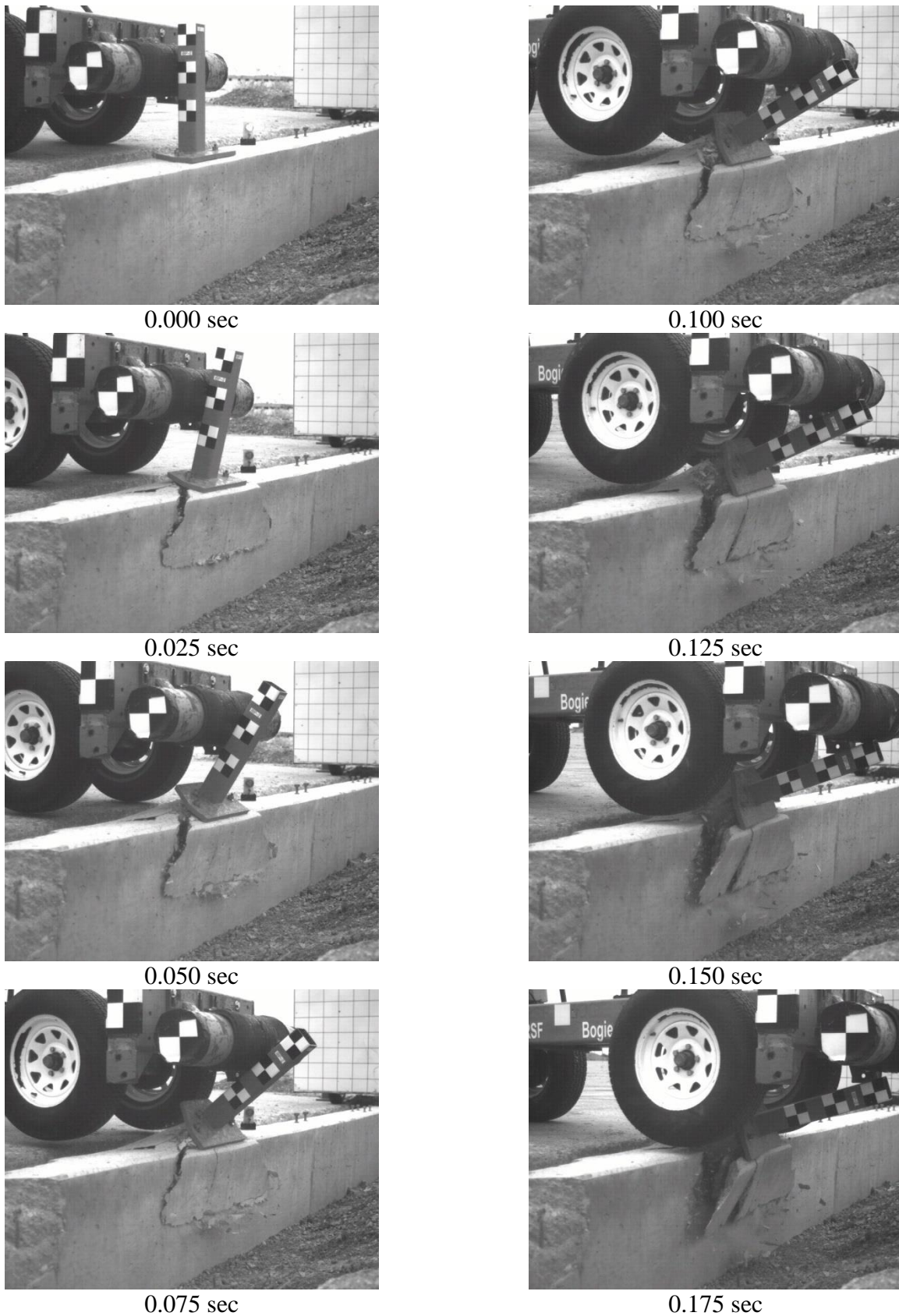


Figure 39. Sequential Photographs, Test No. IBP-2



Figure 40. Four-Anchor Post-Impact Photographs, Test No. IBP-2

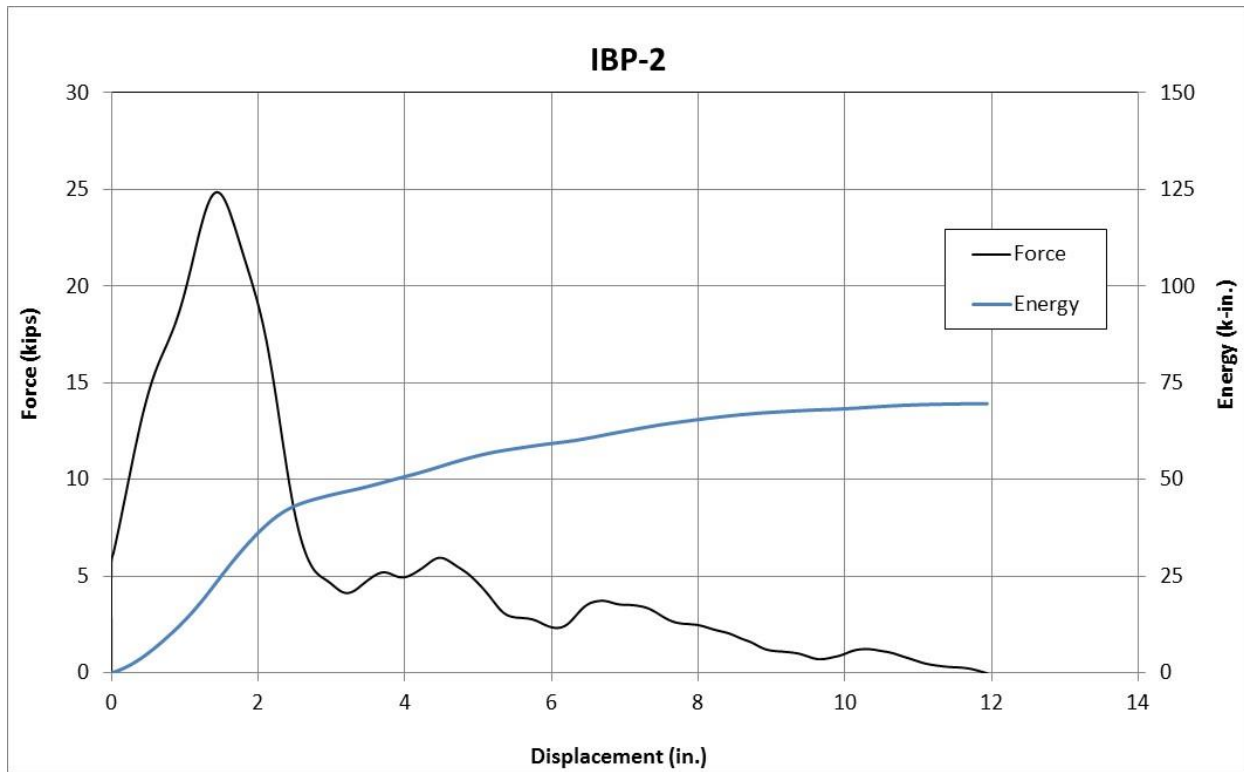


Figure 41. SLICE-2 Force vs. Deflection and Energy vs. Deflection, Test No. IBP-2

4.1.3 Test No. IBP-3

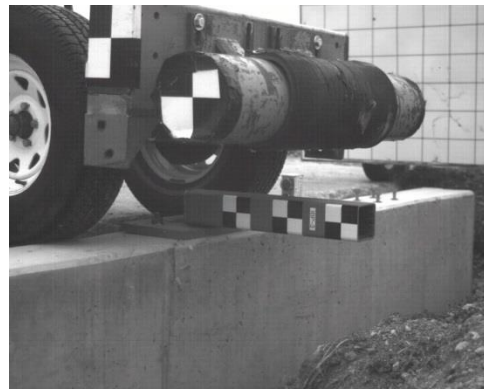
During test no. IBP-3, the bogie impacted the HSS 4-in. x 4-in. x $\frac{3}{16}$ -in. (102-mm x 102-mm x 5-mm) steel post at a speed of 16.3 mph (26.2 km/h), causing the post to deflect backward. During the test, shear cracks formed starting at the anchors and propagated to the backside of the parapet. As the bogie continued to load the post, the weld between the post and the baseplate fractured on the front-side of the post approximately 10 msec after impact. As the post continued to deflect, the weld between the post and the baseplate fractured along both sides of the post, allowing the post to rotate backward. The force on the bogie vehicle dropped to zero at a deflection of 2.7 in. (69 mm). The post completely disengaged from the baseplate at approximately 112 msec, as determined from the high-speed film data. Sequential photographs of the test are shown in Figure 42.

Damage to the system consisted of damage to the concrete parapet and the weld between the post and the baseplate, as shown in Figure 43. The concrete parapet displayed shear cracking along the top of the parapet as well as some cracking of the top of the rear face of the parapet. No significant sections of concrete were disengaged from the parapet in this test. The post and baseplate assembly were not deformed, but the weld between them was completely fractured at the base of the post. The two threaded rod anchors remained embedded in the concrete.

Force versus deflection and energy versus deflection curves were created from the accelerometer data and are shown in Figure 44. A peak force of 28.3 kips (125.9 kN) was reached at a deflection of 1.4 in. (36 mm). The post continued to develop load as the post deflected until the fracture of the weld between the post and the baseplate. At a maximum deflection of 2.7 in. (69 mm), the post assembly absorbed 48.3 kip-in. (5.5 kJ) of energy.



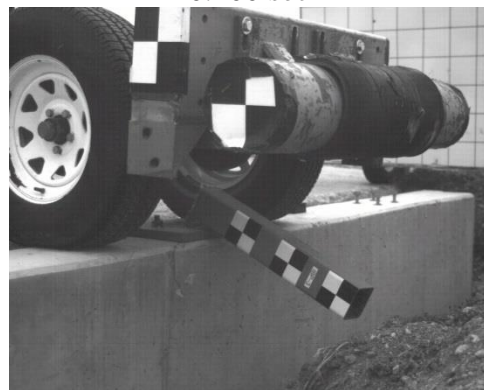
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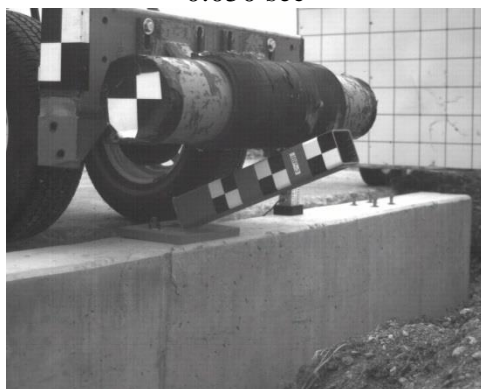
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Figure 42. Sequential Photographs, Test No. IBP-3



Figure 43. Post-Impact Photographs, Test No. IBP-3

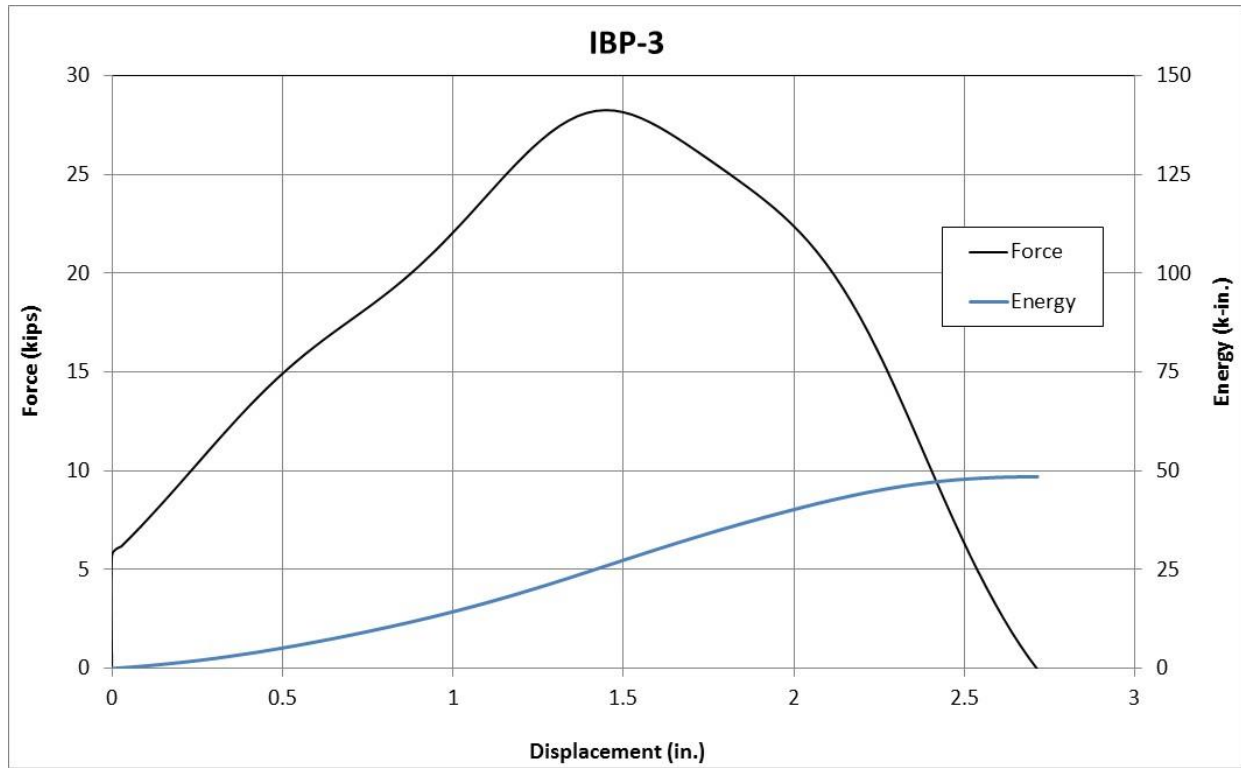


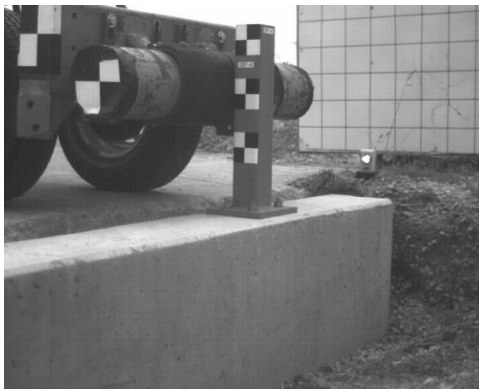
Figure 44. SLICE-2 Force vs. Deflection and Energy vs. Deflection, Test No. IBP-3

4.1.4 Test No. IBP-4

During test no. IBP-4, the bogie impacted the HSS 4-in. x 4-in. x $\frac{3}{16}$ -in. (102-mm x 102-mm x 5-mm) steel post at a speed of 15.4 mph (24.8 km/h), causing the post to deflect backward. During the test, the deflection of the post caused uplift of the front of the baseplate, which caused the front two threaded anchors to fail in tension approximately 12 msec after impact. The post continued to rotate backwards, causing shear cracks to form at the two back anchors and propagate towards the backside of the parapet. The shear cracks and the continued rotation of the steel baseplate caused disengagement of a section of the back of the concrete parapet. The loading of the bogie vehicle dropped to zero at a deflection of 3.4 in. (86 mm). The bogie overrode the top of the post at approximately 166 msec, as determined from the high-speed film data. Sequential photographs of the test are shown in Figure 45.

Damage to the system consisted of damage to the concrete parapet and the anchor rods, as shown in Figure 46. The concrete parapet displayed cracking on the top and disengagement of a section of concrete on the backside. The post and baseplate assembly were largely undamaged. The post and baseplate displayed minimal local deformations due to the impact, and the post did not form a plastic hinge. The threaded rod anchors on the front of the parapet fractured during the test, and the rear anchors were bent backward due to the rotation of the post.

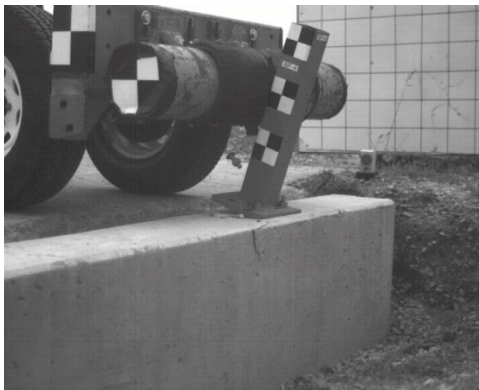
Force versus deflection and energy versus deflection curves were created from the accelerometer data and are shown in Figure 47. A peak force of 23.2 kips (103.2 kN) was reached at a deflection of 1.4 in. (36 mm), prior to the fracture of the two front anchor rods. At a maximum deflection of 3.4 in. (86 mm), the post assembly absorbed 60.3 kip-in. (6.8 kJ) of energy.



