

Investigation of Field Corrosion Performance and Bond/Development Length of Galvanized Reinforcing Steel

**Final Report
December 2014**



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Institute for Transportation

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TABLE OF CONTENTS

ACKNOWLEDGMENTS	vii
EXECUTIVE SUMMARY	ix
1. INTRODUCTION	1
1.1. Background	1
1.2. Objectives	2
2. LITERATURE REVIEW	3
2.1. Bond Characteristics	3
2.2. Bond Strength Test Method	4
2.2.1. Pullout Test	4
2.2.2. Beam End Test	5
2.2.3. Beam Anchorage and Splice Tests	5
2.3. Galvanized Reinforcing Steel	5
3. EXPERIMENTAL PROGRAM	10
3.1. Laboratory Testing	10
3.1.1. Test Specimen	10
3.1.2. Test Setup and Procedure	14
3.1.3. Results	16
3.1.4. Discussion	22
3.2. Field Monitoring	23
4. CONCLUSIONS AND RECOMMENDATIONS	25
4.1. Laboratory Testing	25
4.2. Field Monitoring	25
4.3. Recommendations and Future Research	25
REFERENCES	27

LIST OF FIGURES

Figure 2-1. Force transfer mechanisms [2] (ACI 408R-03)	3
Figure 2-2. Schematic of test specimens [2] (ACI 408R-03)	4
Figure 2-3. Typical stages of corrosion [11] (www.galvanizeit.org)	6
Figure 2-4. Steel dipped into molten zinc bath [16] (www.zinc.org)	7
Figure 2-5. Galvanization process [17] (www.azom.com)	7
Figure 3-1. Typical test specimen	11
Figure 3-2. Wooden forms with epoxy-coated (left) and galvanized (right) test bars	12
Figure 3-3. Concrete cast into the wooden forms	12
Figure 3-4. Sample specimens	13
Figure 3-5. Test apparatus details	15
Figure 3-6. Load versus loaded end slip (#6 galvanized test bars)	18
Figure 3-7. Load versus loaded end slip (#6 epoxy-coated test bars)	18
Figure 3-8. Load versus loaded end slip (#8 galvanized test bars)	19
Figure 3-9. Load versus loaded end slip (#8 epoxy-coated test bars)	20
Figure 3-10. Load versus loaded end slip (#10 galvanized test bars)	21
Figure 3-11. Load versus loaded end slip (#10 epoxy-coated test bars)	21
Figure 3-12. Crack patterns of the specimens	23

LIST OF TABLES

Table 2-1. ASTM standards used for galvanizing steel	8
Table 2-2. Reinforcing steel standards for hot dip galvanizing reinforcing steel	9
Table 2-3. General galvanizing standards	9
Table 3-1. Summary of beam end tests (epoxy-coated test bars)	17
Table 3-2. Summary of beam end tests (galvanized test bars)	17

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EXECUTIVE SUMMARY

In reinforced concrete systems, ensuring that a good bond between the concrete and the embedded reinforcing steel is critical to long-term structural performance. Without good bond between the two, the system simply cannot behave as intended. The bond strength of reinforcing bars is a complex interaction between localized deformations, chemical adhesion, and other factors. Coating of reinforcing bars, although sometimes debated, has been commonly found to be an effective way to delay the initiation of corrosion in reinforced concrete systems.

For many years, the standard practice has been to coat reinforcing steel with an epoxy coating, which provides a barrier between the steel and the corrosive elements of water, air, and chloride ions. Recently, there has been an industry-led effort to use galvanizing to provide the protective barrier commonly provided by traditional epoxy coatings. However, as with any new structural product, questions exist regarding both the structural performance and corrosion resistance of the system.

In the fall of 2013, Buchanan County, Iowa constructed a demonstration bridge in which the steel girders and all internal reinforcing steel were galvanized. The work completed in this project sought to understand the structural performance of galvanized reinforcing steel as compared to epoxy-coated steel and to initiate a long-term corrosion monitoring program. This work consisted of a series of controlled laboratory tests and the installation of a corrosion monitoring system that can be observed for years in the future.

The results of this work indicate there is no appreciable difference between the bond strength of epoxy-coated reinforcing steel and galvanized reinforcing steel. Although some differences were observed, no notable difference in either peak load, slip, or failure mode could be identified. Additionally, a long-term monitoring system was installed in this Buchanan County bridge and, to date, no corrosion activity has been identified.

1. INTRODUCTION

1.1. Background

In reinforced concrete systems, a good bond between reinforcing steel and concrete is critical for developing a desired/required load-carrying capacity of the system. The bond strength of reinforcing bars depends on many factors including the surface geometry of the bar, coatings on the bars, concrete cover, clear spacing, diameter of bars, development and splice length, amount of transverse reinforcing steel, and compressive strength of concrete. Each of these factors contributes differently to the bond strength and it is sometimes difficult to quantify the contribution individually.

As is widely known, compared to reinforcing bars with a smooth surface, bond strength can be increased significantly by deforming the surface of the bars (e.g, ribbed bars). Different coating techniques and materials will also have different effects on the bond strength to concrete and, thus, will offer different overall strength in the reinforced concrete system. The impact of bar coatings is well documented in codified form by standardized multipliers for epoxy-coated bars.

Coating of reinforcing bars has been broadly adopted as one of the measures for providing corrosion protection in concrete as the need to design for durability has become an important practice in civil engineering. Among several general coating systems, two coating materials have been used predominantly in structural practice (although typically in different applications): hot-dip galvanizing and epoxy coating.

Galvanizing provides a metallurgical alloy coating and zinc iron alloys adherent to the steel, which protects the steel from corrosion by providing both an exterior barrier (i.e., zinc coating) as well as sacrificial protection (i.e., anodic function of a zinc) to the underlying steel. Epoxy coating provides a physical barrier that protects the steel from corrosion by isolating the steel base from the elements needed for corrosion to occur (e.g., oxygen, moisture, and chloride ions). The coating also acts as an electrical insulator and minimizes the flow of corrosion current as long as it is not damaged.

While galvanized reinforcing steel has been used frequently by some countries (Australia, etc.) and some studies dispute the effectiveness of epoxy coating, numerous states in the US require the use of epoxy coating as a means of extending reinforced concrete service lives. As the bond between concrete and reinforcing steel is fundamental to the strength-based performance of structural concrete, it is essential to investigate the performance of the structural elements in which these methods are employed.

Although some studies exist comparing performance between galvanized steel and epoxy-coated bars, most studies focus on evaluating either corrosion performance or structural behavior. Quite interestingly, there is very little consistency between the results regardless of the performance metric being assessed. Therefore, a need exists to conduct an experimental investigation to verify the bond characteristics of these two types of reinforcing steel.

In the fall of 2013, Buchanan County, Iowa constructed a new demonstration bridge in which all steel girders and reinforcing steel were galvanized. The galvanized steel was provided by the galvanizing industry at no cost because of their interest to further investigate the issue of galvanizing versus epoxy coating.

This report presents a dual phase study that investigated the bond strength of galvanized reinforcing steel and epoxy-coated bars through laboratory testing, and initiated a mechanism to monitor the effectiveness of galvanizing reinforcement at providing corrosion resistance.

1.2. Objectives

The primary objectives of this study were to investigate the difference in bond strength and development length between galvanized reinforcing steel and epoxy-coated bars by means of beam end tests and to instrument a bridge with sensors to evaluate, over long periods of time, the field performance of the galvanized reinforcing steel used in the bridge.

In the laboratory investigation, the bond strength of reinforcing steel was investigated by using beam end specimens with various bar sizes. A total of 18 specimens were tested and the results from load-slip measurements were used as an indication of the variation in bond strength. Note that this study was not intended to define a new design method or an independent relationship for each specimen tested. It was intended to compare, in a relative manner, the bond strength of concrete-to-galvanized reinforcing steel to concrete-to-epoxy-coated bars.

In the field monitoring, 10 galvanized reinforcing bars in the bridge deck were instrumented with embeddable corrosion sensors and data were collected occasionally to assess their performance in terms of their corrosion resistance.

2. LITERATURE REVIEW

2.1. Bond Characteristics

Bond is the force transfer between two materials [1]. Bond stress in reinforced concrete systems can be thought of as a shear stress at the steel-concrete interface, which modifies the steel stress by transferring the load between the reinforcing bars and surrounding concrete. This stress transfer is only possible with an adequate bond that must be maintained between the two materials.

