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16. Abstract TxDOT Project 0-6348 "Controlling Cracking in Prestressed Concrete Panels and Optimizing Bridge Deck Reinforcing Steel" started on September 1, 2008 and is scheduled to end on August 31, 2012. The project is proceeding on schedule. This report summarizes research progress to date, and lists the principal findings.		
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PCP Cracking and Bridge Deck Reinforcement: An Interim Report

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Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.

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The authors express appreciation to the TxDOT Project Director, Manuel (Bernie) Carrasco, P.E. (Bridge Division). We also acknowledge the contributions of the Project Advisors: Graham Bettis, P.E. (Construction Division); Robert Cochrane, P.E. (Bryan District); David Hohmann, P.E. (Bridge Division); John Holt, PE (Bridge Division); Kirk Krause (Waco District); John Vogel, P.E. (Houston District); and Wade Odell, P.E. (Research and Technology Implementation Office). We are grateful to the management and personnel of the precast plants who furnished panels for field studies, and to the laboratory personnel at Ferguson Lab (UT Austin) and UT Arlington.

EXECUTIVE SUMMARY

Texas Department of Transportation (TxDOT) Project 0-6348, "Controlling Cracking in Prestressed Concrete Panels and Optimizing Bridge Deck Reinforcing Steel," started on September 1, 2008 and is scheduled to end on August 31, 2012. The project is proceeding on schedule. This report summarizes research progress to date and lists the following principal findings:

1) TxDOT specifications for precast, prestressed panels currently require an initial prestress of 16.1 kips per 3/8-in. strand. Because measured prestress losses are less than half those assumed in design, test data acquired in the study indicate that the initial prestress could be reduced to 14.4 kips per 3/8-in. strand. Such a reduction would lessen stresses that lead to collinear panel cracks (cracks that form along prestressing strands).
2) The propagation of cracks that form along the prestressing strands can be controlled by supplementary reinforcing bars placed at the panel edge and oriented perpendicular to the prestressing strands
3) Design of top-mat reinforcement is governed primarily by requirements for control of crack widths. Currently, required longitudinal reinforcement is already at the minimum amount necessary to control crack widths and probably cannot be reduced. Transverse reinforcement, in contrast, might be reduced. Different ways of doing this, including welded-wire reinforcement, are being checked with a field study at a TxDOT bridge near Waco.
4) High-performance steel fibers might replace conventional reinforcement under some conditions. To use these fibers effectively, it is necessary to have reliable and cost-effective tests for evaluating the stress-strain behavior of concrete reinforced with high-performance steel fibers. The "double-punch" test shows promise for this. Standard protocols for performing it have been developed by study researchers, and are being evaluated for reliability.

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Chapter 1. INTRODUCTION

1.1 Background of Project 0-6348

Texas Department of Transportation (TxDOT) Project 0-6348, "Controlling Cracking in Prestressed Concrete Panels and Optimizing Bridge Deck Reinforcing Steel," began September 1, 2008 and is scheduled to end August 31, 2012. The project is proceeding on schedule.

1.1.1 Objectives of Project 0-6348

The objectives of Project 0-6348 are as follows. Each objective is discussed further in subsequent sections of this report.

1) Identify ways of controlling cracking in precast, prestressed bridge deck panels; and
2) Optimize reinforcement in the cast-in-place concrete placed on bridge decks.

1.1.2 Research Team for Project 0-6348

As shown in Table 1.1, the research team for Project 0-6348 consists of Profs. Richard Klingner, Oguzhan Bayrak, and James Jirsa from The University of Texas at Austin (Center for Transportation Research) and Prof. Shih-ho (Simon) Chao from the University of Texas at Arlington.

Table 1.1: Research Team for Project 0-6348

Name	Agency	Duty
Richard E. Klingner	UT Austin / CTR	Research Supervisor
Oguzhan Bayrak.	UT Austin / CTR	Researcher
James O. Jirsa	UT Austin / CTR	Researcher
Shih-Ho (Simon) Chao	UT Arlington	Researcher

1.1.3 Project Monitoring Committee (PMC) for Project 0-6348

The Project Monitoring Committee (PMC) for Project 0-6348 is shown in Table 1.2.

Table 1.2: Project Monitoring Committee (PMC) for Project 0-6348

Name	Agency	Duty
Manuel (Bernie) Carrasco, PE	Bridge Division (BRG)	Project Director
Graham Bettis, PE	Construction Division (CST)	Project Advisor
Robert Cochrane, PE	Bryan District (BRY)	Project Advisor
David Hohmann, PE	Bridge Division (BRG)	Project Advisor
John Holt, PE	Bridge Division (BRG)	Project Advisor
Kirk Krause	Waco District (WAC)	Project Advisor
John Vogel, PE	Houston District (HOU)	Project Advisor
Wade Odell, PE	Research Technology, and Implementation Office (RTI)	Research Engineer

1.2 Purpose and Organization of this Report

This report is intended to summarize the findings of the research team in each area of Project 0-6348, and to confirm the intended direction of the study.

For clarity of presentation, this report is not organized around the specific research tasks of the project contract. Rather, it is organized around the two main technical areas of the research. Principal actions and findings to date in each area are presented.

Chapter 2. TECHNICAL BACKGROUND OF PROJECT 0-6348

The dominant form of TxDOT bridge construction involves precast, prestressed deck panels, spanning transversely between prestressed concrete girders, and which work compositively with CIP concrete to form the bridge deck. The technical background of Project 0-6348 is related to the control of cracking in the precast, prestressed deck panels, and to the optimization of reinforcement in the cast-in-place concrete that is subsequently placed on the panels.

In this section, the technical background of each area is reviewed, and the technical organization of Project 0-6348 is summarized. Details of the technical background are provided in the MS theses of Foreman (2010) and Foster (2010).

2.1 Control of Cracking in Precast, Prestressed Deck Panels

According to the original project statement, about 200,000 square feet of deck panels are rejected by TxDOT every year. Generally, this is due to cracking of the panels along the strands, due to a combination of tensile stresses from release, handling at the precast yard, transportation to the job site, and handling at the job site. As shown in Figure 2.1, circumferential tensile stresses act in the concrete around the strands as a consequence of the bond stresses that are produced along the strands when their initial prestress is released.

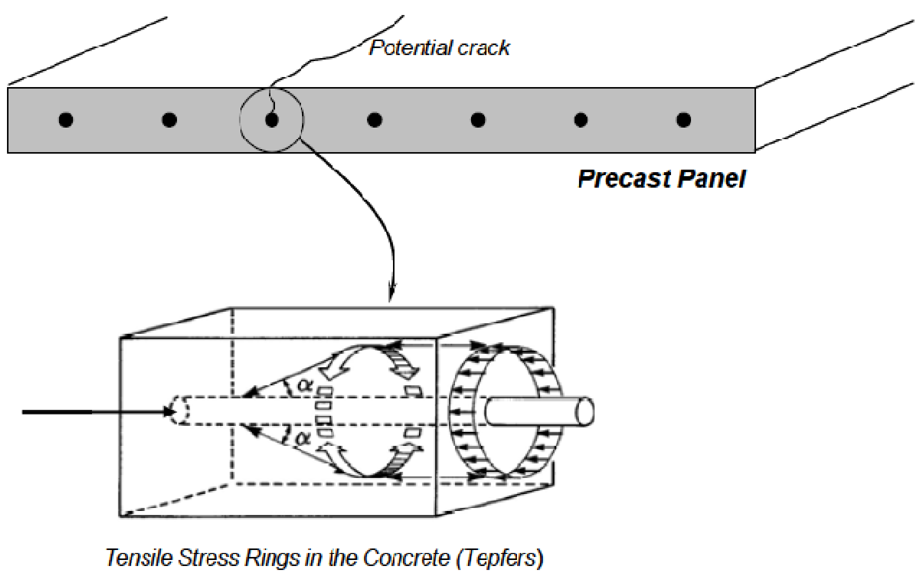

Figure 2.1: Mechanism of Longitudinal Cracking along Prestressing Strands Due to Circumferential Tension around Strands

If these circumferential tensile stresses exceed the tensile strength of the concrete, cracks can form along the strands, and propagate to the surface of the panels. This type of cracking is referred to as "collinear cracking." Collinear cracking can be reduced by reducing the level of initial prestress, or by increasing the level of circumferential tensile stress necessary to produce a surface crack. One way to accomplish the latter is by placing reinforcement perpendicular to the strands. Prior to cracking, that transverse reinforcement resists some of the circumferential tensile stress, and thereby reduces the tensile stress in the concrete around the strand. After local cracking around the strand, that transverse reinforcement controls the opening of cracks and resists their propagation to the surface of the panel.