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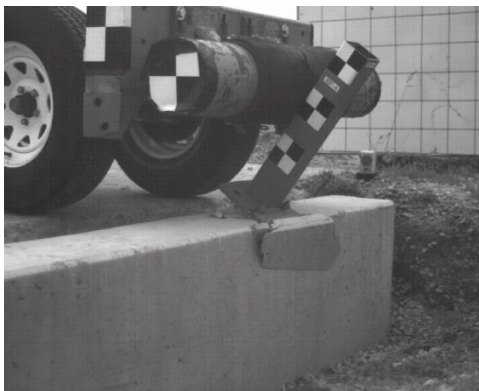
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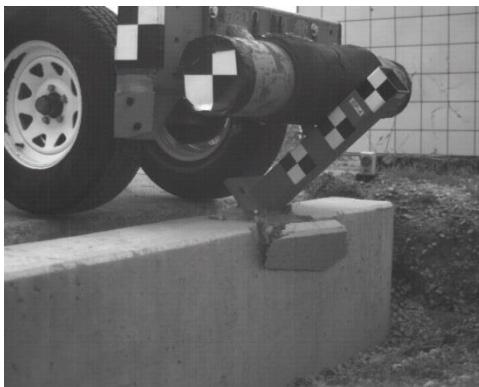
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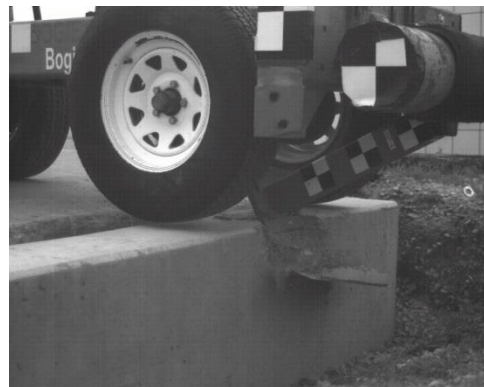
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Figure 45. Sequential Photographs, Test No. IBP-4



Figure 46. US-20 River Bridge Post-Impact Photographs, Test No. IBP-4

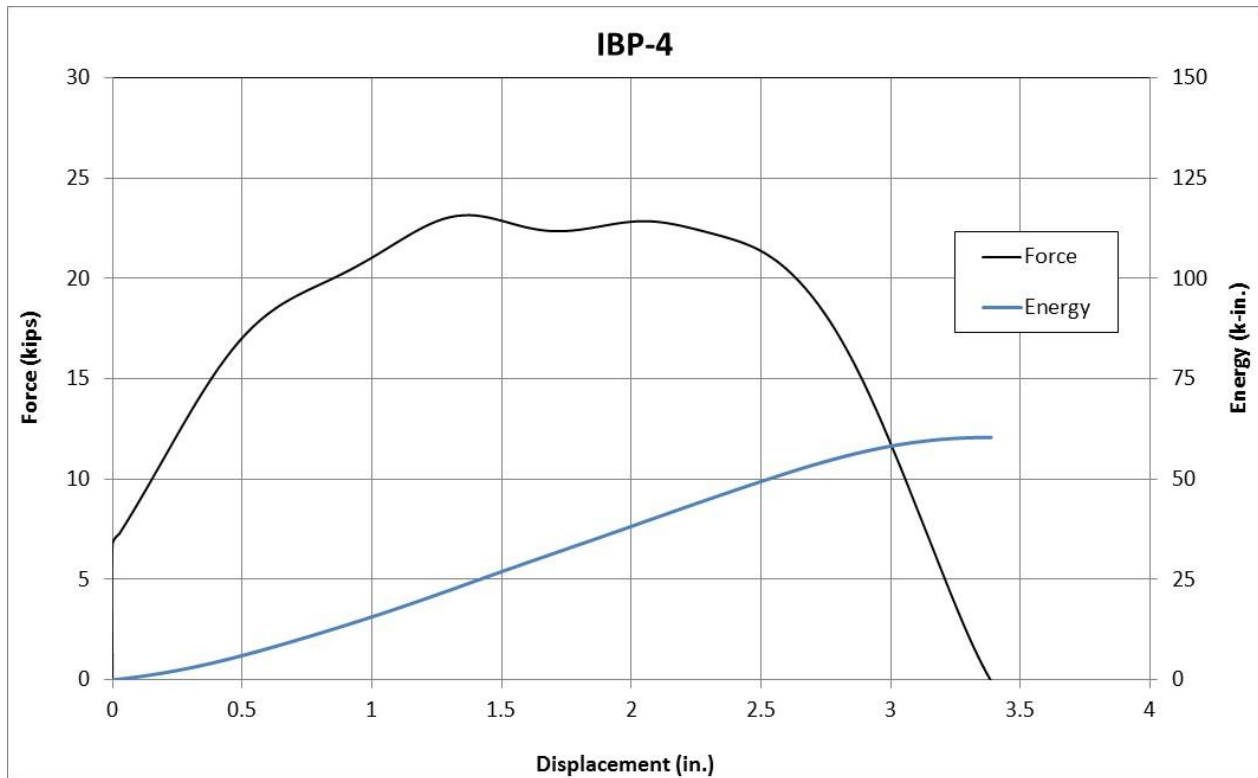


Figure 47. SLICE-2 Force vs. Deflection and Energy vs. Deflection, Test No. IBP-4

4.2 Discussion

The purpose of the dynamic component testing was to determine if two proposed and one currently installed alternative epoxy adhesive anchorages for the BR27C combination bridge rail had sufficient capacity to be used in lieu of the cast-in-place anchorage that was used in the original full-scale crash-tested design. Thus, the dynamic tests were used to evaluate and compare the force versus deflection behavior and the failure modes of the proposed designs to the baseline cast-in-place anchorage. A summary of all dynamic component testing results is shown in Table 2.

Table 2. Dynamic Testing Results

Test No.	Design Configuration	Impact Velocity (mph) [km/h]	Peak Force (kips) [kN]	Max Deflection (in.) [mm]	Total Energy Absorbed (k-in.) [kJ]
IBP-1	Original BR27C Cast-In-Place	16.1 [25.9]	22.9 [101.9]	13.0 [330]	146.0 [16.5]
IBP-2	Four-Bolt Spread	16.2 [26.1]	24.9 [110.8]	11.9 [302]	69.6 [7.9]
IBP-3	Two-Bolt Offset	16.3 [26.2]	28.3 [125.9]	2.7 [69]	48.3 [5.5]
IBP-4	US-20 Bridge	15.4 [24.8]	23.2 [103.2]	3.4 [86]	60.3 [6.8]

The force versus deflection data for the four dynamic component tests as derived from SLICE-2 acceleration transducer, is shown in Figures 48 and 49. Comparison of the results from the four tests found that all three of the alternative epoxy adhesive anchorage designs exceeded the peak force of the original cast-in-place anchorage. The cast-in-place anchorage evaluated in test no. IBP-1 developed the lowest peak force of all the anchorages with a value of 22.9 kips

(101.9 kN) at a deflection of 1.5 in. (38 mm). The US-20 bridge design evaluated in test no. IBP-4 had the next highest peak force with a value of 23.2 kips (103.2 kN) at a deflection of 1.4 in. (36 mm). The four-bolt spread anchorage evaluated in test no. IBP-2 had the third highest peak force with a value of 24.9 kips (110.8 kN) at a deflection of 1.4 in. (36 mm). The two-bolt offset anchorage evaluated in test no. IBP-3 developed the highest peak force with a value of 28.3 kips (125.9 kN) at a deflection of 1.4 in. (36 mm). The forces after the peak force was reached differ for the four anchorages, depending on the failure mode of the anchorage.

The energy versus deflection data for the four dynamic component tests is shown in Figures 50 and 51. Energy levels for all four of the tested anchorages were similar through the first 2 in. (51 mm) of post deflection, but diverged similar to the force levels after that point due to variation in the failure modes.

These results were reviewed to determine the feasibility of the alternative anchorage designs. The original cast-in-place anchorage for the BR27C generated the lowest peak load of the four anchorages. The failure modes observed for this design were a combination of tensile failure of the front anchor rods and breakout of the concrete on the rear of the parapet. This level of damage was much higher than the damage observed in full-scale crash testing. In the full-scale tests, no failure of anchor rods or the concrete parapet was noted. This would indicate that the damage and force levels developed in the component testing were significantly higher than the loading of the post and anchorage during full-scale testing. Thus, alternative designs that exceeded the peak force of the original cast-in-place anchorage should be considered acceptable.

The four-bolt spread anchorage design developed higher peak loads than the original cast-in-place anchorage. Energy levels for the two designs differed, as the cast-in-place anchorage did not completely disengage from the concrete and developed load longer after the initial peak load was reached. Higher peak loads were expected for the four-bolt spread

anchorage based on the increased anchor spacing providing reduction of the overlapped area of influence for the epoxy adhesive anchors, but the peak forces developed in testing found those gains to be minimal. Review of the failure of the anchorage showed that orienting the front and rear anchors for this design diagonal to one another may have allowed shear stresses and cracking to develop along the same plane for both the front and rear anchor simultaneously. This may have contributed to the lower-than-expected improvement in force level of the four-bolt spread anchorage. However, the four-bolt spread anchorage did possess improved capacity to the original cast-in-place anchorage and would be considered an acceptable alternative.

The two-bolt offset anchorage design developed the highest peak load of all of the tested designs. This design also exhibited less damage to the concrete parapet, as the increased offset from the rear face of the parapet increased the shear capacity of the anchorage over the other alternatives. The failure mode for this design was rupture of the weld between the baseplate and the post. Thus, it is the only design tested that did not result in failure of the anchorage itself. The two-bolt offset anchorage was also considered to be an acceptable alternative anchorage. The two-bolt offset anchorage also posed an advantage, in that it required fewer anchors and would be easier to install.

The US-20 bridge anchorage displayed a peak force and failure modes that were quite similar to the original cast-in-place anchorage design. This was not unexpected, as the two designs were similar in terms of the layout and anchor size. The US-20 bridge anchorage was considered to be an acceptable alternative anchorage.

Thus, all three of the alternative anchorage designs were considered to be acceptable alternatives to the original cast-in-place anchorage design. The peak force levels for the alternative anchorages indicated greater capacities than the original anchorage, and the damage levels observed in the dynamic component testing far exceeded the levels observed in full-scale

crash tests. As such, there was no reason to believe that the alternative anchorages would not perform safely. Of the three alternative designs, the two-bolt offset design was deemed the best option due to its potential to reduce parapet damage and improved its ease of installation.

It should be noted that all of the alternative designs were developed and tested on the 12-in. (305-mm) wide version of the IaDOT concrete parapet. These results would likely change if the alternative epoxy anchorages were evaluated on the narrower parapet used with the original cast-in-place anchorage. It should also be noted that the four-bolt spread and two-bolt offset anchorages were designed to develop the full plastic moment capacity of the support post. Based on the test results, the four-bolt spread anchorage was not capable of developing the moment capacity of the post due to concrete breakout in shear. The two-bolt offset design may have had the potential to develop the moment capacity, but the post-to-baseplate weld failed prior to reaching that load. This does not affect the suitability of the alternative anchorages as replacements for the cast-in-place design, but it does suggest that the design calculations for concrete breakout in shear may need further development when considering anchorage for dynamic impact on narrow parapets.

