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Implementation of High Performance Concrete in Louisiana Bridges (Interim Report)

by

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16. Abstract

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The literature review concentrates on practical information that can be used in the design and construction of bridges in Louisiana. Special provisions which emphasize changes in quality control procedures are developed. The instrumentation plan includes the monitoring of temperature effects, camber and deflections, strains, and prestress losses. The concrete materials test program involves measurements of compressive strength, modulus of elasticity, modulus of rupture, coefficient of thermal expansion, creep shrinkage, and permeability.

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IMPLEMENTATION OF HIGH PERFORMANCE CONCRETE IN LOUISIANA BRIDGES

INTERIM REPORT

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ABSTRACT

The report contains a research plan to assist the Louisiana Department of Transportation and Development in the implementation of high performance concrete in the Charenton Canal Bridge in Louisiana. The research involves a literature review, plan review, development of a quality control program for the concrete, development of an instrumentation plan and development of a plan to determine concrete material properties.

The literature review concentrates on practical information that can be used in the design and construction of bridges in Louisiana. Special provisions which emphasize changes in quality control procedures are developed to help ensure the successful use of high performance concrete. Cooperative efforts between the researchers, the state and the contractors are strongly encouraged. The instrumentation plan includes the monitoring of temperature effects, camber and deflections, strains and prestress losses. The concrete materials test program involves measurements of compressive strength, modulus of elasticity, modulus of rupture, coefficient of thermal expansion, creep, shrinkage and permeability.

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On behalf of the Louisiana Transportation Research Center, the work was performed under the administrative direction of Arthur Rogers, special studies/planning research manager. Masood Rasoulian and Craig Duos conducted the laboratory mix design program and developed the report sections dealing with quality control of cast-in-place concrete. Paul Fossier, bridge engineer manager of the Louisiana Department of Transportation and Development, was responsible for design of the bridge and greatly contributed to the program.

Appreciation is also expressed to Mary Lou Ralls of the Texas Department of Transportation for her input on the experiences with high performance concrete on two bridges in Texas and for assistance with the special provisions. The input of Don Theobald of Gulf Coast Pre-Stress, Inc., is also appreciated.

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INTRODUCTION

Objectives and Scope

The Louisiana Department of Transportation and Development (DOTD) plans to use high performance concrete in the Charenton Canal Bridge. The purpose of the research described in this Interim Report is to assist DOTD in the implement-ation of a high performance concrete bridge. Specific objectives of the proposed research are as follows:

1. Develop and monitor a quality control program for the high performance concrete mix and structural members during fabrication.
2. Evaluate the field performance of high performance concrete.
3. Evaluate material properties of high performance concrete used in Louisiana.
4. Assist DOTD in formulating policies regarding the use of high performance concrete in the state.

The objectives of the project are being achieved with the following scope of activities:

1. Literature review and plan review.
2. Development and monitoring of a quality control program for the concrete mix.
3. Development and coordination of an instrumentation plan; installation of instrumentation and subsequent data collection.
4. Analysis of data and development of recommendations.
5. Preparation of Interim and Final Reports.

The research is a cooperative program involving the researchers, DOTD, the Louisiana Transportation Research Center (LTRC), the contractor and the precast/prestressed concrete fabricator.

Methodology

In a previous report entitled *Feasibility Evaluation of Utilizing High Strength Concrete in Design and Construction of Highway Bridge Structures [1]*, the following recommendations were made:

1. Concrete with compressive strengths up to 10,000 psi (69 MPa) should be considered by Louisiana DOTD.
2. A quality control training program should be developed to train local precast concrete fabricators.

3. High-strength concrete with compressive strengths up to 10,000 psi (69 MPa) should be implemented in a bridge. This bridge should be instrumented to measure long-term behavior.
4. An investigation into the effects of steam curing on the properties of high-strength concrete should be performed.

Implementation of the above recommendations is being accomplished with the design and construction of the Charenton Canal Bridge. High-strength concrete will be used in the piles and girders. In addition, high performance concrete will be used in the bridge deck, barrier rails, barrier slabs, approach slabs and pile caps.

A literature review was conducted to identify the latest information on high performance concrete bridges. Emphasis was placed on practical information that could be used in the design and construction of bridges in Louisiana. Where appropriate, this information was used to modify the design criteria and specifications. To help ensure that concretes with the specified properties could be attained, LTRC initiated a program of laboratory trial mixes. Results of the LTRC mixes will be available to contractors bidding on the project. At the same time, a local precast concrete fabricator conducted some in-plant trial mixes.

Based on the literature review and review of the *Louisiana Standard Specifications for Roads and Bridges [2]*, special provisions for high performance concrete were developed. Particular emphasis was placed on changes in the quality control procedures to help ensure the successful use of high performance concrete. It was emphasized in the special provisions that construction of the Charenton Canal Bridge is a cooperative effort between the researchers, DOTD, LTRC, contractor and precast concrete fabricator.

An instrumentation plan for gathering data needed to monitor concrete properties, temperature effects, short- and long-term deflections, strains and prestress losses was developed. A test program to measure compressive strength, modulus of elasticity, tensile strength, coefficient of thermal expansion, creep, shrinkage and permeability was also developed. This test program is being performed in addition to standard tests which are necessary to meet the specifications for quality control.

High Performance Concrete Bridges in Louisiana

In 1988, a bridge project in Louisiana was used as an experiment to determine if the State could obtain a concrete compressive strength of 8,000 psi (55 MPa) on a production project. The specifications required 800 lb of cement per cu yd (475 kg/cu m) of concrete and allowed the use of a high-range water-reducer. The contractor had difficulty meeting the strength requirement and was penalized on 68 percent of the project's 2,370 ft (723 m) of girder.

In 1992, a 24-in. (610 mm) square prestressed concrete pile with a centrally located 12-in. (305-mm) diameter circular void and a length of 130 ft (39.6 m) was fabricated *[1]*. The concrete mix had a cement content of 750 lb/cu yd (445 kg/cu m), a silica fume content of 95 lb/cu yd (56 kg/cu m) and a water-cementitious materials ratio of 0.27. Average concrete compressive strengths at 18 hours and 28 days were 8,449 psi and 10,453 psi (58.3 and 72.1 MPa) respectively. The pile was driven as part of the bridge for State Route 415 over the Missouri Pacific Railroad. The pile was monitored with a pile driving analyzer. Results indicated that driving stresses were within the Federal Highway Administration's (FHWA) driving stress limits and that no pile damage had occurred. The report notes that damage to the pile would have likely resulted if a concrete with a compressive strength of either 5,000 or 6,000 psi (34.5 or 41.4 MPa) had been used *[1]*.

In 1993, bridges on the Inner Loop Expressway over the Ellerbe Road and West 70th Street were constructed in Shreveport using AASHTO Type IV girders. The production concrete used a cement content of 752 lb/cu yd (446 kg/cu m) and 7 percent silica fume. The specification required minimum concrete compressive strengths of 5,000 psi (34 MPa) at release and 8,500 psi (59 MPa) at 28 days. An average concrete compressive strength of 8,400 psi (57.9 MPa) was achieved at 18 hours and an average strength of 11,200 psi (77.2 MPa) at 14 and 28 days.

Although concrete compressive strengths were achieved, some other problems were encountered. These included rapid setting time, some cracks at mid-span from the heat of hydration and more surface air bubbles than normal. The effect of heat of hydration was reduced by lowering the steam temperature. The air bubbles were reduced by allowing the form oil to dry or by using a very thin film of form oil.

LITERATURE REVIEW

Worldwide interest in high performance concrete has grown dramatically during the last decade. As a result, the number of publications produced each year has been steadily increasing. The first major report on high-strength concrete was published by American Concrete Institute (ACI) in 1984 *[3]* and revised in 1992 *[4]*. Another report was published as part of the Strategic Highway Research Program (SHRP) Research Contract C-205 *[5]*. One of the objectives of the SHRP research contract was to conduct an extensive literature search on current knowledge about the mechanical properties of high performance concrete. An annotated bibliography containing 830 references, published from 1974 to 1989 was compiled. From this reference source, about 150 references were selected for critical review and are summarized in the State-of-the-Art Report *[5]*. A subsequent annotated bibliography for the years 1989-94 containing 760 citations has also been published *[6]*. In parallel, a separate State-of-the-Art Report covering the same time period has been prepared *[7]*. This last report concluded that high performance concrete has become widely accepted practically on all continents and that much of the application of high performance concrete is in the area of long-span bridges and high-rise buildings. In many applications, high-strength concrete is used only because of the high durability quality rather than the need for strength.

In addition to papers in technical journals and formal reports, significant information is now available as a result of the FHWA SHRP high performance concrete demonstration projects that are underway in several states. Information obtained from the showcase workshops held in Texas *[8]*, Nebraska *[9]*, Virginia *[10]* and Washington *[11]* is also included in this literature review. Since these are ongoing projects, the reported observations and conclusions represent interim results of these projects.

The objective of the current literature review is to identify information that is particularly relevant to the use of high performance concrete in Louisiana. Particular emphasis is placed on practical information that can be utilized in the design and construction of bridges in Louisiana.

Concrete Compressive Strength

Strength development with age

The variation of concrete compressive strength with age for the four different concrete mixes used in the previous LTRC project *[1]* are shown in Figure 1. All four concrete mixes were proportioned to produce a compressive strength of 10,000 psi (69 MPa) at 28 days. The curve labeled No. 1 is for a concrete that contained cement, fly ash and silica fume and was not heat cured. This concrete showed the least strength at early ages but a relatively larger strength gain at later ages. The curve labelled No. 2 is for a concrete that contained Type III cement and silica fume and was steam cured. The curve labeled No. 3 is for a concrete that contained a Type I-II cement and silica fume and was steam cured. Curve No. 4 is for a concrete that contained a Type I cement and silica fume and was not heat cured. The cement content of each concrete mix was the same; the quantities of silica fume varied slightly between the concrete mixes. Chemical admixtures in the form of air entraining agents, high-range water-reducers and water reducers were included in the concrete mixes.

The data in Figure 1 illustrate that variations in concrete mix proportions and curing methods can have a significant effect on the variation of concrete compressive strength with age. Consequently, it is important that the relationship be determined prior to the production of the actual high performance concrete girders for the Charenton Bridge. It is also important that the concrete used to select the mix be cured in a manner similar to that of the prestressed concrete girders. The necessity of this type of program has been demonstrated in other high performance concrete showcase projects *[8], [10], [11]*. Compressive strength data from three high performance showcase projects are shown in Table 1.

Data published by PCI *[12]* for high performance concretes indicates that release strengths varied from 63 to 87 percent of the 28-day strength. When normal strength concretes are used in prestressed concrete applications, the mix proportions are generally dictated by release strengths, and concrete strengths at 28-days are frequently in excess of the specified 28-day value. Consequently, mix requirements are generally based on the release strengths. The precast concrete fabricator only has to ensure that the mix will provide concrete with a compressive strength in excess of that specified for 28 days. However, with high performance

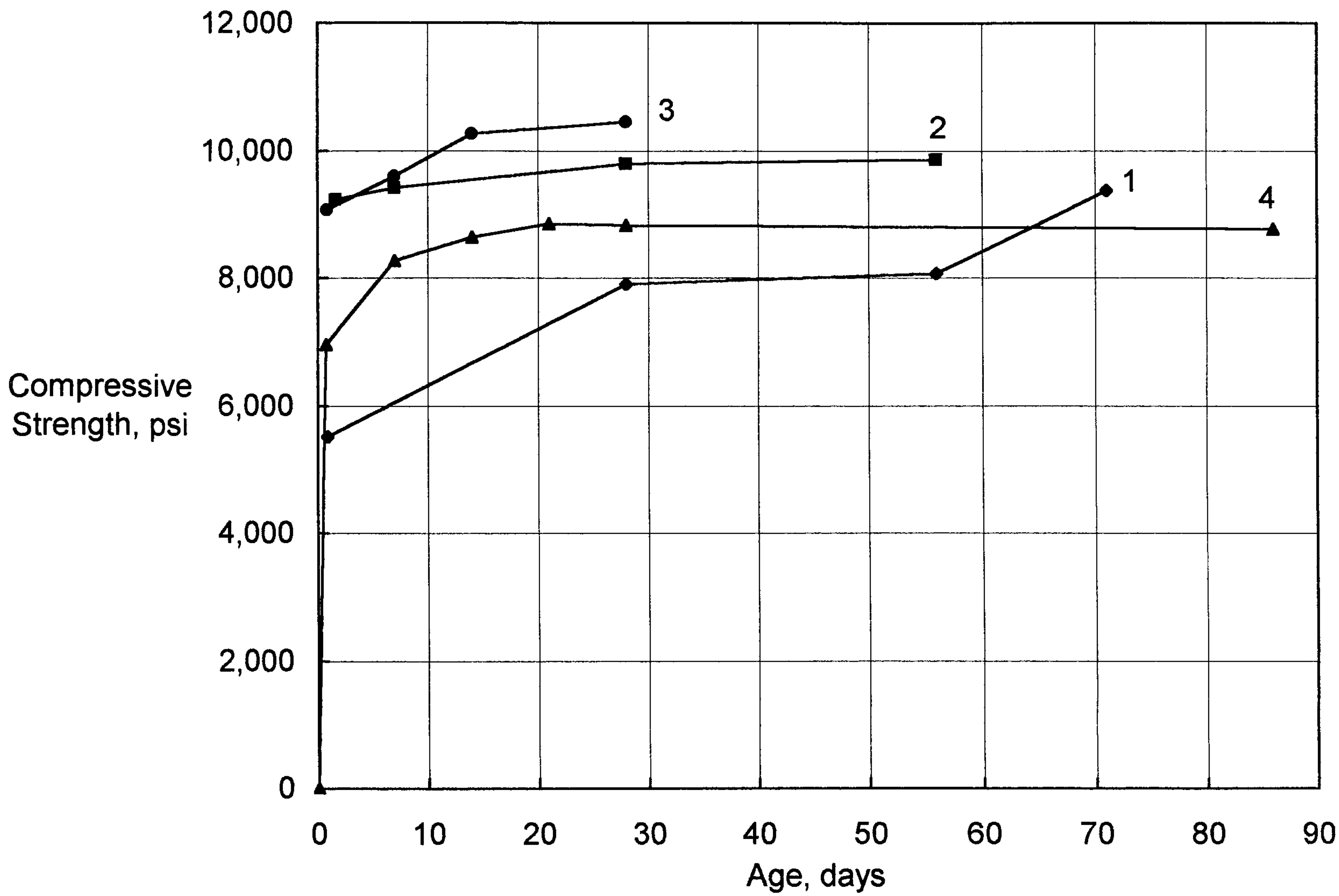

Figure 1
Variation of concrete compressive strength with age

Table 1
Compressive strength data from HPC projects

	Texas(1)	Virginia(2)	Washington(3)
Curing	No Heat	Heat	Heat
Avg. age at release, Hrs	23	-	33
Release strength	9,280	6,540	8,154
28-Day strength, psi	14,129	8,900	11,370
56-Day strength, psi	15,264	-	12,219
Release/28-day strength, %	66	74	72

1. Data from Louetta Road and North Concho River Projects *[13]*.
2. Virginia Route 629 *[10]*.
3. Covington Way Bridge *[11]*.

concretes and the necessity of achieving even higher release strengths, it becomes necessary to verify that the mix proportions are such that the concrete will achieve the desired strength at 28-days. In one project, the precast concrete fabricator was given the option of achieving 10,000 psi (69 MPa) at 56 days or 9,500 psi (66 MPa) at 28 days. This provided the precast concrete fabricator with greater flexibility to produce the high performance concretes.