Those circumferential tensile stresses are increased by flexural tension from bending of the panels about axes parallel to the strands. Consequently, surface cracking parallel to the strands can also be reduced by controlling bending of the panels during handling at the precast yard, transportation to the job site, and handling at the job site. Those additional bending stresses are not addressed further here.

2.2 Optimization of Top Mat Reinforcement

Reinforced concrete bridge decks are generally designed as one-way slabs spanning transversely (between girders). As shown in Figure 2.2, in-plane lateral restraint produces membrane compression ("arching action") in a cracked, reinforced concrete bridge deck. That compression increases the flexural capacity of the deck. As a consequence, the flexural reinforcement required to resist typical vehicle loading is quite small, and requirements for transverse and longitudinal flexural reinforcement are generally governed by requirements for shrinkage and temperature reinforcement. One important exception is transverse reinforcement in cantilever overhangs, because moments are determined by statics and transverse membrane action cannot exist.

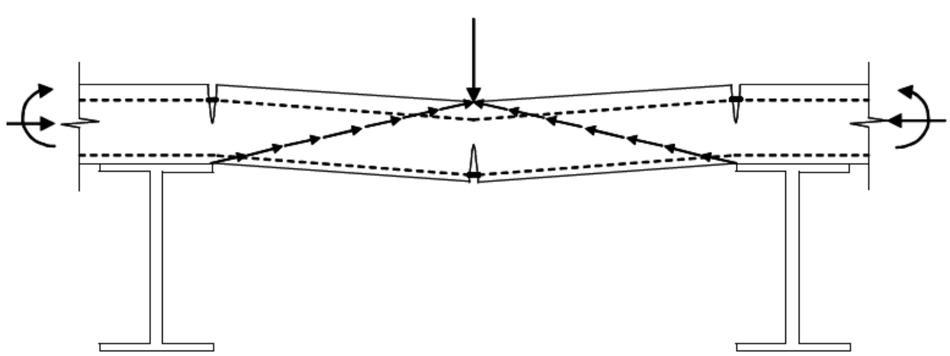

Figure 2.2: Arching Action in Transversely Spanning Bridge Decks

Given that the requirements for transverse and longitudinal flexural reinforcement are largely governed by requirements for shrinkage and temperature reinforcement, the research group has focused on ways of optimizing cast-in-place deck reinforcement to meet these requirements.

2.3 Possible Use of High-Performance Steel Fibers in Concrete

One other aspect of Project 0-6348 deals with high-performance steel fibers, examples of which are shown in Figure 2.3. The red ellipses in the figure indicate the end deformations that are characteristic of high-performance fibers, which increase the mechanical bond between the fiber and the concrete, and which distinguish them from conventional straight fibers.

(a) Type 1 (Royal, single-bend hook)

(b) Type 2 (Bekaert, double-bend hook)

(c) Type 3 (Bekaert, double-bend hook)

Figure 2.3: Examples of High-Performance Steel Fibers

The general behavior of concrete reinforced with high-performance steel fibers has been studied by TxDOT, and is not discussed further here. The particular aspect of that behavior addressed in Project 0-6348 is the development of test methods that will permit comparison of the performance of fibers produced outside of the U.S. (for example, by Bekaert) with fibers produced inside the U.S. (for example, by Royal).

2.4 Technical Organization of Project 0-6348

The technical organization of Project 0-6348 is shown in Figure 2.4. Under the general direction of Prof. Klingner, one group (headed by Prof. Bayrak) is investigating ways to control panel cracking; another group (headed by Prof. Jirsa) is investigating ways to optimize top-mat reinforcement; and a third group (headed by Prof. Chao) is investigating possible ways to use high-performance fibers in concrete.

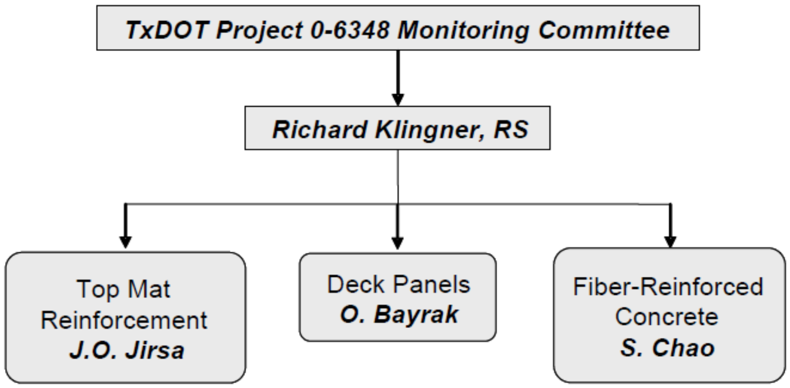

Figure 2.4: Primary Responsibilities of Research Team for Project 0-6348

Chapter 3. WORK TO DATE AND PRINCIPAL FINDINGS IN PROJECT 0-6348

Figure 3.1 shows the organization of work for Project 0-6348. In the following sections, each aspect of the work is summarized, and the principal findings of each aspect are presented.

