RIGID PAVEMENT 100 KSI STEEL LANE TIE BAR SUBSTITUTION ANALYSIS AND DESIGN

Prepared for:

MMFX TECHNOLOGIES CORPORATION

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Introduction
The MMFX Steel Corporation produces a MMFX 2 (ASTM A 1035) Grade 100 and Grade 120 corrosion resistant steel reinforcing bar. Typical bar requirements for pavement reinforcement are for #5 bar, Grade 60 steel. The American Association of State Highway and Transportation Officials (AASHTO) published a standard, MP 18, for uncoated, corrosion resistant bars that allow up to Grade 100 steel. An analysis of the potential to substitute grade 100 steel for the typical Grade 60 is needed. The analysis should include structural (tensile stress) comparisons, development length limitations and identification of any process limitations that may affect the use of certain size bar or steel grade 100. At MMFX’s request, CME Transportation Group has performed this analysis herein.

Steel Reinforcement Design Background
Portland Cement Concrete Pavement is typically constructed in three forms, plain-jointed, jointed-reinforced, and continuously reinforced. All three forms require reinforcing steel to deal with either temperature or load stresses. Reinforcement is typically corrosion resistant, in the form of epoxy coating, to extend the life and performance of the reinforcement. The steel is located in the pavement across joints, either longitudinal or transverse, or within the slab where cracks would be expected to occur.

The primary purpose of the reinforcement is to hold the concrete together after cracking or jointing. Design of the tie bars for longitudinal joints or reinforcing for plain-jointed pavement is typically based on Federal Highway Administration (FHWA) or State Highway Agencies (SHAs) recommended practices. The AASHTO 1993 Guide for Design of Pavement Structures (AASHTO 1993 Guide) is the primary accepted practice for identifying and designing reinforcement at the State Highway Agency level. Specifications for bar size and spacing are loosely based on structural requirements, with significant emphasis placed on past practices and experiences. The American Concrete Institute publication 318 (ACI 318) also has significant influence with recommended practices for designing reinforced concrete.

Current typical reinforcing steel specifications require Grade 40 (40 ksi) or Grade 60 (60 ksi) epoxy coated steel. This is primarily a cost decision based on first cost analysis of pavement construction. Many higher grade steels, such as stainless or low-carbon corrosion resistant (MMFX), have
demonstrated equal or better corrosion resistance performance; however, they are unable to compete in a first cost scenario with equivalent percentages of steel.

Several higher grade steels are designed to meet Grade 100 (100 ksi) strengths. Redesign of the steel size and spacing for the pavement using equivalent strength performance instead of equivalent percentages would potentially allow them to take advantage of enhanced materials properties and compete in first cost comparisons.

**Steel Substitution Analysis**

Structural substitution of Grade 60 steel with Grade 100 steel needs to take into account the tension stresses, development lengths and general practices of specifying agencies. While tension stresses are present in both tie bar and internal reinforcement, development lengths are primarily associated with tie bars. Analysis for longitudinal reinforcement for Jointed-reinforced concrete pavements follows the same approach. Analysis of longitudinal bar in Continuously-reinforced-concrete pavements requires redevelopment of the design models and subsequent field verification trials, and is not addressed within this report.

Tie bar design based on combination of slab movement and historical practice. Slab movement is based on weight of slab, friction factor with base, flexibility of steel need. The AASHTO 1993 Guide3 uses the following process to design tie bars:

Determine percentage of steel required to hold slabs together using

\[ P_S = \frac{LF}{2f_s} \times 100 \]

where

\[ P_S = \text{percent steel reinforcement required}, \]
\[ L = \text{slab length (shortest distance to free edge)}, \]
\[ F = \text{friction factor (Table 2.8), and} \]
\[ f_s = \text{steel working stress (75% of yield strength)}. \]

Percent steel is transformed to bar size by designer selection, and spacing is then determined using

\[ Y = \frac{A_s}{P_tD} \times 100 \]

where

\[ Y = \text{transverse steel spacing (inches)}, \]
\[ A_s = \text{cross-sectional area of transverse reinforcing steel (in}^2\text{)}, \]
\[ P_tD = \text{tensile force in transverse reinforcing steel (ksi)}. \]
P_t = percent transverse steel (P_s), and
D = slab thickness (inches).

