DESIGN GUIDE FOR USE OF ASTM A1035 HIGH-STRENGTH REINFORCEMENT IN CONCRETE BRIDGE ELEMENTS WITH CONSIDERATION OF SEISMIC PERFORMANCE

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BACKGROUND

ASTM A1035 Reinforcement

Reinforcing bars manufactured in accordance with ASTM A1035 are low-carbon, chromium steel bars characterized by a high tensile strength and a non-linear stress-strain relationship with no yield plateau. The yield strength, therefore, is determined using the 0.2% offset method. Specified minimum yield strengths are 100 or 120 ksi for Grades 100 and 120, respectively. The specified minimum tensile strength of both grades is 150 ksi. In addition, the stress corresponding to a tensile strain of 0.0035 is required to have minimum values of 80 and 90 ksi for Grades 100 and 120, respectively. The last requirement is to ensure that the specified reinforcement is at least as stiff at lower strains as lower-strength reinforcing bars. The deformation requirements for ASTM A1035 reinforcement are identical to those required for Grades 60 and 75 reinforcement.

Although ASTM A1035 has a non-linear stress-strain curve, the relationship is essentially linear up to a stress of at least 70 ksi regardless of its ultimate strength. The modulus of elasticity within the linear portion is essentially the same as that of lower strength reinforcement, meaning that the behavior of structural members reinforced on a one-to-one replacement basis will be the same at the service load levels as members reinforced with lower strength reinforcement.

ASTM A1035 bars have a superior corrosion resistance when compared to conventional reinforcing steel grades. For this reason, designers have specified ASTM A1035 bars as a direct, one-to-one replacement for conventional reinforcing bars as an alternative to stainless steel or epoxy-coated bars.
AASHTO Specifications

The American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Construction Specifications (AASHTO 2010A) permits the use of ASTM A1035 reinforcement at locations where specified in the contract documents. The AASHTO LRFD Bridge Design Specifications (AASHTO 2010B), however, limits the yield strength of reinforcement to be used in design to 75 ksi for most applications. Therefore, although ASTM A1035 reinforcement may be specified for its corrosion resistance, its higher yield strength cannot be fully utilized.

The AASHTO specifications do not exclude the use of higher strength grades of reinforcement except when ASTM A706 reinforcement is specified. The specifications currently only limit the value of yield strength that may be used in design. If the enhanced strength of ASTM A1035 reinforcement could be used in design calculations, less reinforcement would be required, resulting in more efficient and economical bridges. To address the design issues involved in utilizing higher strength reinforcement, the National Cooperative Highway Research Program (NCHRP) initiated Project 12-77 titled "Design of Concrete Structures Using High-Strength Reinforcement" (Sharooz et al. 2011). The project included an evaluation of the LRFD specifications to identify the articles affected by the use of high-strength reinforcement. An integrated experimental and analytical program was then performed to develop data required to justify changes to the specifications. The analytical work included all reinforcement with strengths above 60 ksi and having a non-linear stress-strain relationship. The experimental work focused on the use of ASTM A1035 Grade 100 reinforcement. Finally, proposed revisions to the AASHTO specifications were developed. The revisions, which are summarized in Appendix A, are only applicable to bridges in AASHTO Seismic Zone 1. These revisions are currently being considered by a technical committee of the AASHTO Highway Subcommittee on Bridges and Structures, which has responsibility for the AASHTO design and construction (as distinct from materials) specifications.

In 2009, the AASHTO Highway Subcommittee on Materials published a provisional standard MP18 titled "Uncoated, Corrosion-Resistant, Deformed and Plain Alloy, Billet-Steel Bars for Concrete Reinforcement and Dowels." This standard has similar tensile requirements as ASTM A1035 but includes Grade 60 and 75 bars and no Grade 120. The ASTM A1035 requirement that the stress corresponding to a tensile strain of 0.0035 shall be a minimum of 80 ksi is not part of
AASHTO MP18. In addition, the maximum percentages of each chemical element in the two specifications are different. Nevertheless, this should not affect the structural performance of reinforcement meeting both specifications, provided the specified tensile properties are achieved.

In addition to the NCHRP project, private industry has sponsored numerous research projects to show that the higher strength reinforcement can be utilized in bridge structures. The subsequent information in this document is based on both NCHRP and other industry research that was incorporated into the NCHRP project report.

**PART 1: BRIDGES LOCATED IN AASHTO SEISMIC ZONE 1**

**FLEXURAL MEMBERS**

The current flexural design methodology of the AASHTO LRFD Design Specifications assumes that plane sections remain plane, uses a rectangular stress block to model concrete behavior, and assumes an elastic-plastic behavior for the reinforcement. This methodology was shown to be applicable for yield strengths up to 100 ksi.

To ensure adequate ductility of a reinforced concrete member the tension- and compression-controlled strain limits need to be revised from 0.005 and 0.002 for Grade 60 reinforcement to 0.008 and 0.004, respectively, for Grade 100 reinforcement. For intermediate grades of reinforcement, linear interpolation may be used. These strain limits were developed from an analytical study of 286 cases including seven grades of reinforcement, three concrete strengths, and multiple section geometries. Six large-scale beam tests confirmed the appropriateness of the limits and produced measured strengths and ductilities greater than calculated.

