

High-Performance Concrete



SP-189



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PREFACE

The theme of the American Concrete Institute Spring Convention held in Chicago, Ill., March 14-19, 1999, was High-Performance Concrete (HPC). Thirty-nine technical sessions were held at the convention. Twenty-one sessions were identified as theme sessions. The organizers of each theme session were invited to have their speakers contribute papers to an ACI Special Publication on High-Performance Concrete. Selection, review, and approval of each paper was handled by the organizer of the corresponding technical session. Coordination of the special publication was provided by the TAC subcommittee on High-Performance Concrete.

A total of 25 papers are included in this publication. The general topics include HPC bridges, HPC structural lightweight concrete, material science of HPC, and structural safety of HPC. The papers on each topic are grouped according to the organizing committee.

Thanks are due to the authors of the papers, the organizers for each committee—H. M. Abdou, D. A. Lange, H. H. Nassif, M. L. Ralls, and J. P. Ries—and to the many reviewers of the papers.

H. G. Russell
Chairman
TAC Subcommittee on High-Performance Concrete

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Chapter 1

Design and Construction Issues of High-Performance Concrete Bridges

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High-Performance Concrete Lead State Team Activities: Overview

by M. W. Beacham

Synopsis: The AASHTO “Task Force on Strategic Highway Research Program (SHRP) Implementation” developed and instituted the Lead State Program in 1996. The mission of the Task Force was to optimize ways in which SHRP technologies could be implemented at the state level. The Task Force understood the benefits of mutual cooperation in sharing resources, working as teams, and collectively implementing the technologies. In order to achieve their mission the Task Force developed the concept of “Lead States Teams”. A “Lead States Team” is a group of states that are willing to take the lead and assist in the implementation of specific, targeted SHRP technologies in which they have interest and have gained some practical experience.

Keywords: activities; high-strength concrete; technologies

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BACKGROUND

The AASHTO “*Task Force on Strategic Highway Research Program (SHRP) Implementation*” developed and instituted the Lead State Program in 1996. The mission of the Task Force was to optimize ways in which SHRP technologies could be implemented at the state level. The seven high pay off SHRP technologies are: Superpave; High Performance Concrete; Anti-Icing / Roadway Weather Information System; Innovative Pavement Maintenance Materials; Preventive Maintenance; Assessment, Protection & Rehabilitation of Reinforced Concrete Structures, and Alkali Silica Reactivity. Early on, they discovered some technologies were being utilized by states with little or no discussion with other states, and a process for learning from each other’s experiences and information transfer was lacking. The Task Force understood the benefits of mutual cooperation in sharing resources, working as teams, and collectively implementing the technologies.

GENERAL OBSERVATION

Whenever new technologies or procedures are introduced, there are always several state transportation agencies that seem to be ahead of the pack in adopting these innovations. Those leaders may have moved ahead because the technology addresses a problem they find particularly significant or that these agencies and their contractors are particularly comfortable with the change. The agency may even have played a major role in developing the technology. Frequently, a handful of agencies may use a new technology for many years before it is even tried in most states. If this proficiency gained from experience could be readily transferred to other states, the learning period for new users could be greatly shortened. This may be particularly true for contractors and materials suppliers because the expertise gained by contractors in one state is often not readily transferred to counterparts in other states. Mutual exchange of lead state experience among agencies will eliminate unnecessary and costly duplication of effort. In concept, an AASHTO-designated “Lead State” will be expected to share its proficiency and knowledge of a technology with others in order to advise new users of potential benefits and shorten their learning period for the technology.

TASK FORCE

These observations influenced the Task Force to develop the concept of “Lead States Teams”. A “Lead States Team” is a group of states that are willing to take the lead and assist in the implementation of specific, targeted SHRP technologies in which they have interest and have gained some practical experience. The Task Force prepared selection criteria and guidelines for selecting the appropriate states for each technology. The Task Force identified 21 states that met the criteria. Each state was asked to select a “Champion” to represent their state on one of the technology teams. The “Champions” were given the implementation guide-

lines and invited to participate in a workshop jointly sponsored by AASHTO, Federal Highway Administration (FHWA) and the Transportation Research Board (TRB). The "Champions" were asked to invite local industry representatives to the workshop.