The selection of bar size and working stress by the designer will define the spacing requirements. While it is not possible to anticipate designer’s thoughts, a substitution process can be developed to convert original size and spacing to a modified size and/or spacing that will give equivalent performance.

With all constraints held the same, the change from a Grade 60 bar to a Grade 100 bar will affect the bar spacing, diameter and working stress. A simple relational calculation can be made to switch from one bar size to another based on increase working stress. Of the three attributes of tie bar design, working stress, bar size and bar spacing, arbitrary selection of two will define the remaining.

Bar spacing is typically identified based on past practice, recommended maximums or other non-structural factors such as form-hole locations. To address increased working stress in the steel, holding tie-bar spacing the same and allowing bar size to vary gives a more logical solution to the industry. It should also be noted that shorter spacing will lead to smaller bars which are more flexible and less likely to affect the working portion of the joint due to curling and warping. For internal reinforcement in a plain-jointed pavement, the option to use same size bars and reduce the number of bars to be placed may be a more economical solution.

To determine the appropriate bar size required when substituting 100ksi steel for the typical 60 ksi steel, the following equation can be used:

\[ A_{100} = \frac{f_{w60}}{f_{w100}} A_{60} \]

where

- \( A_{100} \) = cross-sectional area of 100ksi bar (in²),
- \( f_{w60} \) = working stress of 60ksi steel,
- \( f_{w100} \) = working stress of 100ksi steel, and
- \( A_{60} \) = cross-sectional area of 60ksi bar (in²).

Appropriate bar size replacements can be determined by comparing the ratio of the bar sizes to the ratio of the working stresses of the steel. For changing from 60ksi to 100ksi steel, the ratio of working stresses is 45/75 or 0.6. Bar size reductions will be appropriate, provided the new bar cross-sectional area is at least 60% of the original bar cross-sectional area. For changing from 60ksi to 100ksi steel, minimum nominal bar size changes are listed in Table 1. Reduction of a #4 Grade 60 bar to a #3 bar Grade 100 bar does not meet the minimum area requirements. Use of Grade 120 steel would satisfy the area requirements.
Table 1. Bar Size Adjustments for Changing from 60ksi to 100ksi Steel

<table>
<thead>
<tr>
<th>Original Bar (60ksi)</th>
<th>New Bar (100ksi)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Number</strong></td>
<td><strong>Area (in²)</strong></td>
<td><strong>Number</strong></td>
</tr>
<tr>
<td>#8</td>
<td>0.79</td>
<td>#7</td>
</tr>
<tr>
<td>#7</td>
<td>0.60</td>
<td>#6</td>
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<tr>
<td>#6</td>
<td>0.44</td>
<td>#5</td>
</tr>
<tr>
<td>#5</td>
<td>0.31</td>
<td>#4</td>
</tr>
</tbody>
</table>

*Tie bar design principles recommend maximum bar size of #7 to maintain joint flexibility.

Development Strength

Development strength is a combination of the chemical and mechanical bond achieved between a deformed steel bar and the concrete it is placed in. Placement of a bar in PCC pavement must include sufficient length of bar to achieve development strengths in excess of the working stress in the bar. ACI 318⁴ gives the following equation for identifying the length of bar to be embedded to develop sufficient bond to prevent the bar from dislodging due to pull-out forces on the bar.

\[
l_d = \frac{f_y \psi_t \psi_e \psi_s \lambda}{25 \sqrt{f'_c}} d_b
\]

where

- \(l_d\) = Development length of deformed bar in tension (in)
- \(f_y\) = Yield stress of bar
- \(\psi_t\) = Reinforcement location factor
- \(\psi_e\) = Coating factor
- \(\psi_s\) = Bar size factor
- \(\lambda\) = Concrete tensile strength factor
- \(f'_c\) = Compressive strength of concrete
- \(d_b\) = Bar diameter

To determine the actual development length needed, the actual stress on the individual bar must be identified. The actual stress is a function of bar size, spacing and overall separation force on the joint.

Using the percent steel and spacing equations previously identified, using a maximum friction factor and a maximum slab depth of 13 inches, the stress in an individual bar can be calculated as a function of bar size and spacing. Expected maximum stress values based on this approach are tabulated in Table 2. These stresses can be used to calculate the minimum development lengths required. These development lengths are identified in Table 3. Table 3 values in the shaded areas
require the standard ACI minimum bar length of 24 inches. The non-shaded areas require bar lengths that are twice the development length shown in the table.