Among many factors controlling the bond strength in a structural member (e.g., yield strength of reinforcing bars, sizes and spacing of bars, and confinement of bars by lateral ties), the three primary mechanisms of stress transfer (or forces) from reinforcement and surrounding concrete are through chemical adhesion, frictional resistance, and mechanical interlock [2] as depicted in Figure 2-1.

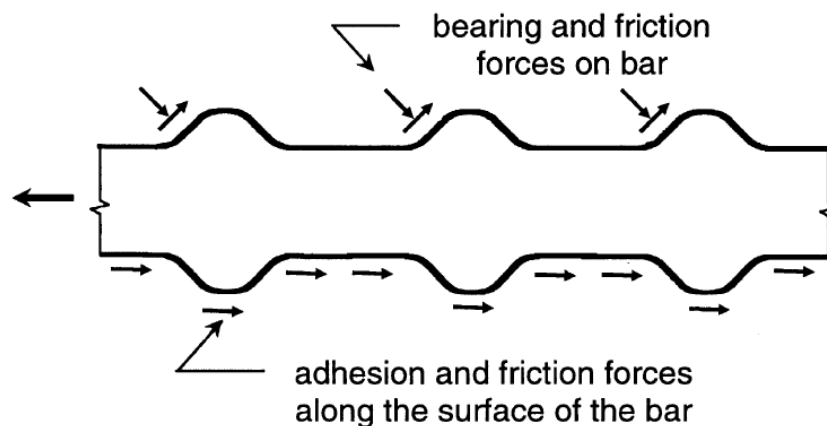


Figure 2-1. Force transfer mechanisms [2] (ACI 408R-03)

Each of these mechanisms contributes to the stress transfer through reinforcing bars and the conditions under which the concrete is placed. Rich mixes have better adhesion than weak mixes. The friction between reinforcing bars and concrete will be primarily influenced by the roughness of the bar surface area, concrete mix, shrinkage, and concrete cover. For deformed reinforcing bars, the main contribution comes from the mechanical interlock, with the frictional and chemical bonds both helping to a lesser degree.

Numerous studies [3-7] have investigated the bond characteristics of conventional uncoated and epoxy-coated reinforcing bars in concrete. The findings of these studies were added to the American Concrete Institute (ACI) Committee 408 database on “Bond and Development of Straight Reinforcing Bars in Tension” [2] and used in formulating the equations in both ACI 318 and ACI 408R that are used to predict the bond strength.

2.2. Bond Strength Test Method

Some of the test methods commonly used to determine the bond strength between reinforcing bars and concrete are the direct pullout test, beam end test, beam anchorage test, and beam splice test (Figure 2-2).

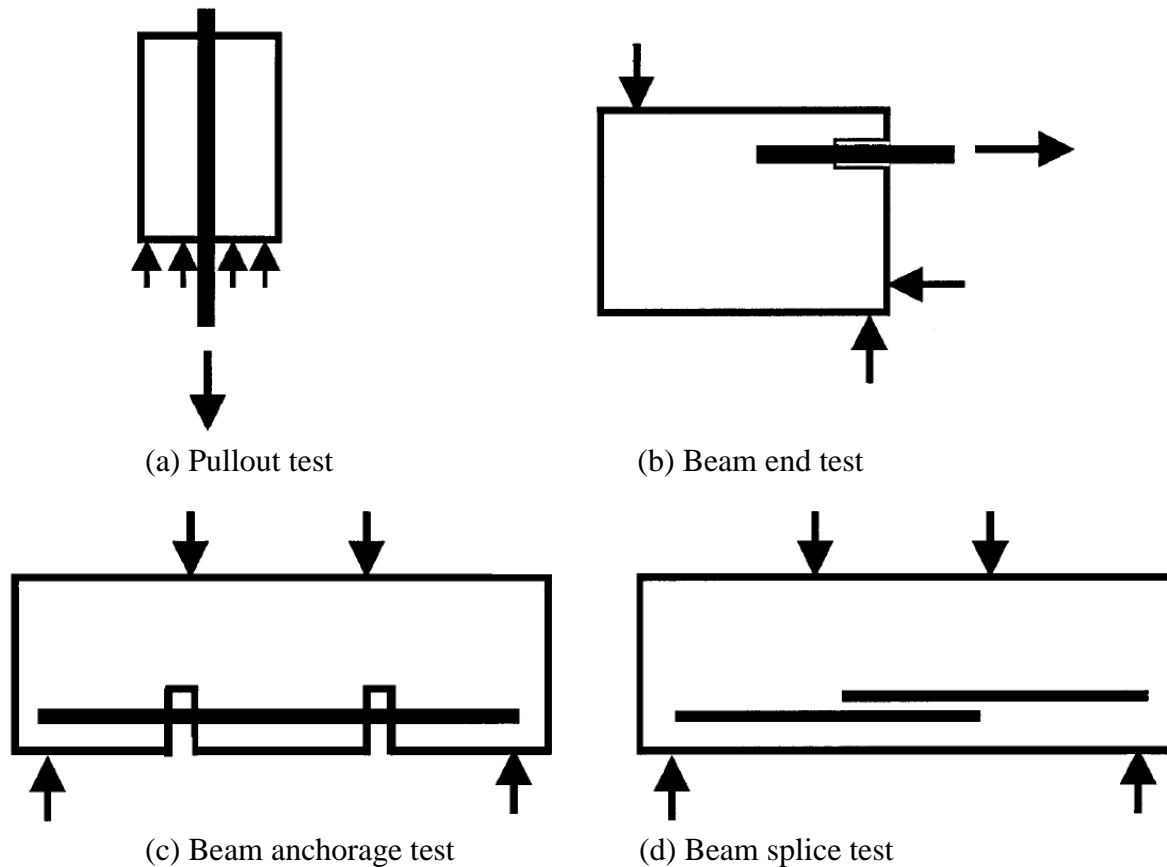


Figure 2-2. Schematic of test specimens [2] (ACI 408R-03)

The direct pullout and beam end tests are considered small-scale test methods while the other two are considered large-scale.

2.2.1. Pullout Test

A typical test specimen used in the direct pullout test consists of a test bar encased in concrete with both ends of a test bar exposed (Figure 2-2a). This test is run such that the test bar is loaded at one end in tension until failure while the other end is left free. Slip is measured on both ends to allow further study of the behavior of the bar-plus-concrete system. Although the pullout test is the simplest method to determine the bond strength of reinforcing steel, it is the least realistic one because, during the testing, the entire concrete surrounding the test bar is in compression; whereas, in real application of flexural members in the tension region, both the reinforcing bars and adjacent concrete are in tension.

2.2.2. Beam End Test

A test specimen used in the beam end test (Figure 2-2b), also known as a semi beam or modified cantilever beam test, is similar to that of the pullout test with a difference being the use of bond breakers along the test bar at both the loaded and unloaded ends of the specimen. The bond breakers are used to control the bonded length of the test bar and to prevent a cone-type failure at the loaded end of the specimen.

The beam end test is relatively easy to setup and conduct and is known to provide results that represent how embedded reinforcing bars would behave in a full-scale beam. During testing, a compressive force (or support reaction) needs to be placed at least the distance as the embedded length of the test bar away from the end of the test bar. This creates a stress state in concrete similar to those in real flexural reinforced concrete members.

2.2.3. Beam Anchorage and Splice Tests

The beam anchorage test is intended to measure the true development length of reinforcement. A specimen used in this test is considered to represent a full-scale beam with two cracked sections at the bottom of the specimen (Figure 2-2c) and a known bonded length. The beam splice test (Figure 2-2d) is used to measure and study the splice length of test bars. The location of the splice and loading configuration are designed such that a test specimen is subject to a constant moment along the length of the splice. Bond strengths determined from these tests are typically similar.

2.3. Galvanized Reinforcing Steel

Galvanized steel has been used throughout the civil engineering and construction industry in many forms including steel reinforcement, bolts, ties, anchors, dowel bars, piping, and other structural elements. Although the application of zinc-coated steel in concrete structures dates back to 1908, its popular use in the US came during the 1930s and its interest continued to increase after World War II and throughout the 1960s and 70s. It was used predominantly in bridge and highway construction across the Snow Belt states that experienced heavy snowfall in winter.

The application of galvanized reinforcing steel diminished, however, in the late 1970s when the Federal Highway Administration (FHWA) temporarily classified galvanizing as an experimental system. This ruling was rescinded in 1983 and, since that time, there has been world-wide use (especially in countries like Australia, New Zealand, South Africa, etc.) of galvanized reinforcement in multiple weather conditions [8].