The first "Lead States Workshop" was held in St. Louis, Missouri, September 18-19, 1996. The purpose of the Workshop was to bring the lead states, local industry and FHWA together and develop an action plan for the implementation of SHRP technologies. At the end of the Workshop each team accomplished the following:

1. Nominated a coordinator or contact person;
2. Developed a mission statement;
3. Set up goals, strategies and action plans to accomplish the mission;
4. Identified where necessary, the needed resources and the person in charge of accomplishing the goals and corresponding time frames.

The HPC Lead State Team included representatives from the following agencies:

Manager Transportation Research Center, Arizona DOT
 PCC Engineer, Iowa DOT
 Director/ Research, Missouri DOT
 Division Engineer, Research & Development, Missouri DOT
 Bridge Research & Development Engineer, Nebraska DOT
 Assistant Material & Test Engineer, Nebraska DOT
 Administrator, Bridge Design, New Hampshire DOT
 Bridge Construction & Maintenance Engineer, Texas DOT
 President, Bayshore Concrete Products Corporation, Virginia
 Principal Research Scientist, Virginia DOT
 Bridge & Structure Engineer, Washington DOT
 Engineering Manager, Central Pre-Mix, Washington
 Assistant Secretary, Washington DOT
 Pavement Design Engineer, Washington DOT
 Bridge Engineer, FHWA Washington Division
 Office of Technology Application, FHWA, D.C.
 Bridge Engineer, FHWA, Nebraska Division
 National Concrete Bridge Council
 Portland Cement Association
 American Concrete Pavement Association

At the first "Lead State" meeting in 1996 the Team defined it's mission to be: Promote implementation of High-Performance Concrete (HPC) Technology for use in pavements and bridges and share knowledge, benefits and challenges with states and their customers.

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GOALS/STRATEGIES/ACTION PLANS

The focus of the first meeting held in St. Louis (1996) was to establish the goals, strategies, and action plans (GSA) in order to implement HPC. The results of that meeting yielded four goals with a "Point of Contact" (POC) person for each goal. Each POC is responsible for facilitating the action plans under each strategy and reporting to the Team Leader. Each action plan had a team member(s) or the entire team assigned to complete the plan by an established date.

One year later the Team reconvened in St. Louis for a reality check. The Team adjusted it's GSAs based on our collective experience during 1996. At the third meeting in late September 1998 the Team again revised the GSA's based on the collective experience during 1997. Pavement experts were integrated into the HPC Lead State Team at the September 1998 meeting. Following are the GSA's that resulted from that meeting.

GOAL A

Develop, deploy, and advance improved performance in concrete pavements and bridges, while maintaining constructibility.

Strategy 1

Provide guidance to the states on research and implementation of HPC.

- a) Complete the HPC deck article and submit to ACI *Concrete International*. **(complete)**
- b) Work with FHWA and industry to develop a summary report of HPC pavement and bridge needs and send it to state contacts. **(in progress)**
- c) Assist with a pooled-fund study to compile and evaluate results from HPC bridge projects to date. **(as requested)**
- d) Work with appropriate state personnel to increase the number of HPC pavement and bridge projects under construction in each Lead State, and include durability specifications as part of the project acceptance. **(ongoing)**

Strategy 2

Provide guidance on the HPC definition.

- a) Provide introduction to HPC definition on website. **(in progress)**

Strategy 3

Provide guidance and assistance to AASHTO on specification changes needed to implement HPC in bridges.

- a) Incorporate evaluation and proposed revisions of specifications as part of the HPC bridge research projects in each Lead State. **(ongoing)**
- b) Assist the AASHTO task force on specification changes through a HPC pooled fund study. **(ongoing)**
- c) With assistance from SHRP Implementation Task Force, develop liaisons with the AASHTO Subcommittees on Bridges and Materials, and Construction to provide for continued implementation past Sunset 2000. **(ongoing)**

High-Performance Concrete 5**GOAL B**

Collect and disseminate information on HPC pavements and bridges, to show the benefits of HPC.

Strategy 1

Establish an information database from the states on existing conventional concrete performance characteristics, HPC performance characteristics and field constructibility evaluations, and HPC design criteria.