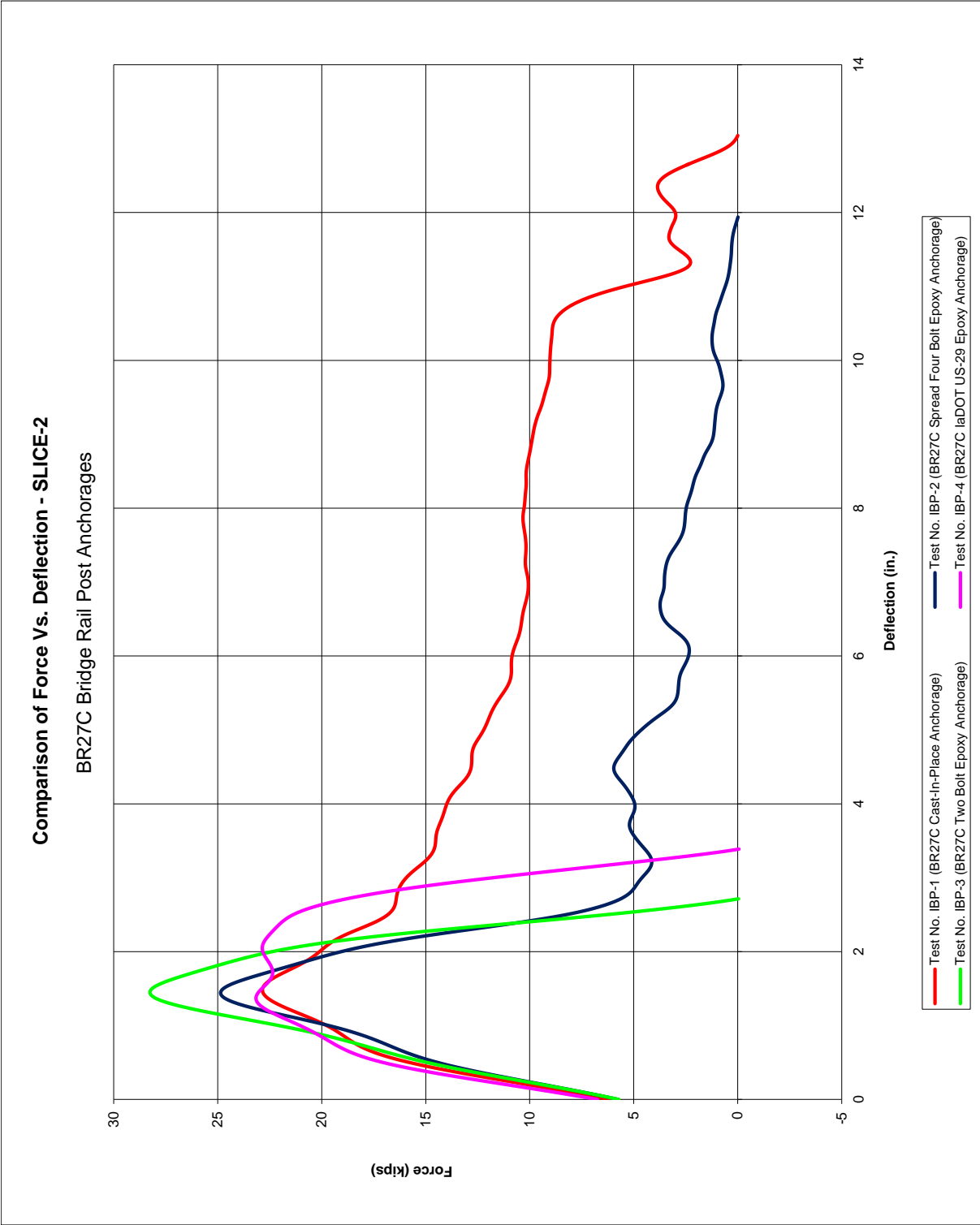


Figure 48. SLICE-2 Force vs. Deflection Comparison, All Bogie Tests

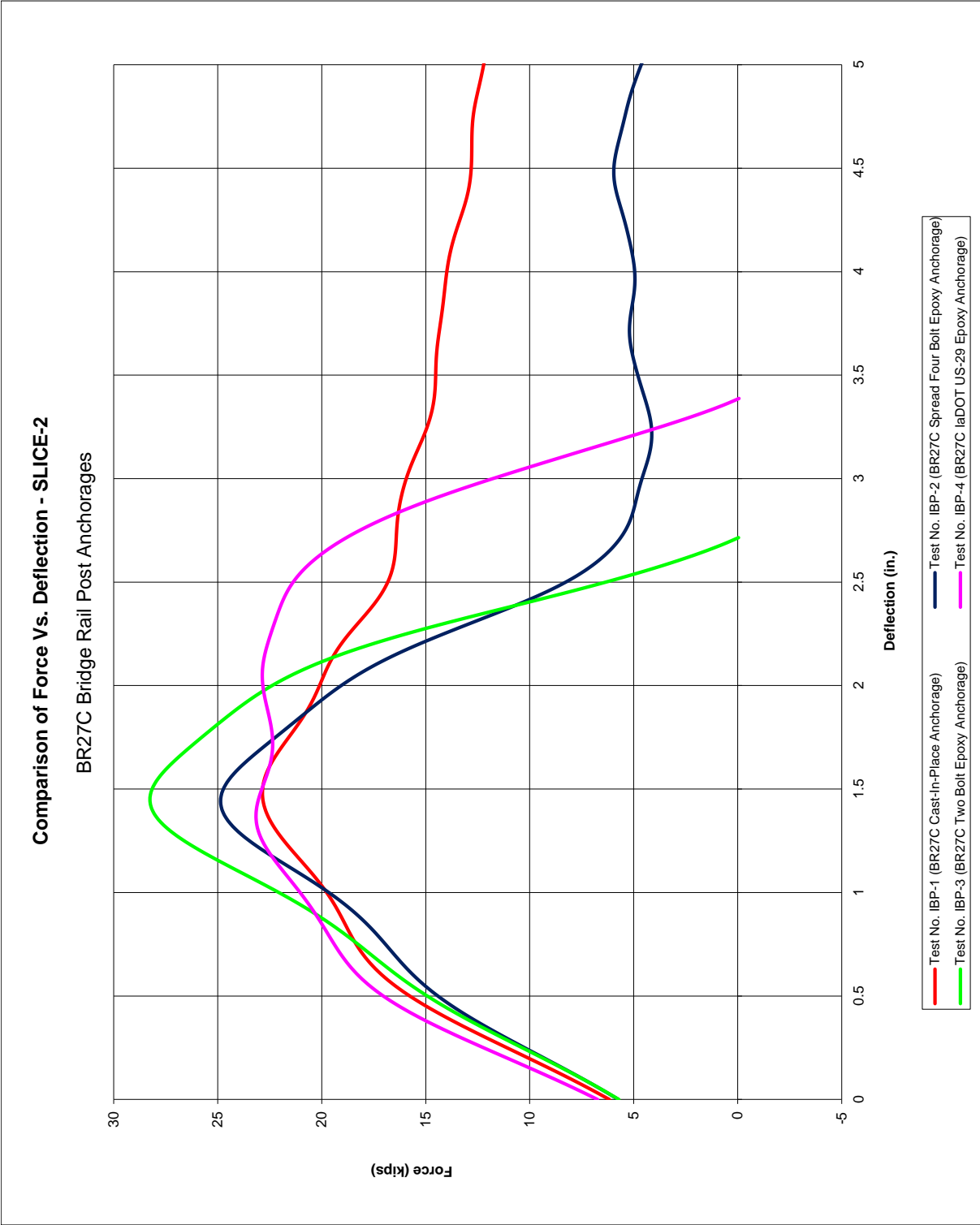


Figure 49. SLICE-2 Force vs. Deflection Comparison Zoom, All Bogie Tests

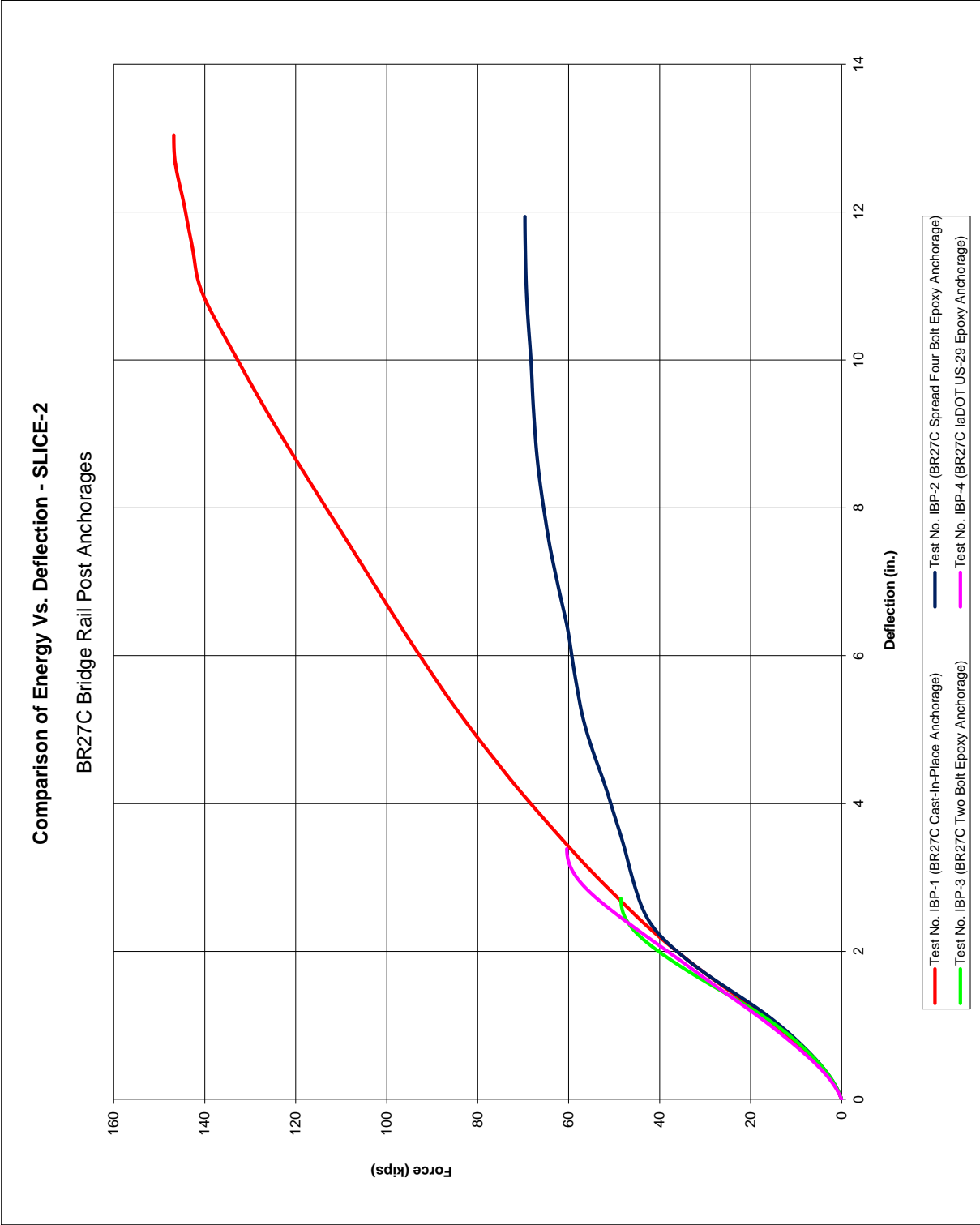


Figure 50. SLICE-2 Energy vs. Deflection Comparison, All Bogie Tests

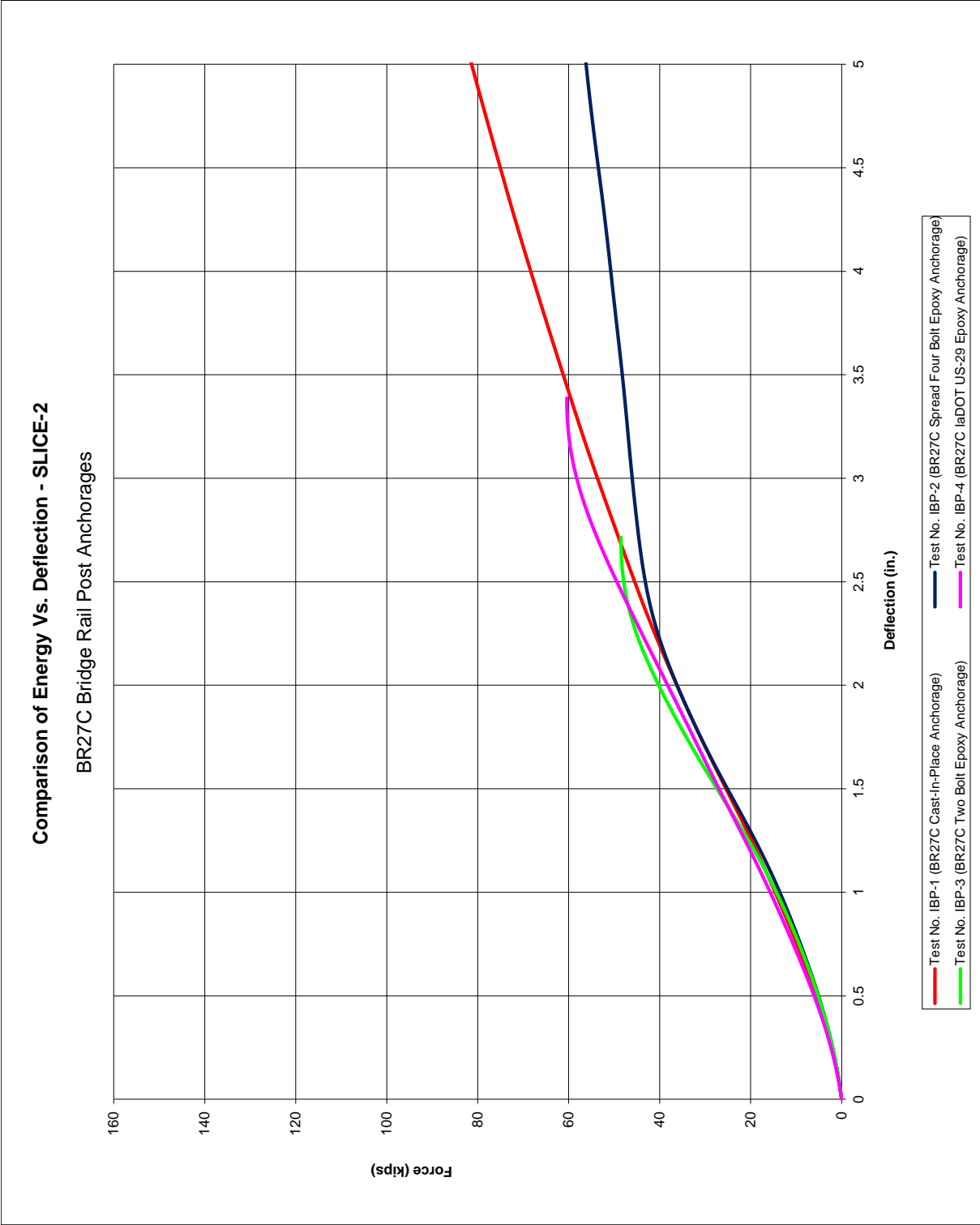


Figure 51 SLICE-2 Energy vs. Deflection Comparison Zoom, All Bogie Tests

5 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The objective of this research was to develop and evaluate alternative epoxy adhesive anchorage systems for the BR27C combination bridge rail system. The BR27C combination bridge rail was originally designed and tested with a 24-in. (610-mm) tall by 10-in. (254-mm) wide vertical concrete parapet with a steel post-and-rail system mounted on top. The steel posts in the combination rail were attached to the concrete parapet with cast-in-place concrete anchors. IaDOT desired an alternative epoxy adhesive anchor design that would be easier to install.

The research effort began with development of several proposed alternative anchorage concepts. The concepts were designed using a modified version of the ACI 318-11 procedures for adhesive anchor design with modifications for dynamic increase factors and the expected failure modes. All of the concepts were designed to develop the full plastic moment capacity of the post. Four design concepts were developed for review by IaDOT, including: (1) a four-bolt, square anchorage, (2) a four-bolt, spread anchorage, (3) a two-bolt, centered anchorage, and (4) a two-bolt, offset anchorage. IaDOT representatives selected the four-bolt spread anchorage and the two-bolt offset anchorage as the preferred designs for evaluation. In addition to these two proposed configurations, IaDOT also requested that the researchers evaluate a third option that had been previously installed on the US-20 bridge near Hardin, IA.

In order to evaluate the performance of the proposed alternative anchorages, dynamic component testing was conducted on the original cast-in-place anchorage as well as the three alternative anchorages using a simulated bridge rail parapet. The test of the original cast-in-place anchorage test no. IBP-1 was used as a baseline for comparison with the alternative designs and developed a peak load of 22.9 kips (101.9 kN) at a deflection of 1.5 in. (38 mm). All three of the tested alternative anchorages provided greater load capacity than the original cast-in-place design and were deemed acceptable surrogates. Of the three alternative designs, the two-bolt offset

design was deemed the best option due to its developing the highest peak loads, the potential for reduced parapet damage, and improved ease of installation. It was also noted that the alternative designs were developed and tested on a 12-in. (305-mm) wide version of the IaDOT concrete parapet. Thus, the alternative anchorages would not be recommended for use on the narrower parapet used with the original cast-in-place anchorage.

6 REFERENCES

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7 APPENDICES

Appendix A. Alternative Epoxy Adhesive Anchor Design Calculations

The anchorage calculations used during the development of the four design concepts presented in this research are detailed herein. The calculations were based on development of the full-plastic moment capacity of the BR27C combination bridge rail post and the corresponding shear and tensile loads when used with the 12-in. (305-mm) wide parapet design provided by IaDOT. Details of the design of the baseplates for the posts are not included.

TENSION ANCHORS (FRONT FACE)

Embedment Depth, h_{ef} :	5 in.
Steel Bar Diameter, d_b :	0.625 in.
Area of Steel, A_s :	0.226 in. ²
Front (Tension) Anchor Spacing, s :	6.5 in.
Front (Tension) Anchor to deck edge, $c_{a,min}$:	4.625 in.
Bond Strength, τ_{cr} :	1800 psi
Steel Ultimate Strength, f_{uta} :	120 ksi
Concrete Strength, f_c :	4000 psi
Deck Reinforced? (γ/n):	γ

Tension Strengths

Failure Mode	Load (kips)
Steel Fracture:	24.00
Concrete Breakout:	10.43
Bond Failure:	7.86
Hybrid:	18.29

Steel DIF, ψ_{sd} :	1.18
Concrete DIF, ψ_{cd} :	1.88
Adhesive/Bond DIF, ψ_{bd} :	1.484

	Tension	Shear
ACI Steel Strength Reduction Factor, ϕ_s :	0.75	0.65
ACI Concrete Strength Reduction Factor, ϕ_c :	0.65	0.75
ACI Adhesive Strength Reduction Factor, ϕ_a :	0.65	NA

TENSION CAPACITY

Steel Fracture: $\Phi N_s = A_s N_s f_{uta} \psi_{sd}$

$\Phi N_s = 24.00$ kips

Concrete Breakout: $\Phi N_{cb} = A_{Nc}/A_{Nco} * \psi_{ed,N} \psi_{cp,N} \psi_{cd} * N_b$

$N_b = k_c * h_{ef}^{1.5} \sqrt{f_c}$
 $k_c = 17$ (24 for cast in place, 17 for post installed)

$\psi_{cp,N} = 1.4$ (1.25 for cast in anchors, 1.4 for post installed, 1.0 if kc taken from external document without further instructions)

$N_b = 12.02$ kips

$c_{ac} = 10$

$\psi_{cp,N} = 1$

$\psi_{ed,N} = 0.885$

$A_{Nco} = 9 * h_{ef}^2 = 225$ in.²

$A_{Nc} = 129$ in.²

$A_{Nc}/A_{Nco} = 0.573333$

Note - Calculated based on influence area per anchor

$\Phi N_{cb} = 10.43$ kips

Adhesive / Bond Failure: $\Phi N_a = A_{Na}/A_{Na0} * \psi_{ed,Na} \psi_{cp,Na} \psi_{bd} * N_{ba}$

$N_{ba} = \tau_{cr} \pi d_b h_{ef}$

$N_{ba} = 17.67$ kips

$A_{Na0} = (2 * C_{Na})^2$

$C_{Na} = 10 * d_b * \sqrt{\tau_{cr}/1100}$

$C_{Na} = 8.00$ in.

$A_{Na0} = 255.68$ in.²

$A_{Na} = 134.9403$ in.²

$A_{Na}/A_{Na0} = 0.527767$

Note - Calculated based on influence area per anchor

$\psi_{cp,Na} = 1$ (should be the same as $\psi_{cp,N}$)

$\psi_{ed,Na} = 0.873545$

$\Phi N_a = 7.86$ kips

Figure A-1. Tensile Adhesive Anchorage Calculations, Four-Bolt Square Anchorage Concept

SHEAR ANCHORS (BACK FACE)

Embedment Depth, h_{ef} :	10 in.
Steel Bar Diameter, d_b :	0.625 in.
Area of Steel, A_s :	0.226 in. ²
Anchor Spacing, s :	6.5 in.
Anchor to Deck Edge Distance, c_{ad} :	4.625 in.
Steel Ultimate Strength, f_{tsa} :	120 ksi
Concrete Strength, f'_c :	4000 psi
Deck Thickness, h_a :	26 in.
Deck Reinforced? (y/n):	y
Bond Strength, τ_{cr} :	1800 psi

Shear Strengths

Failure Mode	Load (kips)
Steel Fracture:	12.48
Concrete Breakout:	7.65
Concrete Pryout:	15.25

Total Anchor Shear for Barrier

LCR: 20 ft

$\Phi V_{s, barrier} = 282.38$ kips

SHEAR CAPACITY

Steel Fracture: $\Phi V_{sa} = 0.6 A_s f_{tsa} \Psi_{ed}$

$\Phi V_{sa} = 12.48$ kips

Concrete Breakout: $\Phi V_{cb} = A_{vc}/A_{vc0} \Psi_{ed,V} \Psi_{c,N} \Psi_{h,V} \Psi_{ed} V_b$

$$V_{b1} = 7 \cdot (l_e/d_b)^{0.2} \cdot \sqrt{d_b} \cdot \sqrt{f'_c} \cdot C_{ad}^{1.5}$$

$$l_e = 5.00$$

$$V_{b1} = 5.28 \text{ kips}$$

$$V_{b2} = 9 \cdot C_{ad}^{1.5} \cdot \sqrt{f'_c}$$

$$5.66 \text{ kips}$$

$$V_b = \min(V_{b1}, V_{b2}) = 5.28 \text{ kips}$$

$$\Psi_{ed,V} = 1 \text{ (only reduced for anchor adjacent to deck discontinuity)}$$

$$\Psi_{c,N} = 1.4 \text{ (1.4 for uncracked deck, 1.2 for cracked reinforced, 1.0 for cracked unreinforced deck)}$$

$$\Psi_{h,V} = 1.00$$

$$A_{vc0} = 4.5 \cdot (c_{ad})^2 = 96.25781 \text{ in.}^2$$

$$A_{vc} = 70.67578 \text{ in.}^2$$

$$A_{vc0}/A_{vc} = 0.734234$$

$\Phi V_{cb} = 7.65$ kips

Concrete Pryout Strength: $\Phi V_{cp} = k_{cp} N_q$

$$k_{cp} = 2$$

$$N_q = \min(N_{cb}, N_a)$$

$$N_b = A_{N0}/A_{N00} \cdot \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} \Psi_{ed} \cdot N_b$$

$$N_b = k_c \cdot h_{ef}^{1.5} \cdot \sqrt{f'_c}$$

$$k_c = 17$$

$$\Psi_{c,N} = 1.4$$

$$N_b = 34.00 \text{ kips}$$

$$c_{ac} = 20$$

$$\Psi_{cp,N} = 1$$

$$\Psi_{ed,N} = 0.7925$$

$$A_{N00} = 9 \cdot h_{ef}^2 = 900 \text{ in.}^2$$

$$A_{Nc} = 129 \text{ in.}^2$$

$$A_{Nc}/A_{N00} = 0.143333$$

$$N_a = 10.17$$

$$N_q = 10.17$$

$\Phi V_{cp} = 15.25$ kips

$$N_a = A_{N0}/A_{N00} \cdot \Psi_{ed,N0} \Psi_{c,N0} \Psi_{cp,N0} \Psi_{ed} \cdot N_{b0}$$

$$N_{b0} = \tau_{cr} \cdot \pi \cdot d_b \cdot h_{ef}$$

$$N_{b0} = 35.34 \text{ kips}$$

$$A_{N00} = (2 \cdot C_{Na})^2$$

$$C_{Na} = 10 \cdot d_b \cdot \sqrt{\tau_{cr}/1100}$$

$$C_{Na} = 8.00$$

$$A_{N00} = 255.68 \text{ in.}^2$$

$$A_{Na} = 134.9403 \text{ in.}^2$$

$$A_{Na}/A_{N00} = 0.527767$$

$$\Psi_{cp,Na} = 1 \text{ (should be the same as } \Psi_{cp,N})$$

$$\Psi_{ed,Na} = 0.873545$$

$$N_a = 24.18$$

Figure A-2. Shear Adhesive Anchorage Calculations, Four-Bolt Square Anchorage Concept

