ABSTRACT
This paper describes the behavior of typical bridge deck slabs reinforced with a high-strength, highly corrosion-resistant steel commercially known as Micro-composite Multi-structural Formable (MMFX) steel. The study included testing three full-scale bridge decks each having a span-to-depth ratio of 12.5. Two decks were reinforced with MMFX steel while another was reinforced with conventional Grade 60 steel for comparison purposes. The bridge decks were tested under static loading up to failure using concentrated loads intended to simulate truck wheel loads. Load-deflection behavior, mode of failure, crack patterns, and strain distribution are reported. The use of MMFX steel as main flexural reinforcement in bridge decks was evaluated in light of the test results. A non-linear finite element analysis was developed to predict the behavior. This prediction was calibrated using measured values, and was employed to study various parameters related to the use of MMFX steel in bridge decks. The paper also presents the effects bending MMFX steel bars on their tensile strength.

INTRODUCTION
Corrosion of steel reinforcement has been identified as one of the leading causes of deterioration in concrete bridges. This fact has led to the development of numerous technologies, such as corrosion-resistant steel, that attempt to mitigate this expensive problem. The recent development of a high-strength, highly corrosion-resistant steel commercially known as Micro-composite Multi-Structural Formable (MMFX) steel is a promising technology. MMFX steel claims high corrosion resistance without the use of the coating technologies. This characteristic was achieved by proprietary alteration of the steel composition and microstructure. In addition, the control of MMFX steel’s morphology of its microstructure has resulted in its higher strength. Use of the new steel could lead to potential savings through lowered reinforcement ratios, due to its higher strength, and to a longer service life of structures, due to its greater corrosion resistance. Recently, many state transportation departments have begun to use MMFX steel as a direct replacement for black steel in concrete bridge decks. However, there is insufficient information about the behavior of such concrete bridge decks utilizing MMFX steel as main flexural reinforcement. In addition, for the new steel to be accepted as transverse reinforcement, the effects of bending on tensile strength must be evaluated.

The experimental program consisted of two phases. The first phase consisted of testing four bent bar specimens in order to evaluate the effects of bending on tensile strength of MMFX bars. The second phase consisted of testing three full-scale bridge decks with a span-to-depth ratio of 12.5 in order to
evaluate the structural performance of MMFX steel as main flexural reinforcement as compared to conventional Grade 60 steel.

This paper evaluates the use of MMFX steel as main flexural reinforcement in concrete bridge decks. Assessment of the impact of arching action, developed due to the use of this new steel, on bridge deck strength is also presented.

PHASE I: TENSILE STRENGTH OF BENT BARS

Specimens and Test Setup

The typical specimen used to evaluate the effect of bending on tensile strength consisted of two concrete blocks which served to anchor the two ends of a bent bar stirrup as shown in Fig. 1(a). Two specimens were tested for each of two bar sizes, #4 (No.13) and #5 (No.16). The radii of the 90° bends used were in accordance with ACI 318-02 (ACI, 2002) and are shown in Fig. 1(a). The length of the stirrups was selected based on the designed dimensions of the concrete blocks, the dimensions of the hydraulic jack, and the thickness of the load cell. The total embedded length (straight and curved portions) of one end of each stirrup was totally debonded from the concrete using a thick rubber tape. On the opposite end each stirrup, only the straight portion of the embedded length was debonded. The details of the test setup for #5 (No.16) specimens are shown in Fig. 1(b). The concrete blocks were heavily reinforced with conventional Grade 60 stirrups to prevent premature failure of the blocks themselves. The concrete was cast using wooden forms which were specially constructed to accommodate the protruding stirrups, and to prevent stresses from developing in the exposed bars prior to testing.