- a) Develop and distribute pavement information form to all states and collect completed forms. **(bridge complete) (pavement in progress)**
- b) Compile summary of completed pavement information and disseminate to all states. **(bridge complete) (pavement in progress)**
- c) Put summary of pavement forms on web site **(pavements in progress) (bridge complete)**
- d) List 50 state DOT contacts on the website. **(in progress)**
- e) Collect bridge information surveys from remaining 19 states and put on website. **(in progress)**

Strategy 2

Collect and disseminate the latest information on HPC research, design, and construction.

- a) Send semi-annual mailouts to contacts. Include latest FHWA calendar updates and other current HPC information. **(ongoing)**
- b) Continue giving presentations and coordinating HPC sessions at local, state and national meetings and conferences. **(ongoing)**
- c) Maintain web site. **(ongoing)**
- d) Partner with FHWA and industry to develop HPC bridge implementation workshop outline and checklist for specific needs of each state. **(ongoing)**
- e) Contact personnel involved in the Long Term Pavement Performance (LTTP) program and establish liaisons. **(in progress)**
- f) Summarize HPC pavement projects and send to states. **(in progress)**

GOAL C

Create and maintain focus teams composed of public/industry/academic representatives in the lead states for HPC research, development, and implementation to serve as a model for other states.

Strategy 1

Members of each Lead State create and maintain focus teams in their States and coordinate focus team activities.

- a) Members of Lead State team to contact industry, universities, appropriate state & FHWA representatives to form focus teams in each state and meet by December 1997. **(complete)**

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- b) Focus Teams meet as needed to ensure HPC projects are initiated in their states and that the number of projects increased. **(ongoing)**
- c) Focus Teams contact all related industries in their states to explore innovations and possible funding sources and methods to promote HPC. **(ongoing)**
- d) Focus Teams contact neighboring non-Lead States by mail and personal contact to encourage formation of focus teams. **(ongoing)**
- e) Provide guidance to focus teams on updates to website (including new initiatives, condition, cost data, etc.), such that at Sunset 2000, the focus team contacts will be able to provide HPC information to the gatekeeper for input on the website. **(ongoing)**
- f) List focus team contacts on the website. **(in progress)**

GOAL D

Provide assistance to states in life-cycle cost analysis.

Strategy 1

Track performance of HPC components (accomplished with Goal C, Strategy 1, Action Plan E).

Strategy 2

Develop guidelines on condition, repair, and maintenance data for use in determining bridge life-cycle costs (LCC) and distribute to all states.

- a) Lead States act as beta test sites for National Institute of Standards and Technology (NIST) Bridge LOC program. **(ongoing)**
- b) Perform life-cycle cost analyses on data from condition forms using NCHRP Report 12-43 or available method. **(in progress)**
- c) Develop guidelines based on Lead State life-cycle costs analyses to serve as examples and distribute to all states. **(in progress)**
- d) Convince at least 8 (minimum 2 per AASHTO Region) AASHTO members to implement guidelines. **(in progress)**

Strategy 3

Encourage States to use life-cycle cost analysis to recognize benefits of NPC pavements.

- a) Identify several existing life-cycle cost analysis examples and disseminate to states. **(in progress)**

ACCOMPLISHMENTS

As you can see from the status of the action plans above, the Team has accomplished a great deal and is continuing its work. The following is a list of the Team's accomplishments.

- The number of states with HPC bridge projects has increased from the 6 Lead states to 16 states.

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- The number of HPC bridge projects in the Lead States has increased.
- Identification of a point-of-contact in each State DOT has been made.
- Co-sponsor and sponsor of HPC Bridge Showcases.
- Conducting presentations and coordinating HPC sessions at numerous local, state, national, and international meetings and conferences.
- Development and distribution of concrete bridge information survey.
- Establishment of focus teams in the Lead States.
- Establishment of the HPC website. Its address is <http://hpc.fhwa.dot.gov>
- Publication of technical articles.
- Provide multiple beta test sites for bridge life-cycle cost program by NIST.
- Initiated pooled-fund study to evaluate needed specification changes.
- Pavements group added to the Team at September 1998 meeting.

