CHROMX®

Material Properties And Design Considerations

PRODUCT INFORMATION

CHROMX® Series 9000, 4000 and 2000

Concrete Reinforcing Steel and Pavement Dowels Bars

Table of Contents

1. Product Description

A. Introduction

ChromX® Technologies' concrete reinforcing bars provide unique material properties of uncoated corrosion-resistant reinforcement (CRR) and high-strength. MMFX steel bars are sold as ChromX® Series 9000, 4000, and 2000 concrete reinforcing products. ChromX® material properties (corrosion-resistance and high-strength) result from both its chemical composition and manufacturing process. ChromX® series include two yield strength Grades with similar corrosion resistance for each of the series with minimum yield strengths of 100 (690) and 120 (830) as illustrated in Table 1. ChromX® Grade 120 [830] rebar series not currently addressed by any of the design codes and guidelines.

Table 1 ChromX® Series

ASTM A1035	Type CL	Type CM	Type CS
ChromX® Series	ChromX® 2000	ChromX® 4000	ChromX® 9000
UNS Designation	K23050	K42050	K81550
Grade 100 (690)	ChromX® 2100	ChromX® 4100	ChromX® 9100
Grade 120 (830)	ChromX® 2120	ChromX® 4120	ChromX® 9120

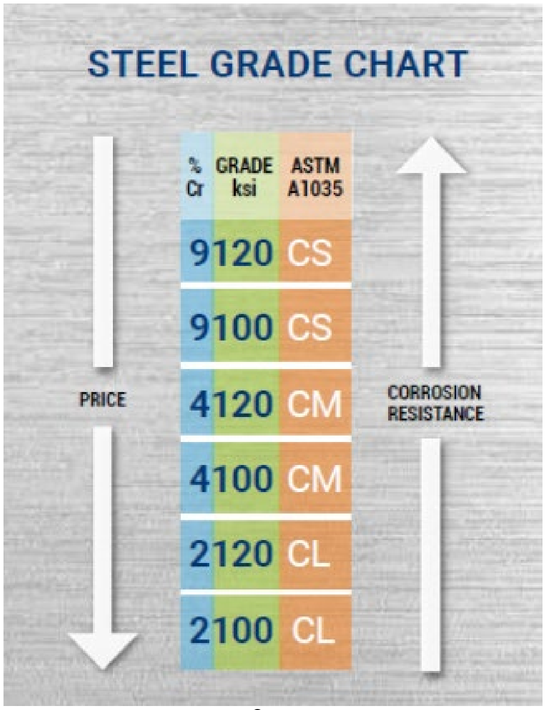


Figure 1 ChromX® Series Grade Chart

ChromX® 9100 and 9120 Concrete Reinforcing Steel Bars

ChromX® 9100 and 9120 (formerly known as $MMFX_2$) steel bars are uncoated, corrosion resistant reinforcing (CRR) and, high-strength steel bar products that meet or exceed the mechanical properties of ASTM A615 Grades 80 and 100, ASTM A1035/A1035M Type CS Grade 100 and 120 [690 and 830] (Ref 5.C.2), and AASHTO Standard Specification M334M/M334 Grade 100 (690), formerly published as AASHTO MP18M/MP18 (5.C.3).


Figure 2 ChromX® 9100

ChromX® 4100 and 4120 Concrete Reinforcing Bars

ChromX® 4100 Grade 100 [690] and 4120 Grade 120 [830] high-strength reinforcing steel and CRR bars are specified per ASTM A1035/A1035 M Type CM Grade 100 [690] and Grade 120 [830]. These bars have a moderate corrosion resistance in comparison to ChromX® 9000 series bars and are produced in a similar manner to ChromX® 9000 series bars with a lower chromium (Cr) content and higher carbon (C) content than the 9000 series bars.


Figure 3 ChromX® 4100

ChromX® 2100 and 2120 Concrete Reinforcing Bars

ChromX® 2100 Grade 100 [690] and 2120 Grade 120 [830] high-strength reinforcing steel bars are specified as ASTM A1035/A1035 M Type CL Grade 100 [690] and Grade 120 [830]. These bars have a lower corrosion resistance than either ChromX® 9000 or 4000 series bars and they are produced in a similar manner to ChromX® 9000 series bars with a lower chromium (Cr) content and higher carbon (C) content than either the 9000 or 4000 series bars.


Figure 4 ChromX® 2100

B. Material Composition

Conventional carbon steels have a microstructure of chemically dissimilar phases of carbide and ferrite. Carbides are strong, yet brittle and pinned to the grain boundaries. In a humid environment, a battery-like effect occurs between the carbides and the ferrite matrix that degrades steels from the inside out. This effect, (so called micro galvanic cell), is the primary corrosion initiator that drives the corrosion reaction. MMFX's patented proprietary steel technology forms a matrix of lath martensite and thin layers of retained austenite with substantially no-carbides at the grain boundaries.

ChromX® steel bars are low-carbon, chromium alloy steel that are produced as part of a controlled-rolling production process (i.e. rolling steel within a well-defined temperature range and cooled at a specific rate). The combination of chemical composition and manufacturing process produces a steel that has a completely different structure at the nanoscale (i.e. a laminated lath microstructure resembling "plywood" – See Figure 3). Steel made using MMFX nanotechnology minimizes the formation of micro galvanic cells (the driving force behind corrosion). ChromX® "plywood" effect provides superior strength, ductility, toughness, and corrosion resistance.

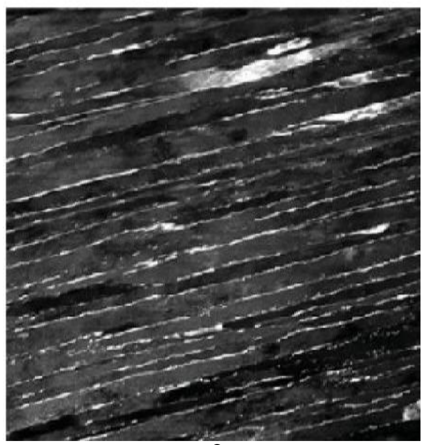


Figure 5 ChromX® Microstructure

Most steel exhibits strength at the cost of ductility (or brittleness). Steel that is made using MMFX's nanotechnology is not only stronger and tougher (not brittle), but is also more corrosion-resistant than conventional steel. This technology and material composition (Table 2) has enabled the development of high-strength, corrosion-resistance and cost-effective ChromX® steels.

Table 2 ChromX® Series Material Composition

(Maximum weight percentage except where noted *, **)

Material Composition	C%	Cr%	Mn%	Si%	S%	P%	N^2 ppm
ChromX® 9000/ASTM A1035 CS	0.15	8 to 10.9 *	1.5	0.50	0.045	0.035	500
AASHTO M334M/M334/ASTM A1035 CS	0.15	9.2**	2.0	N/A	N/A	N/A	2000
ChromX® 4000/ASTM A1035 CM	0.20	4.0 to 7.9 *	1.5	0.50	0.045	0.035	500
ChromX® 2000/ASTM A1035 CL	0.30	2.0 to 3.9 *	1.5	0.50	0.045	0.035	500

* Range as per ASTM A1035 CS, CM and CL

** Minimum Weight % as per AASHTO M334M/M334/ASTM A1035 CS

C. Products

ChromX® Series 9000, 4000, and 2000 bars are available in the North America, South America and Middle East markets with the following concrete reinforcing steel products conforming to ASTM A1035/A1035M Grade 100 [690] and AASHTO M334M/M334 Grade 100 [690]:

North American Market

- #3 through #11, #14 & #18 standard bar sizes
- Standard 60 ft. straight-length-bundle quantities #4 through #11, #14 & #18 standard bar sizes
- Standard 40 ft. straight-length-bundle quantities or coils #3 and #4 standard bar sizes
- Custom-mill-cut lengths up to 72 feet available (80 feet max. - #11, #14 and #18) by special order in 50 ton or greater increments and minimum 20 feet length
- Smooth bar material (i.e. pavement dowels) available in 0.75 to 2.5 inch rounds

Middle East and South American Markets

- 12, 14, 16, 18, 20, 25, 32 mm standard bar sizes
- Available in standard 12-meter length-bundle quantities
- 36 and 40mm bar sizes - special order
- Custom-mill-cut lengths available by special order only in 50 tons or greater increments
- Smooth bar material available in 25 to 40 mm diameters

Mechanical splices (couplers) and anchorages (headed bars)

- High strength and corrosion resistant couplers and headed bar are available from major coupler manufacturers. Please refer to Annex E for Splicing Solutions and The *Mechanical Couplers Made for Use with the ChromX® ASTM A1035/A1035M Grade 100 Steel Reinforcing Bars* for the various types and styles of couplers that are available from the various coupler manufacturers.

2. Material Properties – ChromX® Solution to the Problem

A. The Corrosion Problem

Corrosion of steel is initiated when chlorides penetrate the concrete, which is porous, and reach the steel in sufficient concentration to initiate corrosion. This concentration is known as the Chloride Threshold (CT) of the steel. When the CT is reached, the steel begins to corrode and propagates, expanding the steel and causing the concrete to crack and spall.

Corrosion of the reinforcing steel is the major cause of deterioration of concrete structures. The United States infrastructure is suffering significantly from the corrosion of its bridges, roadways, ports and seawalls. For example, over 56,000 U.S. bridges, approximately 9.1% of the country's total bridges, are deemed structurally deficient or functionally obsolete in 2016. The infrastructure in Chile located in either coastal areas (corrosive marine environment) or in the northern desert regions (highly corrosive soils) face similar corrosion challenges. The Middle East Region is considered to be the most sever environment for reinforcing steels.

B. ChromX® Bars: A Solution to the Corrosion Problem

The corrosion resistance of ChromX® rebar is derived from the unique microstructure of the steel that is fundamentally different than standard carbon steels. Standard carbon steels consist of ferrites and carbides. When exposed to a chloride solution, the ferrites (anodes) and carbides (cathodes) establish a chemical reaction called a micro-galvanic cell that causes the breakdown of the carbides, resulting in the expansion of the steel, i.e. the corrosion of the steel.

In contrast to standard carbon steels, the ChromX® microstructure consists of dislocated laths of Martensite separated by thin layers of retained Austenite, and is relatively free of carbides. This martensitic, austenitic microstructure and the minimizing of carbides in ChromX® inhibit the formation of micro-galvanic cells. Figure 6 represents a diagram of the ChromX® microstructure.

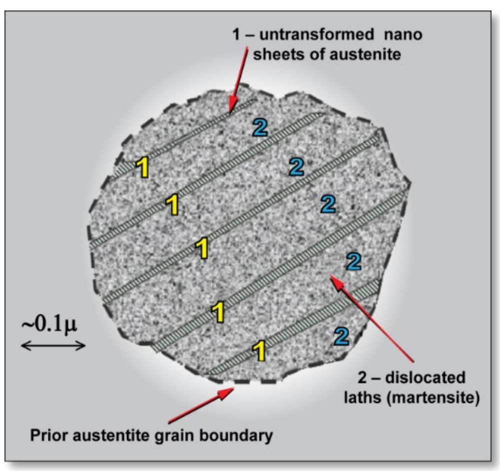


Figure 6 ChromX® Steel Microstructure

The corrosion resistance offered by this unique microstructure is twofold: 1) ChromX® 9000 steel has a CT value of four times that of standard carbon steel, the ChromX® 4000 steel has a CT value of two times that of standard carbon steel, while ChromX® 2000 series has a CT value of one and half times of standard carbon steel, which means it takes longer for corrosion to initiate; and 2) the ChromX® steel corrodes at one-third the rate of standard carbon steel, which further delays the spalling of the concrete.

The corrosion resistance of ChromX® rebar has been studied, validated and measured by numerous universities, government agencies and independent testing facilities. A small selection of these studies are listed under *References* below. ChromX® rebar has been designated a Corrosion Resistant Rebar (CRR) by several state departments of transportation, ministries of transportation, and industry organizations that use the steel to extend the lives of their structures.

ChromX® 9000 Series

ChromX® 9100 (ASTM A1035 Type CS Grade 100 [690] and AASHTO M 334M/M334 Grade 100 [690]) rebar's corrosion resistance, in terms of its critical chloride threshold level (CT - the quantity of chloride in concrete that initiates corrosion), has been demonstrated to be more than four times the CT of ASTM A615 conventional carbon steel bar (2350 vs 550 ppm).

ChromX® 4000 Series

ChromX® 4100 (ASTM A1035 Type CM Grade 100 [690] rebar's corrosion resistance, in terms of its chloride threshold level (CT – the quantity of chloride in concrete that initiates corrosion), has been demonstrated to be more than two times the CT of ASTM A615 conventional carbon steel bar (1150 vs 550 ppm).

ChromX® 2000 Series

ChromX® 2100 (ASTM A1035 Type CL Grade 100 [690] rebar's corrosion resistance, in terms of its chloride threshold level (CT – the quantity of chloride in concrete that initiates corrosion), has been demonstrated to be more than one and half times that CT of ASTM A615 conventional carbon steel bar (850 vs 550 ppm).

ChromX® 9000 (ASTM A1035 Type CS Grade 100 [690] and AASHTO M334M/M334 Grade 100 [690]) and ChromX® 4000 (ASTM A1035 Type CM Grade 100 [690]) steel's corrosion performance has been demonstrated by various test methods at different universities and DOT laboratories, as illustrated under the reference section.

Table 3 summarizes the Chloride Threshold and Estimated Propagation Time for different types of reinforcing bars as referenced by TGC "Reinforcing Steel Comparative Durability Assessment for 100-Years' Service Life" (Ref. 5.A.1). The chloride threshold values of the different competing products are illustrated in Figure 7. A series of tests have been conducted by adding 2 gallons per cubic yard of Calcium Nitrite to the concrete mix. The results of adding the Calcium Nitrite improves the chloride threshold levels of the ChromX® series as illustrated in Figures 8 and 9.

Table 3 Chloride Threshold and Propagation Time

Reinforcing Bar Type	Chloride Threshold, PPM[1]	Estimated Chloride Threshold, ppm with 2 Gpy of CNI	Estimated Propagation Time, Years
Conventional BB (A615)	500		5 - 7
Epoxy Coated Bars (A775)[2]	500		10 - 15
Epoxy Coated Bars (A775) 2 layers[2]	900		10 - 15
Galvanized Bars (A767)[3]	1500		15 - 20
ChromX® 9000 (ASTM A1035 CS)	2000	2400	15 - 20
ChromX® 4000 (ASTM A1035 CM)	1600	2200	15 - 20
ChromX® 2000 (ASTM A1035 CL)	850	1875	15 - 20
UNS S32304 (ASTM A955)	3750		>50

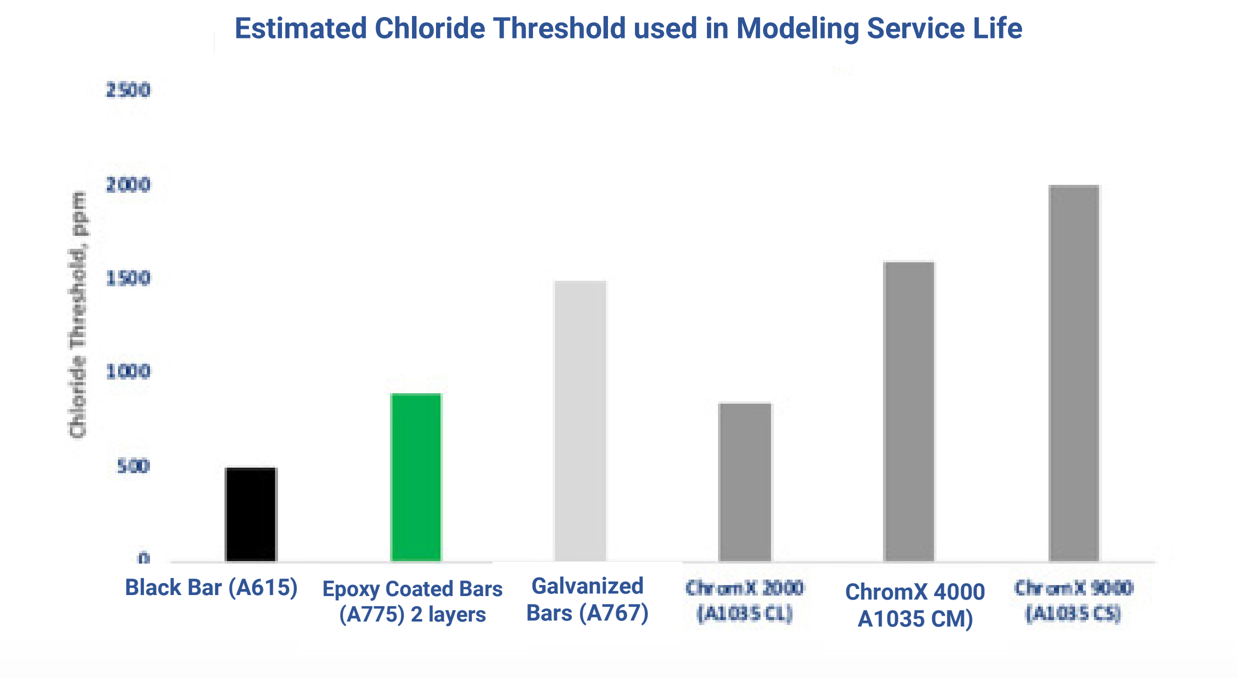


Figure 7 Comparison of Chloride Threshold values of the Different Competing Bars

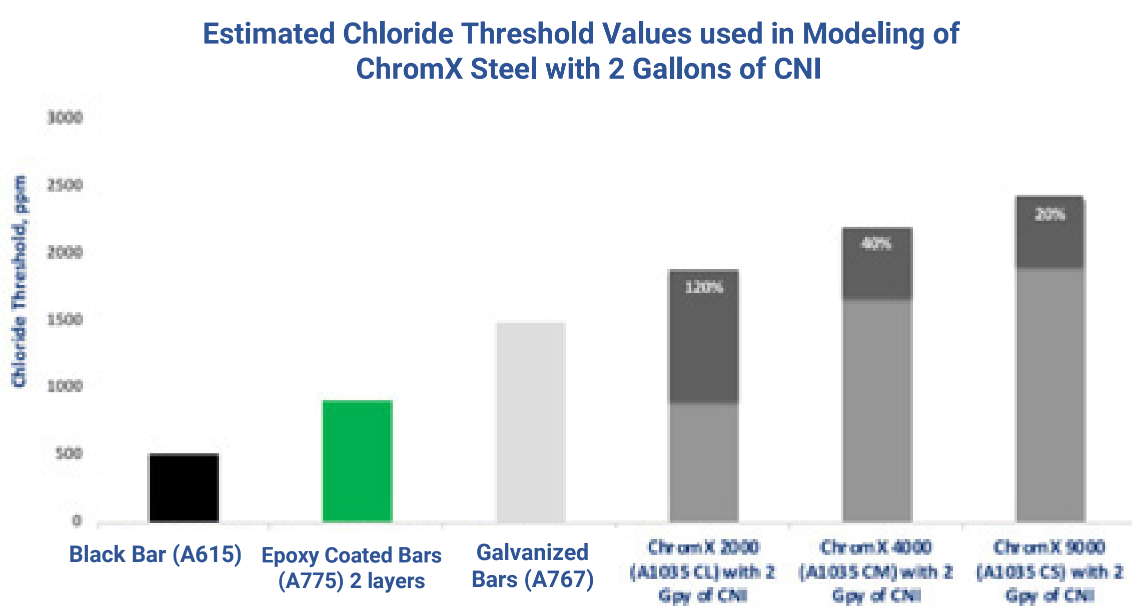


Figure 8 Chloride Threshold Comparison of Competing Products When 2 GPY of Calcium Nitrite is added to Concrete Mix

ChromX 9000 with and without CNI vs Stainless Steel Chloride Threshold

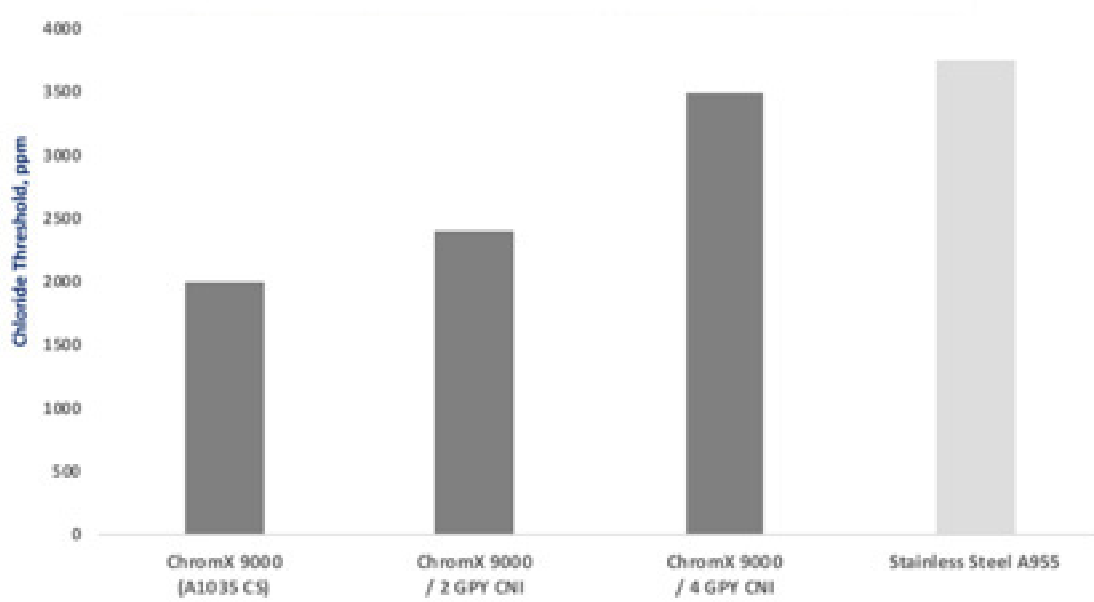


Figure 9 ChromX® 9000 with and without CNI vs Stainless Steel Chloride Threshold

[1] Except as specifically noted, CT data is from Tourney Consulting Group, LLC, "*Performance Modeling of the Corrosion Performance of ChromX® Reinforcing Bars in a Bridge Deck in a Southern Marine Environment*" (11/29/2016). ChromX® 2000 CT set at the bottom of its probability range per the standard deviation.

[2] ECR CT set at black bar's CT. Perfectly applied epoxy-coating performs well in laboratory tests, but field studies prove that the coating does not survive field handling and installation, and therefore provides little to no protection.

[3] Darwin, David et al, "*Critical Chloride Corrosion Threshold for Galvanized Reinforcing Bar*", The University of Kansas Center for Research, Inc. (Dec 2007).

C. Material Service Life

An assessment of ChromX® Steel conforming to ASTM A1035 Type CM and CL and ASTM A1035 Type CS conforming to AASHTO M334M/M334 Grade 100 [690] rebar's corrosion resistance performance is made through the measure of its service life and/or critical chloride threshold level (CT), as noted in 5.A *"Corrosion Test Reports."* The references for service life studies are listed below:

- Berke (Ref 5.A.1) reported that A1035 Type CS and CM with CNI (Calcium Nitrate) bars exceeded a 100-year service life in the study's comparative analysis of various CRR materials in bridge decks, marine and soil piles structures, located within various environmental settings
- Berke (Ref 5.A.2) reported that A1035 Type CS and ASTM A1035 Type CM with CNI (Calcium Nitrate) can provide 100-year service life in bridges.
- Williamson and Weyers (Ref. 5.A.18) indicated that A1035 Type CS, when used in conjunction with LCP (low-permeability concrete), provides greater than 200-year service life.
- Cui and Krause (Ref. 5.A.21) noted that A1035 Type CS would provide about 2 times the service life and had approximately 3 times the chloride threshold of carbon steel.
- Darwin and Browning (Ref. 5.A.23) said that corrosion initiation in un-cracked bridge decks would take about 3.8 times longer than black steel; and 2.4 times longer than galvanized (zinc coated) rebar. This study replicated some of the findings of Pianca and Schell (Ref. 5.A.34) concerning galvanized (zinc coated) rebar.
- LaNier and Springston (Ref. 5.A.33) states that on the US Navy's modular pier project, A1035 Type CS saved about $2.8 million over the original proposed design, while providing a 75-yr service life.
- Clemeña and Virmani (Ref. 5.A.36) reports that A1035's chloride threshold is about 5 to 6 times that of black bar and 2 times that of 2101 LDX bars.

These studies consistently demonstrate ChromX® steel ability to provide long-term corrosion performance and added value to structures in corrosive environments.

The Service Life of a reinforced concrete structure goes through three corrosion related stages. Figure 9 illustrates a schematic graphic comparing ChromX® steel's corrosion protection in relation to conventional black steel, where Stage 1 is time to corrosion initiation, Stage 2 is time for corrosion propagation, and Stage 3 is time through expected repairs until the structure's end of economic service life.

1. Stage one (corrosion initiation) - time for chlorides to penetrate into the concrete to the level of the steel, reach the CT and initiate corrosion.
2. Stage two (propagation) - time from initiation of corrosion to initiation of first concrete spalls.
3. Stage three (repair) - time during the service life interval requiring repairs.

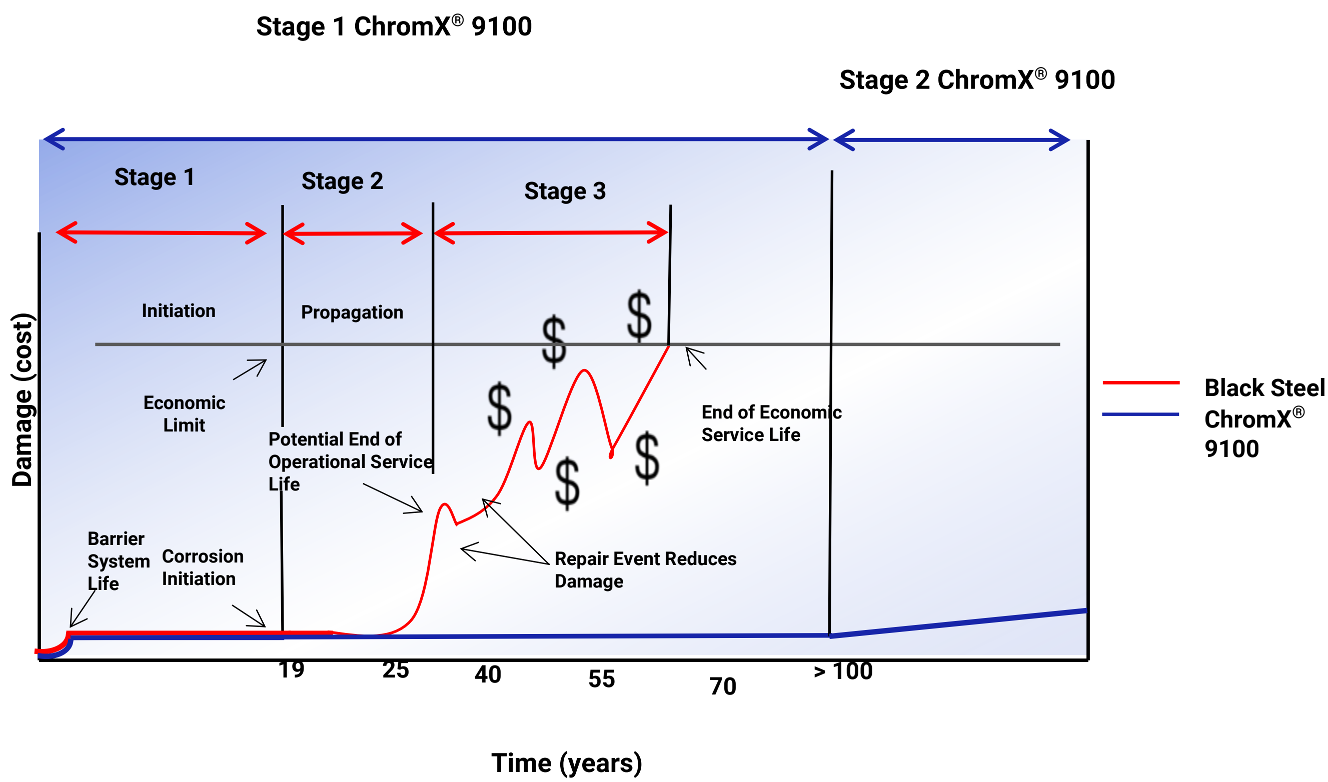


Figure 10 - Schematic Graphic Illustration Comparing ChromX® steel's corrosion protection in relation to conventional black steel.

D. Predictive Service Life Modeling Using STADIUM Model

Durability modeling combines materials properties (chloride threshold and propagation time) and computer simulations to optimize the reinforced concrete system to achieve a service life goal. Typically, this is an iterative process. Each concrete element is evaluated separately based on the exposure conditions with the goal of achieving the required service life. Tourney Consulting Group have analyzed three case studies using the STADIUM model of a) Bridge Deck in Indiana/Kentucky, b) Marine Pile and c) Soil Pile in GCC Ref. 5.A.1)

Table 4 - Case Study Corrosion Service Life for Various Reinforcing Steels in Bridge Decks in Indiana/Kentucky

Concrete Type	Bridge LP	
Exposure Conditions	Deicing Salts	
Cover (min.)	1.5 Inches	
Reinforcement Type	Initiation, Years	Repair, Years
Conventional BB (ASTM A615)	19	25
Epoxy Coated Bars (ASTM A775	19	34
Galvanized Bars (ASTM A767)	59	76
ChromX® 9000 (ASTM A1035 CS)	>100	>100
ChromX® 4000 (ASTM A1035 CM)	43	61
ChromX® 4000 (ASTM A1035) with 2 GPY Calcium Nitrite	>100	>100
UNS S32304 (ASTM A955)	>100	>100

Table 5 - Case Study Corrosion Service Life for Various Reinforcing Steels- Low Permeability Marine Piles

Concrete Type	Pile Mix LP	
Exposure	Marine Environment	
Cover (min.)	2.0	2.5
Reinforcement Type	Service Life Estimate (years)	
Conventional BB (ASTM A615)	26	37
Epoxy Coated Bars (ASTM A775	35	46
Galvanized Bars (ASTM A767)	64	89
ChromX® 9000 (ASTM A1035 CS)	>100	>100
ChromX® 4000 (ASTM A1035 CM)	56	76
ChromX® 4000 (ASTM A1035) with 2 GPY Calcium Nitrite	94	>100
UNS S32304 (ASTM A955)	>100	>100

Table 6 - Case Study Corrosion Service Life for Various Reinforcing Steels- GCC Conditions Soil Pile

Concrete Type	Soil Pile Mix LP		
Exposure	Soil in GCC		
Cover (min.)	37.5 mm	50 mm	62.5 mm
Reinforcement Type	Estimated Service Life to Initiation (years)		
Conventional BB (ASTM A615)	18	32	48
Epoxy Coated Bars (ASTM A775	18	32	48
Galvanized Bars (ASTM A767)	57	96	>100
ChromX® 9000 (ASTM A1035 CS)	>100	>100	>100
ChromX® 4000 (ASTM A1035 CM)	42	71	95
ChromX® 4000 (ASTM A1035) with 2 GPY Calcium Nitrite	98	>100	>100
UNS S32304 (ASTM A955)	>100	>100	>100

E. Applicability of the ChromX® Steel bars in Corrosive Environments:

Taking into consideration of all the available literature on the chloride threshold and corrosion rate of the different ChromX® product series and the estimated service life for initiation and repairs, the following table summarizes the service life and replacement strategy for alternative reinforcements:

Table 7 - The ChromX® Series Choice Replacing Alternate Reinforcement for the Specific Application and Service Life Requirements

Applications	Required Service Life (Years)	ChromX® Series* Choice for Replacing Alternate Reinforcement	Alternate Reinforcement replaced by ChromX®
Bridges	100	• ChromX® 9000 • ChromX® 4100 with 2 gpy CNI	Stainless Steel
Bridges	75	• ChromX® 9000 • ChromX® 4100	Epoxy Coated Steel Galvanized Steel
Marine	100	• ChromX® 9000	Stainless Steel
Marine	75	• ChromX® 9000 • ChromX® 4000 with 2 gpy CNI	Stainless Steel
Marine	50	• ChromX® 4000	Epoxy Coated Steel Galvanized Steel

* Lowest cost for the service life required for the application

F. High-Strength Properties

ChromX® Steels conform to the mechanical properties outlined by the ASTM A1035 specifications as illustrated in Table 8.