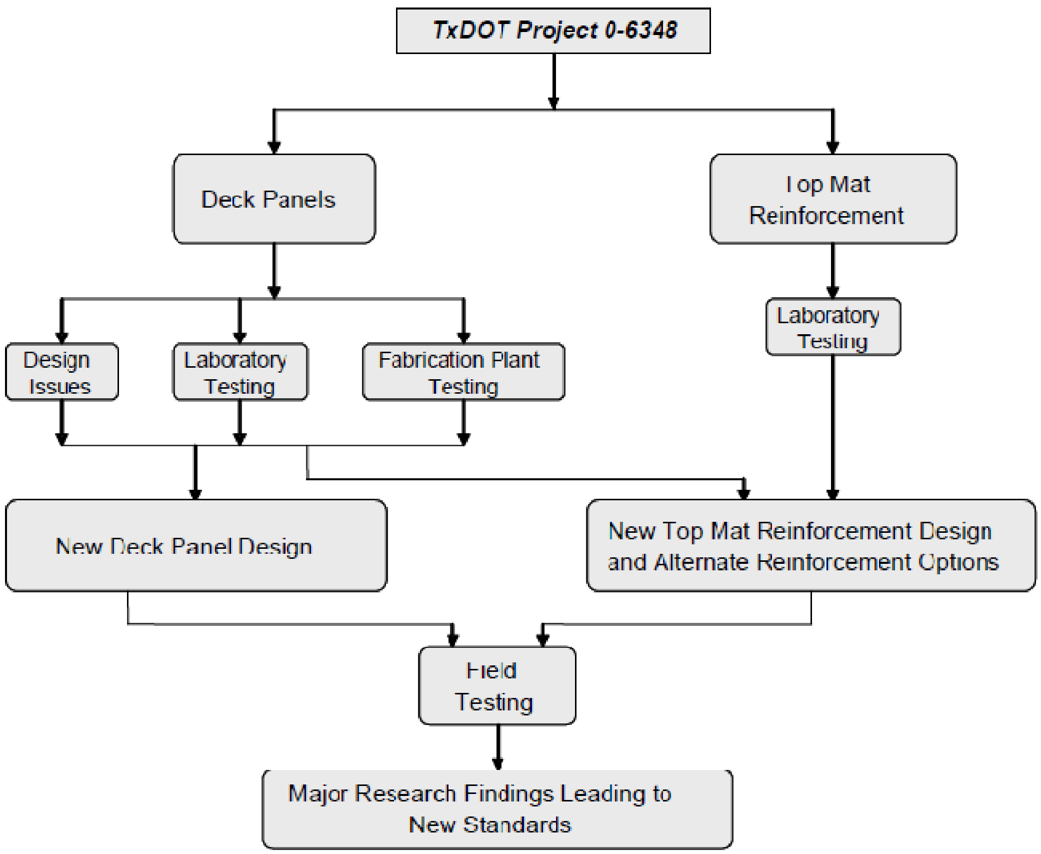

Figure 3.1: Organization of Work for Project 0-6348

3.1 Work to Date on Controlling Cracking in Precast, Prestressed Deck Panels

As explained in Section 2.1, cracking in precast, prestressed deck panels can be controlled by reducing the initial prestress, by placing supplementary reinforcement perpendicular to the prestressing strands, or by reducing flexural tension from handling, transport, and storage. The last of these is not addressed further here.

The initial prestress is determined by the value of final prestress required to resist the design loads on the panels, plus the expected prestress losses. The latter are calculated using a conservative AASHTO lump-sum procedure based on beam testing (Foreman 2010), but had not been verified by panel testing. Therefore, one aspect of Project 0-6348 involves experimental determination of prestress losses in panels cast in two different plants.

The second part of Project 0-6348 related to panel cracking involves a study of the effectiveness of supplementary transverse reinforcement in controlling the propagation of cracks to the surface of panels. The last part of Project 0-6348 related to panel cracking involves an experimental study of the relationship between panel cracking and prestress loss.

3.1.1 Quantify Probable Prestress Losses in Precast, Prestressed Deck Panels

Using a conservative lump-sum procedure permitted by AASHTO, TxDOT calculates prestress losses in panels as 45 ksi, and sets the required initial prestress at 16.1 kips per 3/8-in. strand. In February 2009, a set of six precast, prestressed panels was fabricated at Plant A, using that initial prestress (Figure 3.2).

Figure 3.2: Casting Instrumented Panels at Plant A, February 2009

Using embedded strain gages and vibrating-wire gages in the concrete, the level of prestress in the panels was monitored from the time the panels were cast, through release, and then through handling, transport, and long-term storage at the Phil M. Ferguson Structural Engineering Laboratory (UT Austin). The results are shown in Figure 3.3, and are compared with the value used by TxDOT (conservative AASHTO procedure), and also with the values predicted by more complex procedures of the 2004 and 2008 editions of AASHTO. In that figure, the blue and green curves (C1 and C2, respectively) refer to panels with the current configuration of reinforcement, while the black and red curves (M1 and M2, respectively) refer to panels with a modified configuration with supplemental transverse reinforcement. Measured prestress losses are less than half those predicted by the current TxDOT formula, and also considerably less than those predicted by more complex AASHTO procedures. Similar results were obtained from a second set of panels, fabricated in February 2010 at Plant B, using that same value of initial prestress. Instead of the 45-ksi prestress loss now assumed by TxDOT, losses are actually less than 25 ksi.

For the specimens stressed at 16.1 kips, circumferential tensile stresses perpendicular to the strands were also measured, and were found to be close to the probable tensile strength of the concrete used in the panels. These results, combined with the prestress-loss results discussed above, indicate that initial prestress can safely be reduced from 16.1 kips to 14.4 kips, and that this reduction would also reduce collinear panel cracking.

Based on this, Project 0-6348 investigators are studying the performance of panels with a lower level of initial prestress. Using an initial prestress of 14.4 kips (rather than the currently required 16.1 kips), another set of panels was fabricated at Plant A in July 2010 and at Plant B in September 2010. Those panels are now being monitored.

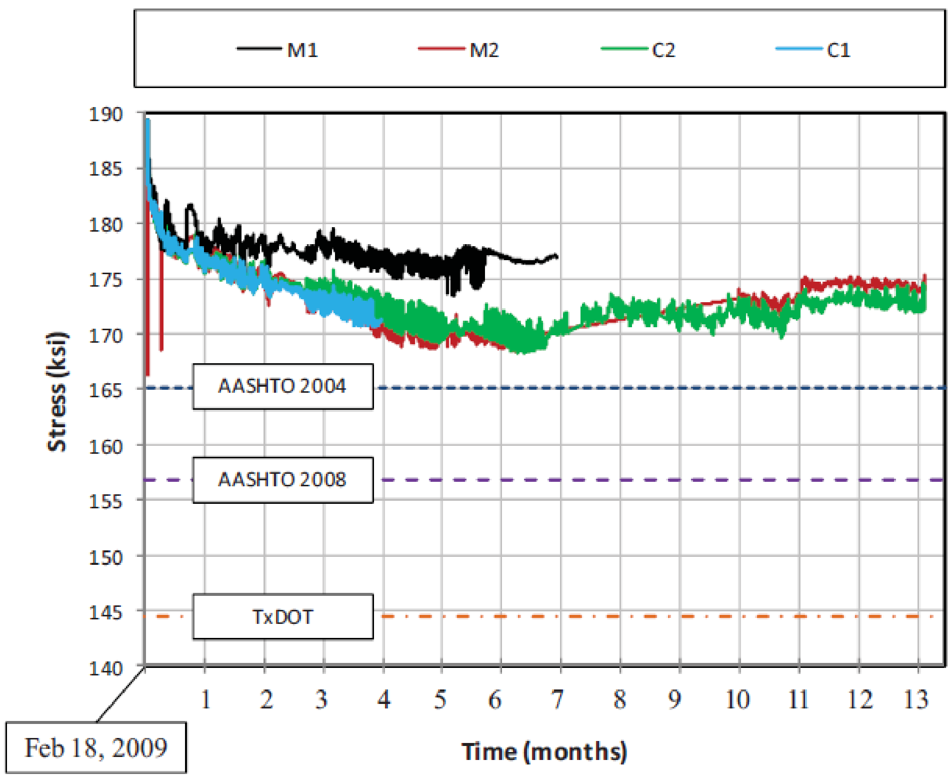

Figure 3.3: Prestress Losses in Panels Cast at Plant A in February 2009 (C1 and C2 are for current panels; M1 and M2 are for modified panels)

3.1.2 Investigate the Use of Supplemental Reinforcement in Precast, Prestressed Deck Panels

Each of the four sets of panels discussed above (Plant A in February 2009 and July 2010, and Plant B in February 2010 and September 2010) included panels with current transverse reinforcement, and panels with supplementary transverse reinforcement (#3 bars) near the ends of the prestressing strands (Figure 3.4 and Figure 3.5).