LESSONS LEARNED

Naturally there are many lessons learned from the Lead State process. The Team feels the Lead State process works for the implementation of SHRP products. It does recognize that adequate funding for research, technology transfer, and construction implementation (including seed money for demonstration projects) is critical to the successful implementation HPC, and help from the SHRP Implementation Task Force is needed to obtain this funding. The SHRP Implementation Task Force is also needed to lobby top management of State Highway Agencies for critical "buy-in" to HPC and the Lead State process. HPC technology transfer is dependent on the active participation of State Highway Agencies. Partnerships with industry and academia are also critical to the success. "Champions" in each State DOT are needed to coordinate activities in the their state. The "Champions" should have an interest in this activity and have the time and energy to accomplish the tasks. "Champions" should not be arbitrarily appointed.

PHASE OUT

The AASHTO/SHRP Lead State Teams activities will be phased out in the year 2000. In order to continue encouraging states to be proactive in HPC, Team activities will be accomplished through existing State Focus Teams in cooperation with assistance from the SHRP Implementation Task Force. Assistance will also be needed from FHWA and AASHTO Subcommittees on Bridges, Materials, and Construction.

Recalling the general observations of the Task Force, they said the mutual exchange of experience among state agencies would eliminate unnecessary, and costly duplication of effort. They also said the learning period for new HPC users could be greatly shortened if the experience gained from one state could be readily transferred to another state. The Lead State Team agrees wholeheartedly with these observations. Therefore, the Lead State Team is encouraging states to

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use the website <http://hpc.fhwa.dot.gov> as a technology transfer tool for current and future exchange of experiences.

Although the website is in its infancy it contains important information. It has a section on *What is High-Performance Concrete?*, and the *Grades of HPC*. It has past and present information on the HPC showcases, including slides and commentary in some cases.

The site has information on *Instrumentation Guidelines*, *Team member information*, and *Other Related Links*. One very informative area on the site is the *Lead States Surveys*.

Each participating State has completed an HPC Survey form. This information has been used to establish a database of specifications and details for each State's concrete bridges. The first section allows you to view (but not modify) the data from any one of the participating States. The second section links you to a spreadsheet containing all of the results of the surveys. The third section allows State representatives to modify the survey results for his or her own State only.

The HPC Lead State Team is confident that the State Focus Teams in cooperation with assistance from the SHRP Implementation Task Force will provide a smooth transition into the new millennium.

REFERENCES

- 1) "Lead States Take the Lead in SHRP Technology Implementation", AASHTO Lead States Workshop Proceedings, Sept. 18-19, 1996, St. Louis, Missouri
- 2) "ASSHTO Lead States Leading The Technology Into The 21St Century" Proceedings-Second Lead States Conference

SP 189-2

Structural Design of High-Performance Concrete Bridges

by M. K. Tadros, X. Huo, Z. (John) Ma, and M. Baishya

Synopsis: Based on four strength parameters testing of three high-performance concrete (HPC) design mixes and parametric studies, the following conclusions have been made. Creep and shrinkage strains of HPC are lower than those in conventional concrete. Amount and type of coarse aggregates affect the value of modulus of elasticity. The modulus of elasticity of HPC should be determined through experiments with local materials. Beam sections that have large bottom flange are efficient for HPC application. The most significant property of HPC prestressed beam is compressive strength at release. Allowable compression at release has the most impact on span capacity, while allowable tension at service has minor impact. Prestress loss can be reasonably predicted by either the proposed method or AASHTO LRFD Lump Sum method. PCI deflection multipliers at final time are not accurate. The proposed multipliers which are the functions of creep coefficient can be used for conventional and HPC members.

Keywords: bridges; compressive strength; creep; deflection; high-performance concrete; modulus of elasticity; structural design

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Xiaoming Huo, Ph. D., P. E., is the Assistant Professor of Civil Engineering at Tennessee Technological University. Her doctoral research was focused on time-dependent analysis and application of high-performance concrete bridges.

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Mautu Baishya, Ph. D., P. E., is the Design Engineer at Tadros Associates, LLC. He has sixteen years of combined experience in teaching, research and professional work experience in bridge engineering and other engineering systems.

INTRODUCTION

The definition of HPC given by the Federal Highway Administration specifies four grades of HPC according to four strength parameters and four durability parameters (1). The four strength parameters of HPC are compressive strength, modulus of elasticity, creep and shrinkage. The four durability parameters are freeze/thaw durability, scaling resistance, chloride permeability and abrasion resistance. Of all these parameters, the strength parameters directly affect the structural design of HPC bridges.