If fly ash is used as one of the cementitious materials, specifying the design concrete strength at 56-days allows advantage to be taken of the later strength gain. It should be noted that specifying concrete strengths at 56 and 90 days for high-strength concrete in building construction has been common practice for many years.

Specimen size

Traditionally, the compressive strength of concrete has been determined by measurements on 6x12-in. (152x305-mm) cylinders. The specimen size has been standard in the industry for many years. However, this specimen size may lead to practical problems when testing high-strength concrete because the required loads to break the cylinders may exceed the capacities of available testing machines. Consequently, the use of smaller size specimens has been suggested.

Cook *[14]* indicated that for a 10,000 psi (69 MPa) design mixture, the strengths of 4-in. (102-mm) diameter cylinders were approximately 5 percent higher than the compressive strengths of 6-in. (152-mm) diameter cylinders. Burg and Ost *[15]* found that the strength of 4-in. (102-mm) diameter cylinders were within 1 percent of the strength of 6-in. (152-mm) diameter cylinders. For concretes in the 10,000-20,000 psi (69-138 MPa) range, Carino et al. *[16]* found that the differences were less than 2 percent. Based on work conducted by Day *[17],* the Canadian Standard A 23.1 *[18]* requires a 5 percent reduction in the measured strength of the cylinder when 4x8-in. (100x200-mm) diameter cylinders are used. Where 4-in. (102-mm) diameter cylinders have been used in the United States, strength reductions have not been applied to the measured strengths.

Core strengths versus cylinder strengths

Based on a statistical analysis of strength data for 771 cores from 31 large elements using 22 concrete mixes, Bartlett *[19]* concluded that the observed ratio of the average in-place concrete strength to the average standard cylinder strength decreased as the maximum temperature that the element was subjected to during hydration increased. The average in-place concrete strength at 28 days roughly equaled the average standard 28-day cylinder strength for HPC containing only portland cements as the cementitious material. If the concretes contained silica fume, slag, Class C fly ash or small quantities of Class F fly ash, the ratio of the average in-place concrete strength at 28 days to the average 28-day control cylinder strength of the same concrete was about 15 percent smaller than that for concretes which did not contain the other cementitious materials.

Burg and Ost *[15]* compared the strength of 4-in (102-mm) diameter concrete cores taken from insulated concrete blocks with the strengths of moist-cured concrete cylinders at the same age. They observed that core strengths for five out of six concrete mixes equalled or exceeded 85 percent of the strength of the moist-cured control specimens when tested at the same ages of 91 days and 14 months. When the core strengths at 91 days and 14 months were compared with the 28-day moist-cured cylinder strengths, the ratio ranged from 0.88 to 1.23.

Cook *[14]* reported tests of 4-in. (102-mm) diameter cores taken from 30x30-in. (760x760-mm) columns of 10,000 psi (69 MPa) compressive strength concrete. Test ages ranged from 7 to 365 days. The average ratio of core strength to moist-cured cylinder strength at each age ranged from 0.84 to 0.99.

Bickley et al. *[20]* reported tests at 1, 2 and 7 years of age on 4-in. (100-mm) cores from 10,000 psi (69 MPa) concrete. Ratio of core strength to 28-day moist-cured cylinder strength ranged from 0.97 to 1.00. Aitcin and Riad *[21]* reported 2-year core strengths of 97 percent of the 28-day moist-cured cylinder strength.

In the previous LTRC project *[1]*, eleven concrete cores were extracted from the bulb-tee girders and tested in compression. Test ages ranged from 40 to 660 days. The ratio of core strength to cylinder strength at the same test age ranged from 0.92 to 1.21 with an average value of 1.02.

Based on the above discussion, it is concluded that the acceptance criteria for core strengths specified in ACI 318 *[22]* are also applicable to high performance concretes. These provisions may be used for interpretation of core strengths if this becomes necessary. However, the preferred method is to establish a correlation curve for each mixture to relate the strength of extracted cores to the strength of the specimens used for acceptance testing.

Tensile Strength

For prestressed concrete, the *AASHTO Standard Specifications [23]* state that temporary allowable stresses before losses due to creep and shrinkage in tension areas with no bonded reinforcement are limited to 200 psi (1.3 MPa) or 3 $\sqrt{f'_{ci}}$. The maximum tensile stress with bonded reinforcement is 7.5 $\sqrt{f'_{ci}}$. After losses have occurred, the allowable tension stress in the precompressed tensile zone for members with bonded reinforcement is limited to 6 $\sqrt{f'_c}$. For severe exposure conditions, such as coastal areas, the limitation is 3 $\sqrt{f'_c}$. Similar allowable stresses given in the *AASHTO LRFD Specifications [24]* are provided so that the concrete will not crack under service load conditions. These provisions are based on the typical assumption that the strength of the concrete is 7.5 $\sqrt{f'_c}$. This empirical relationship was established based on normal strength concretes. Values reported by various investigators *[3], [4]* for the modulus of rupture of both lightweight and normal weight high-strength concrete fall in the range of 7.5 to 12 $\sqrt{f'_c}$.

Based on their test results for high performance concrete, the Texas Department of Transportation increased the allowable tensile stress at release from 7.5 to 10 $\sqrt{f'_{ci}}$ and the allowable tensile stress at design from 6 to 8 $\sqrt{f'_c}$. Results from tests by Mokhtarzadeh et al. *[25]* and Shing et al. *[26]* showed that the modulus of rupture values for both heat-cured and moist-cured specimens were greater than values predicted by the AASHTO equation.

Data for modulus of rupture from the previous LTRC projects are summarized in Table 2. These numbers indicate that the modulus of rupture had an average value of 8.3 $\sqrt{f'_c}$. Although the coefficient is slightly higher than the traditional value of 7.5 $\sqrt{f'_c}$, no change is recommended and Louisiana should still continue to use the allowable tensile stresses as defined in the *AASHTO Standard Specifications*.

Table 2
Modulus of rupture

Specimen No.	Age, days	Compressive Strength, f'_c, psi	Modulus of Rupture MOR, psi	Coefficient k*
P1	28	7,400	720	8.37
P1	56	7,660	733	8.38
P2	28	7,900	750	8.43
P2	56	8,290	883	9.70
P3	28	8,410	665	7.25
P3	56	8,250	817	9.00
BT1-2	40	9,570	810	8.28
BT2-2	40	9,900	840	8.44
BT3-2	660	10,180	890	8.82
BT5-1	28	7,680	750	8.56
BT5-2	28	9,250	580	6.03
BT5-3	28	9,550	820	8.39

$$* k = \frac{MOR}{\sqrt{f'_c}}$$

Modulus of Elasticity

Several equations have been proposed that relate the modulus of elasticity, E_c, to the concrete compressive strength, f'_c, and the unit weight, w_c. They include the following:

AASHTO/ACI *[22], [23]* $E_c = 33\ w_c^{1.5}\ (f'_c)^{0.5}$ (1)

Martinez *[27]* $E_c = (40{,}000\ (f'_c)^{0.5} + 1{,}000{,}000)\ (w_c/145)^{1.5}$ (2)

Canadian Code *[28]* $E_c = (39{,}740\ (f'_c)^{0.5} + 1{,}000{,}000)\ (w_c/143.5)^{1.5}$ (3)

Ahmad *[29]* $E_c = w_c^{2.5}\ (f'_c)^{0.325}$ (4)

The AASHTO/ACI equation was based on an analysis *[30]* for concrete with strengths up to about 6,000 psi (41 MPa). Several investigators *[3], [4]* have indicated that the AASHTO/ACI equation tends to over estimate the modulus of elasticity for the higher strength concretes. The Martinez equation was developed as an alternative for the AASHTO/ACI equation and gives lower values of modulus of elasticity at the higher strength levels. Other authors *[3], [4]* have indicated that the Martinez equation may underestimate the modulus of elasticity at the very high strength levels. The Canadian Code equation was based on the Martinez equation but numbers are rounded off for use with SI units. For this report, the equation has been converted back into English units because rounding off causes slightly different calculated values. The Ahmad equation was based on a statistical analysis of data. The Ahmad equation gives calculated values for the modulus of elasticity between those of the AASHTO/ACI and the Martinez equation. All equations give approximate values for the modulus of elasticity and are suitable when other information is not available. However, it should be noted that these equations are based on a general analysis of measured values from many different sources cured under different conditions and with measurements made by many different investigators.

Khan et al. *[31]* conducted a detailed test program to investigate the stress-strain relationship of high-strength concretes at early ages. The effects of temperature-matched curing, sealed curing and air dried curing on 4,400, 10,000 and 14,500 psi (30, 69 and 100 MPa) compressive strength concretes were studied. The temperature-matched curing resulted in a higher rate of modulus gain than the sealed and air-cured cylinders at early ages. The effect was more apparent with increasing compressive strength.

Mokhtarzadeh et al. *[25]* measured modulus of elasticity values at 1, 28 and 182 days. Values for heat-cured specimens at one day ranged from less than the values predicted by the Martinez equation to more than values predicted by the AASHTO/ACI equation. They concluded that heat-cured specimens exhibit slightly lower moduli of elasticity at any given strength than moist-cured cylinders. Their results indicated that the modulus at one day, measured on heat-cured cylinders, was approximately 98 percent of the 28-day modulus value.

A comparison of the equations with test data obtained on previous and current LTRC projects is shown in Figure 2. In the previous project *[1]*, concrete specimens were obtained from three different precasting plants. For concrete produced in each plant, modulus of elasticity and concrete compressive strength were measured at various ages. These data are plotted in Figure 2. In addition, data obtained during the trial mixes for the current project are shown. It is apparent that the most consistent prediction of modulus of elasticity is obtained utilizing the AASHTO/ACI equation. Consequently, it is recommended that DOTD continue to use the AASHTO/ACI equation for prediction of modulus of elasticity when values for a specific concrete mix are unknown.

Shrinkage

The research reported by ACI Committee 363 *[3], [4]* indicated that the final shrinkage of high-strength concretes was of the same order of magnitude as that for normal strength concretes. However, results by Burg and Ost *[15]* indicated that the shrinkage of moist-cured prisms and cylinders decreased as concrete strength increased.

For many years *[32]*, it has been known that the shrinkage of steam-cured concretes is less than the shrinkage measured on concrete specimens subjected to moist curing. However, little data exist about the shrinkage of high-strength concretes that are heat cured. Data from the previous LTRC project for steam-cured concretes indicated an ultimate shrinkage of 409 millionths. This ultimate shrinkage would be well below the expected range for moist-cured concretes. Test results from the University of Texas in connection with the Texas high performance concrete bridges, indicated ultimate shrinkage values on 4-in. (102-mm) diameter cylinders of 423 millionths. Although the specimens were not steam cured, they were subjected to elevated temperatures as a result of heat of hydration.

Shrinkage tests were made in connection with the Colorado Showcase Project *[26]*. Specimens were steam cured followed by either air curing and moist curing. Measured strains ranged from 500 to 550 millionths at 140 days irrespective of the curing method. Concrete compressive strength was about 10,000 psi (70 MPa).

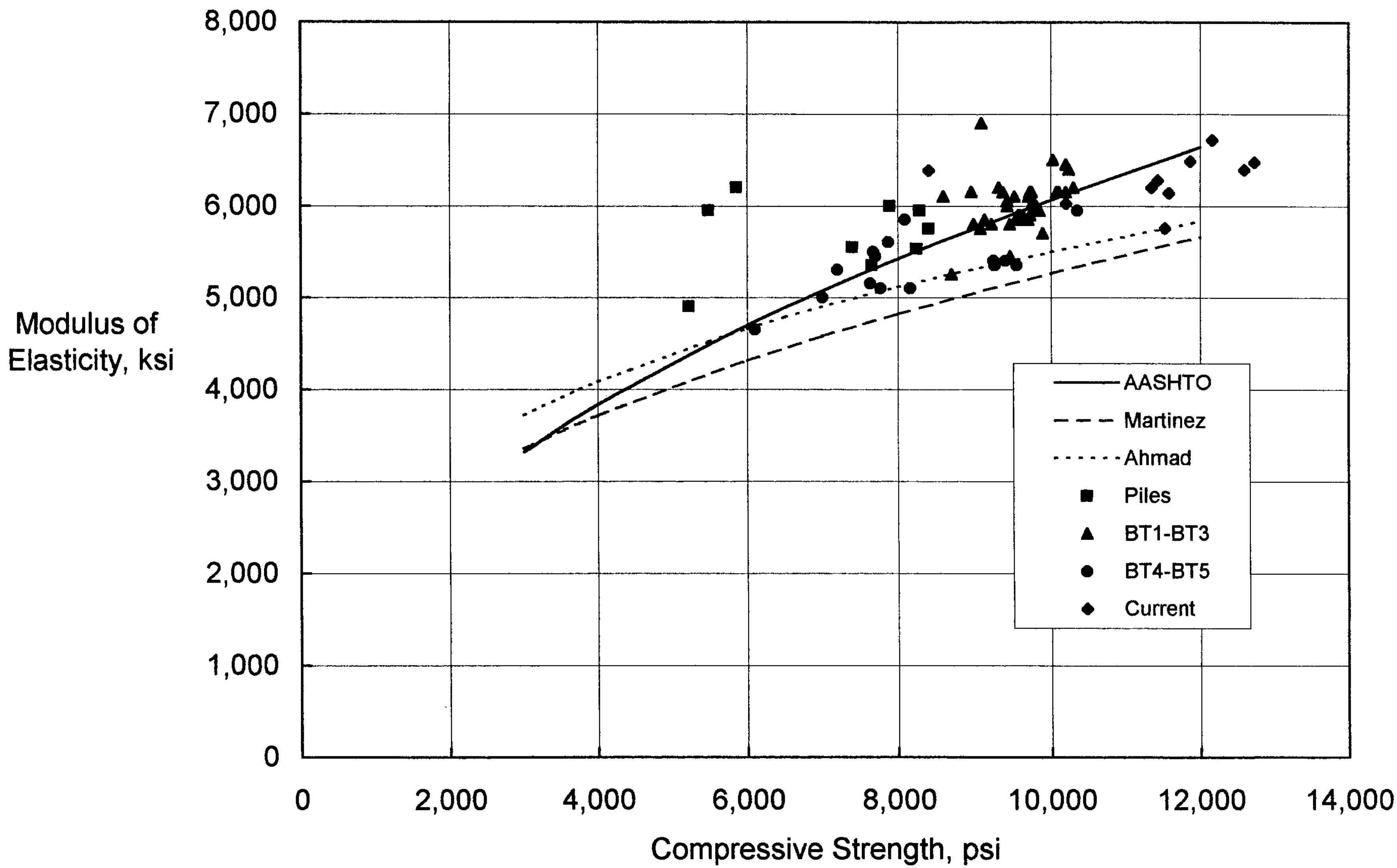

Figure 2
Comparison of modulus of elasticity data

Creep

Creep of concrete can be expressed in terms of creep coefficient or specific creep. The creep coefficient is the ratio of creep strain to the initial strain at loading. The normal range of creep coefficient is from 1.30 to 4.15 *[33]*. Specific creep is defined as the creep strain per unit stress and varies between 0.2 and 1.5 millionths/psi (30 and 215 millionths/MPa). The relationship between creep coefficient and specific creep is as follows:

creep coefficient = specific creep x modulus of elasticity at age of loading

Specific creep data for 6x12-in. (152x305-mm) cylinders obtained by steam curing and published by several authors *[1], [32], [34]* are shown in Figure 3. These data have been obtained for a variety of concrete constituent materials loaded at different ages and maintained under constant load for different lengths of time. To partially eliminate the variables associated with the length of time under load, the published data were corrected to final values on the assumption that the variation of creep with time follows the equation:

$$C_u = C_t \left(\frac{10 + t^{0.6}}{t^{0.6}}\right) \tag{5}$$

Since it is anticipated that high-strength concrete prestressed girders will either be produced by steam curing or will achieve relatively high temperatures from heat of hydration, the effects of curing temperatures on the creep of the concrete are important. Hanson *[32]* indicated that the effect of atmospheric steam curing was to reduce the creep of concrete cylinders containing Type I cement by 20 to 30 percent and that of concretes containing Type III cements by 30 to 40 percent below that of the same concretes moist cured for six days. It is also apparent from Figure 3 that the specific creep decreases fairly rapidly as concrete compressive strength increases. Also shown in Figure 3 is the value of specific creep predicted using the equation of the *AASHTO LRFD Specifications*. It can be seen that the AASHTO equation overestimates the amount of specific creep at the higher concrete compressive strengths.