Table 8 – ChromX® Mechanical Tensile Test Properties

ASTM A1035 Types CS, CM and CL	ChromX® 9100 ChromX® 4100 ChromX® 2100	ChromX® 9120 ChromX® 4120 ChromX® 2120
Tensile strength, min, psi [MPa]	150,000 [1030]	150,000 [1030]
Yield strength (0.2% offset, min, psi [MPa]	100,000 [690]	120,000 [830]
Elongation in 8 in. [200 mm], min.%; Bar Designation No. 3 through 11 [12mm through 36mm]	7	7
Bar Designation No. 14,18, [40mm]	6	6

ChromX® 9100, ChromX® 4100 and ChromX® 2100 rebar are appropriate for use as concrete reinforcement in building, industrial, transportation and other reinforced concrete applications. These bars provide added value for structures designed utilizing a yield strength of 100 ksi (690 MPa), providing up to 40% savings in material, fabrication and installation for structural applications, such as buildings and bridges.

ChromX® 9100 (ASTM A1035 Type CS, AASHTO M 334M/M334, Grade 100 [690]) and ChromX® 4100 (ASTM A1035 Type CM Grade 100 [690]) bars have been used in building (piles, foundations, slabs, beams, columns), bridge (decks, girders, columns, abutments), retaining walls, marine facilities (docks, piers, fenders, etc.), pavement (dowel bars and lane tie bars) and other related cast-in-place and precast reinforced concrete members. Common application of the product is structures that can benefit from both the high-strength and corrosion resistance of ChromX® 9100 and ChromX® 4100.

ChromX® 2100 (ASTM A1035 Type CL Grade 100 [690]) bars are being used in reinforced concrete structures which do not require the same level of corrosion protection as ChromX® 9100 bars but include the benefits of the high yield strength 100 ksi (690 MPa).

ChromX® Series 9000, 4000 and 2000 steel rebar meet or exceed the mechanical requirements of ASTM A615 Grades 80 and 100 [550 and 690], ASTM A1035 Grade 100 and Grade 120, and AASHTO M334M/M334 /ASTM Type CS Grade 100. Figure 10 (Page 11) compares the mechanical properties and the stress vs. strain characteristics of the ChromX® Series conforming to ASTM A1035 vs. ASTM A615

Table 9 – Comparison of Mechanical Properties of ChromX® Steel (A1035) vs. A615

Mechanical Properties	**ASTM A615 Grade 60**	**ASTM A615 Grade 80**	**ASTM A615 Grade 100**	**ASTM A1035 Grade 100**
Min. Yield Strength, Ksi [MPa]	60 [420]	80 [550]	100 [690]	**100 [690]**
Min. Tensile Strength, Ksi [MPa]	90 [620]	100 [690]	115 [790]	**150 [1030]**
Min. Elongation, %	9	7	7	**7**
Min. Tensile/Yield (T/Y Ratio)	-	-	1.15	**1.25**

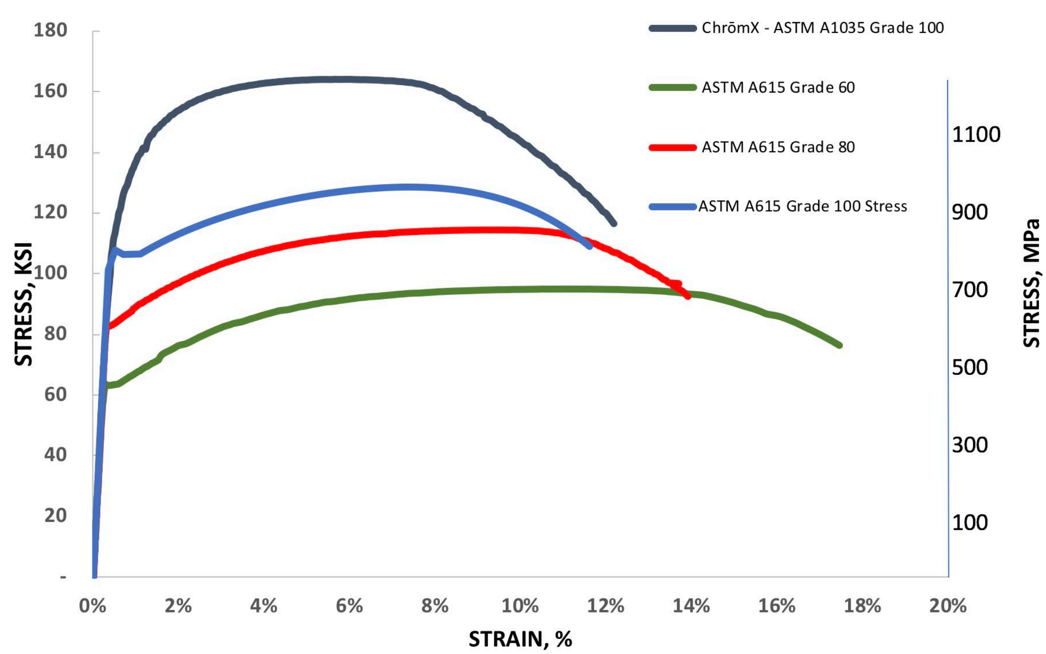


Figure 11 – Stress vs Strain Plot of ASTM A1035 Grade 100 and ASTM A615 Grade 60, 80 and 100

3. Design Considerations

Design requirements for reinforcing steel have increased over time, particularly in concrete columns, shear walls and piles. Reinforcing steel required for these designs may either exceed the practical capacity of conventional steel bars, or result in significant steel congestion, as seen in Figure 11. As a result, reinforcing bar placement and concrete consolidation has become difficult in these reinforced concrete structures.

Figure 12 – Typical Steel Congestions

A. ChromX® Steel Code Compliance

Design using ChromX® steel conforming to ASTM A1035/A1035M Grade 100 (690) can be achieved using the ACI 318-19 Building Code for all gravity loads using f_y = 100,000 psi. However, using prior versions of the ACI 318 code, designers can utilize the ChromX® Steel in accordance to the International Building Code® (IBC), the International Code Council Engineering Services, ICC-AC 429 and ICC ESR 2107 (ref 5.C.8), that allows the use of high-strength ASTM A1035 Grade 100 [690] reinforcing steel, including in high seismic zones when designing according to ACI 318-08, ACI 318-11 and ACI 318-14. AASHTO LRFD Bridge Construction Specifications have also incorporated guidelines for using high-strength reinforcing steel up to 100 ksi [690 MPa] yield strengths. The following sections summarize the incorporation of ASTM A1035 Grade 100 [690] reinforcement into the ACI 318-19, IBC and AASHTO LRFD in all seismic zones with some limitations in Seismic Zones D, E and F.

- **ACI Standards ACI 318-19**

The basis of acceptance and design of ChromX® 9100, 4100 and 2100 reinforcing bars conforming to ASTM A1035/A1035M Type CS, CM and CL with design strength up to 100,000 psi (690 MPa) is permitted for all gravity loads as per the ACI 318-19 building code as defined by Chapter 20.

- **ACI Standards (ACI 318-08, ACI 318-11 and ACI 318-14) and IBC 2012 and IBC 2015.**

The basis of acceptance and design of ChromX® reinforcing bars conforming to ASTM A1035 Type CS, CM and CL with design yield strength up to 100 Ksi [690 MPa] as per ACI 318 (2011 and 2014) and IBC 2012 and 2015 are presented in the following sections.

The 2014 edition of the American Concrete Institute's Standard ACI 318-14 lists ASTM A1035 in section 20.2.1.3 (e) and allows the use of deformed reinforcing bars conforming to ASTM A1035 as lateral support of longitudinal bars or for concrete confinement in spiral and special seismic system applications as per Section 20.2.2.4 in accordance to Table 20.2.2.4a.

In the absence of code language on general use of reinforcement yield strength of 100 ksi [690 MPa] in the ACI 318 standard, the IBC provides a mechanism for the use of new acceptable design criteria under Section 104.11:

IBC Section 104.11 "*Alternative materials, design and methods of construction and equipment*"

> "The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided any such alternative has been *approved*. An alternative material, design or method of construction shall be *approved* where the *building official* finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety."

Design guidelines and criteria have been developed and issued, which allow for the use of ASTM A1035 reinforcement up to 100 ksi [690 MPa] yield strength in accordance to IBC 104.11.

- **International Code Council Evaluation Service (ICC-ES) Acceptance Criteria, ICC-ES AC429**

The Acceptance Criteria, AC429, was issued by ICC Evaluation Service, LLC (ICC-ES) based upon Section 104.11 of the IBC. This acceptance criteria has been issued to provide interested parties with guidelines for demonstrating compliance with performance features of the codes referenced in the criteria.

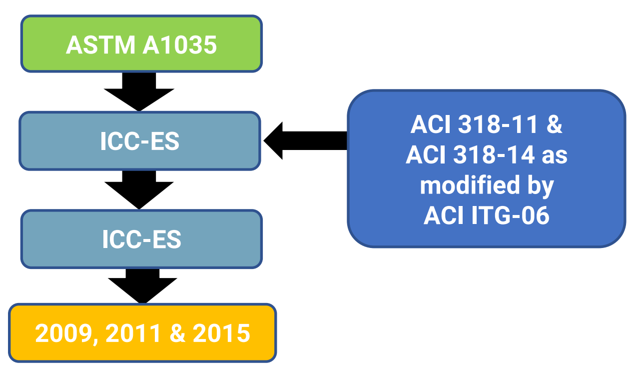
> **ICC-ES AC429 Section 1.2.5** states: "This criteria is applicable to reinforcement under Sections 8.6.2.3, 20.2.1.2, 20.2.1.3, and 20.2.2.4 of ACI 318-14, and Section 4.1.2 of ACI 318.2-14 under the 2015 IBC (Sections 3.5.3.2, 3.5.3.3, 9.4, 11.4.2, 11.5.3.4, 11.6.6, 18.9.3.2, and 19.3.2 of ACI 218-11 or -08 under the 2012 and 2009 IBC). ACI 318 is referenced in Section 1901.2 of the IBC".

> **ICC-ES AC429 Section 1.2.7** states: "For structures assigned to Seismic Design Categories D, E, or F, ASTM A1035 Grade 100 [690] reinforcing bars are limited to placement in slab systems, foundations, and structural components not designed as part of the seismic force-resisting system but explicitly analyzed for induced effects of design displacements in accordance with ACI 318-14 section 18.14 under the 2015 IBC (ACI 318-11 or -08 section 21.13 under the 2012 or 2009 IBC, as applicable)".

ICC-ES AC429 incorporates the findings and design philosophies of the ACI ITG-06 Report described below. The criteria was developed through a transparent process involving public hearings of the ICC-ES Evaluation Committee, and/or on-line postings where public comments were solicited.

- **American Concrete Institute (ACI) ITG-06 Report**

In order to fulfill the requirements of the IBC and ACI 318, ChromX® reinforcing bars meeting ASTM A1035 have been investigated by a number of research institutes and government agencies through major research programs over the past fifteen years. The results of these research programs have been published by journals and publications throughout the engineering community to illustrate the advantages of designing at high yield strengths and the design philosophy behind the same.



ACI put together an Innovative Task Group (ITG) composed of past ACI presidents and leaders in the field of concrete design to formulate a guide on designing concrete structures using ASTM A1035 grade 100 [690] reinforcement. The group reviewed all the published research reports and journals on material characteristics and physical testing of ASTM A1035 reinforcing bars as well as structural components reinforced with ASTM A1035 reinforcing bars in light of the ACI 318 provisions. Applicable code provisions were modified where such modifications were warranted in view of the research findings. The ITG's work including recommendations was published by the American Concrete Institute in the ACI ITG-06 Report as a first step toward general acceptance of the use of ASTM A1035 Grade 100 [690] reinforcement in the design of concrete structures.

- **ICC-ES Evaluation Report ESR-2107 (ref 5.C.8):**

ICC-ES also issued ESR-2107 based on ICC-ES AC429 relating specifically to ChromX® ASTM A1035 bars. This evaluation report confirms the acceptance of ChromX® reinforcing bars conforming to ASTM A1035 pursuant to ICC-ES AC429. While the current ESR 2107 evaluation report addresses the compliance with the 2009 and 2012 IBC, a newer version is underway to include the compliance with the 2015 IBC.

> **ICC-ES ESR-2107 Section 4.1 "*Design*"**
>
> The bars must be designed in accordance with IBC, ACI 318-11 under 2012 IBC (ACI 318-08 under the 2009 IBC), and ACI ITG-6R, as summarized in Annex 1 of this report, entitled, "Design Methodology, ASTM A 1035/A 1035M Grade 100 [690 MPa] Bars."
>
> (6) For structures assigned to Seismic Design Category (SDC) D, E, or F, ASTM A1035 Grade 100 reinforcing bars are limited to placement in slab systems, foundations, and structural components not designated as part of the seismic force–resisting system but explicitly analyzed for induced effects of design displacements in accordance with ACI 318 Section 21.13. Additional requirements pertaining to structures assigned to SDC D, E, or F are described in Section A.2.20 of this report.

- **Use of ASTM A1035 in Compliance with the 2015 IBC Summarized**

ASTM A1035 Grade 100 [690] reinforcement can be used in full compliance with the IBC in concrete structures assigned to Seismic Design Category A, B or C, on the basis of IBC Section 104.11 and ICC-ES ESR-2107. The ESR is based on ICC-ES AC429, which requires design by ACI 318, as modified by ACI ITG-06.

For structures assigned to Seismic Design Category D, E, or F, ASTM A1035 Grade 100 [690] reinforcing bars are limited to placement in slab systems, foundations, and structural components not designated as part of the seismic force–resisting system but explicitly analyzed for induced effects of design displacements in accordance with ACI 318-14 Section 18.14 under the 2015 IBC (ACI 318-11 or -08 Section 21.13 under the 2012 or 2009 IBC, as applicable).

B. Flexural Design

- **Design Methodology for Building Systems Using ACI 318-19 Building Code:**

Flexural design of concrete members reinforced with ChromX® 9100, 4100 and 2100 conforming to ASTM A1035/A1035M Type CS, CM and CL Grade 100 (690) should be followed as permitted in accordance to the ACI 318-19 building code for all gravity loads in flexure, axial loads, and shrinkage and temperature reinforcement.

The ACI 318-19 Building code introduced the following changes when using Grade 100 bars reinforcement:

- Introduce a new definition for tension- and compression-controlled limit for Grade 100 bars. The tension-controlled strain limit is set as "$\varepsilon_t \geq \varepsilon_{ty} + 0.003$" and the compression-controlled strain is set as "$\varepsilon_t \leq \varepsilon_{ty}$".
- Introduce modification factors for Grade 100 bars development length provisions

- Introduce modification factors for Grade 100 hooked and headed bar development length provisions Development and Lap Splice length shall be determined in accordance with ACI 318-19 as illustrated in Annex A.

- **Simplified Design Methodology for Building Systems Using ACI 318-08, ACI 318-11 and ACI 318-14 As Modified in Accordance to ACI ITG 06R and Presented in ICC AC 429 and ESR 2107:**

Flexural design of concrete members reinforced with ChromX® bars (ASTM A1035 Type CS, CM and CL Grade 100 [690]) follow the same methodology as that of members reinforced with conventional steel bars. A simplified flexural design method for ASTM A1035 steel, together with an idealized elastic-plastic stress-strain relationship, form the basis for flexural design.

Compression-controlled and tension-controlled strain limits for members designed with ChromX® bars conforming to ASTM A1035 Types CS, CM and CL at design strength of 100 ksi [690MPa] are modified to 0.004 and 0.009, respectively, to ensure comparable level of displacement and curvature ductility as members with conventional rebar. The specified yield strength for design of structural members using ChromX® ASTM A1035/A1035M Grade 100 [690] reinforcement is outlined in Annex B.

a. The specified yield strength of the reinforcement in tension shall be taken as the 0.2 percent offset with a minimum of 690 MPa.
b. The specified yield strength in compression shall be taken as the stress corresponding to a strain of 0.35 percent.

Development and splice lengths can be adequately determined using ACI 318-08 provisions for confined splices, and using the modified equation in ACI 408R-03 "*Bond and Development of Straight Reinforcing Bars in Tension"* for both confined and unconfined splices. Annex C illustrates some sample development length calculations.

A complete expanded Design Methodology is included in Annex D in accordance to ICC AC 429/ ESR 2107.

1. Structural design using ASTM A1035 Grade 690 high-strength reinforcing bars shall be in accordance with ACI 318-08, ACI 318-11 and ACI 318-14 Building Code Requirements as modified by the ICC AC 429 and the ICC ESR 2107.

2. The tensile requirements of ASTM A1035 Grade 690 high-strength reinforcing bars shall be as follows:
   a. The specified yield strength of the reinforcement in tension shall be taken as the 0.2 percent offset with a minimum of 690 MPa.
   b. The specified yield strength in compression shall be taken as the stress corresponding to a strain of 0.35 percent.

3. Minimum elongation of ASTM A1035 Grade 100 [690] high-strength bars shall not be less than 7% for bar designation #3 through #11 [12 mm through 32 mm] and 6% for bar designation # 14 through #20 [40 mm]. For computing flexural strength, the yield stress shall not exceed 690 MPa. (Annex B)

4. For computing compression strength, stress corresponding to 0.35 percent strain, the yield strength shall not exceed 690 MPa. (Annex B)

5. For computing shear strength, the yield strength shall not exceed 550 MPa and for confinement, the yield strength shall not exceed 690 MPa. (Annex B)

6. Development and Lap Splice length shall be determined in accordance with ACI 318-08, ACI 318-11 and ACI 318-14 as modified by ICC AC 429/ICC ESR 2107 as illustrated in Annex B.

7. Mechanical splices shall be designed in accordance with ACI 318-08, ACI 318-11 and ACI 318-14 as modified by ICC AC 429/ICC ESR 2107.

8. The specified compressive strength for concrete shall range from 4,000 psi [27.6MPa] to 16,000 psi [110 MPa], as per the ICC AC 429 section 1.2.4

9. Compression-Controlled Strain Limit: For ChromX® bars conforming to ASTM A1035 types CS, CM and CL Grade 100 [690] reinforcement, the compression-controlled strain limit is set at 0.004.

10. Tension-Controlled Strain Limit: For ChromX® bars conforming to ASTM A1035 types CS, CM and CL Grade 100 [690] reinforcement, the tension-controlled limit strain is set at 0.009.

11. Deflections shall be computed in accordance with Sections 19.2.34.1 for determination of Modulus of Rupture, f_r and 24.2.3. Replacing ACI 318 Equation 24.2.3.5a with the following equation:

$$\boldsymbol{I_e} = \frac{\boldsymbol{I_{cr}}}{\mathbf{1} - \left(\mathbf{1} - \frac{\boldsymbol{I_{cr}}}{\boldsymbol{I_g}}\right)\left(\frac{\boldsymbol{M_{cr}}}{\boldsymbol{M_a}}\right)^2} \leq \boldsymbol{I_g}$$

12. When using Etabs, Safe or any other design software, the following design parameters shall be verified:

    a. Strength Reduction Factors (0.65 for compression-controlled and 0.9 for tension-controlled designs)
    b. Tension-Controlled Strain Limit (0.009)
    c. Compression-Controlled Strain Limit (0.004)
    d. Control of Deflection using modified I_e
    e. Development Length Equation as per Annex C

- **AASHTO LRFD Bridge Design Specifications Compliance**

The American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications are meant for use in the design, evaluation and rehabilitation of bridges, and the U.S. Federal Highway Administration (FHWA) mandates the specifications for use in all bridges using federal funding. These specifications use LRFD methodologies with factors developed from current statistical knowledge of loads and structural performance. The following updates (code and commentary articles - Table 10) are related to the *AASHTO LRFD Bridge Design Specifications, 7th Edition* (2016 Interim Revisions, Section 5, “Concrete Structures”) (ref 5.B.16) addressing the use of ASTM A1035 Grade 100 [690]/AASHTO M334 steel reinforcing bar at yield strengths up to and including 100 ksi (690 MPa) for all elements and connections in Seismic Zone 1:

Table 10 AASHTO LRFD Bridge Design Specifications 100 Ksi (690 MPa) Related Sections

Reinforcing Bars	5.4.3.1 5.4.3.3	Permits reinforcing bars with minimum yield strength up to 100 ksi (690 MPa) for use in all elements and connections in Seismic Zone 1.
Reinforcing Bars – Fatigue Threshold	5.5.3.2	Equation 5.5.3.2.1 has been calibrated and provides more reasonable values of $(\Delta F)_{TH}$ for higher strength reinforcing bars.
Design of High Strength Steel Rebar	5.7	For nonprestressed reinforcing steel with a specified minimum yield strength of 100 ksi, the compression-controlled strain limit shall be taken as ε_t = 0.004. The tension-controlled strain limit, ε_t, shall be taken as 0.008 for nonprestressed reinforcing steel with a specified minimum yield strength, fy = 100 ksi.
Control of Cracking	5.7.3.4	In certain situations, involving higher strength reinforcement or large concrete cover, the use of Equation 5.7.3.4.1 can result in small or negative values for bar spacing, s. Where higher strength reinforcement is used, s need not be less than 5 inches for control of flexural cracking.

Moment Redistribution	5.7.3.5	In lieu of more refined analysis, where bonded reinforcement that satisfies the provisions of Article 5.11 is provided at the internal supports of continuous reinforced concrete beams, negative moments determined by elastic theory at strength limit states may be increased or decreased by not more than 1000 ε_t percent, with a maximum of 20 percent. Redistribution of negative moments shall be made only where εt is equal to or greater than 1.5 εtl at the section at which moment is reduced, where εtl is the tension-controlled strain limit specified in Article 5.7.2.1. Positive moments shall be adjusted to account for the changes in negative moments to maintain equilibrium of loads and force effects.
Spirals and Ties	5.7.4.6	f_{yh} = specified minimum yield strength of spiral reinforcement (ksi) = 100 ksi for elements and connections specified in Article 5.4.3.3; ≤ 75.0 ksi otherwise
Transverse Reinforcement	5.8.2.4 5.8.2.5 5.8.3.8 5.10.6.1	For members subjected to flexural shear without torsion, reinforcing steel with specified minimum yield strengths up to 100 ksi may be used for transverse reinforcement for elements and connections as specified in Article 5.4.3.3.
Longitudinal Reinforcement	5.8.3.5	Longitudinal or transverse reinforcing steel, or a combination thereof, with specified minimum yield strengths up to 100 ksi, may be used in elements and connections specified in Article 5.4.3.3.

- **DESIGN GUIDE FOR USE OF ASTM A1035 HIGH-STRENGTH REINFORCEMENT IN CONCRETE BRIDGE ELEMENTS WITH CONSIDERATION OF SEISMIC PERFORMANCE ZONES 3 & 4:**

AASHTO LRFD Bridge Design Specification - C5.4.3.3

In 2004, ASTM published A1035/A1035M, Standard Specification for Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement. This reinforcement offers the potential for corrosion resistance. Reinforcing steels with a minimum specified yield strength between 75.0 and 100 ksi may be used in seismic applications, with the Owner's approval, only as permitted in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

A team of researchers and code experts lead by S. K. Ghosh, Henry Russell and Mehdi Saiidi published a report titled "Design Guide for Use of ASTM A1035 High-Strength Reinforcement in Concrete Bridge Elements with Consideration of Seismic Performance as permitted in the AASHTO Guide Specifications for LRFD Seismic Bridge Design" (ref. 5.B.19).

Based on the information presented in this report, Table 11 lists the maximum strengths of reinforcement that may be used in the design of the different structural elements of bridges based on the AASHTO LRFD recommendations and incorporating their conclusions specifically concern applications in Seismic Zones 2, 3 and 4.

Bridge Decks, Girders, and Bent Cap Beams:

In view of the fact that (1) bridge decks, girders, and bent cap beams are capacity-protected, (2) satisfactory performance in field studies has been documented, and (3) research findings that the use of ChromX® bars in close proximity with black steel bars in chloride-contaminated environments does not lead to enhanced corrosion of the reinforcing bars, there should be no reservation about permitting ChromX® reinforcing bars in bridge decks, girders, and bent cap beams in Seismic Zones 3 and 4, provided the guidelines of AASHTO LRFD are followed.

Bridge Columns:

Pending further testing, ChromX® steel should not be used as longitudinal reinforcement in bridge columns in Seismic Zones 3 or 4. The use of ChromX® steel as transverse reinforcement in such members should be permitted, provided the transverse reinforcement yield strength is restricted to no more than 60 ksi for the purposes of computing shear strength. The full yield strength of the transverse reinforcement may be utilized for purposes of confinement of the concrete.

Pier Walls, Back Walls, and Wing Walls:

The above recommendation should also apply to pier walls, back walls, and wing walls, which are preferable locations for inelastic behavior in most bridges.

Foundation Elements:

In general, ChromX® reinforcing bars should be permitted to be used in any foundation element such as a footing or a Type II pile shaft in Seismic Zones 3 and 4 that is capacity protected, provided the guidelines of AASHTO LRFD are followed. When ChromX® reinforcing bars are used as transverse reinforcement in pile shafts, the transverse reinforcement yield strength should be restricted to no more than 60 ksi for the purposes of computing shear strength. The full yield strength of the transverse reinforcement may be utilized for purposes of confinement of the concrete.

Table 11 – Maximum Tensile Strengths of Reinforcement for Use in Design of Bridges

Seismic Zone	**Foundations**			**Columns / Walls**		**Decks**	**Beams / Girders**		
	Abutments	Piles	Pile Caps	Vertical	Confinement	Top & Bottom	Tension	Compression	Shear
Zone 1	100	100	100	100	100	100	100	100	100 (2)
Zone 2	100 (3)	100 (3)	100 (3)	100 (4)	100 (3)	100	100	100	100 (2)
Zone 3	100 (3)	100 (3)	100 (3)	N/R (5)	100 (3)	100	100	100	100 (2)
Zone 4	100 (3)	100 (3)	100 (3)	N/R (5)	100 (3)	100	100	100	100 (2)

(1) Design Guide for Use of ASTM A1035 High-Strength Reinforcement in Concrete Bridge Elements with Consideration of Seismic Performance, H. G. Russell, S. K. Ghosh, Mehdi Saiidi (2011) and Design Guide for Use of ASTM A1035 High-Strength Reinforcement in Concrete Bridge Elements in AASHTO Seismic Zone 2, S. K. Ghosh (2012).
(2) Yield strength limited to 60 ksi for shear friction calculation.
(3) Yield strength of transverse reinforcement limited to 60 ksi for shear strength computations.
(4) Required shear strength must be calculated per Articles 8.3.2 and 8.6.1 and minimum shear reinforcement must be provided per Article 8.6.5 of the AASHTO Guide Specifications for LRFD Bridge Design.
(5) Not recommended. Concrete reinforcing steel used must meet ASTM A706 seismic requirements.

4. Cost Savings Analysis

A. High Strength Savings redesign of ChromX® Grade 100 vs Grade 75 Steel:

Taking advantage of the design strength of ChromX® Grade 100 [690] in a mat foundation and shear wall designs compared to using Grade 75 [520] designs resulted in 12% savings in total steel cost for the project as shown in Table 12 for the mat foundation and 15% reduction in the total steel cost for the shear wall as shown in Table 13 and Summarized in Table 14:

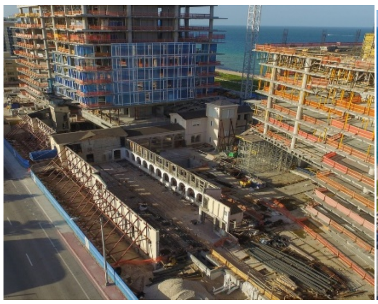


Table 12 - Rebar Cost Comparison for a Mat Foundation Example

Substructure Mat Foundation Steel Reinforcement Construction Costs / tons	**Steel Reinforcement of Structure Members Designed with Grade 75**	**ChromX® Steel Reinforcement of Structure Members Redesigned with Grade 100**	**Added Cost/ (Savings)**	**Added Cost/ (Savings)**
Rebar costs/tons	$580	$900	$320	+55%
Rebar tons	2,000 tons	1,350 tons	(650) tons	-33%
Rebar costs	$1,160,000	$1,215,000	$55,000	+5%
Fabrication costs ($100/ton)	$200,000	$135,000	($65,000)	-33%
Delivery to job site ($25/ton)	$50,000	$33,750	($16,250)	-33%
Placement costs ($425/ton)	$850,000	$573,750	($276,250)	-33%
TOTAL STEEL REINFORCEMENT COSTS	$2,210,000	$1,957,500	($252,500)	-12%

Table 13 - Rebar Cost Comparison for Shear Wall Designs Example

Superstructure Shear Wall Steel Reinforcement Construction Costs / tons	Steel Reinforcement of Structure Members Designed with Grade 75	ChromX® Steel Reinforcement of Structure Members Redesigned with Grade 100	Added Cost/ (Savings)	Added Cost/ (Savings)
Rebar costs/tons	$580	$900	$320	+55%
Rebar tons	1,500 tons	1,000 tons	(500) tons	-33%
Rebar costs	$870,000	$900,000	$30,000	+3%
Fabrication costs ($100/ton)	$150,000	$100,000	($50,000)	-33%
Delivery to job site ($25/ton)	$50,000	$25,000	($12,500)	-33%
Placement costs ($425/ton)	$850,500	$425,000	($212,500)	-33%
TOTAL STEEL REINFORCEMENT COSTS	$1,695,000	$1,450,000	($245,000)	-15%

Table 14 - Total Rebar Cost Savings Based on Reduction in Quantity

	Volume of Steel Reinforcement of Structure Members Designed with Grade 75	Volume of ChromX® Steel Reinforcement of Structure Members Redesigned with Grade 100	Construction Savings Reinforcement Cost	% Change in Cost Savings
Rebar-Substructure	2,000	1,350	($252,500)	-12%
Rebar-Superstructure	1,500	1,000	($245,000)	-15%
Total	3,500	2,350	($497,500)	-15%

Using the same mat foundation in the above example, concrete thickness can be reduced in the mat foundation when the designers can take advantage of the design strength of 100 ksi and the reduction of the number of steel layers as illustrated in the following example in which 20% of concrete was reduced in optimizing the designs with 100 ksi ChromX® 2100 steel rebars.