![Fig. 1: Dimensions, schematic details, and test setup for bent bar specimens.](image-url)
The test setup shown in Fig. 1(c), consists of a 120 kip capacity (534 KN) hydraulic jack used to apply load between the two blocks, a 150 kip capacity (667 KN) load cell used to measure the applied load, and four linear potentiometers used to measure the relative displacement between the two blocks. The hydraulic jack and the load cell were centered between the two branches of the stirrup to assure equal distribution of forces in each branch. An MTS axial mechanical extensometer of 2 in. (51 mm) gage length was mounted on the exposed length of the stirrup to measure elongation during loading. An OPTIM Megadac data acquisition system was used to electronically record the readings of the load cell, the potentiometers, and the extensometer.

**Test Results**

Failure of all four bent bar specimens occurred inside the blocks at the end of the stirrup which was totally debonded. After testing, the concrete blocks of the four tested specimens were cut open using a concrete saw to allow for inspecting of the failure. As expected, all four specimens failed at the location of the bend. Fig. 2 shows the failure location of all specimens as photographed after cutting the concrete blocks. The measured stress-strain characteristics for #4 (No.13) and #5 (No.16) bent bars along with data measured for straight bars are shown in Fig. 3. The mode of failure of the four bent bars, along with the measured stress-strain characteristics, indicate that the behavior of the bent bars was similar to that observed in straight bars, including linear and non-linear behavior up to a strain value of 1.5 percent. Test results also indicate that bending the MMFX bars induced residual strain which influenced both the strength and strain at ultimate. This result reflects the well established phenomenon that a stress concentration resulting from the bending process develops at the bend location. It should be mentioned that in typical structures, bent bars are bonded entirely in concrete and thus, the bend will not usually be subjected to the extreme tension which was applied in this testing program. A complete bond should only enhance behavior due to the confinement provided by the concrete. Based on these limited number of tests, the results suggest that bending of MMFX bars up to 90 degrees reduces the ultimate strength by 6 percent and the ultimate strain by 70 percent.

![Fracture](image1)
(a) #4 (No.13)-first specimen

![Fracture](image2)
(b) #4 (No.13)-second specimen

![Fracture](image3)
(c) #5 (No.16)-first specimen

![Fracture](image4)
(c) #5 (No.16)-second specimen

Fig. 2: Failure location of bent bar specimens.
PHASE II: CONCRETE BRIDGE DECKS

Test Models

The three concrete bridge decks considered in this study were identical in all aspects except for the type and amount of reinforcing steel used in each. Each bridge deck consisted of two spans and two cantilevers which were supported in composite action by three pre-cast, post-tensioned concrete girders each having a cross-sectional dimension of 24x10 in. (610x254 mm). The overall nominal dimensions of each bridge deck were 21'-10"x13'-2"x8$^{1/4}$" (6655x4013x220 mm). The supporting girders were post-tensioned using 1 in. (25 mm) diameter deformed prestressing bars with an ultimate strength of 150 ksi (1034 MPa). Each girder was prestressed by four bars, resulting in a total prestressing force of 360 kips (1601 KN) per girder.

The first and third bridge decks were reinforced with MMFX steel, while the second bridge deck was reinforced with conventional Grade 60 steel for comparison purposes. The test matrix is given in Table 1, and the reinforcement details for all three bridge decks are shown in Fig. 4. It should be noted that the reinforcement ratio ($\rho$) is calculated using the total slab thickness. The first and second bridge decks were constructed with the same reinforcement ratio using MMFX and conventional Grade 60 steel, respectively. The third bridge deck was reinforced with MMFX steel using a reinforcement ratio only two-thirds of that used for the first deck. Reducing the reinforcement ratio was done in an attempt to utilize the higher tensile strength of MMFX steel. It should be noted that the MMFX reinforcement used in first bridge deck was identical to that used in an actual bridge built in 2004 in Johnston County, North Carolina, USA. All three of the tested bridge decks had a span and thickness which matched the deck of the actual bridge. In addition, the tested decks were all supported on girders which were designed to have the same torsional stiffness as those used to support the Johnston County bridge.

Table 1: Bridge decks test matrix.