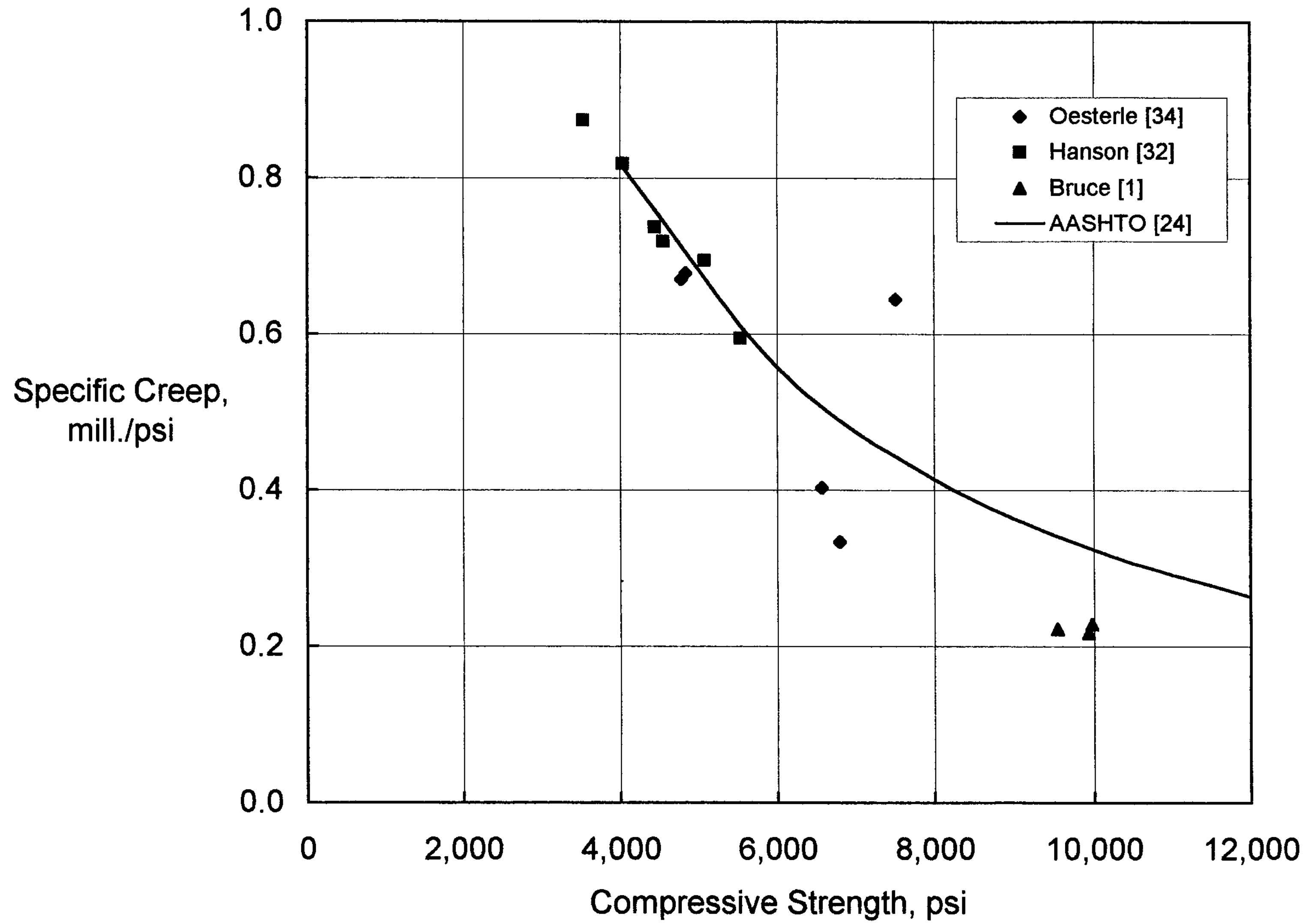

Figure 3
Comparison of creep data

Prestress Losses

Previous research for LTRC *[1]* indicated that measured long term prestress losses on a steam-cured, high-strength concrete girder were approximately 50 percent less than the total prestress loss calculated using the provisions of the *AASHTO Standard Specifications* and design material properties. It was concluded that the AASHTO provisions for creep and shrinkage losses are overly conservative for high-strength concrete.

Prestress losses on two high-strength concrete girders were monitored by Ahlborn et al. *[35]*. Prestress losses measured at the age of 200 days were consistent with those predicted by the *AASHTO Standard Specifications* when actual material properties were used in the calculations.

In the Washington State HPC demonstration bridge *[11]*, the measured prestress losses at 100 days were greater than values calculated using the PCI general method and a modified rate of creep method. The effect of larger prestress losses is to require a higher prestressing force at strand release and a higher concrete compressive strength at release.

For the Texas HPC demonstration projects, Burns et al. *[36]* reported that measured elastic shortening losses were significantly higher than predicted. They attributed the cause to restraint from the prestressing bed prior to release and its influence on the baseline reading of the strain gages. The restraint was developed as the temperature of the girder decreased prior to release of the strand. All readings on the strain gages after release would include an apparent compressive strain in the readings. They also reported that long-term losses were higher for HPC U54 beams than for normal strength concrete Type IV beams.

Based on the above discussion of shrinkage, creep and prestress losses, it is recommended that design of the Charenton Canal Bridge for these effects be based on values for normal strength concrete.

Camber and Deflection

The Texas high performance concrete bridge at San Angelo uses AASHTO Type IV girders. The first girder cast had a length of 153 ft 4 in. (46.74 m). Measured camber at release of the strand was 0.16 in. (4 mm) compared to a calculated camber of 0.67 in. (17 mm). This prompted a detailed study of camber of long-span HPC girders *[37]*. This study indicated that calculated camber at release may be expected to be higher than actual measured values because of differential shrinkage and thermal effects. The study also found that use of the ADAPT computer program *[38]* resulted in calculated cambers that agreed reasonably well with measured values at 100 days. This occurred despite the differences between calculated and measured values at release of the strand. Measured camber at 100 days was approximately 6 in. (150 mm).

Ahlborn et al. compared measured camber with values calculated using the PCI method *[39]* for the two girders tested at the University of Minnesota. They found that measured cambers were less than values predicted using the PCI method. They observed that the girders achieved a maximum camber at about 50 days after which the camber decreased.

In the previous research for LTRC *[1]*, one girder was tested under sustained load for a period of 18 months. Measured data indicated that midspan camber stabilized at an age of about 6 months.

For the Washington State HPC demonstration bridge *[11]*, the measured variation in camber of five 133-ft (40.5-m) long girders was 1.5 in. (38 mm). The differential camber between girders means that additional depth of concrete haunch must be provided to ensure that the grade of the riding surface can be set properly.

Flexural Strength

Previous research for LTRC *[1]* indicated that equations in the AASHTO specifications for calculating flexural strength provided a conservative prediction of girder strength. This conclusion was based on tests of three full-size 54-in. (1.37-m) deep bulb-tee girders with decks which were tested in flexure with a span length of 69 ft (21 m). Concrete compressive strengths at time of test ranged from 7,640 to 10,300 psi (52.7 to 71.0 MPa).

The Virginia Transportation Research Council *[10]* reported flexural tests of 10,000 psi (69 MPa) prestressed concrete beams without decks, in which the load-carrying capacities were at least 9 percent greater than the expected values. Also, the loads to produce the first cracks exceeded expected values by at least 6 percent. In a second program, two AASHTO Type II girders with composite decks were tested in flexure. In four tests, the flexural cracking loads exceeded the theoretical values by at least 10 percent. Flexural strengths exceeded theoretical values by at least 5 percent. Average concrete compressive strength at 28 days was 10,390 psi (71.6 MPa).

Shear Strength

Previous research for LTRC *[1]* indicated that equations in the AASHTO specifications for calculating concrete flexural shear strength, web shear strength and total shear strength provided a conservative prediction of girder strength. This conclusion was based on tests of three full size 54-in. (1.37-m) deep bulb-tee girders tested in shear.

Transfer and Development Lengths

In October 1988, FHWA issued a memorandum that placed the following restrictions on the use of seven-wire strands for pretensioned members in federally funded highway bridge applications:

5. The use of 0.6-in. (15.2-mm) diameter strand in a pretensioned application shall not be allowed.
6. Minimum strand spacing (center to center) will be four times the nominal strand diameter.
7. Development length for all strand sizes up to and including 9/16 in. (14.3 mm) shall be determined as 1.6 x AASHTO Equation 9-32.

Transfer length

As a result of the FHWA memorandum, numerous research projects on transfer and development length were initiated in the United States and Canada. This research has been summarized by Buckner *[40]*. Buckner cited test results by Kaar involving normal weight concrete with compressive strengths at transfer ranging from 1,660 to 5,000 psi (11.5 to 34.5 MPa) that lead to the conclusion that compressive strength at transfer has little influence on transfer length. A more recent study by Mitchell *[41]* with normal weight concrete with compressive strengths from 4,500 to 12,900 psi (31 to 89 MPa) concluded that the transfer length decreased with an increase in compressive strength at transfer.

In the previous study for LTRC *[1]*, transfer length was measured on three bulb-tee specimens and three pile specimens. Transfer lengths interpreted from strain readings were consistently less than the transfer length implied by the development length provision of the *AASHTO Standard Specifications*. Transfer lengths were also measured on two girders at the University of Minnesota *[35]*. The measured transfer lengths were also less than values calculated using the *AASHTO Standard Specifications*. A summary of transfer lengths for various concrete strengths above 6,000 psi (41 MPa) and for 1/2-in. and 0.6-in. (13-mm and 15-mm) diameter strands are shown in Figure 4. Research by Castrodale *[42]* concluded that transfer lengths for high-strength concrete are shorter than for normal strength concrete and that the current AASHTO expression provided a conservative, yet reasonable, estimate for the transfer length of strand in high-strength concrete.

As part of the development process for the use of high performance concrete in Texas, two prototypes of a newly developed U-beam cross section and three full size sections of the U-beams bottom flange were fabricated with 0.6-in. (15-mm) diameter prestressing strands at 1.97-in. (50-mm) spacing. Concrete strains were measured on the outside surface of the specimens. For the U-beam specimens, the transfer length could not be determined from the measured data because of overlapping transfer zones due to the short debonding length increments and because of distortions in the measured strain in the end regions of the beam due to the large end block. Data from the bottom flange test specimens indicated transfer lengths for 0.6-in (15-mm) diameter strand to be less than 27 in. (686 mm) for fully bonded strands and between 18 and 24 in. (455 and 610 mm) for debonded strands.

Development length

Mitchell *[41]* found that an increase in concrete compressive strength resulted in a reduction of the flexural bond length and hence a reduction in the strand development length. He proposed a design expression for the development length of prestressing strand which is a modification of the AASHTO development length expression but includes factors which would account for the concrete compressive strength. His equation indicates that the development length is inversely proportional to the square root of the concrete compressive strength.

Development length tests were also conducted in connection with the Texas high performance concrete bridges *[8]*. These tests involved 0.6-in. (15-mm) diameter strands and a concrete compressive strength of 11,800 psi (81.4 MPa). All of the specimens failed in flexure. Consequently, the true development length was not obtained but was less than 78 in. (1.98 m). This again, is less than would be predicted using the *AASHTO Standard Specifications*. In 1996, FHWA began to allow State agencies to use 0.6-in. (15-mm) diameter strands in prestressed concrete members at 2-in. (50-mm) spacing or 0.5-in. (12.7-mm) diameter strands at 1.75-in (45-mm) spacing. However, no change was issued in the transfer and development length formulas that specified multipliers of 1.6 for fully bonded strands and 2.0 for unbonded strands on the development length calculated by the *AASHTO Standard Specifications*. Based on the limited research on development length for high performance concretes, the use of the multipliers does not appear to be justified.