Table 15 - Typical saving comparison using ChromX® steel redesign and reduction of concrete

Construction Costs	Grade 75 Black Bar	Grade 100 ChromX® 2100	Added Cost / (Savings)	% Change
Steel Related Costs				
Rebar tons	2,000	1,350	(650)	-33%
Rebar costs/ton	$580	$900		
Rebar costs	$1,160,000	$1,215,000	$ 55,000	
Fabrication costs ($100/ton)	$ 200,000	$ 135,000	($ 65,000)	
Delivery to job site ($25/ton)	$ 50,000	$ 33,750	($ 16,250)	
Placement ($425/ton)	$ 850,000	$ 573,500	($ 276,250)	
Total Steel Related Costs	$2,210,000	$1,957,500	($ 252,500)	-12%
Concrete Costs				
Concrete cubic yards	20,842	16,666		
Concrete cost per cy	$160	$160		
Total Concrete Cost	$3,334,682	$2,666,560	($668,122)	-20%
Other Construction Costs				
Excavation cubic yards	31,738	25,379		
Excavation ($10/cy)	$317,377	$253,791		
Total Other Costs	$317,377	$253,791	($63,585)	-20%
Total Construction Costs	($5,862,059)	($4,877,851)	($984,208)	-17%

Table 16 - Summary of Logistics Savings when using ChromX® 2100 vs Grade 75 Steel

Logistics Savings	Grade 75 Black Bar	Grade 100 ChromX® 2100	Added Cost / (Savings)	% Change
Trucks				
Steel delivery (25 tons/truck)	80	54		
Concrete trucks (9 cy/truck)	2,216	1,852		
Total Trucks	2,396	1,906	(490)	-20
Time - Placement				
Placement hours (0.004 MH/lb)	16,000	10,800		
Total Time Hours (50 placers)	320	216	(104)	-33
Total Time Days (12 hours/day)	27	18	(9)	
Time – Concrete Pour				
Concrete pour hours (60 min/truck)	2,316	1,852		
Consecutive unloading (50 trucks)	46	37	(9)	-20
Total Time Days (24 hours/day)	2	2	(0)	
Total Time Hours	366	229	(137)	-37
Total Time Days	29	18	(11)	

B. High Strength Savings redesign of ChromX® Grade 100 vs Grade 60 Steel:

Taking advantage of the design strength of ChromX® Grade 100 [690] in a mat foundation designs compared to using Grade 60 [420] designs resulted in 24% savings in total steel cost for the project as shown in Table 17 for the mat foundation:

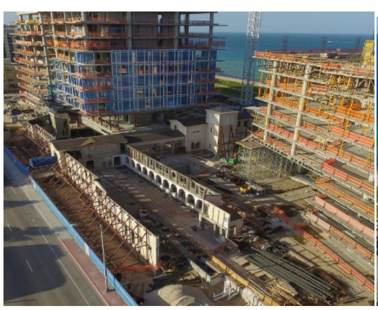



Table 17 - Rebar Cost Comparison for a Mat Foundation Example with Grade 60 Steel

Substructure Mat Foundation Steel Reinforcement Construction Costs / tons	Steel Reinforcement of Structure Members Designed with Grade 60	ChromX® Steel Reinforcement of Structure Members Redesigned with Grade 100	Added Cost /(Savings)	Added Cost / (Savings)
Rebar costs/tons	\$560	\$900	\$340	+60%
Rebar tons	2,000 tons	1,200 tons	(800) tons	-40%
Rebar costs	\$1,120,000	\$1,080,000	(\$40,000)	-4%
Fabrication costs (\$100/ton)	\$200,000	\$120,000	(\$80,000)	-40%
Delivery to job site (\$25/ton)	\$50,000	\$30,000	(\$20,000)	-40%
Placement costs (\$425/ton)	\$850,000	\$510,000	(\$340,000)	-40%
TOTAL STEEL REINFORCEMENT COSTS	\$2,250,000	\$1,713,000	(\$537,000)	-24%

Using the same mat foundation in the above example, concrete thickness can be reduced in the mat foundation when the designers can take advantage of the design strength of 100 ksi and the reduction of the number of steel layers as illustrated in the following example in which 20% of concrete was reduced in optimizing the designs with 100 ksi ChromX® 2100 steel rebars

Table 18 - Typical saving comparison using ChromX® steel redesign and reduction of concrete

Construction Costs	Grade 60 Black Bar	Grade 100 ChromX® 2100	Added Cost / (Savings)	% Change
<u>Steel Related Costs</u> Rebar tons Rebar costs/ton Rebar costs Fabrication costs ($100/ton) Delivery to job site ($25/ton) Placement ($425/ton) Total Steel Related Costs	 2,000 $560 $1,120,000 $ 200,000 $ 80,000 <u>$ 850,000</u> $2,250,000	 1,200 $900 $1,080,000 $ 120,000 $ 30,000 <u>$ 510,000</u> $1,713,000	 (650) ($ 40,000) ($ 80,000) ($ 20,000) <u>($ 340,000)</u> ($ 537,000)	 -40% -24%
<u>Concrete Costs</u> Concrete cubic yards Concrete cost per cy Total Concrete Cost	 20,842 $160 $3,334,682	 16,666 $160 $2,666,560	 ($668,122)	 -20%
<u>Other Construction Costs</u> Excavation cubic yards Excavation ($10/cy) Total Other Costs	 31,738 $317,377 $317,377	 25,379 $253,791 $253,791	 ($63,585)	 -20%
Total Construction Costs	($5,902,059)	($4,633,351)	($1,268,708)	-21%

4. Reference Publications / Reports / Papers

Given that many of these studies were written prior to our product line expansion, they often mention MMFX$_2$ when referring to the ChromX® ® 9000 series. The following reference documents provide a more detailed review of ASTM A1035 Types CS, CM and CL (ChromX® Series 9000, 4000, and 2000) and AASHTO M334 Grade 100 (ASTM A1035 Type CS – ChromX® 9100) rebar's corrosion and structural characteristics.

A. Corrosion Test Reports, Papers and Analysis References

1. Reinforcing Steel Comparative Durability Assessment and 100-year Service Life Cost Analysis Report – **Tourney Consulting Group–N. Berke–June 2016 – (68 pages):** This study focused on modeling and evaluating concrete reinforcement for corrosion; and estimating the service life for various reinforcing steel alternatives. The evaluation included engineering analysis and laboratory test data from Tourney Consulting Group's internal database. The engineering analysis included modeling of concrete service life using STADIUM® software. The investigation centered on the ChromX® 9100 (ASTM A1035 Type CS/AASHTO M334 "MP 18") and ChromX® 4100 (ASTM A1035 Type CM) bars versus black bar (BB), epoxy-coated (ECR), hot-dip galvanized (HDG), and stainless steel (SS2304) reinforcements.

The following three project scenarios were analyzed, using a specific geographical project location, and representing differing climatic and other environmental settings.

- Bridge deck exposed to deicing salts (Ohio River bridge near Louisville, KY)
- Marine pile (Cape Hatteras, NC)
- Pile in corrosive soil with high sulfate content Gulf Cooperation Council (GCC) country

In addition, a Life Cycle Cost Analysis (LCCA) was performed on the bridge deck.

The following table summarizes the result of the bridge deck analyze.

Service Life Analysis for Bridge Deck with Repairs

Concrete Type	Bridge Low Permeability (LP)				
Exposure	Deicing Salts		Rebar Initial Cost	1st Repair NPR	Total Cost Initial +All Repairs NPR
Cover (min.)	1.5 inches		$\$/ft^2$	$\$/ft^2$	$\$/ft^2$
Reinforcement Type	Estimate to Initiation (y) / 1st Repair (y)				
Black Bar (BB)	19	25	7.32	5.62	19.30
ChromX® 9100 Gr 60	>100	>100	14.52	0	14.52
ChromX® 9100 Gr 75	>100	>100	11.52	0	11.52
ChromX® 9100 Gr 100	>100	>100	8.76	0	8.76
ChromX® 4100 GR 60	43	61	9.72	1.37	12.28
ChromX® 4100 GR 75	43	61	7.80	1.37	10.36
ChromX® GR 100	43	61	5.88	1.37	8.44

Concrete Type	Bridge Low Permeability (LP)				
Exposure	Deicing Salts		Rebar Initial Cost	1st Repair NPR	Total Cost Initial +All Repairs NPR
Cover (min.)	1.5 inches		$/ft^2	$/ft^2	$/ft^2
Reinforcement Type	Estimate to Initiation (y) / 1st Repair (y)				
ChromX® 4100 Gr 60 2 gpy CNI	>100	>100	10.03	0	10.03
ChromX® 4100 Gr 75 2 gpy CNI	>100	>100	8.01	0	8.01
ChromX® 4100 Gr 100 2 gpy CNI	>100	>100	6.16	0	6.16
Epoxy Coated (ECR)	19	34	10.08	3.95	18.50
Galvanized (GS)	59	76	13.68	0.76	14.86
UNS S32304 (SS2304)	>100	>100	25.32	0	25.32

Notes:
1. Repairs every 15 years starting at 1st repair at $150/ft2 (Traffic + Repair)
2. 10% of the surface area is repaired at each time
3. Discount rate at 4%
4. Only ASTM A1035-CS (ChromX® 9100), ASTM A1035-CM (ChromX® 4100) with 2 gal/cy CNI, and SS2304 options meet requirement of no significant repairs in either 75 or 100 years.

2. [Case Studies for 100-Year Service Life Utilizing High Strength Low Chromium Reinforcing Bars](#) – NACE- Concrete Service Life Extension Conference- N Berke, R. Vijayakumar, July 2015 (19 pages): This paper is based on testing, which is part of a NCHRP Idea project that included: black steel rebar(BB), A1035 Type CS and CM rebar (A1035 9-Cr "ChromX® 9100") and (A1035 4-Cr "ChromX® 4100"), epoxy-coated rebar (ECR), hot-dipped galvanized rebar (HDG), and stainless steel rebar (S32304) versus zinc coated bar with (TZD) and without (TZE) a hand painted epoxy coating.

The paper concluded:

"All of the corrosion resistant reinforcing bars in this study outperformed black steel in both un-cracked and cracked and fatigued concrete specimens.

For cracked beams after 26 cycles of testing:

- *The corrosion resistant bars have more than a two times reduction in macrocell currents versus the BB.*
- *The A1035 9-Cr and 4-Cr showed a longer time to high macrocell rates vs. the HDG and TZD steels.*
- *Macrocell corrosion for A1035-9Cr+CNI equivalent to better than that of S32304 and the A1035-4CrNI is approximately order of magnitude less than of HDG and TZD.*
- *Cracking data indicate the A1035-9Cr with CNI is the best performer along with S32304. The most severe cracking behavior is on the HDG specimens."*

"A service life and life-cycle case study was performed to compare the various corrosion resistant steels to BB and each other. The results for this case study show that:

- *All of the corrosion resistant steels, with the exception of S32304, reduce life cycle costs, even without taking account of indirect user costs.*
- *Only A1035-9Cr, A1035-4Cr with CNI, and S32304 met 100 years of service before major repairs would be needed.*
- *Significant cost savings are possible when the designer can utilize the higher strength grades available for A1035."*

3. [Exposure Testing of Chloride Exposed Reinforced Concrete and Potential Misrepresentation of Service Performance](#)– Paper No. 5460 NACE Corrosion 2015 Conference–W. Hartt, March 2015 (19 pages): This paper's conclusions included: *"1. Failure to assure that the chloride ingress mechanism (sorption versus diffusion) for test specimens reflects what is anticipated for the structures intended to be represented. This is particularly true when comparing different corrosion resistant reinforcements (CRR). Misleading indications of service performance and of relative performance of CRR can result. 2. Leakage of chlorides from top surface mounted ponding baths onto reinforcements where these penetrate the concrete can result in active potentials and macrocell currents that indicate corrosion. However, embedded steel beneath ponding baths may be un-corroded. 3. Moisture accumulation at reinforcement-concrete interfaces where the former penetrates the latter can result in situations similar to that in (2). 4. Design of concrete exposures such that time to achieve a steady-state mean surface chlorides concentration is not short compared to time-to-corrosion initiation can lead to incorrect conclusions regarding relative performance of different reinforcement types. 5. Failure to assure that the mean surface chloride concentration during testing of concrete slabs is representative of what occurs in the service application at issue can lead to underestimation of actual performance in service of intermediately alloyed CRR such as A1035 and UNS-S41003."*

4.[Increasing Bridge Deck Service Life: Volume I](#)—Technical Evaluation - FHWA/IN/JTRP-2014/16 – Purdue Univ. – R. Frosch, S. Labi, C. Sim - December 2014 (289 pages): This report indicates results of concrete: bond lap splice, slab cracking behavior; and corrosion resistance under uncracked and cracked conditions tests using four types of stainless steel, MMFX (ASTM A1035), hot-dip galvanized, and Zbar (dual-coated) reinforcing bars.

Following test results were reported:

- *Stainless-steel, MMFX II microcomposite [ASTM A1035], hot-dip galvanized, and Zbar (dual-coated) reinforcing bars have bond strengths comparable to black bars. Coated bars other than galvanized and dual-coated have reduced bond strengths. Epoxy coated bars had on average 11% less bond strength than black while un-plated zinc-clad and tin-plated zinc-clad bar had on average 18% and 26% less bond strength than black bars, respectively.*

- *Slabs with MMFX II [ASTM A1035] bars at 6 in. and 12 in. bar spacing produced crack widths comparable to slabs with black bars, specimens with MMFX bars at an 18 in. spacing resulted in the smallest crack widths compared to the other bars including black bars at every stress level.*
- *All four stainless steels also perform significantly better compared to the control black bar. MMFX II microcomposite [ASTM A1035] steel, however, demonstrates more corrosion than the stainless-steel reinforcement. This result is expected because the steel includes less chromium. In addition, the MMFX II [ASTM A1035] reinforcement also retains its mill scale. However, for MMFX II [ASTM A1035], it is evident that the initiation of corrosion is delayed as compared to the black bar.*

5. Corrosion Sensitivity of Concrete Mix Designs FHWA/VCTIR 14-R19 - Virginia Center for Transportation Innovation and Research - S. Sharp, C. Ozyildirim, D. Mokarem June 2014 (32 pages): This study compared the durability of concrete mixtures containing supplementary cementitious materials (SCMs) by evaluating the permeability, absorption, and corrosion resistance of seven mix designs and two types of reinforcement. This study also demonstrated that the corrosion-resistant reinforcement plays the most vital role in minimizing corrosion. SCMs provide durable concretes and in combination with the corrosion-resistant reinforcement ensure reinforced concrete structures with longer service lives. ASTM A615 and ASTM A1035 were used as a baseline to enable comparisons with other corrosion studies.

6. Acceptance Procedures for New and Quality Control Procedures for Existing Types of Corrosion-Resistant Reinforcing Steel FHWA/VCTIR 11-R21- 7. S. Sharp, L. Lundy, H. Nair et al. - June 2011 (80 pages): This study was made to provide VDOT's Materials Division with a method/specification for evaluating CRR bars. Bar types and tests are indicated in the following table, which included "MMFX 2" AASHTO M 334 "MP 18"/ASTM A1035 Type CS bars.

Table 3. Tests Performed on Various Types of Rebar

Rebar Type	X-Ray Fluorescence	Magnetic Sorting	Hardness Test	Uniaxial Tensile Test	Elongation	Reduction in Cross - Sectional Area	Bar Finish	Mill ID Markings	Relative Rib Area	Concrete / Steel Bond Strength	Corrosion Resistance
2101 LDX	X	X									X
2205	X	X		X	X			X	X	X	X
2304	X	X			X		X				X
304		X									
316L Clad (NX)	X			X	X	X	X	X	X	X	X
316LN	X	X		X		X			X	X	X
ASTM A615 Grade 60				X	X	X			X	X	
ASTM A615 Grade 75	X	X		X			X		X	X	X
ASTM A615 Grade 75 w/Zn	X										X
Duracorr	X		X		X	X	X	X			X
EnduraMet 32	X	X		X	X	X		X	X	X	X
MMFX 2	X	X	X	X	X	X			X	X	X
ECR	X						X				X
Zbar	X			X			X		X	X	X

7. A Critical Review of Corrosion Performance for Epoxy-Coated and Select Corrosion Resistant Reinforcements in Concrete Exposed to Chlorides–W. Hartt, Hartt and Associates, Inc.-April 2011 (8 pages): This paper reviews the corrosion performance of Corrosion Resistant Reinforcing (CRR) materials, including epoxy coated reinforcement (ECR), using numerous studies from 1987 thru 2010. It notes: *"A finding from a number of the above referenced studies was that ECR experiences corrosion at coating defects along with cathodic disbondment of the adjacent coating. Also noted was wet adhesion loss and under film* *corrosion*." The report states: "*A fundamental difficulty in projecting future ECR performance is that, first, long-term deterioration processes are, for the most part, not necessarily reflected by results from short-term exposures and, second, it is not possible to perform first-principles based modeling of the coating disbondment, embrittlement, and under film corrosion processes. Such modeling can be performed for uncoated bars, …*" The paper indicates that long term studies of uncoated bars including MMFX 2 ChromX® 9100 (ASTM A1035- Type CS) and UNS-S41003 (3Cr12) indicate these uncoated bars provide mean critical chloride threshold (C_T) values at 4 times that of black steel. In conclusion, the paper states: *"In summary, long-term performance of ECR in chloride contaminated concrete is uncertain; and no first-principles based analytical tools are available for projecting this. On the other hand, sufficient data are available, along with a first-principles based methodology, for projecting long-term performance of uncoated BB and CRR such as 3Cr12 and MMFX2™. Results from the latter analysis indicate that in the case of these CRR a low maintenance service life of 75 and even 100 years can be expected."*

8. Corrosion Initiation Projection for Reinforced Concrete Exposed to Chlorides – Part II: Corrosion Resistant Bars–NACE Corrosion 11–W. Hartt-March 2011 (11 pages): This paper presents the results of a study utilizing an equation from the literature to analyze the growth of reinforcement corrosion initiation (Ti) in concrete exposed to chlorides. The study's analytical method was comparatively applied to black bar (BB) and to corrosion resistance reinforcement (CRR) steels (ASTM A1035 Type CS and UNS-S41003 - 3Cr12 stainless steel), varying 1) the surface Cl^- concentration (C_s), 2) the effective diffusion coefficient (D) in the equation. The analysis used previously published CRR steel's critical Cl^- concentration threshold values (C_T), at four (4) times that of BB. The study's findings included:

- Analyses projected the T_i for initial occurrence of CRR corrosion to be almost ten (10) times greater than for BB
- An analysis comparing the T_i trend of BB in high performance concrete (HPC) with the T_i trend of CRR in Portland Cement Concrete (PCC), indicated that almost 50 percent less of the CRR/PCC system initiated active corrosion than the BB/HPC system after 100 years. If the analysis had considered the greater tendency of HPC compared to PCC to exhibit shrinkage cracking, then the difference between the two trends would have been greater.

9. Critical Chloride Corrosion Threshold of Galvanized Reinforcing Bars- ACI Material Journal -Technical Paper M106-M22–D. Darwin, J. Browning et. al. -Mar-Apr 2010 (8 pages): This technical paper based on research as part of the International Lead Zinc Research Organization, Inc. under Project Code ZC-24-2 (See also Ref. 5.A.23) indicated the following: "*The average critical corrosion threshold for galvanized reinforcement, 2.57 lb/yd³ (1.5 kg/m³), is higher than the observed critical corrosion threshold of conventional (A615) steel, 1.63 lb/yd³ (0.97 kg/m³), and lower than the value for A1035 steel, 6.34 lb/yd³ (3.76 kg/m³), and the lower-bound value for 316LN stainless steel, 19.14 lb/yd3 (11.36 kg/m³).* " "*Based on chloride surveys of cracked bridge decks, galvanized steel can be expected to increase the average time to corrosion initiation at crack locations from 2.3 years for conventional steel to 4.8 years for bars with 3 in. (76 mm) of concrete cover. Corrosion initiation would be expected to occur at an average age of 15 years for ASTM A1035 reinforcement and not to occur for bars consisting of 316LN stainless steel.*" (Copy of this paper may be obtained from the American Concrete Institute (ACI) Farmington Hills, MI.)

10. The Use of Corrosion Resistant Reinforcement as a Sustainable Technology for Bridge Deck Construction - TRB Annual Meeting 2010 Paper #10-2214 - A. Moruza, S. Sharp–January 2010- (24 pages): "*As part of the Innovative Bridge Research and Construction Program (IBRCP), this study used the full-scale construction project of the Route 123 Bridge over the Occoquan River in Northern Virginia to identify differences in the installation practices and comprehensive placement costs of epoxy-coated reinforcing steel (ECR) and corrosion-resistant reinforcing steel (CRR), specifically an ASTM A1035 steel. Internal VDOT construction records provided construction costs specifically associated with one of the two reinforcement materials. … For this project, the cost advantage of ECR at the time of contract award was not preserved after inclusion of unanticipated construction costs directly related to ECR. Specifically, adding the cost of deck sealing operations to the bid cost of ECR produced an in-place cost estimate of $.804/lb compared to $.780/lb for CRR. Inclusion of the indirect costs of the sealing operations, however, more than quadrupled the unit cost of ECR over bid, predominantly because of road user costs to the public. … CRR was ultimately* *cost-competitive with ECR in this project when costs of common VDOT practices related to ECR were included, but CRR potentially possesses superior longevity benefits as a sustainable choice for deck reinforcement, especially since significant traffic growth is expected on this new structure."*

11. Laboratory Study of Corrosion Performance of Different Reinforcing Steels for Use in Concrete Structures - National Research Council of Canada - Research Report IRC-RR-284 - J. Zhang, S. Qian, B. Baldock-September 2009 (55 pages): This report presents the results of a two-year program comparing the corrosion performance of ASTM A 615 (carbon steel), ASTM A 1035 Type CS "ChromX® 9100" (referred to as "Cr steel" in the report); and stainless steels 316LN, 304LN and 2205 bars. The primary testing program was performed in electrochemical cells by using different electrochemical techniques (simulated concrete pore solutions), and concrete prisms using sand blasted bars. Comparing A615 to ASTM A1035 bars, the report states: "*In the simulated concrete pore solution (pH=12.6), it was found that the average chloride threshold of carbon steel was about 1.0% by weight; the average chloride threshold of Cr steel was about 5.0%." … "Pitting corrosion was observed to initiate on Cr steel at a range of chloride concentrations from 0.18% to 0.3%. Pitting corrosion was initiated on carbon steel, at a much lower chloride concentration around 0.06%."* The following observations were made concerning concrete prism testing program: "*In concrete containing 1.5% of chlorides, Cr steel remained in passive state before undergoing active corrosion at ten months of age… As the chloride concentration increased from 1.5% to 3.0%, Cr steel showed lower corrosion rate than carbon steel". "…the corrosion rate of Cr steel did not increase significantly as the chloride concentration was increased from 3.0% to 6.0%. …" The corrosion rate was within a range of low to moderate at these concentrations."* The report also made the following observation concerning concrete prism testing of A1035 bars with mill scale: "*The corrosion rate of as-received Cr steel (non-sandblasted) was found to be much higher than that of sandblasted Cr steel, and the difference became greater with increasing chloride concentrations. The difference was about half an order of magnitude at 1.5% and 3.0% of chlorides, and increased to about one order of magnitude at 4.5% and 6.0% of chlorides."*

12. Field Comparison of the Installation and Cost of Placement of Epoxy-Coated and MMFX 2 Steel Deck Reinforcement: Establishing a Baseline for Future Deck Monitoring-S. Sharp, A. Moruza - VTRC 09-R9– May 2009 (84 Pages): This study identifies and compares differences in the installation practices and comprehensive placement costs of epoxy-coated reinforcing steel (ECR) and MMFX 2 ChromX® 9100 (ASTM A1035 Type CS/AASHTO M 334 "MP 18"), as part of the FHWA Innovative Bridge Research and Construction Program (IBRCP). The report was based on costs associated with Virginia DOT's construction of the Route 123 Bridge over the Occoquan River. Two separate bridge decks were constructed as part of the project: A. southbound deck using ECR, and the northbound deck using MMFX2, a corrosion-resistant reinforcing steel (CRR). Field surveys conducted after completion of construction resulted in crack sealing of certain spans of the ECR portion of the deck and modification for added bolster material above one of the northbound (MMFX 2) abutments. Final installation costs, including cracking sealing of certain ECR deck span sections and the additional work for the MMFX deck, indicated that that total direct in place unit costs for ECR were $0.90/lb and MMFX were $0.87/lb. The report indicates that indirect costs (i.e. VDOT inspection, special traffic control, and public travel delay) for required sealing of ECR deck cracking placed the total installed ECR unit cost between $2.34 and $2.90/lb. Labor records for the contract's ironworkers indicated that they placed: 329.9 lb/labor hour for ECR and 358.8 lb/labor hour for MMFX 2. The study notes: "*Average labor productivity estimates from this study suggested that the handling requirements of ECR led to additional supervisory costs and additional ironworker costs relative to those associated with uncoated CRR. Inspectors' records indicated that the subcontractor billed 15% more supervisor hours to place ECR in the southbound deck than to place MMFX 2 in the northbound deck, yet almost 16% less ECR than MMFX 2 was placed by weight …. The special handling requirements for ECR are a plausible explanation for the lower average labor productivity in the placement of ECR compared to that for MMFX 2…"* The report concludes by stating: *"ECR appears to have been far less cost-effective per unit than MMFX 2 when both anticipated and unanticipated costs of ECR in this study are estimated. MMFX 2 showed both labor productivity and comprehensive in-place cost advantages over ECR in this application."*

13. Risk of Macro-Cell Corrosion Associated with Black Bar – MMFX 2 (ASTM A1035) Combinations in Concrete - W. Hartt - Hartt and Associates, Inc.-May, 2009 (10 pages): This report was based on testing of concrete specimens to determine if adverse dissimilar metal effects occur when connecting MMFX 2 "ChromX® 9100" (ASTM A1035 Type CS) bars to black steel bars. The report makes the following statement: "*… there is no technical reason why black bar and MMFX2 reinforcements cannot be combined in concrete construction, including situations involving, first, field repairs and, second, new construction.*"

14. Corrosion Resistant Alloys for Reinforced Concrete – FHWA HRT 09-020 – W. Hartt, R. Powers, P. Virmani et. al – May 2009 (150 pages): This report documents the findings of a 6 year study of the corrosion resistance of various concrete reinforcing bars using four specimen types: simulated deck slabs (SDS), macrocell slabs (MS), 3 Bar tombstone columns (3BTC) and Field columns (FC) and three types of concrete mixtures: STD 1 (5 bag W/C 0.50), STD 2 (7 bag W/C 0.41) and STD 3 (7 bag W/C 0.50) Reinforcements included stainless steels: 316, 304, 2304, 2101, and 3Cr12 ; two types of 316 clad, AASHTO MP 13M/MP 13-04, and MMFX-2 "ChromX® 9100" (ASTM A1035 Type CS); and BB (ASTM A615), with BB used a comparator. The report indicates: " *The reinforcements, other than BB, were classified into two groups as either improved performance*" … (alloys with corrosion initiation during project) "*or high performance*" (alloys without corrosion initiation during project). *"Improved performers were 3Cr12, MMFX-2, and 2101 …. These alloys ranked according to time for corrosion to initiate as BB < 2101 < 3Cr12 < MMFX-2."* "Chloride threshold for corrosion initiation of 3Cr12 and MMFX-2 reinforced SDS specimens was about four times greater than for BB specimens and slightly less than four times greater in the case of 2101 specimens. For STD2 MS specimens, however, T_i for MMFX-2 and 2101 was from 3.4 to more than 5.7 times greater than for BB (limited data precluded this determination for 3Cr12)."

15. Effect of Concrete Crack Width on Corrosion of Embedded Reinforcement– W. Hartt - Hartt and Associates, Inc.- March, 2009 (10 pages): This commentary reviews existing literature pertaining to the effect of concrete crack width on reinforcement corrosion in various project environments with different exposure zones encountered in the Middle East. The literature indicates that crack width does not significantly influence long-term durability of reinforced concrete, for crack widths up to 1.6 mm (0.063 in.). The report indicates that the time to corrosion initiation is six times greater and the corrosion rate for MMFX 2 "ChromX® 9100" (ASTM 1035 Type CS) is six times less than that for black steel in cracked concrete.

16. Effect of crack width on corrosion of reinforcing steel– D. Darwin- February, 2009 (2 pages): This report states: "*Within the range of 0.1 to 0.6 mm, crack width does not affect the corrosion of reinforcing steel. Corrosion is, however, affected by concrete cover, water-cement ratio, and the orientation of the crack with respect to the reinforcing steel*." This discussion notes that the steel's corrosion rate is not affected when crack widths are under 0.6mm but are by their critical chloride threshold level. A 75mm concrete cover, the report indicates "…*provides protection against not only corrosion in the presence of cracks abut also against carbonation …*" and states "*For severe chloride exposure conditions, the ACI Building Code limits the water-cement ratio to a maximum of 0.40*."