The primary purpose for galvanizing steel is to protect the steel from corrosion. Corrosion in reinforcing steel results in deterioration of the concrete within a structural system. Galvanizing provides reinforcing steel with a zinc coating to protect the steel from moisture and other

corroding compounds (such as chlorides). The zinc coating helps the element retain its structural integrity, as well as allowing early detection of corrosion [9].

The two most common causes of steel corrosion are chloride induced corrosion and carbonation [10]. Chlorides are the most detrimental corroding element for steel reinforcement. The chlorides typically come from the materials used in mixing, saltwater exposure, and application of deicing agents. As with other types of corrosion, the result is an expansion of voluminous corrosion products causing cracks and spalling of the surrounding concrete (Figure 2-3), leading to a reduction in remaining service life of the structure.

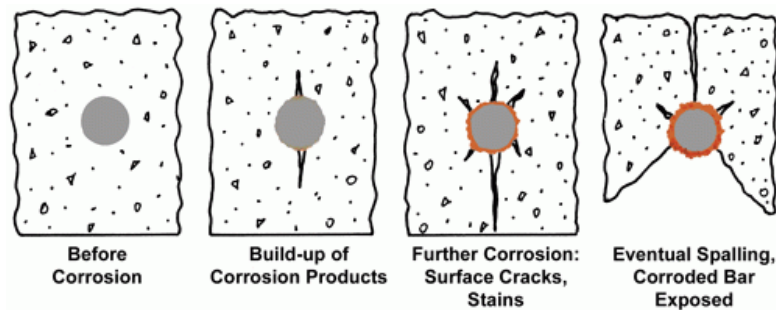


Figure 2-3. Typical stages of corrosion [11] (www.galvanizeit.org)

Galvanized steel has a chloride threshold of nearly 5 to 6 times higher than conventional uncoated steel. Carbonation occurs due to the difference in pH level between water and concrete. Concrete has a higher alkalinity in comparison to acidic water (e.g., rainwater) causing a reaction to neutralize the difference. The time it takes for carbonation to penetrate into concrete depends on the condition of concrete. Once carbonation occurs inside the concrete mass, the pH level drops from about 11.5 to 7 [12].

When conventional uncoated steel is used as the reinforcement, the surface of the steel depassivates as its pH level decreases, allowing corrosion to commence. With galvanized steel, however, the zinc coating corrodes first and at a slower rate than the uncoated steel. This reduction in corrosion rate makes galvanized reinforcement more adequate in carbonated concrete.

Some studies [8, 9, 13 and 14] report that concrete bonds better to galvanized reinforcement than it does to uncoated steel. Chemical reactions that occur with galvanized steel result in a stronger adhesion between the reinforcement and concrete as well as increased frictional resistance to slipping. When galvanized reinforcement comes in contact with wet cement, a layer of calcium hydroxyl-zincate is formed at the surface [15]. Once formed, this layer firmly adheres to the zinc coating as well as its surrounding concrete, resulting in an increase in bond strength compared to uncoated steel. Tests have shown that as the zinc coating corrodes, its surrounding areas are densified, which would lead to further bonding in that area. These chemical reactions do not occur to uncoated or epoxy-coated steel.

Among several ways to galvanize reinforcing steel such as hot dipping, electroplating, spraying, and mechanical alloying, hot dipping is the most accepted and effective method for galvanizing structural steel [9]. It involves immersing clean steel in a bath of molten zinc (Figure 2-4) at about 840°F (450°C), during which a metallurgical reaction occurs between the steel and the zinc.



Figure 2-4. Steel dipped into molten zinc bath [16] (www.zinc.org)

A key feature of a galvanized coating is that it is metallurgically bonded to the steel and becomes an integral part of the steel. A hot-dip galvanizing (Figure 2-5) provides a thicker layer of zinc coatings that are better bonded to the underlying steel in comparison to the other galvanizing techniques.

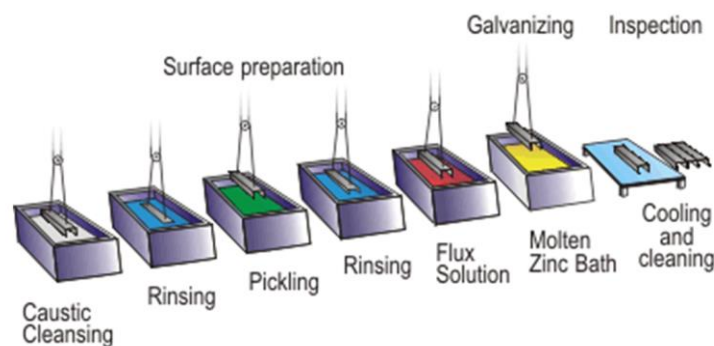


Figure 2-5. Galvanization process [17] (www.azom.com)

There have been concerns of potential adverse effects of hot-dip galvanizing on the microstructure or mechanical properties of steel [8]. The temperature to which steel is heated and the rate at which it is allowed to cool during fabrication is what determines the steel's properties.

Usually, steel is heated above 1,200°F (650°C) during fabrication. Hot-dip galvanizing is performed at a much lower temperature of about 840°F (450°C). Therefore, it does not reach a temperature that would result in adversely affecting the strength and other mechanical properties of the steel that is galvanized.

Also, while some research studies have indicated that galvanizing may soften cold worked steel and embrittle high strength steel, other studies show that the extent of softening and the possibility of embrittlement are minor, and that the mechanical properties of the steel used in today's construction industry are not heavily affected by the process of galvanizing [8, 9]. The use of new techniques such as thermo-mechanically treated and micro-alloyed steels might further decrease the possibility.

Galvanized reinforcing steel can be handled, stored, and transported using the basic methods that are used for conventional uncoated steel and no special requirements need to be considered. However, some minor cautions need to be taken. For example, it is suggested that when maneuvering galvanized rebar of long lengths, a spreader bar and additional nylon straps should be used to prevent sag and any rubbing of the bars, which could damage the coating surface [9].

Galvanized reinforcement can be adequately welded using any welding technique. Before the weld is made, the zinc coating needs to be removed, usually by grinding. This will prevent zinc from entering the weld and ensures full weld penetration. For reinforced concrete details that require the use of hooks, damage to the coating can be minimized by using large bend diameters.

The standards and regulations for hot dip galvanized reinforcement are handled differently around the world [8, 9]. While some countries treat reinforcing steel in the same way as any other steel products, such that hot dip galvanized steel falls under a general galvanizing standard, others use dedicated standards relating only to reinforcing steel. Examples of specifications and standards used in the US and in other countries are given in Tables 2-1 through 2-3.

Table 2-1. ASTM standards used for galvanizing steel

Designation	Title
ASTM A 90	Test method for weight (mass) of coating on iron and steel articles with zinc or zinc-alloy coatings
ASTM A 143	Safeguarding against embrittlement of hot-dip galvanized structural steel products and procedure for detecting embrittlement
ASTM A 653	Specification for steel sheet, zinc-coated (galvanized), or zinc-iron alloy-coated (galvanized) by the hot-dip process
ASTM A 767	Specification for zinc-coated (galvanized) steel bars for concrete reinforcement
ASTM A 780	Practice for repair of damaged and uncoated area of hot-dip galvanized coatings

Table 2-2. Reinforcing steel standards for hot dip galvanizing reinforcing steel

Country	Designation	Title
France	NF A35-025	Hot-dip galvanized bars and coils for reinforced concrete
Italy	UNI 10622	Zinc-coated (galvanized) steel bars and wire
India	IS 12594	Hot-dip coatings on structural steel bars for concrete reinforcement specifications
International Standards Organization	ISO 14657	Zinc-coated steel for the reinforcement of concrete

Table 2-3. General galvanizing standards

Country	Designation	Title
Australia / New Zealand	AS/NZS 4680	After-fabrication hot dip galvanizing
Canada	CAN/CSA G164	Hot dip galvanizing of irregularly shaped articles
South Africa	SABS/ISO 1461	Hot dip galvanized coatings on fabricated iron and steel articles
Sweden	SS-EN ISO 1461	Hot dip galvanized coatings on fabricated iron and steel articles
United Kingdom	BS EN ISO1461	Hot dip galvanized coatings on fabricated iron and steel articles
International Standards Organization	ISO 1461	Hot dip galvanized coatings on fabricated iron and steel articles

