CORROSION RESISTANT ALLOY STEEL (MMFX) REINFORCING BAR IN BRIDGE DECKS

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16. Abstract

Epoxy coated reinforcement (ECR) has gained mainstream acceptance since the early 1980s as a means to extend the useful life of highway structures. Research to date estimates additional service life between 5 to 15 years is provided by the epoxy coating. To achieve a 75 year service life, however, alternate materials must be considered. Typically, the only alternative to ECR has been the use of more expensive corrosion resistant materials, such as stainless steel. Another alternative, MMFX reinforcement, has potential to resist corrosion and yet remain economical in initial cost to use.

Corrosion resistance, mechanical properties, and design criteria were investigated with MMFX reinforcement. The MMFX steel does exhibit corrosion resistance, higher yield strength, and a lower life cycle cost than ECR. The bond strength for MMFX was comparable to uncoated reinforcement, but results from flexure testing of reinforced concrete beams imply that the lap length needs further study. Therefore, the designer should reduce or eliminate lapped joints, either by mechanical splices, or requiring the contractor to supply the exact length reinforcing bars as detailed on the plan sheets. 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, in highly congested urban areas, when life cycle costs are justified.

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MICHIGAN DEPARTMENT OF TRANSPORTATION
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Corrosion Resistant Alloy Steel (MMFX) Reinforcing Bar in Bridge Decks

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Construction and Technology Division
Research Project TI-1980
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EXECUTIVE SUMMARY

Epoxy-coated reinforcement (ECR) has gained mainstream acceptance since the early 1980s as a means to extend the useful life of highway structures. Research to date estimates additional service life between 27 to 40 years is provided by the epoxy coating. To achieve a 75 year service life, however, alternate materials must be considered.

Typically, the only alternative to ECR has been the use of more expensive corrosion resistant materials, such as stainless steel. Another alternative, MMFX reinforcement, has potential to resist corrosion and yet remain economical in initial cost to use.

The mechanical, chemical, and corrosion properties of MMFX were investigated. Evaluation samples as well as quality control samples from its use in bridge construction were tested. The MMFX was found to have substantially higher yield and tensile strengths (140 ksi and 160 ksi, respectively) than conventional Grade 60 steel, with somewhat decreased elongation. The MMFX reinforcement tested, however, met the ASTM A615 Grade 75 criteria for steel reinforcement.

The corrosion properties of MMFX were investigated through a literature review of projects conducted by other state DOT's and from limited in-house experiments. The research to date indicates a corrosion resistance typically four to eight times that of uncoated reinforcement, and a one-third to two-thirds lower corrosion rate. That translates to an initial bridge deck service life estimate of 52 years before repairs are needed. Life cycle cost analysis over a 90-year analysis period indicated a $31/yd^2 lower cost of MMFX compared to ECR.

Design criteria were investigated due to the significantly higher yield and tensile strengths of MMFX reinforcement. For instance, the AASHTO LRFD code limits design reinforcement yield strength to 75 ksi. Therefore, it is possible to have an over-reinforced section using MMFX due to its higher yield strength, and possible reduction of ductility at ultimate load levels. Other research indicated that the bond strength for MMFX was comparable to uncoated reinforcement, but results from flexure testing of reinforced concrete beams imply that the lap length needs further study. Therefore, the designer should reduce or eliminate lapped joints, either by mechanical splices, or requiring the contractor to supply the exact length reinforcing bars as detailed on the plan sheets.

Despite these potential limitations, 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, in highly congested urban areas, when life cycle costs are justified.

ACTION PLAN

1. Identify bridge construction projects where MMFX reinforcement should be used (Construction and Technology/Design).

2. Monitor design and construction phases of new projects implementing MMFX reinforcement to identify any design issues and verify supply timeliness (Construction and Technology).

3. Continue monitoring performance of bridge deck constructed in 2003 with MMFX reinforcement (R01 of 82022) and note any performance issues as compared to the adjacent bridge deck constructed with epoxy coated reinforcement (Construction and Technology).

4. After ten successful bridge projects implementing MMFX reinforcement, place the special provision for microcomposite steel on the frequently used special provision (FUSP) list, with corresponding use statement and criteria (Construction and Technology/Design).

INTRODUCTION

The deterioration of reinforced concrete structures has become a major liability for highway agencies. The cost due to corrosion of steel and reinforced concrete alone is significant, at $3.9 billion annually *[1]*. Bridges built in the early 1970's in urban areas generally have had deck overlays and even replacement in less than 30 years.

The primary cause of this deterioration is the corrosion of steel reinforcing bars due to chlorides. During the winter months, many highway agencies use a large amount of chloride-based deicing chemicals. Chloride ions then reach the reinforcing steel by penetrating the concrete via the pore water and through cracks. The corrosion products of steel reinforcing bars occupy a volume three to six times the volume of the original steel. This increase in volume induces stresses in the concrete that result in cracks, delaminations, and spalls. Cracked or delaminated concrete accelerates the corrosion process by providing an easy pathway for the water and chlorides to reach the steel. When 30 percent of the deck surface area is deteriorated (spalling, delamination, or patch areas), the useful service life has expired and rehabilitation is needed.

Epoxy-coated reinforcement (ECR) has gained mainstream acceptance since the early 1980s as a means to extend the useful life of highway structures. The epoxy coating prevents moisture and chlorides from reaching the surface of the reinforcing steel by acting as a barrier. Research to date estimates additional service life between 27 to 40 years is provided by the epoxy coating. To achieve a 75 year service life, however, alternate materials must be considered.

Typically, the only alternative to ECR has been the use of more expensive corrosion resistant materials such as stainless steel. Although highly effective in resisting corrosion, the initial cost of stainless steel precludes its widespread use. Another alternative, MMFX reinforcement (defined later), has potential to resist corrosion and yet remain economical in initial cost to use. Therefore, the Michigan Department of Transportation (MDOT) is conducting research into MMFX steel for use as an alternative material to achieve a 75 year bridge service life.

LITERATURE REVIEW

<u>Iowa</u> - The first State DOT to use MMFX was Iowa, and their demonstration project was completed in 2001. The structure, IA 520 E.B. over South Beaver Creek, is a 274 ft long, 39.4 ft wide deck on prestressed concrete I-beams. A total of 82,669 lb of MMFX steel was used for deck reinforcement. The westbound structure was constructed with conventional ECR. Reference electrodes were installed at various locations to monitor corrosion activity. The research project was to continue monitoring the structure with load testing, reference electrode readings, and visual observation, over a 12 month period. Initial corrosion readings indicated the epoxy-coated deck had about six times the corrosion current as compared to MMFX, possibly due to defects in the coating that may have exposed the steel. The corrosion currents stabilized as the concrete cured, at about 150 days *[2]*.

Subsequently, the following phase was to focus on accelerated laboratory testing, as field monitoring would necessarily take years to complete. After 40 weeks of exposure using the ASTM G109 accelerated corrosion test procedure, no corrosion was present on either the MMFX or the undamaged epoxy coated reinforcement. However, epoxy coated reinforcement with intentionally induced defects underwent corrosion within 15 to 30 weeks. The uncoated reinforcement (control) underwent corrosion in five weeks. It was concluded that the 40 week testing may not constitute a prediction of life expectancy without further study *[3]*.