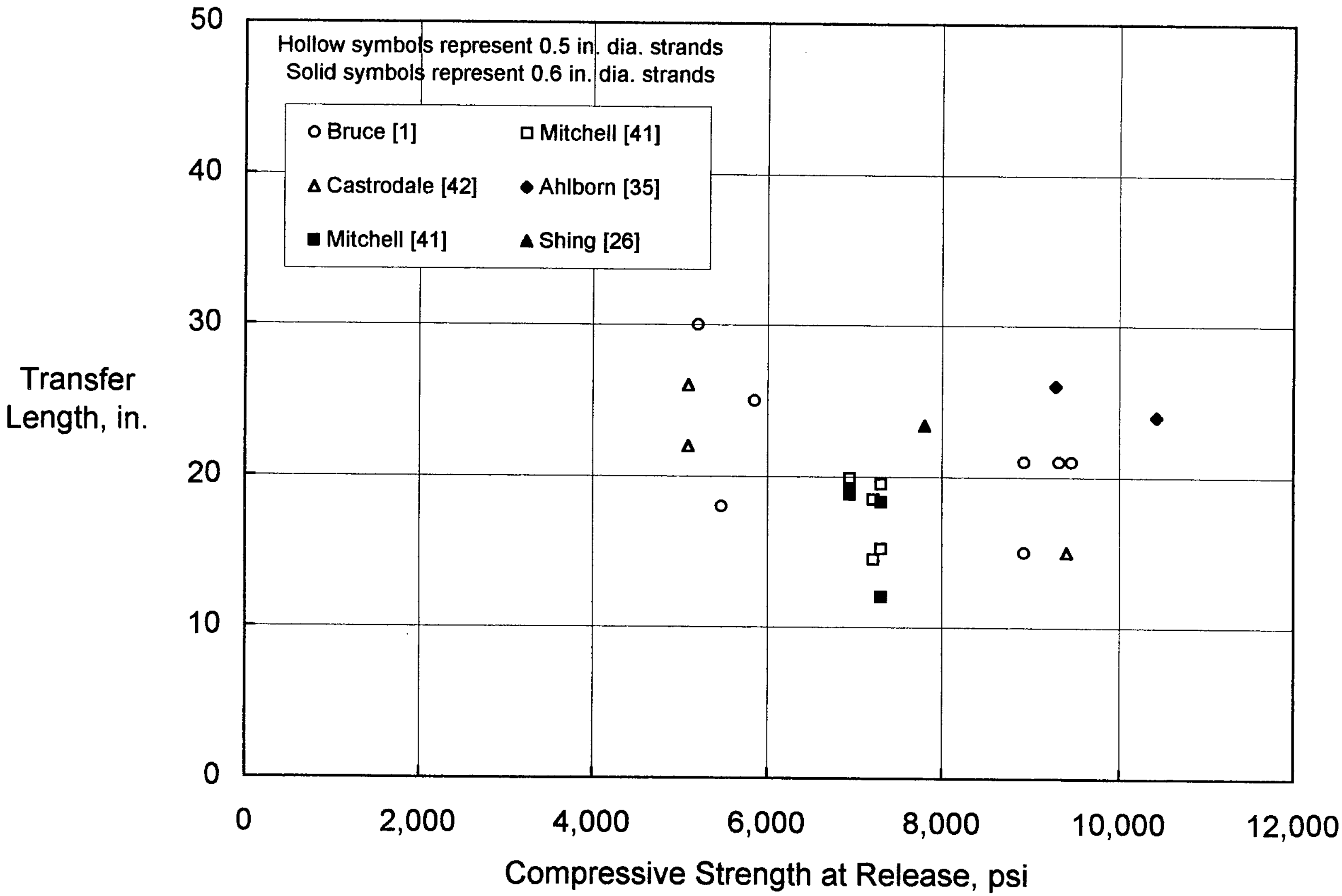

Figure 4
Comparison of transfer lengths

Girder Curing and Heat of Hydration

In 1993, Zia and Caner *[43]* reported a cracking problem associated with the production of large-size, long-span prestressed concrete AASHTO girders. Vertical cracks often developed near the mid-third of the span after the girders were cured overnight and before the prestressing strands were detensioned. The authors reported that the most probable cause of the cracking problem was due to restrained thermal contraction during the cooling period after the overnight heat curing. It was recommended that the total length of exposed tendon be increased or the cooling period be reduced. Cracks were also observed near midspan in the girders manufactured for the previous LTRC project *[1]*.

High-strength concretes contain more cementitious material than used in normal strength concretes. Therefore, the heat generated during hydration is greater. If the heat is retained, high curing temperatures occur. This, in turn, affects the strength gain. With high curing temperatures, high early-age strengths are achieved, but strengths at later ages do not increase as much. With low curing temperatures, achieving high early strengths is difficult, but higher strength gains occur at later ages. Ozyildirim and Gomez *[44]* have concluded that proper temperature management is needed to achieve high early strengths as well as high strengths at 28 days. Their recommendations were based on trial mixes in the laboratory and in a prestressed concrete plant. In the plant, AASHTO Type II beams were cast and steam cured. Measured temperatures in one beam reached 220°F (104°C) even though the steam was turned off when the temperature of the beam approached 190°F (88°C) and the bed was vented. Ozyildirim and Gomez also recommended that the maturity method or temperature-matched curing be used for a representative determination of compressive strength of the concrete in the member at an early age.

On high-strength concrete beams produced for the Louetta Road Overpass in Houston *[8]*, maximum temperatures achieved inside different portions of a precast U-beam varied depending on the mass of the concrete at that location. End blocks achieved higher temperatures than the bottom flanges which were hotter than the webs. It was also found that the maximum temperature achieved by the concrete had an influence on the strength that the concrete achieved. In match-cured specimens tested at 24 days, the concrete strength decreased when the maximum concrete temperature was greater than 152°F (67°C). It was also reported that curing of concrete at elevated temperatures increased the strength of the concrete at one day but resulted in decreases of up to 30 percent in compressive strength at later ages when compared to strengths of concrete subjected to standard moist curing at 73° F (23° C). The higher the temperature at which the concrete was cured at early ages, the lower the relative strength gain at later ages.

In the Sarpy County HPC showcase bridge in Nebraska *[9]*, the specifications limited the steam temperature and the temperature at the centroid of the bottom flange of the girder to 160°F (71°C). In a trial pour for a length of girder, maximum measured temperatures were between 130 and 150° F (54 and 66° C) depending on location. Temperature of the concrete at time of placement was about 45° F (7° C). Because the high temperatures were found to be detrimental to development of long-term strengths, it was subsequently agreed with the precast concrete producer to limit the maximum temperature of the girder concrete to 80-90° F (23-32° C). As a result, actual strengths at 56 days exceeded the specified strength of 12,000 psi (83 MPa).

In the manufacturing of some test beams in Virginia *[10]*, it was found that the temperature of concrete cylinders stored in the enclosure surrounding a girder had significant temperature drop after the steam was cut off. When tested, these cylinders had lower strengths than cylinders that were cured to match the temperature of the girder.

During production of girders for the Louetta Overpass Bridge in Houston, nine castings were monitored using match-cured, member-cured and moist-cured concrete cylinders. Test results showed that the match-cured cylinders had the highest average compressive strength at detensioning. However, at 28 and 90 days, the moist-cured specimens had the highest strength. A similar pattern was observed with the modulus of elasticity values.

In the Washington State demonstration bridges, the temperature of the top flange increased more rapidly and had a higher maximum value than the temperature of the bottom flange. This, in turn, affected the strength development of the concrete, particularly for strand release. Careful consideration must therefore, be given to the location of the thermocouples that are used to monitor temperatures for the match-cured specimens.

Bridge Deck Durability

The Virginia Department of Transportation Special Provisions for Low Permeability Concretes *[45]* require bridge deck concrete to be moist cured for a minimum of 7 days and until 70 percent of the specified strength is reached. Burlap is to be maintained wet and covered with plastic sheeting. Immediately after removing the burlap and plastic sheeting, white pigmented curing compound is to be applied while the surface is still damp.

The Federal Highway Administration has developed a working definition of high performance concrete *[46]*. Four parameters in the definition that relate to durability are freeze/thaw resistance, deicer scaling, abrasion resistance and permeability. For each parameter, various grades are established based on the exposure conditions. The parameter that is used most frequently by state DOTs is chloride permeability which is measured using the test procedures of AASHTO T277 or ASTM C 1202.

The Virginia Department of Transportation Special Provisions for Low Permeability Concretes require a maximum laboratory permeability at 28 days of 1,500 coulombs for concrete used in prestressed concrete members, 2,500 coulombs for deck concrete and 3,500 coulombs for substructure concrete. Specimens for permeability testing are moist cured at 73°F (23°C) for one week followed by three weeks moist curing at 100°F (38°C). Permeability values obtained from trial batches are required to be 500 coulombs below the specified values. For bridge decks, the trial batches need to have a permeability of 2,000 coulombs. The same specification mandates the use of fly ash, granulated iron blast-furnace slag, silica fume or other approved mineral admixture with Types I, II (if above 0.40 percent alkali content) and III cements.

The Texas Department of Transportation Special Provisions for an HPC *[47]* bridge deck to be used for a redecking project requires that the permeability of the concrete shall be less than 2,000 coulombs when tested at 28 days in accordance with AASHTO T277.

Recommendations

Based on the literature review summarized in previous sections, the following recommendations are made.

1. Structural design of the Charenton Canal Bridge should be based on the *AASHTO Standard Specifications for Highway Bridges* with no modifications for HPC. This approach will not provide any economical benefits which may be achieved by modifying the design criteria but will provide a conservative approach.
2. Temperature control during production of the prestressed concrete piles and girders should be based on temperature of the concrete and not on temperature of the enclosure surrounding the members.
3. Maximum temperature of the concrete should be limited to 160°F (71°C).
4. Quality control cylinders for determining concrete strengths for release and acceptance should be match cured according to the temperature of the bottom flange of the girders.
5. The precast concrete fabricator should be given the option of achieving the specified concrete strength by 56 days.
6. Trial placements of concrete for the girders and deck should be made prior to production.
7. The specifications for the deck concrete should include a requirement for maximum permeability of 2,000 coulombs. Prescriptive requirements should be minimized.
8. The instrumentation program on the Charenton Canal Bridge should include measurements of girder temperatures during production, camber and deflections, strains and prestress losses.
9. A test program should be conducted to measure compressive strength, modulus of elasticity, modulus of rupture, coefficient of thermal expansion, creep, shrinkage and permeability of the high performance concrete at different ages.
10. Release strength of the concrete for the piles and girders should be between 60 and 80 percent of the specified design strength.
11. The deck concrete should be water cured for a minimum of seven days.
12. The precast concrete fabricator should be allowed to use 4x8-in. (102x203-mm) cylinders instead of 6x12-in. (152x305-mm) cylinders.

BRIDGE PLANS

The information contained in this chapter is based on the Advance Check Prints for the Charenton Canal Bridge dated October 1, 1997.

Bridge Description

The Charenton Canal Bridge is located in St. Mary Parish on Highway LA 87. The bridge is a 365-ft (111.3-m) long continuous structure consisting of five 73-ft (22.3-m) long spans. A 40-ft (12.2-m) long, 12-in. (305-mm) thick approach slab is provided at each end of the structure. The superstructure of the bridge consists of five prestressed concrete AASHTO Type III girders spaced at 10-ft (3.1-m) centers supporting an 8-in. (2.03-mm) thick reinforced concrete deck. The total width of the bridge deck is 46 ft 10 in. (14.3 m). The Type III prestressed concrete girders contain thirty-four 1/2-in (12.7-mm) diameter Grade 270 strands. Eight strands are debonded in pairs for various lengths at each end of the girders. Specified compressive strengths for the prestressed concrete girders as shown on the drawings are 7,000 psi (48 MPa) initial and 10,000 psi (69 MPa) final.

Negative moment continuity over the piers is provided by longitudinal reinforcement in the deck. No positive moment connection is provided. Diaphragms are provided at each abutment, over each pier and at midspan.

The substructure for the bridge consists of cast-in-place reinforced concrete bents supported on precast prestressed concrete piles. The eight piles at the end bents are 24-in. (610-mm) square. The four piles at the intermediate bents are 30-in. (760-mm) square.

Review Comments

The following comments are based on Sheet Nos. 101 through 142 of the Advance Check Prints. The purpose of the review is to identify only information relevant to the use of high performance concrete and to identify any inconsistencies between the plans and the *Proposed Special Provisions.*

Several sheets of the drawings include references to Class A (HPC) concrete for use in the end and intermediate bents. Class A (HPC) concrete and Class AA (HPC) concrete are considered to be the same concrete and are identified in the

Proposed Special Provisions as Class AA (HPC) concrete. The use of two different classes of concrete adds complexity to the project. However, the authors recognize that DOTD may decide to retain the two classes for purposes of quantities and costs.

Sheet No. 102

The last paragraph of the general notes on Sheet No. 102 addresses high performance concrete (HPC) and instrumentation. It is suggested that this paragraph be divided into two notes, since these are separate topics. The first note should address HPC. The second note should address instrumentation.

Concrete in the bridge deck should be added to the list of items for cast-in-place HPC concrete as shown in the last paragraph of the general notes. This list also includes the barrier rail. The *Proposed Special Provisions* of this report do not address the use of HPC in the barrier rail. The Class AA (HPC) concrete may not be suitable for this application if the contractor decides to slipform the barrier. Subsection 810.03 of the *Louisiana Standard Specifications for Roads and Bridges* requires the concrete for slip forming barriers to have a slump of 1/2 to 1-1/2 in. (25 to 40 mm). This is inconsistent with the required slump of 2 to 8 in. (50 to 200 mm) for Class AA (HPC) concrete.

The last paragraph of the general notes does not list the use of HPC in the wing walls. However, the quantities on Sheet No. 106 imply that Class A (HPC) concrete used in the bents is also used in the wing walls. There does not appear to be any reason to utilize HPC concrete in the wing walls.

The spelling of Specifications in the first paragraph and salvageable in the fourth paragraph from the end need to be corrected.

Sheet No. 110

The position for Section AA, as marked on the elevation drawing, is a section through the approach slab. The actual drawing of Section AA shows a cross section through the barrier slab. The location of the cross section needs to be moved to the correct location.

Sheet No. 115

This sheet indicates that the allowable camber for exterior and interior girders has a minimum value of 1-1/16 in. (27 mm) and a maximum value of 1-13/16 in. (46 mm). As indicated in Chapter 2 of this report, measured cambers of girders in Texas have indicated that the camber at release was less than would be calculated using normal design procedures. However, at time of erection, the camber was close to the calculated value. Consideration should be given to what measures are necessary if the actual girder camber is less than the minimum value. This may involve more precise calculation of camber such as using the ADAPT program *[38]* or accepting the girders with less than the specified camber.

Sheet No. 117

The structural design of the bridge girders is based on a concrete compressive strength at release of 6,000 psi (41 MPa) and a final design strength of 9,000 psi (62 MPa). However, on Sheet No. 117, these strengths are identified as 7,000 psi (48 MPa) initial and 10,000 psi (69 MPa) final. The increase of 1,000 psi (7 MPa) in the final strength may be justified on the basis that the long-term strength of the girders is important. It will also demonstrate that prestressed concrete girders can be produced with a specified strength of 10,000 psi (69 MPa). Increasing the strength at release from 6,000 to 7,000 psi (41 to 48 MPa) may impose an undue difficulty on the precast concrete fabricator. An increase of 1,000 psi (7 MPa) represents a significant increase in the required concrete strength at release. Therefore, the authors recommend that the strength at release be specified as 6,000 psi (41 MPa).

The typical strand pattern at the end of a girder shows a pattern of debonded strands for the girders. The pattern of debonding is consistent with recommendations given in Article 5.11.4.2 of the *AASHTO LRFD Specifications [24]*. The pattern shows that the debonded strands are positioned next to each other at the center of the beam in the two bottom rows. The authors recommend that consideration be given to staggering the debonded strands across the width of the girder so that all the debonded strands are not adjacent to each other. A symmetrical pattern of debonding should be retained. The number of debonded strands in each row should be retained.

Sheet No. 122

Sheet No. 122 contains standard notes for prestressed concrete girders. For this project, Paragraphs 3 and 8 need to be modified for the use with Class P (HPC) concrete. Paragraph 4 needs to be modified for use with debonded strands.

Sheet No. 126

Sheet No. 126 is a standard drawing for precast prestressed concrete piles. Paragraph 3 of the General Notes needs to be modified for use with Class P (HPC) concrete.

Paragraphs ten and eleven are inconsistent with the *Proposed Special Provisions* that require precast prestressed concrete members made with Class P (HPC) concrete not be shipped until an age of 28 days and only after the concrete has attained a compressive strength of 10,000 psi (69 MPa).