Table 2. Maximum expected typical in-place stresses for variable bar size and spacing, Based on 13 inch slab and lean concrete.

<table>
<thead>
<tr>
<th>Spacing (in)</th>
<th>Bar Size</th>
<th>12</th>
<th>15</th>
<th>24</th>
<th>30</th>
<th>36</th>
<th>48</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8</td>
<td></td>
<td>2,085</td>
<td>2,607</td>
<td>4,171</td>
<td>5,213</td>
<td>6,256</td>
<td>8,341</td>
</tr>
<tr>
<td>#7</td>
<td></td>
<td>5,491</td>
<td>6,864</td>
<td>10,982</td>
<td>13,728</td>
<td>16,474</td>
<td>21,965</td>
</tr>
<tr>
<td>#6</td>
<td></td>
<td>9,360</td>
<td>11,700</td>
<td>18,720</td>
<td>23,400</td>
<td>28,080</td>
<td>37,440</td>
</tr>
<tr>
<td>#5</td>
<td></td>
<td>13,285</td>
<td>16,606</td>
<td>26,570</td>
<td>33,213</td>
<td>39,855</td>
<td>53,141</td>
</tr>
<tr>
<td>#4</td>
<td></td>
<td>20,592</td>
<td>25,740</td>
<td>41,184</td>
<td>51,480</td>
<td>61,776</td>
<td>82,368*</td>
</tr>
</tbody>
</table>

* Exceeds working stress (75% of yield) of Grade 100 steel.

Table 3. Development lengths for stresses identified in Table 2.

<table>
<thead>
<tr>
<th>Spacing (in)</th>
<th>Bar Size</th>
<th>12</th>
<th>15</th>
<th>24</th>
<th>30</th>
<th>36</th>
<th>48</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8</td>
<td></td>
<td>1.32*</td>
<td>1.65*</td>
<td>2.64*</td>
<td>3.30*</td>
<td>3.96*</td>
<td>5.28*</td>
</tr>
<tr>
<td>#7</td>
<td></td>
<td>3.04*</td>
<td>3.80*</td>
<td>6.08*</td>
<td>7.60*</td>
<td>9.12*</td>
<td>12.16</td>
</tr>
<tr>
<td>#6</td>
<td></td>
<td>4.44*</td>
<td>5.55*</td>
<td>8.88*</td>
<td>11.10*</td>
<td>13.32</td>
<td>17.76</td>
</tr>
<tr>
<td>#5</td>
<td></td>
<td>5.25*</td>
<td>6.56*</td>
<td>10.50*</td>
<td>13.13</td>
<td>15.75</td>
<td>21.01</td>
</tr>
<tr>
<td>#4</td>
<td></td>
<td>6.51*</td>
<td>8.14*</td>
<td>13.02</td>
<td>16.28</td>
<td>19.54</td>
<td>26.05</td>
</tr>
</tbody>
</table>

*Minimum length required by ACI 318 is 12 inches.

For standard bar substitutions, since working stress is typically not exceeded, replacement is based on equivalent structural design. Using a combination of Table 1 values for appropriate bar size reduction from Grade 60 to Grade 100, and table 3 minimum development lengths for each bar size, the appropriate bar size and length substitutions can be determined. The total bar length is twice the development length, with development length needed for both ends of the bar. Using constant spacing and reducing bar size only, appropriate substitutions are identified in Table 4.

Table 4. Bar size and minimum lengths for switching from Grade 60 steel to Grade 100 steel.