Florida - Experimental flexural testing on concrete beam specimens with # 6 bar MMFX and Grade 60 continuous reinforcement has shown that the ductile behavior is identical, up to the yield loading of Grade 60 reinforcement, and the cracking loads also were similar. When 10 in lapped sections were tested, however, it was noted that the beam specimens with # 6 bar MMFX and Grade 60 reinforcement failed before reinforcement yielding. At a lap length of 30.5 in, the Grade 60 reinforcement yielded prior to flexural failure of the beam, but the MMFX specimen did not. Although the Grade 60 reinforced beam failed progressively from 30 to 40 kips, with increased deflection of 2.5 in, the MMFX specimen failed at 48 kips, with only 0.9 in deflection. Lap splices or hook embedment that are adequate for yielding of Grade 60 reinforcement will not be appropriate for yielding of MMFX reinforcement. Therefore special attention is needed to detailing the deck reinforcement, minimizing or eliminating lapped segments *[4]*.

MMFX/West Virginia - A study commissioned by MMFX Steel Corporation in 2002 investigated bending behavior of high strength concrete beams (8 and 11 ksi) reinforced with MMFX. Although the beams exhibited significant elongation before compression failure under four point bending, it was noted that the MMFX Young's modulus (E) varied at different stress levels. Service load stress levels need to be limited to about 40 ksi, based on a limiting crack width of 0.016 in, and deflection of L/360. MMFX strain values will be 30 percent higher at 40 ksi stress compared to conventional Grade 60 steel with $E = 29 \times 10^6$ psi *[5]*.

Virginia - The Virginia Transportation Research Council (VTRC) had compared the chloride corrosion threshold of MMFX to stainless and uncoated reinforcement, and the test used stock MMFX bars with mill scale. The research results showed that "*Perhaps because of the presence of mill scale, the new MMFX and the 2101 LDX materials had significant corrosion rates even from the very beginning of the exposure of the concrete blocks to salt [6].*" The paper gave a time to corrosion of 244 to 247 days, as compared to 90 to 95 days with uncoated bar, or 2.6 times as long. However, the corrosion rate of MMFX was considerably less (3.55 $\mu A/in^2$) than that of uncoated bar (11.7 $\mu A/in^2$), and near the low end of 'moderate' corrosion (3.23-6.45 $\mu A/in^2$). The authors recommend the use of MMFX or stainless-clad reinforcing bars for urban and heavily traveled interstate routes.

FHWA - In a report published by the FHWA, the concrete deck time-to-repair for uncoated bar reinforcement is estimated at 9 years with 2.5 percent delamination *[7]*. The corrosion rate for uncoated bar in this study was given as 20.62 $\mu A/in^2$. The concrete deck time-to-repair for epoxy-coated reinforcement is estimated at 27 years with 2.5 percent surface delamination, and a maximum of 42 years (repairing epoxy coating and sealing cracks in deck).

Although MMFX was not studied, the service life of decks reinforced with uncoated, epoxy coated, and stainless reinforcement was determined.

South Dakota - The University of Kansas Center for Research was commissioned by South Dakota DOT (SDDOT) to perform a study evaluating MMFX reinforcement, its material properties, corrosion resistance, and applicability to AASHTO standard (16^{th} edition) bridge design criteria. This paper had concluded that epoxy-coated reinforcement performed better than MMFX, and that "*The corrosion rate for MMFX Micro composite steel is between one-third and two-thirds that of conventional reinforcing steel…MMFX micro composite steel appears to corrode when the surface is exposed to moist air and chlorides but not in contact with concrete or submerged in water [8].*"

The conclusion was based, in part, on corrosion rates of (simulated) damaged epoxy-coated bar that were normalized over the entire surface area. Normalizing corrosion current density across the entire bar area would suggest more uniform corrosion would occur, versus the very rapid pitting and section loss observed from testing, and should be used to characterize the corrosion mechanism of epoxy coated steel. The authors acknowledge the fact that "*…very high corrosion rates can occur in localized areas, especially when the cathode is unprotected as it is in these tests ([8], pg. 63).*"

In terms of repeatability, the measured corrosion rates' variance ranged from 41 to 110 percent. The large variances are due to taking measurements of corrosion rates at discrete intervals of time, rather than monitoring reinforcement corrosion rates continuously over time.

The report authors mixed laboratory results with field observations when attempting to calculate life cycle cost estimates. Despite the laboratory results showing very high corrosion rates, ECR is given a 30 to 40 year life before repair. The report states, "*The estimates for epoxy-coated steel are based on the experience that, over the past 25 years, no bridge decks with epoxy-coated reinforcement required repair due to corrosion damage ([8], pg. 76).*"

The time-to-repair estimates for all materials other than ECR were based on chloride content required for corrosion initiation, the time it takes for the chlorides to reach the threshold concentration via penetration through cracks, and the time to produce 0.001 in of corrosion products. The chloride diffusion rate of 0.0427 lb/yd^3 per month was taken from a previous study (chloride accumulation from migration through cracks only).

For MMFX, the average chloride corrosion threshold of 5.36 lb /yd^3 means that accumulation of chlorides would take 125.5 months (10.5 years) and with a corrosion rate of 0.024 mil/yr, the time to produce one mil of corrosion products would take 41.6 years. The MMFX initial time-to-repair should be given as a total of 10.5 + 41.6 = 52.1 years instead of the 35 years in Table 5.12 *([8], pg. 76)*. An appraisal report concluded, "*In short, the …arbitrary selection of certain data points in the case of MMFX steel are unacceptable practices that destroy confidence in the results and conclusions of the research [9].*"

EVALUATION OF MMFX

The manufacturer, MMFX Steel Corporation, had produced a prototype bar (MMFX-1), but by 2001 had phased it out for MMFX-2, with a different chemical composition. Evaluation samples of MMFX-2 reinforcement in nominal sizes of 1/2 in, 5/8 in, and 3/4 in (# 4, 5, and 6), were sent to MDOT in August of 2001. Originally named for the steel's microstructure (thus requiring the numerical suffix as the formula changed), MMFX is now the trade name for the product. The steel's microstructure consists of *"…untransformed nano sheets of austenite between laths of dislocated martensite, resulting in a virtually carbide free steel."* The product literature also states that *"...without the creation of continuous paths of carbides, micro galvanic cell formation is minimized during production. Hence, the control of MMFX steel's morphology (form and structure) has resulted in its significantly superior material properties [10]."* The evaluation and testing was designed to characterize the physical and mechanical properties of the MMFX reinforcement, as well as study its relative corrosion resistance.

<u>Mechanical Properties</u> - The MMFX evaluation samples had a uniform dark grey-black color, with tightly adhered mill scale evident, as shown in a bend test specimen in Figure 1. The evaluation samples were tested in accordance with current MDOT practice, which references the American Society for Testing and Materials (ASTM) test method A370 for yield, tensile, and ductility (bend test) *[11]*. For the bend test, the bars were bent around mandrels of diameter 3d (#4), and 5d (#5, #6). The bars exhibited no signs of cracking on the outside bend radius, nor wrinkling on the inside radius. The results of MMFX yield, tensile, supplementary fatigue, and Charpy impact tests are summarized in Table 1.