Recommendations

Based on the above review, the authors recommend that Class AA (HPC) concrete be used in the deck slab, approach slabs, barrier slabs and bents (excluding wing walls); and that Class P (HPC) concrete be used in the precast prestressed concrete girders and piles.

The use of Class AA (HPC) concrete in the barrier rail is currently shown in the drawings. However, this concrete may not be suitable if the contractor decides to slipform the barrier.

RECOMMENDED QUALITY CONTROL PROGRAM AND SPECIAL PROVISIONS

Introduction

Experience with the production of HPC bridges *[8], [9], [10], [11]* has indicated that production techniques used to produce normal strength prestressed concrete girders and cast-in-place concrete bridge decks may not be appropriate with HPC. This chapter describes several aspects related to HPC and the need to provide a higher degree of quality control for HPC bridges than is customary with conventional concrete. Although the increased quality control may have a cost impact on the construction costs of the first bridges built with HPC in a particular location, it is anticipated that the increased costs will diminish with subsequent projects as contractors become more familiar with the requirements of HPC production. In addition, the long-term goal is that any increases in construction cost will be more than offset by decreased maintenance costs and a longer life for bridge structures.

Precast Girders

Numerous projects have been conducted that demonstrate the benefits of using HPC with high compressive strength for precast, prestressed bridge girders. However, very few precast concrete fabricators have experience at producing HPC with high compressive strengths on a regular basis. Since the production of high-strength concrete requires the optimal use and careful selection of all constituent materials, it is extremely important that the quality of these materials not change during the production cycle of the girders. All materials used in girder construction must be the same as those used in the preproduction trial mixes. Substitution of alternate materials is not acceptable with high-strength concrete without verification of performance through trial mixes.

As with normal strength concretes, the specified release and design compressive strengths must be achieved in the final product. At the same time, the concrete must have adequate workability to facilitate placement. As indicated in Chapter 2, there is a trade-off between early strength gain and later-age strength gain. It is possible to achieve high release strengths through a combination of procedures such as the use of high curing temperatures and silica fume. However, slow strength gains at later ages may make it difficult to achieve the design strength. On the other hand, a combination of lower curing temperature and fly ash may make it easier to achieve the design strength but more difficult to achieve the release strength at an age that is acceptable to the producer.

In conjunction with developing a concrete mix design and prior to girder production, the precaster will need to establish a curing cycle that will allow the release and design compressive strengths to be achieved. Consequently, the mix proportion submittal by the precast concrete producer should include the proposed curing cycle as well as data showing the compressive strength versus time relationship.

For normal strength concretes, the required concrete strength at release has generally been the controlling factor in establishing the concrete mix proportions because of the desire to achieve a 24-hour production cycle. Achievement of the design compressive strength at 28 days has, generally, been easy, and, in many cases, the actual strengths have been well in excess of the specified design strength. With HPC, it can be more difficult to achieve the design strength because the strength gain at later ages may be slower. Consequently, it is proposed that the design strength be achieved at an age no later than 56 days. In addition, girders may be shipped as soon as the mix design strength is reached but no earlier than 28 days. This will provide the precaster with more flexibility in achieving the design strength.

Control of concrete temperature during the curing cycle is extremely critical. Since high-strength HPC contains more cementitious material than is used in normal strength concretes, the heat generated during hydration is greater. It is, therefore, important that the temperature of the concrete be controlled during the curing period. When radiant or steam-curing methods are used, temperature control of the heat or steam must be based on temperature of the concrete and not on the temperature of the enclosure surrounding the girders.

High-strength concretes can also produce sufficient heat of hydration that the addition of external heat may not be necessary. It is, therefore, appropriate in the specifications to refer to heat-cured concrete rather than steam-cured concretes because the heat curing may be generated within the concrete member.

Since the compressive strength versus time relationship is a function of concrete temperature, it is important that the quality control cylinders to be used for determination of release strength and design strength undergo a temperature-time history similar to that of the concrete member. This requires the use of match-curing procedures prior to release, irrespective of the type of curing. Following release, the test specimens should be stored alongside the girders according to normal practice in Louisiana.

Prior to production of the actual bridge girders, it is essential that the producer conduct a series of trial placements using the selected mix design and required curing cycle. These trials should be made in the plant using girder cross sections with the required amount of strand and reinforcement. It is assumed that, if the concrete can be placed in the girders without segregation and honeycombing, placement in the piles can also be accomplished without segregation and honeycombing. The placements are an integral part of the design mix development and serve to accomplish the following:

1. Verify that concrete of the required strength can be produced on a consistent basis.
2. Ensure that quality of the concrete in the girder is adequate.
3. Verify the selected curing cycle.
4. Verify that temperature sensors and match-curing systems work.

Cast-In-Place Deck

The primary performance requirement of an HPC bridge deck is that of durability. This requires a crack-free concrete with low permeability. Cracking caused by shrinkage can be minimized by using a concrete with a low water content to reduce the amount of shrinkage and by ensuring that the concrete develops higher tensile strength before drying begins. The latter requirement can be met through proper curing for sufficient length of time. A longer curing period allows for greater strength development and delays the start of drying shrinkage.

Low permeability can be achieved through the use of supplementary cementitious materials such as fly ash and silica fume. However, concretes containing fly ash or silica fume can be more difficult to finish when unfavorable weather conditions exist. It is, therefore, necessary to provide continuous fogging when dry or windy conditions exist.

Consequently, for high performance concrete, it is necessary to pay special attention to the mix proportions so that a low shrinkage, low permeability concrete is developed. Special attention must also be given to placing, finishing and curing the concrete so that cracking is minimized and a low permeability concrete surface is achieved. Prior to construction of the deck, the contractor should cast a test slab to demonstrate that the concrete mix can be placed and finished properly using the proposed concrete mix proportions, placing and finishing equipment and curing procedures.

Proposed Special Provisions

The following proposed special provisions are written so that they may be incorporated directly into the Contract Documents. They are written on the assumptions that Class P (HPC) concrete will be used in the piles and girders and Class AA (HPC) concrete will be used in the bent caps, bridge deck, approach slabs and barrier slabs.

Introduction. The design and construction of the Charenton Canal Bridge is part of a project to implement the use of High Performance Concrete (HPC) in Louisiana bridges. A research team headed by Tulane University and including Henry G. Russell, Inc. and Construction Technology Laboratories, Inc. (CTL) is assisting DOTD and Louisiana Transportation Research Center (LTRC) in the implementation program. The success of both the construction project and the research project requires that the researchers play an integral part in the construction process and that the contractor and subcontractors cooperate fully with the researchers. The following section describes special provisions required of the contractor and outlines the role of the researchers and LTRC in various aspects of the construction process. Details of the research program are given in "Implementation of High Performance Concrete in Louisiana Bridges - Interim Report."

Coordination of work with contractors. All aspects of the researchers' work shall be coordinated with the contractor. The contractor shall take all actions necessary to incorporate the research activities into the development of the construction schedule. The researchers shall cooperate with the contractor and shall minimize all delays.

Attendance at an open meeting prior to bid letting, where the researchers, DOTD and LTRC will give presentations on details concerning the HPC bridge, is mandatory for all contractors including the prestressed concrete girder fabricator and the deck concrete contractor bidding on this contract. After letting, a preconstruction meeting will be held with the contractor, pertinent subcontractors, researchers and sponsors.

At all times, including during construction, coordination between the contractors' and researchers' representatives will be required to ensure implementation of the necessary measures for design and control of HPC. The researchers will be provided access to the work area, will install the instrumentation and will be responsible for measurements. Facilities necessary for installing and protecting the instrumentation and equipment will be provided by the contractor.

Definition of high performance concrete (HPC). For this contract, the cast-in-place concrete used for the approach slabs, barrier slabs, deck slab and bent caps shall be high performance concrete Class AA (HPC). The precast concrete used in the prestressed concrete piles and girders shall be high performance concrete Class P (HPC).

High performance concrete mix development. The researchers and LTRC will provide technical expertise to assist the contractor in developing and evaluating the HPC mix design and curing cycle. The design and control of the HPC will be in accordance with the Standard Specifications, Special Provisions and Contract Plans.

Laboratory and field testing for research. During HPC girder and deck construction, HPC specimens in addition to those required by the specifications and contract plans will be made by the researchers and/or LTRC personnel. The contractor shall make the necessary provisions to allow sampling of the HPC as described in "Implementation of High Performance Concrete in Louisiana Bridges—Interim Report."

Concrete specimens made by the researchers at the precasting plant will be stored with the girders at the plant and at the bridge site. Concrete specimens made at the bridge site will be stored alongside the bridge deck at the bridge site. Contractor shall provide adequate space for storage and proper containers to protect cylinders from damage during shipping of specimens to CTL for testing. Contractor shall pay shipping costs.

Structural monitoring. The researchers have developed an instrumentation program to monitor the structural performance of the bridge and its components as described in "Implementation of High Performance Concrete in Louisiana Bridges—Interim Report." The contractor shall make available selected components and provide access to various locations to allow researchers to attach instrumentation and lead wires. It is planned to instrument four girders that will be cast in one bed at the same time. Instrumentation will also be installed in the deck. With proper planning and coordination, installation of instrumentation and data collection will not cause any significant delays to the contractor.

Louisiana Standard Specifications for Roads And Bridges. For this project, the *Louisiana Standard Specifications for Roads and Bridges*, 1992 Edition, is amended with respect to the subsections cited below:

<u>Subsection 105.05 Cooperation by Contractor</u>

Add the following paragraph:

The contractor shall provide access to selected components and access to various locations to allow researchers to install instrumentation and lead wires and to collect data. The precast concrete producer shall provide 110v electrical power at required locations for use by the researchers. The contractor shall make the necessary provisions to allow sampling of concrete by the researchers. The contractor shall provide adequate space for the manufacture and storage of test specimens at the precast plant and bridge site. The contractor shall be responsible for shipping test specimens to the researchers' facilities.

<u>Subsection 805.02 Materials</u>

Add the following class of concrete:

Concrete Class	Use
P (HPC)	High-strength concrete precast bridge members
AA (HPC)	High performance concrete cast-in-place superstructure

<u>Subsection 805.10 Curing</u>

Add the following paragraphs:

For Class AA (HPC) concrete used in the bridge deck, approach slabs and barrier slabs, the contractor shall comply with ACI 302—Guide for Concrete Floor and Slab Construction, ACI 308—Standard Practice for Curing Concrete and ACI 305—Hot Weather Concreting. As a minimum, if silica fume is used, the contractor shall under finish concrete by limiting finishing operations to screeding, bull floating and grooving. Continuous fogging above the surface of the concrete during the finishing operation shall be required. Fogging shall continue until the surface will support wet burlap without deformation. Free-standing water on the concrete surface prior to concrete final set shall not be allowed to occur.

As soon as the surface will support the burlap without deformation, apply pre-wetted burlap to the textured concrete surface. The concrete shall be kept continuously wet with a fog nozzle system or soaker hoses for seven curing days as defined in Subsection 805.11 and until a concrete compressive strength of 3,200 psi is reached. Materials, equipment and labor necessary for continuous curing will be supplied by the contractor. The use of polyethylene sheeting or plastic coated burlap blankets shall not be permitted.

The Project Engineer may require placement to be made at night or during early morning hours if satisfactory surface finish cannot be achieved. Weather conditions (current and forecasted) shall be within limits of Subsection 901.11.

Subsection 805.11 Removal of Falsework and Forms

Add the following to the second paragraph:

Supporting forms and falsework for HPC bridge decks, approach slabs and barrier slabs shall not be removed until both criteria determined by Methods 1 and 2 are met.

Add the following to Method 1:

Concrete Class	Compressive Strength (psi)
AA (HPC)	3,200

Subsection 805.13 (e) (1) Striking Off

Replace last paragraph with the following:

Addition of water to the surface of Class AA (HPC) concrete to assist in finishing shall not be permitted.

Subsection 805.14 (e) Curing

Revise as follows:

To establish adequacy of curing methods and to determine whether concrete has attained the required compressive strength, a minimum of eight test cylinders shall be made from the last batch of concrete and match cured under the same condition as the corresponding member. Three cylinders will be tested no later than 56 calendar days after casting to determine that the required strength has been achieved. The remaining five cylinders may be tested at any time as required by the contractor. However, no more than three cylinders will be tested in one day. If all five cylinders have been tested and concrete has not attained required strength, the members involved shall be held at the plant until the 56-day cylinders are tested. If the average 56-day concrete cylinder strength has not achieved the required strength, all members involved will be subject to rejection. Acceptance will be made in accordance with the Department's manual entitled "Application of Quality Assurance Specifications for Precast-Prestressed Concrete Plants." Curing methods other than heat curing shall be in accordance with Subsection 805.10. Hot weather concrete limitations as stipulated in Subsection 901.11(b) shall not be applicable for heat curing; however, precautions such as cooling of forms will be required.

Heat curing shall be done under a suitable enclosure to contain the heat in order to minimize moisture and heat losses. Initial application of heat shall begin only after concrete has reached its initial set as determined by ASTM C 403. When used, steam shall be at 100 percent relative humidity. Application of heat shall not be directly on concrete. During application of heat, concrete temperature shall be increased at a rate not to exceed 40°F per hour until the desired concrete temperature is achieved. The concrete temperature shall not exceed 160°F. Heat curing may continue until concrete reaches release strength. At the contractor's option, the application of heat may be reduced or discontinued to ensure that the concrete temperature does not exceed 160°F. If structural defects occur, the defective members will be rejected.

Contractor shall detension strands before the internal concrete temperature has decreased to 20°F less than its maximum temperature. Two recording thermometers showing time-temperature relationship in the concrete shall be furnished for each 200 ft of bed. For girders, one thermometer shall be located at the center of gravity of the top flange and one at the center of gravity of the bottom flange. For piles, one thermometer shall be located in the top half of the pile and one in the bottom half. Both thermometers shall be located midway between the outside corners of the pile and the nearest edge of the center void. If a void is not provided, only one thermometer shall be provided at the center of gravity of the cross section.

Subsection 805.14 (f) Transportation and Storage

Replace fourth paragraph with the following:

Prestressed members shall be held at the plant for at least 28 days after casting and until the concrete has attained the specified compressive strength. Specified compressive strength shall be attained no later than 56 days after casting.

Subsection 805.14 (g) Pretensioning Method

Replace the first paragraph with the following:

Prestressing strands shall be accurately held in position and stressed by approved jacks. A record shall be kept of the jacking force and tendon elongation produced. Several units may be cast in a continuous line and stressed at one time. Sufficient space shall be left between ends of members to permit access for cutting strands after concrete has attained required strength. Sufficient free strand shall be left in the line to ensure that cracking of the girders does not occur as the temperature of the girders decreases prior to detensioning of the strand. No bond stress shall be transferred to concrete nor shall end anchors be released until concrete has attained specified release strength as shown by cylinders made in accordance with DOTD TR 226 and match cured identically with members and tested in accordance with DOTD TR 230. Strand shall be cut or released in such order that lateral eccentricity of prestress will be a minimum in accordance with approved shop drawings.

Subsection 810.03 Construction, Fabrication, Erection and Painting

Replace the first sentence of the second paragraph with the following:

After completing the deck pour, a minimum of 7 days shall elapse before placing of reinforcing steel and forms for railings.