<table>
<thead>
<tr>
<th>Bridge Deck</th>
<th>Steel Type</th>
<th>Bottom Reinforcement</th>
<th>Top Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Transverse</td>
<td>Longitudinal</td>
</tr>
<tr>
<td>First</td>
<td>MMFX</td>
<td>#5 @ 6.75&quot;</td>
<td>#5 @ 10&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(#16 @ 170)</td>
<td>(#16 @ 250)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\rho = 0.54%$</td>
<td>$\rho = 0.36%$</td>
</tr>
<tr>
<td>Second</td>
<td>Grade 60</td>
<td>#5 @ 6.75&quot;</td>
<td>#5 @ 10&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(#16 @ 170)</td>
<td>(#16 @ 250)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\rho = 0.54%$</td>
<td>$\rho = 0.36%$</td>
</tr>
<tr>
<td>Third</td>
<td>MMFX</td>
<td>#5 @ 10&quot;</td>
<td>#5 @ 10&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(#16 @ 250)</td>
<td>(#16 @ 250)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\rho = 0.36%$</td>
<td>$\rho = 0.36%$</td>
</tr>
</tbody>
</table>
Material Properties

The average concrete compressive strengths at the day of testing the three bridge decks were 7000, 4500, and 5278 psi, respectively (48.2, 31, and 36.4 MPa). The concrete compressive strengths were determined using 4x8 in. (102x204 mm) concrete cylinders cast with each deck and cured under the same conditions as that deck. Concrete was batched and delivered by a local supplier.

Tension coupons of MMFX and Grade 60 steels were tested according to ASTM-A370 specifications. The measured stress-strain characteristics of the MMFX and Grade 60 steel are shown in Fig. 5. The MMFX reinforcing bars exhibit a linear stress-strain relationship up to 100 ksi (689 MPa), followed by a nonlinear behavior and ultimate strength of 173 ksi (1193 MPa). According to the ASTM-A370 offset method (0.2% offset), the yield strength was determined to be 120 ksi (827 MPa). The initial modulus of elasticity was determined to be 29,000 ksi (200 GPa), followed by a nonlinear behavior and reduction in the modulus of elasticity after the stress exceeded 100 ksi (689 MPa). The yield strength of the Grade 60 steel was determined to be 68 ksi (469 MPa).

Fig. 5: Stress-strain characteristics of Grade 60 and MMFX steel.
Test Setup

Two 440 kip capacity (1957 kN) MTS hydraulic actuators were used to apply a concentrated load to both spans simultaneously to simulate the effects of truck wheel loading. Two 10x20 in. (254x508 mm) steel plates were used to transfer the load from the actuators to the deck. This size plate is specified as a representative tire contact area by the AASHTO LRFD Bridge Design Specifications (AASHTO, 1998). A ½ in. (13 mm) thick neoprene pad was placed under each loading plate to prevent possible local crushing of the concrete. The supporting girders rested on concrete reaction blocks which transferred the applied load to the strong floor. This configuration resulted in a clear span of 96 in. (2438 mm) for each girder. The clear span was carefully set based on calculations equating of torsional stiffness of the prestressed girders used in the test to that of the steel girders used in the Johnston County bridge. Fig. 6 shows an isometric view of the test setup and also shows the first bridge deck prior to testing.

A total of 72 channels of instrumentation were used for each bridge deck test. A 440 kip capacity (1957 KN) load cell was mounted to each actuator to measure the applied load. Twenty-four string potentiometers (string pots) were used to record deflection profiles along the longitudinal and transverse directions of each bridge deck. In addition, six linear potentiometers were used to measure the deflections and rotations of each girder. Twenty PI gages were used to measure the concrete strain at various locations. The measured strains were used to determine the strain profiles of the sections at each measured location. Twenty electrical resistance strain gages of 120 ohm resistance and 6 mm gage length were attached to selected reinforcing bars to determine the strains in these bars. Data were electronically recorded by an Optim Megadac data acquisition system. Fig. 7 shows the locations of the PI gages used and establishes the notation adopted hereafter.

![Test setup and first bridge deck prior to testing](image)

Fig. 6: Test setup and first bridge deck prior to testing.