Figure 1 MMFX Bend Specimen, #4 bar

Table 1 Summary of mechanical properties for MMFX								
Bar Size[1]	Testing Results (ASTM A370), US Customary Units					Charpy Impact Toughness (ASTM E23), ft-lb		
	Yield (ksi)	Ultimate (ksi)	Elongation (%)[2]	Bend Test	Fatigue Cycling[3]	10 °F	40 °F	70 °F
E4	144	162	8.3	Pass	1,465,000	---	---	---
E5	145	160	8.3	Pass	n/a	---	---	---
E6	169	177	10	Pass	n/a	---	---	---
3	130	152	7.2	Pass	n/a	---	---	---
4	130	153	7.3	Pass	n/a	---	---	---
6	150	165	8.9	Pass	n/a	14	29	45
7	136	158	8.3	Pass	n/a	---	---	---

1. E = Evaluation samples (8/2001), as-received from the manufacturer.
2. Percent elongation based on a gage length of 8 inches.
3. Tensile fatigue loading was for 30-50 ksi range, at 10 Hz, for 2,000,000 cycles or failure. However, fatigue criteria are not currently specified for reinforcement.

Note that the elongation is within ASTM A615 Grade 75 requirements (where applicable to bar size). In any case, the results indicate a ductile, tough material. The deformation patterns conform to ASTM A615, and the unit weights are also comparable. See Table 2.

Table 2 Summary of MMFX bar unit weights (US customary units) from evaluation samples				
Bar Size	Length, ft.	Weight, lb.	Unit Weight, lb/ft.	Nominal Weight, lb/ft *
4	2.79	1.86	0.667	0.668
5	2.57	2.55	0.992	1.043
6	1.10	1.63	1.482	1.502

*Note: Can be at most 6 % under nominal weight, and over weight is allowed.

Chemical Properties - The chemical analysis results of the evaluation samples (heat # 3101I4004) listed in Table 3 were obtained from the manufacturer. For illustrative purposes, the heat analysis was compared to the specified chemical composition of stainless steel type 409 (ASTM A240 Grade S40900). As can be seen, other than below the minimum chromium (Cr) content by 0.5 to 1.0 percent, the MMFX steel meets chemical requirements for ASTM A240 Grade S40900.

Table 3 Chemical analysis of MMFX rebar, test report supplied by manufacturer										
Steel	C %	Mn %	Si %	S %	P %	Cu %	Cr %	Ni %	Mo %	V %	N_2 ppm
MMFX	0.07	0.45	0.14	0.012	0.010	0.08	9.980	0.090	0.009	0.018	220
S40900	0.08	1.00	1.00	0.020	0.040	---	10.5-11.7	0.50	---	---	300

Reference: ASTM A240 *'Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications'*, Table 1, S40900. N_2 = Nitrogen content, in parts per million (ppm). Unless shown, values given are maximums for Grade S40900.

The chromium content is part of what gives stainless steel exceptional corrosion resistance, and generally is a minimum of 11 percent. The MMFX formulation, therefore, did not readily fit into existing specifications for either carbon steel or stainless steel. MMFX pursued the development of ASTM specification A1035 *"Standard Specification for Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement,"* which follows ASTM A615 criteria except for allowing higher yield and tensile strengths, and optional corrosion resistance. The MDOT special provision for MMFX steel, written in metric units for a demonstration project in 2003, required the reinforcement to meet ASTM A615 Grade 60 criteria, but no corrosion resistance criteria were specified. The special provision for MMFX steel is located in Appendix A.

Corrosion Performance - Four sets of #6 bar segments were directly exposed, and 10 concrete blocks were cast with #4 bars inside, and subject to wet/dry cycling in 3.5 percent by weight salt solution (NaCl) in water, to simulate a severe corrosion service environment. ASTM G-44 *"Standard Practice for Evaluating Stress Corrosion Cracking Resistance of Metals and Alloys by Alternate Immersion in 3.5% Sodium Chloride Solution,"* was followed for the corrosion test. The concrete mix proportions were based on MDOT Grade 45D specifications, except that the maximum coarse aggregate size was limited to 3/8 in, due to the sample size, concrete cover (1/2 in), and tank dimensions. See Figure 2 and Figure 3 for corrosion tank setup and sample configuration.

Figure 2 Corrosion tank with cover raised

Figure 3 Concrete blocks with #4 bars embedded with 1/2 in cover, before testing (above), and after 1,184 days (above right). Note the corrosion staining in specimen 1 (uncoated bar) in top row, left block of right photo.

The corrosion sample testing was started on November 19, 2001. The samples cycled for 20 minutes immersed in the 3.5 percent salt solution (NaCl), and 40 minutes exposed to ultraviolet light and drying per hour. Sample 10 had MMFX bars tied to uncoated bars with steel ties to determine effects of any accelerated galvanic corrosion. On February 15, 2005, all blocks were removed and examined.

This 1,184 day (28,416 cycles) cumulative exposure time for the onset of corrosion for the uncoated bar specimen represents approximately 9 years of real-world exposure, based on estimated time-to-cracking of uncoated bars in concrete *[7]*. All bar types were removed from the concrete and examined. Corrosion staining and minor section loss were observed on all uncoated bar specimens, and isolated corrosion stains were found on some MMFX samples. See Figure 4. The epoxy coated reinforcement was intact, as was the solid stainless 304 reinforcement. The results of this corrosion study were inconclusive except that the MMFX, epoxy coated, and stainless steel reinforcement were more corrosion resistant than uncoated bar; and the mill scale present on MMFX bars may be detrimental to corrosion performance, but to an unknown degree. For the electrically coupled sample, galvanic corrosion was not observed on the MMFX or the uncoated bar.

Figure 4 Uncoated reinforcement removed from concrete blocks. Note the general corrosion on all samples.

Along with the concrete blocks, eight MMFX #6 size reinforcement samples were placed in the corrosion tank for direct exposure to the 3.5 percent salt solution and drying cycle to evaluate pitting corrosion potential. The samples were subjected to 14,660 wet/dry cycles over 611 days. Although extensive pitting corrosion occurred along the surface, the maximum pit depth was 0.070 inches as measured with a depth micrometer, or 9.4% of the nominal cross sectional diameter, after nearly two years of this severe environment exposure. See Figure 5. It is estimated that uncoated bar directly exposed to this environment would have had nearly complete loss of cross section.

Figure 5 MMFX reinforcement directly exposed to salt solution

In 2000, the MMFX Corporation applied for a HITEC (Highway Innovative Technology Evaluation Center) evaluation to assess the corrosion resistance of MMFX steel rebar in reinforced concrete applications. By 2002, HITEC had proposed an evaluation plan for corrosion resistance of MMFX by standardization of an accelerated laboratory scale test procedure, to be carried out in a round-robin study. Four state departments of transportation (DOT), namely CALTRANS, Penn DOT, Texas DOT and Virginia DOT participated in the round-robin study, with corrosion testing of MMFX reinforcement, along with other reinforcing steel materials using the accelerated chloride threshold (ACT) test method. The test quantitatively measures the corrosion resistance of different steel reinforcement materials by determining the critical chloride concentration threshold necessary to initiate reinforcement corrosion.

Chlorides are migrated into the mortar by applying a potential difference between a wire mesh embedded at the reinforcement, and the solution reservoir at the surface. The corrosion rate of the reinforcement is concurrently monitored. The critical chloride concentration is determined by analyzing mortar sampled directly above the steel when the corrosion rate reaches 10 $\mu A/cm^2$. Unfortunately, the results of the round-robin study were never published. The ACT procedure is not being investigated further at this time, due to suspension of funding.