<table>
<thead>
<tr>
<th>Original Design Bar Size (Grade 60)</th>
<th>Spacing (in)</th>
<th>12</th>
<th>15</th>
<th>24</th>
<th>30</th>
<th>36</th>
<th>48</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8</td>
<td>#7 x 24&quot;</td>
<td>#7 x 24&quot;</td>
<td>#7 x 24&quot;</td>
<td>#7 x 24&quot;</td>
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<td>#7 x 24&quot;</td>
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<td>#7</td>
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<td>#6 x 24&quot;</td>
<td>#6 x 24&quot;</td>
<td>#6 x 24&quot;</td>
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<td>#6 x 25&quot;</td>
<td></td>
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<tr>
<td>#6</td>
<td>#5 x 24&quot;</td>
<td>#5 x 24&quot;</td>
<td>#5 x 24&quot;</td>
<td>#5 x 24&quot;</td>
<td>#5 x 27&quot;</td>
<td>#5 x 36&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>#4 x 24&quot;</td>
<td>#4 x 24&quot;</td>
<td>#4 x 24&quot;</td>
<td>#4 x 27&quot;</td>
<td>#4 x 32&quot;</td>
<td>#4 x 42&quot;</td>
<td></td>
</tr>
</tbody>
</table>
For example, if the original bar design specifies a #6 Grade 60 bar at 36” centers, the substituted Grade 100 bar would be a #5 bar, 28 inches long, also at 36” centers. The possibility exists to modify the spacing along with bar size to accomplish equivalent design. This is discussed in detail in the Specialized Substitutions section.

It should be noted that for substitution of #4 Grade 60 bar, reduction to a #3 Grade 100 bar does not provide sufficient cross-sectional area to provide equivalent design. As tie-bar performance is a combination of bar size and spacing, substitution of #4 bars should be based on increased spacing. The working stress ratio of the steel strengths (G60/G100) is 0.6. The spacing of #4, Grade 60 bars can be increased inversely to a spacing 1.6 times that specified to give equivalent structural design (i.e #4 Grade 60 @ 30” = #4 Grade 100 @ 48”). This is discussed further in the Specialized Substitutions section below.

Specialized Substitutions
Design of pavement reinforcing is based on a combination of expected stresses, governing guidelines and designer practices. Substitution of Grade 60 steel with Grade 100 steel will have similar characteristics. If a combination of bar size and spacing adjustment is desired, the proper combination of cross-sectional area and bar spacing for equivalent design must be maintained.

Combining the previous percent steel equation

\[ P_t = \frac{A_s}{YD} \times 100 \]

with

\[ P_{s60} \times f_{s60} = P_{s100} \times f_{s100} \]

and simplifying the gives

\[ A_{s60} \times \frac{f_{s60}}{Y_{60}} = A_{s100} \times \frac{f_{s100}}{Y_{100}} \]

where

- \( A_{s60} \) = Cross-sectional area of Grade 60 bar (in²)
- \( f_{s60} \) = Working stress of Grade 60 bar (ksi)
- \( Y_{60} \) = Spacing of Grade 60 bar (in)
- \( A_{s100} \) = Cross-sectional area of Grade 100 bar (in²)
- \( f_{s100} \) = Working stress of Grade 100 bar (ksi)
- \( Y_{100} \) = Spacing of Grade 100 bar (in)

This can be reformed as
In simple terms, whether for specified tie-bar design or calculated CRC design, substitution of Grade 60 steel with Grade 100 steel can be based on a reduction of cross-sectional area, increase in spacing, or combination of both, based on a ratio of working stresses. The working stress ratio of Grade 60 to Grade 100 is 45/75 or 0.6. Substituted cross-sectional areas must be greater than 60% of those specified. Modified spacing must be less than 160% of those specified. Designer preference or material availability may weigh on the decision.

This equation can also be used to change from Grade 40 steel using the appropriate working stress ratio.

**NON-ENGINEERING IMPACT AND LIMITATIONS**

Structural substitution can be performed using acceptable mathematical equations related to changes in steel strength; however, equations are not limited by practicality or designer preference. These issues need to be included in each project-level substitution analysis.

ACI 318 requires a minimum 12 inch length of bar for development. While some of the stresses that may be endured by individual tie bars may only be a fraction of their working or yield stresses, requiring only several inches of bar to provide proper development, industry minimums and standards must still be accounted for. It is possible for bars on individual projects to be shortened with the designer’s concurrence; however, a minimum 24 inch bar length should be considered the practical minimum, with shorter bars considered individual design exceptions.

The FHWA recommends maximum 48” tie bar spacing in a Technical Advisory: Concrete Pavement Joints. Again, mathematically, it is possible to meet structural requirements with a variety of bar sizes and strengths; however, based on empirical practices, a minimum number of bars (or maximum spacing) has been shown to be effective.