TENSION ANCHORS (FRONT FACE)

Embedment Depth, h_{ef} :	5 in.
Steel Bar Diameter, d_b :	0.625 in.
Area of Steel, A_s :	0.226 in. ²
Front (Tension) Anchor Spacing, s :	6.5 in.
Front (Tension) Anchor to deck edge, $c_{a,min}$:	4.625 in.
Bond Strength, τ_{cr} :	1800 psi
Steel Ultimate Strength, f_{uta} :	120 ksi
Concrete Strength, f'_c :	4000 psi
Deck Reinforced? (γ/n):	Y
Steel DIF, ψ_{sd} :	1.18
Concrete DIF, ψ_{cd} :	1.88
Adhesive/Bond DIF, ψ_{bd} :	1.484

Tension Strengths

Failure Mode	Load (kips)
Steel Fracture:	24.00
Concrete Breakout:	10.43
Bond Failure:	7.86
Hybrid:	18.29

	Tension	Shear
ACI Steel Strength Reduction Factor, ϕ_s :	0.75	0.65
ACI Concrete Strength Reduction Factor, ϕ_c :	0.65	0.75
ACI Adhesive Strength Reduction Factor, ϕ_a :	0.65	NA

TENSION CAPACITY

Steel Fracture: $\phi N_s = A_s N f_{uta} \psi_{sd}$

$\phi N_s = 24.00$ kips

Concrete Breakout: $\phi N_{cb} = A_{Nc}/A_{Nco} * \psi_{ed,N} \psi_{c,N} \psi_{cp,N} \psi_{cd} * N_b$

$N_b = k_c * h_{ef}^{1.5} \sqrt{f'_c}$

$k_c = 17$ (24 for cast in place, 17 for post installed)

$\psi_{c,N} = 1.4$ (1.25 for cast in anchors, 1.4 for post installed, 1.0 if k_c taken from external document without further instructions)

$N_b = 12.02$ kips

$c_{ac} = 10$

$\psi_{cp,N} = 1$

$\psi_{ed,N} = 0.885$

$A_{Nco} = 9 * h_{ef}^2 = 225$ in.²

$A_{Nc} = 129$ in.²

$A_{Nc}/A_{Nco} = 0.573333$

Note - Calculated based on influence area per anchor

$\phi N_{cb} = 10.43$ kips

Adhesive / Bond Failure: $\phi N_a = A_{Na}/A_{Na0} * \psi_{ed,Na} \psi_{cp,Na} \psi_{bd} * N_{ba}$

$N_{ba} = \tau_{cr} \pi d_b h_{ef}$

$N_{ba} = 17.67$ kips

$A_{Na0} = (2 * C_{Na})^2$

$C_{Na} = 10 * d_b * \sqrt{\tau_{cr}/1100}$

$C_{Na} = 8.00$ in.

$A_{Na0} = 255.68$ in.²

$A_{Na} = 134.9403$ in.²

Note - Calculated based on influence area per anchor

$A_{Na}/A_{Na0} = 0.527767$

$\psi_{cp,Na} = 1$ (should be the same as $\psi_{cp,N}$)

$\psi_{ed,Na} = 0.873545$

$\phi N_a = 7.86$ kips

Figure A-3. Tensile Adhesive Anchorage Calculations, Four-Bolt Spread Anchorage Concept

SHEAR ANCHORS (BACK FACE)

Embedment Depth, h_{ef} :	10 in.
Steel Bar Diameter, d_b :	0.625 in.
Area of Steel, A_s :	0.225 in. ²
Anchor Spacing, s :	13.5 in.
Anchor to Deck Edge Distance, c_{al} :	4.625 in.
Steel Ultimate Strength, f_{uta} :	120 ksi
Concrete Strength, f'_c :	4000 psi
Deck Thickness, h_d :	26 in.
Deck Reinforced? (y/n):	y
Bond Strength, τ_{cr} :	1800 psi

Shear Strengths

Failure Mode	Load (kips)
Steel Fracture:	12.48
Concrete Breakout:	10.28
Concrete Pryout:	30.85

Total Anchor Shear for Barrier

LCR:	20 ft
$\Phi V_{barrier}$:	182.67 kips

SHEAR CAPACITY

Steel Fracture: $\Phi V_{sa} = 0.6 * A_s * f_{uta} * \Psi_{sa}$
 $\Phi V_{sa} = 12.48$ kips

Concrete Breakout: $\Phi V_{cb} = A_{vc} / A_{vco} * \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} \Psi_{cd} * V_b$

$V_{b1} = 7 * (l_e / d_b)^{0.2} * \sqrt{d_b} * \sqrt{f'_c} * C_{al}^{1.5}$
 $l_e: 5.00$

$V_{b1} = 5.28$ kips

$V_{b2} = 9 * C_{al}^{1.5} * \sqrt{f'_c}$
 5.66 kips

$V_b = \min(V_{b1}, V_{b2}) = 5.28$ kips

$\Psi_{ed,v}:$ 1 (only reduced for anchor adjacent to deck discontinuity)

$\Psi_{c,v}:$ 1.4 (1.4 for uncracked deck, 1.2 for cracked reinforced, 1.0 for cracked unreinforced deck)

$\Psi_{h,v}:$ 1.00

$A_{vco} = 4.5 * (c_{al})^2 = 96.25781$ in.²

$A_{vc} = 94.95703$ in.²

$A_{vco} / A_{vc} = 0.986486$

$\Phi V_{cb} = 10.28$ kips

Concrete Pryout Strength: $\Phi V_{cp} = K_{cp} * N_p$

$K_{cp} = 2$

$N_{cp} = \min(N_{cb}, N_a)$

$N_{cb} = A_{Nc} / A_{Nco} * \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} \Psi_{cd} * N_b$

$N_b = k_c * h_{ef}^{1.5} \sqrt{f'_c}$

$k_c:$ 17

$\Psi_{c,N}:$ 1.4

$N_b = 34.00$ kips

$C_{ac}:$ 20

$\Psi_{cp,N}:$ 1

$\Psi_{ed,N}:$ 0.7925

$A_{Nco} = 9 * h_{ef}^2:$ 900 in.²

$A_{Nc}:$ 261 in.²

$A_{Nc} / A_{Nco}:$ 0.29

$N_{cb} = 20.57$

$N_a = A_{Na} / A_{Nao} * \Psi_{ed,Na} \Psi_{c,Na} \Psi_{cp,Na} \Psi_{cd} * N_{ba}$

$N_{ba} = \tau_{cr} * \pi * d_b * h_{ef}$

$N_{ba} = 35.34$ kips

$A_{Nao} = (2 * C_{Na})^2$

$C_{Na} = 10 * d_b * \sqrt{(\tau_{cr} / 1100)}$

$C_{Na} = 8.00$

$A_{Nao} = 255.68$ in.²

$A_{Na} = 176.9403$ in.²

$A_{Na} / A_{Nao} = 0.692033$

$\Psi_{cp,Na}:$ 1 (should be the same as $\Psi_{cp,N}$)

$\Psi_{ed,Na}:$ 0.873545

$N_a = 31.71$

$N_{cp} = 20.57$

$\Phi V_{cp} = 30.85$ kips

Figure A-4. Shear Adhesive Anchorage Calculations, Four-Bolt Spread Anchorage Concept

TENSION ANCHORS (FRONT FACE)

Embedment Depth, h_{ef} :	6 in.
Steel Bar Diameter, d_b :	0.75 in.
Area of Steel, A_s :	0.334 in. ²
Front (Tension) Anchor Spacing, s :	6.5 in.
Front (Tension) Anchor to deck edge, $c_{a,min}$:	6 in.
Bond Strength, τ_{cr} :	1800 psi
Steel Ultimate Strength, f_{uta} :	120 ksi
Concrete Strength, f'_c :	4000 psi
Deck Reinforced? (y/n):	y
Steel DIF, ψ_{sd} :	1.18
Concrete DIF, ψ_{cd} :	1.88
Adhesive/Bond DIF, ψ_{bd} :	1.484

Tension Strengths

Failure Mode	Load (kips)
Steel Fracture:	35.47
Concrete Breakout:	11.04
Bond Failure:	9.12
Hybrid:	20.16

	Tension	Shear
ACI Steel Strength Reduction Factor, ϕ_s :	0.75	0.65
ACI Concrete Strength Reduction Factor, ϕ_c :	0.65	0.75
ACI Adhesive Strength Reduction Factor, ϕ_a :	0.65	NA

TENSION CAPACITY

Steel Fracture: $\phi N_s = A_s f_{uta} \psi_{sd}$
 $\phi N_s = 35.47$ kips

Concrete Breakout: $\phi N_{cb} = A_{Nco} / A_{Nco} * \psi_{ed,N} \psi_{cp,N} \psi_{cd} * N_b$

$$N_b = k_c * h_{ef}^{1.5} \sqrt{f'_c}$$

$k_c = 17$ (24 for cast in place, 17 for post installed)

$\psi_{cp,N} = 1.4$ (1.25 for cast in anchors, 1.4 for post installed, 1.0 if k_c taken from external document without further instructions)

$N_b = 15.80$ kips

$C_{ac} = 12$

$\psi_{cp,N} = 1$

$\psi_{ed,N} = 0.9$

$A_{Nco} = 9 * h_{ef}^2 = 324$ in.²

$A_{Nc} = 147$ in.²

$A_{Nc} / A_{Nco} = 0.453704$

Note - Calculated based on influence area per anchor

$\phi N_{cb} = 11.04$ kips

Adhesive / Bond Failure: $\phi N_a = A_{Na} / A_{Na0} * \psi_{ed,Na} \psi_{cp,Na} \psi_{bd} * N_{ba}$

$N_{ba} = \tau_{cr} \pi d_b h_{ef}$

$N_{ba} = 25.45$ kips

$A_{Na0} = (2 * C_{Na})^2$

$C_{Na} = 10 * d_b * \sqrt{\tau_{cr} / 1100}$

$C_{Na} = 9.59$ in.

$A_{Na0} = 368.18$ in.²

$A_{Na} = 154.1284$ in.²

$A_{Na} / A_{Na0} = 0.41862$

Note - Calculated based on influence area per anchor

$\psi_{cp,Na} = 1$ (should be the same as $\psi_{cp,N}$)

$\psi_{ed,Na} = 0.887617$

$\phi N_a = 9.12$ kips

Figure A-5. Tensile Adhesive Anchorage Calculations, Two-Bolt Centered Anchorage Concept

SHEAR ANCHORS (BACK FACE)

Embedment Depth, h_{ef} :	12 in.
Steel Bar Diameter, d_b :	0.75 in.
Area of Steel, A_s :	0.334 in. ²
Anchor Spacing, s :	6.5 in.
Anchor to Deck Edge Distance, c_{ad} :	6 in.
Steel Ultimate Strength, f_{ut} :	120 ksi
Concrete Strength, f'_c :	4000 psi
Deck Thickness, h_d :	26 in.
Deck Reinforced? (y/n):	y
Bond Strength, τ_{cr} :	1800 psi

Shear Strengths

Failure Mode	Load (kips)
Steel Fracture:	18.44
Concrete Breakout:	11.24
Concrete Pryout:	16.01

Total Anchor Shear for Barrier

LCR:	20 ft
$\Phi V_{barrier}$:	414.96 kips

SHEAR CAPACITY

Steel Fracture: $\Phi V_{sa} = 0.6 * A_s * f_{ut} * \Psi_{ed}$
 $\Phi V_{sa} = 18.44$ kips

Concrete Breakout: $\Phi V_{cb} = A_{vc} / A_{vco} * \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} \Psi_{ed} * V_b$

$$V_{b1} = 7 * (l/d_b)^{0.2} * \sqrt{d_b} * \sqrt{f'_c} * c_{ad}^{1.5}$$

$$l_c = 6.00$$

$$V_{b1} = 8.54 \text{ kips}$$

$$V_{b2} = 9 * c_{ad}^{1.5} * \sqrt{f'_c}$$

$$8.37 \text{ kips}$$

$$V_b = \min(V_{b1}, V_{b2}) = 8.37 \text{ kips}$$

$$\Psi_{ed,v} = 1 \text{ (only reduced for anchor adjacent to deck discontinuity)}$$

$$\Psi_{c,v} = 1.4 \text{ (1.4 for uncracked deck, 1.2 for cracked reinforced, 1.0 for cracked unreinforced deck)}$$

$$\Psi_{h,v} = 1.00$$

$$A_{vco} = 4.5 * (c_{ad})^2 = 162 \text{ in.}^2$$

$$A_{vc} = 110.25 \text{ in.}^2$$

$$A_{vc} / A_{vco} = 0.680556$$

$$\Phi V_{cb} = 11.24 \text{ kips}$$

Concrete Pryout Strength: $\Phi V_{cp} = k_{cp} N_{cp}$

$$k_{cp} = 2$$

$$N_{cp} = \min(N_{cb}, N_a)$$

$$N_{cb} = A_{Nc} / A_{Nco} * \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} \Psi_{ed} * N_b$$

$$N_b = k_c * h_{ef}^{1.5} * \sqrt{f'_c}$$

$$k_c = 17$$

$$\Psi_{c,N} = 1.4$$

$$N_b = 44.69 \text{ kips}$$

$$c_{ac} = 24$$

$$\Psi_{cp,N} = 1$$

$$\Psi_{ed,N} = 0.8$$

$$A_{Nco} = 9 * h_{ef}^2 = 1296 \text{ in.}^2$$

$$A_{Nc} = 147 \text{ in.}^2$$

$$A_{Nc} / A_{Nco} = 0.113426$$

$$N_{cb} = 10.67$$

$$N_{cp} = 10.67$$

$$\Phi V_{cp} = 16.01 \text{ kips}$$

$$N_a = A_{Na} / A_{Nao} * \Psi_{ed,Na} \Psi_{c,Na} \Psi_{cp,Na} \Psi_{ed} * N_{ba}$$

$$N_{ba} = \tau_{cr} \pi d_b h_{ef}$$

$$N_{ba} = 50.89 \text{ kips}$$

$$A_{Nao} = (2 * C_{Na})^2$$

$$C_{Na} = 10 * d_b * \sqrt{(\tau_{cr} / 1100)}$$

$$C_{Na} = 9.59$$

$$A_{Na} = 368.18 \text{ in.}^2$$

$$A_{Na} = 154.1284 \text{ in.}^2$$

$$A_{Na} / A_{Nao} = 0.41862$$

$$\Psi_{cp,Na} = 1 \text{ (should be the same as } \Psi_{cp,N})$$

$$\Psi_{ed,Na} = 0.887617$$

$$N_a = 28.06$$

Figure A-6. Shear Adhesive Anchorage Calculations, Two-Bolt Centered Anchorage Concept

TENSION ANCHORS (FRONT FACE)

Embedment Depth, h_{ef} :	6 in.
Steel Bar Diameter, d_b :	0.75 in.
Area of Steel, A_s :	0.334 in. ²
Front (Tension) Anchor Spacing, s :	6.5 in.
Front (Tension) Anchor to deck edge, $c_{a,min}$:	4.625 in.
Bond Strength, τ_{cr} :	1800 psi
Steel Ultimate Strength, f_{uts} :	120 ksi
Concrete Strength, f'_c :	4000 psi
Deck Reinforced? (Y/N):	Y

Tension Strengths

Failure Mode	Load (kips)
Steel Fracture:	35.47
Concrete Breakout:	10.48
Bond Failure:	8.68
Hybrid:	19.16

Steel DIF, ψ_{sd} :	1.18
Concrete DIF, ψ_{cd} :	1.88
Adhesive/Bond DIF, ψ_{bd} :	1.484

	Tension	Shear
ACI Steel Strength Reduction Factor, ϕ_s :	0.75	0.65
ACI Concrete Strength Reduction Factor, ϕ_c :	0.65	0.75
ACI Adhesive Strength Reduction Factor, ϕ_a :	0.65	NA