INNOVATIVE BRIDGE RESEARCH AND CONSTRUCTION (IBRC) DEMONSTRATION PROJECT

An IBRC demonstration project, sponsored by the Federal Highway Administration (FHWA), was completed in 2003. The project utilized MMFX reinforcement for the deck. This 4-span prestressed box beam structure, R01 of 82022, I-94 eastbound over Shook Road and CSX Railroad, is 285 ft length, 78 ft - 7 in width, has a 6 in wearing surface, and carries 4 lanes of traffic with an average daily traffic (ADT) of 122,600 and 13% commercial (trucks). A total of $137,000 in Federal funds was obligated for the innovative material (MMFX) portion of this project. The I-94 westbound structure was constructed with conventional ECR. See Table 4 for a cost summary.

Table 4 Summary of demonstration project costs for MMFX reinforcement

Reinforcement Material	Quantity, lb	Engineer's Estimate, $/lb	Contractor's Price, $/lb	Total Cost, $	Percent of Deck Cost ($2,444,678)
MMFX	116,755	$1.02	$1.18	$145,898.49†	6.0
Epoxy-Coated Reinforcement (ECR)	168,473	$0.77	$0.75	$126,068	5.2

† Includes contract extras of $8,228.49 (materials and labor) due to supply limitations on available reinforcement length.

The Engineer's cost estimate of $1.02/lb for MMFX reinforcement was based on taking the manufacturer's quoted price of $800 per ton ($0.40/lb), adding the estimated delivered and installed cost of $0.50/lb (based on discussions with other suppliers). For comparison purposes, an equivalent amount of epoxy-coated reinforcement substituted for the MMFX would have cost $92,306.60. The additional $53,591.89 paid for the MMFX reinforcement amounted to 0.8 percent of the deck cost and only 0.5 percent of the structure cost.

The MMFX bar tested for the demonstration project exceeded ASTM A615 Grade 75 mechanical properties (see Table 1), and it is anticipated that the corrosion properties will be superior to ECR. However, the bars delivered to the site had mill scale on the surface that differed in appearance from the evaluation specimens. At this time it is unclear as to how the mill scale will affect the long-term performance of the MMFX bar. The manufacturer stated that the additional cost to remove the mill scale would be close to $0.30/lb, making it cost prohibitive. See Figure 6.

Figure 6 Mill scale on MMFX reinforcement, R01 of 82022

IBRC project supply issues, R01 of 82022 - The bridge deck slab was designed using ASTM A615 Grade 60 reinforcement. The reason for the selection of Grade 60 for design purposes was due to the uncertainty in funding for the innovative material, and concerns of its availability. Had the application for IBRC funds been denied, or if the MMFX was not available, substitution using ECR would have been straightforward.

In April 2002, during the design phase of the IBRC project, the manufacturer's General Manager indicated that with enough lead time, they can supply lengths outside of the typical 40 and 60 ft. sections. MDOT Bridge Design Manual (BDM) specified a maximum length of 40 ft for #3 bars, and recommends that other size bar lengths be kept under 50 ft (*[12], Section 7.04.03*). The #4 reinforcement had a length of 42 ft – 8 in specified on the plans. All reinforcement lengths were within recommended guidelines, and were assumed to be available from discussions with the manufacturer.

In June 2003, however, as work proceeded on Stage I, a claim was made by the contractor against MDOT for extra work required for additional lapping (the fabricator had ordered reinforcement bar lengths in 40 ft sections) and material. Upon investigation, it was found that the manufacturer's Midwest Regional Sales Manager advised the Contractor in a letter dated June 20, 2003, that special lengths were available at additional cost. However, another letter to the contractor dated October 9, 2003, stated that only 40 and 60 ft lengths were available, and *"...the 42 foot length requirements for the recent Michigan DOT bridge project on I-94 would have either required the fabricator to order 60 foot length material (and have significant waste) or, (sic) to splice material on the deck [13]."*

Due to the manufacturer's change in policy, claim extras of $8,228.49 were awarded to the contractor for the additional cost of lapping and materials. Stage I was completed in July 2003, and Stage II completed by September 2003. See Figure 7. The latest inspection in November 2006 shows the MMFX and ECR reinforced decks to be in good condition.

Figure 7 Completed Stage I, R01 of 82022, August 2003.

DESIGN ISSUES

Because MMFX does not exhibit a defined yield point, the yield strength (f_y) of 130 ksi is determined by the 0.2 percent offset method (ASTM A370). The higher MMFX yield strength may reduce the steel reinforcement requirements, but f_y is limited to 60 ksi (17^{th} edition, Subsection 8.3.3), and 75 ksi (LRFD Subsection 5.4.3.1) for design purposes *[14, 15]*. The MDOT current practice of reinforced concrete design utilizes AASHTO 17^{th} edition Standard Specifications for Bridge Design, load factor method. Direct substitution of MMFX reinforcement for ASTM A615 Grade 60 reinforcement therefore presents some issues.

The MMFX yield strength of 130 ksi has implications for use in a reinforced concrete bridge deck, as the failure modes in flexure and shear may be affected when limiting the design reinforcement yield strength to 60 or 75 ksi. Current design provisions (AASHTO 17^{th} edition) require the steel reinforcement ratio to be at most 75 percent of the reinforcement ratio that would produce a balanced strain condition in the reinforced concrete section. Balanced strain conditions are when the tension reinforcement reaches a strain corresponding to its yield strength, at the same time as the concrete in compression reaches its ultimate strain of 0.003 (AASHTO 8.16.3.1.2). A steel reinforcement ratio of 0.75 leads to yielding of the steel prior to compressive failure of the concrete, thus exhibiting ductile failure. Concrete sections with reinforcement yield strength greater than the 60 ksi yield strength for design could exhibit a brittle failure governed by crushing of the concrete prior to yielding of the reinforcement. Therefore a closer investigation of the ductility of sections reinforced with MMFX steel should be conducted for each design case.

The steel to concrete bond is a function of reinforcement geometry, and thus not an issue when substituting A615 Grade 60 reinforcement with MMFX. A report published by Michigan Technological University showed that *"...a one-to-one replacement of MMFX [-2] reinforcing bar for A615 Gr. 60 bar would be suitable from a bond perspective of # 4 and # 6 reinforcing bars in normal strength concrete typical of that used in a Michigan bridge deck [16]."*

Current practice in Michigan for lapped tension splices is based on AASHTO 17^{th} edition, and tables are published in the MDOT Bridge Design Guides (BDG). For reference, the minimum lap length in 4 ksi concrete is 35 in for # 6 uncoated tensile reinforcement at 8 in spacing (BDG §7.14.01A). Lap lengths up to 30.5 in were tested in a Florida DOT study *[4]* with #6 bar at 8 in spacing embedded in 5 ksi concrete beams and found insufficient for yielding of MMFX reinforcement. Because of the higher yield strength of MMFX, and lack of a distinct yield point, lapping should be further investigated and mechanical reinforcement splices used. Tensile tests of #4, 8, and 9 bars coupled with BarSplice mechanical threaded splices showed acceptable performance of the mechanical splices by reinforcement failure outside the splice region *[17]*.

ECONOMIC ANALYSIS

Reinforced concrete bridge decks using MMFX reinforcement would necessarily have to be designed with the AASHTO LRFD code, to utilize the higher permitted reinforcement yield strength. A model bridge deck was designed using LRFD to determine appropriate quantities and spacing for MMFX reinforcement, and compared to MDOT standard design (LFD) for the ECR deck. As can be seen in Table 5, the LRFD design resulted in reduced steel weight, although the material cost is greater by $\$1.05/ft^2$. The calculations are included in Appendix B.