In the same advisory, the FHWA recommends using #4 and #5 bar, allowing for bars as large as #7. The advisory is based on use of Grade 40 and Grade 60 steel and does not address the potential for higher grade steel as seen in the ASTM Specification A 1035 and AASHTO Specification MP 18. If proper detail regarding development lengths and steel properties are addressed, it is logical to assume the bar sizes can be adjusted with minimal or no impact on performance of the pavement.
Currently, the American Concrete Pavement Association (ACPA) has commissioned the development of a new tie bar design procedure\cite{6}. It is understood that the procedure development will be finished by summer, 2009. Preliminary indications are that it will be based more on structural and mechanistic performance, instead of industry empirical practices. Should the practice be adopted by AASHTO, substitution practices for the tie-bars will need to be reviewed to determine if some of the non-engineering limitations and impacts can be removed.

**Substitution Procedures**

The substitution of higher grade steel for lower grade steel can be performed using Step by Step Process to follow for substitution.

For basic tie bar design in jointed or continuously reinforced pavements, or for internal reinforcement in jointed reinforced concrete pavements, where typical #5 Grade 60 bar or similar is specified, a simple substitution can be identified using Table 4. For specialized substitutions, where the desire exists to change spacing and/or bar size more than a single size, the following steps can be used.

Step 1. Select a combination of bar size and spacing that satisfies the following equation:

$$\frac{A_{sNew}}{Y_{New}} = \frac{A_{sDesign}}{Y_{Design} \times \left(\frac{f_{sDesign}}{f_{sNew}}\right)}$$

Step 2. Calculate development lengths for the new steel using project specific data and the following equation:

$$l_a = \frac{f_y \psi_s \psi_e \psi_p \lambda}{25 \sqrt{f'_c}} d_b$$

Step 3. Determine appropriate bar length by doubling development length and rounding up to an appropriate bar length for production. For spacings larger than 48 inches or development lengths less than 12 inches will likely require significant justification and discussion with the project pavement designer for approval as they are less than current standards.

In simple terms, whether for specified tie-bar design or reinforced concrete pavement design, substitution of lower Grade steel with higher Grade steel can be based on a reduction of cross-sectional area, increase in spacing, or combination of both, based on a ratio of working stresses and development length, and then accounting for industry standards.
Summary
The process of substituting higher grade steel for specified lower grade steel incorporates a combination of mathematical equivalency calculations, industry standards and designer preferences. Some portions of the substitution process can be reduced to simple tables for selection of appropriate bar sizes and lengths, however, significant portions of the substitution process require subjective selection of bar size and/or spacing and iterative review of design calculations. It is recommended that these portions of the substitution process be performed by individuals with knowledge of reinforced concrete pavement design practices and industry limitations.

References
2. AASHTO Subcommittee on Materials; *Uncoated, Corrosion-Resistant, Deformed and Plain Alloy, Billet-Steel Bars for Concrete Reinforcement and Dowels –Specification MP 18 M/MP 18-09*; American Association of State Highway and Transportation Officials, Washington D.C., 2009

Limitations
The conclusions and recommendations presented are professional opinion, based on engineering experience and judgment. As the extent of the review and analysis was limited, identification of all possible conditions is not guaranteed. This investigation was not able to include review of actual field performance correlated to predicted design expectations based on actual field materials properties and geometry.
**Author’s Bio**

Timothy Biel, P.E., MS in Civil Engineering.

Mr. Biel spent 14 years with the Utah Department of Transportation where, at different times, he was responsible for performance and review of pavement designs, and the programmatic oversight of the pavement design program.

As a Pavement Management Engineer in the Salt Lake Region, Mr. Biel was responsible for the performance of over 50 pavement designs, using the AASHTO 1993 procedure. Designs ranged from parking lots to interstates and included new and rehabilitation of HMA and PCC surfaces, recycling, full-depth reclamation and maintenance treatments. Mr. Biel was also the review and approval authority for over 50 consultant and UDOT performed designs, similar to those previous, including the I-15 corridor rebuild, the Legacy Highway and the Parley’s Canyon profile-milling and recycling.

As State Engineer for Materials, Mr. Biel’s programmatic pavement responsibilities included Pavement Design oversight, and policy development and implementation. Mr. Biel was Project Manager for three research projects related to Mechanistic-Empirical Pavement Design practice calibration and implementation, including development of materials property values.

In conjunction with ARA, Inc., Mr. Biel is currently under contract with the Utah DOT to perform training in and guide the implementation of the M-E guide in Utah.