Although fatigue is unlikely to control the design of typical flexural bridge members, two large-scale tests and review of available publications demonstrated that presently accepted values for the fatigue limit for reinforcing bars are applicable with Grade 100 bars.

**COMPRESSION MEMBERS**

**Column Strengths**

Analytical parametric studies were performed to determine the moment-curvature response and axial load-moment interaction diagrams for columns reinforced with ASTM A1035 longitudinal
and transverse reinforcement. Variables included in the 270 cases analyzed included column type, reinforcement type, column size, transverse reinforcement size, and concrete strength. Results indicated that the current provisions of the LRFD Design Specifications are applicable for calculating column strengths with reinforcement yield strengths up to 100 ksi.

**Spiral Reinforcement**

For cases not controlled by seismic requirements, the volumetric ratio of spiral reinforcement in a column must be such that the axial load capacity after spalling of the concrete cover is at least equal to the capacity before spalling. The additional capacity after spalling is provided by confinement of the core by the spiral reinforcement. Based on the analyses performed in NCHRP Project 12-77, the authors concluded that the existing equation to calculate the volumetric ratio of spiral reinforcement is applicable for spiral reinforcement yield strengths up to 100 ksi. In comparison, ACI 318 (ACI 2008) also allows the use of yield strengths up to 100 ksi for spiral reinforcement in columns of buildings.

**SHEAR DESIGN**

**Shear Strength**

The AASHTO LRFD Design Specifications for shear include a Sectional Design Model, derived from the Modified Compression Field Theory, for determining the amount of shear reinforcement. The Sectional Design Model provides strain-based relationships to account for contributions from the concrete and the transverse reinforcement to overall shear capacity.

To determine if the method is applicable with ASTM A1035 reinforcement with a yield strength of 100 ksi, five rectangular reinforced concrete beams and four AASHTO Type I prestressed concrete beams were built and tested to destruction. One end of each beam used ASTM A615 stirrup reinforcement. The other end used ASTM A1035 reinforcement. Both ends were designed to have the same shear strength by changing the reinforcement size and spacing. During testing, little difference between the behavior of the opposite ends of each beam was observed.

Following testing, the shear capacities were calculated using the as-built material properties and the Sectional Design Method. Measured capacities ranged from 22 to 83% greater than the calculated values. The use of the current specification procedures for calculating shear capacity was found to be acceptable for yield strengths of shear reinforcement up to 100 ksi.
Interface Shear Transfer

The AASHTO LRFD Design Specifications limits the value of reinforcement yield strength to be used in shear friction design to 60.0 ksi. This limit restricts the strain at the interface crack to ensure adequate aggregate interlock capacity across the interface. Eight push-off tests were conducted using ASTM A1035 or ASTM A615 bars to determine if the limit of 60.0 ksi could be increased when using ASTM A1035 reinforcement. The results clearly demonstrated that the limit of 60.0 ksi needs to be retained.

This design provision primarily affects design of the interface between precast concrete beams or cast-in-place bent caps and cast-in-place concrete decks where stirrups from the beams or bent caps protrude into the deck. A yield strength of 100.0 ksi may still be used to determine the amount of shear reinforcement whereas 60.0 ksi must be used to determine the amount of reinforcement across the interface.

Punching Shear

Three full-scale concrete bridge deck specimens were tested for punching shear by Seliem et al. (2008). The first two specimens had the same reinforcement ratio using either high-strength reinforcement or Grade 60 reinforcement representing a 1:1 substitution. The third specimen used 33% less high-strength reinforcement. The ultimate load-carrying capacity of the three decks was about 10 times greater than the service load prescribed by the AASHTO LRFD Design Specifications. The cracking load of the three specimens was more than twice the service load prescribed by the AASHTO LRFD Design Specifications. Using 33% less reinforcement did not change the serviceability behavior of the bridge decks. Based on these results, the authors recommended that design of reinforced concrete bridge decks can be based on a reinforcement yield strength = \((60/2/3) = 90\) ksi. Given the conservative nature of the test results, it is likely that design with yield strength of 100 ksi would also provide adequate punching shear strength.

DEVELOPMENT LENGTHS AND HOOK ANCHORAGES

The LRFD Design Specifications for bond and development lengths of reinforcement are empirically based and were developed for Grades 40, 60, and 75 bars. In NCHRP Project 12-77, proof tests of eight straight bar specimens and 18 hooked bar specimens were made assuming that the existing provisions for development lengths would be applicable with a yield strength of
100 ksi. The tests demonstrated that bar stresses of at least 125 ksi could be developed. However, to control bursting stresses, it is necessary to provide confining reinforcement based on current design requirements when ASTM A1035 bars are used. Existing equations for development length where no confinement was present were unconservative. The presence of confining reinforcement effectively mitigates potential splitting failures and results in suitably conservative strengths.

**SERVICEABILITY**

A fundamental issue in using ASTM A1035 reinforcement is that the stress level at service load will be higher than when conventional Grade 60 reinforcement is used. In many bridge applications, the stress at service load is assumed to be about 60% of the yield strength. Consequently, service load reinforcing strains are greater with ASTM A1035 reinforcement than with Grade 60 reinforcement. The large strains affect both deflection and crack widths at service loads.