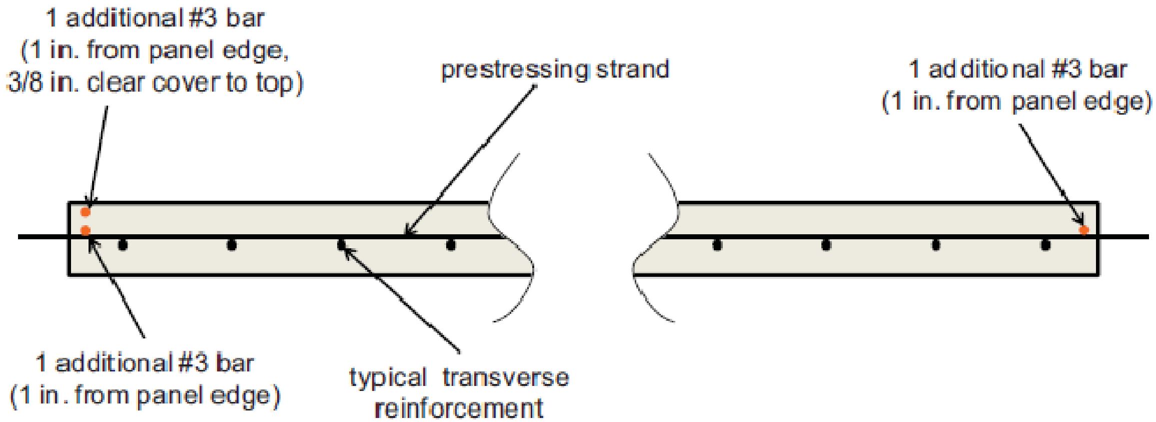

Figure 3.4: Section View of Layout of Reinforcement in Panels M3, M4, and M5 (Modifications Shown in Orange)

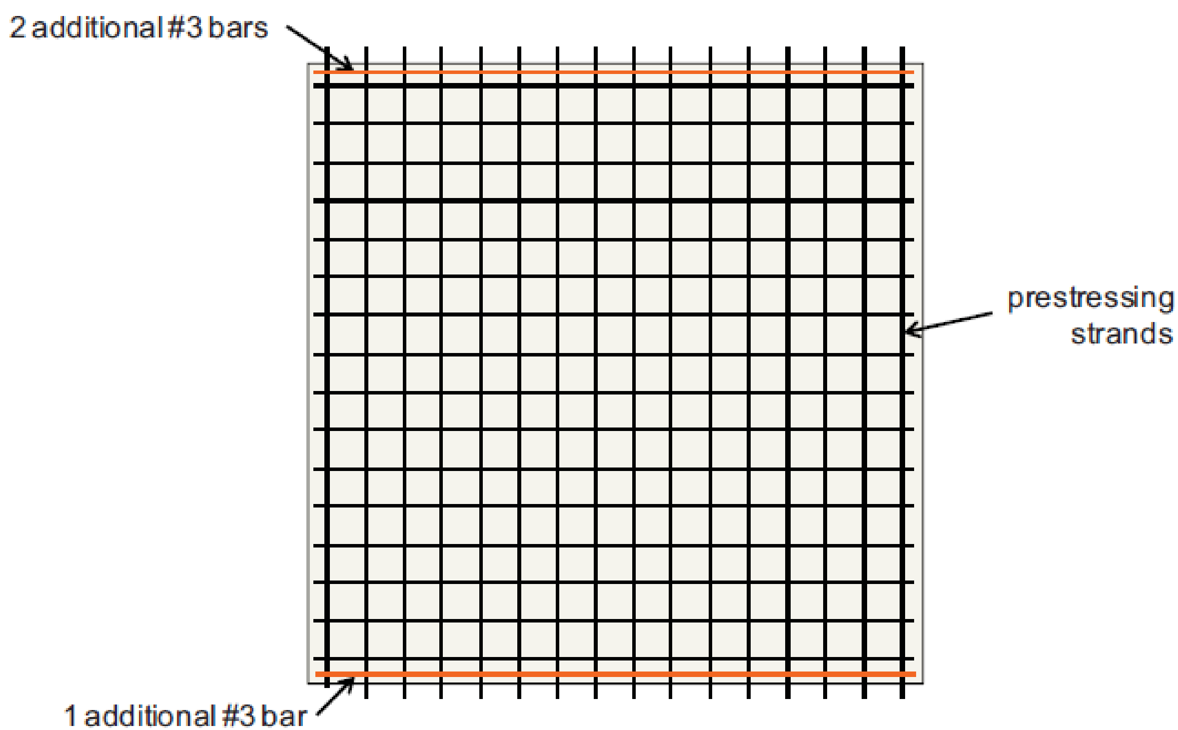

Figure 3.5: Plan View of Layout of Reinforcement in Panels M3, M4, and M5 (Modifications Shown in Orange)

Prestress losses in the modified panels with supplemental transverse reinforcement were no greater than, and were sometimes less than, prestress losses in panels with conventional transverse reinforcement. This suggests that supplementary transverse reinforcement near panel ends could be useful in controlling collinear cracking. This is examined further in the next section.

3.1.3 Quantify the Effects of Cracking on Precast, Prestressed Deck Panels

The next aspect of Project 0-6348 was to examine and quantify the effects of panel cracking on the behavior of precast, prestressed deck panels. Using the sets of panels that had been fabricated at Plants A and B, the specific objective was to see how prestress loss might increase with increasing width of surface cracks.

Because only one crack, hairline in width, formed in the first set of panels fabricated at Plant A, it was not useful to study prestress losses using existing cracks. It was necessary to generate and widen collinear cracks by supporting panels on a knife edge parallel to the strands, and bending the panels about that knife edge as shown in Figure 3.6.

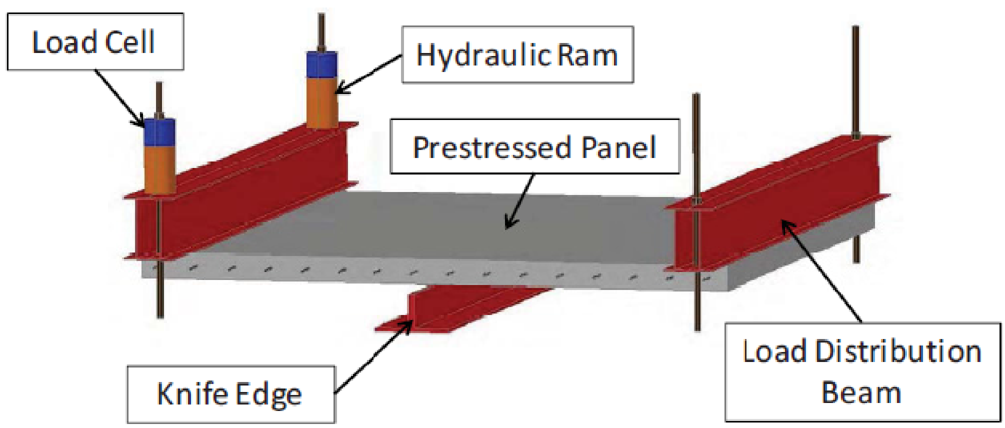

Figure 3.6: Setup with Knife-Edge Support, used to Generate Collinear Cracks and Measure Prestress Loss

Figure 3.7 shows prestress losses versus surface crack width for the current panel reinforcement configuration (solid blue lines, denoted by “C”) and for the modified reinforcement configuration with supplemental transverse reinforcement (dashed red lines, denoted by “M”). The loss in prestress is small even for very wide cracks. Supplementary transverse reinforcement is effective in controlling surface crack width, and therefore reduces prestress loss somewhat.