The model bridge characteristics were as follows: Single span, simply supported, concrete deck on steel girders, 75 ft length, 40 ft width, 8 ft beam spacing, and 9 in deck composite with 1.5 in future wearing surface.

Table 5 Comparison of LFD designed ECR concrete deck to LRFD designed MMFX reinforced concrete deck

Reinforcement Bar Size and Spacing	LRFD (MMFX)	LFD (ECR)
Longitudinal (top mat)	# 3 at 12 in	# 3 at 10 in
Transverse (top mat)	# 5 at 7 in	# 6 at 9 in
Longitudinal (bottom mat)	# 5 at 12 in	# 5 at 10 in
Transverse (bottom mat)	# 5 at 8 in	# 6 at 9 in
Total weight	13,992 lb	17,149 lb
Total reinforcement cost	$16,510.56	$13,547.71
Reinforcement cost per SFT (3,000 SFT total)	$5.50	$4.52

<u>Life Cycle Cost Analysis</u> - Life cycle cost analysis (LCCA) is a tool used to compare the total user and agency costs of competing project implementation alternatives that would yield the same level of service and benefits. The LCCA is used when comparing alternatives to replace a bridge that has reached the end of its service life, where each design alternative will result in the same level of service to the user. Costs measured in LCCA typically include expenses to MDOT, such as construction, operation, and maintenance costs. The LCCA is performed during the bridge scoping process and the annual call for projects, where alternative rehabilitation strategies are compared. The bridge deck preservation matrix (*[12], Appendix 12.09.02*) gives recommended repair methods for various deck conditions. The repair strategies are based on the National Bridge Inventory ratings provided from bridge inspection and scoping documents. For bridge project scoping, several repair and replacement scenarios are proposed considering the initial time to repair and time to subsequent repairs over a defined analysis period to give present value costs. The bridge deck preservation matrix is based on rehabilitation of uncoated steel reinforced structures.

The 90 year timeframe is the standard analysis period for MDOT LCCA when considering repair options during the bridge scoping phase. The MDOT LCCA scenario used for bridge scoping is summarized as:

Year 1 – new construction
Year 40 – deep overlay (below top mat steel, approximately 4 in)
Year 65 – shallow overlay (approximately 1.5 in)
Year 80 – bituminous (HMA) overlay (for ride quality)
Year 90 – new deck

The LCCA scenario used in the SDDOT report is based on a 75 year timeframe, and the typical rehabilitation after new construction is the placement of a low slump dense concrete overlay. The repair life assumption of 25 years in the SDDOT report (*[8], pg. 32*) is a reasonable estimate for the placement of a low slump dense concrete overlay, comparable to the MDOT Bridge Design Manual (BDM) estimate of a 25 year repair life for a deep overlay (*[12], Appendix 12.09.02*).

The SDDOT LCCA scenario is also based on rehabilitation of uncoated steel reinforced structures and summarized as follows:

Year 1 – new construction
Year 40 – low slump dense concrete overlay
Year 65 – low slump dense concrete overlay
Year 75 – new deck

The SDDOT report used their typical costs of bridge construction to model their LCCA example. The model bridge was 150 ft length, 36 ft width, 8.5 in deck, with steel density of 210 lb/yd^3, and deck surface area of 600 yd^2. No information was given as to the number of spans, or girder type. The analysis focused on deck costs, so user costs were not considered.

For comparison purposes to the SDDOT report, their LCCA scenario was used except for the longer 90 year analysis period. The life expectancy of bridge decks constructed with ECR has been estimated at 27 to 40 years. Some estimates are based on observations, while others consider corrosion rates and chloride threshold levels. The SDDOT report authors chose 40 years as the life expectancy of ECR bridge decks based on observed field performance of bridges no older than 25 years.

Listed in Table 6 are selected pay item quantities and cost per square yard of bridge deck for the LCCA case study. Note that most items are not specifically related to the bridge deck but part of the construction process. For a comparison, the MDOT bridge repair cost estimate for similar pay items is included (*[12], Appendix 2.02.19.A.4*), with the most recent unit pricing. There is an error in the SDDOT cost estimate for bridge rail modification. The report calculated the cost per yd^2 based on the 36 ft width, instead of the 150 ft length. The corrected cost is stated in Table 6. Also, mobilization and traffic control costs that were considered in the SDDOT report were not included in this report because of the highly variable pricing from project to project.

Table 6 Pay items selected for bridge LCCA example, SDDOT and MDOT							
Bridge Deck Repair Pay Item (SDDOT)	Unit	Cost	Cost/yd^2	Pay Item (MDOT)	Unit	Cost	Cost/yd^2
Low slump dense concrete overlay	SYD	$80	$80	Deep overlay (includes joint replacement and hydro)	SFT	$18	$162
Bridge Rail Modification	LFT	$45.25	$23	Bridge Railing, remove and replace	LFT	$135	$68
Approach guardrail	LS	$16,000	$27	Guardrail, type B or T	LFT	$15	$8
Approach pavement work	LS	$16,500	$28	Approach pavement, 40 ft each end	SFT	$9	$81
Total Repair Costs per yd^2			$158	Total Repair Costs per yd^2			$319
New Bridge Deck Pay Item (SDDOT)	Unit	Cost	Cost/yd^2	New Bridge Deck Pay Item (MDOT)	Unit	Cost	Cost/yd^2
Concrete	CYD	$350	$83	Superstructure Concrete	CYD	$199	$47
Epoxy coated steel	LB	$0.60	$30	Reinforcement, steel, epoxy coated	LB	$0.94	$47
MMFX steel	LB	$0.84	$42	MMFX reinforcement	LB	$1.18	$59
Total New Construction Cost per yd^2 ECR			$112	Total New Construction Cost per yd^2 ECR			$94
Total New Construction Cost per yd^2 MMFX			$124	Total New Construction Cost per yd^2 MMFX			$106

If the appropriate initial time-to-repair values for each material are used, with a real discount rate of 3.0 percent *[18]*, the LCCA over a 90 year period gives present value costs of \$207/yd^2 for MMFX, and \$238/yd^2 for epoxy-coated reinforcement. See Table 7 for a summary. Note that at year 90, the ECR deck would be due for another repair, but was not included because costs are incurred at the beginning of each year. The MMFX deck would have an additional 13 years beyond the analysis period before repair.

The present value of construction and repair costs were calculated using Equation (1), using information from the SDDOT report:

$$P = F \times (1 + i)^{-n}, \qquad \text{Equation (1)}$$

where:
P = present worth, F = cost of construction and repair, i = discount rate, and n = time to repair or replacement (years).

Table 7 Life cycle costs for 90-year period using time to initial repair of 52 years for MMFX based on corrosion rate data, and 25 years for subsequent repairs, as indicated in the SDDOT report.

Reinforcement	New Cost (\$/yd^2)		Repair Cost (\$/yd^2)		Time to Initial Repair (years)	Time to Next Repair (years)	Time to Next Repair (years)	Present Value of Costs (\$/yd^2) with 3.0 % Discount Rate	
	SDDOT	MDOT	SDDOT	MDOT				SDDOT	MDOT
Epoxy-coated	112	94	158	319	40	65	90	184	238
MMFX	124	106			52	77	103	174	207

CONCLUSIONS

From life cycle cost analysis, with MMFX reinforcement providing an estimated 12 years service life over epoxy-coated reinforcement, an increase in the structure cost of $12/ yd^2 for the MMFX reinforcement is worth the investment. Bridge decks incorporating MMFX, however, will have to be designed using AASHTO LRFD code to accommodate the higher yield strength, and it is recommended that mechanical reinforcement splices are specified for lap locations. It is recommended that the usage criteria for MMFX reinforcement follow that established for stainless steel reinforcement and in highly congested urban areas when life cycle costs are justified.