17. Periodic Overload Corrosion Fatigue of MMFX and Stainless Reinforcing Steels– Journal of Materials in Civil Engineering, Vol. 21, No. 1 - S. DeJong, P. Heffernan, C. MacDougall - January 2009 (9 pages): "*This study presents the periodic overload and corrosion-fatigue resistance of machined specimens made from two corrosion resistant reinforcing steels: MMFX Microcomposite [ASTM A1035/AASHTO MP18] and 316LN stainless steel. MMFX had reduced constant amplitude performance in the corrosive environment, whereas 316LN stainless steel showed no environmental reduction under constant amplitude loading (except at high loads). Corrosion fatigue reduced the periodic overload performance of both materials, although both materials retained their intrinsic fatigue limit (250 MPa stress range) in the corrosive environment, a drastic improvement over the periodic overload corrosion-fatigue performance of conventional reinforcing steel*."

18. Concrete and Steel Type Influence on Probabilistic Corrosion Service Life- ACI Materials Journal V. 106, No. 1 - G. Williamson, R. Weyers, et. al., Jan.-Feb. 2009 (7 pages): This technical paper presents comparative service life predictions for Microcomposite (MMFX 2 "ChromX® 9100" /ASTM A1035 Type CS), stainless steel (SS), galvanized steel (GS) and carbon (black) steel reinforcing bars, utilizing low-permeability concrete (LPC with admixtures) and standard (without admixtures) concrete. Service lives were established using a computer model based on: a. time to corrosion initiation of 2% of the reinforcing steel; b. time from corrosion initiation to concrete cracking and spalling of the concrete over 2% of the reinforcing steel; and c. time for corrosion propagation from 2% to 12%. ASTM A1035 steel was noted to provide > 200 years of service life when used in conjunction with LPC; and 2 to 3 times the service life of black steel depending on the surface chloride concentration levels of standard concrete. (Copy of this paper may be obtained from the American Concrete Institute (ACI) -, Farmington Hills, MI.)

19. Evaluation of Corrosion Resistance of Steel Dowels Used for Concrete Pavements- Journal of Materials in Civil Engineering- M. Mancio, C. Carlos, J. Zhang, J. Harvey, P. Monteiro; A. Ali– October 2008 (9 pages): This study investigated the corrosion performance of several types of steel dowels cast in concrete beams using accelerated laboratory tests. This paper indicates: ". *that the microcomposite [ASTM A1035 Type CS] steel dowels exhibit much greater resistance to corrosion propagation than carbon steel dowels, but not as much as the stainless clad and stainless hollow bars*"." In addition the study states: "*Visual inspections of the corroded dowels revealed heavy and mostly uniform corrosion along the carbon steel dowels, light corrosion in the microcomposite steel dowels, and no visible corrosion in the stainless steel clad and stainless steel hollow bars. For the epoxy coated dowels, the visual inspections generally revealed that visible corrosion was not widespread, but did occur at a few localized defective areas, generally at holidays and at the edges of the bar ends. No significant difference was observed on the performance of nonflexible and flexible epoxy-coated dowels*."

20. Electrochemical and in-situ SERS study of passive film characteristics and corrosion performance of microcomposite steel in simulated concrete pore solutions - University of California Berkeley (UCB) – M. Mancio, G. Kusinski, et. al. – March 2008 (104 pages). This testing program provides the basis for AASHTO Standard Method of Test for *Comparative Qualitative Corrosion Characterization of Steel Bars Used for Concrete Reinforcement (Linear Polarization Resistance and Potentiodynamic Polarization Tests)*. The report states: *"After passivation, the current density for microcomposite steel [ASTM A1035 Type CS] was around 11 μA/cm^2 while that for carbon steel was about 140 μA/cm^2 (~13 times higher). If these values were used to estimate corrosion rates, one would get approximately 1627 μm/yr (1.63 mm/yr) for carbon steel and 128 μm/yr (0.128 mm/yr) for microcomposite steel [ASTM A1035]."*

21. "Comparative Corrosion Testing and Analysis of MMFX 2 Rebars for Reinforced Concrete Applications" – Wiss, Janney, Elstner Associates, Inc. - Final Report WJE No. 2003.0707.0 – F. Cui, P. Krauss, et. al.- January 2008 (57 pages): This report is based on ASTM G109 and time-to-corrosion (a.k.a. Southern Exposure) tests, stating: *"MMFX 2 [ASTM A1035 Type CS] steel bars have higher chloride thresholds than A615 bars, but less than Type 304 stainless steel. The ASTM G-109 test program suggested that the chloride threshold of MMFX bars is about three times of that of black bars ... The removal of mill scale was found to have slightly increased the corrosion resistance of MMFX bars in the G-109 test. … Modeling analyses of a marine pile and a northern bridge deck exposed to deicers showed that the use of MMFX bars in lieu of black bars may extend the structure service life by about 1.8 times, assuming a chloride threshold for the MMFX bars of three times that of the A615 black bars. Slower corrosion rates for the MMFX bars during the propagation period could increase the service life further, to about twice that of A615 bars."*

22. Corrosion of Reinforcing Bars in Concrete - R&D Serial No. 3013 Portland Cement Association - C. Hansson, A. Poursaee; S. Jaffer– 2007 (33 pages): The primary focus of this report is chloride-induced corrosion of steel reinforcement: the factors affecting it and its influence on durability. This review describes the corrosion process, the lack of a single chloride threshold concentration for initiation of corrosion and the relative contributions of micro-cell and macro-cell corrosion in sound concretes. The report indicates that a two-fold approach to corrosion resistant structures should include:
"The use of high performance concrete (HPC) to lower concrete's permeability and reduce the rate of ingress of chlorides or carbonation and, thereby, increase the effectiveness of the physical barrier.
The use of more resistant reinforcing bar materials to provide better chemical resistance. In those parts of structures exposed to very severe chloride environments, stainless steel is recommended. Despite the initial expense, it is a cost-effective solution in these circumstances when both direct and indirect costs (such as user costs) are taken into account. In the somewhat less severe chloride environments, corrosion resistant alloys such as MMFX [ASTM A1035 Type CS] or 2101LDX, which are more resistant to chlorides than black

steel - but less corrosion resistant and much less costly than stainless steel - should be considered."

23. [Critical Chloride Corrosion Threshold for Galvanized Reinforcing Bars](#) - SL Report 07-2 –Univ. of Kansas Center for Research– D. Darwin, J. Browning et. al.- Dec. 2007 (36 pages): This report prepared for the International Lead Zinc Research Organization indicated the following: "*… test results show that galvanized reinforcement has an average critical chloride corrosion threshold of 2.57 lb/yd^3, which is greater than conventional steel (1.63 lb/yd^3) and lower than MMFX [ASTM A1035 Type CS] steel (6.34 lb/yd^3)." "Based on chloride surveys of cracked bridge decks in Kansas, galvanized steel can be expected to increase the average time to corrosion initiation at crack locations from 2.3 years for conventional steel to 4.8 years for bars with 3 in. of concrete cover. Corrosion initiation can be expected to occur at an average age of 14.8 years for MMFX [ASTM A1035] steel. All three systems will exhibit significantly longer times to corrosion initiation in uncracked concrete." …." the average times to corrosion initiation in uncracked regions on bridge decks would be 26, 41, and 100 years for conventional, galvanized, and MMFX reinforcement, respectively, demonstrating that uncracked concrete provides excellent protection against chloride penetration. These values closely match those used in life-cycle models in which chloride penetration is based on diffusion through uncracked concrete."*

24. [Corrosion Resistant Alloy Steel (MMFX) Reinforcing Bar in Bridge Decks](#)– Michigan DOT Report R-1499– S. Kahl– September 2007 (36 pages): This report compiles the findings of Michigan DOT corrosion research and structural analysis (See Appendix B "MMFX Reinforced Bridge Deck LRFD Design Example") of bridges using MMFX "ChromX® 9100" (ASTM A1035 Type CS) reinforcing bars. The report states: "*...MMFX steel does exhibit corrosion resistance, higher yield strength, and a lower life cycle cost than epoxy coated reinforcement. Due to the high yield strength, MMFX use in bridge deck construction should be limited to structures that are designed in accordance with AASHTO LRFD code, and for 75 ksi steel reinforcement design yield strength…*" This statement was made after making an economic analysis comparing a MMFX (ASTM A1035 Gr 75) AASHTO LRFD design to an epoxy-coated rebar (ECR) design using Michigan's standard design (LFD).

25. [Corrosion Resistant Alloys for Reinforced Concrete - FHWA-HRT-07-039](#) -W. Hartt, R. Powers et. al.- July 2007 (135 pages): This test program included MMFX-II™ "ChromX® 9100" (ASTM A1035 Type CS), solid stainless steels 3Cr12 (UNS-S41003), 2201LDX (ASTM A955-98), 2205 (UNS 31803), and two 316L (UNS S31603) alloys; and two 316 stainless steel clad black bar products, black bar (ASTM A615) reinforcement included for comparison purposes. The report states: "*For black bar slabs, it was considered that active corrosion commenced once potential dropped to −280 mVSCE, at which point the average macro-cell current density was about 0.26 μA/cm^2. If it is assumed that this same current density denotes onset of active corrosion for the other reinforcement types as well, then the corresponding potentials are −390, −350, and −195 mVSCE for 3Cr12, MMFX-II™, and 2201, respectively. These potentials were achieved after 35 days (black bar), 64 to 140 days (3Cr12), 91 to 140 days (MMFX-II™), and 64 to 94 days (2201)."*

26. [Laboratory Evaluation of Corrosion Resistance of Steel Dowels in Concrete Pavement](#) –Final Report UCPRC-RR-2005-10 – FHWA No S/CA/RI-2006/27 – M. Mancio, J. Harvey, et al.- January 2007 (127 pages): This pavement dowel corrosion report indicates that Microcomposite (MMFX 2 "ChromX® 9100") pavement dowel bar had approximately 35 times the polarization resistance of carbon steel dowels. The report makes the following recommendations: *"It is recommended that the use of stainless steel-clad, hollow stainless steel, or microcomposite [MMFX 2] steel dowels be considered for locations with high risk of high chloride exposure (such as on mountain passes and marine environments) ..."* Furthermore, the report states**: "***Epoxy dowels present some risk of corrosion, primarily localized at holidays and the ends of the bars." The report notes that: "Bar ends should be coated with epoxy and care must be taken with epoxy-coated dowels during shipping, storage, and installation. Corrosion will be exacerbated if the bar ends are not coated (observed on various Caltrans construction sites) or if the coated ends are damaged during storage, transport, and installation***."**

27. "[Comparative Performance of MMFX Microcomposite Reinforcing Steel and Other Types of Steel with Respect to Corrosion Resistance and Service Life Prediction in Reinforced Concrete Structures](#)"– AMEC

Earth & Environmental- D. Morgan-- June 2006 (48 pages): This report makes the following conclusion after evaluating 14 studies and reports concerning the corrosion resistance properties of MMFX "ChromX® 9100") (Microcomposite) Steel reinforcement and other products: *"Studies evaluated in this report indicate that MMFX corrosion resistance is similar to or better than that of certain stainless steels such as 2101 and 3Cr12. … stainless steels (i.e. SS304 and SS316 series) appear to be more effective than MMFX for use in bridge and other structures exposed to chlorides, the lack of availability in North America of many the types of stainless steel evaluated, and their high costs compared to MMFX, make them less attractive from a life-cycle cost perspective for most applications."*

28. Summary Report on the Performance of Epoxy-Coated Reinforcing Steel in Virginia- VTRC Report 06-R29- R. Weyers, M. Sprinkel, M. Brown- June 2006 (37 pages): This report based on 14 years of research by VTRC of corrosion resistant reinforcing steel alternates states: *"because ECR cannot provide adequate corrosion protection for structures designed for a 100-year+ service life as currently recommended by FHWA, the report recommends that the Virginia Department of Transportation amend its specifications regarding the use of ECR to require the use of corrosion-resistant metallic reinforcing bars such as MMFX 2 ["ChromX® 9100"], ." Based on the times to cracking, MMFX2 reinforcement is worth 5 times more than ECR."* This study reiterated some of the findings of Hansson, Haas et al (ref 5.A.40) concerning epoxy-coated rebar (ECR)

29. Evaluation of corrosion resistance of different steel reinforcement types - Final Report - Iowa State University Bridge Center - CTRE Project 02-103 - May 2006 (75 pages): Voltage and current results from field monitoring of a instrumented bridge constructed half with MMFX steel and half with ECR indicated: 1. The MMFX half remained within the normal range at less than 100mV; appeared to have no ongoing corrosion activity. 2. In contrast, ECR had readings that were two times greater than MMFX, close to 200 mV. This led to the report's speculation that defects in the coatings had occurred during construction.

30. ASM Handbook, Volume 13C, Corrosion: Environments and Industries -Corrosion in Bridges and Highways –ASM International – J. Tinnea, W. Hartt, F. Pianca; et. al.- 2006 (39 pages): This handbook chapter discusses the various aspects of corrosion associated with bridge structural elements in corrosive environments and describes alternative corrosion-resistant reinforcement systems. ASTM A1035 Type CS (MMFX 2 Steel "ChromX® 9100") is noted as having the same Cl^-/OH^- ratio of 4.9, as 316 stainless steel cladded reinforcement, as a measure of its corrosion resistance. (Copy of this reference is available from the American Society for Metals - ASM).

31. Evaluation of MMFX Steel For NCDOT Concrete Bridges– FHWA/NC/2006-31, NCDOT Report 2004-27– S. Rizkalla, P. Zia et. al.– December 2005 (131 pages): This publication states the following conclusions based on testing of full scale bridge deck sections and corrosion tests at North Carolina State University: *"1. Substituting MMFX steel directly for Grade 60 steel in a design … is an overly-conservative approach. 2. MMFX steel [ASTM A1035 Type CS] can be used as the main flexural reinforcement for cast-in-place concrete bridge decks at a reinforcement ratio corresponding to 33% less than that required for Grade 60 steel. Therefore, a design of reinforced concrete bridge decks using MMFX steel may utilize an equivalent yield stress of 90 ksi for the MMFX steel bars. 3. Design of concrete bridge decks utilizing the high tensile strength characteristics of the MMFX steel should satisfy all minimum reinforcement ratios required by the AASHTO LRFD Bridge Design Specifications as well as the serviceability requirements of the specifications. 4. MMFX steel [ASTM A1035] has a much lower corrosion rate compared to conventional Grade 60 steel. Therefore, the use of MMFX steel could increase the service life of concrete bridges and lower repair costs."*

32. Corrosion Resistance of Duplex Stainless Steels and MMFX Microcomposite Steel for Reinforced Concrete Bridge Decks -University of Kansas Center for Research, Inc.- SM Report No.80- J. Ji, D. Darwin; et. al- December 2005 (507 pages): This report indicated that MMFX 2 "ChromX® 9100" (ASTM A1035 Type CS) bars had ½ the corrosion rate of conventional black bar and approximately 3 to 4 times black bars critical chloride threshold value. 2101 stainless steel (SS) was reported to have 1.35 times the corrosion rate of black bar in an un-pickled surface condition, with a slight higher corrosion rate as ASTM A1035 with a pickled surface condition.

33. New Technologies Proven in Precast Concrete Modular Floating Pier for U.S. Navy– PCI Journal – M. LaNier, P. Springston; et. al.- Jul-Aug 2005 (26 pages): This article notes that the Navy's Modular Hybrid Pier (MHP) project received Precast/Pre-stressed Concrete Institute's (PCI's) Henry N. Edwards award and updates - Preston Springston's ASCE paper. Project review procedures are discussed demonstrating why MMFX ASTM (A1035 Type CS) rebar was included in one of the project's two Navy MHP modules. The article noted that use of MMFX saved approximately $2.8 million over the original proposed design, while providing a 75-yr service life. MMFX's corrosion resistance performance was analyzed by the STADIUM computer model.

34. The Long Term Performance of Three Ontario Bridges Constructed with Galvanized Reinforcement – Ontario Ministry of Transportation– F. Pianca; H. Schell– June 2005 (29 pages): This report makes the following conclusions: *"Corrosion of galvanized reinforcing bars was initiated soon after the chloride corrosion threshold (for black steel) was reached…"* [and] *"…caused significant damage to the concrete, in the form of delamination and cracking." "…they* [galvanized bars] *do not provide effective long-term protection from corrosion."*

35. Surface Condition Effects on Critical Chloride Threshold of Steel Reinforcement - ACI Materials Journal 102- M12 – D. Trejo; R. Pillai– Mar–Apr 2005 (6 pages): This publication compared the critical chloride threshold level (CCTL) values of various uncoated steel reinforcement types using their as-received (with mill scale) and polished surface conditions. MMFX 2 "ChromX® 9100" (ASTM A1035 Type CS) was determined to have approximately 9 times the CCTL value of A615 in the as-received condition and approximately 12 times the CCTL of A615 in the polished condition. These values were determined using the accelerated chloride threshold (ACT) test procedure developed at Texas A&M University. (Copy of this paper may be obtained from the American Concrete Institute (ACI), Farmington Hills, MI)

36. Comparing the Chloride Resistances of Reinforcing Bars - Concrete International **- G. Clemeña, P. Virmani–- November 2004 (11 pages):** This article evaluates new, economical metallic reinforcement for its ability to withstand high salt concentration. The comprehensive study, on which the article is based shows that the chloride threshold of MMFX "ChromX® 9100" (ASTM A1035 Type CS) Microcomposite bars is about 5 to 6 times better than A615 steels and approximately 2 times chloride threshold of stainless steel 2101 LDX bars. (Copy of this paper may be obtained from the American Concrete Institute (ACI), Farmington Hills, MI)

37. Evaluation of Mechanical and Corrosion Properties of MMFX Reinforcing Steel for Concrete - University of Kansas Center for Research, Inc. Report No. FHWA-KS-02-8 - SM Report No. 70 – L. Gong, D. Darwin; et. al. - February 2004 (132 Pages): This report based on Southern Exposure corrosion testing, indicated that MMFX 2 "ChromX® 9100" (ASTM A1035 Type CS) has a corrosion rate of approximately 30% less, when compared to black (ASTM A615) steel bars.

38. Investigation of the Resistance of Several New Metallic Reinforcing Bar to Chloride-Induced Corrosion In Concrete -Virginia Transportation Research Council (VTRC) Report 04-R7 –G. Clemeña– December 2003 (27 pages): This report describes testing, analysis and recommendations concerning various metallic bars, including MMFX 2 "ChromX® 9100" (ASTM A1035 Type CS), that were found to be more durable and corrosion resistant than epoxy-coated rebar, with the program's investigation serving as the basis for an ACI Materials Journal paper co-authored by Dr. Gerardo Clemeña of the VTRC and Dr. Y. Paul Virmani of the FHWA. In conclusion, the report recommends MMFX 2 rebar for use by Virginia DOT in corrosive environments.

39. Appraisal Report High Corrosion Resistance MMFX Microcomposite Reinforcing Steels CIAS (Concrete Innovations Appraisal Service) Report 03-2– P. Zia, T. Bremner, M. Malhotra, M. Schupack, P. Tourney– May 2003 (50 pages): This document reports on the findings of the CIAS's MMFX corrosion panel concluding that MMFX 2's "ChromX® 9100" corrosion resistance provides a longer service life and is more cost effective than A615 reinforcement.

40. Corrosion Protection Strategies for Ministry Bridges - Final Report Amended - University of Waterloo -C. Hansson, R. Haas, R. Green, R. C. Evers, O. Gepraegs, and R. Al Assar- July 2000 (210 pages): This report states: "*Major concerns exist with the inability of maintaining a flaw-free coating on ECR during handling, placement and compaction of the concrete, and with disbondment of the coating ... In turn, concern exists that this provides easy access to chlorides and, thus, allows corrosion at flaws and along the bar under the dis-bonded coating. … There is additional concern regarding the difficulty of monitoring the condition of ECR and of repair/rehabilitation cycles over the 75 years." …. "The conclusion is that options involving ECR present no cost or performance advantages over Black Steel Reinforcement. …the further use of ECR is not recommended on the basis of both technical and life cycle cost analysis.*"

B. Structural Test Reports, Papers and Analysis References

1. Design, Installation, and Condition Assessment of a Concrete Bridge Deck Constructed With ASTM A1035 CS No. 4 Bars- Final Report VTRC 17-R16 - A. Salomon; C. Moen- June 2017 – 21 pages: This report indicated: *Recently developed corrosion-resistant reinforcing structural design guidelines were used to design, construct, and assess a reinforced concrete bridge deck with high-strength ASTM A1035 CS steel bars. The bridge replacement is located along the North Scenic Highway over the Wolf Creek in Bland County, Virginia. The bridge deck design used the higher yield stress available from ASTM A1035 CS steel to replace No. 5 bars with No. 4 bars that saved 23% by weight of steel in the deck and reduced reinforcement bar congestion, especially near the traffic barrier-bridge deck splice.* This project's design yield strength was 75 ksi in the positive moment region and 83 ksi in the negative moment region

2. **Anchorage Strength of Reinforcing Bars with Standard Hooks - University of Kansas Center for Research, Inc. - SM Report No. 125 – A. Ajaam, D. Darwin and M. O'Reilly - April 2017 (372 pages):** This report that included testing of ChromX® 9100 bars indicated:

The following conclusions are based on the data and the analysis presented in the report:

1. The provisions in ACI 318-14 for the development length for hooked bars overestimate the contribution of concrete compressive strength and bar size on the anchorage strength.

2. The incorporation of the modification factors based on concrete cover and confining reinforcement in the current Code provisions for development length overestimate the anchorage strength of hooked bars, particularly for large hooked bars and closely-spaced hooked bars.

3. The contribution of concrete compressive strength on the anchorage strength of hooked bars is best represented by the concrete compressive strength to the 0.295 power. Compressive strength to the 0.25 power works well for design.

4. The anchorage strength of hooked bars increases with an increase in the amount of confining reinforcement, even for confining reinforcement below the value required by ACI 318-14 to reduce development length by 20 percent.

5. Hooked bars with a center-to-center spacing below six bar diameters exhibit lower anchorage strengths than hooked bars with wider spacing. The reduction in anchorage strength of closely-spaced hooked bars is a function of the spacing between the hooked bars and amount of confining reinforcement.

6. The straight portion of hooked bars contributes to anchorage strength of hooked bars even at failure.

7. For hooked bars with a bend angle of 90°, at peak load, confining reinforcement provided in form of hoops within the joint region generally exhibit the greatest strain at the hoop closest to the straight portion of the bar, with strains decreasing as the distance from the bar increases. For hooked bars with a bend angle of 180°, at peak load, the hoop adjacent to the tail extension of the hooked bars exhibits the greatest strain; the strains in hoops above and below the hoop with the highest strain decrease as the distance from the hoop with the highest strain increases.

8. The anchorage strength of staggered hooked bars can be represented by considering the minimum spacing between hooked bars.

9. Hooked bars anchored in beam-column joints with a ratio of beam effective depth to embedment length (d/ℓ_{eh}) *greater than 1.5 exhibit low anchorage strengths.*

10. The amount of confining reinforcement provided above the joint region, within a range of 0.25 to 1.29 times the area of the hooked bars, does not affect the anchorage strength of the hooked bars within the

joint region.
11. The proposed provisions for ACI 318 provide conservative criteria for the development length of reinforcing bars anchored with standard hooks for reinforcing steel with yield strengths up to 120,000 psi and concrete with compressive strengths up to 16,000 psi.

3. **Anchorage Strength of Standard Hooked Bars in Simulated Exterior Beam Column Joints - University of Kansas Center for Research, Inc. - SM Report No. 124 – S. Yasso, D. Darwin, M. O'Reilly- April 2017 (330 pages):** This report that included testing of ChromX® 9100 bars indicated:
Based on the current study, the following conclusions are drawn:
1. Front failure was the dominant failure mode for specimens containing more than two hooked bars.
2. The anchorage strength of hooked bars in joints with three or four bars decreased for values of center-to-center spacing below seven bar diameters. The addition of confining reinforcement mitigated but did not eliminate this effect.
3. The modification to the descriptive equation by Sperry et al. (2015b, 2017b) to calculate the anchorage strength of two widely-spaced hooked bars to account for the effect of low spacing between hooked bars provides a reasonable representation of the anchorage strength of closely-spaced hooked bars.
4. As the force per bar decreased as the number of bars within a given width increased, the total anchorage force for the hooked bars in the simulated beam-column joints remained constant or increased moderately as the number of hooked bars increased.
5. Placing hooked bars outside the column core results in a significantly lower anchorage strength than placing hooked bars inside the column core. In this study, the reduction ranged from 4 to 34%, producing an average anchorage strength equal to about 84% of the average strength of hooked bars placed inside the column core.
6. For hooked bars are placed outside the column core, the dominant failure mode was front failure for No. 5 (No. 16) bars and side failure for No. 8 and No. 11 (No. 25 and No. 36) hooked bars.
7. Hooked bars anchored halfway through the column depth exhibit reductions in anchorage strength compared to those anchored at the far side of the column, with front failure as the dominant mode of failure for all specimens.
8. Hooked bars extended to the far side of the column in simulated beam-columns joints exhibit reduced strength where the ratio of effective depth to the embedment length is greater than 1.5 compared to specimens where the ratio of effective depth to the embedment length less than 1.5.
9. The anchorage strength of hooked bars with a 90 bend angle is not affected by tail kickout at failure or hook tail covers as low as 0.75 in. (19 mm). The likelihood of tail kickout increases with increasing bar size and for hooks with tail cover less than 2 in. (50 mm) and no confining reinforcement.
10. The proposed descriptive equations for anchorage force and design expressions for development length that include the spacing modification factor account for the spacing effect between hooked bars.
11. A development length modification factor of 1.25 accounts for lower anchorage strength resulting from placing hooked bars outside the column core or not extending hooked bars to the back of the column.
12. Hooked bars not in a beam-column joint with side cover more than 7db behave similarly to those inside the column core of a beam-column joint.
13. The proposed design provisions show a better correlation with the experimental results and less scatter than those in ACI 318-14.

4. Defining Structurally Acceptable Properties of High-Strength Steel Bars through Material and Column Testing Part II: Column Testing Report – Univ. of Texas, Austin – CPF Research Grant Agreement #05-14 - D. Sokoli, W. Ghannoum, et.al. – February 2017 (219 pages): The report indicated: *"Four column tests were conducted in this study that was part of a broader research effort aimed at setting the minimum acceptable T/Y ratios and elongations in new ASTM specifications for seismic grade 80 and 100 reinforcing bars. Three of the specimens were reinforced with grade 100 bars produced by different manufacturers and therefore having different mechanical properties. The fourth column was reinforced with conventional grade 60 ASTM A706 bars. Column specimens were tested under constant axial load and reverse cyclic lateral loading of increasing amplitude until fracture of longitudinal bars."*

"ASTM A1035 grade 100 longitudinal bars braced using A1035 grade 100 hoops and cross-ties exhibited much less pronounced buckling than grade 100 bars having a yield plateau and a shallower stress/strain hardening slope. This was attributed to the higher buckling capacity in the inelastic hardening range of the A1035 bars compared with that of other grade 100 bars having shallower inelastic tangent modulus of elasticity. Additionally, A1035 cross-ties did not open up as much at their bends during testing as other grade 100 bars did, thereby restraining longitudinal bars more effectively. By sustaining significantly less buckling amplitudes than other grade 100 bars, A1035 bars likely incurred less strain concentrations due to buckling. Results therefore indicate that the rounded hardening shape of the stress-strain curve of A1035 bars may have offset any detrimental effects on column deformation capacity that their lower elongation capacities may have had."

5.**Defining Yield Strength for Nonprestressed Reinforcing Steel – ACI Structure Journal - 113-S16 – C. Paulson, J. Rautenberg, S. Graham, & D. Darwin - Jan-Feb 2016 (16 pages)**: This ACI article, which includes concrete reinforcing bars such as ASTM A1035/AASHTO MP 18 Grade 100 states: "*Analytical strengths of reinforced concrete beams and columns incorporating reinforcing steel stress-strain curves with and without a sharp yield plateau* [i.e. ASTM A1035] *and realistic representations of the nonlinear stress-strain behavior of concrete were compared with strengths obtained using an idealized elastic-plastic stress-strain curve and equivalent concrete stress block, as permitted by the ACI Building Code."*
"The comparisons demonstrate that: 1. For the practical range of reinforcement ratios in beams (below three-fourths of the balanced ratio), realistic stress-strain curves representing both sharply yielding and gradually yielding reinforcement produce analytical strengths that equal or exceed the corresponding Code-calculated nominal strengths. 2. For columns, the greatest sensitivity of column strength to steel stress-strain curve shape occurs at the point of maximum nominal bending capacity. At this combination of bending moment and axial load, columns with compressive strengths of 5000 and 8000 psi (34 and 55 MPa) and total reinforcement ratios of 1 or 2% exhibit little impact of the stress-strain relationship selected. At the same reinforcement ratios combined with a concrete compressive strength of 12,000 psi (83 MPa), the analytical strengths are at least 97% of the Code-calculated strength. The relative strengths drop as the reinforcement ratio increases. The lowest ratio of analytical to Code-calculated strength for columns containing reinforcement with yield strength defined based the 0.2% offset method (93%) occurs at total reinforcement ratios of 6 to 8%; in all cases, the lower relative strengths occur under combinations of moment and axial load where the strength reduction factor is 0.65 for tied columns and 0.75 for spirally reinforced columns, thus maintaining an adequate margin of safety. 3. Use of the 0.2% offset method to define the yield strength of gradually yielding reinforcing steel is safe and realistic."

6. Plasticity Spread in Columns Reinforced with High Strength Steel - 16th World Conference on Earthquake, 16WCEE 2017 Santiago Chile, - Paper N° 234 - D. Sokoli, A. Limantono, M. Ghannoum - Jan. 2017 (11 pages): This paper indicated: *Test results therefore indicate that higher grade reinforcing bars, up to Grade 100 (100 ksi specified yield strength (690 MPa)) may be suitable for use in both longitudinal and transverse reinforcement in regions of high seismicity.*
A limit in the form of shear stresses or member minimum length is advised for higher strength bars to avoid bond failures. Accounting for the loss of bond strength within plastic hinge regions is also advised for all bar grades. Higher strain demands were observed in higher strength longitudinal bars. This trend was attributed in part to differences in the tensile-to-yield strength (T/Y) ratios of various bars used in the experimental program. T/Y ratio of the bars used in this study dropped as the bar strength increased. Lower T/Y ratios reduce the ability of bars to spread their inelastic strains away from flexural crack locations, which results in amplified peak strain demands. The higher strain demands on higher-strength bars in turn raised concerns about their low-cycle fatigue behavior and possible premature fractures in application with high strain-cycle demands.