TENSION CAPACITY

Steel Fracture: $\phi N_s = A_s N_f \psi_{sd}$
 $\phi N_s = 35.47$ kips

Concrete Breakout: $\phi N_{cb} = A_{Nco} / A_{Nco} * \psi_{ed,N} \psi_{cp,N} \psi_{bd} * N_b$
 $N_b = k_c * h_{ef}^{1.5} \sqrt{f'_c}$
 $k_c = 17$ (24 for cast in place, 17 for post installed)
 $\psi_{cp,N} = 1.4$ (1.25 for cast in anchors, 1.4 for post installed, 1.0 if k_c taken from external document without further instructions)
 $N_b = 15.80$ kips

$c_{ac} = 12$
 $\psi_{cp,N} = 1$
 $\psi_{ed,N} = 0.854167$
 $A_{Nco} = 9 * h_{ef}^2 = 324$ in.²
 $A_{Nc} = 147$ in.²
 $A_{Nc} / A_{Nco} = 0.453704$

Note - Calculated based on influence area per anchor

$\phi N_{cb} = 10.48$ kips

Adhesive / Bond Failure: $\phi N_a = A_{Na} / A_{Na0} * \psi_{ed,Na} \psi_{cp,Na} \psi_{bd} * N_{ba}$
 $N_{ba} = \tau_{cr} \pi d_b h_{ef}$
 $N_{ba} = 25.45$ kips

$A_{Na0} = (2 * c_{Na})^2$
 $c_{Na} = 10 * d_b * \sqrt{(\tau_{cr} / 1100)}$
 $c_{Na} = 9.59$ in.
 $A_{Na0} = 368.18$ in.²
 $A_{Na} = 154.1284$ in.²
 $A_{Na} / A_{Na0} = 0.41862$

Note - Calculated based on influence area per anchor

$\psi_{cp,Na} = 1$ (should be the same as $\psi_{cp,N}$)
 $\psi_{ed,Na} = 0.844621$

$\phi N_a = 8.68$ kips

Figure A-7. Tensile Adhesive Anchorage Calculations, Two-Bolt Offset Anchorage Concept

SHEAR ANCHORS (BACK FACE)

Embedment Depth, h_{ef} :	12 in.
Steel Bar Diameter, d_b :	0.75 in.
Area of Steel, A_s :	0.334 in. ²
Anchor Spacing, s :	6.5 in.
Anchor to Deck Edge Distance, c_{ad} :	7.375 in.
Steel Ultimate Strength, f_{uta} :	120 ksi
Concrete Strength, f'_c :	4000 psi
Deck Thickness, h_d :	26 in.
Deck Reinforced? (y/n):	y
Bond Strength, τ_{cr} :	1800 psi

Shear Strengths

Failure Mode	Load (kips)
Steel Fracture:	18.44
Concrete Breakout:	14.56
Concrete Pryout:	16.47

Total Anchor Shear for Barrier

LCR:	20 ft
$\Phi V_{barrier}$:	537.52 kips

SHEAR CAPACITY

Steel Fracture: $\Phi V_{sa} = 0.6 \cdot A_s \cdot f_{uta} \cdot \Psi_{sa}$
 $\Phi V_{sa} = 18.44$ kips

Concrete Breakout: $\Phi V_{cb} = A_{vc} / A_{vco} \cdot \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} \Psi_{ed} \cdot V_b$

$$V_{b1} = 7 \cdot \left(\frac{l_d}{d_b} \right)^{0.2} \cdot \sqrt{d_b} \cdot \sqrt{f'_c} \cdot C_{ad}^{1.5}$$

$$l_d = 6.00$$

$$V_{b1} = 11.64$$
 kips

$$V_{b2} = 9 \cdot C_{ad}^{1.5} \cdot \sqrt{f'_c}$$

$$11.40$$
 kips

$$V_b = \min(V_{b1}, V_{b2}) = 11.40$$
 kips

$$\Psi_{ed,v} = 1 \text{ (only reduced for anchor adjacent to deck discontinuity)}$$

$$\Psi_{c,v} = 1.4 \text{ (1.4 for uncracked deck, 1.2 for cracked reinforced, 1.0 for cracked unreinforced deck)}$$

$$\Psi_{h,v} = 1.00$$

$$A_{vco} = 4.5 \cdot (c_{ad})^2 = 244.7578$$
 in.²

$$A_{vc} = 158.332$$
 in.²

$$A_{vco} / A_{vc} = 0.646893$$

$$\Phi V_{cb} = 14.56$$
 kips

Concrete Pryout Strength: $\Phi V_{cp} = k_{cp} \cdot N_p$

$$k_{cp} = 2$$

$$N_{cp} = \min(N_{cb}, N_a)$$

$$N_{cb} = A_{Nc} / A_{Nco} \cdot \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} \Psi_{ed} \cdot N_b$$

$$N_b = k_c \cdot h_{ef}^{1.5} \cdot \sqrt{f'_c}$$

$$k_c = 17$$

$$\Psi_{c,N} = 1.4$$

$$N_b = 44.69$$
 kips

$$c_{ac} = 24$$

$$\Psi_{cp,N} = 1$$

$$\Psi_{ed,N} = 0.822917$$

$$A_{Nco} = 9 \cdot h_{ef}^2 = 1296$$
 in.²

$$A_{Nc} = 147$$
 in.²

$$A_{Nc} / A_{Nco} = 0.113426$$

$$N_{cb} = 10.98$$

$$N_{cp} = 10.98$$

$$\Phi V_{cp} = 16.47$$
 kips

$$N_a = A_{Na} / A_{Nao} \cdot \Psi_{ed,Na} \Psi_{c,Na} \Psi_{ba} \cdot N_b$$

$$N_{ba} = \tau_{cr} \cdot \pi \cdot d_b \cdot h_{ef}$$

$$N_{ba} = 50.89$$
 kips

$$A_{Nao} = (2 \cdot C_{Na})^2$$

$$C_{Na} = 10 \cdot d_b \cdot \sqrt{\tau_{cr} / 1100}$$

$$C_{Na} = 9.59$$

$$A_{Nao} = 368.18$$
 in.²

$$A_{Na} = 154.1284$$
 in.²

$$A_{Na} / A_{Nao} = 0.41862$$

$$\Psi_{cp,Na} = 1 \text{ (should be the same as } \Psi_{cp,N})$$

$$\Psi_{ed,Na} = 0.930612$$

$$N_a = 29.42$$

Figure A-8. Shear Adhesive Anchorage Calculations, Two-Bolt Offset Anchorage Concept

Appendix B. Material Specifications

The bill of materials and material specifications are all included in this appendix. This includes concrete cylinder test reports, chemical composition of concrete reports, and chemical composition of rebar reports.

Cast-in-Place (IBP-1)			
Item No.	Description	Material Spec	Reference
a1	Baseplate (3/4")	ASTM A36	R# 14-0486 various heat numbers
a2	5/8" [16] Dia. UNC, 12" [305] Long Threaded Rod	ASTM A193 Grade B7	R# 14-0481 Grainger COC "4FHF5"
a3	5/8" [16] Dia. Hex Nut	ASTM A563 DH	Item# DHHN063CG L# 124738C H# 1W259 Grainger# "1AY80"
a4	5/8" [16] Dia. SAE Flat Washer	ASTM F436	R# 14-0481 Grainger COC "6PU24"
a5	2"x7 3/8"x3/8" [51x187x10] Washer Plate	ASTM A36	R# 14-0486 H# 63134357/02
e5	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	R# 14-0486 H# NC4272
Four-Anchor Spread (IBP-2)			
Item No.	Description	Material Spec	Reference
b1	Baseplate (7/8")	ASTM A36	R# 14-0486 H# A3Q1399
b2	5/8" [16] Dia. UNC, 12" [305] Long Threaded Rod	ASTM A193 Grade B7	R# 14-0481 Grainger COC "4FHF5"
b3	5/8" [16] Dia. Hex Nut	ASTM A563 DH	Item# DHHN063CG L# 124738C H# 1W259 Grainger# "1AY80"
b4	5/8" [16] Dia. SAE Flat Washer	ASTM F436	R# 14-0481 Grainger COC "6PU24"
e5	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	R# 14-0486 H# NC4272
e6	Epoxy	Hilti RE-500 SD Epoxy	Same as SFH, See Tech Data Sheet

Figure B-1. Bill of Materials, Test Nos. IBP-1 through IBP-2

Two-Anchor Offset (IBP-3)			
Item No.	Description	Material Spec	Reference
c1	Baseplate (1")	ASTM A36	R# 14-0486 H# B4M5434
c2	3/4" [19] Dia. UNC, 14 1/2" [368] Long Threaded Rod	ASTM A193 Grade B7	R# 14-0481 Grainger COC "4FHF7"
c3	3/4" [19] Dia. Hex Nut	ASTM A563 DH	R# 12-0364(HTCT) L# 133507 Item# CHHNO75CP H# 330900831
c4	3/4" [19] Dia. SAE Flat Washer	ASTM F436	R# 14-0481 Grainger COC "6PU26"
e5	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	R# 14-0486 H# NC4272
e6	Epoxy	Hilti RE-500 SD Epoxy	Same as SFH, See Tech Data Sheet
US-20 River Bridge (IBP-4)			
Item No.	Description	Material Spec	Reference
d1	Baseplate (13/16" is shaved down from 3/4" plate)	ASTM A36	R# 14-0486 various heat numbers
d2	5/8" [16] Dia. UNC, 12" [305] Long Threaded Rod	ASTM A193 Grade B7	R# 14-0481 Grainger COC "4FHF5"
d3	5/8" [16] Dia. Hex Nut	ASTM A563 DH	Item# DHHN063CG L# 124738C H# 1W259 Grainger# "1AY80"
d4	5/8" [16] Dia. SAE Flat Washer	ASTM F436	R# 14-0481 Grainger COC "6PU24"
e5	HSS 4"x4"x3/16" [102x102x5] Steel Tube	ASTM A500 Grade B	R# 14-0486 H# NC4272
e6	Epoxy	Fastenal Pro-Poxy 300	L# 1305 EXP. 11/15 see (Tech Data Sheet)
Bill of Bars			
Item No.	Description	Material Spec	Reference
e1	1/2" [13] Dia., 79 1/2" [2019] Long Bent Rebar	ASTM A615 Grade 60	H# 112090 Red Paint R# 14-0497
e2	1/2" [13] Dia., 78 1/2" [1994] Long Bent Rebar	ASTM A615 Grade 60	H# 112090 Red Paint R# 14-0497
e3	1/2" [13] Dia., 73 1/2" [1867] Long Bent Rebar	ASTM A615 Grade 60	H# 112090 Red Paint R# 14-0497
e4	1/2" [13] Dia., 236" [5994] Long Rebar	ASTM A615 Grade 60	H# 57134866 Red Paint R# 14-0497

Figure B-2. Bill of Materials, Test Nos. IBP-3 through IBP-4

Concrete industries 6300 Cornhusker Highway P.O. Box 29529 Lincoln, NE 68529- Phone: (402)434-1800 FAX: (402)434-1899				JOB NUMBER 8000MISC.		RELEASE NUMBER DAVID-617		REQ. DELIVERY DATE		PAGE 1 of 1								
				JOB NAME JOB COMPLETE						CC 4HQ								
				CUSTOMER MIDWEST ROADSIDE SAFETY FACILITY						BY MEB								
MATERIAL TYPE Rebar, Grade 60, Black				REFERENCE		DRAWING ID		DESCRIPTION IOWA BRIDGE RAIL										
Item	Qty	Size	Length	Mark	Shape	Lbs	A	B	C	D	E	F/R	G	H	J	K	O	BC
1	14	4	7-05	E3	T1	69	0-042	2-02	1-02	2-02	1-02		0-042					107
2	15	4	7-00	E1	17	70		3-02	0-08	3-02								805
3	8	4	6-08	E2	S11	36		6-08						3-022				0-07
4	12	4	19-08	E4		158												0
49						333												
Total Weight: 333 Lbs																		
Longest Length: 19-08																		

REBAR PLANT

WEIGHT SUMMARY												
TOTAL				STRAIGHT			LIGHT BENDING			HEAVY BENDING		
SIZE	ITEMS	PIECES	LBS	ITEMS	PIECES	LBS	ITEMS	PIECES	LBS	ITEMS	PIECES	LBS
Rebar, Grade 60, Black												
4	4	49	333	1	12	158	2	22	105	1	15	70
	4	49	333	1	12	158	2	22	105	1	15	70
Total Weight: 333 Lbs												
Longest Length: 19-08												

Heat # Please

Coil - Evraz 112090

4 - Garden 57134866

APR 30 2014

FOR CONSTRUCTION

Iowa Bridge Rail Rebar
R# 14-0497 May 2014 SMT

Iowa Bridge Rail Rebar & Concrete portfolio
September 9, 2014

v13.01.4015 (T) (LIN)
©2014 ASA UNAUTHORIZED REPRODUCTION PROHIBITED
Wednesday, April 30, 2014 12:19 PM

Figure B-3. Rebar Material Specification, Test Nos. IBP-1 through IBP-4



ROCKY MOUNTAIN STEEL
A DIVISION OF EVRAZ INC. NA

P.O. Box 316
Pueblo, CO 81002 USA

MATERIAL TEST REPORT

Date Printed: 21-APR-14

Date Shipped: 21-APR-14 Product: DEF #4 (1/2") Specification: ASTM A-706/A-615
FWIP: 52815362 Customer: CONCRETE INDUSTRIES INC Cust. PO: 106081

CHEMICAL ANALYSIS												
Heat Number	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Al	V	Ti

(Heat chemistry entered 03/24/14)

0.016 0.0060

Carbon Equivalent = 0.497

MECHANICAL PROPERTIES						
Heat Number	Sample No.	Yield (Psi)	Ultimate (Psi)	Elongation (%)	Reduction (%)	Bend

112090	01	0.2% offset	69715	16.9	ok	0.671
		(MPa)	480.7			
112090	02	0.0035 EUL	64851	15.2	ok	0.670
		(MPa)	447.1			


All melting and manufacturing processes of the material subject to this test certificate occurred in the United States of America.
ERMIS also certifies this material to be free from Mercury contamination.

This material has been produced and tested in accordance with the requirements of the applicable specifications. We hereby certify that the above test results represent those contained in the records of the Company.

Mark F. Espinosa

Quality Assurance Department

Figure B-4. Rebar Material Test Report, Test Nos. IBP-1 through IBP-4



65-MI-KNOXVILLE
1919 TENNESSEE AVENUE N. W.
KNOXVILLE, TN 37921
USA

CERTIFIED MATERIAL TEST REPORT

Page 1/1

CUSTOMER SHIP TO NEBCO INC STEEL DIVISION HAVELOCK, NE 68529 USA		CUSTOMER BILL TO CONCRETE INDUSTRIES INC LINCOLN, NE 68529-0529 USA		GRADE 60 (420) TMX	SHAPE / SIZE Rebar / #4 (13MM)	WEIGHT 111.576 LB	HEAT / BATCH 57134866/02
SALES ORDER 507829/000010		CUSTOMER MATERIAL N°		SPECIFICATION / DATE OF REVISION 1-ASTM A615/A615M-09			
CUSTOMER PURCHASE ORDER NUMBER 101828		BILL OF LADING 1326-0000008527		DATE 08/22/2013			

CHEMICAL COMPOSITION C % 0.25 Mn % 0.62 P % 0.015 S % 0.005	Si % 0.20 Cu % 0.30 Ni % 0.16 Cr % 0.18 Mo % 0.025 V % 0.004	UTS MPa 580 UTS MPa 98030 UTS MPa 676	CE _{eq} A706 % 0.39
--	---	---	------------------------------

MECHANICAL PROPERTIES YS MPa 580 UTS MPa 98030 UTS MPa 676	G _L inch 8.000 G _L mm 203.0
--	--

MECHANICAL PROPERTIES Yield % 11.90 Bend Test OK

GEOMETRIC CHARACTERISTICS %Light 4.79 Def Hgt inch 0.031 Def Gap inch 0.108 Def Space inch 0.332

COMMENTS / NOTES

The above figures are certified chemical and physical test records as contained in the permanent records of company. This material, including the billets, was melted and manufactured in the USA. CMTR complies with EN 10204 3.1.

Shackay
 BHASKAR YALAMANCHILI
 QUALITY DIRECTOR

Lisa Churnetski
 LISA CHURNETSKI
 QUALITY ASSURANCE MGR.