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Appendix A – MDOT Special Provision for Microcomposite Steel Reinforcement

MICHIGAN
DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION
FOR
REINFORCEMENT, MICROCOMPOSITE STEEL

a. Description. This work shall consist of furnishing and installing reinforcing steel in structural concrete. This work shall be done in accordance with Section 706 of the Standard Specifications for Construction, except as modified herein.

b. Materials. This special provision covers deformed microcomposite steel bars for use in reinforcement of concrete exposed to conditions requiring resistance to corrosion. The bars (designated MMFX-2) are to be supplied by MMFX Steel Corporation, 8000 Corporate Center Dr., Suite 207, Charlotte, NC, 28226. Contact Person is Tim Knaus, Tel. (704)-752-9155. All materials required shall be as specified in Section 706 of the Standard Specifications for Construction, except for the following modifications:

1. The delivery time for this material is approximately 60 calendar days from date of order.

2. Unless otherwise stated, all aspects of the bars shall conform to ASTM A 615. Grade 60 deformed reinforcement bars shall be specified. The standard sizes and dimensions of deformed bars, their numerical designation, and the spacing, height and gap of deformations shall be equivalent to those listed in Table 1 of ASTM A 615.

3. Physical Properties - The MMFX-2 microcomposite steel reinforcement bars shall have the following physical properties:

Minimum Yield Strength:	60 ksi
Minimum Tensile Strength:	90 ksi
Minimum Elongation in 8 in:	9 percent

4. Acceptance testing shall be performed on a per project basis. Sample size shall be 2 bars per size, per manufacturer; 1 bar of 24 inch minimum length, and 1 bar of 36 inch minimum length. Samples shall be submitted to MDOT for testing. Testing shall be in accordance with ASTM A 370 and the ASTM specifications applicable to the material referred to herein.

5. Mill Certificate - The certificates shall be provided per project, with one copy submitted with acceptance samples, and:

   a. Be from the supplying mill verifying that the MMFX-2 microcomposite steel reinforcement provided has been sampled and tested and the test results meet the contract requirements;
   b. Include a copy of the chemical analysis of the steel provided, with the heat lot identification, rolling condition, and the source of the metals if obtained as ingots from another mill;
   c. Include a copy of tensile strength, yield strength, bend tests, and elongation tests on

each of the sizes (diameter) of MMFX-2 microcomposite steel reinforcement provided;

d. Permit positive determination that the reinforcing provided is that which the test results cover.

7. Bar Chairs and Wire Ties - The bar chairs and wire ties required for shipping, placing and fastening the microcomposite steel reinforcement shall meet the following:

Bar chairs shall be plastic coated, epoxy coated, or plastic. Legs of chairs shall be turned up a minimum of 1/8 inch.

Wire ties shall be plastic coated, plastic, or stainless steel conforming to the requirements of ASTM A493, Type 316 (UNS number S31600), annealed. Wire size shall be the same as used for steel reinforcement.

Tie-down wires shall be plastic coated, epoxy coated, or stainless steel conforming to the requirements of ASTM A493, Type 316 (UNS number S31600), annealed.

8. Marking - When loaded for mill shipment, bars shall be properly separated and tagged with the manufacturer's two batch numbers referring respectively to the cladding and the core or test identification numbers. The producer(s) shall identify the symbols of marking system(s) used.

c. Construction Methods. Construction of microcomposite steel reinforced concrete shall conform to Section 706 of the Standard Specifications for Construction except for the following modifications:

Notification - The Engineer shall notify Construction & Technology Division, Structural Research Unit, phone number (517) 322-5655, two weeks prior to placement of microcomposite steel reinforcement.

Splices - Splices shall generally be of the lap type. Mechanical splices may be used in certain situations, subject to the approval of the Engineer.

Approval - After placing reinforcement in any member have it inspected and approved by the Engineer before placing concrete. Concrete placed in violation of this provision may be rejected and removal required.

d. Measurement and Payment. The completed work as measured will be paid for at the contract unit price for the following pay item:

Contract Item (Pay Item)	**Pay Unit**
Reinforcement, Microcomposite Steel	Pound

The item of **Reinforcement, Microcomposite Steel** will be paid for by the pound and consist of furnishing and installing all component materials as specified in the Special Provision and on the plans.

Appendix B – MMFX Reinforced Bridge Deck LRFD Design Example

MMFX Reinforced Bridge Deck LRFD Design Example

Code Reference: AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

Design Criteria:

Deck thickness	9 in
Wearing surface	1.5 in
Beam spacing	5 @ 8 ft
Span	75 ft

Bridge barrier railing is not included in analysis. Figure B 1 shows a cross section of the design bridge, not to scale.

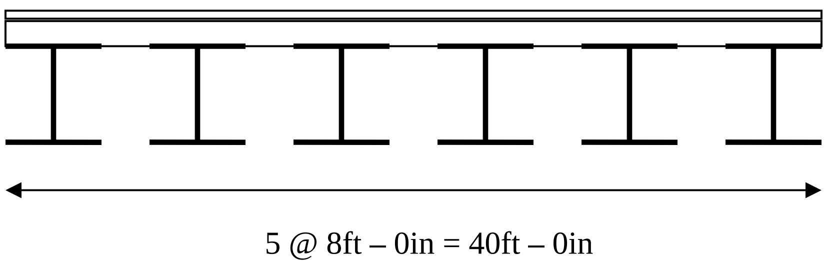

Figure B 1. Design bridge cross section. Not to scale.

f'_c	4 ksi	
E_c	$1820 * \sqrt{f'c}$ (ksi) = 3,640 ksi	(C5.4.2.4-1)
E_s	29,000 ksi	
β_1	0.85	
f_y	75 ksi for design	

Number of design lanes = 40 ft / (12 ft/lane) = 3.33, use 3.0

A. Dead Loads (DC, DW)

9 in concrete slab (density 0.150 kcf):	0.150 kcf * 9 in / (12 in/ft)	= 0.1125 ksf
Stay in place forms:		= 0.0150 ksf
Future wearing surface (2 in):	0.150 kcf * 2 in / (12 in/ft)	= 0.0250 ksf

A. Dead Load Moments (M_{DC}, M_{DW}) from Beam Analysis Program (BAP)

M_{DC}	= + 0.636 k-ft / ft	M_{DW}	= + 0.125 k-ft / ft
	= - 0.859 k-ft / ft		= - 0.168 k-ft / ft

B. Live load moments

For the strip design method, when strips are transverse and the span is less than 15 ft, use 32 kip axle of design truck (3.6.1.3.3). From the strip method, width equals (Table 4.6.2.1.3-1):

+M width (in) = 26.0 + 6.6S = 26.0 + 6.6 * 8 = 78.8 in (6.6 ft)
-M width (in) = 48.0 + 3.0S = 48.0 + 3.0 * 8 = 72.0 in (6.0 ft)

There are three design lanes; therefore up to three trucks can be placed on the deck. MDOT Bridge Design Manual (BDM) Subsection 7.02.19 specifies HS20-44 truck loading for slab design. Although the HL-93 truck loading is used in LRFD, MDOT current design truck loading will be used in place. Moments are calculated using BAP, and adjusted with multiple presence factors (m) for 1 to 3 lanes (Table 3.6.1.1.2-1). Results are summarized in Table B 1 with maximum moments in boldface.