3. EXPERIMENTAL PROGRAM

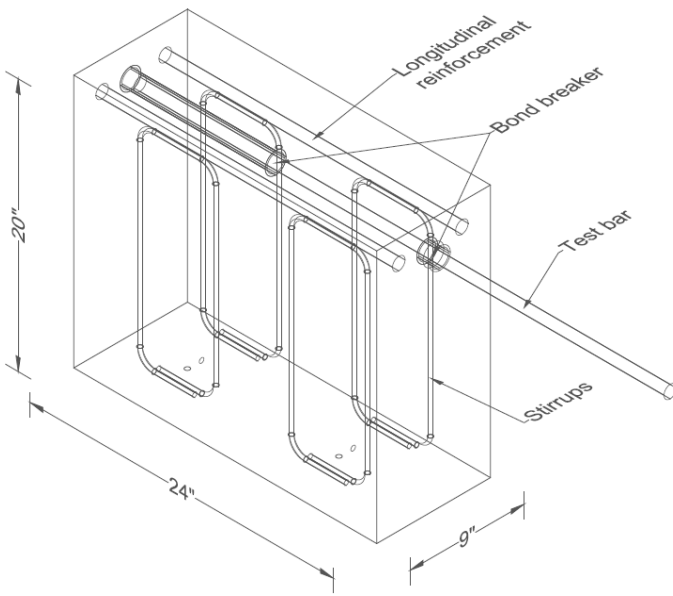
Section 3.1 describes the beam end tests performed in accordance with ASTM A 944 [18]. During the testing, the pullout performance of both epoxy-coated and galvanized reinforcing bars were evaluated to compare their bond strength. Section 3.2 presents a brief summary of instrumentation installed in this Buchanan County bridge to evaluate, over the next several years, the field performance of galvanized reinforcing steel used in the bridge deck through corrosion monitoring.

3.1. Laboratory Testing

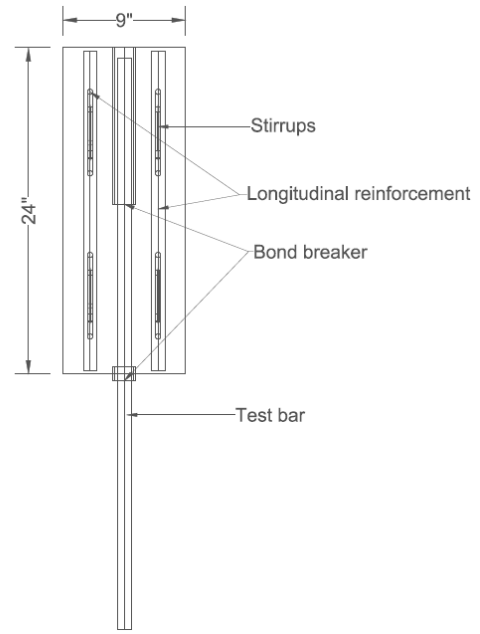
The laboratory testing program consisted of a series of beam end tests, conducted with a total of 18 specimens, to investigate the bond strength of galvanized reinforcing steel as compared with that of epoxy-coated bars. The specimens were fabricated and tested in accordance with ASTM A 944-10.

3.1.1. Test Specimen

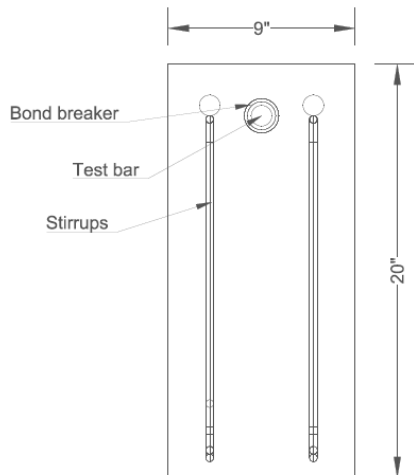
The specimens were divided into two sets based on the coating material used on the reinforcing bars in the specimens: one set of nine specimens with galvanized test bars and one set of nine specimens with conventional epoxy-coated test bars. To investigate the common types of bars used in bridge construction, three different bar sizes were evaluated. Each specimen had a single test bar of either #6, #8, or #10 bar (three of each size per coating combination) cast into a concrete block that was reinforced with four double-legged closed shear stirrups, oriented parallel to the sides of the concrete block and positioned to avoid confining the test bar along its bonded length, and two #8 flexural reinforcing bars running parallel to the test bar. Shear stirrups used were #3 bars for the specimens with #6 and #8 test bars, and #4 bars for the specimens with #10 test bars. In all cases, both the longitudinal reinforcement and shear stirrups were uncoated reinforcing steel. A typical test specimen is illustrated in Figure 3-1.



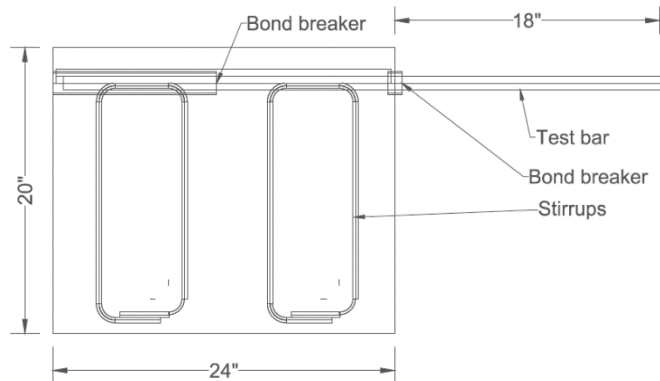
(a) Isometric view



(b) Plan view



(c) Front view



(d) Side view

Figure 3-1. Typical test specimen

The test bar was extended from the front surface of the specimen at a distance that was compatible with the test apparatus. Two PVC pipes were used as bond breakers to control the bonded length of the test bar and to avoid a localized conical failure of the concrete at the loaded end of the specimen (and as specified in ASTM A 944-10). As shown in Figure 3-1, each test bar was unbonded a short distance through the bond breaker at the loaded end, extended along a bonded length, and had an additional unbonded length through the bond breaker placed near the unloaded end. All concrete blocks had the same length and depth of 24 in. and 20 in., respectively. The width of the specimens with the #6 and #8 test bars were 9 in., while the specimens with #10 test bars had a width of 10 in. The compressive strength of the concrete used

in the specimens was between 6,500 psi and 7,000 psi with an average value of 6,827 psi. Photographs taken during the specimen fabrication process are shown in Figures 3-2 and 3-3.

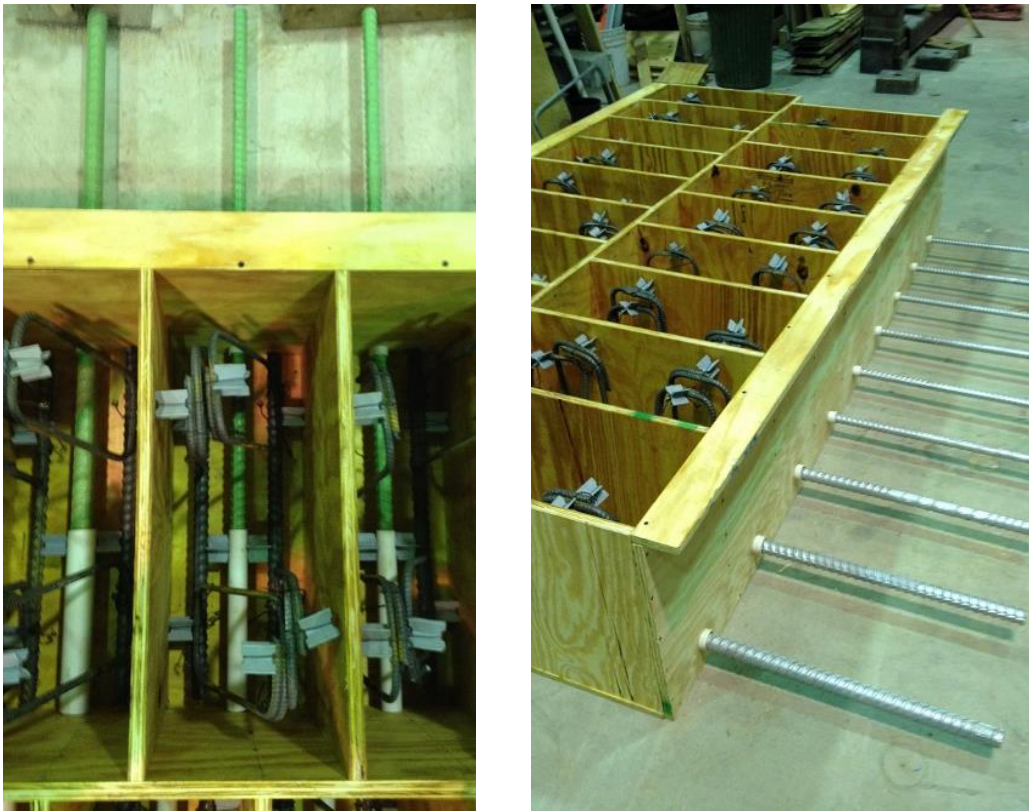


Figure 3-2. Wooden forms with epoxy-coated (left) and galvanized (right) test bars



Figure 3-3. Concrete cast into the wooden forms

Each specimen was labeled in the following format: size of longitudinal rebar, followed by a letter E for epoxy-coated or G for galvanized, followed by the specimen number. For example, 8G-2 corresponded to the second of the three test specimens containing a #8 galvanized test bar. Sample photographs of specimens used in the beam end test are presented in Figure 3-4.