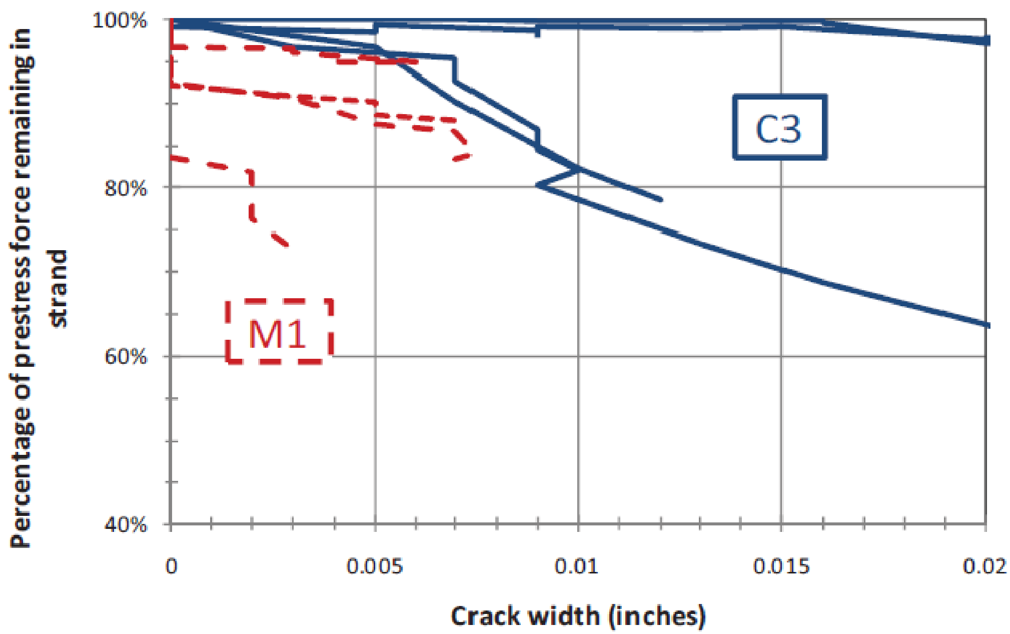

Figure 3.7: Typical Variation of Prestress Loss with Increasing Surface Width of Collinear Panel Cracks (C and M denote current and modified reinforcement configurations, respectively)

3.2 Work to Date on Optimizing Top Mat Reinforcement

As explained in Section 2.2, the requirements for transverse and longitudinal flexural reinforcement are largely governed by requirements for shrinkage and temperature reinforcement. The research group has focused on ways of optimizing cast-in-place deck reinforcement to meet these requirements. Research has included a study of probable requirements, laboratory studies of the performance of different configurations of reinforcement, and the extension of those laboratory studies to the field. In this section, each is discussed further, and principal findings are summarized.

3.2.1 Study of Probable Requirements as Limited by Shrinkage and Temperature Reinforcement

TxDOT now requires top-mat reinforcement consisting of No. 4 reinforcing bars spaced at 9 in. on centers in the longitudinal direction, and No. 5 reinforcing bars spaced at 6 in. on centers in the transverse direction.

Using the predictive formulas of CEB-FIP, the surface widths of transverse cracks were predicted for different configurations of longitudinal deck reinforcement. Results are shown in Figure 3.8. Qualitatively similar results were obtained with the Gergely-Lutz formulas used in the US.

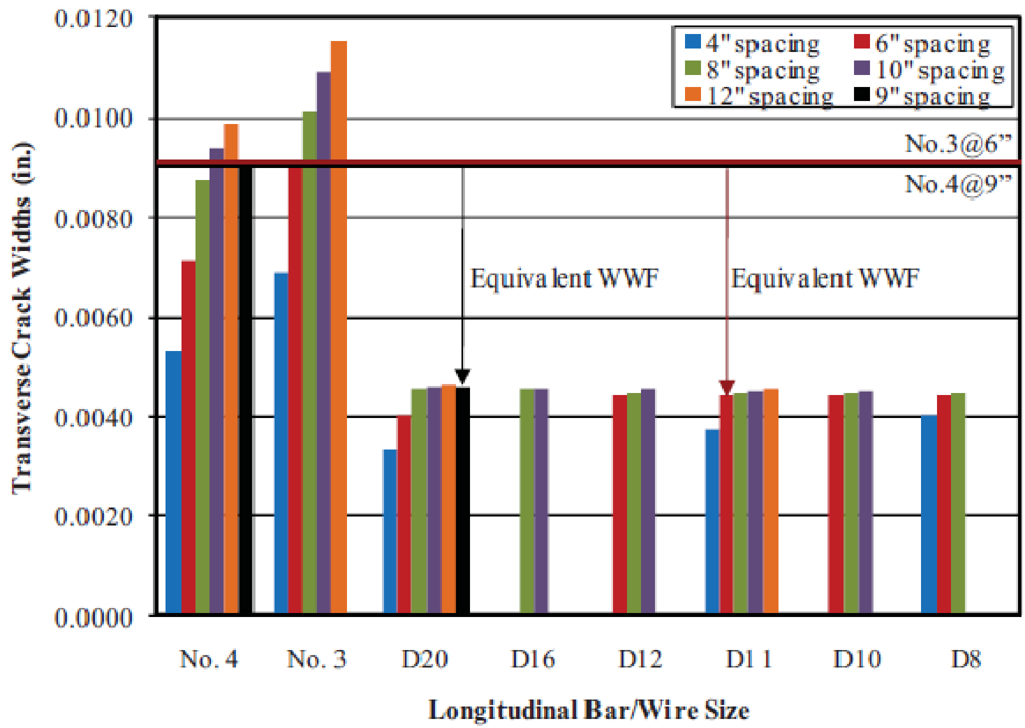

Figure 3.8: Predicted Width of Transverse Cracks for Different Configurations of Longitudinal Deck Reinforcement (CEB-FIP equations)

As shown in Figure 3.8, using No. 4 bars at 9 in. (the current TxDOT requirement for longitudinal deck reinforcement), indicated by the black bar above the "No. 4" on the horizontal axis, has a predicted surface crack width of almost 0.01 in. This is a relatively wide crack, and indicates that current TxDOT requirements for longitudinal deck reinforcement probably cannot be relaxed.

This indication is corroborated by simple hand calculations suggesting that the probable tensile strength of a section of CIP concrete 4-in. thick and 9-in. wide (the tributary area corresponding to current TxDOT requirements), is approximately equal to the specified yield strength of a No. 4, Grade 60 reinforcing bar. Simply put, current TxDOT requirements for longitudinal reinforcement (No. 4 bars @ 9 in.) provide barely enough reinforcement to remain unyielded under the force that is released when the deck cracks. If longitudinal reinforcement is reduced, it can be expected to yield when transverse deck cracks form, and will therefore be ineffective in controlling the width of those cracks.

As shown by the black bar in the "D20" group on the horizontal axis of Figure 3.8, predicted surface crack widths could be reduced by using welded-wire reinforcement instead of deformed reinforcement. The first reason for this is the welded cross-wires of welded-wire reinforcement, which give more efficient anchorage, reducing the spacing between cracks and therefore reducing the average width of each crack. The second reason for this is the higher specified yield strength of welded wire reinforcement (75 ksi rather than 60 ksi).

Predictions for transverse reinforcement show smaller crack widths, and hence the possibility of a reduction in required reinforcement. As shown in Figure 3.9, using No. 5 bars at 6 in. (the current TxDOT requirement for transverse deck reinforcement), indicated by the red bar above the "No. 5" on the horizontal axis, has a predicted surface crack width of about 0.005 in., considerably less than that noted above for the longitudinal reinforcement.