Subsection 901.01 General

Add a new paragraph as follows:

Concrete Class P (HPC) shall conform to the following requirements:

Average Compressive Strength at 56 days	10,000 psi
Slump	≤ 10 in.

Concrete slump shall be selected by contractor to ensure that concrete does not segregate.

Concrete Class AA (HPC) shall conform to the requirements of Table 1 and the following:

Permeability (Total Charge Passed) shall be less than or equal to 2,000 coulombs at 28 days.

If used, silica fume shall be added as early as possible in the concrete batching and as directed by a technical representative of the admixture supplier to ensure uniform distribution.

High-range water-reducers may be used to control slump, water/cementitious material ratio and proper distribution of fly ash or silica fume. Admixtures shall be plant added. Retempering at the jobsite, if necessary, will be permitted. Air entraining and set controlling admixtures may be used. All admixtures shall be compatible. Compatibility shall be demonstrated with trial batches. Admixtures containing chlorides shall not be used.

Specimens for compressive strength testing and permeability testing shall be manufactured by the contractor and supplied to LTRC for testing.

Subsection 901 Table 1

Add the following:

Structural Class AA (HPC)[m]	
Average Compressive Strength at 28 days	4,200 psi
Grade of Coarse Aggregate	A[n]
Minimum Bags of Cement (94 lb) per Cu Yd of Concrete[i]	7.0
Maximum Water per Bag of Cement[a i] (Gallons)	4.51[p]
Total Air Content (Percent by volume)[d]	5.5±1.5%
Slump Range (Inches)	2-8

m Cement type shall be I, IB or III conforming to Subsection 1001.01

n Aggregates shall conform to Subsection 1003.02.

p Water content shall include weight of water, if any, in the admixtures. Cement content shall include all mineral admixtures.

Revise footnote i as follows:

i For mixes containing combinations of cement, fly ash and silica fume, the minimum cement and maximum water contents shown shall apply to the total cement/fly ash/silica fume content of the mix.

Subsection 901.02 Materials

Add the following:

The use of silica fume conforming to AASHTO M307 with the exception of LOI which shall not exceed 6.0 percent or ASTM C 1240 shall be permitted.

Subsection 901.06 Quality Control of Concrete

Add the following paragraph:

A representative of the admixture manufacturer shall be present for batching start up and during initial concrete placement.

Subsection 901.06 (a) Mix Design

Add the following paragraphs:

For Class P (HPC) concrete, the contractor shall make two demonstration trial batches, of at least 3 cu yd, on separate days at the prestressed concrete girder plant to show that the girder concrete sections can be cast with the proposed mix design. Materials used in concrete batches shall be identical to those that will be used in production. These demonstration batches and girders shall be made sufficiently before production girders are cast to demonstrate that design compressive strength can be achieved. Cylinders shall be made and match cured with the girder section. The cylinders shall be cured and tested in the same manner as acceptance cylinders in a production mode. The design trial batch shall meet the minimum design compressive strength before mix design approval will be given. Test results for slump, air content, wet unit weight and compressive strengths at concrete ages of 1, 3, 7, 28 and 56 days shall be submitted. The verified time-temperature history of the concrete during the initial curing period shall be submitted. If requested, the contractor shall furnish materials to the Department for verification of trial mixes.

For Class AA (HPC), the concrete producer shall make trial batches as necessary to determine the proportions of the basic ingredients as well as the amount and proper sequencing of admixtures to produce the required concrete mix. Specimens for compressive strength testing and permeability testing shall be manufactured by the contractor and supplied to LTRC for testing. At least 28 days prior to placement of Class AA (HPC) for the bridge deck, contractor shall construct a test slab 12x30 ft. The test slab shall be constructed using the proposed Class AA (HPC) concrete and shall be finished and cured in accordance with the proposed procedures for the bridge deck. The Materials Engineer will approve the mix design when trial batching and test slab demonstrate the desired results.

The Contractor shall strictly adhere to the manufacture's written recommendations regarding the use of admixtures, including storage, transportation and method of mixing.

Subsection 901.07 Substitutions

Add the following:

P (HPC) No substitutions

AA (HPC) No substitutions

Subsection 901.08

Add a new subsection as follows:

901.08 (g) Permeability

Permeability of concrete shall be determined in accordance with AASHTO T277 or ASTM C 1202. The permeability samples shall have a 4-in. diameter and a length of at least 4 in. They shall be moist cured until testing at 28 days after casting. The average value of three specimens shall be reported.

Subsection 901.08 (a) Cement and Aggregates

Add the following paragraphs:

For Class P (HPC) and Class AA (HPC) concretes, the contractor will be permitted the use of silica fume to a maximum of 10 percent by weight of the total combination of cement, fly ash and silica fume.

For Class P (HPC) concrete, the contractor will be permitted the use of fly ash with Type I, I(B), I(C), II or III portland cement up to a maximum of 35 percent by weight for the total combination of cement, fly ash and silica fume.

For Class AA (HPC) concrete, the contractor will be permitted the use of fly ash with Type I, I(B) and III portland cement up to a maximum of 30 percent by weight of the total combination of cement, fly ash and silica fume. A combination of fly ash and silica fume may be used with the total substitution by weight not to exceed 30 percent of the total combination of cement, fly ash and silica fume.

Subsection 901.08 (f) (1) Structural Concrete

Add the following paragraph:

Cylinders by which strength of Class P (HPC) concrete is to be determined shall be cured using the match-curing technique until detensioning of the strand. Thereafter, cylinders shall be cured alongside the members that they represent. For girders, thermocouples for use with the match-curing system shall be placed within 1 in. of the center of gravity of the bottom flange. For piles, thermocouples for use with the match-curing system shall be placed at the center of gravity of the cross section when a void is not present or midway between the outside corner of the pile and the nearest edge of the void in piles with voids.

Subsection 901.12 Acceptance and Payment Schedule

Add the following paragraph:

Acceptance and payment for Class AA (HPC) concrete shall be in accordance with the schedule in Table 2 for Class AA concrete except the concrete will not be accepted and shall be removed if the specified 28-day compressive strength is not achieved by 56 days.

Subsection 1003.02 (a) Fine Aggregate

Add the following sentence at the end:

For Class P (HPC) concrete, other gradations of concrete sand will be permitted if demonstrated in trial mixes to produce the required concrete properties and accepted as part of the proposed mix designs.

Subsections 1003.02(b) Coarse Aggregate

Add the following sentence at the end:

For Class P (HPC) concrete, other gradations of uncrushed and crushed coarse aggregate will be permitted if demonstrated in trial mixes to produce the required concrete properties and accepted as part of the proposed mix design.

Subsection 1009.05 Steel Strand for Pretensioning

Add the following sentence:

The contractor shall obtain certification from the strand supplier that the strand will bond to concrete of a normal strength and consistency in conformation with the prediction equations for transfer and development length given in the *AASHTO Standard Specifications for Highway Bridges.*

TR 226M/226-95

For this project, DOTD Designation: TR 226M/226-95 is amended with respect to the following: Part II.

Apparatus

At the end of the first paragraph of A. Cylinder molds, add the following:

Match-cured cylinders shall have an inside diameter of 100 mm (4 in.) and a length of 200 mm (8 in.).

Add the following new section:

3. Match-cure molds - Sure Cure Cylinder Mould System from Products Engineering.

Section IV. A. Compression Test Specimens

Add the following new section:

1.b. Match-cure molds - Follow manufacturer's instructions.

TR 230M/230-95

For this project, DOTD Designation: TR 230M/230-95 is amended with respect to the following:

Section II. G. Testing Machines

Add the following at the end of the second paragraph:

For testing Class P (HPC) concrete, the testing machine shall have been calibrated within 6 months prior to the time of testing.

Part IV. Sample

Add the following paragraph:

Match-cure cylinders shall be molded to have a diameter of 102 mm (4.0 in.) and a nominal height of 203 mm (8 in.).

Section V. B. Determining the Cross-Sectional Area

Add a new section as follows:

3. For match-cured cylinders, determine cross-sectional area in accordance with V. B. 2.

Section V. D. Determining Compressive Strength

Add the following to the first paragraph:

Neoprene caps with a durometer hardness of at least 70 shall be used for testing Class P (HPC) concrete.

INSTRUMENTATION PLAN

Introduction

The application of high performance concrete in highway structures has been receiving increased attention in recent years. The actual construction of bridges with high performance concrete is providing opportunities to learn more about the production of high performance concrete as well as the actual behavior of high performance concrete bridges. Construction of the Charenton Canal Bridge provides a unique opportunity to learn about the performance of a bridge built in Louisiana with local materials. Consequently, it is proposed to instrument the structure to determine its performance both during and after construction. Information from this instrumentation program can then be used to refine the design and construction procedures and specifications for bridges built of high performance concrete in Louisiana.

The Instrumentation Plan is based on the following assumptions:

1. Instrumentation will be installed in a total of four girders.
2. Three interior girders and one exterior girder in one span of the bridge will be instrumented.
3. The instrumented girders will be cast at the same time in the same prestressing bed.
4. A limited amount of instrumentation will be placed in the deck.
5. The specific span and girders to be instrumented will be determined after the contract is awarded.

The instrumentation will be used to monitor girder curing temperatures, prestressing forces, prestress losses, strains and deflections. Details of each type of instrumentation are given in the following sections.

Girder Curing Temperatures

As discussed in the literature review, concrete temperatures during initial curing are extremely critical. Since high-strength concretes contain more cementitious material than used in normal strength concretes, the heat generated during hydration is greater. Since this heat does not escape from the concrete, the temperature of the concrete increases. This, in turn, affects the development of concrete compressive strength and related material properties.

The temperature rise within a concrete section during initial curing depends on the heat generated by hydration of the cementitious materials, the size of the cross section, insulation properties of the formwork and coverings and the external temperature. Little is known about the effects produced by variations of temperature within cross sections as a result of heat of hydration. The effect of interaction between the heat of hydration, application of external heat and rate of heat loss is also an area of needed research.

The objectives of the instrumentation to measure concrete curing temperatures are as follows:

1. Measure the variation with time of concrete temperatures from heat of hydration and curing environment.
2. Measure the variation of concrete temperatures in a girder cross section.
3. Measure the longitudinal variation of concrete temperatures of a girder.
4. Measure the variation of concrete temperatures between girders in the same bed.

The objectives of the measurements will be accomplished by installing thermocouples in four girders, as shown in Figure 5. Nine thermocouples will be located at midspan, one quarter point and one end of Girder 1 for a total of 27 thermocouples. Three thermocouples will be located at midspan in Girders 2, 3 and 4 for a total of nine thermocouples. In addition to the thermocouples in the concrete, one thermocouple will be installed outside the formwork and underneath the protective covering at midspan of each of the four girders and one thermocouple will be used to monitor outside air temperature, for a total of five thermocouples. Also, thermocouples will be placed in three concrete cylinders that will be cured alongside each of the four prestressed concrete girders for a total of 12 thermocouples. The complete program will require 53 thermocouples.

Prestressing Forces

Although calibrated hydraulic jacks are used to tension prestressing strands, the jacks only provide a measurement of the force before they are released. Transfer of force from the hydraulic jack to the strand anchorage and the subsequent increase in strand temperature during initial curing, result in a decrease in strand force prior to release. Consequently, when the strands are released, the force applied to the prestressed concrete girder will be less than the force applied to

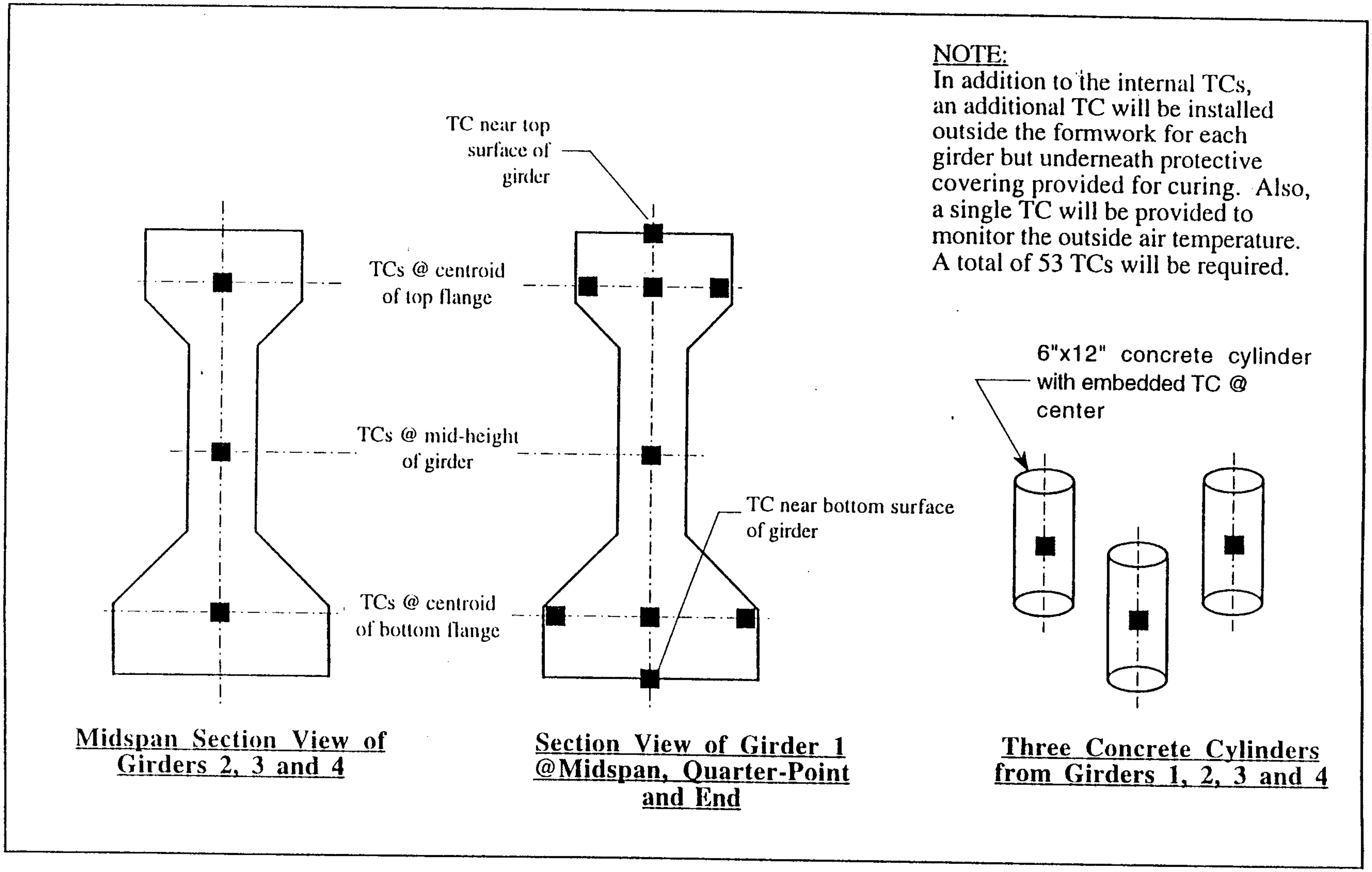

Figure 5
Thermocouple (TC) instrumentation

the strands initially by the prestressing jack. It is anticipated that the decrease in force may be greater with high performance concretes due to the higher temperatures. Research work on high performance concrete girders in Texas *[37]*, has indicated that measured camber of the girders has been less than calculated. One contributing factor may be the force in strands prior to release. In addition, in the previous project for LTRC *[1]*, a large crack was observed through the full depth of each of three girders after form removal. The crack widths were widest at the top flange and gradually diminished towards the bottom flange. The presence of these cracks and the variation in crack widths could be attributed to an increase in the prestressing force caused by the decreasing temperature of the girder prior to release of the strands.