Use of high-strength concrete for pretensioned concrete bridge girders has become accepted practice by many state highway agencies because of its technical and economic benefits. High strength concrete gives designers the flexibility to design bridges with longer spans and/or wider girder spacing. High modulus of elasticity decreases the camber, deflection, and elastic shortening losses in bridge girders when subjected to prestress and service loads. Lower creep and shrinkage decreases the prestress losses and affects the time dependent performance of HPC girders. HPC bridges with lighter superstructures and fewer girders than the ones with conventional concrete will result in initial economic savings, especially after

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more experience is gained in HPC construction. The increased service life and reduced maintenance of HPC bridges would also bring the long term economic savings. The objective of this paper is to focus on the four strength parameters of HPC and their impact on structural design.

FOUR STRENGTH PARAMETERS OF HPC

As indicated earlier, four strength parameters of HPC are compressive strength, modulus of elasticity, creep and shrinkage. Numerous recent studies (2,3,4,5) have been conducted on creep and shrinkage properties and modulus of elasticity of HPC, especially as a result of the FHWA HPC Bridge Showcase program. Nebraska was one of the sites of the Showcase program. Given below is a summary of the results of the Nebraska program.

In order to determine creep, shrinkage and modulus of elasticity three concrete mixes were used in the experiments. The materials for all mixes are local Nebraska materials. The mixes were designated as 12SF, 12FA and 8FA. The 12SF mix contained silica fume and fly ash and was designed to have a compressive strength of 12,000 psi at 28 days, 12FA mix contained fly ash only and was designed to have a compressive strength of 12,000 psi at 28 days and 8FA mix contained fly ash only and was designed to have a compressive strength of 8,000 psi in 28 days. Tables 1 to 3 provides the design mixes.

Compressive Strength

The compressive strengths of concrete for the three mixes at different age intervals are listed in Table 4.

Shrinkage

A total of 22 shrinkage specimens in six groups were tested. The shrinkage specimens were the same size as the creep specimens. As the temperature and relative humidity were not controlled, the so-called shrinkage specimens here are actually unloaded specimens that are subjected to lab temperature and relative humidity. The shrinkage specimens were cast at the same time and cured under the same conditions as creep specimens. These specimens had demac points affixed in the same manner as the creep specimens.

The ACI Committee 209 equation (6) for shrinkage coefficient is

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$$\epsilon_{sh} = (\epsilon_{sh})_u \frac{t}{K_s + t} \quad (\text{Equation 1})$$

where, $K_s = 35$. The Nebraska experiments confirmed observations by others (4, 5) that accelerated shrinkage strains occur at an early age in high performance concrete. In order to reflect this trend, K_s in Eq. 1 is suggested as a variable value and is expressed as a function of strength instead of a constant. According to test results, it would be reasonable to keep $K_s = 35$ for normal strength concrete $f'_c = 4,000$ psi (28 MPa) and apply $K_s = 15$ for $f'_c = 12,000$ psi (82.7 MPa) to reflect rapid development of shrinkage at an early age. The recommended K_s can be expressed as

$$K_s = 45 - 2.5 f'_c \quad (\text{Equation 2})$$

where f'_c is the compressive strength of concrete at 28 days in ksi.

The other value that dramatically changes compared to ACI 209 is $(\epsilon_{sh})_u$ as shown in Table 5. The values shown in the table are based on the assumption that all other adjustment factors for relative humidity, curing method, etc., in the ACI 209 method are considered valid. In order to reflect the effects of concrete strength on shrinkage strains, a formula of strength adjustment factor $\gamma_{st,s}$ was developed. The $\gamma_{st,s}$ can be expressed as:

$$\gamma_{st,s} = 1.20 - 0.05 f'_c \leq 1 \quad (\text{Equation 3})$$

The proposed modified equation for concrete strengths in the 4000 to 12000 psi range is thus:

$$\epsilon_{sh} = 780 \times 10^{-6} \gamma_{sh} \frac{t}{K_s + t} \quad (\text{Equation 4})$$

where γ_{sh} is the product of adjustment factors specified in ACI 209 and the proposed ultimate shrinkage reduction factor $\gamma_{st,s}$, obtained from Eq. 3, and K_s is the early age accelerated shrinkage factor (Eq. 2).