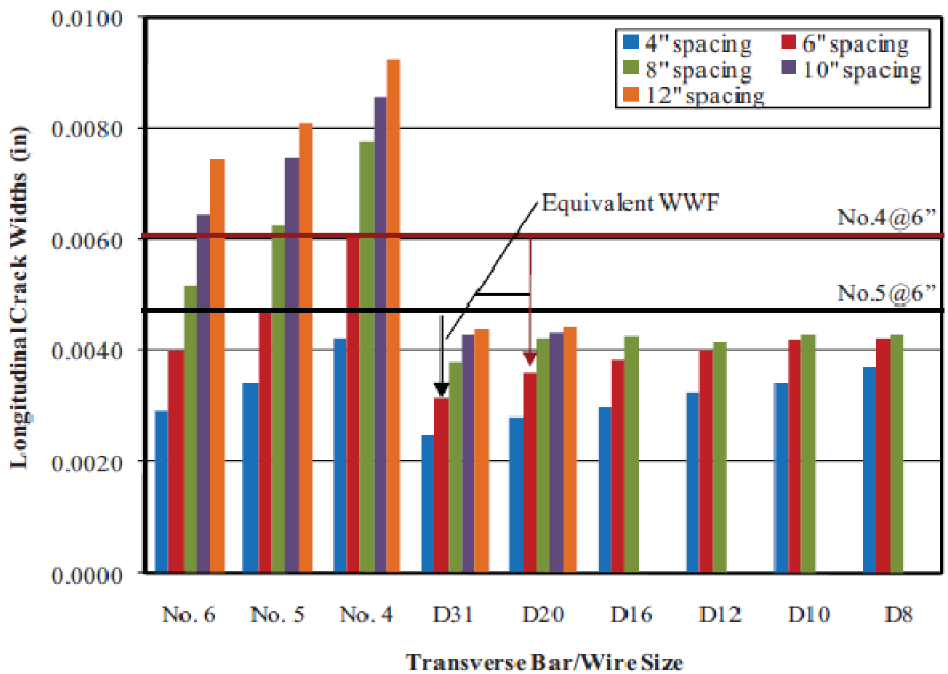

Figure 3.9: Predicted Width of Longitudinal Cracks for Different Configurations of Transverse Deck Reinforcement (CEB-FIP Equations)

This indication is again corroborated by simple hand calculations. Again, welded-wire reinforcement is predicted to be more effective in controlling crack widths than the same area of conventional deformed reinforcement, due to its welded cross-wires and its higher specified yield strength.

As shown in Figure 3.9, crack widths of about 0.005 in. could also be obtained using welded-wire reinforcement with a smaller cross-sectional area per foot of bridge deck than the current No. 5 bars. For example, D20 welded-wire reinforcement spaced at 12 in. (the orange line above the "D20" group on the horizontal axis of Figure 3.9) is predicted to control cracking better than the current reinforcement. Current requirements are equivalent to 0.62 in.^{2} of conventional reinforcement per foot of bridge deck, oriented transversely. If crack width could be controlled equally well with 0.20 in.^{2} of welded-wire reinforcement per foot of bridge deck, considerable cost savings would result, even considering the higher cost of welded-wire reinforcement and the possible additional costs associated with its detailing, handling, and placement.

3.2.2 Laboratory Studies of the Performance of Different Configurations of Top Mat Reinforcement

The crack-control performance of different configurations of top mat reinforcement was first investigated by UT Austin using the complex laboratory specimen shown in Figure 3.10. Because that specimen was complex, so were the boundary conditions at the loading beams, the combined stresses acting on the top-mat reinforcement, and the stresses acting on the panels near the panel butt joint. As a consequence, the results from this complex specimen were difficult to interpret.

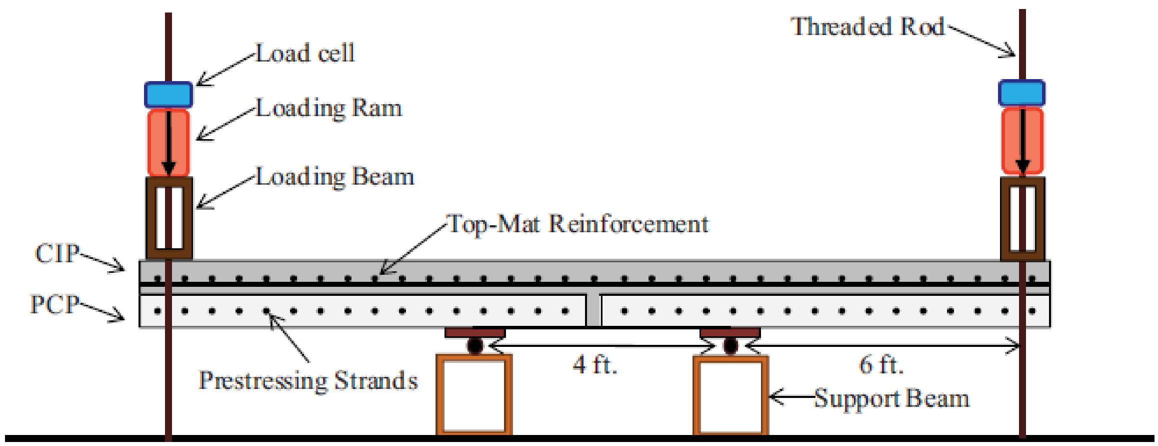

Figure 3.10: Complex Laboratory Specimen for Evaluating the Crack-Control Performance of Different Configurations of Top-Mat Reinforcement (UT Austin)

Progressive simplification led to the simple laboratory specimen shown in section view Figure 3.11. That specimen had a layer of reinforcement (2 or 3 bars or wires) at mid-thickness. Reinforcement within the layer was spaced at different distances for test purposes. Specimens with different types and spacing of reinforcement were tested under precise displacement control, permitting step-by-step evaluation of the crack width as displacement and corresponding stress in reinforcement were increased.

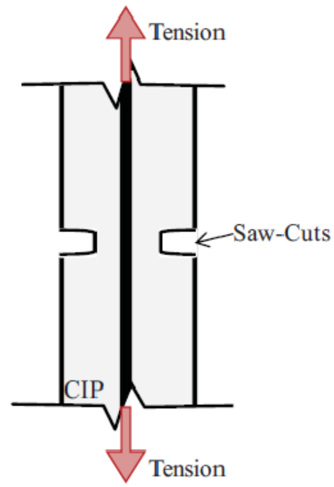

Figure 3.11: Simple Laboratory Specimen for Evaluating the Crack-Control Effectiveness of Top-Mat Reinforcement

Typical results from one of these simple laboratory specimens are shown in Figure 3.12. They permit precise measurement of steel stress and crack width as axial deformation increases.

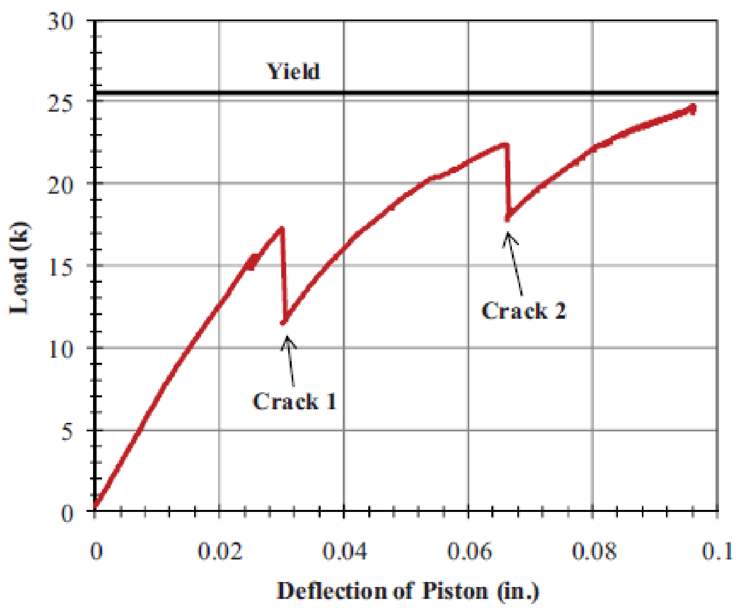

Figure 3.12: Typical Load-Deflection Results from Simple Laboratory Specimen

They also permit a correspondingly precise evaluation of the probable effectiveness of the reinforcement in controlling crack widths. An example of such a comparison is shown in Figure 3.13. The higher slope of the stress-elongation curves for welded-wire reinforcement indicates that the welded-wire reinforcement is more effective than conventional deformed reinforcement for controlling crack width.