7. Steel Reinforcement Maximum Design Yield Strength Presentation 2016 - Florida DOT -Steve Nolan – (57 pages): This presentation reviewed various types of High Strength Reinforcing (HSR) bars which are included in FL Dot's Standard Specification 931-1 *Reinforcement Steel (for Pavement Structures)*. ASTM A1035 Grade 100 bars HSR bars were part of this presentation's analysis. The presentation used the provisions for use of 100 ksi reinforcing steel (for Seismic Zone 1) AASHTO-LRFD Bridge Design

Specifications (BDS) to analyze use of bars with 100 ksi design yield strengths, along with BDS provisions for use of 10 ksi –15 ksi concrete.
The presentation stated that design benefits using HSR bars included:
•*Increased Flexural and Shear Strength;*
•*Reduced congestion;*
•*Reduced transportation and placement cost;*
•*Many high strength reinforcing materials also have improved durability properties.*

8. Continuous Transverse Reinforcement—Behavior and Design Implications - ACI Structural Journal, V. 113, No. 5, B. Shahrooz, M. Forry, N. Anderson, H. Bill, A. Doellman - Sep-Oct 2016 (10 pages): This paper, which included ASTM A1035 longitudinal bars used to prevent premature flexure failure, as shear was the intended failure mechanism stated: *Evaluation of the experimental data from full-scale members and subassemblies showed current design provisions and detailing practices are applicable to CTR* [continuous transverse reinforcement]*, and CTR can be used in lieu of conventional transverse reinforcement. Members and connections reinforced with CTR have better post-peak behavior, and generally higher strength and stiffness than comparable elements with conventional transverse steel. Nevertheless, the direction of applied torque in comparison to the direction in which CTR "spirals" is an important consideration and impacts the member performance.*
The following conclusions and recommendations were made regarding the performance of CTR:

a) *CTR provides the same level of shear performance as conventional U-shaped stirrups. The post-failure shear response is improved as a result of the enhanced confinement that CTR provides.*
b) *Well-established design procedures are applicable to members using CTR.*
c) *The torsion design provisions as stated in the Code* [ACI 318 Building Code] *are applicable to members using CTR without modification.*
d) *The torsional strength with CTR is as good as, and sometimes exceeds, that obtained from conventional transverse reinforcement. However, the strength is reduced if the diagonal cracks due to torsion are in the same direction as how CTR "spirals."*
e) *For cases where the torque direction can alternate, the member strength needs to be limited to the cracking torsional strength.*
f) *Available interaction equations for the combined actions of bending, shear, and torsion are apparently adequate for computing strength when CTR is used.*
g) *Current equations for axial load design of columns are applicable to columns using CTR. Conventional column ties and CTR achieve the same strength, with the latter offering constructability advantages.*
h) *Exterior beam-column connections with conventional seismic hoops and CTR performed equally well. The current seismic design provisions and detailing requirements in ACI 318 are applicable to CTR.*

9. Setting Bar-Bending Requirements for High-Strength Steel Bars (RGA#01-15) - Charles Pankow Research Grant #01-15 – W. Ghannoum, S. Zhao (Univ. Texas – Austin) – June 2016 (102 pages): - This paper that included results of testing of ASTM A1035 bars indicated: *Bend/re-bend tests were conducted on reinforcing bars with yield strengths ranging from 60 ksi to approximately 120 ksi. Strain aging tests were also conducted on the bars to ensure that the bar bends were re-bent after most of the strain aging embrittlement effects had occurred. The bend/re-bend test variables were: bar grade, bar manufacturing process, bar diameter (db), and bend inside diameter.*
U.S. Bar sizes were #5, #8, and #11 and the bars were bent to meet the minimum specified ACI 318-14 bend diameters for each of the sizes. Performance measures used to quantify the performance of bends included:

a) *The remaining bend angle at fracture during re-bending* (θ_b)
b) *The axial strain at fracture normalized by the bar uniform elongation strain* ($\varepsilon_b / \varepsilon_{un}$)
c) *The axial stress at fracture normalized by the bar yield strength* (f_{ub} / f_y)
d) *The axial stress in bar at fracture normalized by the bar tensile strength* (f_{ub} / f_t)

Overall, for all bar sizes and types, as bar strength (or grade) increased, bend performance decreased as demonstrated by lower stresses, strains, and changes in the bend angle at fracture during re-bending. Moreover, for all bar types, as the bend inside diameter increased, bend performance was seen to improve.
Overall, bends in #8 and #11 bars were found to perform adequately at the current ACI 318-14 minimum bend diameters for all grades.

Bends in #5 bars showed significantly varied performance. Grade 60 #5 bars, bent to achieve a target inside diameter of 4db, were able to reach stresses close to yield prior to fracture during re-bending. Bends in grade 80 and 100 bars, however, only reached fractions of their yield strength during re-bending when bent to achieve a 4db inside diameter. The performance of bends in higher grade #5 bars reached larger stresses and strains as the bend diameter was increased, with grade 80 bars reaching stresses close to their yield with 5db bends and grade 100 bars reaching yield strengths with bend diameters of 6db.
A615 and A706 #5 and smaller transverse bars were recommended to be bent to a bend diameter ratio of at least 5.0 for grade 60 and 80 and 6.0 for grade 100. Grade 100 A615 longitudinal bars are recommended to be bent to a ratio of at least 9.0 for #9 to #11 bars and 8.0 for #6 to #8 bars.

10. Defining Yield Strength for Nonprestressed Reinforcing Steel – ACI Structure Journal - 113-S16 – C. Paulson, J. Rautenberg, S. Graham, D. Darwin - Jan-Feb 2016 (16 pages): This ACI article, which includes concrete reinforcing bars such as ASTM A1035/AASHTO M 334 Grade 100 states: "*Analytical strengths of reinforced concrete beams and columns incorporating reinforcing steel stress-strain curves with and without a sharp yield plateau* [i.e. ASTM A1035] *and realistic representations of the nonlinear stress-strain behavior of concrete were compared with strengths obtained using an idealized elastic-plastic stress-strain curve and equivalent concrete stress block, as permitted by the ACI Building Code."*
"The comparisons demonstrate that: 1. For the practical range of reinforcement ratios in beams (below three-fourths of the balanced ratio), realistic stress-strain curves representing both sharply yielding and gradually yielding reinforcement produce analytical strengths that equal or exceed the corresponding Code-calculated nominal strengths. 2. For columns, the greatest sensitivity of column strength to steel stress-strain curve shape occurs at the point of maximum nominal bending capacity. At this combination of bending moment and axial load, columns with compressive strengths of 5000 and 8000 psi (34 and 55 MPa) and total reinforcement ratios of 1 or 2% exhibit little impact of the stress-strain relationship selected. At the same reinforcement ratios combined with a concrete compressive strength of 12,000 psi (83 MPa), the analytical strengths are at least 97% of the Code-calculated strength. The relative strengths drop as the reinforcement ratio increases. The lowest ratio of analytical to Code-calculated strength for columns containing reinforcement with yield strength defined based the 0.2% offset method (93%) occurs at total reinforcement ratios of 6 to 8%; in all cases, the lower relative strengths occur under combinations of moment and axial load where the strength reduction factor is 0.65 for tied columns and 0.75 for spirally reinforced columns, thus maintaining an adequate margin of safety. 3. Use of the 0.2% offset method to define the yield strength of gradually yielding reinforcing steel is safe and realistic."

11. Innovative Modular High Performance Lightweight Decks for Accelerated Bridge Construction - Florida International University- S. Ghasemi (Master's Thesis)– November 2015 (160 pages) –This program is based on experimental work using Ultra-High Performance Concrete (UHPC) deck reinforced with High Strength Steel (HSS – ASTM A1035 Grade 100) bars. The test program was compared to an ABAQUS finite element analysis for UHPC slab reinforced with HSS rebars. Program conclusions included:
"In a two-step optimization process, both the size and the reinforcement of the deck were modified, reducing the weight by over 37%. Test results showed that the optimized section can suitably meet the load demand, ductility, and serviceability requirements, while staying within the weight limits for movable [ABC Accelerated Bridge Construction] bridges.

12. MMFX Steel Alternative to Post-Tensioning for Pier Cap –eConstruct USA -A. Girgis, A. Sevenker, M. Tadros- July 2015 (10 pages): This report indicated: "*The results of the study given below demonstrate that use of ASTM A1035 steel reinforcing bars supplied by MMFX result in several advantages including simplified construction steps, reduced pier cap weight and concrete quantities and reduced reinforcement cost."*
"The alternative design resulted in a much lighter and prismatic section which will significantly improve the production and construction cost." (As noted in Table 1)

Table 1. Total material quantities of flexural steel and concrete

Item	Original Design		Alternative Design - MMFX Conforming to ASTM A1035	
	Weight (tons)	Volume (Yard^3)	Weight (tons)	Volume (Yard^3)
Concrete	123	61	58	29
	Weight (tons)	Grade (Ksi)	Weight (tons)	Grade (Ksi)
Flexure Reinforcement	2.8	270	2.7	100
Shear Reinforcement	5.6	60	3.5	75
Ledger Reinforcement			2.2	75

13. Anchorage of High-Strength Reinforcing Bars with Standard Hooks - Univ. of Kansas SM Report No. 111 -J. Sperry, S. Yasso, N. Searle, M. DeRubeis, D. Darwin, M. O'Reilly, A. Matamoros, L. Feldman, A. Lepage, R. Lequesne; A. Ajaam- June 2015 (266 pages): This study was based on testing of hooked ASTM A615 Grade 60, ASTM A615 Grade 80 and A1035 Grade 120 bars. The report indicated: "*The results of this study show that current ACI 318-14 code provisions are conservative for larger hooked bars and higher compressive strength concrete. The effect of concrete compressive strength on the anchorage capacity of hooked bars is less than represented by the 0.5 power currently used in ACI provisions; the 0.25 power provides a more realistic estimate of capacity. The addition of confining transverse reinforcement in the hook region increases the anchorage capacity of hooked bars–the value of the increase depends on the quantity of confining reinforcement per hooked bar. Hooked bars with 90° and 180° bend angles exhibit similar capacities, and no increase in capacity was observed when increasing side cover from 2.5 to 3.5 in. Anchoring a hooked bar outside the column core or outside the compressive region of a column provides less capacity than anchoring the hooks at the far side of a beam-column joint or in a wall with a high side cover. Hooked bars also exhibit a reduction in capacity if the center-to-center spacing is less than seven bar diameters.*"

14. Structural Design Guidelines for Concrete Bridge Decks Reinforced with Corrosion-Resistant Reinforcing Bars -VCTIR 15-R10 -A. Salomon; C. Moen –October 2014 (49 pages): The findings of this experimental structural report indicate: "*This research program develops and validates structural design guidelines and details for concrete bridge decks with corrosion-resistant reinforcing (CRR) bars. A two-phase experimental program was conducted where a control test set consistent with a typical Virginia Department of Transportation bridge deck design using Grade 60 steel (ASTM A615, f_y = 60 ksi) and epoxy-coated reinforcing steel was compared to deck slab specimens where Grade 60 is replaced with CRR bars.*

The experimental program was designed to evaluate how flexural performance at service and ultimate limit states are affected by a one-to-one replacement of Grade 60 with CRR bars, a reduction of concrete clear cover, and a reduction in rebar size. Structural analysis models were developed using Response 2000 in order to predict the CRR bridge deck moment-curvature and the moment-crack width relationships."

Program conclusions include:
• For ASTM A1035 and UNS S32304 specimens, a decrease in bar size and clear cover (2.0 in instead 2.50 in) proved to have similar deformability ratios and crack widths that comply with current AASHTO requirements, with as much as 36% less steel. Bridge deck slabs employing high strength rebar without a defined yield plateau can still provide ductility consistent with AASHTO and ACI ductility limits at an ultimate limit state.
• CRR bridge deck designs can be identified that meet current code serviceability and strength requirements, with the added benefit of corrosion resistance, by using programs like Response 2000.

15. Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures -NIST GCR 14-

917-30 - NEHRP Consultants -D. Kelly, A. Lepage, D. Mar, J. Restrepo, J. Sanders, A. Taylor – March 2014 (231 pages): Applications for use of ASTM A1035 Grade 100 and 120 bars were included as a part of this; high strength seismic research and study program. The report concluded*: "Based on the research reviewed and the studies performed, it was observed that concrete members reinforced with reduced amounts of high-strength reinforcement (with yield strengths of 80 ksi or stronger) are capable of reaching comparable strength and deformation capacity to those achieved by members reinforced with conventional strength reinforcement. This observation applies to members having reinforcement details that provide concrete confinement and inhibit brittle failures related to shear, bond stress, and bar buckling*.
Use of high-strength reinforcement can result in cost reductions and improved constructability. Benefits include reduced reinforcement quantity, reduced reinforcement congestion, improved placement of concrete, and accelerated reduced construction schedule. The cost benefits will not be fully achieved in the United States without the associated increase in production, however, until there is increased demand for high-strength reinforcement.

16. [AASHTO LRFD Bridge Design Specifications 7th Ed - 2016 Interim Revisions] – **American Association of State Highway and Transportation Officials (AASHTO)**: Following is a summary to the LRFD Bridge Design Specifications Section 5 "*Concrete Structures*" that include use of ASTM A1035 Gr. 100/AASHTO M 334 steel bars at yield strengths up to and including 100 ksi (690 MPa).

- 5.3 – Includes higher yield strengths as part of f_y. Added definitions of ε_{cl} and ε_{tl}-compression- and tension-controlled strain limits
- 5.4.3.1, 5.4.3.3, 5.5.4.2.1, 5.7, 5.10.11.1 – Permits reinforcing bars with minimum yield strengths up to 100 ksi (690 MPa) in non-seismic elements.
- 5.5.3.2 - Revised fatigue equation to include 100 ksi (690 MPa) bars
- 5.5.4.2.1 - Modifies Eq. 5.5.4.2.1-1, and Figure C5.5.4.2.1-1 to use ε_{cl} and $\varepsilon_{tl,}$ (compression- and tension- controlled strain limits) in place of 0.002 and 0.005.
- 5.7.2.1 - Provides 100 ksi (690 MPa) bars compression- and tension-controlled strain limits of 0.004 and 0.008.
- 5.7.3.4 - Eq. 5.7.3.4-1 limits f_{ss} to 60 ksi (410 MPa). Indicates that only Class 1 requirement need to be satisfied with greater corrosion resistant reinforcement.
- 5.7.3.5 - Modifies strain limit for moment redistribution in structures using 100 ksi (690 MPa) bars.
- 5.7.3.2.5 - Limits steel stress in strain compatibility calculations to the specified minimum yield strength
- 5.7.4.6 - Allows spirals and ties made from 100 ksi (690 MPa) bars in non-seismic elements.
- 5.8.2.4, 5.8.2.5, 5.8.2.8 - Authorizes transverse reinforcement for 100 ksi (690 MPa) bars with flexural shear without torsion.
- 5.8.3.5 - Permits longitudinal reinforcing steel using 100 ksi (690 MPa) bars.
- 5.8.4.1 – Provides provisions for transverse reinforcing bars with yield strength greater than 60.0 (410 MPa) ksi, noting that f_y limited to 60.0 ksi (410 MPa) in Eq. 5.8.4.1.3.
- 5.10.2 - Approves hooks for 100 ksi (690 MPa) bars with transverse confining steel in non-seismic elements.
- 5.10.6.1 - Permits 100 ksi (690 MPa) bars in spirals used in non-seismic elements.
- 5.11.1.1 - Indicates that the development length equations are applicable to 100 ksi (690 MPa) bars.
- 5.11.2.1 - Necessitates transverse confining steel for development of bars with yield strengths above 75 ksi (515 MPa).
- 5.11.2.4 - Calls for the use of modification factors or ties for hooks with bar yield strengths greater than 60 ksi (410 MPa).
- 5.11.5 - Permits splices of 100 ksi (690 MPa) bars and provides transverse confining steel requirements.
- 5.11.5.3.1 Requires transverse confining steel of lap splices for bars with yield strengths exceeding 75 ksi (515 MPa).

(Document available through AASHTO (American Association of State Highway and Transportation Officials)

17. The Impact of High Strength Reinforcing Steel on Current Design Practice - Charles Pankow Foundation Research Grant Agreement #01-13– K. Price, D. Fields, L. Lowes –July 2013 (36 pages): This study reviewed comparative designs for high strength reinforcement (HSR) using ACI 318-11, for design strengths from 60 to 120 ksi (410 to 830 MPa), such as ASTM A1035 Gr 120. Design comparisons were made for flexural members (slabs and beams), gravity columns and structural walls.
The report stated: "*The results of this study show that if ACI Code limits on steel yield strength are ignored, HSR can be used to reduce steel volumes for RC building components. However, results show also that for some components, there is a limit beyond which increasing the yield strength of the reinforcing steel does not result in reduced steel volume due to serviceability requirements.*"

The report's following table lists the recommended maximum reinforcement yield strength for use in design of the components included in the study.

Recommended Maximum Steel Yield Strength for Use in Design

Component	Recommended Maximum Fy	Note
One-Way Slabs	Longitudinal– 80 ksi	Use of 120 ksi steel requires the use of a larger number of smaller bars (No. 3) to meet maximum flexural crack width limits.
Two-Way Slabs	Longitudinal– 60 ksi	Use of HSR requires the use of a larger number of smaller bars (No. 3) to meet maximum flexural crack width limits.
Beams	Longitudinal – 120 ksi Shear– 60 ksi	
Gravity Columns, low axial & high shear and moment demand	Longitudinal– 120 ksi Confinement– 100 ksi Shear– 120 ksi	Recommendations are appropriate for columns with axial load less than 0.2 $f'_c A_g$ subjected to high shear demand.
Gravity Columns, high axial & low shear and moment demand	Longitudinal– 60 ksi Confinement– 100 ksi Shear– 60 ksi	Recommendations are appropriate for columns with axial load greater than 0.3 $f'c A_g$ subjected to low shear demand.
Structural Walls	Longitudinal– 120 ksi Confinement– 120 ksi Shear– 120 ksi	Recommendations are appropriate for walls with axial load less than 0.2 $f'_c A_g$ subjected to shear demand in excess of $4 A_{cv} \sqrt{f'_c}$ psi.

18. Interim 5 Year Report Alternative Dowel Bar Material Study -WAY-30-11.5 -Ohio DOT – March 2013 (11 pages): This report concerning MMFX "ChromX® 9100" (ASTM A1035) and three other dowel bars types indicated: "*After five years, the epoxy coated steel dowels, the MMFX dowels, and the Lifejacket [zinc bonded] dowels have LTE [load transfer efficiency] greater than the 70% criteria typically used*" and that the FRP bars had LTEs that varied between 55 and 23%. Visual observation of the bars after taking joint core samples indicated: "*The MMFX bar was also in very good condition with slight pitting on the top of the bar in the area of the joint.*" It also noted under film corrosion occurred on some of the epoxy coated dowel bars, and that some of zinc bonded dowels were oxidized and slightly abraded.

19. Design Guide For Use Of ASTM A1035 High-Strength Reinforcement In Concrete Bridge Elements In AASHTO Seismic Zone 2 - S. K. Ghosh - S. K. Ghosh Associates Inc.– July 2012 (20 pages): This document is a supplement to Design Guide for Use of ASTM A1035 High-Strength Reinforcement in Concrete Bridge Elements with Consideration of Seismic Performance **– H. Russell, S. Ghosh, M. Saiidi- August 2011 (27 pages),** as AASHTO Seismic Zone 2 was not a part of the original design guide. This paper also analyzes the revisions voted to be included into the *AASHTO LRFD Design Specifications*, as result of the approval of AASHTO Committee T-10 "Concrete" Agenda Item (WAI 163) by AASHTO Subcommittee on Bridges and Structures (SCOBS) at their July 2012 meeting.

The following table summarizes the report's conclusions concerning the maximum yield strengths of reinforcement that may be used in the design of the different structural elements of bridges in AASHTO Seismic Zone 2.

Maximum Yield Strengths of Reinforcement for Use in Design in Seismic Zone 2

Yield Strength, ksi	Foundations			Columns/Walls		Decks	Beams/Girders		
	Abutments	Piles	Pile Caps	Vertical	Confinement	Top and Bottom	Tension	Compression	Shear
100	X	X	X	X (2)	X	X	X	X	X
75				X (2)					
60	X (3)	X (3)	X (3)	X (2)	X (3)				X (1)

(1) Yield strength limited to of 60 ksi for shear-friction calculations.
(2) Required shear strength must be calculated per Articles 8.3.2 and 8.6.1 of the *AASHTO Guide Specifications for LRFD Bridge Design* and minimum shear reinforcement must be provided per Article 8.6.5.
(3) Yield strength of transverse reinforcement limited to 60 ksi for shear strength computations.

20. High-Performance Steel Bars and Fibers as Concrete Reinforcement for Seismic-Resistant Frames - Hindawi Publishing Corporation -Advances in Civil Engineering -A. Lepage, H. Tavallali, S. Pujol, J. Rautenberg- February 2012 (13 pages): This paper presents the results of nonlinear cyclic response testing of concrete frame members with drift reversals up to 5%. The test members were reinforced with advanced high-strength steel (AHSS). Testing was conducted on concrete members longitudinally reinforced with steel bars 60 ksi (410 MPa), 97 ksi (670 MPa) and 120 ksi (830 MPa) along with a volume fraction of hooked steel fibers ranging between 0 and 1.5%.

The paper states: "The test data indicate that replacing conventional Grade-410 longitudinal reinforcement with reduced amounts of Grade-670 or Grade-830 steel bars did not cause a decrease in usable deformation capacity nor a decrease in flexural strength. The evidence presented shows that the use of advanced high-strength steel as longitudinal reinforcement in frame members is a viable option for earthquake-resistant construction.

a. *Replacing conventional Grade-410 longitudinal reinforcement with reduced amounts of AHSS reinforcement maintained flexural strength and did not decrease the usable member deformation capacity. The tested beams tolerated drift ratios in excess of 10% without failure while the column specimens tolerated drift ratios of 5% before failure. Column failures in RC specimens were due to buckling of the longitudinal reinforcement while failures in HPFRC specimens were due to fracture of the longitudinal reinforcement.*
b. *Increasing the spacing of Grade-410 transverse reinforcement from d/4 in RC specimens to d/2 in HPFRC specimens, did not reduce the member deformation capacity.*
c. *Reducing the amount of longitudinal reinforcement while increasing the yield strength of the reinforcement decreased the post cracking stiffness and increased the yield deformation of the member, leading to a reduction of the area inside the load deformation hysteresis loops. Reductions in the amount of longitudinal reinforcement achieved by reducing bar diameter may lead to increased vulnerability to bar buckling.*
d. *Nonlinear seismic analyses of SDOF systems with identical strength indicated that the mean ratio of calculated maximum displacements for systems representing RC with AHSS reinforcement to those calculated for RC with conventional Grade-410 reinforcement were about 1.1 for Grade-670 systems and 1.3 for Grade-830 systems.*
e. *These observations suggest that AHSS reinforcement is a viable option for frame members in earthquake-resistant construction. Additional studies are needed to investigate the nonlinear seismic response of multistory concrete frames reinforced with AHSS bars."*

21. Design Guide for Use of ASTM A1035 High-Strength Reinforcement in Concrete Bridge Elements with Consideration of Seismic Performance– H. Russell, S. K. Ghosh, M. Saiidi- August 2011– (25 pages)– This study evaluates the maximum strengths of reinforcement that may be used in the design of the different structural elements of bridges in Seismic Zones 1, 3 and 4. The report's conclusions are summarized the Table 1 (below), based on the information presented in this report, which differentiates

Seismic Zone 1 conclusions from Seismic Zones 3 and 4. It is noted that the application of high-strength reinforcing steel in bridges Seismic Zone 2 was beyond the scope of this study.

The report states: *"In view of the fact that (1) bridge decks, girders, and bent cap beam***s** *are capacity-protected, (2) satisfactory performance in field studies has been documented, and (3) research findings that the use of MMFX [ASTM A1035 Type CS Gr 100] bars in close proximity with black steel bars in chloride-contaminated environments does not lead to enhanced corrosion of the reinforcing bars, there should be no reservation about permitting MMFX reinforcing bars in bridge decks, girders, and bent cap beams in Seismic Zones 3 and 4, provided the guidelines in Part 1 of this report are followed.*
Pending further testing, MMFX steel should not be used as longitudinal reinforcement in bridge columns in Seismic Zones 3 or 4. The use of MMFX steel as transverse reinforcement in such members should be permitted, provided the transverse reinforcement yield strength is restricted to no more than 60 ksi for the purposes of computing shear strength. The full yield strength of the transverse reinforcement may be utilized for purposes of confinement of the concrete.
The above recommendation should also apply to pier walls, back walls, and wing walls, which are preferable locations for inelastic behavior in most bridges.
The welding of ASTM A1035 reinforcement should be prohibited in plastic hinge regions of columns and other structural members that are not capacity-protected in bridges located in AASHTO Seismic Zones 3 and 4, until approved procedures for butt-welding of hoops become available.
In general, MMFX reinforcing bars should be permitted to be used in any foundation element such as a footing or a Type II pile shaft in Seismic Zones 3 and 4 that is capacity protected, provided the guidelines in Part 1 of this report are followed.
When MMFX reinforcing bars are used as transverse reinforcement in pile shafts, the transverse reinforcement yield strength should be restricted to no more than 60 ksi for the purposes of computing shear strength. The full yield strength of the transverse reinforcement may be utilized for purposes of confinement of the concrete."

Table 1 Maximum Tensile Strengths of Reinforcement for Use in Design

Yield Strength, ksi	Foundations			Columns/Walls		Decks	Beams/Girders		
	Abutments	Piles	Pile Caps	Vertical	Confine-ment	Top and Bottom	Tension	Compres-sion	Shear
Non-Seismic (Zone 1)									
100	X	X	X	X	X	X	X	X	X
75									
60									X (1)
Seismic (Zones 3 and 4)									
100	X	X	X	N (2)	X	X	X	X	X
75				N (2)					
60	X (3)	X (3)	X (3)	N (2)	X (3)				X (1)

(1) Yield strength limited to of 60 ksi for shear-friction calculations.
(2) Not recommended.
(3) Yield strength of transverse reinforcement limited to 60 ksi for shear strength computations.
Note: Application of high-strength reinforcement in bridges located in Seismic Zone 2 was beyond the scope of this study.

22. Design of Concrete Structures Using High-Strength Steel Reinforcement - National Cooperative Highway Research Program -NCHRP Report 679 –B. Shahrooz, R. Miller, K. Harries, H. Russell– April, 2011 (Pages 83): This report, prepared as part of NCHRP Project 12-77, provides an evaluation of existing *AASHTO LRFD Bridge Design Specifications* relevant to the use of high-strength reinforcing steel. The report identifies aspects of reinforced-concrete design and of the AASHTO specifications that may be affected by the use of high-strength reinforcing steel. Experimental and analytical studies, conducted as part of the program, provide the background and engineering basis to support recommendations for changes to the specifications necessary for the use of high-strength reinforcing steel. The report includes proposed recommended language which specifically permits the use of high-strength reinforcing steel for yield strengths not greater than 100 ksi (690 MPa). This study did not address seismic applications and therefore, is limited in its application to Seismic Zone 1.

The report's supporting documents include the following: Appendix A—*Material Properties*, Appendix B—*Flexural Resistance of Members with Reinforcing Bars Lacking Well-Defined Yield Plateau*, Appendix C—*Strain Limits for Tension-Controlled/Compression-Controlled and Strains to Allow Negative Moment Redistribution*, Appendix D—*Flexure Specimens*, Appendix E—*Fatigue of High-Strength Reinforcing Steel*, Appendix F—*Shear Specimens*, Appendix G—*Analytical Studies of Columns*, Appendix H—*Beam Splice Specimens*, Appendix I—*Crack Control*, Appendix J—*Survey Instruments and Results*, Appendix K—*Design Examples*, Appendix L—*Proposed Changes to Section 5 of the AASHTO LRFD Specification*, Appendix M—*2010 AASHTO Bridge Committee Agenda Item.*

The following conclusions are made as part of this report; which cover the ensuing design aspects based on the program's experimental and analytical studies: A. Yield Strength **–** "*A value of yield strength, fy, not exceeding 100 ksi was found to be permissible without requiring significant changes to the specifications*." B. Flexure- *"All beam specimens met and exceeded their designed-for strength and ductility criteria and exhibited predictable behavior and performance similar to beams having conventional reinforcing steel"*. C. Fatigue- "*…tests…and a review of available published data demonstrate that presently accepted values for the fatigue or 'endurance' limit for reinforcing steel are applicable, and likely conservative, when applied to higher strength bars*." D. Shear – "*The use of current specifications procedures for calculating shear capacity were found to be acceptable for values of shear reinforcement yield fy ≤ 100 ksi.* E. Shear Friction - "*…restriction that fy be limited to 60 ksi when calculating shear friction capacity must be maintained regardless of the reinforcing steel used."* F. Compression - "*Results indicate the current specifications requirements for both longitudinal and transverse reinforcement design in compression members are applicable for fy ≤ 100 ksi*." G. Bond and Development **–** "*…it is recommended that development, splice, and anchorage regions be provided with cover and confining reinforcement based on current design requirements when high-strength bars are used."* H. Serviceability—Deflections and Crack Widths - "*Based on the results of the flexural tests conducted in this study, deflections and crack widths at service load levels were evaluated. Both metrics of serviceability were found to be within presently accepted limits and were predictable using current specifications provisions*."