**Deflection Predictions**

Instantaneous deflections of reinforced concrete flexural members are generally calculated using an effective moment of inertia in equations for elastic deflections. The effective moment of inertia takes into account the reduced stiffness caused by cracking of the concrete. The method used in the AASHTO Specifications was developed by Branson (1963).

Vertical deflections at midspan were measured during the testing of the six reinforced concrete beams used to verify flexural strength design. Following the initial flexural cracking, the load-deflection curves were essentially linear up to at least the assumed service stress level of 60 ksi. Comparisons of the measured deflections with calculated values using Branson's method showed very poor agreement because his method was developed for lower strength reinforcement. However, comparison of measured and calculated deflections using a computer program called Response 2000 provided adequate agreement.
Crack Control

In the AASHTO LRFD Design Specifications, crack widths are indirectly controlled through spacing limits for the longitudinal reinforcement. Two exposure conditions are considered. Class 1 corresponds to an assumed crack width of 0.017 in. Class 2 corresponds to a crack width of $0.75 \times 0.017 = 0.013$ in. However, the Authority having jurisdiction may decide on alternative crack widths if desired.

Extensive crack width data were collected during the flexural strength tests. The maximum and average crack widths corresponding to a variety of stresses in the reinforcement were then determined. The data showed that average crack widths at reinforcement stresses up to 72 ksi were all below the present AASHTO assumed limits of 0.017 and 0.013 in. for Class 1 and Class 2 exposures, respectively. Analysis of the data also showed that the measured crack widths were always less than the calculated crack widths for Class 1 exposure and measured and calculated crack widths were about the same for Class 2 exposure.

Serviceability Conclusions

Both deflections and crack widths were found to be within presently accepted limits and were predictable using the current specifications. Nevertheless, the researchers did recommend that the stress in the reinforcement be limited to a maximum of 60 ksi at service load levels.
PART 2: BRIDGES LOCATED IN AASHTO SEISMIC ZONES 3 AND 4
This part of the report specifically examines the applicability of the conclusions in Part 1 of the report to bridges located in AASHTO Seismic Zones 3 and 4. This part of the report is organized by the bridge component, such as decks, columns, and so on.

BRIDGE DECKS, GIRDERS, AND BENT CAP BEAMS

The following are reproduced from Memo to Designers (MTD) 20-1, issued by Caltrans in July 2010 (Caltrans 2010A).

Concrete Superstructure Design
All Ordinary Bridges shall be proportioned to direct inelastic damage into the columns, pier walls, and abutments. The superstructure shall have sufficient overstrength to remain essentially elastic when the bent reaches its most probable plastic moment capacity. The superstructure-to-substructure connection for non-integral caps may be designed to fuse prior to generating inelastic response in the superstructure.

The girders, bent caps, and columns shall be proportioned to minimize joint stresses. Moment resisting connections shall have sufficient joint shear capacity to transfer the maximum moments and shears, including overstrength demands without causing joint distress.

Section 3.4 of Caltrans’ *Seismic Design Criteria* (SDC), November 2010, Version 1.6 (Caltrans 2010b) requires:

*Capacity protected concrete elements such as footings, Type II pile shafts, bent cap beams, joints and superstructure shall be designed flexurally to remain essentially elastic when the column reaches its overstrength capacity.*

Even when a bridge is not an Ordinary Bridge by Caltrans definition, a designer will typically not deviate from the above principles. In view of this, there is absolutely no reason why the conclusions from Part 1 should not be applicable to bridge decks, girders, and bent cap beams.

Utah Department of Transportation Study

The following is excerpted from a recent report prepared for the Utah Department of Transportation Research Division (Barr and Wixom 2009):

_pressed by increases in construction costs coupled with insufficient funding for maintenance and new construction, many state and federal agencies are now specifying a service life of 75 years for all concrete bridges without significant repairs. In order to achieve this longer service life, better materials are required. MMFX Microcomposite Steel is a proprietary alloy that the company claims has greater corrosion resistance and structural properties which can achieve a service life of up to 75 years._
The research that has been performed thus far on the corrosive properties of MMFX Microcomposite Steel has demonstrated that it has a critical chloride threshold that is approximately four times higher than that of mild reinforcement (not epoxy-coated rebar).

Researchers have found that the rate of corrosion of MMFX is smaller (between one-third and two-thirds) of mild reinforcement. Some studies have shown that the corrosive rate increases over time.

Most stainless steel specimens tested performed better than MMFX Microcomposite Steel, but cost more likely due to their higher chromium contents.

While many of the rapid tests that have been performed for this research do allow for a quick evaluation that can be used to rate different types of steel, they do not provide a reliable correlation between short-term test results and in-situ results that are required to make accurate life-cycle costs.

In an effort to gain experience with the performance of MMFX for an in-service bridge, UDOT replaced the conventional epoxy-coated reinforcement in the US-6/White River Bridge with MMFX Microcomposite Steel. The contractor and employees noted that there was no additional labor associated with the placement of MMFX Steel in comparison to epoxy-coated reinforcement. However, MMFX steel does have a higher initial cost and there is some question in regards to its availability. As such the researchers made the following recommendations:

- For critical concrete bridge decks that are going to be exposed to large amounts of traffic and salting, UDOT should consider using MMFX steel or some other type clad or stainless steel rebar.
- UDOT should not use different types of steel for the top and bottom mats until more research is performed to insure that cracking does not occur at the bottom of the deck.