![PI gages locations and notations for the three bridge decks](image)

Fig. 7: PI gages locations and notations for the three bridge decks.
Test Results

Load-Deflection Behavior
The load-deflection envelopes up to failure of all three bridge decks are given in Fig. 8. It should be noted that deflection plotted in Fig. 8 is measured at the center of the respective deck span directly underneath the applied load. It is readily apparent from Fig. 8 that the first bridge deck, reinforced with MMFX steel using the same reinforcement ratio as the actual bridge, exhibited smaller deflections in comparison to the other two bridge decks. Due to the use of a higher reinforcement ratio in the first bridge deck, stiffness was higher than in the other two decks; this could also be attributed to the higher compressive strength of the concrete used in the first deck. Despite the lower reinforcement ratio of the third bridge deck (33 percent less than that of the first two decks), this deck was capable of sustaining the same load as the second bridge deck reinforced with Grade 60 steel. This behavior is attributed to the utilization of the higher tensile strength of the MMFX steel reinforcement. The slight increase in deflection measured in the third bridge deck in comparison to the second is possibly due to the slight reduction of the modulus of elasticity of MMFX steel at higher stress levels.

According to the AASHTO LRFD Bridge Design Specifications (AASHTO, 1998), the design tandem consists of a pair of 25 kip (111 KN) loads. Therefore, at a load level of 25 kips (111 KN), which is less than the cracking load; deflection at service was almost identical for the three bridge decks.

Deflection Profiles
The deflection profiles along the longitudinal direction of the three bridge decks are given in Figures 9, 10, and 11, respectively. It should be noted that each deflection profile is plotted for the final loading cycle only. The initial deflections shown for each deck represent the residual deflections from previous loading cycles. The deflection profiles for the three bridge decks indicate that the deflection at the edge of the bridge decks was very small. This implies that selection of the length of the model was effective in carrying the total load, and therefore, is representative to the actual bridge deck.
The deflection profiles along the transverse direction of the three bridge decks are given in Figures 12, 13, and 14, respectively. Again, it should be noted that the deflection profiles are plotted for the last loading cycle only and therefore, residual deflections are shown at the beginning of each loading cycle (zero load). The profiles indicate that the maximum deflections in each case occurred at the mid-span under the applied load. Also, it is clear that the spans which failed in punching shear (right span) exhibited less deflection than the spans which failed due to flexure. This observation will be discussed further in the following sections.
Mode of Failure

In general, the behavior of the experimentally tested bridge decks was a two-way flexural mode, followed by the development of an arching action. This arching action was supported by membrane forces which developed in the bottom layer of reinforcement. At the first peak load of the first bridge deck, a sudden drop in load occurred due to the formation of flexural-shear cracks along the top surface of the bridge deck on both sides of the middle girder. Further loading led to the widening of those cracks, this widening associated with a slight increase in load resistance until punching failure occurred. Punching failure of both spans occurred simultaneously at a load level of 229 kips (1019 N) and 216 kips (961 N) for the left and right spans, respectively. Fig. 15 shows the first bridge deck at the conclusion of the test, where the punching areas under the two concentrated loads can be seen clearly along with the shear cone at the bottom of the left span.
The behavior of the second bridge deck, reinforced with Grade 60 steel using the same reinforcement ratio as the first deck, was similar to the behavior of the first deck. At the peak load of the left span, a sudden drop in load occurred due to the formation of a flexural-shear crack on the top surface of the bridge deck to the left of the middle girder only (left span only). This drop in the load made the left span incapable of carrying the higher load equivalent to the punching shear capacity of the deck. The test was terminated due to excessive deflections in the left span. The smooth decrease of the load carrying capacity of the left span reveals that flexural-shear failure was the mode of failure in the left span. The maximum measured load for the left span prior to termination was 185 kips (823 KN), corresponding to a deflection of 2.2 in. (56 mm). Failure of the right span was due to punching shear at a load level of 204 kips (907 KN).