Figure B-5. Rebar Material Test Report, Test Nos. IBP-1through IBP-4

 benesch engineers • scientists • planners	LINCOLN OFFICE 825 "J" Street Lincoln, NE 68508 Phone: (402) 479-2200 Fax: (402) 479-2276
Client Name: Midwest Roadside Safety Facility	COMPRESSION TEST OF CYLINDRICAL CONCRETE SPECIMENS - 6x12
Project Name: Iowa Bridge Rail Footing	ASTM Designation: C 39
Placement Location: Iowa Bridge Rail Head, 8/5/14	Date 04-Sep-14
Mix Designation: 3600	Required Strength: 3600

Laboratory Test Data									
Laboratory Identification	Field Identification	Date Cast	Date Received	Date Tested	Days Cured in Field	Days Cured in Laboratory	Age of Test, Days	Length of Specimen, in.	Diameter of Specimen, in.
IBR- 8	E	8/5/2014	9/4/2014	9/4/2014	30	0	30	12	6.00
IBR- 9	F	8/5/2014	9/4/2014	9/4/2014	30	0	30	12	6.00
1 cc: Mr. Shaun Tighe Midwest Roadside Safety Facility									Maximum Load, lbf
									28.27
									115,800
									4,100
									psi
									Required Strength, psi
									3,600
									Type of Fracture
									5
									ASTM Practice for Capping Specimen
									C 1231
									C 1231







Remarks: Concrete test specimens along with documentation and test data were submitted by Midwest Roadside Safety Facility. Test results presented relate only to the concrete specimens as received from Midwest Roadside Safety Facility. This report shall not be reproduced except in full, without the written approval of Alfred Benesch & Company. Report Number 2147366596 Page 1	Sketches of Types of Fractures  Type 1 Reassess well-formed cones on both ends, less than 1 in. (25 mm) of cracking through caps  Type 2 Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end  Type 3 Columnar vertical cracking through both ends, no well-formed cones  Type 4 Diagonal fracture with no cracking through both ends; top with hammer to distinguish from Type 1  Type 5 Side fractures at top or bottom (occur commonly with unbonded caps)  Type 6 Similar to Type 5 but end of cylinder is pointed	ALFRED BENESCH & COMPANY CONSTRUCTION MATERIALS LABORATORY By <u>Brant Wells</u> Brant Wells, Coordinator
---	---	--

Figure B-6. Concrete Material Test Report for Footing Pour

 <p>benesch engineers · scientists · planners</p>	<p>LINCOLN OFFICE 825 "J" Street Lincoln, NE 68508 Phone: (402) 479-2200 Fax: (402) 479-2276</p>	<p>COMPRESSION TEST OF CYLINDRICAL CONCRETE SPECIMENS - 6x12</p> <p>ASTM Designation: C 39</p>
<p>Client Name: Midwest Roadside Safety Facility</p> <p>Project Name: Iowa Bridge Rail Footing</p> <p>Placement Location: Iowa Bridge Rail Head.</p> <p>Mix Designation: 3600</p>		
		<p>Date 27-Aug-14</p>
		<p>Required Strength: 3600</p>

Laboratory Identification	Field Identification	Date Cast	Date Received	Date Tested	Days Cured in Field	Days Cured in Laboratory	Age of Test, Days	Length of Specimen, in.	Diameter of Specimen, in.	Cross-Sectional Area, sq. in.	Maximum Load, lbf	Compressive Strength, psi	Required Strength, psi	Type of Fracture	ASTM Practice for Capping Specimen
IBR- 6	C	8/5/2014	8/26/2014	8/27/2014	21	1	22	12	6.01	28.37	108,740	3,830	3,600	6	C 1231
IBR- 7	D	8/5/2014	8/26/2014	8/27/2014	21	1	22	12	6.02	28.46	112,200	3,940	3,600	6	C 1231

1 cc: Mr. Shaun Tighe
Midwest Roadside Safety Facility

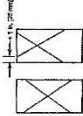





<p>Remarks:</p> <p>Concrete test specimens along with documentation and test data were submitted by Midwest Roadside Safety Facility.</p> <p>Test results presented relate only to the concrete specimens as received from Midwest Roadside Safety Facility.</p> <p>This report shall not be reproduced except in full, without the written approval of Alfred Benesch & Company.</p> <p>Report Number 2147366574</p> <p>Page 1</p>	<p>Sketches of Types of Fractures</p> <div style="display: flex; justify-content: space-around;"> <div style="text-align: center;">  <p>Type 1</p> </div> <div style="text-align: center;">  <p>Type 2</p> </div> <div style="text-align: center;">  <p>Type 3</p> </div> <div style="text-align: center;">  <p>Type 4</p> </div> <div style="text-align: center;">  <p>Type 5</p> </div> <div style="text-align: center;">  <p>Type 6</p> </div> </div> <p>ALFRED BENESCH & COMPANY CONSTRUCTION MATERIALS LABORATORY</p> <p>By _____ Brant Wells, Coordinator</p>
--	---

Figure B-8. Concrete Material Test Report for Parapet Pour

**CAUTION
FRESH CONCRETE**

Body and or eye contact with fresh (moist) concrete should be avoided because it contains alkali and is caustic.

**Ready Mixed
Concrete Company**

6200 Cornhusker Highway, P.O. Box 29288
Lincoln, Nebraska 68529
Telephone 402-434-1844

PLANT 01	MIX CODE 235131PF	YARDS 3.50	TRUCK 0106	DRIVER	DESTINATION NTE	CLASS	TIME 09:30AM	DATE 06/23/14	TICKET 1178830
CUSTOMER 00003	JOB	CUSTOMER NAME CIA---MWRS			TAX CODE	PARTIAL	NIGHT R	LOADS 1	
DELIVERY ADDRESS 4800 NW 35TH				SPECIAL INSTRUCTIONS NW 36TH & W CUMINS NORTH OF OLD N/ GOODYEAR HANGER			P.O. NUMBER 402-4506250 / J		

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UNIT PRICE	AMOUNT
3.50	3.50	3.50	235131PF	47B(1PF) MINIMUM HAUL	4.00	98.25 343.88 35.00

WATER ADDED ON JOB 0 GAL

AT CUSTOMER'S REQUEST

RECEIVED BY

SUBTOTAL **378.88**

TAX

TOTAL **378.88**

TRUCK **USER LOGIN** **DISP** **TICKET NUM** **TICKET NUM** **TICKET ID** **TIME** **DATE**

0106 USER 1178830 198703 49780 09:30 06/23/2014

LOAD SIZE **MIX CODE**

3.50 yd 235131PF

SOURCE **DESIGN QTY** **REQUIRED** **BATCHED** **VAR** **% VAR** **%MOISTURE** **ACTUAL WAT**

G47B 47B GRAVEL 2070 lb 7375 lb 7360 -15 -20% 1.80 M 15.59 gl

L47B 47B ROCK 901 lb 3179 lb 3160 -19 -60% 0.80 M 3.01 gl

CEM1PF CEM1PF 564 lb 1974 lb 1965 -9 -46%

LRWR POZZ 322N 17.00 oz 59.50 oz 59.00 -50 -84%

AIR MB-AE 90 A 5.50 oz 19.25 oz 19.00 -25 -1.30%

WATER WATER 29.0 GL 86.3 GL 86.3 0.0 0.00%

WATER2 WATER2 0.0 gl # 0.0 gl 0.0 0.0 0.00%

NON-SIMULATED NUM BATCHES: 1

LOAD TOTAL: 13210 lb DESIGN W/C: 0.429 WATER/CEMENT: 0.445A DESIGN WATER: 101.5 gl ACTUAL WATER: 104.9 gl TO ADD: 0.0 gl

SLUMP: 4.00 " # WATER IN TRUCK: 0.0 gl ADJUST WATER: 0.0 gl /load TRIM WATER: 0.0 gl /yd

Iowa Bridge Rail Concrete Footing

ORIGINAL

Figure B-9. Concrete Material Specification, Footing Pour

**CAUTION
FRESH CONCRETE**

Body and or eye contact with fresh (moist) concrete should be avoided because it contains alkali and is caustic.

**Ready Mixed
Concrete Company**
6200 Cornhusker Highway, P.O. Box 29288
Lincoln, Nebraska 68529
Telephone 402-434-1844

PLANT 04	MIX CODE 235131PF	YARDS 2.00	TRUCK 0203	DRIVER 010	DESTINATION	CLASS	TIME 09:55AM	DATE 08/05/14	TICKET 4162072
CUSTOMER 00003	JOB	CUSTOMER NAME CIA---MIDWEST ROADSIDE SAFETY			TAX CODE	PARTIAL	NIGHT R	LOADS 1	
DELIVERY ADDRESS 4800 NW 35TH STREET			SPECIAL INSTRUCTIONS NORTHEAST OF THE NORTH GOODYEAR HANGER				P.O. NUMBER 7709121 KEN		

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UNIT PRICE	AMOUNT
2.00	2.00	2.00	235131PF	47B (1PF) MINIMUM HAUL	98.25	196.50
						50.00

WATER ADDED ON JOB
AT CUSTOMER'S REQUEST

RECEIVED BY

SUBTOTAL 246.50

TAX

TOTAL 246.50

TRUCK	USER LOGIN	DISP	TICKET NUM	TICKET NUM	TICKET ID	TIME	DATE	
0203	USER		4162072	4162324	197331	09:55	08/05/2014	
LOAD SIZE	MIX CODE	SEQ	LOAD ID					
2.00 yd	235131PF	W	199449					
MATERIAL	SOURCE	DESIGN QTY	REQUIRED	BATCHED	VAR	% VAR	%MOISTURE	ACTUAL WAT
647B	47B GRAVEL	2070.0 lb	4255.9 lb	4280.0	24.1	0.57%	2.80 M	13.97 gl
L47B	47B ROCK	901.0 lb	1811.0 lb	1800.0	-11.0	-.61%	0.50 M	1.07 gl
CEM1PF	INTERGRDUN	564.0 lb	1128.0 lb	1125.0	-3.0	-.27%		
LRWR	POZZ 322N	17.0 oz	34.0 oz	34.0	0.0	0.00%		
AIR	MB-AE 90 A	4.8 oz	9.6 oz	10.0	0.4	4.17%		
WATER	WATER	27.0 gl	41.0 gl	42.4	1.4	3.41%		42.42 gl
WATER2	RECYCLE WA	0.0 gl	0.0 gl	0.0	0.0	0.00%		
NON-SIMULATED NUM BATCHES: 1								
LOAD TOTAL: 7562 lb DESIGN W/C: 0.400 WATER/CEMENT: 0.426A DESIGN WATER: 54.0 gl ACTUAL WATER: 57.5 gl								
SLUMP: 4.00 # WATER IN TRUCK: 0.0 gl								

IOWA
BRIDGE
(HEA)
15-0084

ORIGINAL

Figure B-10. Concrete Material Specification, Parapet Pour



Central Plains Cement Company

Type IP (25) Mill Test Report

Month of Issue: Jun-14

Plant: Omaha Terminal
Product: Type IP (25)
Manufactured: May-14

The current version of ASTM C 595 and AASHTO M 240 Standard Requirements

CHEMICAL ANALYSIS			PHYSICAL ANALYSIS		
Item	Spec limit	Test Result	Item	Spec limit	Test Result
Rapid Method, X-Ray (C 114)			Air content of mortar (%) (C 185)	12 max	9
SiO ₂ (%)	---	27.3	Blaine Fineness (m ² /kg) (C 204)	---	340
Al ₂ O ₃ (%)	---	9.0	Fineness, Residue retained on a 45 um sieve (%) sieve (%)	---	8.0
Fe ₂ O ₃ (%)	---	8.3	Autoclave expansion (%) (C 151)	0.80 max -0.20 min	-0.01
CaO (%)	---	48.2	Compressive strength (PSI) (C 109)		
MgO (%)	6.0 max	1.1	1 days		1710
Sulphate as SO ₃ (%)	4.0 max	2.4	28 days (Reflects previous month's data)	3620 min	5900
Loss on ignition (%)	5.0 max	1.9	Time of setting (minutes)		
Total Alkalis	---	0.96	Vicat Initial (C 191)	45 - 420	125
			Specific Gravity (C188)	---	2.98

We certify that the above described cement meets the chemical and physical requirements of the current version of ASTM C 595 and AASHTO M 240.

Sugar Creek Plant
2200 N Courtney Rd.
Sugar Creek, MO 64050
816-257-3604

Certified By:


Adam Doppenberg - Quality Coordinator

6/13/2014

Figure B-11. Concrete Material Specification, Footing Pour

ASH GROVE CEMENT COMPANY

Quantity (tons):
Trailer/Car:
Shipped:



16215 Highway 50
Louisville, NE 68037
Phone: 402-234-2415
FAX: 402-234-4825

Type IP (25)
Duracem® F

Production Period: June 1 thru 30, 2014

Date: 7/9/2014

The following information is based on average test data during the production period. The data is typical of cement shipped from the Louisville, Nebraska plant.
Individual shipments may vary.

STANDARD REQUIREMENTS ASTM C595-13

CHEMICAL				PHYSICAL			
A.S.T.M. Test				A.S.T.M. Test			
Item	Method	Spec. Limit	Test Result	Item	Method	Spec. Limit	Test Result
SiO ₂ (%)	C114	A	30.6	Air content of mortar (volume %)	C185	12 max	5
Al ₂ O ₃ (%)	C114	A	8.7	Fineness			
Fe ₂ O ₃ (%)	C114	A	3.5	Air permeability (m ² /kg)	C204	A	496
CaO (%)	C114	A	48.0	325 mesh (%)	C151	A	94.6
MgO (%)	C114	6.0 max	2.8	Autoclave expansion (%)	C151	0.80 max	0.00
SO ₃ (%)	C114	4.0 max	3.2	Compressive strength (psi)			
Loss on ignition (%)	C114	5.0 max	1.4	1 Day	C109	A	2550
Na ₂ O (%)	C114	A	0.26	3 Days	C109	1890 min	3700
K ₂ O (%)	C114	A	0.71	7 Days	C109	2900 min	4470
Equivalent alkalies (%)	C114	A	0.73	28 Days	C109	3620 min	C
				Time of setting (minutes)			
				(Vicat)			
				Initial: Not less than	C191	45	85
				Not more than		420	
				Sulfate resistance ¹	C1012	0.10	0.04
				Specific Gravity	C188		2.95
				Heat of hydration (kJ/kg)	C186		
				7 Days		B	73

¹ Optional requirement

A = Not applicable.

B = Test result represents most recent value and is provided for information only.

C = Test results for this period not available.

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirement of the ASTM C595/C595M-13 or (other) _____ specification.

Signature:

Douglas R. Jaquier
Title: Chief Chemist

Figure B-12. Concrete Material Specification, Parapet Pour

GENERAL TESTING LABORATORIES

TELEPHONE (402)434-1891

August 4, 2014

P.O. BOX 28529

FAX (402)434-2161

LINCOLN, NEBRASKA 68529

Mix Design Number 235131PF

NDOR 3500 - 1PF

Mix proportions (per cubic yard)

PROJECT INFORMATION			
PROJECT NAME	Iowa DOT Test Mix		
CITY, ST	Lincoln, NE		
CONTRACTOR INFORMATION			
CONTRACTOR NAME	University of Nebraska - Lincoln		
CONTACT PERSON			
CONTACT NUMBER	CONTACT EMAIL		
CONCRETE INFORMATION			
Supplier Mix Design Number	235131PF	NDOR 3500 - 1PF Mix Type	
Design Strength (f' c)	3500 - 1PF	psi	
Design Water / Cementitious Ratio	0.43	=	29 Gallons
Specification Water / Cementitious Allowed	0.48	=	32.5 Gallons
Total Air Content	7.25	+ 1.5%	
Mix Developed From		Density (Estimated)	
Trial Mix Test Data		Fresh	140.2 pcf
Field Experience		Cured	139.5 pcf
Slump (Spread if SCC)			
3" (+ 1.5") WITHOUT WR Admixture			
(+ 1.5") WITH Type A, B, D Water Reducer Admixture			
(+ 2.5") WITH TYPE F, G Admixture			
Specification Maximum			
ADMIXTURE INFORMATION			
ASTM Admix Type	ASTM Designation	Product (Manufacturer/Brand)	Dosage (ounces) See RATE
Air Entraining	ASTM C-260	Master Builders / MasterAir AE 90	No Limit 8.5 1.5
Air Entraining	ASTM C-260	Master Builders / MasterAir AE 400	
Water Reducing *	A, B, D ASTM C-494	Master Builders / MasterPozzolith 322	3 - 5 17 3.0
Set Retarding *	B, D ASTM C-494	Master Builders / MasterSet R 300	
Set Retarding * (Note 1)	B, D ASTM C-494	Master Builders / MasterSet DELVO	
Mid Range Water Reducing (Note 2)	A ASTM C-494	Master Builders / MasterPolyheed 900	
Full Range Water Reducing (Note 2)	F ASTM C-494	Master Builders / MasterGlenium 3030 NS	
Full Range Water Reducing (Note 2)	F ASTM C-494	Master Builders / MasterGlenium 7700	
Accelerating Chloride Base *	C, E ASTM C-494	Master Builders / MasterSet AC 122	
Accelerating Non Chlor Base	C ASTM C-494	Master Builders / MasterSet AC 534	
Corrosion Inhibiting	C ASTM C-494	Master Life CI 30	
Rheological	S ASTM C-494-08a	Master Builders / MasterMatrix VMA 362	
Rheological **	S ASTM C-494-10	ActiveMinerals/Acti-Gel 208	
Consistency Control	S ASTM C-494	Master Builders / MasterSure Z-60	
Water Repellence	S ASTM C-494	Master Builders / MasterPel 240	
Durability Enhancing	C ASTM C-494	Master Builders / MasterLife CI 30	
Drying Enhancing	S ASTM C-494	Specialty Products Group - Vapor Lock 20/20	
Fibermesh	ASTM C-116-03	Proper Concrete Systems - Fibermesh 300	
Fibermesh	ASTM C-116-03	Proper Concrete Systems - Novomesh 500	
Type A - Water Reducing	Type D - Water Reducing & Retarding	Type G - Water Reducing High Range & Retarding	
Type B - Retarding	Type E - Water Reducing & Accelerating	Type S - Specific Performance (Rheology)	
Type C - Accelerating	Type F - Water Reducing High Range		
RATE: Admixture quantities will vary due to external forces including ambient temperatures, humidity, wind, etc.			
* Meets multiple ASTM specifications according to dosage volume.			
** Acti-Gel addition calculated on total weight of all dry materials included in the mix design.			
*** As required at contractor request to adjust mix physical properties.			
Note 1: May be required for set control and slump retention with environmental conditions.			
Note 2: Jobsite conditions and contractor requests may require use of this admixture.			
Bayer Color System			
Color	No Color Added	Color Originator	Color 1100 Color 9200 Color 1300 Color 3000
Product Code			Lt. Red Yellow Dark Red Black
Total Load Rate			