It should be noted that much of Utah, including the Salt Lake City area is in AASHTO Seismic Zone 3.

Kentucky Transportation Center Study

The following is the official abstract of a report prepared by the Kentucky Transportation Center in cooperation with the Kentucky Transportation Cabinet and the U.S. Department of Transportation, Federal Highway Administration (Chiaw and Harik 2006):

This report investigates the performance of bridge decks reinforced with stainless steel clad (SSC) and micro-composite multistructural formable steel (MMFX) rebars. The two-span Galloway Road Bridge on route CRS218 over North Elkhorn Creek in Scott County, KY, was reinforced with SSC rebars in one span and MMFX rebars in the second span. The reinforcements are intended to prolong the service life of the newly constructed bridge decks due to the expected corrosion-resistance capability.
Moment-curvature analyses indicated that MMFX RC decks had 57% and 85% higher strengths than SSC RC decks in positive and negative moment regions, respectively. The areas under the moment-curvature curves, a ductility indicator, of the MMFX RC decks, however, were 5% and 14% less than that of SSC RC decks in similar regions.

Field performance of the bridge decks was monitored beginning in August 2001, following its completion in July 2001. Field investigation consists of locating and measuring crack formation. As of September 2005, the cracks in the deck were not measurable since the maximum observed crack width was less than the smallest unit (e.g. 1/100 in.) on the crack comparator. This is also less than the maximum crack width of (0.013 in.) allowed by the AASHTO Standard for exterior exposure.

A review of the report indicated that the moment-curvature diagrams referred to in the second paragraph were “plotted based on the maximum usable AASHTO limiting strain of 0.003 in./in.”

We do not believe that the area under a moment-curvature diagram terminating where the extreme compression fiber strain in the concrete is 0.003 is a “ductility indicator.” The areas under the moment-curvature curves so calculated have little relevance to the applicability of MMFX reinforcement in bridge decks as discussed in this report.

Potential for Adverse Interaction when MMFX and Other Types of Reinforcing Bars are Placed in Close Proximity

Caltrans has expressed concern in the past about the possibility that the differences in electrical potential between different reinforcing bar types can actually accentuate and promote corrosion when such dissimilar reinforcing bars are in close proximity. The negative effect may be more pronounced in chloride-contaminated environments.

MMFX Corporation has noted that the electro-potential difference between MMFX and black steel is approximately 100 millivolts; the corresponding difference between stainless steel and black steel is well over 100 millivolts.

MMFX Corporation has had tests performed to determine if the differences in electrical potential between MMFX bars and other reinforcing bars may have a detrimental effect on performance. A report on this testing is available (Hartt 2009). The following are excerpted from that report.

Concern has been raised that contiguous placement of MMFX 2 (ASTM A1035) and black bar (BB) reinforcements in concrete exposed to chlorides could result in accelerated macro-cell corrosion of the latter metal (BB). This concern arises because of the higher chloride threshold concentration for MMFX2 compared to BB. Consequently, BB should initiate active corrosion before MMFX2; and this could result in an active-passive cell that accelerates corrosion of the BB. At the same time, macro-cell corrosion in an active-passive cell involving reinforcement in concrete is normally controlled by oxygen availability at the cathode; and on this basis, it can be argued that macro-cell current should be the same irrespective of whether BB or MMFX2 is serving as the cathode. The present testing program was performed to evaluate this situation.
from a corrosion standpoint and determine the extent to which, if any, macro-cell corrosion of a BB-MMFX2 combination in chloride contaminated concrete exceeds one where BB alone is present.

The experimental program consisted of eight concrete test block specimens. Each of these consisted of two overlapping but mutually electrically isolated (bars not in physical contact) lengths of No. 5 reinforcing bars. For four of the specimens, one bar was MMFX2 and the other BB; and for the remaining four, both bars were BB. The specimens were subjected to accelerated corrosion of the reinforcement. An electrical lead was attached to each exposed bar end to complete the circuit between bar pairs. Macro-cell current between the individual specimen bar pairs was calculated from the voltage drop measured across a resistor that was temporarily inserted into the circuit. Comparison of this macro-cell current for the BB only specimens with that of the MMFX-BB ones indicated no tendency for greater corrosion of the latter type specimen compared to the former. The following conclusion was drawn on the basis of the research:

The analysis of data from chloride exposure of concrete specimens reinforced with 1) black bars only and 2) both black and MMFX2 bars indicated that macro-cell corrosion was essentially the same in the two cases. On this basis, there is no technical reason why black bar and MMFX2 reinforcements cannot be combined in concrete construction, including situations involving, first, field repairs and, second, new construction.

Conclusion Concerning Bridge Decks, Girders, and Bent Cap Beams

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

COLUMNS

The AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Edition (AASHTO 2009) specifically states in Section 8.4.1:

*Use of high-strength high-alloy bars with an ultimate tensile strength of up to 250 ksi shall be permitted for longitudinal column reinforcement for seismic loading provided that it can be*
demonstrated through testing that the low-cycle fatigue properties are not inferior to normal reinforcing steel with yield strengths of 75 ksi or less.