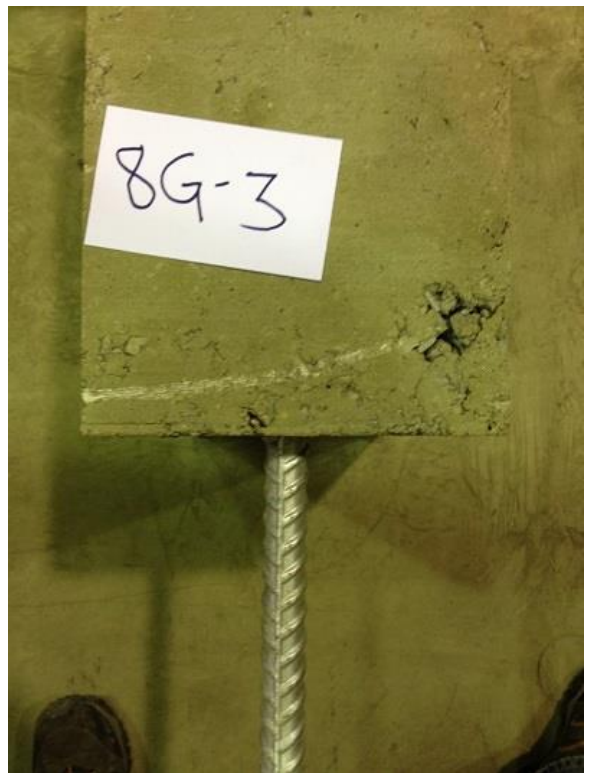
(a) Isometric view showing the loaded end



(b) Isometric view showing the unloaded end



(c) Specimen with poor concrete consolidation

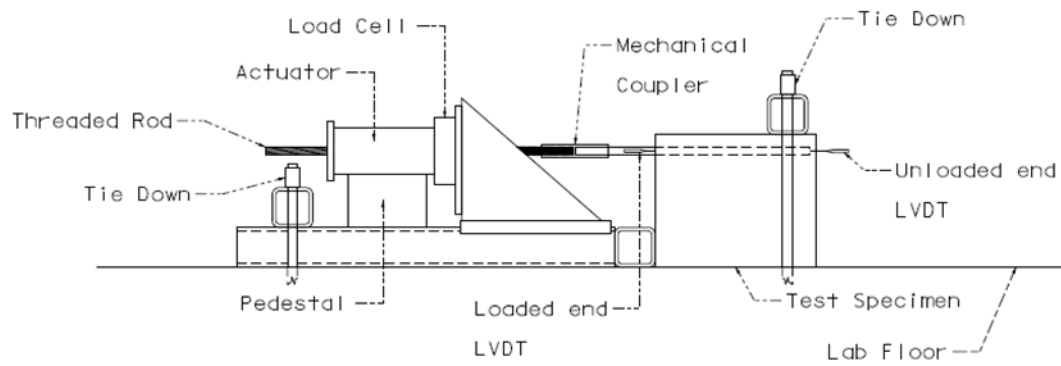


(d) Specimen with #8 galvanized test bar

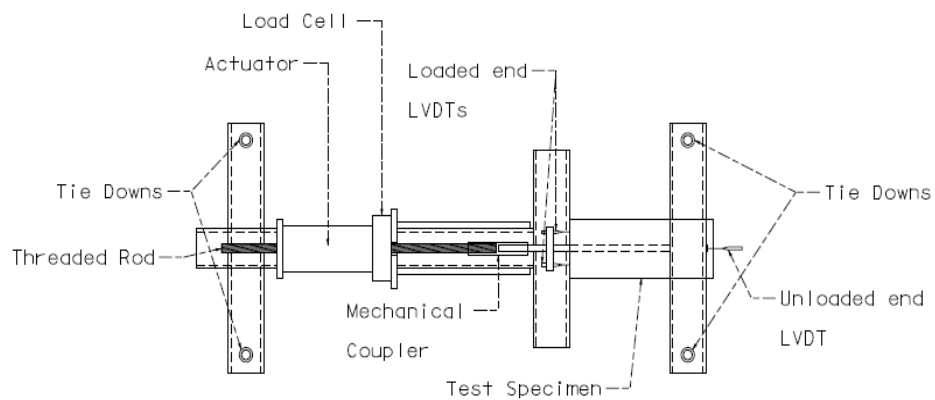
Figure 3-4. Sample specimens

3.1.2. Test Setup and Procedure

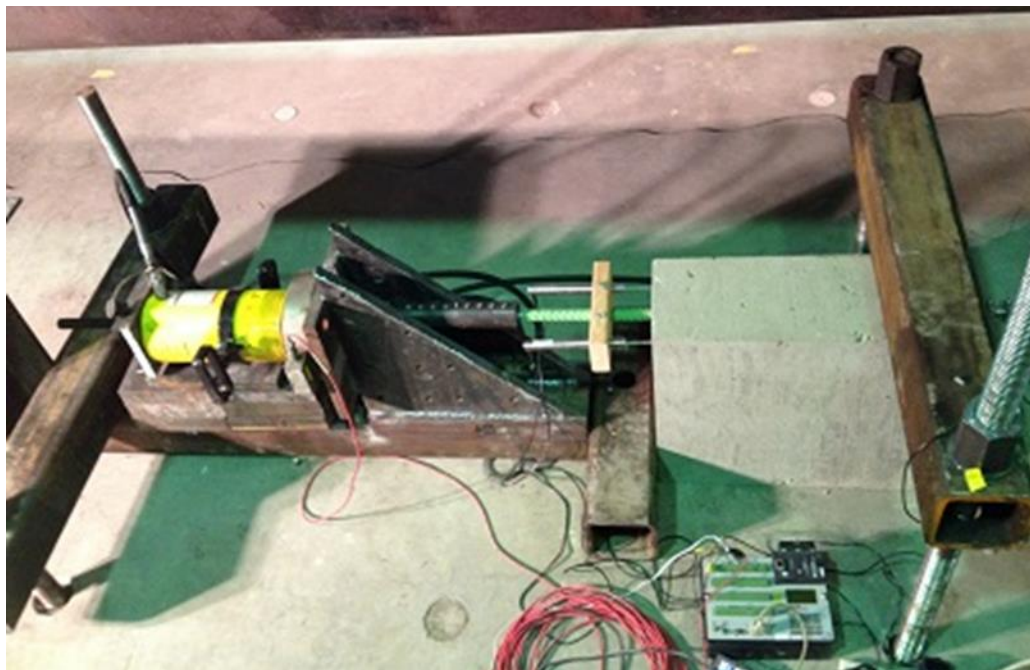
The test setup (Figure 3-5) was assembled following the guidelines given in ASTM A 944-10 with a minor modification made in assembling the apparatus: the double hydraulic ram and yoke system was replaced with a single actuator pulling on the threaded bar coupled with a test bar.



(a) Elevation view



(b) Plan view



(c) Test setup

Figure 3-5. Test apparatus details

This was done to prevent uneven loading from the two-jack system. The free end of the test bar was butted against a hollow steel conduit by means of a mechanical coupler to provide access to the free end of the test bar for measuring slip. The test system was assembled such that it had sufficient capacity to prevent yielding of the various components during testing.