Measurements will be made to determine the change of force in the strand from time of tensioning, during curing and until the strands are detensioned. The most accurate method to determine prestressing forces prior to release is with load cells. As shown in Figure 6, six load cells will be positioned on strands at either the dead end or jacking end in the prestressing bed.

Prestress Losses

Previous research *[1]* has indicated that prestress losses in high performance concrete girders are less than the losses in girders with normal strength concretes. However, additional data are required to justify a reduction in the prestress losses currently assumed in design. It is, therefore, proposed to instrument Girders 1 through 4 to determine prestress losses caused by elastic shortening, creep and shrinkage. Measured values will be compared with design assumptions. The prestress losses will be measured using vibrating wire strain gages. Three gages will be placed in each of four girders. The gages will be placed at midspan and at the level of the centroid of the strand group at the approximate locations shown in Figure 7. A total of twelve vibrating wire strain gages will be installed. Each vibrating wire strain gage will be equipped with a sensor to measure temperature at the gage location.

Four vibrating wire strain gages will also be placed in 6x12-in. (152x305-mm) concrete cylinders to provide a calibration curve for the effect of temperature on the apparent strain during the initial curing period. One cylinder will be made for each

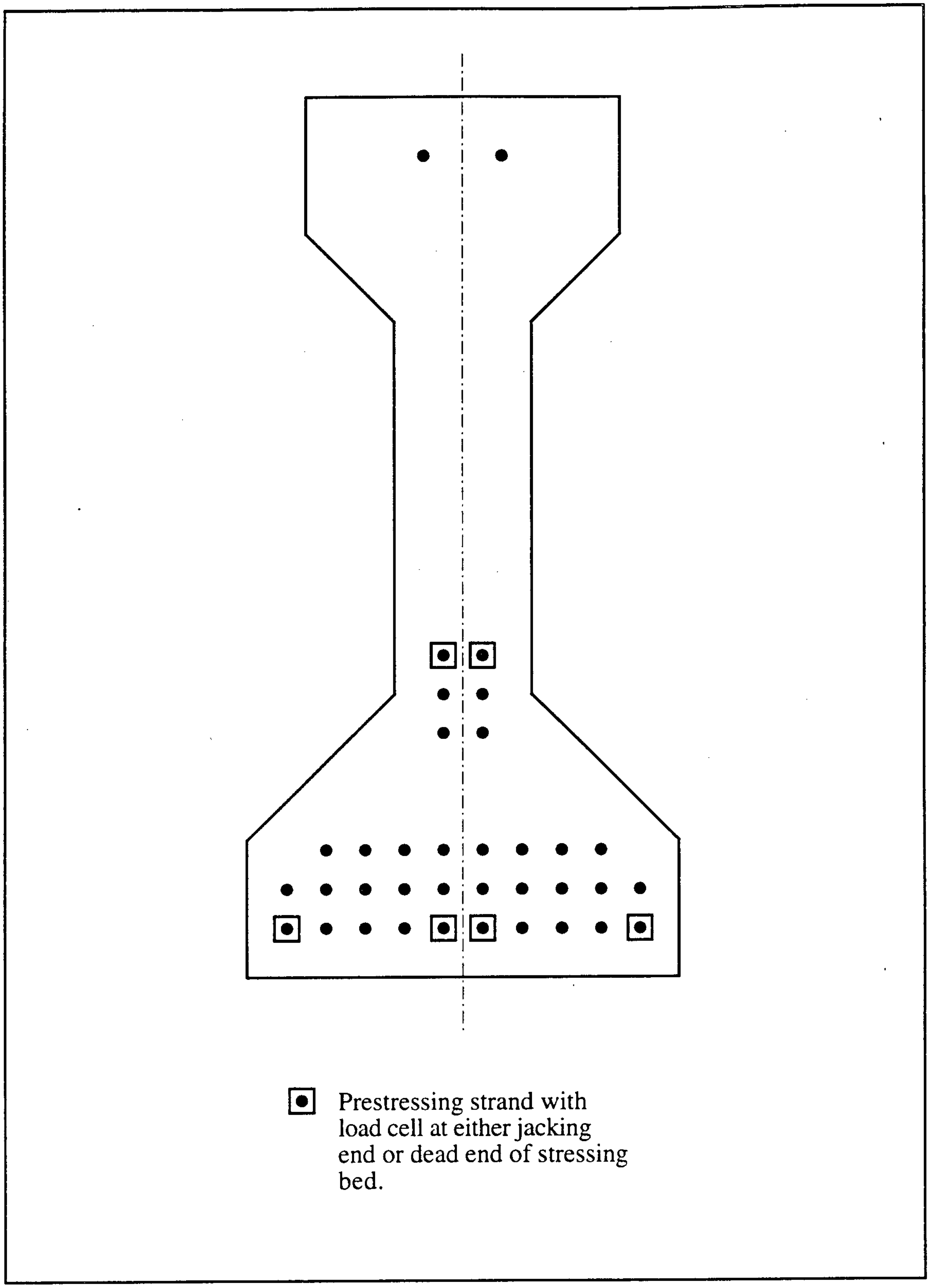

Figure 6
Location of load cells

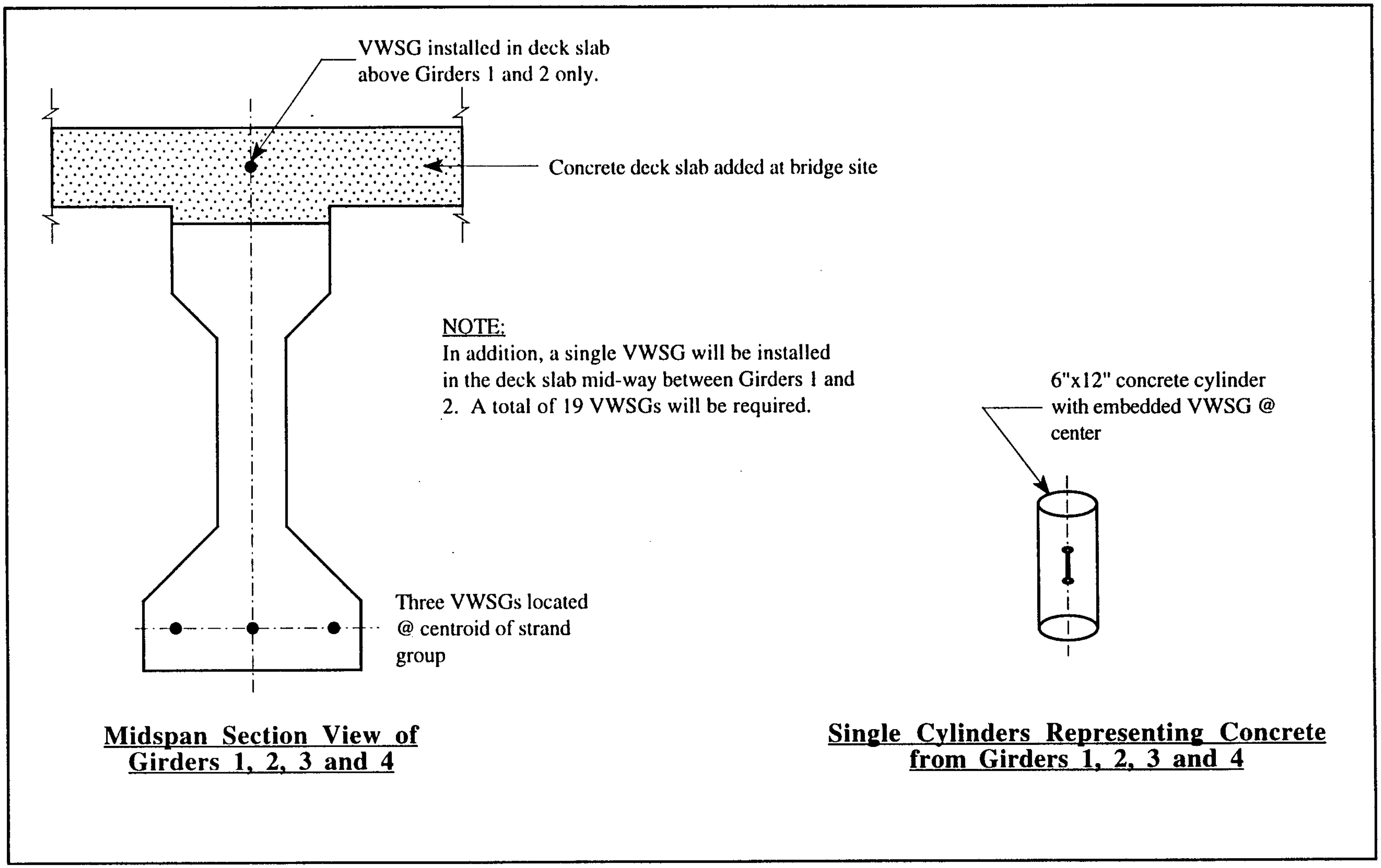

Figure 7
Vibrating wire strain gage (VWSG) instrumentation

instrumented girder. Each cylinder will be cured alongside the corresponding girder under any enclosure that is placed over the girders. The same cylinder will be used in the concrete materials testing program to determine the coefficient of thermal expansion.

Deck Strains

Strains in the concrete deck will be measured at three locations using vibrating wire strain gages. All three gages will be installed at mid-depth of the deck at midspan. One gage will be installed above each center line of two instrumented interior girders. The third gage will be installed midway between the girders. These gages will measure strains in the concrete deck caused by the combined effects of shrinkage and creep of the deck and girders.

Deflections

As mentioned previously, prestress losses with high performance concretes are likely to be less than with normal strength concretes. As a result, camber and long-term deflections may be different from those predicted using the properties of normal strength concretes. Modifications of design procedures and assumptions may be needed to provide better prediction of long-term camber. It is, therefore, proposed to measure midspan deflections relative to each girder end on Girders 1 through 4. Immediately after casting and while the concrete is still plastic, steel bolts will be embedded in the top surface of each girder at midspan and near both ends to provide permanent fixed reference points for camber measurements. The embedded bolts near each end will be centered above the sole plate. Camber measurements will be made using a level to sight elevations at each reference point.

DATA COLLECTION PLAN

Precasting Plant

Automated data acquisition systems will be utilized in the precasting plant to monitor thermocouples, load cells and vibrating wire strain gages. The acquisition systems will be connected prior to casting the concrete girders and will collect data until the girders are moved from the precasting bed. The data acquisition systems will be programmed to collect data every fifteen minutes during this period.

Prior to moving the girders from the prestressing bed, the instrumentation in Girders 2, 3 and 4 will be disconnected from the data acquisition systems. After moving these girders to the storage area, readings on selected gages will be taken using manual readout boxes. If practical, the data acquisition system will remain connected to the instrumentation in Girder 1 as long as possible. Thereafter, readings will be taken using manual readout boxes.

Manual readings of thermocouple data will continue on a daily basis until the girder temperature is within 10°F (5.5°C) of ambient temperature. Readings of load cells will be discontinued after detensioning of the prestressing strands. The vibrating wire strain gages will be read manually before and after the girders are moved to storage, once a day for the first week, once a week for the first month, once a month until the girders are moved to the bridge site and immediately before the girders are moved to the bridge site. A manual readout box will be provided by Construction Technology Laboratories (CTL) for this purpose.

Deflections will be measured using surveying techniques. Readings will be taken before and after release of the prestressing strands, before and after the girders are moved to storage, once a day for the first week, once a week for the first month, once a month until the girders are moved to the bridge site and immediately before the girders are moved to the bridge site. Since girder camber changes during the day as the top surface of the girder becomes hotter, readings should be taken in the early morning before the girders are exposed to sunlight.

Bridge Site

Following erection of the precast girders at the bridge site, lead wires from the vibrating wire strain gages in each of the four girders will be routed through the concrete deck to a convenient location at the side of the bridge. It is anticipated that this location will be on the span with the four instrumented girders. This will facilitate manual readings of the instrumentation. Strain gage leads will be tied to the underside of reinforcing steel used in the deck.

Prior to casting the concrete deck, the deflection reference bolts will be extended by the research team to the top surface of the deck so that deflection readings can continue at the same locations after the deck is cast.

Vibrating wire strain gages and deflection measurements will be taken after erection of the girders, before and after the deck is cast, once a week for the first month after casting the deck, once a month for the next two months and then every three months for the duration of the project.

Concrete Material Properties

A test program to measure compressive strength, tensile strength, modulus of elasticity, coefficient of thermal expansion, creep, shrinkage and permeability of the concrete will be conducted. To minimize project costs and to ensure timely testing after detensioning of the strands, it is proposed that some of the testing be performed by LTRC. This test program will be in addition to that required by the precast girder producer for quality control.

Girder concrete

A test program to determine the material properties of the concretes used in the instrumented girders is shown in Table 3. Concrete compressive strength per AASHTO T23 and modulus of elasticity per ASTM C 469 will be measured on test specimens exposed to two different methods of initial curing. These are designated as match curing and field curing.

Match-cured 4x8-in. (102x203-mm) cylinders will be produced using the Sure Cure Cylinder Mould System. It is anticipated that the equipment for the match-cured specimens will be provided on loan from the Federal Highway Administration. In the event that the equipment is not available from FHWA, it is anticipated that the equipment will be provided by LTRC or the precast concrete producer. The Federal Highway Administration has a total of seven controllers and 28 moulds available. The proposed program will use seven controllers and 19 moulds. Following the initial curing period, the specimens will be stripped from their moulds and stored alongside the girders until a few days before the age of testing. One specimen from each of the four instrumented girders will be tested at release, 7, 28 and 90 days and one specimen from each of three girders will be tested at 56 days for compressive strength and modulus of elasticity. The same specimens will be used for compressive strength and modulus of elasticity. Tests will be conducted by LTRC.

The field-cured specimens for measurement of compressive strength will be 6x12-in. (152x305-mm) cylinders cured alongside the prestressed concrete girder under any enclosure that is placed over the girders. This curing procedure represents the current standard Louisiana DOTD curing procedure for Class P and P(M) concretes. Following the initial curing period, field-cured cylinders will be stripped and stored alongside the girders until a few days before the age of testing. To minimize the number of required cylinders, it is proposed to utilize the same cylinders for measurement of modulus of elasticity and concrete compressive strength. Five cylinders will be needed from each of the four instrumented girders for the measurement of compressive strength and modulus of elasticity. Tests will be made by LTRC.

The modulus of rupture will be measured on a total of 16 beam specimens. These specimens will be cured initially alongside the prestressed concrete girder. Following the initial curing, the beams will be stripped and stored alongside the prestressed concrete girders until a few days before the age of testing. Forms will be supplied and tests will be made by LTRC in accordance with ASTM C 78. Tests at release will be made in the plant using LTRC equipment.