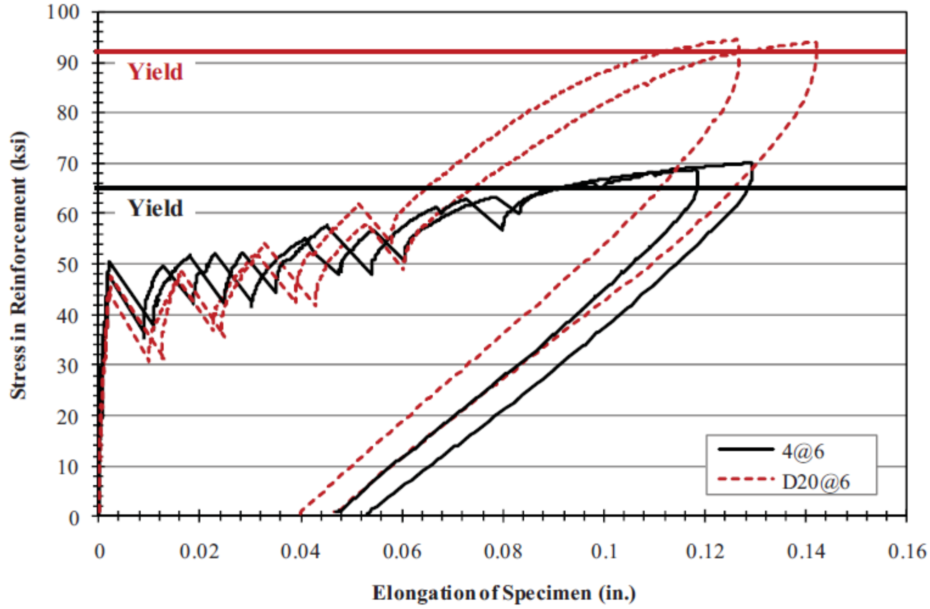

Figure 3.13: Stress versus Elongation for No. 4 and D20 Specimens

3.2.3 Field Studies of the Performance of Different Configurations of Top Mat Reinforcement

Once welded-wire reinforcement had been shown in the laboratory to be potentially better than conventional deformed reinforcement for controlling crack width, Project 0-6348 investigators began work with the TxDOT Project Monitoring Committee to identify possible sites for field studies.

Beginning in June 2010, Project 0-6348 investigators developed a reinforcement and instrumentation plan for a bridge in the Houston District. When unforeseen circumstances made that project unsuitable, the Project Monitoring Committee suggested another bridge project near Waco. Project 0-6348 investigators at UT Austin have developed a reinforcement and instrumentation plan for that bridge, using welded-wire reinforcement. Part of that plan is presented in Figure 3.14. Its details are being reviewed by industry experts in welded-wire reinforcement, and will soon be submitted to the Project Monitoring Committee for implementation.

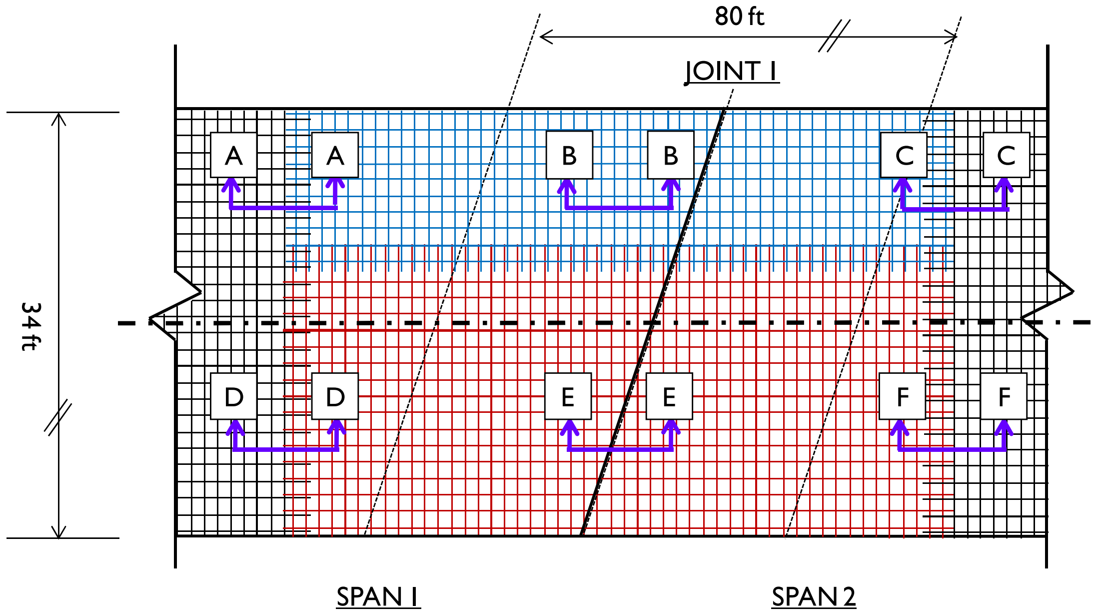

Figure 3.14: Section of Waco Bridge to be used for Field Study of Top-Mat Reinforcement

3.3 Work to Date on Possible Applications of High-Performance Steel Fibers in Concrete

As explained in Section 2.3, the focus of this aspect of Project 0-6348 is the development of test methods that will permit comparison of the performance of fibers produced outside of the US (for example, by Bekaert) with fibers produced inside the US (for example, by Royal).

3.3.1 Laboratory Development of Standard Test Methods for Evaluating Performance of Fiber-reinforced Concrete

Several standardized test methods of the American Society for Testing and Materials (ASTM) are available for evaluating the performance of fiber-reinforced concrete, typically in terms of stress-strain behavior. These methods include beam flexure tests with third-point loading (ASTM C1609). As shown in Figure 3.15, results from C1609 tests can vary considerably among replicates, making it difficult to use C1609 testing to evaluate the performance of concrete reinforced with high-performance fibers.

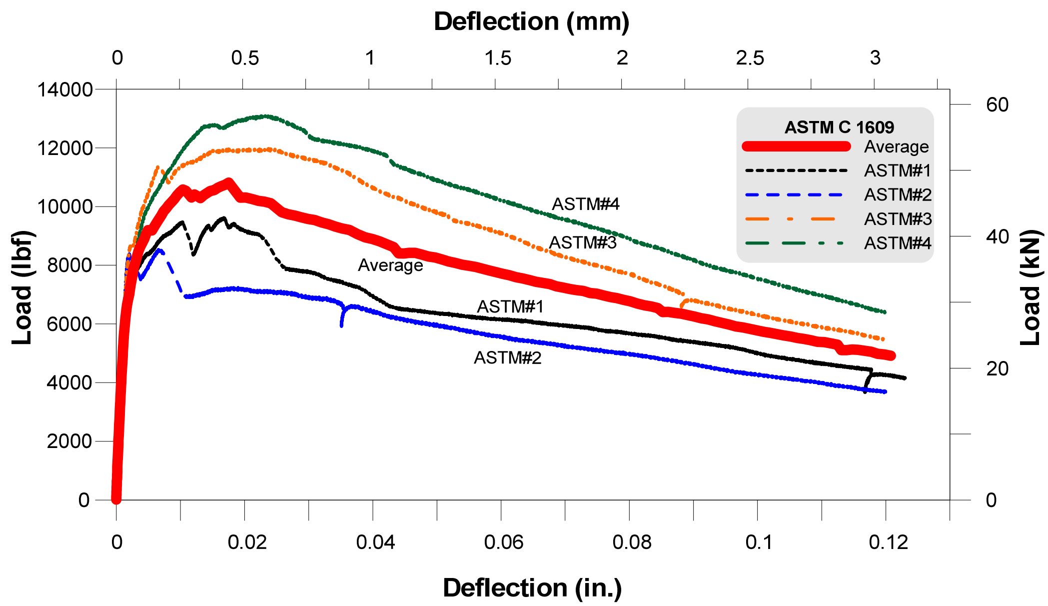

Figure 3.15: Variability Associated with Existing ASTM Beam-Flexure Standard (C1609) used to Investigate the Performance of Concrete Reinforced with High-Performance Fibers

In an effort to develop more reliable test methods for evaluating the performance of fiber-reinforced concrete, the researchers of Project 0-6348 originally proposed the so-called “double punch” test, in which a concrete cylinder (typically a sawn half-cylinder) is loaded in axial compression, in a conventional universal testing machine, between cylindrical steel punches (Figure 3.16). The presence of the steel punches creates transverse tension in the test specimen.