Two linear variable differential transformers (LVDTs) were attached to the loaded end by means of a clamp with the sensor core touching the front face of each specimen. A third LVDT was attached to the rear of the specimen for measuring slip of the unloaded end by means of an L bracket with the sensor core touching the unloaded end of the test bar through the PVC bond breaker. The entire apparatus was placed on the floor and secured to the Iowa State University Structural Engineering Laboratory floor with a hydraulically secured tie down.

The testing was performed by manually pumping the actuator at a constant rate and applying a tensile load until cracks formed on the top and front face of the specimen. Each specimen was positioned in the apparatus so that the test bar was pulled slowly from the specimen. As the specimen was pulled, the bottom of the test specimen reacted in compression against the loading apparatus.

This system created a self-contained loading apparatus. A tie-down at the back end of the specimen restrained the specimen against overturning. A load cell was placed in line with the actuator to read the applied loads. As per ASTM A 944-10, the tensile load was applied parallel to the axis of the test bar and the target loading rate was such that failure does not occur within the first three minutes (180 seconds) of testing. The process was repeated for all specimens.

An average measurement between the two LVDTs was reported as the test bar slip at the loaded end. Both the magnitude of the applied load and the specimens' corresponding slip were recorded. The time taken for the failure of the test specimen after the application of the load was also recorded.

3.1.3. Results

During the testing, it was considered a failure of a specimen when the applied load caused cracks to develop along the specimen and/or the test bar was observed to have slipped. Tables 3-1 and 3-2 summarize the results of the beam end tests for the galvanized reinforcing steel and epoxy-coated bars, respectively. In these tables, the load at which a failure occurred and its corresponding slips at the loaded and unloaded ends are given for each specimen.

Table 3-1. Summary of beam end tests (epoxy-coated test bars)

Specimen	Load at failure (kips)	Slip at failure	
		Loaded end (in.)	Unloaded end (in.)
6E-2	25.710	0.0319	0.0044
6E-4	30.148	0.0353	0.0051
6E-5	29.855	0.0446	0.0031
6E-avg	28.571	0.0372	0.0042
(Std. Dev.)	(2.5)	(0.0066)	(0.0010)
8E-1	30.468	0.0185	0.0033
8E-2	28.745	0.0200	0.0039
8E-3	27.268	0.0179	0.0040
8E-avg	28.827	0.0188	0.0038
(Std. Dev.)	(1.6)	(0.0011)	(0.0004)
10E-1	32.911	0.0147	0.0040
10E-2	30.663	0.0119	0.0039
10E-3	33.136	0.0148	0.0029
10E-avg	32.237	0.0138	0.0036
(Std. Dev.)	(1.4)	(0.0016)	(0.0006)

Table 3-2. Summary of beam end tests (galvanized test bars)

Specimen	Load at failure (kips)	Slip at failure	
		Loaded end (in.)	Unloaded end (in.)
G-1	31.584	0.0335	0.0041
6G-3	27.059	0.0332	0.0046
6G-4	26.757	0.0273	0.0046
6G-avg	28.467	0.0313	0.0044
(Std. Dev.)	(2.7)	(0.0035)	(0.0003)
8G-1	28.565	0.0170	0.0017
8G-2	30.595	0.0124	0.0015
8G-3	29.394	0.0201	0.0030
8G-avg	29.518	0.0165	0.0020
(Std. Dev.)	(1.0)	(0.0039)	(0.0008)
10G-3	35.559	0.0140	0.0025
10G-4	33.365	0.0153	0.0029
10G-5	40.064	0.0211	0.0194
10G-avg	36.329	0.0168	0.0083
(Std. Dev.)	(3.4)	(0.0038)	(0.0096)

The average bond strength of #6 epoxy-coated bar was 28.6 kips with an average slip at the loaded and unloaded ends of 0.037 inches and 0.004 inches, respectively. For the #6 galvanized bars, the average bond strength was 28.5 kips with an average slip at the loaded and unloaded ends of 0.031 inches and 0.004 inches, respectively. The relationship between the loads at failure

versus the average loaded end slip for the #6 galvanized bars and the #6 epoxy-coated bars are shown in Figures 3-6 and 3-7, respectively.

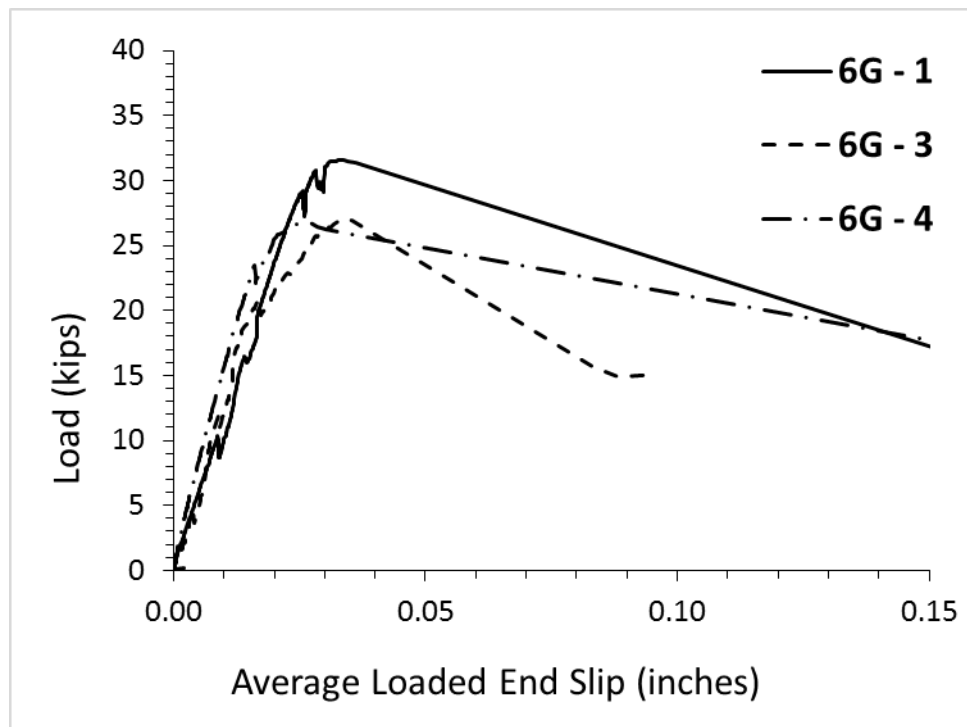


Figure 3-6. Load versus loaded end slip (#6 galvanized test bars)

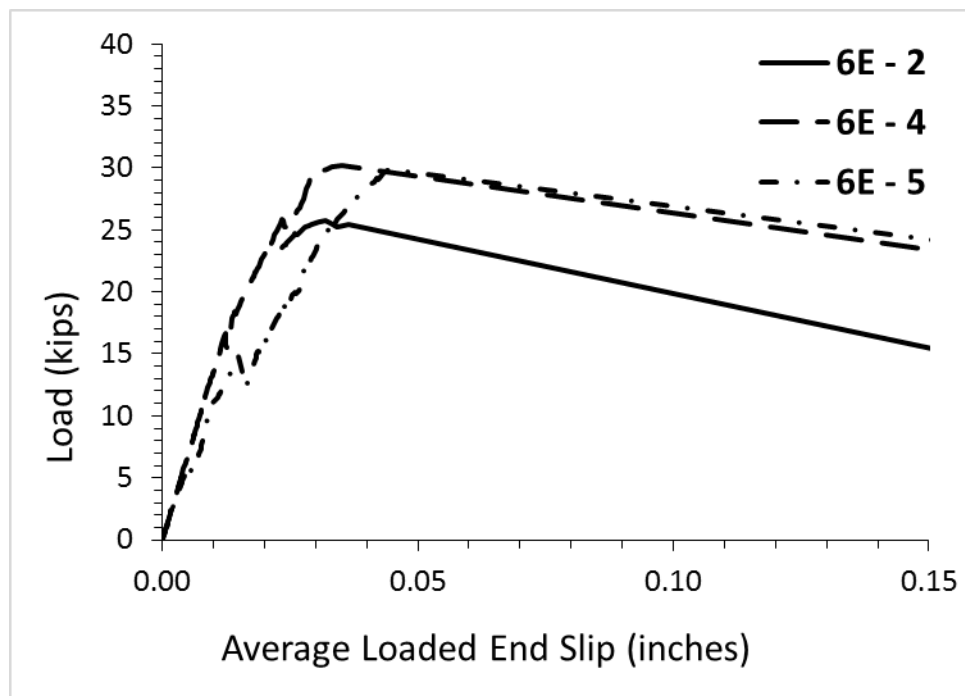


Figure 3-7. Load versus loaded end slip (#6 epoxy-coated test bars)

The average bond strength for the #8 epoxy-coated rebar was 28.8 kips with an average slip at the loaded and unloaded ends of 0.019 inches and 0.004 inches, respectively. The average bond strength of the #8 galvanized rebar was 29.5 kips with an average slip at the loaded and unloaded ends of 0.016 inches and 0.002 inches, respectively. The relationship between the loads at failure versus the average loaded end slip for the #8 galvanized bars and the #8 epoxy-coated bars are shown in Figures 3-8 and 3-9, respectively.