The coefficient of thermal expansion of concrete is required for the correction of measured strains for the effects of temperature. The coefficient of thermal expansion will be measured on one cylinder from each of the four instrumented girders in accordance with CRD C-39. Measurements will be made as soon as possible after release and at 28 and 90 days. The same four specimens will be used for measurements at the different ages. Tests will be made by CTL.

Table 3
Concrete materials testing program—girders

Material Property	Initial Curing	Number of Tests at Each Age, days						Total Specimens
		R[1]	7	28	56	90	S[2]	
Compressive Strength—LTRC 5 cylinders from each of 3 girders 4 cylinders from 1 girder	Match	4	4	4	3	4		19
Compressive Strength—LTRC 5 cylinders from each of 4 girders	Field	4	4	4	4	4		20
Modulus of Elasticity—LTRC 5 cylinders from each of 3 girders 4 cylinders from 1 girder	Match	4	4	4	3	4		19[3]
Modulus of Elasticity—LTRC 5 cylinders from each of 4 girders	Field	4	4	4	4	4		20[3]
Modulus of Rupture—LTRC 4 beams from each of 4 girders	Field	4	4	4		4		16
Coef. of Thermal Expansion—CTL 1 cylinder from each of 4 girders	Field	4		4		4		4
Creep and Shrinkage—CTL 8 cylinders from each of 2 girders — Creep	Field	2		2		2	4[4]	10
Shrinkage	Field	2		2		2		6
Permeability—LTRC 1 cylinder from each of 4 girders	Field			4				4

1. R = Release or as early as possible.
2. S = Spare
3. Same specimens as compressive strength.
4. Three spares to be used at end of creep frames.

Totals

Match-cured cylinders: 5 from each of 3 girders and 4 from 1 girder = 19 cylinders

Field-cured cylinders: 15 from each of 2 girders and 7 from each of 2 girders = 44 cylinders

Field-cured beams: 4 from each of 4 girders = 16 beams

Creep and shrinkage tests, in accordance with ASTM C 512, will be conducted by CTL on cylinders from two of the four instrumented girders. Only two girders have been selected for these measurements due to the size of the available creep frames and costs of conducting creep tests. Measurements will be made on 6x12-in (152x305-mm) cylinders that are field cured alongside the precast concrete girders. The tests will begin at three different concrete ages. The earliest age will correspond to the earliest time that the cylinders can be shipped to CTL and prepared for testing. These cylinders will need to be shipped overnight and will probably not be loaded until an age of three days. Other tests will commence at concrete ages of 28 and 90 days. These specimens will also be stored alongside the girders until it is necessary to ship them to CTL. Measured compressive strength and modulus of elasticity at the ages of loading will be supplied to CTL by LTRC.

Costs for packaging and shipping cylinder specimens to CTL will be included in the Special Provisions for the concrete girder production.

Permeability tests in accordance with AASHTO T 277 will be conducted by LTRC on samples of concrete cut from one cylinder from each of the four instrumented girders. These cylinders will be field cured alongside the prestressed concrete girders.

The overall program will require the manufacture of 19 match-cured cylinders, 44 field-cured cylinders and 16 field-cured beams.

Deck concrete

A test program to determine the material properties of concrete used in the deck of the Charenton Canal Bridge is shown in Table 4. Concrete compressive strength, modulus of elasticity and coefficient of thermal expansion will be determined at concrete ages of 7, 28 and 90 days on concrete from the span that incorporates the instrumented girders.

Permeability will be measured at an age of 28 days on concrete cut from one cylinder representing each of the five spans of the structure. All other tests will be made on 6x12-in. (152x305-mm) cylinders. All tests of the deck concrete except the coefficient of thermal expansion will be performed by LTRC. The program will require the manufacture of 17 cylinders to be cured at the bridge site.

Table 4
Concrete materials testing program—decks

Material Property	**Number of Tests at Each Age, days**				**Total Specimens Required**
	7	28	56	90	
Compressive Strength—LTRC 9 cylinders from 1 span	3	3		3	9
Modulus of Elasticity—LTRC 9 cylinders from 1 span	3	3		3	9(1)
Coefficient of Thermal Expansion—CTL 3 cylinders from 1 span	3	3		3	3
Permeability—LTRC 1 cylinder from each of 5 spans		5			5

1. Same specimens as compressive strength.

Totals

Field-cured cylinders: 13 from one span and
1 from each of 4 spans = 17 cylinders

RECOMMENDATIONS

Based on the information presented in this report, the following recommendations are made:

1. Design of the Charenton Canal Bridge should be based on the *AASHTO Standard Specifications* with no modifications for high performance concrete.
2. Specified strength of high performance concrete in the girders and piles should be 10,000 psi (69 MPa) at 56 days.
3. Specified permeability of high performance concrete in the deck should be at least 2,000 coulombs at 28 days.
4. Compressive strength of the concrete in the girders and piles should be determined using match-cured specimens.
5. The Quality Control Program and Special Provisions listed in this report should be adopted for the Charenton Canal Bridge.
6. The instrumentation and data collection plan described in this report should be implemented.

NOTATIONS

E_c = modulus of elasticity, psi

f'_c = concrete compressive strength, psi

f'_{ci} = concrete compressive strength at release, psi

C_t = creep at time t, millionths/psi

C_u = final value of creep, millionths/psi

t = time under load, days

w_c = unit weight of concrete, pcf

$$k = \frac{MOR}{\sqrt{f'_c}}$$

MOR = modulus of rupture

REFERENCES

1. Bruce, R. N., Russell, H. G., Roller, J. J. and Martin, B. T., "Feasibility Evaluation of Utilizing High Strength Concrete in Design and Construction of Highway Bridge Structures," Final Report, Louisiana Transportation Research Center, Research Report No. FHWA/LA-94-282, Baton Rouge, LA, 1994, 168 pp.
2. *Louisiana Standard Specifications for Roads and Bridges,* 1992 Edition, State of Louisiana, Department of Transportation and Development, Baton Rouge, 1992.
3. ACI Committee 363, "State-of-the-Art Report on High-Strength Concrete (ACI 363R-92)," American Concrete Institute, Detroit, 1984.
4. ACI Committee 363, "State-of-the-Art Report on High-Strength Concrete (ACI 363R-92)," American Concrete Institute, Detroit, 1992.
5. Zia, P., Leming, M. L. and Ahmad, S. H., "High Performance Concretes: A State-of-the-Art Report," Report No. SHRP-C/FR-91-103, Strategic Highway Research Program, National Research Council, Washington, DC, 1991.
6. Zia, P., Ahmad, S. H. and Leming, M. L., "High Performance Concretes: An Annotated Bibliography (1989-1994)," Report No. FHWA-RD-96-112, FHWA, U. S. Department of Transportation, Washington, DC, 1996.
7. Zia, P., Ahmad, S. H. and Leming, M. L., "High Performance Concretes, A State-of-the-Art Report (1989-1994)," Report No. FHWA-RD-97-030, FHWA U. S. Department of Transportation, Washington, DC, 1997.
8. SHRP High Performance Concrete Bridge Showcase, Houston, TX, March 25-27, 1997.
9. SHRP High Performance Concrete Bridge Showcase, Omaha, NE, November 18-20, 1996.
10. SHRP High Performance Concrete Bridge Showcase, Richmond, VA, June 24-26, 1997.
11. SHRP High Performance Concrete Bridge Showcase, Belleview, WA, August 18-20, 1997.
12. *Precast/Prestressed Concrete Bridge Design Manual,* Precast/Prestressed Concrete Institute, Chicago, IL, 1997.

13. Patton, B. B., "Fabrication of Prestressed Concrete Beams for Two High Performance Concrete Bridge Projects in Texas," *Symposium Proceedings*, PCI/FHWA International Symposium of High Performance Concrete, New Orleans, LA. October 20-22, 1997, Precast/Prestressed Concrete Institute, Chicago, IL, 1997, pp. 488-503.
14. Cook, J. E., "10,000 psi Concrete," *Concrete International*, Vol. 11, No. 10, Oct. 1989, pp. 67-75.
15. Burg, R. G. and Ost, B. W., "Engineering Properties of Commercially Available High-Strength Concretes," PCA Research and Development Bulletin RD104T, Portland Cement Association, Skokie, IL, 1992.
16. Carino, N. J., Guthrie, W. F. and Lagergren, E. S., "Effects of Testing Variables on the Measured Compressive Strength of High-Strength (90 MPa) Concrete," NISTIR 5405 National Institute of Standards and Technology, Gaithersburg, MD, Oct. 1994, 141pp.
17. Day, R. L., "Strength Measurements of Concrete Using Different Cylinder Sizes: A Statistical Analysis," *Cement, Concrete and Aggregates,* Vol. 16, No. 1. June, 1997, pp. 21-30.
18. CSA A23.1-94 *Concrete Materials and Methods of Construction*, Canadian Standards Association, Rexdale, ON, 1994.
19. Bartlett, F. M. and MacGregor, J. G., "In-place Strength of High-Performance Concretes," *High-Strength Concrete - An International Perspective,* Publication SP-167, American Concrete Institute, Detroit, 1996, pp. 211-228.
20. Bickley, J. A., Ryell, J., Rogers, C. and Hooton, R. D., "Some Characteristics of High-Strength Concrete: Part 2," *Canadian Journal of Civil Engineering*, Vol. 21, No. 6, 1994, pp. 1084-1087.
21. Aitcin, P. C. and Riad, N., "Curing Temperature and Very High-Strength Concrete," *Concrete International*, Vol. 10, No. 10, Oct. 1988, pp. 69-72.
22. Committee 318, *Building Code Requirements for Structural Concrete*, American Concrete Institute, Detroit, 1995.
23. *Standard Specifications for Highway Bridges*, sixteenth edition, American Association of State Highway and Transportation Officials, Washington, DC, 1996, 677 pp.
24. *AASHTO LRFD Bridge Design Specifications,* first edition, American Association of State Highway and Transportation Officials, Washington, DC, 1994.

25. Mokhtarzadeh, A., Kriesel, R., French, C. and Snyder, M. B., "Mechanical Properties and Durability of High Strength Concrete for Prestressed Bridge Girders," Paper No. 950873, Transportation Research Board, Jan. 1995, 42 pp.
26. Shing, B. B., Cooke, D., Fangopol, D. M., Leonard, M. A., McMullen, M. L. and Hutter, W., "Colorado Showcase on HPC Box-Girder Bridge: Development and Transfer Length Tests," *Symposium Proceedings*, PCI/FHWA International Symposium on High Performance Concrete, New Orleans, LA. October 20-22, 1997, Precast/Prestressed Concrete Institute, Chicago, IL, 1997, pp. 705-716.
27. Martinez, S., Nilson, A. H. and Slate, F. O., "Spirally Reinforced High-Strength Concrete Columns," Research Report No. 82-10, Department of Structural Engineering, Cornell University, Ithaca, NY, Aug. 1982.
28. *CSA A23.3-94 Design of Concrete Structures*, Canadian Standards Association, Rexdale, ON, 1994.
29. Ahmad, S. H. and Shah, S. P., "Structural Properties of High Strength Concrete and its Implications for Precast Prestressed Concrete," *PCI Journal,* Vol. 30, No. 6, Nov./Dec. 1985, pp.92-119.
30. Pauw, A., "Static Modulus of Elasticity of Concrete as Affected by Density," *ACI Journal*, Vol. 32, No. 6, Dec. 1960, *Proceedings*, Vol. 57, Paper No. 32, pp. 679-687.
31. Khan, A. A., Cook, W. D. and Mitchell, D. "Early Age Compressive Stress-Strain Properties of Low-, Medium, and High-Strength Concretes," *ACI Materials Journal*, Vol 92, No. 6, November-December 1995, pp. 617-624.
32. Hanson, J. A., "Prestress Loss as Affected by Type of Curing," PCA Development Bulletin D75, Portland Cement Association, Skokie, IL, 1964.
33. ACI Committee 209, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures (ACI 209-92)," American Concrete Institute, Detroit, 1992.
34. Oesterle, R. G., Gliken, J. D. and Larson, S. C., "Design of Simple-Span Precast Prestressed Girders Made Continuous," National Cooperative Highway Research Program, Report No. 322, Transportation Research Board, Washington, DC, 1989, 97 pp.
35. Ahlborn, T. M., French, C. W. and Leon, R. T., "Application of High-Strength Concrete to Long-Span Prestressed Bridge Girders," Paper No. 950400, Transportation Research Board, Washington, DC, Jan. 1995, 32 pp.

36. Burns, N. H., Gross, S. P. and Byle, K. A., "Instrumentation and Measurements - Behavior of Long-Span Prestressed High Performance Concrete Bridges," *Symposium Proceedings*, PCI/FHWA International Symposium of High Performance Concrete, New Orleans, LA. October 20-22, 1997, Precast/Prestressed Concrete Institute, Chicago, IL, 1997, pp. 566-577.
37. Gross, S. P., Byle, K. A. and Burns, N. H., "Camber of Long-Span High Performance Concrete Girders," Technical Memorandum No. 1, High Performance Concrete for Bridges, The University of Texas, Austin, October 1996, 34 pp.
38. ADAPT-ABI Version 2.00 Computer Program, Adapt Structural Engineering Consultants,
39. PCI Design Handbook, Fourth Edition, Precast/Prestressed Concrete Institute, Chicago, 1992.
40. Buckner, C. D., "An Analysis of Transfer and Development Lengths for Pretensioned Concrete Structures," Report No. FHWA-RD-94-049, FHWA, U. S. Department of Transportation, Washington, DC, 1994.
41. Mitchell, D., Cook, W. D., Khan, A. A. and Tham, T., "Influence of High Strength Concrete on Transfer and Development Length of Pretensioning Strand," *PCI Journal,* Vol. 38, No. 3, May/June, 1993, pp. 52-66.
42. Castrodale, R. W., Kreger, M. E. and Burns, N. E., "A Study of Pretensioned High-Strength Concrete Girders in Composite Highway Bridges - Design Considerations," Report 381-4, University of Texas for Transportation Research, Austin, TX, 1988.
43. Zia, P. and Caner, P., "Cracking in Large-Sized Long-Span Prestressed Concrete AASHTO Girders," Final Report - Center for Transportation Engineering Studies, North Carolina State University, Report No. FHWA/NC/94-003, Raleigh, NC, 1993, 87 pp.
44. Ozyildirim, C. and Gomez, J. P., "Design of High-Performance Concrete Mixtures and Test Beams for a Bridge in Virginia," Virginia Transportation Research Council, Report No. FHWA/VA-96R27, Charlottesville, VA, 1996, 38 pp.
45. *Special Provisions for Low Permeability Concretes,* Virginia Department of Transportation, August 29, 1994, Revised May 21, 1996, Richmond, VA.
46. Goodspeed, C. H., Vanikar, S. and Cook, R., "High Performance Concrete Defined for Highway Structures," *Concrete International,* Vol. 18, No. 2, February 1996, pp. 62-67.

47. *Special Provisions for the North to East Loop Overpass, Amarillo, Texas,* Texas Department of Transportation, Austin, TX, 1995.

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