One Lane (1 truck), m = 1.2

$+M_{LL}$ = 1.2 * 26192 / (6.6 * 1000) = 4.76 k-ft / ft
$-M_{LL}$ = 1.2 * -23214 / (6.0 * 1000) = -4.64 k-ft / ft

Table B 1. Design Moments for LRFD deck slab

Design Moments (k-ft / ft)	1 lane m = 1.2	2 lanes m = 1.0	3 lanes m = 0.85
+ M_{LL}	**4.76**	4.02	3.52
- M_{LL}	-4.64	**-4.80**	-4.08

C.1. Strength Limit State (3.4.1)

$M_u = \eta\ [Y_p * M_{DC} + Y_p * M_{DW} + 1.75 * (IM + 1)*M_{LL}]$

$\eta = \eta d* \eta r* \eta i \geq 0.95 = 1.0*1.0*1.0 = 1.0$ (1.3.2)

Y_p = 1.25 for DC (Table 3.4.1-2)
= 1.50 for DW

IM = 33 % (Table 3.6.2.1-1)

$+M_u$ = 12.06 k-ft / ft = ΦM_n (+)
$-M_u$ = -12.50 k-ft / ft = ΦM_n (-)

$\Phi = 0.9$

C.2. Service Limit State III (3.4.1)

$+M_r = 1.0\ [1.0 * (M_{DC} + M_{DW}) + 0.80 * (IM + 1)*M_{LL}] = 0.636 + 0.125 + 0.80*1.33*4.76$
$+M_r$ = 5.83 k-ft/ft
$-M_r$ = -6.13 k-ft/ft

D. Primary steel reinforcement requirements

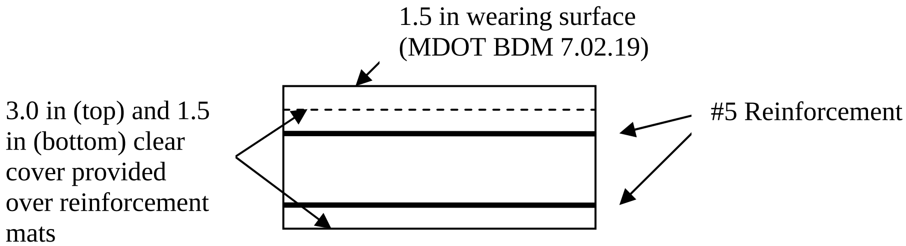

Figure B 2. Design deck slab cross-section, total thickness 9 in, including wearing surface.

Figure B 2 shows a cross section of the design deck slab. Design is for 75 ksi yield strength steel reinforcement as allowed by code (5.4.3.1).

D. 1. Positive moment steel (A_s) required (5.7.3.3.2):

d_s = 9 in – 1.5 in – (0.625 in/2) = 7.19 in
a = A_s*fy / (0.85*f'c*b)
b = 12 in

$$M_n = A_s f_y \left(d_s - \frac{a}{2} \right) = f_y A_s d_s - \left(\frac{f_y^2}{1.7 f_c' b} \right) A_s^2 \qquad 5.7.3.2.2\text{-}1$$

$$M_n = \left(12.06 k \cdot ft \times 12 \frac{in}{ft} \right) \times \frac{1}{0.9} = 75ksi \times A_s \times 7.19in - \left(\frac{75ksi^2}{1.7 \times 4ksi \times 12in} \right) \times A_s^2 = 539.25 A_s - 68.93 A_s^2$$

By numerical solution, A_s = 0.31 in^2/ft, use #5 reinforcing bars spaced at 12 in on centers (A_s = 0.31 in^2/ft). This area of steel provides nominal moment capacity (M_n) of 13.40 kip-ft / ft.

Check minimum reinforcement – according to 5.7.3.3.2, unless otherwise specified, at any section of a flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:

1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, f_r, on the concrete as specified in 5.4.2.6.

OR

1.33 times the factored moment required by the applicable strength load combinations specified in Table 3.4.1-1.

1.2 M_c = 1.2 *S_c* f_r
S_c = (12 in * (9 in)^2)/6 = 162 in^3

$f_r = 0.24\sqrt{f'c} = 0.24\sqrt{4}$ ksi = 0.48 ksi (5.4.2.6)
1.2 M_r = 1.2 *162 in^3*0.48 ksi = 93.3 k-in = **7.78 k-ft** GOVERNS - OK
1.33 M_u = 1.33 * 12.06 k-ft = 16.04 k-ft

Check maximum positive moment reinforcement (C5.7.2.1)
$c \leq 0.6d_s$
$a = \beta_1 c = 0.85c$ = 0.85 *0.6*5.69 in = 2.90 in
A_s (max) = [0.85*f'c*a*b] / f_y = [0.85*4 ksi*2.90 in*12 in]/75 ksi = 1.58 in^2 OK

Check spacing, S = 12 in $\leq$ S (max)
S (max) = lesser of (1.5 * 9 in = **13.5 in**, governs) or 18 in (5.10.3.2). OK

D. 2. Negative Moment Reinforcement:

d_s = 9 in – 3 in – (0.625 in/2) = 5.69 in
a = A_s*fy / (0.85*f'c*b)
b = 12 in

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) = f_y A_s d_s - \left(\frac{f_y^2}{1.7 f_c' b}\right) A_s^2 \qquad 5.7.3.2.2\text{-}1$$

$$M_n = \left(12.50k \cdot ft \times 12\frac{in}{ft}\right) \times \frac{1}{0.9} = 75ksi \times A_s \times 5.69in - \left(\frac{75ksi^2}{1.7 \times 4ksi \times 12in}\right) \times A_s^2 = 426.75A_s - 68.93A_s^2$$

By numerical solution, A_s = 0.42 in^2/ft, use #5 reinforcing bars spaced at 8 in on centers (A_s = 0.47 in^2/ft). This area of steel provides nominal moment capacity (M_n) of 13.90 kip-ft / ft.

Check for minimum reinforcement:
1.2 M_c = 1.2 *S_c* f_r
S_c = (12 in * (9 in) ^{2})/6 = 162 in^3
$f_r = 0.24\sqrt{f'c} = 0.24\sqrt{4}$ ksi = 0.48 ksi (5.4.2.6)
1.2 M_r = 1.2 *162 in^3*0.48 ksi = 93.3 k-in = **7.78 k-ft** GOVERNS - OK
1.33 M_u = 1.33 * 12.50 k-ft = 16.63 k-ft

Check maximum negative moment reinforcement (C5.7.2.1)
$c \leq 0.6d_s$
$a = \beta_1 c = 0.85c$ = 0.85 *0.6*5.69 in = 2.90 in
A_s (max) = [0.85*f'c*a*b] / f_y = [0.85*4 ksi*2.90 in*12 in]/75 ksi = 1.58 in^2 OK

8 in spacing (S) $\leq$ max spacing of 13.5 in OK
Nominal moment capacity (M_n) = 13.90 kip-ft / ft $\geq$ M_u of 12.50 kip-ft / ft OK

D. 5. Distribution Reinforcement (9.7.3.2):
For primary reinforcement perpendicular to traffic:

Percent distribution reinforcement = 220 / $\sqrt{S_e}$ < 67%
S_e = 8 ft (full span length, due to assumed girder spacing)
Percent distribution reinforcement = 220 / $\sqrt{8}$ = 77.8%, use 67%

A_s (distribution reinforcement) = 0.67*As = 0.67*0.41 = 0.27 in^2/ft
Use #5 bars at 12 in spacing on center, bottom mat.
A_s (distribution reinforcement) = 0.31 in^2/ft OK

D. 6. Shrinkage and Temperature Reinforcement (5.10.8.2):

$$A_s = \frac{1.30bh}{2(b+h)f_y} = \frac{1.30 \times 12in \times 9in}{2 \times (12in + 9in) \times 75ksi} = 0.045\frac{in^2}{ft}$$, with 0.11 ≤ A_s ≤ 0.60

Therefore set A_s equal to 0.11 in^2/ft, distributed equally between both faces of the deck slab.
Due to distribution reinforcement of 0.31 in^2/ft on bottom slab, place shrinkage and temperature reinforcement on top slab only. Use #3 bars at 12 in spacing on center, top mat (A_s = 0.11 in^2/ft).