23. Drift Capacity of Concrete Columns Reinforced with High-Strength Steel- Purdue University- J. Rautenberg (Thesis-Dissertation)- February 2011 (289 pages): This paper describes the testing and results with columns constructed using combinations of reinforcing bars with the following yield stresses:

A. Longitudinal reinforcement: -1. 60 ksi (410 MPa) (ASTM A706), 2. 80 (550 MPa) ksi (ASTM A706), 3. 120 ksi (830 MPa) (ASTM A1035)
B. Transverse reinforcement: -1. 60 ksi (410 MPa) (ASTM A615), 2. 120 ksi (830 MPa) (ASTM A1035), 3. 180 ksi (1240 MPa) (JIS G3137) [Japanese Industrial Standard]

Following are conclusions from this research:

- *For axial loads below the balanced point, flexural strength is nearly linearly proportional to the product of the reinforcement ratio and the yield stress of the longitudinal reinforcement. As a consequence, a column reinforced with Gr.-60 steel has approximately the same flexural strength as a column with half as much Gr.-120 steel.*
- *The drift capacity of a column reinforced with Gr.-80 or Gr.-120 longitudinal steel is comparable to the drift capacity of a similar column reinforced with Gr.-60 steel. All columns with transverse reinforcement spaced at d/4 [effective depth] had drift capacities between 4 and 8%.*
- *Numerical simulations indicate that, for a given ground motion, a multi-story moment-frame building with columns reinforced with Gr.-60 longitudinal steel will experience approximately the same mean roof drift ratio as a similar building with columns reinforced with half as much Gr.-120 longitudinal reinforcement.*
- *Specimens reinforced with Gr.-120 or Gr.-180 hoops spaced at d/2 had computed shear strengths that were comparable to specimens reinforced with Gr.-60 hoops. But, the specimens with high-strength hoops (spaced at d/2) exhibited inclined cracks not crossing any hoops that led to shear failures (after flexural yielding) at drift ratios between 3 and 4%.*

24. Behavior of Concrete Beams Reinforced with ASTM A1035 Grade 100 Stirrups under Shear– ACI Structural Journal– A. Munikrishna, A Hosny, S. Rizkalla, P. Zia – Jan-Feb 2011 (9 pages): This paper stated the following conclusions based on test of large beams reinforced with high-strength longitudinal and transverse steel:

1. *The shear strength of flexural members can be achieved by using a lesser amount of high-strength stirrups and a lower high-strength longitudinal reinforcement ratio in comparison with using Grade 60 reinforcement.*
2. *The use of the lower longitudinal reinforcement ratio for the beams reinforced with high-strength steel caused higher deflections compared to the beams reinforced with conventional Grade 60 steel at the same load levels.*
3. *The measured shear crack widths for all beams reinforced with high-strength stirrups designed with a yield strength of 80 and 100 ksi (552 and 690 MPa) were within the allowable limit recommended by ACI 318-08.*
4. *At ultimate, failure is typically due to crushing of the concrete strut for beams with and without stirrups. For beams with high-strength stirrups, the measured strains in the stirrups were equal to or greater than the strain of 0.0035 corresponding to 100 ksi (690 MPa) prior to crushing of the concrete strut.*
5. *The ACI, CSA, and AASHTO LRFD design codes can all be used to predict the shear strength of concrete beams reinforced with high-strength stirrups, with ACI 318-08 being the most conservative. The predictions by the CSA and AASHTO codes are quite accurate and are very close to each other. A yield strength up to 100 ksi (690 MPa) can be used in the design of high-strength transverse reinforcement for flexural members without impairing the ultimate load-carrying capacity and without exceeding the limits of the crack width. The stirrups, however, should have 135-degree hooks to provide better anchorage when it is designed for such high stresses. More testing is recommended to validate this detail.*
6. *The ultimate load-carrying capacities recorded for all of the beams were at least five times the service load specified by ACI 318-08*

25. Cyclic Response of Concrete Columns Reinforced with High-Strength Steel - 10th Canadian Conference on Earthquake Engineering - Paper No 996 - J. Rautenberg, S. Pujol et. al – July 2010 – (9 pages): This paper compares the results of columns designed with ASTM A1035 120 ksi (830 MPa) longitudinal reinforcement to those of columns with ASTM 706 60 ksi (410 MPa) reinforcement and variable axial loads with all columns having the same confinement steel. It is reported that the flexural strength of these columns is controlled by the strength of the steel; and that two sections with different grades of steel have similar moment capacities as long as the product of reinforcement ratio and yield stress is similar for both sections. Tests of columns under cyclic load reversals show that columns reinforced with ASTM A1035 120-ksi (830 MPa) steel reinforcement can reach drift ratios of 4%; and have smaller drift capacities than columns reinforced with (twice as much) A706 60-ksi steel.

26. Design Guide for the Use of High-Strength Steel Bars (ASTM 1035-07) for Structural Concrete - ACI ITG-6R-10 - August 2010 - *ACI Innovation Task Group 6:* P. Zia, A. Luba, S. Ghosh, C. Paulson, A. Lepage, H. Russell, K. Luttrell, J. Sanders, R. Mast– (90 pages): This guide provides recommendations on appropriate design procedures for the use of ASTM A1035 Grade 100 high-strength deformed reinforcing bars, for reinforced concrete members with regard to safety and serviceability. It was developed to address certain requirements in ACI 318-08 that limit more efficient use of high-strength steel bars.

This document includes a discussion of the material characteristics of ASTM A1035 steel bars and recommends design criteria for beams, columns, slab systems, walls, and footings in low and moderate seismic applications (Seismic Design Category A, B, or C). For high seismic areas, the application of this guide is currently limited to slab systems, foundations, and structural components not designated as part of the seismic-force-resisting system but explicitly checked for the induced effects of the design displacements. The only exception to this, is the use of transverse reinforcement for concrete confinement with a specified yield strength, f_y, up to 100,000 psi (690 MPa) as permitted by Section 21.1.5.4 of ACI 318-08.

Design examples are included as part of this document to illustrate design procedures and proper applications of the recommended design criteria. Also, included as part of these design examples are commentaries, which are provided to highlight the differences in design when using ASTM A1035 high-strength steel bars as opposed to the conventional ASTM A615 steel bars

27. [Flexural Behavior And Design With High-Strength Bars And Those Without Well-Defined Yield Point]() Transportation Research Board Annual Meeting 2010 Paper #10-1599- Jan. 2010- B. Shahrooz, K. Harries, H. Russell; et al.- (16 Pages): This paper focuses on behavior and design of flexural members reinforced with high-strength reinforcement (ASTM A1035 Grade 100) as well as other types of steel without well-defined yield plateaus, as part of National Cooperative Highway Research Program (NCHRP) Project 1277 *Structural Concrete Design with High-Strength Steel Reinforcement*. Analytical formulations and experimental testing of full-scale beams are reported. The following conclusions were made from these formulations and experimental testing.

The results of this study make the following general conclusions:

"For beams with reinforcement ratios less than 3%, flexural capacity of members reinforced with high-strength and other grade bars with no clear yield point can be established by using well-established strain compatibility analysis procedures in which the steel stress-strain behavior is idealized as being elastic-perfectly plastic with the yield point taken as the stress at strain equal to 0.0035 or 0.005. For beams with reinforcement ratios larger than 3% and concrete strength exceeding 69 MPa (10 ksi), the use of stress corresponding to strain equal to 0.0035 is conservative and recommended. For A1035 reinforcing bars, the yield strength at this strain maybe taken as 100 ksi.
The strain limit to achieve tension-controlled behavior for members using high-strength reinforcement should be taken as 0.008 (instead of the current value of 0.005). The corresponding strain limit for compression-controlled members is 0.004 (versus 0.002 in current AASHTO LRFD Bridge Design Specifications).
Members reinforced with high-strength ASTM A1035 bars exhibit adequate ductility and do not suggest any unexpected response characteristics."

28. [Bond and Anchorage of High Strength Reinforcing Steel](), TRB 2010 Paper #10-1328, K. Harries, B, Shahrooz, H. Russell; et al January 2010 (12 pages): This paper, which was prepared as part of TRB's NCHRP 12-77 project: **[Structural Concrete Design with High-Strength Steel Reinforcement]()**, stated: ***"****The study clearly demonstrates that the present AASHTO, and indeed ACI requirements for both straight bar tension development and hooked anchorage tension development may be extended to develop bar stresses of at least 125 ksi (860 MPa) for concrete strengths up to 10 ksi (69 MPa). In using higher strength steel, greater bar strain and slip will occur prior to development of the bar. The results of this study and previous work clearly indicate that confining reinforcement should always be used when developing, splicing or anchoring ASTM A1035 reinforcing steel."* The results reported in this paper will be part of NCHRP 12-77 Project Report 679, which will provide an evaluation of existing *AASHTO LRFD Bridge Design Specifications* relevant to the use of high-strength reinforcing steel and other grades of reinforcing steel having no discernable yield plateau. NCHRP Report 679 will include recommended language to the *AASHTO LRFD Bridge Design Specifications,* Sections 3, 5 and 9 that specifically permits the use of high-strength reinforcing steel with specified yield strengths not greater than 100 ksi (830 MPa).

29. [Use Of High-Strength Steel Reinforcement In Shear Friction Applications]() - Univ. of Pittsburg- G. Zeno (Master's Thesis)- November 2009 (Pages 91): This thesis reports the results of a study associated with Task 8.4b of the National Cooperative Highway Research Program (NCHRP) Project 12-77 *Structural Concrete Design with High-Strength Steel Reinforcement*. This study's test results showed that the shear friction mechanism occurs in stages and that the concrete component contributes to the majority of the shear friction capacity prior to cracking when the steel component develops. Therefore, the concrete and steel components of the shear friction mechanism do not act simultaneously as implied by the present AASHTO shear friction equation. In addition, the test results showed that, contrary to the assumptions of the AASHTO and ACI equations to calculate the shear friction capacity of concrete members, the interface steel reinforcement never reaches its yield strain. Therefore, the use of high-strength reinforcing steel does not affect the shear friction capacity of concrete members because the clamping force is a function of the elastic modulus of the steel rather than its yield strength. Based on these findings and using the experimental data from current and previous tests, an equation was proposed as an alternative to the existing AASHTO and ACI equations to calculate the shear friction capacity of concrete members. While the proposed equation is still semi-empirical, it represents the actual shear friction behavior better than the existing equations.

30. Bond Characteristics of ASTM A1035 Steel Reinforcing Bars - ACI Structural Journal,- J. Jirsa, D. Darwin, S. Rizkalla, P. Zia; et al. Jul-Aug 2009 (10 Pages): – This paper states: **"***The study shows that using high-strength steel [ASTM A1035 Grade 100] alters the mode of failure from diagonal tension to shear compression failure and results in higher shear strength compared with using conventional steel. It was also found that the current ACI shear design provisions are un-conservative for large-size concrete beams without web reinforcement"*. Among the paper's conclusions are the following: "*The use of transverse reinforcement to confine the spliced bars allowed splitting cracks to develop along the spliced bars and spalling of the cover was more gradual."* – This conclusion is made when comparing beams with and without transverse reinforcement. *"By confining the ASTM A1035 spliced bars with transverse reinforcement, bar stresses at bond failure of up to 150 ksi (1035 MPa) were reached for No. 8 and No. 11 (No. 25 and No. 36) bars." … "The ACI Committee 408 equation provides a reasonable estimate of the strength for both unconfined and confined splices using a strength reduction factor (φ-factor) of 0.82 and design parameters (cover, spacing, and concrete strengths) comparable to those used in this test program. The design equations in ACI 318 are less conservative, with a large percentage of the developed/calculated strength ratios below 1.0, and should not be used for development and splice design with high-strength reinforcing steel in their present form*" (Copy of this paper may be obtained from the American Concrete Institute (ACI) Farmington Hills, MI)

31. Rigid Pavement 100 KSI Steel Lane Tie Bar Substitution Analysis and Design- CME Transportation Group– July 2009– T. Biel (10 Pages): This report provides an analysis and design methodology for substituting 100 ksi (690 MPa) corrosion resistant MMFX 2 "ChromX® 9100" (ASTM A1035 Grade 100/AASHTO M 334) bars for either lower strength (i.e. 60 ksi [410 MPa]) coated or uncoated black steel bars. Design procedures provide a new economical procedure for optimizing lane tie bar materials and installation costs by utilizing MMFX 2's high strength and corrosion resistant material properties.

32. Behavior of High-Performance Steel as Shear Reinforcement for Concrete Beams - ACI Structural Journal - M. Sumpter S. Rizkalla; P. Zia –Mar-Apr 2009 (7 Pages): This document states: **"***This paper describes the behavior of high-performance (HP)[ASTM A1035] steel as shear reinforcement for concrete beams. HP steel is characterized by enhanced corrosion resistance and higher strength in comparison to ASTM A615-06 Grade 60 steel." ... "Test results indicate that using HP steel reinforcement increases the shear capacity and enhances the serviceability in terms of strength gain and reduction of shear crack width. Current design codes can conservatively be used for the design of HP steel using a yield strength of 80 ksi (552 MPa)."* Among the paper's conclusions are the following: *"Direct replacement of conventional Grade 60 stirrups with ASTM A1035 steel stirrups increased the shear load capacity of flexural members and enhanced the serviceability in terms of distributing cracks and reducing crack width." "Direct replacement of conventional Grade 60 longitudinal reinforcement with ASTM A1035 longitudinal reinforcement further increased the shear strength and enhanced serviceability;" "Shear crack widths were within the allowable limit of 0.016 in. (0.41 mm) using an increased service stress level of 48 ksi (331 MPa) for all beams reinforced with HP steel;" "The ACI, CSA, and AASHTO LRFD design codes can conservatively predict the shear behavior of concrete beams reinforced with HP steel using a yield strength of 80 ksi (552 MPa)." "Current research could not fully use the strength of ASTM A1035 steel stirrups beyond 80 ksi (552 MPa) because the failure was controlled by crushing of the concrete in the strut. Pairing high-strength concrete with ASTM A1035 steel could provide a better use for HP steel;" (*Copy of this paper may be obtained from the American Concrete Institute (ACI) Farmington Hills, MI.

33. Mechanical Properties of ASTM A1035 High Strength Steel Bar Reinforcement -Wiss, Janney, Elstner Associates, Inc. -WJE No. 2008.9901.0 –S. Graham, C. Paulson- December 2008 (49 pages): This report indicates the results of laboratory tests to measure mechanical properties of ASTM A1035 steel reinforcing bars with specified yield strengths of 100,000 psi (Grade 100) and 120,000 psi (Grade 120). The tests were performed in support of the activities of ACI Innovation Task Group 6 (ITG-6) - *High Strength Reinforcing*

Steel. Tests measured axial tension stress (ASTM A370), at 0.0035 in/in strain, yield strength (0.2% offset), and ultimate tensile strength, along with axial compression (ASTM E9) stress at 0.0035 in/in strain, and yield strength (0.2% offset). The following table compares the average tensile test results to ASTM A1035 required values. Individual test specimen elongation curves are included as part of the report. Modulus of Elasticity (E) tests were conducted in accordance with ASTM E111, indicating ASTM A1035's E value of 29,000 ksi (200 MPa) is similar to carbon steel.

Average Tensile Test Properties of ASTM A1035 Bars

ASTM A1035 Bar Grade	Stress corresponding to extension of 0.0035 in./in. (ksi)	Yield strength (0.2% offset) (ksi)	Tensile Strength (ksi)	Total Elongation (percent)
Ave Gr 100 Bar Tests	92.4	126.2	158.1	9.0
A1035 Gr 100 Spec	80	100	150	7
Ave Gr 120 Bar Tests	94.4	137.0	172.9	10.6
A1035 Gr 120 Spec	90	120	150	7

34. Analytical Evaluation of Structural Concrete Members with High-Strength Steel Reinforcement– University of Cincinnati– E. Ward (Master's Thesis)– December 2008 (352 pages): This report investigates the use of reinforcing steel, which exhibits no well-defined yield plateau, in the design of structural concrete members through analytical studies. Steel reinforcement considered includes ASTM A1035 Grade 100, A955 (stainless steel), A706, A496, and A82. This study made the following conclusions and observations: "*The analytical studies suggest that concrete members designed with a yield strength of 100 ksi behave similarly to members designed with a yield strength of 60 ksi. Therefore, allowing concrete members to be designed with reinforcing bars having a yield strength of 100 ksi is deemed reasonable. Because the stress-strain diagram for A1035 reinforcing steel has no well-defined yield plateau, the currently accepted strain limits of 0.005 and 0.002 were reevaluated. … the strain limits for tension-controlled behavior and compression-controlled behavior of members reinforced with A1035 bars were found to be, respectively, 0.008 and 0.004. Columns designed with A1035 transverse reinforcement and using the current equations for transverse steel spacing behave in the same manner as columns designed with A615 reinforcement. A1035 shear reinforcement with a yield strength of 100 ksi provides adequate shear resistance while maintaining acceptable diagonal crack widths and pattern.*"

35. Towards Earthquake-Resistant Concrete Structures With Ultra High-Strength Steel Reinforcement– 14th World Conference on Earthquake Engineering–A. Lepage, H. Tavallali, S. Pujol; J. Rautenberg- October 2008– (9 pages) This paper describes a collaborative experimental program between Penn State University and Purdue University is underway to investigate the deformation capacity of reinforced concrete members reinforced with ultra-high-strength steel reinforcement well in excess of 80 ksi (550 MPa). Test beams and columns are subjected to combined shear, moment, and axial load, applied through controlled increasing displacement reversals. Main variables of the experiments include: yield strength of main longitudinal reinforcement, 60, 100, and 120 ksi [410, 690, and 830 MPa]; yield strength of transverse reinforcement, 60, 120, and 185 ksi [410, 830, and 1280 MPa]; spacing of transverse reinforcement, $d/2$ and $d/4$; volume fraction of steel fibers, 0 and 1.5%; ratio of compression-to-tension longitudinal reinforcement, ρ'/ρ = 0.5 and 1.0; and type of loading, monotonic and cyclical. Within the range of code-accepted limits on reinforcement ratios, shear stress levels, and length-to-depth ratios, it is expected that the deformation capacity of ultra-high-strength steel reinforced concrete members is going to be increased by (1) reductions in spacing of transverse reinforcement; (2) increases in the ratio of compression-to-tension longitudinal reinforcement, and/or (3) addition of engineered fibers.

36. Flexural Strength Design of Concrete Beams Reinforced with High-Strength Steel Bars - ACI Structural Journal - B. Mast, M. Dawood, S. Rizkalla; P. Zia –Sep-Oct 2008 (8 Pages) This paper presents a methodology for the flexural strength design of concrete beams reinforced with high-strength reinforcing steel that conforms to the requirements of ASTM A1035-07 (MMFX 2 "ChromX® 9100" bars). The design method is based on simple analysis techniques that satisfy fundamental principles of equilibrium and compatibility. Strain limits for tension-controlled sections and compression-controlled sections are proposed that are consistent with the approach of the current and past ACI 318 Codes. The proposed method is compared with experimental results previously reported by others. The application of the proposed method is demonstrated by a numerical design example. (Copy of this paper may be obtained from the American Concrete Institute (ACI) Farmington Hills, MI)

37. Shear Behavior of Concrete Beams Reinforced with High Performance Steel Shear Reinforcement- North Carolina State Univ.– Constructed Facilities Laboratory– A. Munikrishna (Master's Thesis)– July 2008 (152 pages): This paper reports on a program utilizing the strength of ASTM A1035 (MMFX 2 "ChromX® 9100") as shear reinforcement for reinforced concrete flexure members at selected yield strengths of 80 ksi (550 MPa) and 100 ksi (690 MPa) in comparison to conventional reinforcement designed at 60 ksi (410 MPa), which were tested to failure under static loading conditions. Conclusions from this program are: *"1. The shear capacity of flexural members can be achieved with lesser amount of MMFX stirrups … attributed to the higher tensile strength of the MMFX steel in comparison to Grade 60 steel. 2. The beams reinforced with MMFX steel exhibited the same deflections at service load as the beam reinforced with Grade 60 steel. 3. Shear crack widths measured for all tested beams reinforced with MMFX steel designed with yield strength of 80 ksi and 100 ksi were within the allowable limit specified by the ACI Code. 4. The ACI, CSA and AASHTO LRFD design codes can conservatively predict the shear behavior of concrete beams reinforced with MMFX steel. 5. Design stress up to 100 ksi can be for MMFX transverse reinforcement for flexure members without impairing the ultimate load carrying capacity and serviceability."*

38. Review of Port Authority of NY & NJ (PANYNJ) Testing of MMFX Reinforcing Steel, #11 rebars –S. Ghosh (S. K. Ghosh Associates Inc.) –April 2008 (Pages 44): This report is based on the included PANYNJ's report and "Material Test Report – ASTM A1035 – Grade 120." The review notes A1035 Grade 120's certified: tensile, yield (0.0035 strain and 0.2% offset) and elongation at facture properties, indicating that all 30 test specimens exceeded the certified values. The author concludes: *"Overall, I think the results are quite reassuring. The shape of the stress-strain curve should not matter until we get into high-seismic applications. And it can be accounted for in design. There is now an Innovation Task Group (ITG) within ACI, working on a comprehensive design document for concrete structural members using MMFX steel. Such a document is expected to be available within the next year or so."*

39. Shear Behavior of Large Concrete Beams Reinforced with High-Strength Steel- ACI Structural Journal– T. Hassan, Rizkalla, P. Zia; et. al.- Mar-Apr 2008 (7 Pages): This paper states: *"The study shows that using high-strength steel alters the mode of failure from diagonal tension to shear compression failure and results in higher shear strength compared with using conventional steel. It was also found that the current ACI shear design provisions are unconservative for large-size concrete beams without web reinforcement."* Among the paper's conclusions are the following: *"Despite the reduction in the reinforcement ratio by 40%, the shear strength of concrete beams reinforced with high-strength steel was significantly higher than that of the beams reinforced with Grade 420 MPa (60 ksi) steel. The high yield strength of the material maintained the capacity of the tension tie, and thus enabled the beams to resist more load until crushing of the diagonal strut occurred;" "A significant reserve in strength was observed for beams reinforced with high-strength steel after diagonal cracking. Failure was due to crushing of the diagonal concrete strut at much higher loads compared with beams reinforced with conventional steel;" "The ACI 318-05 simplified expression for the shear contribution of concrete is un-conservative for large-size concrete beams without web reinforcement. The expression needs to account for the size effect and the reinforcement characteristics."* (Copy of this paper may be obtained from the American Concrete Institute (ACI) Farmington Hills, MI)

40. Behavior of Concrete Bridge Decks Reinforced with High-Performance Steel ACI Structural Journal, V. 105, No. 1– G. Lucier, S. Rizkalla, P. Zia, P. Hatem - Jan --Feb 2008 (9 pages): This paper describes the behavior of concrete bridge decks reinforced with MMFX 2 "ChromX® 9100" (ASTM A1035) high-performance (HP) steel, characterizing its high strength in comparison with conventional ASTM A615-06

Grade 60 steel. The paper makes the following conclusions: *"1. The ultimate load-carrying capacity of the three bridge decks investigated in this study was on the order of 10 times the service load prescribed by the AASHTO Specifications; 2. Punching shear was the primary mode of failure for the three bridge decks. Due to continuity used in the test models, flexural-shear failure was observed as a secondary mode of failure; 3. The cracking load of the three tested bridge decks was more than twice the service load prescribed by the AASHTO Specifications. Hence, under service load level, the three bridge decks behaved as uncracked sections. Therefore, using 33% less HP [ASTM A1035] steel should not alter the serviceability behavior of concrete bridge decks; 4. The bridge deck reinforced with 33% less HP steel developed the same ultimate load-carrying capacity as that reinforced with Grade 60 steel. This performance is attributed to the higher strength of HP [ASTM A1035] steel compared with Grade 60 steel; and 5. Behavior of bonded HP [ASTM A1035] steel bent bars is similar to the behavior of straight bars. De-bonded bent bars exhibit similar behavior to straight bars, including the linear and the nonlinear behavior up to a strain of 1.5%. Its ultimate strength, however, is reduced by 6% and its ultimate strain by 70%."* The papers also make the following design guideline recommendations: *"1. Substituting HP [ASTM A1035] steel directly for conventional Grade 60 steel in a design, … is a conservative approach; 2. HP [ASTM A1035] steel can be used as the main flexural reinforcement for cast-in-place concrete bridge decks at a reinforcement ratio corresponding to 33% less than that required for Grade 60 steel. Therefore, design of reinforced concrete bridge decks using HP [ASTM A1035] steel can use a yield stress of 90 ksi (621 MPa) for the HP [ASTM A1035] steel bars; 3. Reduced reinforcement ratio of HP [ASTM A1035] steel shall satisfy all minimum reinforcement ratios prescribed by the AASHTO Specifications. In addition, the reduced reinforcement ratio of HP [ASTM A1035] steel must comply with the crack control requirement of the AASHTO Specifications; and 4. HP [ASTM A1035] steel bars can be bent up to 90 degrees without reducing their ultimate strength or strain provided that the bend is fully encased and bonded to concrete."*

41. Behavior of Concrete Deep Beams With High Strength Reinforcement - Structural Engineering Report 277 University of Alberta J. Garay-Moran, A. Lubell - January 2008 (315 Pages): This paper reports on the testing of large-scale beams containing ASTM A1035 steel, at an effective yield strength of 860 MPa (125 ksi) as the main tension reinforcement. The program's testing examined the adequacy of CSA A23.3-04, ACI 318-05 and Eurocode 2 design models predict the behavior of reinforced concrete deep beams containing high strength steel reinforcement. The report conclusions indicated: "*Capacity predictions made using the Strut and Tie Method provisions from the CSA A23.3-04, ACI 318-05 and Eurocode2 were in good agreement with the results from deep beam specimens constructed with ASTM A1035 reinforcing steel. … Current design yield strength limits (500 MPa for CSA A23.3-04 and Eurocode 2 and 550 MPa for ACI 31805) can be increased to magnitudes closer to the effective yield strength according to the 0.2% offset method*."

42. Bond Behavior of MMFX (ASTM A 1035) Reinforcing Steel – Cooperative Research Program – NC State Univ., S. Rizkalla et. al. – Univ. of Kansas, D. Darwin et. al. – Univ. of Texas Austin, J. Jirsa et. al.– November 2007 (32 pages): This report summarizes the findings of a cooperative research program on the bond behavior of MMFX (ASTM A1035). Findings indicate, based on sixty-six MMFX "ChromX® 9100" (ASTM A 1035) test specimens, that: *"… ACI 318-05 code design equation overestimates the strength of unconfined spliced MMFX bars, especially for high strength concrete. On the other hand, the bond equation for design recommended by ACI Committee 408 (as best-fit to the database but including a strength-reduction factor φ of 0.82) underestimates the stresses for unconfined spliced bars for all but two out of 31 cases, but with less scatter than those obtained using the ACI 318-05 equation. … ACI Committee 408 equation with a strength-reduction factor φ of 0.82 is recommended for development and splice design using MMFX steel."*

43. Report on Structural Design and Detailing for High-Strength Concrete in Moderate to High Seismic Applications - ACI ITG -4.3R-07 – ACI Innovation Task Group 4 (S. Ghosh, Chairman) -2007 (Pages 62): This report presents a literature review on seismic design using high-strength concrete. Included as part of the report are a series of recommended modifications to "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE" ACI 318-05. Subsequently, ACI 318-08 allowed an upper limit of 100 ksi (690 MPa) on the yield strength of high-strength confinement reinforcement for members resisting earthquake-induced forces in structures assigned to SDC (Seismic Design Category) D, E, or F. (Copy of this report may be obtained from American Concrete Institute (ACI), Farmington Hills, MI)

44. "High-Strength Rebar Called Revolutionary" – McGraw Hill Construction - Engineering News Record - July 2007 (2 pages): This magazine article describes the first use of MMFX 2 "ChromX® 9100" (ASTM A1035) rebar in a high seismic zone (Seattle, WA) high rise building for column and shear wall boundary element confinement." The article notes that use of ASTM A1035 at 100 ksi (690 MPa) design, reduced rebar requirements by 40% in comparison to conventional design practices. In addition, use of ASTM A1035 rebar simplified the project's beam to column connections, reducing the construction time to make these connections by up to 25%.

45. Behavior of High Performance Steel as Shear Reinforcement for Concrete Structures – Final Report– North Carolina State Univ.–Constructed Facilities Laboratory– M. Sumpter, S. Rizkalla; P. Zia-June 2007 (91 pages): This report concludes that: *1. "Direct replacement of conventional Grade 60 longitudinal reinforcement with MMFX [ASTM A1035] longitudinal reinforcement showed an optimum design by further increasing the shear strength and enhancing serviceability." 2. "The use of MMFX [ASTM A1035] steel, with a yield strength of 80 ksi, increases the allowable service stress level to 48 ksi. Shear crack widths measured for all tested beams reinforced with MMFX steel were within the allowable limit specified by the ACI Code."*

46. Evaluation of Bond Characteristics of MMFX Steel –North Carolina State Univ.–Constructed Facilities Laboratory, Technical Report No. RD-07-02 –H. Seliem, A. Hosny; S. Rizkalla–June 2007 (71 pages): This report concludes that: A. Stress levels of 90 and 70 ksi (620 and 480 MPa)can be achieved by No. 8 and No. 11 ASTM A1035 spliced bars without the use of transverse reinforcement (confinement). B. Spliced bar transverse reinforcement was able to develop a stress of 150 ksi (1030 MPa) for No. 8 and No. 11 A1035 bars and increased the ultimate load and ductility of the beams. C. Increasing the splice length, proportional to the square root of the ratio of the splice length and the bar diameter, increased the strength of the splice. D. Increasing the concrete cover by the square root of the ratio of the cover to the bar diameter, increases the stress developed in the spliced bars. E. Use of ACI 408 equation provides better prediction of stresses and less scatter than use of the ACI 318-05 equation.

47. Effects of Confinement and Gauging on the Performance of MMFX High Strength Reinforcing Bar Tension Lap Splices--University of Texas (Austin)–K. Hoyt (Master's Thesis)–May 2007 (65 pages): This program reports on testing of beam-splice specimens using ASTM A1035 Grade 100 No. 8 bar splices in a constant moment region, with varied amounts of No. 4 Grade 60 transverse reinforcement and spacing. It was found that: 1. ACI 408 equation provided a good estimate of failure stresses at high stress levels, but with predicted lower strengths than measured in beams with confinement. 2. The linear nature of the current development length code equation is acceptable. 3. Behavior of the interior splices was nearly identical to that of the exterior splice. 4. High steel stresses resulted in greater crack widths than currently acceptable for service load stresses using Grade 60 steel. The equation used to determine serviceability limits only appears to be effective for stress levels of 60 ksi (410 MPa) or less.

48. Performance of Tension Lap Splices with MMFX High Strength Reinforcing Bars - University of Texas (Austin)–G. Glass–May 2007 (141 pages): This paper reports on tests from beam-splice specimens at the University of Texas, North Carolina State University, and the University of Kansas, making the following conclusions concerning ASTM A1035 reinforcement: A. A1035 lap splices developed bar stresses up to 155 ksi (410 MPa). B. ACI 408 development length equation provided relatively accurate estimates of failure stresses for splices with and without confining transverse reinforcement. C. ACI 318 and AASHTO LRFD development length equations provided un-conservative calculated failure stresses for unconfined splices, while providing reasonable calculated failure stresses for confined splices. D. The addition of confining transverse reinforcement provided an increase in failure stress and was greater than predicted by either the ACI 408 or ACI 318 equation. E. The addition of confining transverse reinforcement provided an increase in beam deflections at failure; and was greater than proportional to the increase in confining reinforcement. F. Service level crack widths were greater than the limits used as a basis for serviceability provisions included in pre-1999 editions of ACI 318. G. Bar splices with stresses greater than 75 ksi (515 MPa) should be designed using the ACI 408 development length equation with the modification factor,ϕ, equal to 0.82. H. A minimum level of transverse reinforcement should be included for all splices above 75 ksi (515 MPa) except for those with No. 5 or smaller bars with large bar spacing and cover.