Creep

Testing was performed in accordance with ASTM Standard C512-87 (7). The only modification to the specifications was that one 4 in. x 4 in. x 24 in. (100 mm x 100 mm x 610 mm) prism, was used instead of three stacked 4 in. x 8 in. (100 mm x 200 mm) cylinders. This was consistent with recent testing at the University of Texas (4) where a single 4 in. diameter by 24 in. long cylinder is used in each test frame. One-piece specimen in each test frame provides better accuracy and reduces the possibility of loading eccentricity and boundary

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discontinuity. Table 6 shows the number of creep specimens used in this program.

The measured strains in the loaded creep specimens are the sum of shrinkage and creep strains. The creep strain can be determined by subtracting the shrinkage strains, determined as discussed earlier, from the total strains.

The ACI 209 prediction equation for creep coefficient in conventional concrete:

$$C_t = (C_i)_u \frac{t^{0.6}}{K_c + t^{0.6}} \quad (\text{Equation 5})$$

The value of K_c in Eq. (5) is a constant = 10. Similar to the shrinkage studies, the Nebraska testing confirmed earlier observations that the ultimate creep coefficient is lower in higher strength concretes, while growth of that coefficient accelerates at an early age more rapidly than in conventional concrete. Table 7 shows that the ultimate measured creep can be as much as 41% lower than ACI 209 prediction. To reflect these observation, two coefficients were proposed to be introduced to Eq.5:

$$K_c = 12 - 0.50 f'_c \quad (\text{Equation 6})$$

$$\gamma_{st,c} = 1.18 - 0.045 f'_c \quad (\text{Equation 7})$$

The values of K_c and $\gamma_{st,c}$ reduce to 10 and 1.0 for 4000 psi concrete, and to 6 and 0.64 for 12000 psi concrete. The proposed modified Eq. 5 thus becomes:

$$C_t = 2.35 \gamma_c \frac{t^{0.6}}{K_c + t^{0.6}} \quad (\text{Equation 8})$$

where γ_c is a product of the adjustment factors in ACI 209 for conventional concrete and $\gamma_{st,c}$ from Eq. 7 above, and K_c is the coefficient in Eq. 6.

The result of this modification is a creep coefficient that has the same order of magnitude as that for conventional concrete in the early stages and is considerably reduced at the ultimate condition. Thus prestress loss and camber, which are heavily influenced by the creep coefficient, are not affected significantly at the time of erection of a precast concrete girder, about 7 to 60 days after casting. However, the lower ultimate creep of higher strength concrete can considerably impact the ultimate time-dependent deflections and prestress losses.

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Modulus of Elasticity

The modulus of elasticity can be measured according to ASTM Standard C469 (8). Testing at the University of Nebraska, using the previously discussed HPC mix designs, and several hundred 4 in. × 8 in. (100 mm x 200 mm) cylinders, confirmed earlier findings by others (2,5). No single prediction formula can adequately represent the wide variations from one concrete mix to another and from one location to another. It has been widely recognized that the modulus of elasticity of concrete is not just a function of compressive strength of concrete and its unit weight, but it also significantly depends on the type of aggregates and the ratio of coarse to fine aggregates used in the mix.

An attempt in the early 1980s at Cornell University (9) to modify E_c formula to account for high strength was adopted by ACI Committee 363 on High Strength Concrete (10). However, research has found it to significantly underestimate the value of E_c for HPC and the current formula for conventional concrete to give closer predictions to test results.

Based on testing results of the three mixes developed in Nebraska, the following predicting equation is recommended:

$$E_c = a (w)^{1.5} \sqrt{f'_c} + b \text{ (psi)} \quad \text{(Equation 9)}$$

where w is the unit weight of the concrete in pcf. Values of a and b are 33.4 and 462,600 for mix 12SF, 34.1 and 0 for mix 12FA, and 24.7 and 1,789,000 for mix 8FA respectively.