Mander et al. (1994) studied the low-cycle fatigue behavior of reinforcing steel. ASTM A615 Grade 40 ordinary deformed steel reinforcing bars with a minimum specified yield strength of 40 ksi and ASTM A722 high-strength prestressing threaded bars with a specified ultimate strength of 157 ksi were experimentally evaluated for their low-cycle fatigue behavior under axial-strain-controlled reversed cyclic tests with strain amplitudes ranging from yield to 6%. All tests were performed on virgin (unmachined) specimens to closely simulate seismic behavior in structural concrete members. The study demonstrated that the modulus of toughness and low-cycle fatigue life for both the low- and high-strength materials were similar. Based on fatigue considerations, it was concluded that existing design codes were overly restrictive in not permitting the use of high-strength thread bars in seismic force-resisting elements.

Although MMFX reinforcing bars were not tested in the Mander study, it is difficult to see why the above conclusions would not be applicable to MMFX steel.

While low-cycle fatigue is an important consideration in seismic applications, so is overall member ductility – particularly, members of the seismic force-resisting system that are expected to undergo inelastic behavior in the design earthquake ground motion. Basically the only investigation available to guide the authors in this regard was reported by Stephan et al. (2003). In this University of California, San Diego investigation, two 35%-scale bridge column units were tested. The reinforced concrete columns were expected to form plastic hinges at the base during the design earthquake.

Unit 1 was built using conventional ASTM A706 Grade 60 reinforcement. Unit 2 was built using only high-strength MMFX reinforcement. Unit 2 was proportioned to give approximately the same flexural strength as the benchmark Unit 1.

The columns were 36 in. in diameter and were tested using a shear span of 9.5 ft. A concrete strength, $f'_c$, of 8 ksi was specified. The reinforcement for Unit 1 consisted of two cages, each containing 42 No. 5 bars with No. 3 hoops spaced at 1.56 in. on center. The longitudinal reinforcement ratio was 2.54% and the volumetric transverse steel ratio was 1.75%. Unit 2 incorporated only a single cage with 42 No. 5 MMFX longitudinal bars with No. 3 MMFX hoops spaced at 1.56 in. on center. The longitudinal reinforcement ratio for this column was 1.27% and
the volumetric transverse steel ratio was 0.85%. The hoops in both column models were butt-welded.

The test units were subject to quasi-static reversed cyclic loading, applied at 9.5 ft above the base. A constant axial load of $0.074A_g f'_c$ was applied to the specimens, where $A_g$ = gross area of the column. Figures 1 and 2 plot the hysteretic lateral force-drift responses for Units 1 and 2, respectively. In comparison with the conventionally reinforced Unit 1, the MMFX-reinforced Unit 2 showed a softer initial response, smaller displacement ductility capacity and smaller residual drifts. The following discussion from the report on the UCSD investigation (Stephan et al. 2003) is most useful.

![Figure 1: Hysteretic Response of Conventionally Reinforced Column Tested at UC San Diego](image)
The main implication of the limited ductility capacity observed for the MMFX unit is that components designed for earthquake resistance will have to be designed for lateral forces that are greater than those used currently in conventional designs [this implies design using lateral force reduction factors that are smaller than those used in the design of conventional components] ... The current design philosophy of ductile design gives significant weight to ... displacement ductility ... Little, if any, emphasis is currently given to the loss of performance due to residual drifts. However, newer trends in seismic design are moving more towards giving more emphasis to the impact that residual drifts can have, mainly to issues related to loss of function and reparability after moderate, but more common, seismic events. From this point of view the higher strength reinforcements could be encouraged in seismic design. An additional advantage of higher strength reinforcements used as transverse reinforcement is the effectiveness to restrain the longitudinal reinforcement and avoid buckling. This is due to the fact that due to the stress-strain characteristics of this type of steel, the hoops do not show unrestricted yielding that is characteristic of hoops made of conventional steel.

In view of the utter paucity of test results and in view of the above findings from the test results that are available, we recommend that, pending the availability of results from further comprehensive and systematic tests that would enable a possible adjustment of R-values, MMFX steel should not be used as longitudinal reinforcement in a bridge column in Seismic Zones 3 or 4. The use of MMFX steel as transverse reinforcement in such members is permitted, provided $f_y$ is restricted to no more than 60 ksi for the purposes of computing shear strength.

It should be noted that ACI 318-08 (ACI 2008) permits the use of transverse reinforcement with specified yield strength up to 100 ksi in special moment frames and special structural walls and coupling beams (special detailing is required in buildings assigned to Seismic Design Category...
D, E, or F, which, to revert to old terms, would roughly mean buildings located in Zones 3 and 4). The yield strength used in the design of shear reinforcement, however, is restricted to no more than 60 ksi, in order to avoid wide shear cracks.

The UC San Diego report cited above (Stephan et al. 2003) contained the following observation on Unit #2:

Degradation in the response began with fracture of a hoop at 125 mm above the base of the column. The hoop fractured in the heat affected region adjacent to the fuse weld... This led to the buckling of the column longitudinal bars in compression and to crushing of the concrete core. Further lateral displacement led to successive ductile fracturing of the longitudinal bars on the tension side of the column....