49. Behavior of Concrete Bridges Reinforced with High-Performance Steel Reinforcing Bars––North Carolina State Univ.–H. Seliem (Dissertation)–2007 (287 pages): This paper describes the testing of reinforced concrete structural members with MMFX "ChromX® 9100" (ASTM A1035 Grade 100) reinforcing bars and made the following conclusions: *"Yield strength of 90 ksi (621 MPa) can be used in design of bridge decks reinforced with MMFX steel reinforcing bars without impairing the ultimate load carrying capacity or altering the serviceability behavior. … Up to #8 (NO. 25), spliced MMFX reinforcing bars can develop a stress of 90 ksi (621 MPa) without the use of confinement by transverse reinforcement. … Minimum amount of transverse reinforcement is required to confine spliced bars to ensure a ductile behavior of concrete members with spliced bars as well as sufficient warning prior to failure. … MMFX steel bars can be bent up to 90 degrees without impairing their ultimate strength if they are fully bonded to concrete."*

50. Behavior of Minimum Length Splice of High Strength Reinforcement--University of Texas (Austin)–K. Donnelly (Honors Thesis)–2007 (37 pages): This paper describes testing of MMFX "ChromX® 9100" (ASTM A1035 Grade 100) beams, using No. 5 MMFX bars spliced with minimum splice lengths and varying levels of transverse reinforcement. It was concluded that splice length designs using ACI 408 equations with transverse reinforcement were more accurate than use of ACI 318, while ACI 318 designs were better suited for unconfined splice designs.

51. Fatigue Behavior of MMFX Corrosion-Resistant Reinforcing Steel -Canada 7th International Conference on Short and Medium Span Bridges, Montreal, Canada—S. DeJong, C. MacDougall- 2006 (11 pages): This study indicates that MMFX "ChromX® 9100" (ASTM A1035 Grade 100) was tested to have a fatigue life of 1 x 10^6 cycles at a stress range of approximately 310 MPa [45 ksi], compared to conventional steel 1 x 10^6 cycles at a stress range of approximately 166 MPa [24 ksi]. The study made the following conclusion: "Thus, MMFX exhibits superior fatigue resistance under constant amplitude loading in an air environment than conventional steel reinforcing bars."

52. Bond Characteristics of High-Strength Steel Reinforcement -ACI Structural Journal Vol. 103, No. 6 - R. El-Hacha, H. El-Agroudy, S. Rizkalla- Nov-Dec 2006 (12 pages): This paper summarizes the findings of a study concerning the bond characteristics of MMFX 2 "ChromX® 9100" (ASTM A1035 Grade 100) steel bars, based on testing of a series of beam end specimens, comparing MMFX 2 bars to A615 Grade 60. The bond behavior of the MMFX 2 bars was found to be similar to that of A615 Grade 60 ksi (410 MPa) steel up to the proportional limit of 80 ksi (550 MPa), using splice length to bar diameter (L_s/d_b) of 30 d_b. A splice length of 45 d_b was found to be adequate for a MMFX 2 bar yield strength of 110 ksi (760 MPa). (Copy of this paper may be obtained from American Concrete Institute (ACI), Farmington Hills, MI).

53. Shear Behavior of Concrete Beams Reinforced with MMFX Steel without Web Reinforcement–– Technical Report: IS-06-08 - NC State Univ. –S. Rizkalla, H. Seliem; et. al.- April 2006 (13 pages): This study tested large size concrete beams reinforced with MMFX steel without web reinforcement under static loading up to failure to evaluate their shear behavior. Among the report's conclusion were: *"reduction of the longitudinal reinforcement area (40 percent less) of MMFX [ASTM A1035] steel used, the shear capacity of the beams with a/d ratio of 1.79 and reinforced with MMFX steel was 80 percent higher than those reinforced with grade 60 steel. For the beams with a/d ratio of 2.6, the beam reinforced with MMFX steel had a capacity of 12 percent more than the beam reinforced with conventional Grade 60 steel. … The higher failure loads achieved by the beams reinforced with MMFX steel compared to the beams reinforced with Grade 60 steel is due to the high-strength characteristics of the MMFX steel which is more than twice of the Grade 60 steel."*

54. Application of ASTM A 1035 MMFX Steel Reinforcement in Building Applications: An Appraisal – S.

Ghosh- S. K. Ghosh Associates Inc.- April 2006 (19 pages): This report examines various design aspects for use of MMFX 2 "ChromX® 9100" (ASTM A1035 Grade 100) rebar in building structural applications, relating the design to appropriate ACI 318 Sections. Conclusions of the report describe considerations for: a. allowable flexural tension design at 100 ksi (690 MPa), 80 ksi (550 MPa) in flexural compression, and 60 ksi (410 MPa) for shear strength, and b. one-way slab tension design at 100 ksi (690 MPa) limitations, among design aspects presented.

55. Evaluation of MMFX Steel For NCDOT Concrete Bridges –FHWA/NC/2006-31, NCDOT Report 2004-27– S. Rizkalla, P. Zia et. al.– December 2005 (131 pages): This publication states the following conclusions based on testing of full-scale bridge deck sections and corrosion tests at North Carolina State University: *"1. Substituting MMFX steel directly for Grade 60 steel in a design … is an overly-conservative approach. 2. MMFX steel [ASTM A1035] can be used as the main flexural reinforcement for cast-in-place concrete bridge decks at a reinforcement ratio corresponding to 33% less than that required for Grade 60 steel. Therefore, a design of reinforced concrete bridge decks using MMFX steel may utilize an equivalent yield stress of 90 ksi for the MMFX steel bars. 3. Design of concrete bridge decks utilizing the high tensile strength characteristics of the MMFX steel should satisfy all minimum reinforcement ratios required by the AASHTO LRFD Bridge Design Specifications as well as the serviceability requirements of the specifications. 4. MMFX steel [ASTM A1035] has a much lower corrosion rate compared to conventional Grade 60 steel. Therefore, the use of MMFX steel could increase the service life of concrete bridges and lower repair costs."*

56. MMFX Rebar Evaluation for I-95 Service Road Bridge 1-712-B –University of Delaware- M. Chajes, M. McNally; et. al – March 2005 (162 pages): The following is a summary of results from the four point bending tests of the "standard" beam [60 ksi (410 MPa) yield design], "MMFX4" beam [same reinforcement as 60 ksi (410 MPa) yield design], "MMFX2" beam [100 ksi (690 MPa) yield design], and the "CFRP" beam [ACI 440.1 R-01 design guideline]. Both ultimate loads and mode of failure were predicted with good accuracy using traditional equations for the MMFX reinforced beams. Yield deflection calculations were smaller and load at L/800 calculations were greater than the actual measured yield deflection and load at L/800 values for all beams. This may have been due to early cracking. All beams cracked at a similar load level. Both MMFX beams failed in the desired mode. For both beams, the MMFX rebar yielded prior to failure.

57. Tensile Test – Coupled Reinforcing Steel Bars (w/ Stress vs. Strain Graphs)– Smith Emery Laboratories– February 2005 (26 pages): This report covers the successful testing of #4, #8, #9, #10, and #11 MMFX Bars fitted with Barsplice® couplers. The report covers test results and photographs of tested samples.

58. Tensile Testing of Mechanical Bar Splices for MMFX Steel–Florida DOT**- A. Michael- February 2004 (15 pages):** Two types of commercially available mechanical splices for #6 bars were tested to establish compatibility with MMFX 2 "ChromX® 9100" (ASTM A1035 Grade 100) rebar. Both splice types exceeded the capacity of the MMFX bar and failure occurred in the steel bar. The average stress in the bars at failure was 173.6 ksi (1200 MPa).

59. "Seismic Behavior of Bridge Columns Built Incorporating MMFX Steel" – University of California, San Diego– Report No. SSRP–2003/09– B. Stephan, J. Restrepo, F. Seible – October 2003 (37 pages): Testing was performed on two similar column units constructed using ASTM A706 Grade 60 and MMFX 2 "ChromX® 9100" (ASTM A1035 Grade 100) reinforcing bars. The ASTM unit was designed according to the CALTRANS Bridge Design Specifications (July 2002) and the MMFX unit incorporated MMFX's design strength resulting in approximately half the steel requirement of the ASTM unit. The tests conclusively showed that both units can be designed to form ductile flexural plastic hinges and can sustain drift levels of approximately 4% without failure and complied with CALTRANS column seismic failure criteria. (See also "**Seismic Testing of Bridge Columns Incorporating High-Performance Materials" –ACI Structural Journal Vol. 103, No. 4 -J. Restrepo, F. Seible, B. Stephan, M. J. Schoettler - Jul-Aug 2006 - 9 pages**) (Copy of this paper may be obtained from American Concrete Institute (ACI), Farmington Hills, MI).

60. **Development Length of Micro-composite (MMFX) Steel Reinforcing Bars Used In Bridge Applications- University of Massachusetts Amherst– S. Peterfreund- June 2003 (59 pages):** This study reports on the laboratory testing of beams using MMFX "ChromX® 9100" (ASTM A1035 Grade 100) No. 4 and No. 5 bars for tensile reinforcement, varying the lap splice length in the constant moment region. Tests were compared to ACI 318-02 development length code (Equation 6-1); and the tested lap splice lengths were determined to be *"more than adequate to develop the flexural capacity of the beam"*.

61. **Fundamental Material Properties of MMFX Steel Rebars, North Carolina State University, NCSU-CFL Report No. 02-04 –R. El-Hacha. S. Rizkalla- July 2002 (61 pages):** This report provides preliminary data for the fundamental mechanical material properties of MMFX steel reinforcing rebars. The testing focused on the mechanical properties in tension and in compression, shear strength, fatigue strength, effect of bend on tensile strength of the bent rebar (stirrup), bond strength and development length, and the behavior of MMFX rebars as compression steel in reinforced concrete columns.

62. **Bending Behavior of Concrete Beams Reinforced with MMFX Steel Bars, Constructed Facilities Center, West Virginia University - Vijay P. V.; et. al.- July 2002 (34 pages):** Theoretical moments can be predicted very well using current theories. Beams exhibited a significant amount of elongation before compression failures (secondary) occurred. Deflection values can be well approximated up to a stress level of 75 ksi (515 MPa) within the serviceability stress limits) using actual stiffness of the bar at a given stress level and also by accounting the corresponding increase in strain as compared to Es = $29x10^6$ psi. The crack width values evaluated by using stress in tension steel and also by accounting for the corresponding strain value at that stress level led to very good prediction of crack widths.

C. Supplemental References

1. **"Product Guide Specification – Uncoated Corrosion Resistant Reinforcement (CRR) – High Strength Steel Microcomposite (ChromX® 9100, 4100 and 2100) Plain and Deformed Bars For Concrete Reinforcement" – MMFX Technologies Corporation – July 2017 (9 pages)** This guide product and construction specification provide a guideline to assist design engineers in specifying ChromX® 9100, 4100 and 2100 (ASTM A1035 Types CS, CM and CL and AASHTO M 334) rebar, referencing applicable codes and standards that apply to it, along with MMFX's material properties and recommendations for fabrication, and field installation.

2. **ASTM A1035/A1035M-16b Specification "Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement" - American Society For Testing And Materials - West Conshohocken, PA 19428 – 2016 (7 pages):** This ASTM specification, which ChromX® Series 9000, 4000 and 2000 rebars qualifies as, was prepared by ASTM's 1.05 Committee based on material properties that ChromX® series 9000, 4000, and 2000 rebar possesses. Bars are of two minimum yield strength levels as 100 ksi [690 MPa], and 120 ksi [830 MPa] designated as Grade 100 [690] and Grade 120 [830], respectively.

Included as part of ASTM A615-15 is the following "NOTE 1: Grade 100 [690] reinforcing bars were introduced in this specification in 2015. In contrast to the lower grades, which have ratios of specified tensile strength to specified yield strength that range from 1.31 to 1.5, Grade 100 [690] reinforcing bars have a ratio of specified tensile strength to specified yield strength of 1.15. Designers should be aware that there will, therefore, be a lower margin of safety and reduced warning of failure following yielding when Grade 100 [690] bars are used in structural members where strength is governed by the tensile strength of the reinforcement, primarily in beams and slabs. If this is of concern, the purchaser has the option of specifying a minimum ratio of tensile strength to actual yield strength. Consensus design codes and specifications such as "Building Code Requirements for Structural Concrete (ACI 318)" may not recognize Grade 100 [690] reinforcing bars: therefore the 125 % of specified yield strength requirements in tension and compression are not applicable. Mechanical and welded splices should meet a minimum specified tensile strength of 115 000 psi [790 MPa]". ASTM A1035 and ASTM A615 Documents available through ASTM (American Society for Testing and Materials)."

3. AASHTO M 334M/M 334 17 Standard Specification: Uncoated, Corrosion-Resistant, Deformed and Plain Alloy, Billet-Steel Bars for Concrete Reinforcement and Dowels– 2017 (15 pages): This specification was prepared in conjunction with AASHTO SOM (Subcommittee on Materials) Technical Section 4f "Metals". ASTM A1035 Type CS (ChromX® 9100) and various stainless steel reinforcing bars are included in this specification, which defines corrosion resistant bars by testing them in accordance with the document's reference Standard Methods of Test T 472M/T 472,T 373M/T 373, T 374M /T374, T 375M/T375, or T 376M/T 376**.** These standards are available through AASHTO (American Association of State Highway and Transportation Officials).

4. Quality Assurance Manual 9th Edition– MMFX Technologies Corporation– April 2016 (12 pages): This manual provides the quality control basis for the manufacture of all ChromX® series 9000, 4000, and 2000 steel bars while ensuring that the manufacturing practices and tolerances used in ChromX® series 9000, 4000, and 2000's production, provide both the certified chemical composition and mechanical properties are met or exceeded.

5. ASTM A1035/AASHTO M 334 - Material Safety Data Sheet (MSDS) – Cascade Steel Rolling Mills – February 2012 (6 pages): This document provides information concerning ASTM A1035 "ChromX® series 9000, 4000, and 2000" and AASHTO M 334 bars, including physical and chemical properties, handling and storage, toxicological information of the main components, and disposal.

6. Chemical, Mechanical Analysis, Tests and Measurements performed on Bar numbers 3, 4, 5, 6, 7, 8, 9, 10 and 11 MMFX-2 (AASHTO MP 18/ASTM A 1035) grade 100 steel rebar samples –Professional Service Industries, Inc. (PSI)- PSI Project: 0689492-2a- P. Irish - June 2011 (6 pages): This test report provides results of mechanical and other testing of MMFX 2 "ChromX® 9100" (AASHTO M 334, formerly MP 18/ASTM A1035) bar sizes 3 through 11, certified to AASHTO M 334 and ASTM A1035 in accordance with AASHTO M 334 "MP18" Sections 6, 7, 8, 9, 10, 12 13 and 21. All test bars met the requirements of AASHTO Standard Specification M 334 "MP 18" as indicated in the test data table included in the report.

7. MMFX 2 (ASTM A 1035, Grade 100) Steel Rebar Corrosion Performance Testing in Accordance with AASHTO MP 18M/MP 18-09 – Tourney Consulting Group (TCG) – Report TCG # 11072- August 2011 (29 pages): This test report indicates the result of corrosion test of AASHTO M 334 "MP 18"/ ASTM A1035 Type CS certified bars in accordance with AASHTO T 334M (formerly AASHTO MP 18 – Annex A) Standard Method of Test for *Comparative Qualitative Corrosion Characterization of Steel Bars Used for Concrete Reinforcement (Linear Polarization Resistance and Potentiodynamic Polarization Tests)* on No. 3 through No. 11 test bars. Results of these corrosion tests indicated that all bars met the corrosion test requirements of AASHTO M 334. In addition, micrographs of etched specimens were taken as an index for confirmation of the microstructure for each of the test bars. The metallographic examination showed Martensite, fine grain microstructure structure for each of the MMFX 2 test bars at a 500x magnification.

8. ICC ES Evaluation Report ESR 2107. International Code Council (ICC) Subsidiary ICC ES (Evaluation Service) January 2017 (8 pages): This report provides guidance how ChromX® 9100, 4100, & 2100 (ASTM A1035 Types CS, CM & CL Gr. 100) high strength bars can be designed into structures in accordance to the ICC AC 429 Acceptance Criteria, which references the ACI ITG-06 required changes to the ACI 318-14 according to the IBC 2015.

9.Chemical, Mechanical Analysis, Tests and Measurements performed on Bar numbers 3, 4, 5, 6, 7, 8, 9, 10, 11, 14 and 18, ASTM A1035, Grade 100 steel rebar samples – Professional Service Industries, Inc. (PSI)- PSI Project: 0689492-11- P. Irish – August 2012 (7 pages): This test report provides results of mechanical and other testing of MMFX 2 "ChromX® 9100" (ASTM A1035 Type CS) bar sizes 3 through 11, 14 and 18 certified to ASTM A1035 in accordance with ASTM Sections 6, 7, 8, 9, 10, 12 13 and 21. All test bars met the requirements of ASTM Standard Specification A1035/A1035 M -11 as indicated in the test data tables included in the report.

10. [CRSI – Specialty & Corrosion Resistant Steel Reinforcement – Product Guide](#) - Concrete Reinforcing Steel Institute– July 2013 (28 pages): CRSI's Product Guide provides a comparison of fabrication, estimating, detailing and placement information for Corrosion Resistant Reinforcing (CRR) bars: ChromX® 9100 (AASHTO M 334/ASTM A1035 Type CS), epoxy coated (ECR), galvanized, stainless steel (SS) and dual coated (Z-bar). A summary comparison of these CRR bars is provided in **MMFX's CRR CRSI Product Guide Comparison**, which indicates that fabrication, estimating, detailing and placement of ChromX® 9100 bars is easier than the other CRR bars included in the CRSI Product Guide.

11. Frequently Asked Questions (FAQ) About Low-Carbon, Chromium ASTM A1035 Types CS, CM and CL Steel Reinforcing Bar CRSI ETN-M-11-17 FAQ – April 2017 (6 pages): *CRSI's FAQ is divided into the following Sections: 1. Basic Material Characteristics, 2. Engineering Design Issues, 3. Fabrication and Construction Issues, 3. Finish Quality and 4. References. This FAQ provides information concerning: A. ASTM A1035 types chemical composition, B. Available bar types, sizes and lengths, C. Figure of typical stress-strain curve, D. IBC building code and AASHTO LRFD Bridge Design Specifications high strength design guidelines, E. Corrosion resistance properties, F. High strength design benefits, G. Fabrication equipment and procedures, H. Handling, storage, or placing requirements, I. Effect of mill scale oxidation on the bars service life, J. Reference documents identified in the FAQ.*

6. Annexes

Annex A: DETERMINING THE PROPER DEVELOPMENT LENGTH FOR LAP SPLICING HIGH STRENGTH "GRADE 100 CHROMX® STEEL" AS PER ACI 318-19

The process for determining the development length of ChromX® steel conforming to ASTM A1035/A1035M design strength at 100 ksi is in accordance to the requirements of ACI 318-19 section ***25.4.2.1***.

> ***25.4.1.4*** *The values of* $\sqrt{f'_c}$ *used to calculate development length shall not exceed 100 psi.*
>
> ***25.4.2.1*** *The development length* ℓ_d *for deformed bars and deformed wires in tension shall be the greater of (a) and (b):*
>
> *(a) Length calculated in accordance with* ***25.4.2.3*** *or* ***25.4.2.4*** *using the applicable modification factors of 25.4.2.5*
> *(b) 12 in.*
>
> ***R25.4.2.1*** *This provision gives a two-tier approach for the calculation of tension development length. The user can either use the simplified provisions of 25.4.2.3 or the general length equation (Eq. (25.4.2.4a)), which is based on the expression previously endorsed by ACI 408.1R.*
>
> *In Table* ***25.4.2.3***, ℓ_d *is based on two preselected values of* $(c_b + K_{tr})/d_b$, *whereas* ℓ_d *from Eq.* ***(25.4.2.4a)*** *is based on the actual* $(c_b + K_{tr})/d_b$.

Additional requirements are imposed when designing with 100 ksi design strength when the bars are spaced closer than 6 in. on center, transverse reinforcement shall be provided such that K_{tr} shall not be smaller than 0.5 d_b.

> ***25.4.2.3*** *For deformed bars or deformed wires,* ℓ_d *shall be calculated in accordance with Table 25.4.2.3*
>
> *Clear spacing of bars or wires being developed or lap spliced not less than* d_b, *clear cover at least* d_b, *and stirrups or ties throughout* ℓ_d *not less than the Code minimum or*
> *Clear spacing of bars or wires being developed or lap spliced at least 2db and clear cover at least* d_b
>
> *For #3, #4, #5 and #6 the development length* $\left(\dfrac{f_y \Psi_t \Psi_e \Psi_g}{25\lambda\sqrt{f'_c}}\right) d_b$ *is*

For #7 and larger bars the development length is $\left(\frac{f_y \psi_t \psi_e \psi_g}{20\lambda\sqrt{f'_c}}\right) d_b$

25.4.2.4 For deformed bars or deformed wires, ℓd shall be calculated by:

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \psi_g}{\left(\frac{c_b + K_{tr}}{d_b}\right)} \right) d_b \qquad (25.4.2.4a)$$

in which the confinement term ***(c_b + K_{tr})/d_b*** *shall not exceed 2.5, and*

$$K_{tr} = \frac{40 A_{tr}}{sn} \qquad (25.4.2.4b)$$

where ***n*** *is the number of bars or wires being developed or lap spliced along the plane of splitting. It shall be permitted to use* ***K_{tr}*** *= 0 as a design simplification even if transverse reinforcement is present or required*

25.4.2.5 *For the calculation of* ***ℓd****, modification factors shall be in accordance with Table* ***25.4.2.5.***

Typical Development Length with a minimum clear cover of 2 in. according to ACI 318-19 section 25.4.2.4

	Concrete Compressive Strength, psi											
Bar size:	**3,000**	**4,000**	**5,000**	**6,000**	**7,000**	**8,000**	**9,000**	**10,000**	**11,000**	**12,000**	**14,000**	**16,000**
3	21	18	17	15	14	13	12	12	12	12	12	12
4	28	25	22	20	19	17	16	16	16	16	16	16
5	36	31	28	25	23	22	21	20	20	20	20	20
6	43	37	33	30	28	26	25	23	23	23	23	23
7	62	54	48	44	41	38	36	34	34	34	34	34
8	71	62	55	50	47	44	41	39	39	39	39	39
9	80	70	62	57	53	49	46	44	44	44	44	44
10	90	78	70	64	59	55	52	50	50	50	50	50
11	100	87	78	71	66	61	58	55	55	55	55	55

Modification Factors used in the example

Reinforcement grade factor ψ_g = 1.3 for Grade 100
Lightweight, λ = 1.0 for Normal concrete
Epoxy Factor, ψ_e = 1.0
Size Factor, ψ_s = 0.8 for Sizes #3 through #6,
Size Factor, ψ_s = 1.0 for sizes #7 and larger
Casting position, ψt = 1.0
A_{tr} = total cross-sectional area of all transverse reinforcement within spacing s, that crosses the potential plane of splitting through the reinforcement being developed, in^2
c_b = lesser of: (a) the distance from center of a bar or wire to nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires being developed, in.
d_b = nominal diameter of bar, in.
ℓd = development length in tension of deformed bar, in.
K_{tr} = transverse reinforcement index, in.
s = center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, in.
f_c' = specified compressive strength of concrete, psi
f_y = specified yield strength for nonprestressed reinforcement,
psi

Annex B Specified Yield Strength for Design of Structural Members Using ChromX® ASTM A1035/A1035M Grade 100 [690] Reinforcement According to ICC AC 429

Specified Yield Strength for Design of Structural Members
Using ChromX® ASTM A1035/A1035M Grade 100 [690] Reinforcement

TYPE OF MEMBER	LONGITUDINAL REINFORCEMENT		TRANSVERSE REINFORCEMENT		
	Tension, psi (MPa)	Compression, psi (MPa)*	Shear, psi (MPa)	Torsion, psi (MPa)	Confinement, psi (MPa)
Beams and one-way slabs	100,000 (690)	100,000 (690)	80,000 (550)	60,000 (410)	N/A
Columns	100,000 (690)	100,000 (690)	80,000 (550)	60,000 (410)	100,000 (690)
Tension ties	80,000 (550)	N/A	N/A	N/A	N/A
Compression struts	N/A	100,000 (690)	N/A	N/A	N/A
Two-way slabs	100,000 (690)	100,000 (690)	60,000 (410)	60,000 (410)	N/A
Walls	100,000 (690)	100,000 (690)	80,000 (550)	N/A	100,000 (690)
Footings and pile caps	100,000 (690)	100,000 (690)	80,000 (550)	60,000 (410)	N/A
Mat foundations	100,000 (690)	100,000 (690)	80,000 (550)	N/A	N/A

* As per the ICC AC 429, May 2017, The specified yield strength in compression shall be taken as the stress corresponding to a strain of 0.35 percent.

Annex C Splicing Solution for ASTM A1035 Grade 100 According to ACI 318-08, ACI 318-11 and ACI 318-14

For ChromX® ASTM A1035 Grade 100 Type CS, CM and CL deformed bars, l_d shall be in accordance with Eq. A2-7 or Eq. A2.7M:

$$l_d = \frac{\left(\frac{f_y}{f_c'^{\,1/4}} - \phi 2400\omega\right)\alpha\beta_c\lambda}{\phi 76.3\left(\frac{c\omega + K_{tr}}{d_b}\right)}\, d_b \text{ (in.)} \qquad \text{(Eq. A2-7)}$$

where:

ϕ = 0.8
α = Ψ_t defined in ACI 318
= 1.3 for top cast bars
β_c = 1.0 for uncoated bars
λ = 1.0 for normal-weight concrete
c = spacing or cover dimension for reinforcing bar = $c_{min} + {d_b}/{2}$
c_b = bottom concrete cover for reinforcing bar being developed
c_{max} = maximum of (c_b, c_s)
c_{min} = small of minimum concrete cover or ½ of the spacing between bars
c_{si} = ½ of the bar spacing
c_{so} = side concrete cover for reinforcing bar
c_s = minimum of [c_{so}, c_{si} + 0.25 in. (6.35 mm)]
d_b = diameter of bar
n = number of bars being developed
R_r = relative rib area of the reinforcement
= $\frac{project\ rib\ area\ normal\ to\ bar\ axis}{nominal\ bar\ diameter \times center{-}to{-}center\ rib\ spacing}$
s = spacing of transverse refinement
ω = $0.1\left(\frac{c_{max}}{c_{min}}\right) + 0.9 \le 1.25$
K_{tr} = $(0.52 t_r t_d A_{tr}/sn)\sqrt{f'_c}$ (in.) (Eq. A2-8)
t_r = $9.6Rr + 0.28 \le 1.72$ (Eq. A2-9)
t_d = $0.78d_b + 0.22$ (in.) (Eq. A2-10)
$\left(\frac{c\,\omega + K_{tr}}{d_b}\right) \le 4.0$ (Eq. A2-11)

Table C1 – Sample Development Length Calculations Based on ICC ESR 2107 with Concrete Cover of 2 inches

	Development Length, in. (**Concrete Cover = 2 inches**)											
	Concrete Compressive Strength, psi											
Bar size:	3000	4000	5000	6000	7000	8000	9000	10000	11000	12000	14000	16000
3	17	16	15	14	13	13	12	12	12	11	11	10
4	23	21	20	19	18	17	17	16	16	15	14	14
5	29	26	25	23	22	21	21	20	19	19	18	17
6	35	32	30	28	27	26	25	24	23	23	22	21
7	41	38	35	33	32	31	30	29	28	27	26	25
8	53	48	45	43	41	39	38	37	36	35	33	32
9	66	61	57	54	51	49	47	46	44	43	41	40
10	82	75	70	67	64	61	59	57	55	54	51	49
11	99	91	85	81	77	74	71	69	67	65	62	59

Table C2 – Sample Development Length Calculations Based on ICC ESR 2107 with Concrete Cover of 4 inches

	Development Length, in. **(Concrete Cover = 4 inches)**											
	Concrete Compressive Strength, psi											
Bar size:	3000	4000	5000	6000	7000	8000	9000	10000	11000	12000	14000	16000
3	17	16	15	14	13	13	12	12	12	11	11	10
4	23	21	20	19	18	17	17	16	16	15	14	14
5	29	26	25	23	22	21	21	20	19	19	18	17
6	35	32	30	28	27	26	25	24	23	23	22	21
7	40	37	35	33	31	30	29	28	27	27	25	24
8	46	42	40	38	36	34	33	32	31	30	29	28
9	52	48	45	42	40	39	37	36	35	34	33	31
10	59	54	50	48	45	44	42	41	40	38	37	35
11	65	60	56	53	51	48	47	45	44	43	41	39

Annex C Splicing Solutions for ASTM A1035 Grade 690 (SI Units)

Development Length and Lap Splices Calculations as per ICC AC 429 / ICC ESR 2107

In SI units

$$l_d = \frac{\left(\frac{f_y}{f_c'^{1/4}} - \phi 57.4\omega\right)\alpha\beta_c\lambda}{\phi 1.83\left(\frac{c\omega + K_{tr}}{d_b}\right)} d_b \text{ (mm)} \quad \text{(Eq. A2-7M)}$$

$K_{tr} = (6.26 t_r t_d A_{tr} / sn)\sqrt{f'_c}$ (mm) (Eq. A2-8M)

$t_r = 9.6Rr + 0.28 \leq 1.72$ (Eq. A2-9M)

$t_d = 0.03db + 0.22$ (mm) (Eq. A2-10M)

$\left(\frac{c\,\omega + K_{tr}}{d_b}\right) \leq 4.0$ (Eq. A2-11M)

where:

ϕ = 0.8, strength reduction factor
α = Ψ_t defined in ACI 318
= 1.3 for top cast bars
β_c = 1.0 for uncoated bars
λ = 1.0 for normal-weight concrete
c = spacing or cover dimension for reinforcing bar = $c_{min} + {d_b}/{2}$
c_b = bottom concrete cover for reinforcing bar being developed
c_{max} = maximum of (c_b, c_s)
c_{min} = smaller of minimum concrete cover or ½ of the spacing between bars
c_{si} = ½ of the bar spacing
c_{so} = side concrete cover for reinforcing bar
c_s = minimum of [c_{so}, c_{si} + 0.25 in. (6.35 mm)]
d_b = diameter of bar
n = number of bars being developed
R_r = relative rib area of the reinforcement
$= \frac{project\ rib\ area\ normal\ to\ bar\ axis}{nominal\ bar\ diameter \times center-to-center\ rib\ spacing}$
s = spacing of transverse refinement
ω = $0.1\left(\frac{c_{max}}{c_{min}}\right) + 0.9 \leq 1.25$

Table C3 – Sample Development Length Calculations Based on ICC ESR 2107 with Concrete Cover of 75 mm

	Development Length, mm (with 75 mm clear cover)							
	Concrete Compressive Strength, MPa							
Bar size:	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**
16	660	606	566	535	510	490	472	456
20	826	757	708	669	638	612	590	570
25	1128	1035	967	914	872	836	805	779
32	1780	1633	1526	1443	1375	1319	1271	1229

Annex D Design Methodology -ChromX® ASTM A1035/A1035M Grade 100 [690] Bars According to ICC ESR 2107, ICC AC 429 and ACI 318-08, ACI 318-11 and ACI 318-14

A1.0 Structural design with high-strength reinforcing bars must be in accordance with ACI 318-08, ACI 318-11 and ACI 318-14, as modified by ACI ITG-6R and issued by the ICC ESR 2107. Table A1 provides the highest permissible values of the specified yield strengths for the ASTM A1035/A1035M Grade 100 bars to be used in the design of structural members.