It is observed that the modulus of elasticity of concrete is not a function of compressive strength only. Other effects on the modulus of elasticity included the type of aggregates and content percentage of coarse aggregates. When the content percentage of coarse aggregates is increased the modulus of elasticity of HPC is increased also. Since it would be very hard to give a unified equation of the modulus of elasticity for different HPC mixes, it would be better to consider each mix separately. Test results of modulus of elasticity from each mix provide reliable information on the development of modulus of elasticity. For this reason, the Nebraska Department of Roads has in the past year implemented a new policy, recommended by the University of Nebraska, to require the precast concrete suppliers to supply results of modulus of elasticity measurements of the mixes used in production of precast prestressed concrete bridge girders. However, this approach is a long-term solution aimed at assembling a reliable database. Most designers will continue for the foreseeable future to need reasonably accurate estimates of the modulus of elasticity ahead of decisions on mix design and girder production.

IMPACT OF SECTION PROPERTIES

In order to utilize HPC efficiently in structural design, the section properties of girders and the type of prestressing strands used should be considered first. A detailed investigation was conducted at the University of Nebraska for NU girders (11). A simply supported bridge with NU2000 girders is studied. Fig. 1 shows the maximum span capacity of NU2000 with the changes in girder spacing and number of strands. The strength of concrete is $f'_c = 6,000$ psi with 0.5 in. strands. It can be seen from Fig. 1 that, with a decrease of girder spacing, the number of strands has to be increased to accommodate the increase of the maximum span length. When girder spacing equals 6 ft, the required number of prestressing strands is 58 which is the maximum number of strands that the bottom flange of NU girder can hold.

Based on the research results above, the following comment can be made on effectively utilizing the section properties and HPC together. Designers usually do not have a lot of choice on available sections of girders. However it should be realized by designers that whenever using HPC a girder with a bulky bottom flange would be preferable. The effectiveness of using HPC is largely dependent on the number of strands that a bottom flange can hold. The more strands a bottom flange can hold, the longer span capacity the girder can achieve.

IMPACT OF COMPRESSIVE STRENGTH

Among all property parameters of HPC, compressive strength has most impact in the design of HPC bridge girders. To show the impact of compressive strength the comparison of a bridge with two types of concrete was conducted. The analyzed bridge is a simply supported NU girder bridge. The sections studied in the bridge were NU1100 and NU2000 girders respectively. The girder spacing was changed from 6 to 14 ft. The bridge design program CONSPAN (12) was used in the analysis. Figs 2 and 3 show the analysis results. Compressive strength of concrete was 6,000 psi for conventional concrete and 12,000 psi for HPC. Two types of low relaxation strands, 0.5 in. or 0.6 in. in diameter, were used in the analysis.

Using HPC, especially with 0.6 in. strands, shows great improvement in the girder span capacity. Three curves for NU1100 are shown in the Fig. 2 for conventional concrete and 0.5 in. diameter strands, HPC with 0.5 in. diameter strands and HPC with 0.6 in. diameter strands. By using HPC with 0.6 in. strands, the span capacity can be increased 7%-10% compared with HPC with 0.5 in. diameter strands and 34%-50% compared with conventional concrete with 0.5 in. strands.

Another three curves of the NU2000 girders are shown in Fig. 3. Using HPC with 0.6 in. diameter strands, the span capacity of the girders will be increased by 20%-32% comparing with using conventional concrete with 0.5 in. diameter strands. The higher compressive strength, and the corresponding ability of a member to accept more prestressing that has the most impact on improving span capacities.

IMPACT OF ALLOWABLE TENSION

There has been discussion about increasing concrete modulus of rupture from $7.5\sqrt{f'_c}$ for conventional concrete to $12\sqrt{f'_c}$ for HPC. Since the allowable tension stress is directly proportional to the modulus of rupture, the increased allowable tension has been suggested by some designers and researchers. In order to see the effect of increasing allowable tension stresses on structural design, a study was conducted for NU1100 girders. NU1100 girders were used in a HPC bridge at 120th and Giles Road, Sarpy County, Nebraska. The compressive strength of HPC are 5,500 psi at release and 12,000 psi at service. In this study simply supported girders were considered. Fig. 4 shows two curves developed with different allowable tensile limits.

The first curve represents the design with allowable stresses specified in AASHTO Standard Specifications while the second curve represents the new allowable tensile limits, which are $10\sqrt{f'_c}$ at release and $8\sqrt{f'_c}$ at service. However, as Fig. 4 shows, this adjustment on allowable tensile limits does not result in significant improvement in span capacity. Since the effect of using higher allowable tensile stresses for HPC bridge design is minor, it is reasonable to use the AASHTO Standard design criteria (13) in the design of HPC bridge girders.