The above observation should be weighed against the background that butt-welded hoops of mild steel are specifically allowed even in plastic hinge regions of bridge columns by Caltrans’ *Seismic Design Criteria* (SDC), November 2010, Version 1.6 (Caltrans 2010b). Section 3.8.2 of that document entitled “Lateral Column Reinforcement Inside the Plastic Hinge Region” reads in part: “The lateral reinforcement shall be either butt-welded hoops or continuous spiral.” Out of abundant caution, we recommend that the welding of ASTM A1035 reinforcement should be prohibited in plastic hinge regions of columns and other structural members that are not capacity-protected in bridges located in AASHTO Seismic Zones 3 and 4, until approved procedures for butt-welding of hoops become available.

**Conclusions Concerning Bridge Columns**

Pending further testing, MMFX steel should not be used as longitudinal reinforcement in bridge columns in Seismic Zones 3 or 4.

The use of MMFX steel as transverse reinforcement in such members should be permitted, provided the transverse reinforcement yield strength is restricted to no more than 60 ksi for the purposes of computing shear strength. The full yield strength of the transverse reinforcement may be utilized for purposes of confinement of the concrete.

The welding of ASTM A1035 reinforcement should be prohibited in plastic hinge regions of columns and other structural members that are not capacity-protected in bridges located in AASHTO Seismic Zones 3 and 4, until approved procedures for butt-welding of hoops become available.
The above recommendation should also apply to pier walls, back walls, and wing walls, which are preferable locations for inelastic behavior in most bridges.

FOUNDATIONS AND OTHER CAPACITY-PROTECTED ELEMENTS

The following are reproduced from Memo to Designers (MTD) 20-1, issued by CalTrans in July 2010 (Caltrans 2010A).

Pre-Determined Locations of Damage

Inelastic behavior shall be limited to pre-determined locations within the bridge that can be easily inspected and repaired following an earthquake. Continuous column/pile shaft combinations are an exception since inelastic behavior may occur below ground. Preferable locations for inelastic behavior on most bridges include columns, pier walls, back walls, wing walls, seismic isolation and damping devices, bearings, shear keys and steel end diaphragms. Significant inelastic response in concrete superstructures is not desirable because of the potential to jeopardize public safety. Furthermore, superstructure damage in continuous bridges is difficult to repair to a serviceable condition.

Capacity Protected Design

Bridges shall be designed with ductile members to attract seismic energy and form successful plastic hinges. All other elements shall be “capacity protected” such that they remain essentially elastic. An appropriate margin of safety (referred to as “overstrength”) shall be used for capacity protected elements to ensure that fusing occurs in the ductile elements. Desired locations of plastic hinging shall be identified and detailed for ductile response. A large enough overstrength factor shall be provided to ensure the desired yielding mechanism occurs and non-ductile failure mechanisms such as concrete crushing, shear cracking, elastic buckling, and fracture are prevented. Capacity protected members shall also have some ductility to provide insurance against the unexpected propagation of damage.

Essentially Elastic Behavior

Components not explicitly designed as ductile or sacrificial shall be designed as capacity protected components that remain essentially elastic under seismic loads. The effects of the inelastic response in capacity-protected components shall not diminish the bridge’s ability to meet its specified performance criteria and shall not prevent the bridge from eventually being repaired and restored to normal service conditions. The inelastic response of capacity protected concrete components shall be limited to minor cracking and/or incremental material strains that will not significantly diminish the component’s stiffness. The force demands in capacity-protected concrete components shall not exceed the seismic capacity limits identified in the Caltrans SDC.

Foundations

Bridge foundations in competent soil shall be designed to remain essentially elastic when resisting the plastic hinging moments, associated shears, and axial force at the base of columns and piers with two exceptions. Pile shaft foundations are permitted to respond inelastically if they are designed and detailed in a ductile manner. Also, the pile foundations for pier walls cannot be economically designed to resist the transverse seismic shear elastically. However, the designer should attempt to minimize the inelastic response in pier wall foundations, and shall verify global stability is maintained under the anticipated seismic demand. For bridge foundations in soft or liquefiable soil, designing the piles to remain essentially elastic may be uneconomical due to the excessive demand imposed on the piles. In that case plastic hinging of the pile at the fixed connection to the footing, to a maximum displacement ductility of 2.5, may be allowed. However, the formation of a second hinge in the piles shall not be allowed.

Caltrans’ Seismic Design Criteria (SDC), November 2010, Version 1.6 (Caltrans 2010B) has defined Type I and Type II pile shafts.
Type I shafts are designed so the plastic hinge will form below ground in the shaft. The concrete cover and area of transverse and longitudinal reinforcement may change between the column and Type I shaft, but the cross section of the confined core is the same for both the column and the shaft.

Type II shafts are designed so the plastic hinge will form at or above the shaft/column interface, thereby, containing the majority of inelastic action to the ductile column element. Type II shafts are usually enlarged shafts characterized by a reinforcing cage in the shaft that has a core diameter larger than that of the column it supports. Type II shafts shall be designed to remain elastic.