	Identification (Type,size,source,etc.)	Weight (lb.)	Density SSD	Volume (cubic feet)	% Aggregate Absorption				
Cement	III								
Cement	III								
Cement	Max Cem								
Cement	1PF	Ash Grove, Central Plains	564	2.95	3.06				
Cement	White								
Fly Ash	Class C								
Slag	Size 120								
Komponent									
Interplast N									
Silica Fume									
Coarse Agg. No. 1	47B, Size 57 Limestone	901	2.66	5.43	1.2%				
Coarse Agg. No. 2									
Coarse Agg. No. 3									
Fine Agg. No. 1	47B Sand/Gravel	2071	2.62	12.67	0.6%				
Fine Agg. No. 2									
Fine Agg. No. 3									
		Gal	Lbs						
Water	Lincoln Water System	29	242	1	3.88				
Air Content	Master Builders	7.25%	0	1.96					
Other									
	TOTALS	3778	Lbs.	27.00	Cubic Feet				
Coarse & Fine Aggregate Gradation Information									
	% Passing Each Sieve (All Sieves Sizes must be entered)					Combined % Retained			
Sieve	Coarse # 1 Limestone	Coarse # 2 Limestone	Coarse # 3	Fine # 1 Sand/Gravel	Fine # 2 Sand	Fine # 3 Sand	Combined	Cumulative	Individual
Size	3. Size 57 Limestone	Size 47 Limestone		17B Sand/Gravel	C.33 Sand	Masonry Sand	% Passing		
1-1/2"	100			100			100%	0%	0%
1"	94.6			100			98%	2%	2%
3/4"	74.9			100			92%	8%	6%
1/2"	43.0			100			83%	17%	10%
3/8"	26.1			98			76%	24%	6%
No. 4	6.7			88			64%	36%	13%
No. 8	4.6			68			49%	51%	15%
No. 16	4.0			47			34%	66%	15%
No. 30	3.6			26			19%	81%	15%
No. 50	3.3			7			6%	94%	14%
No. 100	3.2			0.5			1%	99%	4%
No. 200	3.0			0.1			1.0%	99.0%	0.3%
% of Vol	30%	0%		70%	0%	0%			
Fineness Modulus:				3.34					
Aggregate Ratios									
Combined % cumulative retained 3/8" sieve									
Coarseness Factor =		Combined % cumulative retained # 8 sieve					=	46	
Workability Factor =		Combined % passing # 8 sieve					=	49	
Adj-Workability Factor =		WF + [(Cementitious Material - 564) / 37.6]					=	49	
Allowable Adj-WF =		[(11.25 - .15 CF) + 36] + 2.5					= Low	38	High 43
Mortar Fraction								65.7%	

Figure B-13. Concrete Gradation Specification, Test Nos. IBP-1 through IBP-4

GENERAL TESTING LABORATORIES

Typical Aggregate Quality Analysis (Supplied by Nebraska Department of Roads)

Western Sand & Gravel		Kereford Limestone	
Typical Sand/Gravel Qualities all Sizes		Typical Limestone Qualities all Sizes	
Bulk Specific Gravity (SSD):	2.62	Bulk Specific Gravity (SSD):	2.66
24 Hour Absorption:	0.6%	24 Hour Absorption:	1.1%
LA Abrasion Loss:	26%	LA Abrasion Loss:	30%
Sulfate Soundness Loss:	2.0%	Sulfate Soundness Loss:	3.1%
Deleterious Materials:	< 0.5%	Deleterious Materials:	< 0.5%
Soluble Chloride Ion Content:	< 0.001%	Soluble Chloride Ion Content:	< 0.001%
Organic Impurities:	None		

Figure B-14. Aggregate Quality Analysis, Test Nos. IBP-1 through IBP-4

Appendix C. Bogie Test Results

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

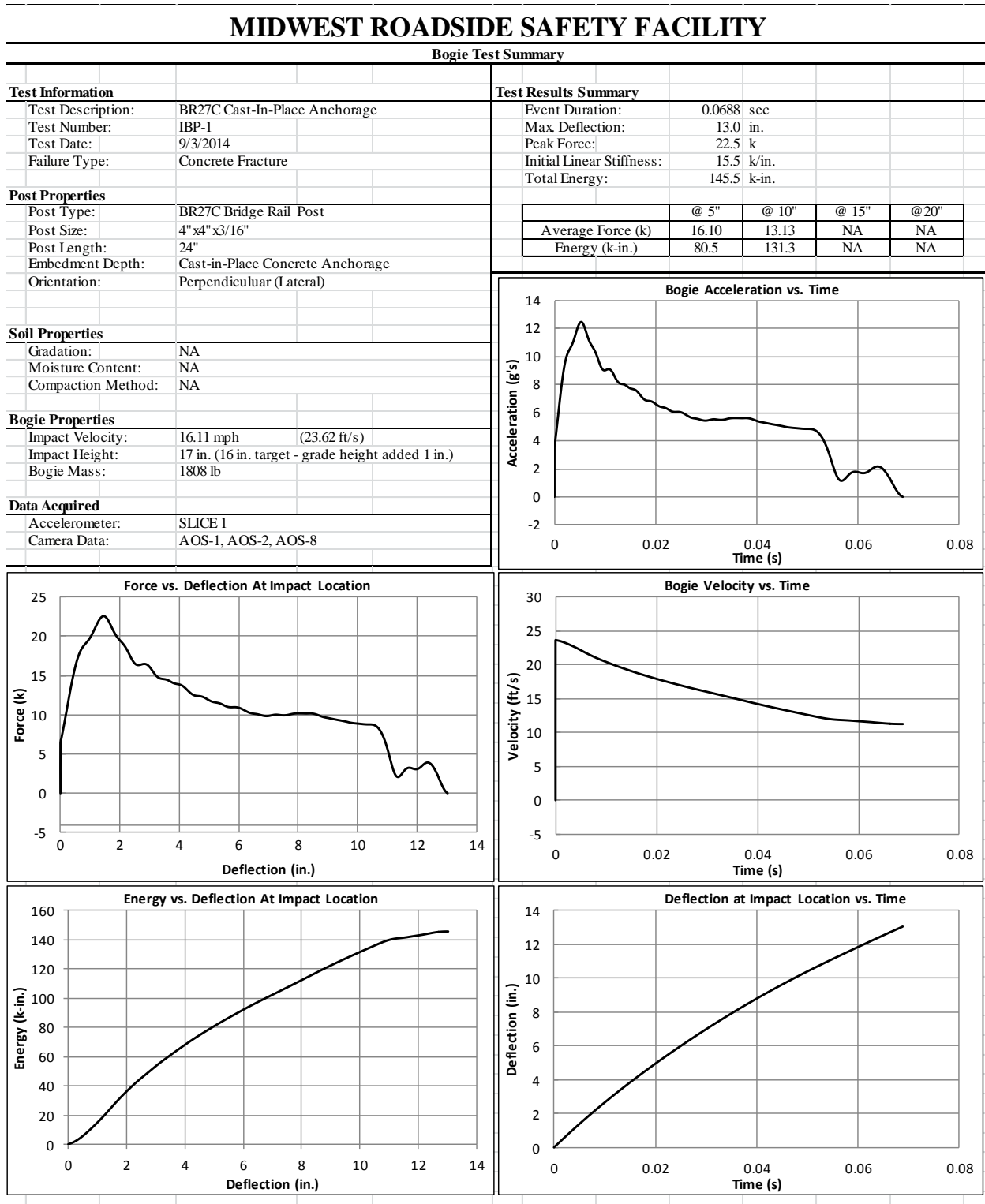


Figure C-1. Test No. IBP-1 Results (SLICE-1)

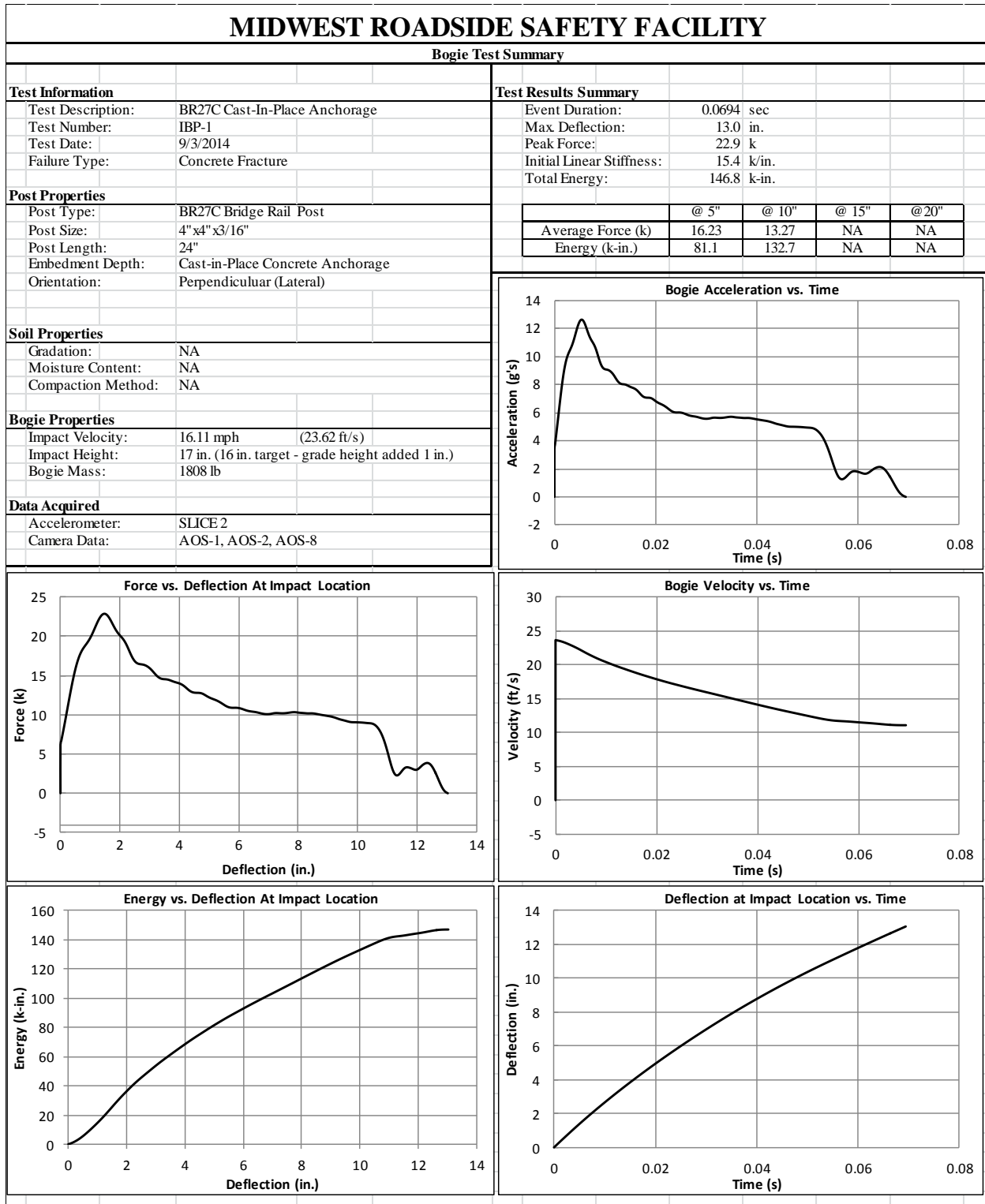


Figure C-2. Test No. IBP-1 Results (SLICE-2)

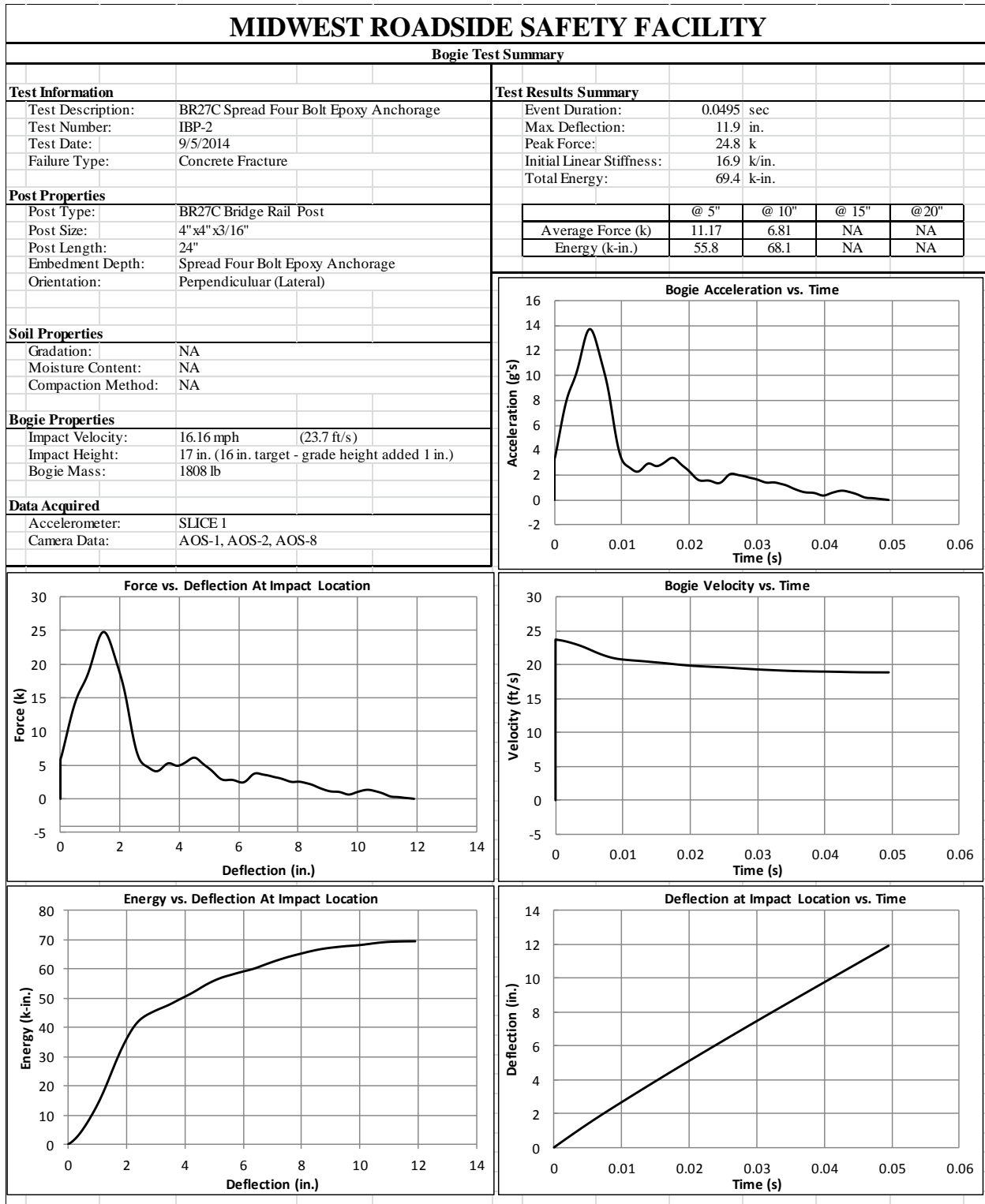


Figure C-3. Test No. IBP-2 Results (SLICE-1)