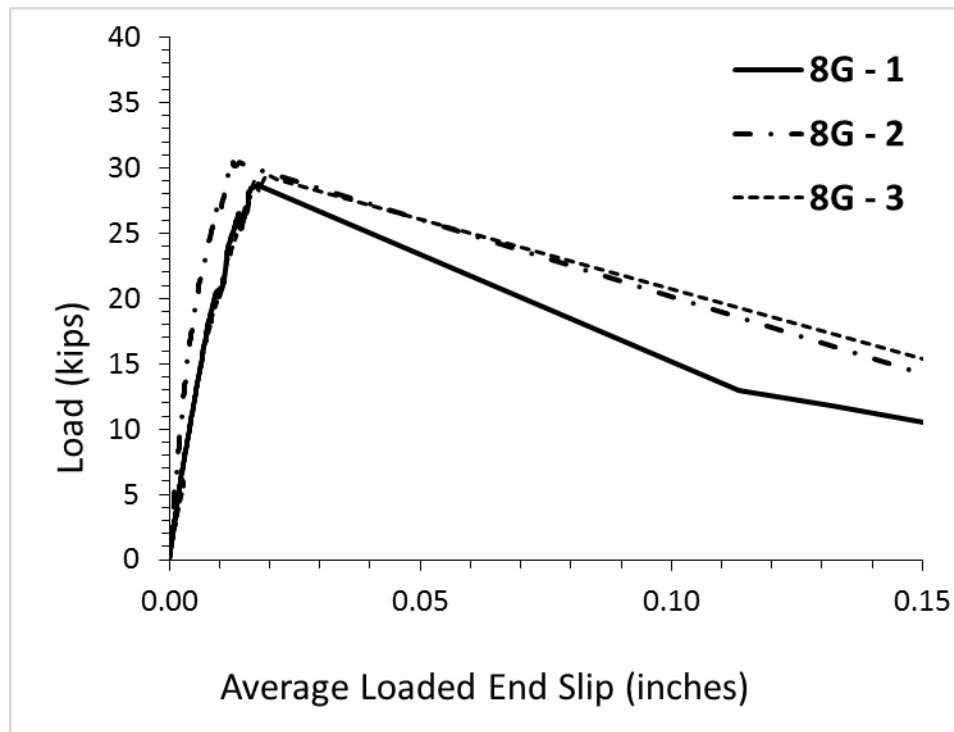


Figure 3-8. Load versus loaded end slip (#8 galvanized test bars)

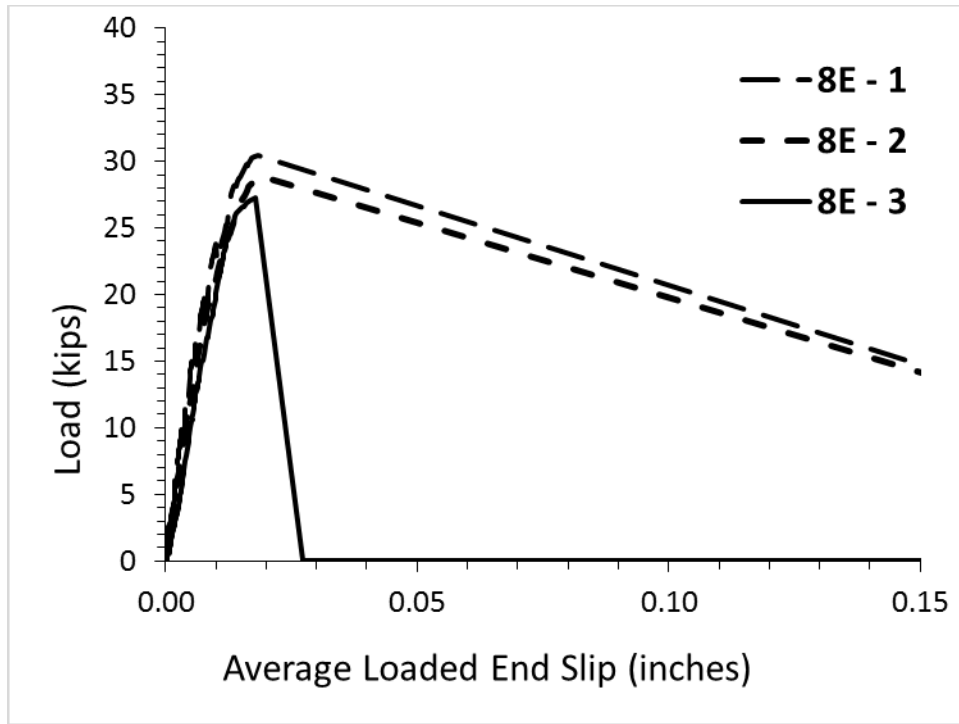


Figure 3-9. Load versus loaded end slip (#8 epoxy-coated test bars)

The average bond strength of #10 epoxy-coated rebar was 32.2 kips with an average slip at the loaded and unloaded ends of 0.014 inches and 0.004 inches, respectively. The average bond strength of the #10 galvanized rebar was 36.3 kips with an average slip at the loaded and unloaded ends of 0.017 inches and 0.008 inches, respectively. The relationship between the loads at failure versus the average loaded end slip for the #10 galvanized bars and the #10 epoxy-coated bars are shown in Figures 3-10 and 3-11, respectively.

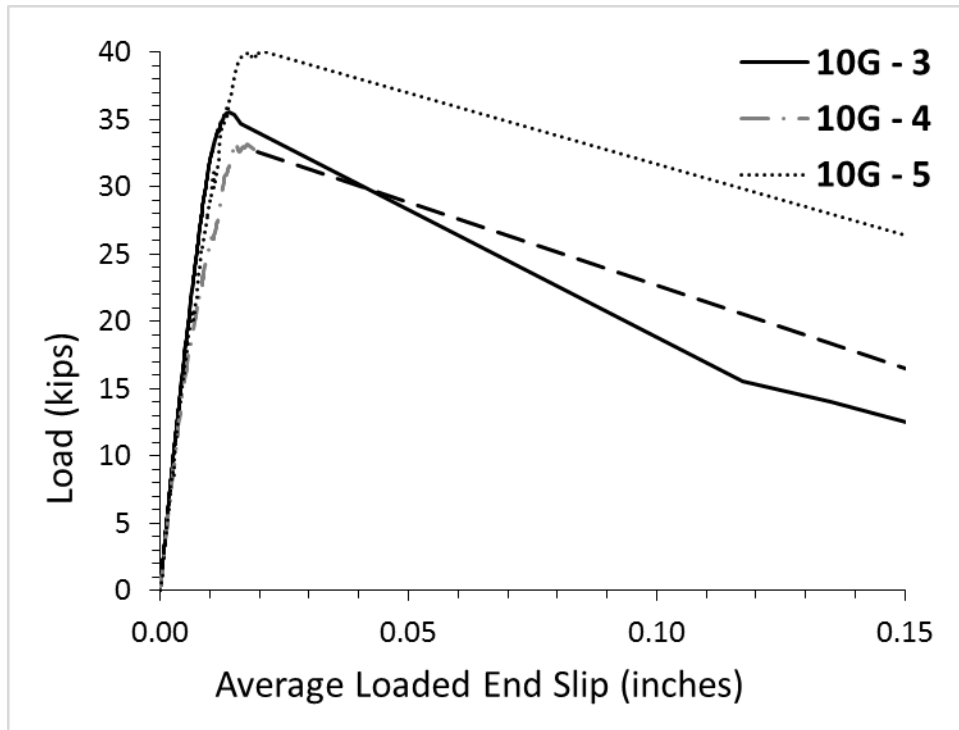


Figure 3-10. Load versus loaded end slip (#10 galvanized test bars)