SDC Section 3.4 requires:

*Capacity protected concrete elements such as footings, Type II pile shafts, bent cap beams, joints and superstructure shall be designed flexurally to remain essentially elastic when the column reaches its overstrength capacity.*

**Conclusions Concerning Foundation Elements**

In general, MMFX reinforcing bars should be permitted to be used in any foundation element such as a footing or a Type II pile shaft in Seismic Zones 3 and 4 that is capacity protected, provided the guidelines in Part 1 of this report are followed.

When MMFX reinforcing bars are used as transverse reinforcement in pile shafts, the transverse reinforcement yield strength should be restricted to no more than 60 ksi for the purposes of computing shear strength. The full yield strength of the transverse reinforcement may be utilized for purposes of confinement of the concrete.

**ACTUAL APPLICATIONS OF MMFX REINFORCEMENT IN BRIDGES IN AASHTO ZONES 3 AND 4**

MMFX Technologies Corporation has provided the authors with two example applications briefly described in Appendix B.
OVERALL CONCLUSIONS (PARTS 1 and 2)

Based on the information presented in this report, Table 1 lists the maximum strengths of reinforcement that may be used in the design of the different structural elements of bridges with the modifications described Part 1 and summarized in Appendix A. The remaining conclusions specifically concern applications in Seismic Zones 3 and 4. The application of high-strength reinforcing steel in bridges located in Seismic Zone 2 was beyond the scope of this study.

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

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

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

The welding of ASTM A1035 reinforcement should be prohibited in plastic hinge regions of columns and other structural members that are not capacity-protected in bridges located in AASHTO Seismic Zones 3 and 4, until approved procedures for butt-welding of hoops become available.

In general, MMFX reinforcing bars should be permitted to be used in any foundation element such as a footing or a Type II pile shaft in Seismic Zones 3 and 4 that is capacity protected, provided the guidelines in Part 1 of this report are followed.

When MMFX reinforcing bars are used as transverse reinforcement in pile shafts, the transverse reinforcement yield strength should be restricted to no more than 60 ksi for the purposes of
computing shear strength. The full yield strength of the transverse reinforcement may be utilized for purposes of confinement of the concrete.

Table 1 Maximum Tensile Strengths of Reinforcement for Use in Design

<table>
<thead>
<tr>
<th>Yield Strength, ksi</th>
<th>Foundations</th>
<th>Columns/Walls</th>
<th>Decks</th>
<th>Beams/Girders</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Abutments</td>
<td>Piles</td>
<td>Pile Caps</td>
<td>Vertical</td>
</tr>
<tr>
<td>Non-Seismic (Zone 1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>75</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismic (Zones 3 and 4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>N(^{(2)})</td>
</tr>
<tr>
<td>75</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>X(^{(3)})</td>
<td>X(^{(3)})</td>
<td>X(^{(3)})</td>
<td>N(^{(2)})</td>
</tr>
</tbody>
</table>

(1) Yield strength limited to 60 ksi for shear-friction calculations.
(2) Not recommended.
(3) Yield strength of transverse reinforcement limited to 60 ksi for shear strength computations.

Note: Application of high-strength reinforcement in bridges located in Seismic Zone 2 was beyond the scope of this study.

REFERENCES

Referenced Standards

ASTM A706, Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement.

ASTM A1035, Standard Specification for Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement.

Cited References


ACI, 2008, ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)," American Concrete Institute, Farmington Hills, MI, 473 pp.


# APPENDIX A

## Table A1 Summary of Proposed Changes to Section 5 of the AASHTO LRFD Bridge Design Specifications Based on NCHRP Project 12-77.