Similar to the second bridge deck, the right span of the third bridge deck failed due to punching shear prior to the failure of the left span. A flexural-shear crack formed in the left span, causing a sudden drop in load. This drop rendered the left span incapable of carrying more load, and thus it was not able to reach a load corresponding to its punching shear resistance. Flexural-shear failure was the mode of failure of the left span, as shown by the smooth decrease in the load carrying capacity of the load. The right span failed in punching shear at a load level of 203 kips. The test terminated due to excessive deflections in the left span after a maximum recorded load for the left span of 181 kips (805 KN). Fig. 16 shows the second and third bridge decks at failure, where the punching area under the actuator in the right span, and the flexural-shear crack formed in the left span are clearly visible.

**Crack Pattern**

No cracks were observed up to a load level of 50 kips (222 KN) for any of the three bridge decks. However visible top cracks started to occur at a load level of roughly 60 kips (267 KN) for each deck. Fig. 17 shows the top cracks at a load level of 100 kips (444 KN). Negative flexural cracks formed before the positive cracks due to the higher values of negative moments in comparison to the positive moments.
Positive moment flexural cracks at load levels of 100 and 150 kips (444 and 666 KN) for the first bridge deck are shown in Fig. 18, where the crack pattern confirms the two way distribution of the load. Further loading led to a spreading and widening of the flexural cracks until the formation of the flexural-shear crack at the top surface of the deck close to the middle girder. The formation of this flexural-shear crack led to a sudden drop in the load as was previously discussed. However, a flexural-shear crack formed symmetrically on both sides of the middle girder of the first bridge deck, as shown in Fig. 19. This phenomenon allowed a gradual yet simultaneous increase of the load to cause a punching shear failure in both spans. For the second and third bridge decks, the flexural-shear crack occurred on the left side of the middle girder only, and allowed the load to increase in the right span, thus causing punching shear failures in the right spans only as shown in Fig. 16.
Strain Profiles and girders rotation

Based on the deformations measured by the PI gages, strain profiles were determined using the measured strain at the extreme top and bottom fibers of each bridge deck. It should be noted that all the strain profiles are plotted for the final loading cycle only, and therefore, residual strains are shown at zero load. The strain profile obtained from the two PI gages located in the right span at 14 in. (356 mm) from the centerline of the deck (T6 and B10 in Fig. 7) is depicted in Fig. 20. The strain profile indicates that the top surface of the concrete at the vicinity of the punching area exceeded the limiting compressive strain value. The strain profile obtained from the PI gages at the edge of the right span (T8 and B12 in Fig. 7) is shown in Fig. 21. The strain profile shows that the strain values were very small, which is another indication that the length of the bridge deck is effective and representative to the behavior of typical bridges.

Fig. 20: Strain profiles from T6 and B10 PI gages for the three bridge decks.
Rotation of the three supporting girders was monitored throughout the tests of the three decks as shown in Fig. 22. For all three decks, the two outside girders exhibited larger rotations in comparison to the middle girder due to the unbalanced moment effect.

**Predicted Strength**

The predicted shear strengths for the three bridge decks according to several different design codes are given in Fig. 23 along side the values measured experimentally. The design codes included are: AASHTO LRFD Bridge Design Specifications (AASHTO, 1998), American Concrete Institute (ACI, 2002), and Ontario Highway Bridge Design Code (OHBDC, 1990).

The equations used are as follows:

**AASHTO**:

$$ V_c = \min \left[ 0.063 + \frac{0.126}{\beta_c} + 0.126 \sqrt{f_c' b_o d} \right] \text{ units: kips & in. } \quad \text{Equation(1)} $$

**ACI**:

$$ V_c = \min \left[ 2 + \frac{4}{\beta_c} + 4 \frac{\alpha_s d}{b_o} + 2 \sqrt{f_c' b_o d} \right] \text{ units: lbs & in. } \quad \text{Equation(2)} $$
OHBDC: 

\[ V_c = \left[ 0.6f_r + 0.25f_{pc} \right] p_o d + 0.9V_p \]  

units: N & mm  \hspace{1cm} \text{Equation}(3)

Where; \( V_c \) = punching shear capacity of bridge deck; \( \beta_c \) = ratio of long side to short side of loading plate; \( f_c' \) = concrete compressive strength; \( b_o \) = perimeter of critical section at a distance of \( d/2 \) from loading plate; \( d \) = effective section depth; \( \alpha_s \) = constant; \( f_t \) = concrete tensile strength; \( f_{pc} \) = compressive stress in concrete due to prestressing; and \( V_p \) = component of effective prestressing force in direction of applied shear.