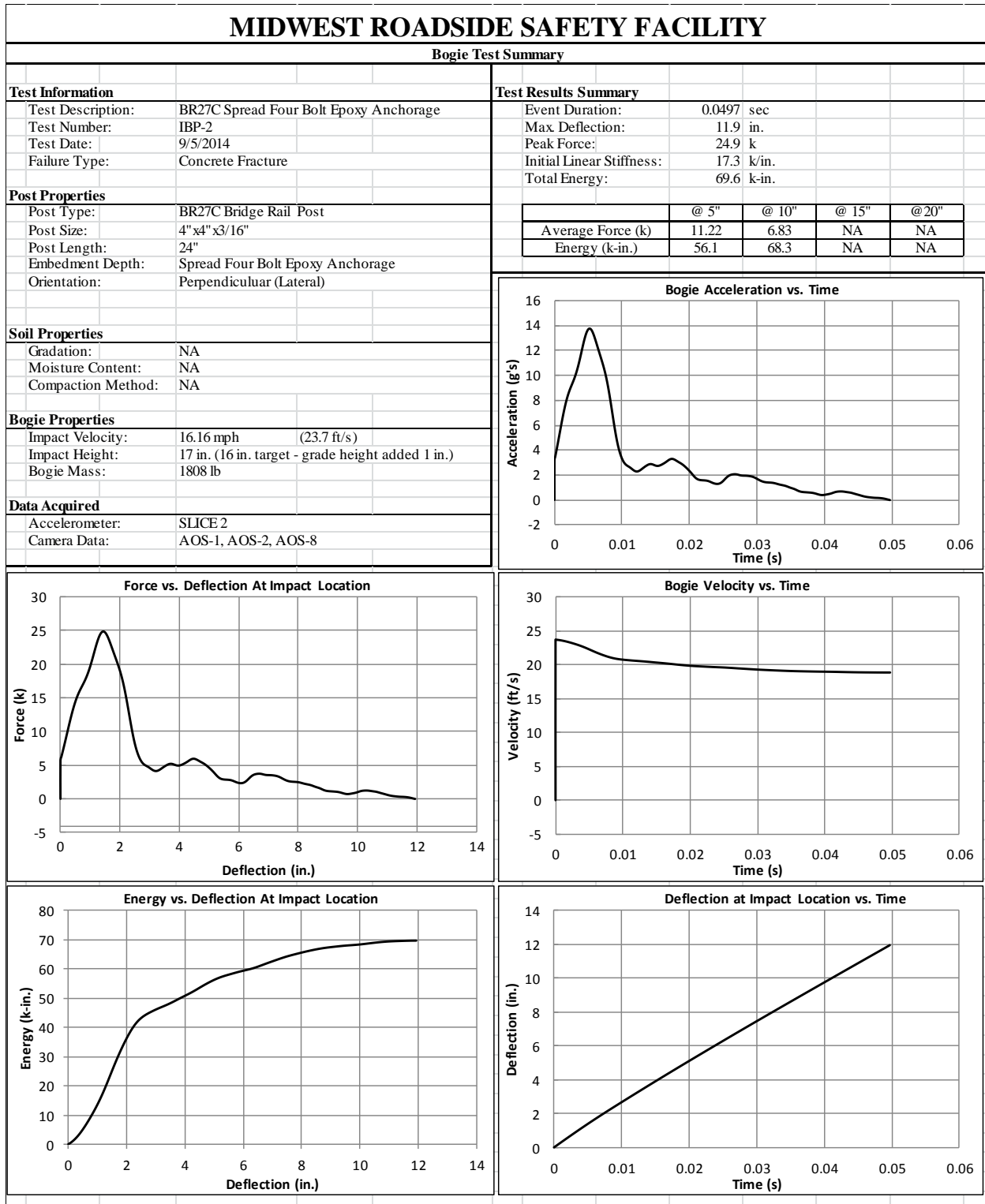


Figure C-4. Test No. IBP-2 Results (SLICE-2)

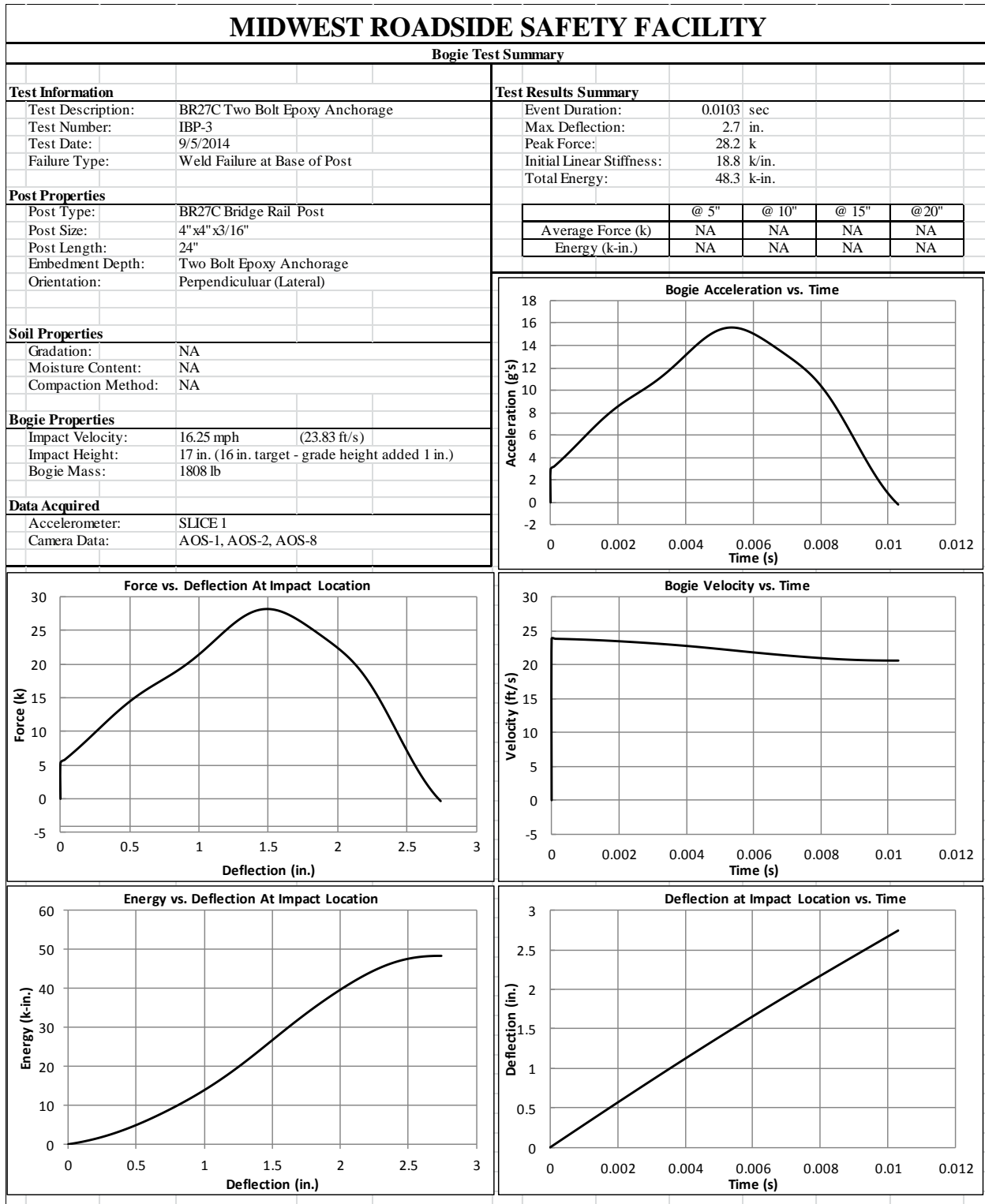


Figure C-5. Test No. IBP-3 Results (SLICE-1)

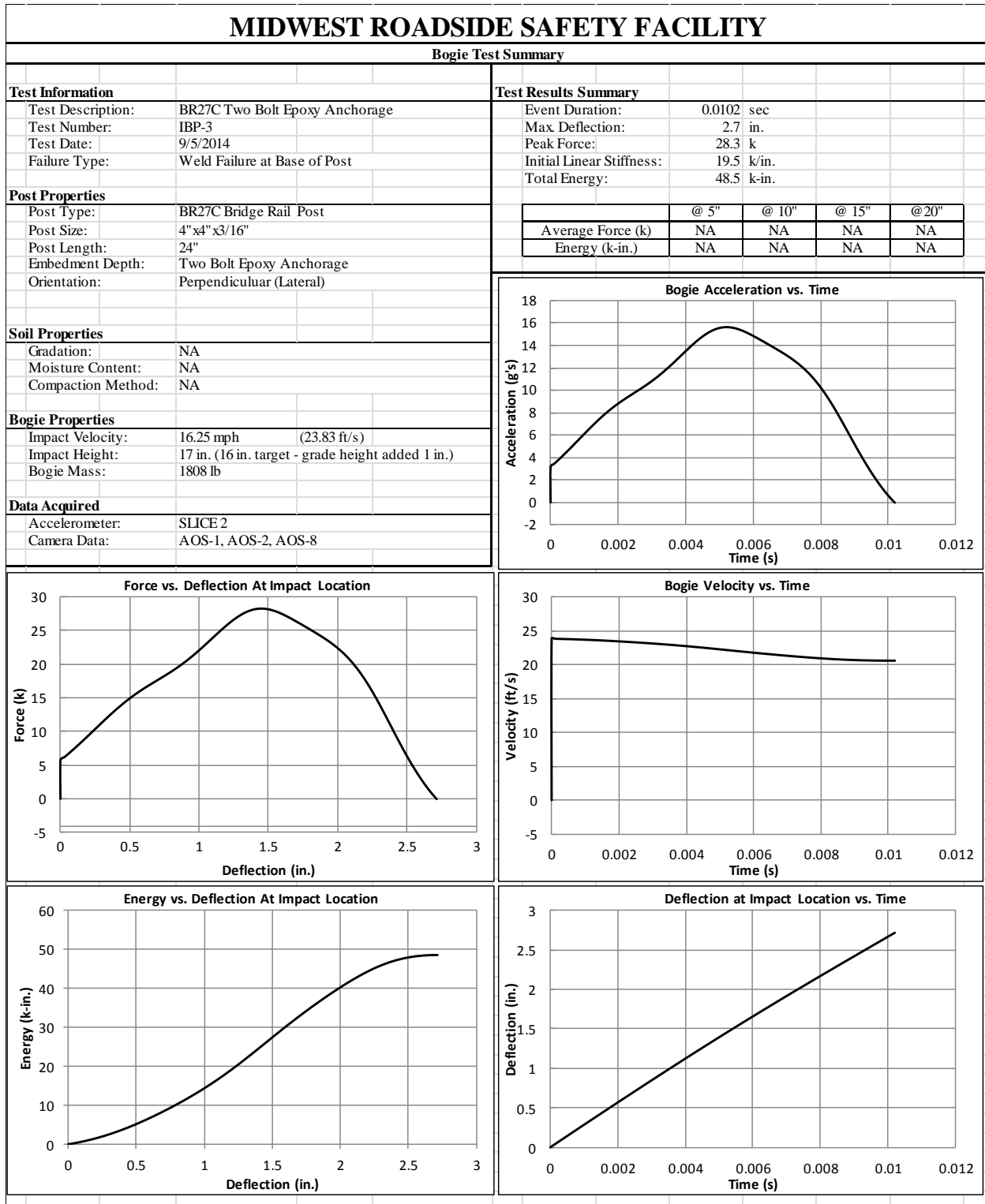


Figure C-6. Test No. IBP-3 Results (SLICE-2)

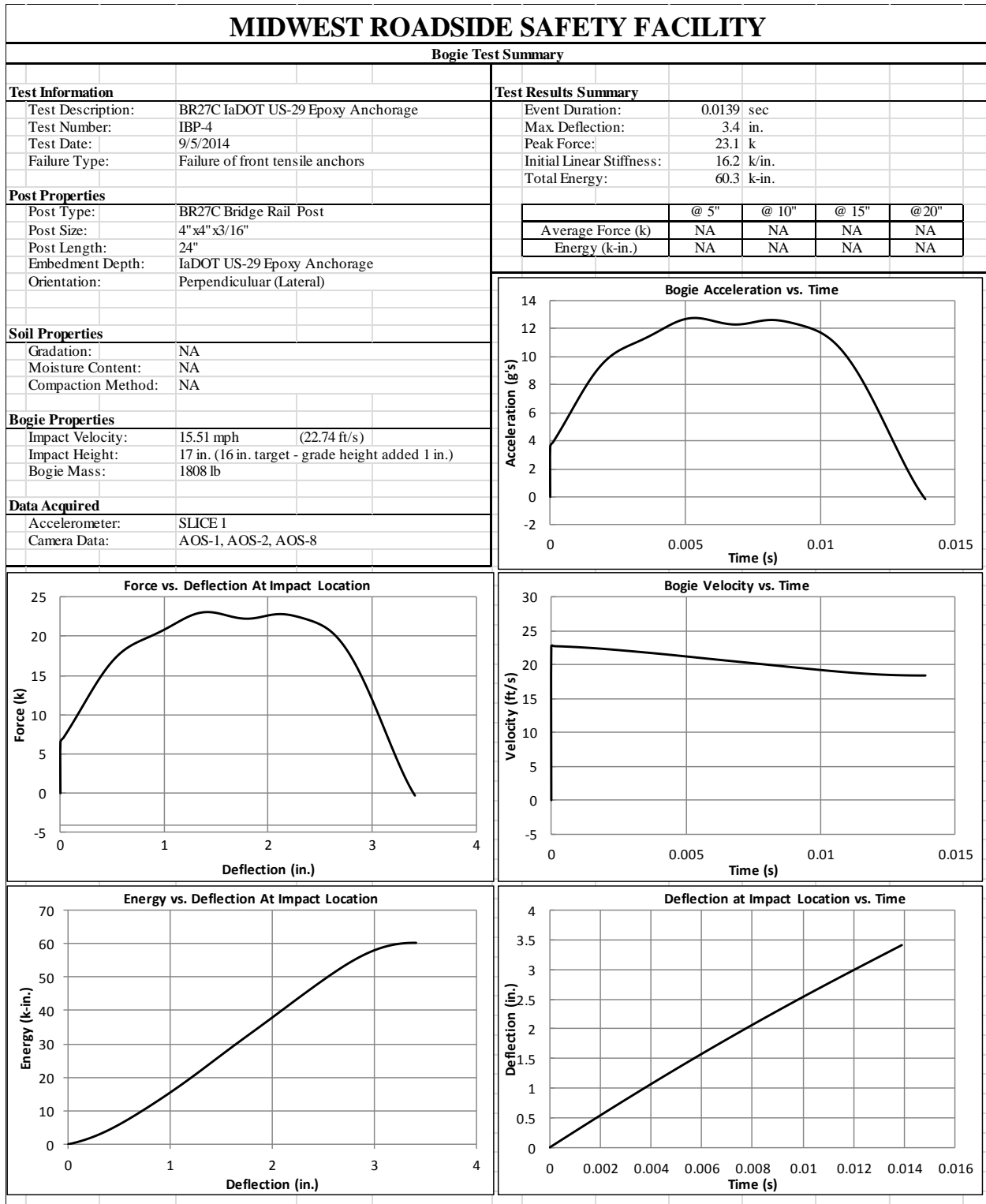


Figure C-7. Test No. IBP-4 Results (SLICE-1)

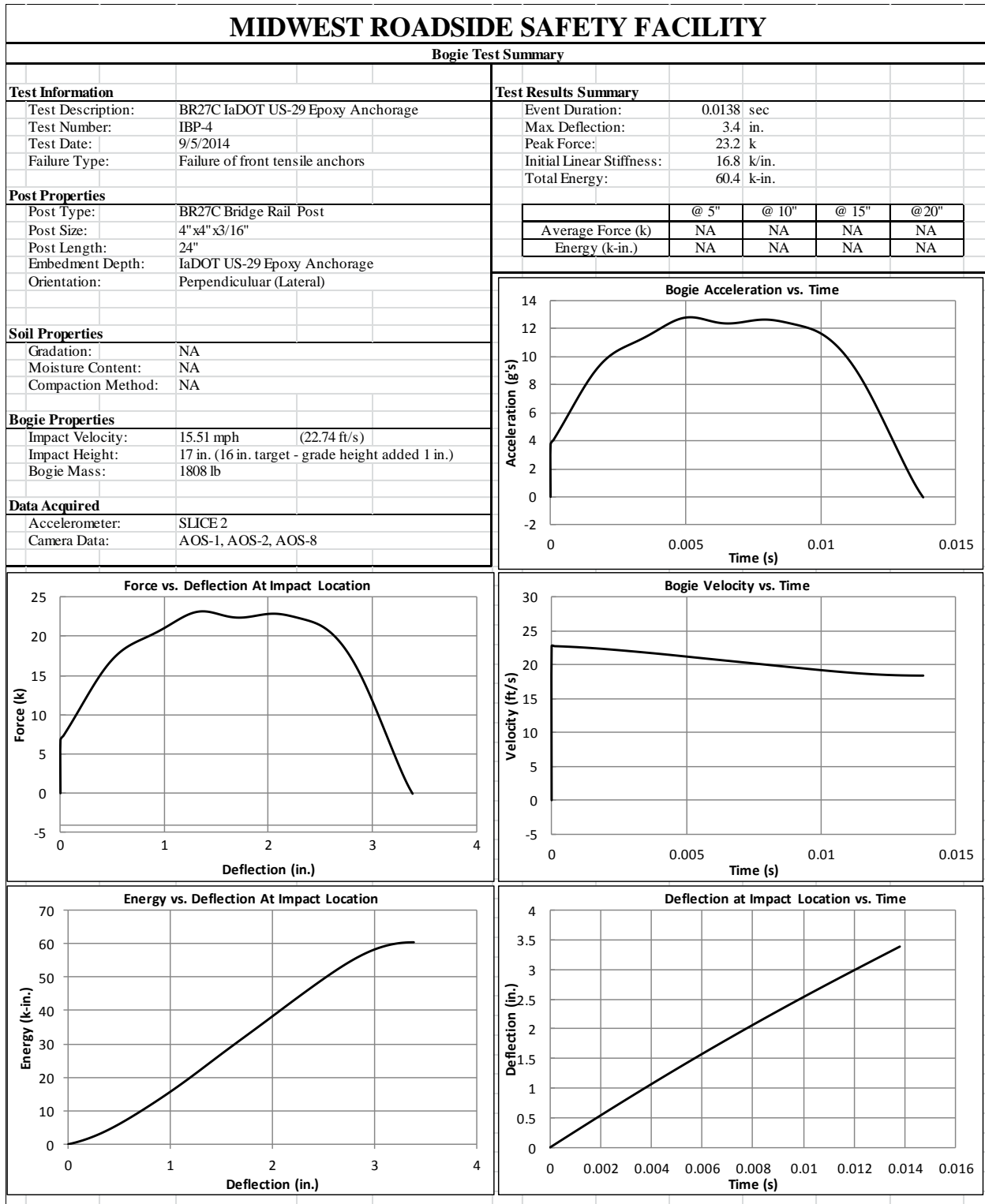


Figure C-8. Test No. IBP-4 Results (SLICE-2)

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