A2.0 ACI ITG-6R provides further guidance for using high-strength steel reinforcing bars beyond current ACI 318-08, ACI 318-11 and ACI 318-14 limitations. The design and installation of ASTM A1035/A1035M Grade 100 bars must observe the following modifications to ACI 318, taken from ACI ITG-6R, identified in Sections A2.1 through A2.20 with the changes highlighted in BLUE.

The ACI 318 modified language in presented in boxes:

A2.1 Deformed Reinforcement: Modify ACI 318-14 Section 20.2.1.2 (ACI 318-11 or -08 Section 3.5.3.3) to read as follows:

20.2.1.2 Yield strength of nonprestressed bars and wires shall be determined by either (a) or (b):

(a) The offset method, using an offset of 0.2 percent in accordance with ASTM A370
(b) The yield point by the halt-of-force method, provided the nonprestressed bar or wire has a sharp-kneed or well defined yield point

Deformed Grade 100 reinforcing bars conforming to ASTM A1035 Types CS, CM and CL shall be permitted to be used subject to the specific modifications to ACI 318 given in this ICC ESR 2107 Report

20.2.1.3 Deformed bars shall conform to (a), (b), (c), (d), or (e):

(a) ASTM A615 – carbon steel
(b) ASTM A706 – low-alloy steel
(c) ASTM A996 – axle steel and rail steel; bars from rail steel shall be Type R
(d) ASTM A955 – stainless steel
(e) ASTM A1035 – low-carbon chromium steel

A2.2 Exposure Categories and Classes: Modify ACI 318-14 Section 19.3.1 (ACI 318-11 or -08 Section 4.2 by adding new Section 19.3.1.2 in ACI 318-14 (Section 4.2.2 in ACI 318-11 or -08):

19.3.1 Exposure categories and classes
19.3.1.2 *—ASTM A1035 Types CS, CM and CL Grade 100 bars shall be permitted to be in direct contact with other grades of steel except where the structure is in an aqueous environment. Aqueous environments include concrete exposure categories and classes where moisture or water contact is anticipated, as set forth in Section 4.2.1.*

A2.3 Bending: Modify ACI 318-14 Section 26.6.3 (ACI 318-11 or -08 Section 7.3) by adding new Section 26.6.3.1 (d) in ACI 318-14 (Section 7.3.3 in ACI 318-11 or -08):

26.6.3 Bending

26.6.3.1 Compliance requirements:

Reinforcement shall be bent cold prior to placement, unless otherwise permitted by the licensed design professional.

(b) Field bending of reinforcement partially embedded in concrete shall not be permitted, except as shown in the construction documents or permitted by the licensed design professional.

I Offset bars shall be bent before placement in the forms.

(d) Unbending of ASTM A1035 Grade 100 Types CS, CM and CL bars is prohibited.

A2.4 Redistribution of Moments: Modify ACI 318-14 Section 6.6.5 (ACI 318-11 or -08 Section 8.4) by adding the following:

6.6.5 Redistribution of moments in continuous flexural members

Redistribution of moments shall not apply to members containing ASTM A1035 Grade 100 types CS, CM and CL reinforcing bars.

A2.5 Strength Reduction Factor: **Under the 2012 and 2009 IBC:** Modify by ACI 318-11 or -08 Section 9.3.2.2 by replacing the second paragraph in ACI 318-11 or -08 with the following:

For sections reinforced with ASTM A1035 Grade 100 Types CS, CM and CL bars in which the net tensile strain in the extreme tension steel at nominal strength, ε_t, is between the limits for compression-controlled and tension-controlled sections, φ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as ε_t increases from the compression-controlled strain limit to 0.009 in accordance with Eq. A2-1:

$$\varphi = 0.45 + 50\varepsilon_t \qquad \text{(Eq. A2-1)}$$

Under the 2015 IBC: Modify Section 21.2.2 in ACI 318-14 as follows:

The strength reduction factor, *φ, shall be determined in accordance with (Eq. A2-1), where ε_t is the net tensile strain in the extreme tension steel at nominal strength as described in the paragraph above in Section A2.5.*

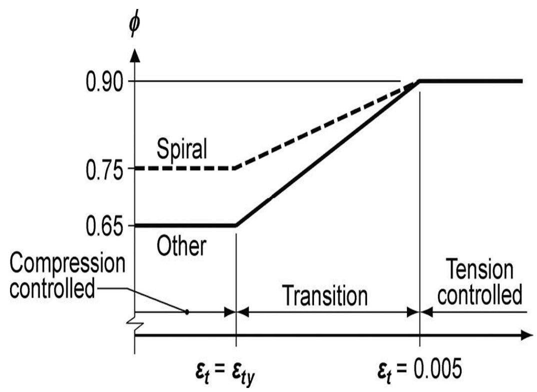
21.2.2 The strength reduction factor, *φ, shall be determined in accordance with (Eq. A2-1, where ε_t is the net tensile strain in the extreme tension steel at nominal strength.*

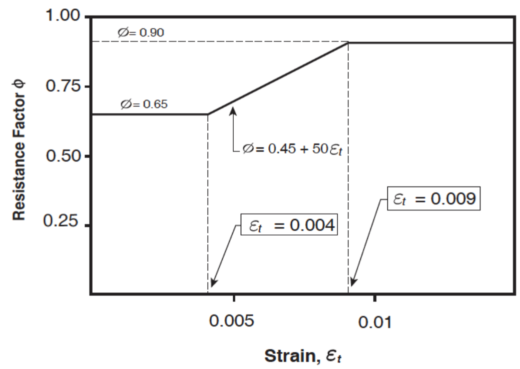
$$\Phi = 0.45 + 50\varepsilon_t \qquad \text{(Eq. A2-1)}$$

Table 21.2.2—Strength reduction factor ϕ for moment, axial force, or combined moment and axial force

$\varepsilon_t \leq \varepsilon_{ty} = 0.004$	Compression-controlled	0.65
$\varepsilon_{ty} < \varepsilon_t < 0.009$	Transition [1]	$0.45 + 50\varepsilon_t$
$\varepsilon_t \geq 0.009$	Tension-controlled	0.90

[1] For sections classified as transition, it shall be permitted to use ϕ corresponding to compression-controlled sections.





A2.6 Design Strength for Reinforcement: Modify ACI 318-14 Section 20.2.2.4 and Table 20.2.2.4a (ACI 318-11 or -08 Section 9.4) to read as follows:

20.2.2.4 *The specified yield strengths of ASTM A1035 Grade 100 bars Types CS, CM and CL used in the design of structural members shall be no more than the values shown in Table A1.*

A2.7 Control of Deflections: Modify ACI 318-14 Tables 7.3.1.1 and 9.3.12.1 (ACI 318-11 or -08 Table 9.5(a)) by adding the following note:

ACI 318-14 note 2 (ACI 318-11 or 08 note c) This table does not apply to members reinforced with ASTM A1035 Grade 100 bars. Deflections shall be computed in accordance with Sections 19.2.3.1 (for determination of modulus of rupture, f_r) and 24.2.3 of ACI 318-14 (Section 9.5.2.3 of ACI 318-11 or -08).

Replace ACI 318-14 Eq. (24.2.3.5a) (ACI 318-11 or -08 Eq. (9-8)) with Eq. A2-2.

24.2.3.5 For nonprestressed members, effective moment of inertia, I_e, shall be calculated by Equation A2-2 unless obtained by a more comprehensive analysis, but I_e shall not be taken greater than I_g.

$$I_e = \frac{I_{cr}}{1-\left(1-\frac{I_{cr}}{I_g}\right)\left(\frac{M_{cr}}{M_a}\right)^2} \leq I_g \quad \text{(Eq. A2-2)}$$

A2.8 Tensile stress: Modify ACI 318-14 Section 20.2.2.1 (ACI 318-11 or -08 Section 10.2.4) to read as follows:

20.2.2.1 *Tensile stress in ASTM A1035 Grade 100 bars Type CS, CM and CL shall be computed in accordance with Eq. A2-3 or A2-4:* *$f_s = 29{,}000\varepsilon_s$ (ksi) for $\varepsilon_s \le 0.00345$* (Eq. A2-3) *$f_s = 100$ ksi for $\varepsilon_s > 0.00345$* (Eq. A2-4)

A2.9 Compression-controlled Strain Limit: Modify ACI 318-14 Section 21.2.2.1 and Table 21.2.2 (ACI 318-11 or -08 Section 10.3.3) to read as follows:

21.2.2.1 *Sections are compression-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For ASTM A1035 Grade 100 Type CS, CM and CS reinforcement, it shall be permitted to set the compression-controlled strain limit equal to 0.004.*

A2.10 Tension-controlled Strain Limit: Modify ACI 318-14 Section 21.2.2 and Table 21.2.2 (ACI 318-11 or -08 Section 10.3.4) to read as follows:

21.2.2 *Sections reinforced with ASTM A1035 Grade 100 bars Type CS, CM and CL are tension-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or greater than 0.009 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with ε_t between the compression-controlled strain limit and 0.009 constitute a transition region between compression-controlled and tension-controlled sections.*

A2.11 Volumetric Spiral Reinforcement Ratio: Modify ACI 318-14 Section 25.7.3.3 (ACI 318-11 or -08 Section 10.9.3) to read as follows:

25.7.3.3 Volumetric spiral reinforcement ratio ρ_s shall satisfy Eq. (25.7.3.3). $\rho_s \ge 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ (25.7.3.3) where the value of f_{yt} shall not be taken greater than 100,000 psi. *For f_{yt} greater than 60,000 psi (410 MPa) and ASTM A1035 Grade 100 reinforcement, lap splices according to ACI 318-14 Section 25.7.3.6 (ACI 318-11 or -08 Section 7.10.4.5(a)) shall not be used.*

A2.12 Slenderness Effects: Modify ACI 318-14 Section 6.2.6 (ACI 318-11 or -08 Section 10.10.2) to read as follows:

6.2.6 *When slenderness effects are not neglected as permitted by Section 6.2.5 in ACI 318-14 (Section 10.10.1 in ACI 318-11 or -08), the design of compression members, restraining beams, and other supporting members reinforced with ASTM A1035 Grade 100 bars Type CS, CM and CL shall be based on the factored forces and moments from a second-order analysis satisfying Section 6.6.4 in ACI 318-14 (Section 10.10.5 in ACI 318-11 or -08). These members shall also satisfy Sections 6.2.6 and 6.6.4.6.4 in ACI 318-14 (Sections 10.10.2.1 and 10.10.2.2 in ACI 318-11 or -08). The dimensions of each member cross section used in the analysis shall be within 10 percent of the dimensions of the members shown on the contract documents or the analysis shall be repeated.*

A2.13 Shear Strength Provided by Concrete for Nonprestressed Members: Modify ACI 318-14 Section 22.5.5.1 (ACI 318-11 or -08 Section 11.2.1.1) by adding the following:

> 22.5.5.1 For nonprestressed members without axial force, V_c shall be calculated by:
>
> $V_c = 2\lambda\sqrt{f_c{}'}b_w d$ (22.5.5.1)
>
> Unless a more detailed calculation is made in accordance with Table 22.5.5.1.
>
> *For one-way slabs reinforced with ASTM A1035 Grade 100 bars Type CS, CM and CS where (1) ρ is less than one percent; (2) no significant axial load is present; and (3) no shear reinforcement is provided, V_c shall be computed in accordance with Eq. A2-6 or Eq. A2-6M:*
>
> $V_c = \frac{73}{39+2.1d}\sqrt{f'}\ b_w d$ (lb) *(Eq. A2-6)*
>
> $V_c = \frac{154}{1000+2.1d}\sqrt{f'}b_w d$ (N) *(Eq. A2-6M)*

A2.14 Shear Reinforcement Design Strength: Modify ACI 318-14 Section 20.2.2.4 and Table 20.2.2.4a (ACI 318-11 or -08 Section 11.4.2) to read as follows:

> 20.2.2.4 *The values of f_y and f_{yt} used in design of shear reinforcement shall not exceed 60,000 psi (410 MPa), except the value may be increased to 80,000 psi (550 MPa) for beams and walls with ASTM A1035 Grade 100 Type CS, CM and CS reinforcement where appearance and serviceability due to shear cracking are not critical design considerations.*

TABLE A1—SPECIFIED YIELD STRENGTHS FOR DESIGN OF MEMBERS USING ASTM A1035/A1035M Type CS, CM, and CL GRADE 100 REINFORCEMENT

TYPE OF MEMBER	LONGITUDINAL REINFORCEMENT		TRANSVERSE REINFORCEMENT		
	Tension, psi (MPa)	Compression, psi (MPa)**	Shear, psi (MPa)	Torsion, psi (MPa)	Confinement, psi (MPa)
Beams and one-way slabs	100,000 (690)	100,000 (690) **	80,000 (550)	60,000 (410)	N/A
Columns	100,000 (690)	100,000 (690) **	80,000 (550)	60,000 (410)	100,000 (690)[2]
Tension ties	80,000 (550)	N/A	N/A	N/A	N/A
Compression struts	N/A	100,000 (690) **	N/A	N/A	N/A
Two-way slabs	100,000 (690)	100,000 (690) **	60,000 (410)	60,000 (410)	N/A
Walls	100,000 (690)	100,000 (690) **	80,000 (550)	N/A	100,000 (690)[3]
Footings and pile caps	100,000 (690)	100,000 (690) **	80,000 (550)	60,000 (410)	N/A
Mat foundations	100,000 (690)	100,000 (690) **	80,000 (550)	N/A	N/A

[1]N/A = Not applicable
[2]Spirals and ties
[3]Ties

**** AC 429 2017 permits the compression Yield Strength up to stress corresponding to 0.0035 strain (690 MPa)**

A2.15 Minimum Shear Reinforcement:

Concrete members reinforced with ASTM A1035 Grade 100 Type CS, CM and CS bars shall comply with the minimum shear reinforcement requirements in ACI 318-14 (ACI 318-11 or -08)

A2.16 Development of Deformed Bars in Tension: Modify ACI 318-14 Section 25.4.2 (ACI 3218-11 or -08 Section 12.2) by deleting Sections 25.4.2.2 and 25.4.2.3 in ACI 318-14 (Sections 12.2.2 and 12.2.3 in ACI 318-11 or -08), renumbering Section 25.4.2.4 in ACI 318-14 to become Section 25.4.2.3 (Section 12.2.4 in ACI 318-11 or -08 to become 12.2.3) and replacing Section 25.4.2.2 in ACI 318-14 (Section 12.2.2 in ACI 318-11 or -08) with the following:

25.4.2.2 *For ASTM A1035 Grade 100 Type CS, CM and CL deformed bars, l_d shall be in accordance with Eq. A2-7 or Eq. A2.7M:*

$$l_d = \frac{\left(\frac{f_y}{f_c'^{1/4}} - \phi 2400\omega\right)\alpha\beta_c\lambda}{\phi 76.3\left(\frac{c\omega + K_{tr}}{d_b}\right)} d_b \text{ (in.)} \qquad \text{(Eq. A2-7)}$$

where:

ϕ = 0.8
α = Ψ_t defined in ACI 318
= 1.3 for top cast bars
β_c = 1.0 for uncoated bars
λ = 1.0 for normal-weight concrete
c = spacing or cover dimension for reinforcing bar = $c_{min} + {}^{d_b}/_2$
c_b = bottom concrete cover for reinforcing bar being developed
c_{max} = maximum of (c_b, c_s)
c_{min} = small of minimum concrete cover or ½ of the spacing between bars
c_{si} = ½ of the bar spacing
c_{so} = side concrete cover for reinforcing bar
c_s = minimum of [c_{so}, c_{si} + 0.25 in. (6.35 mm)]
d_b = diameter of bar
n = number of bars being developed
R_r = relative rib area of the reinforcement
$= \frac{project\ rib\ area\ normal\ to\ bar\ axis}{nominal\ bar\ diameter \times center-to-center\ rib\ spacing}$
s = spacing of transverse refinement
ω = $0.1\left(\frac{c_{max}}{c_{min}}\right) + 0.9 \leq 1.25$
K_{tr} = $(0.52 t_r t_d A_{tr}/sn)\sqrt{f'_c}$ (in.). (Eq. A2-8)
t_r = $9.6Rr + 0.28 \leq 1.72$ (Eq. A2-9)
t_d = $0.78d_b + 0.22$ (in.) (Eq. A2-10)
$\left(\frac{c\,\omega + K_{tr}}{d_b}\right) \leq 4.0$. (Eq. A2-11)

In SI units

$$l_d = \frac{\left(\frac{f_y}{f_c'^{1/4}} - \phi 57.4\omega\right)\alpha\beta_c\lambda}{\phi 1.83\left(\frac{c\omega + K_{tr}}{d_b}\right)} d_b \text{ (mm)} \qquad \text{(Eq. A2-7M)}$$

K_{tr} = $(6.26 t_r t_d A_{tr}/sn)\sqrt{f'_c}$ (mm) (Eq. A2-8M)
t_r = 9.6Rr + 0.28 ≤ 1.72 (Eq. A2-9M)
t_d = 0.03db + 0.22 (mm) (Eq. A2-10M)
$\left(\frac{c\,\omega + K_{tr}}{d_b}\right) \leq 4.0$ (Eq. A2-11M)

A2.17 Development of Deformed Bars in Compression: Modify ACI 318-14 Section 25.4.9 (ACI 318-11 or -08 Section 12.3) by adding the following:

> 25.4.9 Development of deformed bars and deformed wires in compression. *The specified yield strength f_y in compression of ASTM A1035 Grade 100 Type CS, CM, and CL bars is limited to 80,000 psi (550 MPa) to stress corresponding to 0.0035 strain maximum.*
>
> 25.4.9.1 Development length ℓdc for deformed bars and deformed wires in compression shall be the greater of (a) and (b)
> (a) Length calculated in accordance with 25.4.9.2
> (b) 8 in.
>
> 25.4.9.2 ℓ_{dc} shall be the greater of (a) and (b), using the modification factors of 25.4.9.3:
> (a)$\left(\frac{f_y \psi_r}{50\lambda\sqrt{f_c{}'}}\right)d_b$
>
> (b) $0.0003 f_y \psi r d b$
>
> 25.4.9.3 For the calculation of ℓdc, modification factors shall be in accordance with Table 25.4.9.3, except ψ_r shall be permitted to be taken as 1.0.

A2.18 Mechanical and Welded Splices: Modify ACI-14 Section 25.5.7.1 (ACI 318-11 or -08 Section 12.14.3.1) to read as follows:

> ~~**25.5.7.1** A mechanical or welded splice shall develop in tension or compression, as required, at least 1.25f_y of the bar.~~
> ***25.5.7.1*** *Mechanical splices shall be permitted when specifically recognized for use with ASTM A1035 Grade 100 reinforcing bars in an ICC-ES evaluation report. Welding of bars is prohibited.*

A2.19 Modification of Moments for Two-way Slab Systems: Modify ACI 318-14 Section 8.10.4.3 (ACI 38-11 or -08 Section 13.6.7) to read as follows:

> 8.10.4.3 *Modification of negative and positive factored moments using moment redistribution is prohibited in two-way slabs reinforced with ASTM A1035 Grade 100 Type CS, CM, and CL bars.*

A2.20 Use in Earthquake-resistant structures: Modify ACI-14 Section 20.2.2.4 (ACI 318-11 or -08 Section 21.1.5) by adding new section as follows:

> 20.2.2.5 *Use of ASTM A1035 Grade 100 Type CS, Cm, and CL reinforcing bars as longitudinal reinforcement in a structural member that is part of the seismic-force-resisting system of a building assigned to SDC D, E or F is prohibited. The use of ASTM A1035 Grade 100 Type CS, CM, and CL reinforcing bars as transverse reinforcement is permitted, provided f_{yt} is limited to 60,000 psi (410 MPa) maximum for computing shear strength.*

Modify ACI 318-14 Section 18.4.1.1 (ACI 318-11 or -08 Section 21.3.2) to read as follows:

> 18.4.1.1 *Where Grade 60 (420 MPa) reinforcing bars are used for column longitudinal reinforcement and ASTM A1035 Grade 100 Type CS, CM, and CL reinforcing bars are used for beam longitudinal reinforcement in intermediate moment frames, the flexural strength requirements (strong-column-weak beam) in Section18.7.3.2 in ACI 318-14 (ACI 318-11 or -08 Section 21.6.2.2) shall be met using the actual strengths of the beams containing ASTM A1035 Grade 100 Type CS, CM and CL reinforcement. A nonlinear analysis shall be conducted to determine M_{nb}. The required shear strength of the beam shall be determined based on M_{nb} from such nonlinear analysis. Permitted methods of nonlinear analysis are given in ACI ITG-6R.*

Annex E ChromX® Design Guidelines and Specifications Compared to Other High Strength Reinforcement

Summary of Specifications, Codes, Design Values permitted, allowed Structures, and Restrictions for High Strength Steel Rebars Grade 100 [690]

Product	**DYWIDAG Grade 100 THREADBARS**	**SAS STRESSTEEL Grade 97 THREADBARS**	**ASTM A615 Grade 100**	**CHROMX® - 100ksi**
ASTM Standard	No ASTM Standard as Reinforcing Bar	No ASTM Standard As a Reinforcing Bar	ASTM A615	ASTM A1035
ICC ESR Report #	ICC ESR 3367 *	ICC ESR 1163	No ICC ESR REPORT	ICC ESR 2107
ICC AC Report #	ICC AC 237	ICC AC 237	No ICC AC Report	ICC AC 429
NYC Bulletin #	No Bulletin	2016-001	No Bulletin	2015-036
ICC ACCEPTANCE APPROVAL Basis	Based on AC 237 approved February 2015	Based on AC237 approved February 2015	No Acceptance Criteria	Based on AC429 approved May 2017
Applicable Design Code	ACI 318 -14 with ICC ESR 3367 Modifications	ACI 318 -11 with ICC ESR 1163 Modifications	ACI 318-19	ACI 318-19 ACI 318 -08, ACI 318-11 and ACI 318-14 as modified by ICC ESR 2107
Permitted Seismic Design Categories (SDC)	Structural elements in SDC A and B only except beams and slabs. Shall not be used in SDC C, D, E, and F	Structural elements in SDC A and B only except beams and slabs. Shall not be used in SDC C, D, E, and F	ACI 318-19	All structural elements of SDC A, B, and C. For SDC D, E, and F, limited to slab systems, foundations and structural components not designated as part of the seismic force-resisting system.
Design Values				
f_y for tension	*fy can be used as per AC 237 up to 100 ksi*	80 ksi < $f_y \leq$ 100 ksi	100 ksi per ACI 318-19	100 ksi
f_y for compression		80 ksi < $f_y \leq$ 100 ksi	80 ksi per ACI 318-19	80 ksi per ACI 318-19 Up to 100 ksi per ICC AC 429
f_y for shear	60 ksi	60 ksi	60 ksi as per ACI 318-19	60 ksi per ACI 318-19 80 ksi per ICC AC 429
f_y for torsion	60 ksi	60 ksi	60 ksi	60 ksi
f_y for lateral support to longitudinal steel and confinement by spirals	100 ksi	100 ksi	100 ksi (as per ACI 318)	100 ksi (as per ACI 318)
f_y for lateral support to longitudinal steel and confinement by non-spiral reinforcement	80 ksi	80 ksi	100 ksi (as per ACI 318)	100 ksi (as per ACI 318)

Restrictions	DYWIDAG Grade 100 THREADBARS	SAS STRESSTEEL Grade 97 THREADBARS	ASTM A615 Grade 100	CHROMX® - 100ksi
Specified compressive strength of concrete	Must be in the range from 6 to 12 Ksi	Must be in the range from 6 to 12 ksi	As per ACI 318	Must be in the range from 4 to 16 ksi
Permitted applications as reinforcement and limitations	1) Longitudinal reinforcement for resisting flexure, axial force and shrinkage and temperature 2) Lateral support of longitudinal bars or confinement 3) Shear reinforcement, including shear friction 4) Torsional reinforcement (longitudinal and transverse) 5) Must not be used in Beams or slabs 6) AC237 is limited to uncoated reinforcement in normal-weight-concrete with no more than 12 in. of fresh concrete placed below horizontal bars. 7) Shall not be used in buildings assigned to SDC C, D, E, r F.	1) Longitudinal reinforcement for resisting flexure, axial force and shrinkage and temperature 2) Lateral support of longitudinal bars or confinement 3) Shear reinforcement, including shear friction 4) Torsional reinforcement (longitudinal and transverse) 5) Shall not be used in Beams or Slabs 6) AC237 is limited to uncoated reinforcement in normal-weight-concrete with no more than 12 in. of fresh concrete placed below horizontal bars. 7) Shall not be used in buildings assigned to SDC C, , E, or F.	1) Allowed to be used in Gravity loads as per ACI 318-19 2) Allowed to be used as confinement reinforcement in columns of special moment frames and special shear wall boundary elements by ACI 318. ASTM A615 NOTE 1—Grade 100 [690] reinforcing bars were introduced in this specification in 2015. In contrast to the lower grades, which have ratios of specified tensile strength to specified yield strength that range from 1.31 to 1.5, Grade 100 [690] reinforcing bars have a ratio of specified tensile strength to specified yield strength of 1.15. Designers should be aware that there will, therefore, be a lower margin of safety and reduced warning of failure following yielding when Grade 100 [690] bars are used in structural members where strength is governed by the tensile strength of the reinforcement, primarily in beams and slabs. If this is of concern, the purchaser has the option of specifying a minimum ratio of tensile strength to actual yield strength. Consensus design codes and specifications such as "Building Code Requirements for Structural Concrete (ACI 318)" may not recognize Grade 100 [690] reinforcing bars: therefore the 125 % of specified yield strength requirements in tension and compression are not applicable. Mechanical and welded splices should meet a minimum specified tensile strength of 115000 psi [790 MPa].	1) Allowed to be used in all gravity loads as per ACI 318-19. 2) Allowed to be used as confinement reinforcement in columns of special moment frames and special shear wall boundary elements by ACI 318. .

Allowed structural elements	DYWIDAG Grade 100 THREADBARS	SAS STRESSTEEL Grade 97 THREADBARS	ASTM A615 Grade 100	CHROMX® - 100ksi
Beam	No	No	Yes	Yes
Column	Yes**	Yes**	Yes	Yes
Moment Frame	Columns only**	Columns only**	Yes up to 80 KSI	Yes**
Slab	No	No	Yes up to 80 KSI	Yes**
Wall (including boundary elements)	Yes**	Yes**	Yes up to 80 KSI	Yes**

* ESR 3367 does not mention what fy stress to be used in Tension. Or compression. It is mentioned only in AC 237.
** Only if it is not part of a special seismic System.

Annex F Mechanical Couplers Made for Use with ChromX® ASTM A1035 Grade 100 [690] Steel Reinforcing Bars

Various types and styles of couplers specified by the structural design engineer are available from the coupler manufacturers below. Please contact the couplers suppliers for details and pricing.

Couplers Manufacturers	APPLICATIONS			
	Grade 60 – 80 [420 – 550] Designs		Grade 100 [690] Designs	
	Type 1[(1)]	Type 2[(2)]	Type 1[(3)]	Type 2[(4)]
	125 percent of specified yield strength	150 percent of specified yield strength	125 percent of specified yield strength	Actual Tensile strength
Barsplice[5] *www.barsplice.com* Grip-Twist® 'XT' Zap Screwlok® 'FXT' BPI® 100KSI Barsplicer Zap Screwlok® 'XT' Grip-Twist® Bargrip® XL BPI® 100KSI Barsplicer	#4 through #18 #4 through #18 #4 through #18 #4 through #18 #4 through #18 #4 through #18 #4 through #11	#4 through #18 #4 through #18 #4 through #11	#4 through #18 #4 through #18 #4 through #18 #4 through #18 #4 through #11	#4 through #11
Dextra[5] *www.dextragroup.com* Standard Coupler Small Anchor Plate Large Anchor Plate	#4 through #18 #4 through #18 #4 through #18	#4 through #18 #4 through #18 #4 through #18	#4 through #18 #4 through #18 #4 through #18	#4 through #18 #4 through #18 #4 through #18
Erico- PENTAIR[5] *www.erico.com* Lenton® Standard A32 Lenton® Lock	#3 through #14 #3 through #18	#3 through #14 #3 through #18	#3 through #14 #3 through #18	#3 through #14 #3 through #18
NMB Splice Sleeve[5] www.splicesleeve.com	#6 through #14	#6 through #14	#6 through #14	#6 through #14
HRC[5] *www.hrc-usa.com* 100 Series T-Headed 400 Series Couplers	#5 through #18 #4 through #18	#5 through #18 #4 through #18	#5 through #18 #4 through #18	#5 through #18 #4 through #18

(1) Type 1 mechanical couplers for Grade 60-80 designs are full mechanical splice connections that shall develop in tension or compression, as required, at least 125 percent of specified yield strength of the spliced bar as per ACI 318.
(2) Type 2 mechanical couplers for Grade 60-80 designs are full mechanical splice connections that shall meet Type 1 requirements and develop in tension or compression, as required, at least 150 percent of specified yield strength, which equates to the specified tensile strength of the spliced bar as per ACI 318.
(3) Type 1 mechanical couplers for Grade 100 designs are full mechanical splice connections that shall develop in tension or compression, as required, at least 125 percent of specified yield strength of the spliced bar as per ACI 318 and ACI ITG-6.
(4) Type 2 mechanical couplers for Grade 100 designs are full mechanical connections that shall meet Type 1 requirements and develop in tension or compression, as required, the actual tensile strength of the spliced bar as per ACI ITG-6 and ICC-ES AC429.
(5) Couplers made from MMFX materials are available through these coupler suppliers in select sizes and styles.