A_s = 0.11 in^2/ft OK

E. Crack Control (5.7.3.4):

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$, where (5.7.3.4-1)

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

γ_e = exposure factor (1.00)
d_c = actual concrete cover thickness, in
f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)
h = overall thickness (9 in)

E. 1. Positive Moment Region:
d_c = 1.5 in + (0.625 in)/2 = 1.81 in

$$\beta_s = 1 + \frac{1.81in}{0.7(9in - 1.81in)} = 1.36$$

$$f_{ss} = \frac{M_s}{A_s\, jd_s} \times \frac{12\ in}{ft}$$

M_s = 5.83 k-ft/ft

$$j = 1 - \frac{k}{3}; \quad k = \sqrt{2\rho n + (\rho n)^2} - \rho n; \quad \rho = \frac{A_s}{bd_s}; \quad n = \frac{E_s}{E_c}$$

E_c = 3640 ksi
E_s = 29000 ksi
n = 29000/3640 = 7.97
ρ = (0.31 in^2/ft)/(12 in * 7.19 in) = 0.0036

$$k = \sqrt{2 \times 0.0036 \times 8 + (0.0036 \times 8)^2} - 0.0036 \times 8 = 0.213$$

j = 1 – 0.213/3 = 0.929

$$f_{ss} = \frac{5.83k \cdot ft / ft}{0.31in^2 / ft \times 0.929 \times 7.19in} \times \frac{12in}{ft} = 33.8ksi$$

$$s = \frac{700 \times 1.00}{1.36 \times 33.8} - 2 \times 1.81in = 11.6in > 8 \text{ in}$$ OK

<u>E. 2. Negative Moment Region:</u>
d_s = 5.69 in
d_c = 3.0 in + (0.625 in)/2 = 3.31 in
β_s = 1.83
M_s = -6.13 k-ft/ft
ρ = 0.47 in^2/ft / (12 in * 5.69 in) = 0.0069
k = 0.277
j = 1 – 0.277/3 = 0.908
f_{ss} = 30.3 ksi
s = 6.0 in < 8 in NG
<u>Reduce spacing of #5 bars to 7.0 in on centers, A_s = 0.53 in^2/ft.</u>
ρ = 0.53 in^2/ft / (12 in * 5.69 in) = 0.0078
k = 0.296
j = 1 – 0.296/3 = 0.901
f_{ss} = 27.1 ksi
s = 7.5 in OK

F. Summary of LRFD Deck Slab Design using MMFX Reinforcement

See Figure B 3. Beams are not shown for clarity.

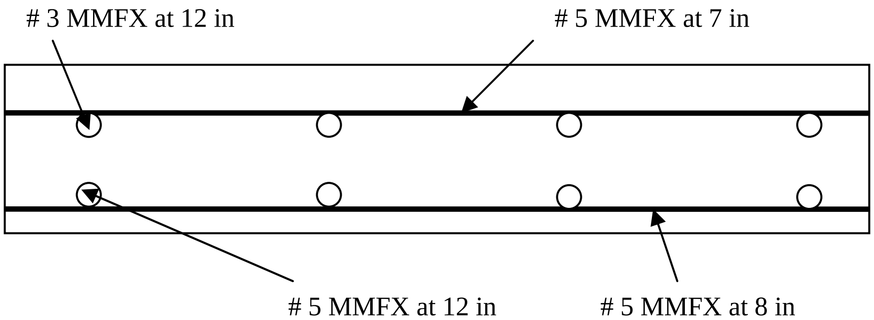

Figure B 3. Reinforcement Layout (Not To Scale)

G. MDOT Standard Deck Slab Design (MDOT Bridge Design Guide Table 6.41.01):

See Figure B 4. Beams are not shown for clarity.

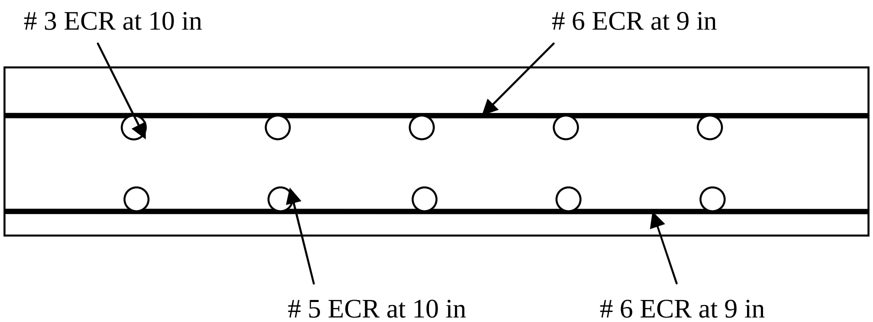

Figure B 4. Standard MDOT Bridge Deck Slab (Not To Scale)

H. Economic Analysis and Quantity Summary
Material cost for MMFX reinforcement, including placement …………………$1.18/lb
Material cost for epoxy coated reinforcement, including placement ………..…..$0.94/lb

Unit Weights of Reinforcement (lb/ft) – ASTM A615, Table 1:
#3 0.376
#5 1.043
#6 1.502
Primary Reinforcement (PS), lb
= (unit weight, lb/ft)*(40 ft)*[75 ft / (spacing, in / 12 in/ft)]

Distribution, temperature, and shrinkage reinforcement (DS), lb
= (unit weight, lb/ft)*(75 ft)*[40 ft / (spacing, in / 12 in/ft)]

The LRFD designed bridge deck has reduced steel requirements, due to the design allowable maximum of 75 ksi yield strength, as compared to LFD design allowable maximum of 60 ksi yield strength. As can be seen in Table B 2, although the total quantity of MMFX reinforcement is less than Grade 60 steel by 3,157 lb, or 18.4 percent, the total cost is higher by $2,962.85, or 21.9 percent.

Table B 2 Economic analysis of bridge deck design scenarios

Location	PS Size	PS Spacing, in	DS Size	DS Spacing, in	Quantity, lb	Cost, $
MMFX Reinforced Bridge Deck (LRFD Design)						
Top Mat	#5	7	#3	12	6,492	$ 7,660.56
Bottom Mat	#5	8	#5	12	7,500	$ 8,850.00
				Totals	13,992	$ 16,510.56
Epoxy Coated Reinforced Bridge Deck (ASD Design)						
Top Mat	#6	9	#3	10	7,374	$ 5,825.46
Bottom Mat	#6	9	#5	10	9,775	$ 7,722.25
				Totals	17,149	$ 13,547.71