<table>
<thead>
<tr>
<th>Article</th>
<th>Summary of Proposed Changes</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.2 DEFINITIONS</td>
<td>Modify the definition of tension-controlled section by changing “0.005” to “tension-controlled strain limit.” Add a definition of tension-controlled strain limit.</td>
</tr>
<tr>
<td>5.3 NOTATION</td>
<td>Modify the definition of $f_y$ to allow higher yield strengths. Add definitions of $\varepsilon_{cl}$ and $\varepsilon_{tl}$, compression- and tension-controlled strain limits, respectively.</td>
</tr>
<tr>
<td>5.4.3.1 and C5.4.3.1 Reinforcing Steel, General</td>
<td>Permit the use of reinforcing steel with specified yield strengths up to 100.0 ksi when allowed by specific articles.</td>
</tr>
<tr>
<td>5.4.3.2 Reinforcing Steel, Modulus of Elasticity</td>
<td>$E_s = 29,000$ ksi may be used for specified yield strengths up to 100.0 ksi.</td>
</tr>
<tr>
<td>5.4.3.3 and C5.4.3.3 Reinforcing Steel, Special Applications</td>
<td>Permit the use of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1.</td>
</tr>
<tr>
<td>5.5.3.2 and C5.5.3.2 Fatigue Limit State, Reinforcing Bars</td>
<td>Modify the fatigue equation for reinforcing bars to allow the equation to be used for specified yield strengths up to 100.0 ksi.</td>
</tr>
<tr>
<td>5.5.4.2.1 and C5.5.4.2.1 Resistance factors, Conventional Construction</td>
<td>Allow the use of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1. Modify the equation, figure, and commentary for $\phi$. These will use $\varepsilon_{cl}$ and $\varepsilon_{tl}$, (compression- and tension-controlled strain limits) in place of 0.002 and 0.005.</td>
</tr>
<tr>
<td>5.7 and adds C5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS</td>
<td>Allow the use of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1.</td>
</tr>
<tr>
<td>5.7.2.1 and C5.7.2.1 Assumptions for Strength and Extreme Event Limit States</td>
<td>Keep compression- and tension- controlled strain limits of 0.002 and 0.005, respectively, for reinforcing steels with specified yield strengths up to 60.0 and 75.0 ksi. Provide compression- and tension-controlled strain limits of 0.004 and 0.008 for reinforcing steel with a specified yield strength equal to 100.0 ksi. Linear interpolation is used for reinforcing steels with specified yield strengths between 60.0 or 75.0 ksi and 100.0 ksi. Equations are provided for when $f_y$ may replace $f_s$ or $f_s'$ in 5.7.3.1 and 5.7.3.2.</td>
</tr>
<tr>
<td>5.7.3.2.5 Strain Compatibility Approach</td>
<td>Limit the steel stress in a strain compatibility calculation to the specified yield strength.</td>
</tr>
<tr>
<td>C5.7.3.3.1 Maximum Reinforcement</td>
<td>Replace 0.005 with “tension-controlled strain limit.”</td>
</tr>
<tr>
<td>5.7.3.5 and C5.7.3.5 Moment Redistribution</td>
<td>Adjust strain limit to allow moment redistribution in structures using reinforcing steel with specified yield strengths up to 100.0 ksi.</td>
</tr>
<tr>
<td>C5.7.4.2 and C5.7.4.4. Limits for Reinforcement</td>
<td>Warn that designs should consider that columns using higher strength reinforcing steel may be smaller and have lower axial stiffness.</td>
</tr>
<tr>
<td>Section</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>5.7.4.6</td>
<td>Spirals and Ties</td>
</tr>
<tr>
<td>5.8.2.4</td>
<td>and C5.8.2.4 Regions Requiring Transverse Reinforcement</td>
</tr>
<tr>
<td>5.8.2.5</td>
<td>and C5.8.2.5 Minimum Transverse Reinforcement</td>
</tr>
<tr>
<td>C5.8.2.7</td>
<td>Maximum Spacing of Transverse Reinforcement</td>
</tr>
<tr>
<td>5.8.2.8</td>
<td>and C5.8.2.8 Design and Detailing Requirements.</td>
</tr>
<tr>
<td>C5.8.3.3</td>
<td>Nominal Shear Resistance</td>
</tr>
<tr>
<td>5.8.3.5</td>
<td>Longitudinal Reinforcement</td>
</tr>
<tr>
<td>5.8.4.1</td>
<td>Interface Shear Transfer, General</td>
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<tr>
<td>5.10.2</td>
<td>and C5.10.2 Hooks and Bends</td>
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<tr>
<td>5.10.6.1</td>
<td>and C5.10.6.1 Transverse Reinforcement for Compression Members, General</td>
</tr>
<tr>
<td>5.10.11.1</td>
<td>Provisions for Seismic Design, General</td>
</tr>
<tr>
<td>5.11.1.1</td>
<td>and C5.11.1.1 DEVELOPMENT AND SPLICES OF REINFORCEMENT, Basic Requirements</td>
</tr>
<tr>
<td>5.11.2</td>
<td>and C5.11.2 Development of Reinforcement</td>
</tr>
<tr>
<td>5.11.2.1</td>
<td>Deformed Bar and Wire in Tension</td>
</tr>
<tr>
<td>5.11.5</td>
<td>and adds C5.11.5 Splices of Bar Reinforcement</td>
</tr>
<tr>
<td>5.11.5.3</td>
<td>and C5.11.5.3 Splices of Reinforcement in Tension</td>
</tr>
<tr>
<td>Table 5.11.5.3.1-1 Classes of Tension Lap Splices</td>
<td>Require transverse confining steel in splices of reinforcing steel with specified yield strengths exceeding 75.0 ksi.</td>
</tr>
</tbody>
</table>

**Note:** The full details of these proposed changes are provided in NCHRP Report 679 (Shahrooz et al. 2011).
APPENDIX B

Project Name: **DAGGETT ROAD BRIDGE**
Location: Daggett Road at Burns Cut-off - Stockton, CA
Construction Date: 2006

Owner: Port of Stockton
Stockton, CA

Engineer: DMJM Harris / AECOM
Sacramento, CA

Project Description: New 4 lane Daggett Road Bridge over Burns Cut-off for Port of Stockton, California

Material Use/ Design Criteria: MMFX 2 (ASTM A1035) bar used as negative moment compression girder reinforcement at a 100 ksi yield design strength.

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Project Name: **LIGHT RAIL TRANSIT SYSTEM - ALDER CREEK BRIDGE**
Location: Alder Creek - Folsom, CA
Construction Date: 2004

Owner: Sacramento Regional Transit Authority
Sacramento, CA

Engineer: DMJM Harris / AECOM
Sacramento, CA

Project Description: New light rail transit system bridge, which connects Sacramento to north east suburbs.

Material Use/ Design Criteria: MMFX 2 (ASTM A1035) bar used in girder, columns and columns confinement, bridge abutment and column foundation reinforcement designed for 100 ksi yield strength.