It is clearly seen from Fig. 23 that the predicted values according to the AASHTO and ACI design codes match very well to the measured values for the bridge decks using MMFX and Grade 60 steel.

**ANALYTICAL MODELLING**

**General**

The three bridge decks were modeled using the finite element analysis program “ANACAP” (Anatech Concrete Analysis Program) Version 3.0, (James, 2004). The concrete material model is based on smeared cracking methodology developed by Y. R. Rashid (Rashid, 1960). Within the concrete constitutive model, cracking and all other forms of material non-linearity are treated at the finite element integration points. Cracks are assumed to form perpendicular to the principal tensile strain direction in which the criterion is exceeded and they are allowed to form at each material point. When cracking occurs, the normal stress across the crack is reduced to zero and distribution of cracks around the crack is recalculated. Cracks may close or re-open under load reversals. Concrete modeling also includes residual tension stiffness for the gradual transfer of load to the reinforcement during crack formation. In addition, the program accounts for the reduction in shear stiffness due to cracking, and allows for further decay as the crack opens. Reinforcement is modeled as individual sub-elements within the concrete elements. The stiffness of the bar sub-element is superimposed on the concrete element stiffness in which the bar resides. The anchorage loss is modeled as an effective stiffness degradation of the bar as a function of the concrete strain normal to the bar.

A 3-D analysis was conducted for the three bridge decks using 20-node hexahedral continuum elements. Only one quarter of the deck was modeled due to its symmetry about both axes. The depth of the deck was divided into five layers within its thickness with a total of 1040 elements, as shown in Fig. 24.
Analytical Results

The predicted and experimental load-deflection envelopes for the three bridge decks are compared in Fig. 25. It can be seen that the predicted load-deflection behaviors of the three bridge decks compared very well with the measured values. The initial and post-cracking stiffnesses were accurately predicted by the analytical model. In addition, the ultimate load was very reasonably predicted considering the fact the two spans of the second and third bridge decks failed in two different modes. However, the predicted ultimate deflection was slightly less than the experimental values; this is due to the nature of the smeared cracking methodology adopted by the program. For validation purposes, the contours of the principal strain at failure and the portion of the first bridge deck that failed due to punching failure are shown in Fig. 26. The strain contours depict a punching shear cone which matches very well to the experimental failure shear cone.

Fig. 24: Mesh used for analytical model.

Fig. 25: Analytical and experimental load-deflection envelopes for three bridge decks.
CONCLUSIONS

1. The ultimate load carrying capacity of the three bridge decks tested in this investigation was eight to ten times the service load specified by AASHTO Design Specifications (1998).
2. Punching shear failure was the primary mode of failure for the three bridge decks tested.
3. Punching failure resulted in sudden decrease of the load carrying capacity, while flexural failure resulted in gradual decrease of the load carrying capacity.
4. When compared to decks reinforced with Grade 60 steel at the same reinforcement ratio, bridge decks reinforced with MMFX steel exhibited the same deflection at service load, but developed more load carrying capacity.
5. Bridge decks reinforced with 33 percent less MMFX steel developed the same ultimate load carrying capacity and deflection at service load as those reinforced with Grade 60 steel. This is attributed to the higher strength of the MMFX steel compared to Grade 60 steel.
6. Behavior of bent MMFX bars is similar to that of straight bars including the linear and the non-linear behavior up to strain of 1.5 percent. However, bending MMFX bars severely impacts ductility as it reduces ultimate strength by 6 percent and ultimate strain by 70 percent.

ACKNOWLEDGMENT

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