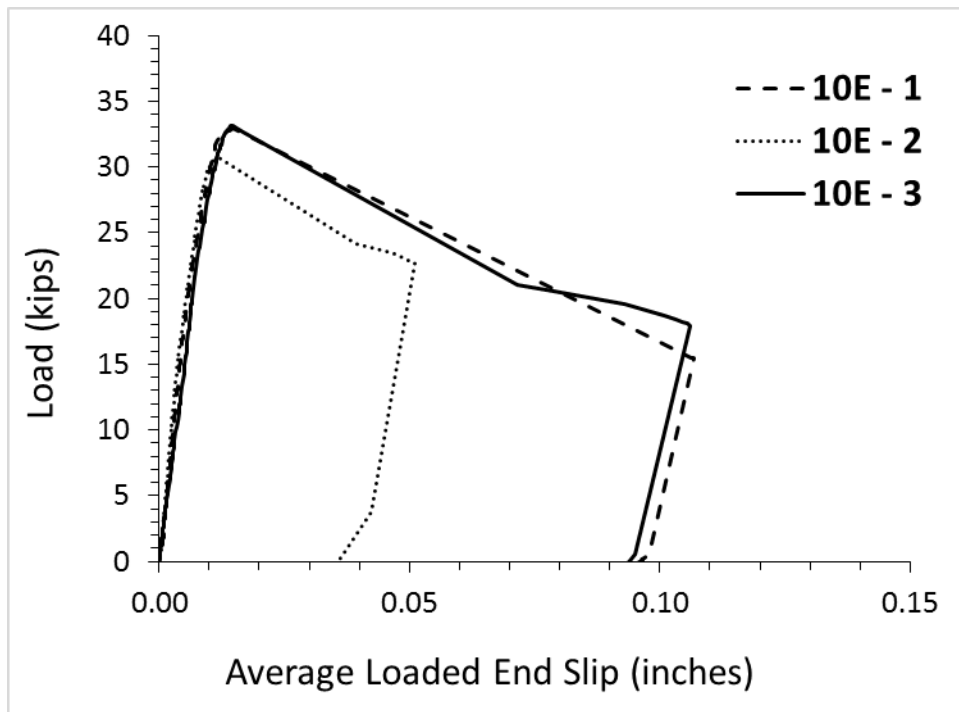


Figure 3-11. Load versus loaded end slip (#10 epoxy-coated test bars)

3.1.4. Discussion

From the results presented in Tables 3-1 and 3-2 and plots in Figures 3-6 through 3-11, the bond strength of galvanized reinforcing bars can be compared with that of epoxy-coated bars. In general, the galvanized reinforcing bars performed comparably to the epoxy-coated bars.

The bond strengths of the #8 and #10 galvanized bars were higher than those of the same-sized epoxy-coated bars, while the specimens with the #6 epoxy-coated bars showed a slightly higher bond strength than that of the #6 galvanized bars. In terms of percentages, #6, #8, and #10 galvanized bars had a failure load that was 0.37% less, 2.4% greater, and 12.7% greater than their epoxy-coated counter-parts, respectively. Note that the specimen 10G-5 seemed to have outperformed the other specimens with the same size of galvanized test bars. If this specimen is considered an outlier and is excluded from the data, the average failure load for the specimens with the #10 galvanized bars decreases to 34.462 kips, only 6.9% greater than their counterparts.

The average slip of the epoxy-coated bars decreased as the bar size increased as expected. For the specimens with galvanized steel, however, the minimum average slip was obtained from the specimens with #8 test bars while the specimens with #10 test bars had the greatest slip. This may be due to the specimen 10G-5, which had the slips at the loaded and unloaded ends of 0.0168 inches and 0.0083 inches, respectively, which are significantly larger than the other two specimens with the same test bar size.

As can be seen in Figures 3-6 through 3-11, the load-slip relation generally softens as the load reaches the maximum load, followed by reduction in tensile force associated with bond failure. Note that the force reductions of specimens 8E-3 and 10E-2 were more abrupt than those of others. It is speculated that this phenomenon was due to poor consolidation of the concrete along the top and front faces of these specimens (Figure 3-4c).

After testing was completed, the failed specimens were visually inspected. Figure 3-12 shows typical crack patterns of the specimens observed during the testing.



(a) Specimen with #8 epoxy-coated test bar



(b) Specimen with #8 galvanized test bar

Figure 3-12. Crack patterns of the specimens

In general, longitudinal cracks initiated around the loaded end and propagated to the top surface and then toward the unloaded end of the specimen along the test bar. This indicates a typical splitting mode of failure, which was observed, to various extents, in all specimens. In general, the width of the longitudinal crack increased as the applied load increased. In most cases, the failure was by pullout mode with relatively smaller radial cracks developing around the loaded end as the test bar was pulled from the specimen.

Finally, note that the time elapsed before failure for a few specimens (i.e., 8E-2, 8E-3, 8G-3, and 10G-4) were less than what was recommended by ASTM A 944. Therefore, the results from those specimens may need to be disregarded. However, it is not anticipated that the slightly shorter test times had a notable influence on the overall test results.

3.2. Field Monitoring

The field monitoring portion of this project was not intended to provide any immediate answers regarding the corrosion resistance of galvanized reinforcing steel. Rather, the intent of this portion of the project was to take advantage of a unique opportunity to monitor the corrosion resistance of galvanized reinforcing steel in this Buchanan County bridge.

In total, 10 galvanized bars in the bridge deck were instrumented with passive corrosion monitoring sensors. These sensors have been used by the Bridge Engineering Center for a number of years on a variety of projects where corrosion performance was particularly important.

The corrosion sensors installed in the bridge deck were strategically placed with two groups of five sensors located near the gutterline of the bridge. In all cases, the corrosion sensors were 10 feet long. Longitudinally, the first group of five sensors were placed three feet from the end of the bridge, starting one foot from the edge of the deck and placed at a spacing of approximately one foot. The second group of sensors started at a distance of 26.5 feet from the bridge end (extending over the 10-foot length). Given the bridge had a total length of 63 feet, the second group of sensors had the sensor mid-length aligned with the bridge mid-span.

As of the preparation of this report, as expected, no corrosion activity had been detected. The bridge will continue to be monitored for future corrosion activity.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1. Laboratory Testing

The laboratory testing program was carried out to evaluate and compare the bond strength of galvanized reinforcing steel to that of epoxy-coated bars. The evaluation process was based on the ASTM A 944 test protocols. To perform the beam end tests, 18 specimens—9 with galvanized test bars and 9 with epoxy-coated test bars—were constructed. The load at failure and the slip at the loaded and unloaded ends were noted and analyzed with the help of LVDTs.

The following conclusions were made based on the laboratory test results:

- The bond behavior of the galvanized reinforcing steel was similar to that of the conventional epoxy-coated bars. The difference in bond strength between them was not significant.
- In general, #8 and #10 galvanized reinforcing bars had an average bond strength that was greater than that of the same-sized epoxy-coated bars. Although this may indicate that galvanized steel could potentially be an adequate replacement for epoxy-coated bars, this may be disputed based on the test results on the slip at failure.
- The force reduction after reaching the peak load was abrupt for the specimens with poor concrete consolidation; whereas, it was gradual for other specimens.
- The during- and post-test observation revealed that the failure was by typical splitting and putout mode for most specimens.

4.2. Field Monitoring

A field monitoring program was successfully initiated that will allow the future corrosion performance of this Buchanan County bridge to be monitored. Additional monitoring will be conducted on an as-needed basis.

4.3. Recommendations and Future Research

While the results of this study could be used as a foundation for understanding the bond strength of galvanized steel reinforcement in concrete compared to that of conventional epoxy-coated steel bars, a further study with a larger pool of specimens is needed for producing more reliable results.

A more in-depth investigation regarding the bond properties may also be needed before the use of galvanized reinforcing steel is considered and incorporated into the Iowa DOT's current design codes. In such a study, the parameters or variables to be considered may include test bar location, embedment and/or splice length, amount of transverse reinforcing steel, size of concrete specimens, coating thickness, etc. In addition, large-scale flexural beam tests may allow for establishing deflection and cracking behavior of beams reinforced with galvanized reinforcing steel.

With regard to corrosion performance, it is recommended that an accelerated corrosion study be conducted. With such a study, the corrosion performance of galvanized reinforcing steel can be made in a matter of months rather than decades. Such studies have been completed successfully on other corrosion-resistant reinforcing steel.

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