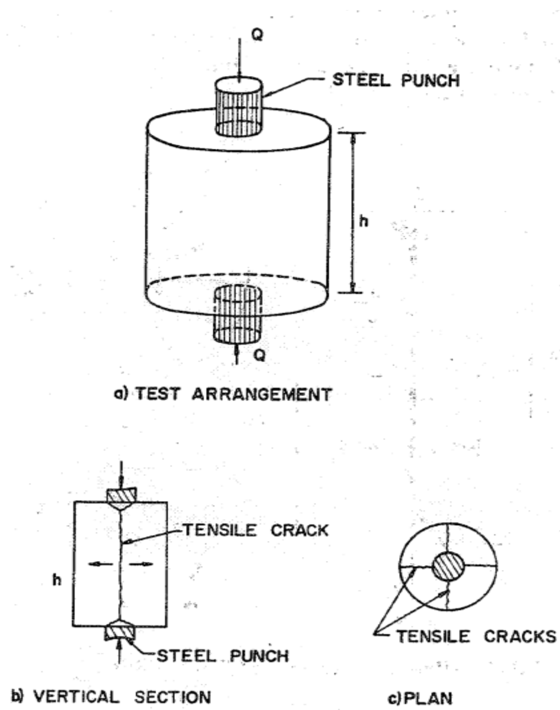

Figure 3.16: "Double-Punch" Test Specimen

Continued investigation by Project 0-6348 researchers at the University of Texas at Arlington has confirmed the potential value of the "double-punch" test for this purpose. Figure 3.17 shows a typical set of load-deformation curves obtained by the "double-punch" test, using test protocols developed by Project 0-6348 researchers. As shown by the red ellipses, results are very consistent among four replicates with respect to maximum load, to the ascending branch of the curve, and to the descending branch of the curve at a deflection of 0.1 in. This suggests that the "double-punch" test, used with a precise testing protocol, can be used to clearly and effectively compare different high-performance steel fibers.

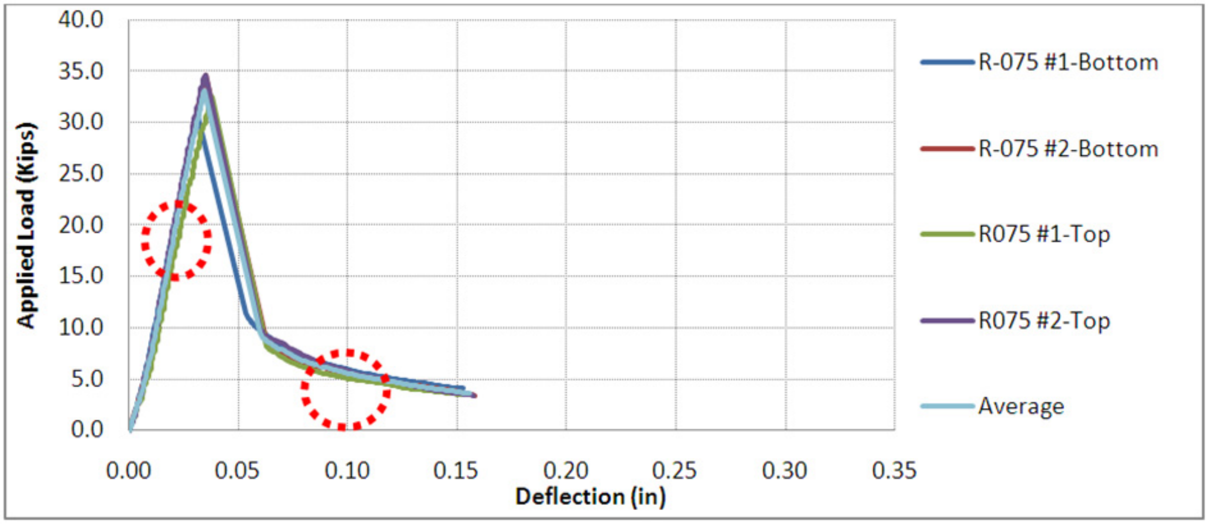

Figure 3.17: Typical Load-Deformation Curves for "Double-Punch" Tests of Concrete Reinforced with High-Performance Steel Fibers

3.3.2 Laboratory Verification of Standard Test Methods for Evaluating Performance of Fiber-reinforced Concrete

In a visit to UT Arlington in September 2010, UT Austin researchers participated in hands-on casting and testing of "double-punch" test specimens, and took several previously fabricated specimens back to Austin. UT Austin researchers will test those specimens in Austin, and will also prepare specimens themselves, some of which will be tested in Austin and others in Arlington.

Comparison of these results is expected to establish the repeatability (within-laboratory reliability) and the reproducibility (among-laboratory reliability) of the "double-punch" test, and to open the possibility of a draft ASTM test method.

Preliminary results like those shown in Figure 3.17 suggest that in order to achieve performance with U.S.-made fibers comparable to that previously seen by TxDOT with non-U.S. fibers, quite high fiber volume fractions may be necessary. Ongoing fiber work at UT Austin is intended to confirm or refute this conjecture.

Chapter 4. FUTURE WORK IN PROJECT 0-6348

In this section, future work in Project 0-6348 is briefly discussed and arranged according to each major work area.

4.1 Future Work on Controlling Cracking in Precast, Prestressed Deck Panels

At UT Austin, the researchers will continue to monitor the behavior of the panels stored there, with particular attention to differences in behavior between the panels meeting current requirements, and the modified panels with additional supplementary transverse reinforcement.

4.2 Future Work on Optimizing Top Mat Reinforcement

At UT Austin, the researchers will oversee and monitor the planned field application of modified top-mat reinforcement at the selected bridge near Waco. UT Arlington colleagues have been invited to participate, and have enthusiastically accepted.

4.3 Future Work on Possible Applications of High-Performance Steel Fibers in Concrete

At UT Austin, 0-6348 researchers will independently follow the "double-punch" fabrication and testing protocol used by colleagues at UT Arlington, and will confirm or refute the repeatability (within-laboratory reliability) and reproducibility (among-laboratory reliability) of that protocol.

Results are expected to clarify the feasibility of using high-performance steel fibers, in some cases, as a substitute for deformed reinforcement. If such fibers prove to be feasible, Project 0-6348 researchers are expected to conduct additional work on their field application.

REFERENCES

Foreman (2010): Foreman, James M., Jr., "Controlling Cracking in Prestressed Concrete Panels," MS Thesis, Department of Civil, Architectural, and Environmental Engineering, The University of Texas at Austin, May 2010.

Foster (2010): Foster, Stephen W., "Reducing Top Mat Reinforcement in Concrete Bridge Decks," MS Thesis, Department of Civil, Architectural, and Environmental Engineering, The University of Texas at Austin, May 2010.