PRESTRESS LOSSES

In order to harvest the full benefits of HPC it is necessary to estimate accurately the prestress losses of pretensioned concrete girders. Recent research has indicated that, in many cases, bridge designers use the same prestress loss equations developed for calculating prestress losses in normal-strength concrete for high-strength concrete bridge girders. These calculation methods do not appear to give reasonable estimates of long term prestress losses. This is documented in recent research performed by Seguirant (14) in developing deep

precast, prestressed concrete girder sections that have been adopted as Washington State Department of Transportation (WSDOT) Standards. The four methods used to calculate prestress loss are the AASHTO Refined Method (15), AASHTO Approximate Lump Sum Method (15), ACI-ASCE Committee 423 Method and PCI General Method (16). All these methods provided different amounts of prestress loss. For the entire range, the loss of span capability from one extreme to the other is 5 to 13 percent. At higher levels of pretensioning, the available span can be reduced by more than 18-ft (5.49 m). This will undoubtedly be accentuated with longer-span post-tensioned girders.

Time dependent prestress loss is caused by a combination of the effects of creep and shrinkage of concrete and of stress relaxation of the prestressing strands. There is continuous interaction between these three influencing effects. The interaction of creep, shrinkage and relaxation is partially taken into account in the AASHTO prediction formulas and other approximate procedures. However, full account of these effects cannot be achieved without time-step computer analysis. Perhaps, the treatment of interaction between creep, shrinkage and relaxation in AASHTO is adequate for conventional concrete, with no mild reinforcement. But, it must be applied carefully for HPC with or without mild reinforcement. Luckily, prestress losses have negligible impact on flexural strength. They only affect the serviceability limit state.

Proposed Methods of Prestress Loss Prediction

Research at the University of Nebraska has resulted in a proposed modification of the AASHTO prestress loss formulas to reflect the above-mentioned changes in creep and shrinkage properties of HPC concrete.

Prestress loss due to concrete shrinkage:

$$SH = (17 - 0.15 H) \gamma_{st, s} \text{ (ksi)} \quad \text{(Equation 10)}$$

where $\gamma_{st, s}$ is the strength modification factor for concrete shrinkage given by Equation 3.

Prestress loss due to concrete creep:

$$CR_c = (12 f_{cir} - 7 f_{cds}) \gamma_{st, c} \text{ (ksi)} \quad \text{(Equation 11)}$$

where $\gamma_{st, c}$ is strength modification factor for concrete creep given by Equation 7.

Prestress loss due to strand relaxation:

$$CR_s = 5 - 0.1 ES - 0.05 (SH + CR_c) \text{ (ksi)} \quad \text{(Equation 12)}$$

PCI Bridge Design Manual Procedure

The approach proposed in the PCI Bridge Design Manual (17) was based on earlier studies by Tadros (18). The Tadros procedure is based on the aging coefficient theory of Trost (19) and Bazant (20). Tadros et al initially presented their method in 1975 (21) and in an award winning paper in 1977 (22). The simplified steps published in 1985 produce (18) results that are very close to those from elaborate computer step function procedures. The steps are given below.

Stress loss due to shrinkage:

$$SH = K \epsilon_{sh} E_s \quad (\text{Equation 13})$$

$$\text{where: } K = 1 / \left[1 + (E_s A_{ts} / E_{ci} A_g) \cdot (1 + e_{ts}^2 / r^2) (1 + \chi C_u) \right] \quad (\text{Equation 14})$$

The symbols A_{ts} and e_{ts} are the total steel area and its eccentricity, respectively.

Stress Loss Due to Creep:

$$CR = K \left[n_i C_u f_{cir} + n(1 + C'_u) f_{cds} \right] \quad (\text{Equation 15})$$

where: $n_i = E_s / E_{ci}$, $n = E_s / E_c$,

f_{cir} is concrete stress at centroid of total steel area due to initial prestress and member weight:

$$f_{cir} = (P_{co} / A_g) + (P_{co} e_{ps} e_{ts} / I_g) - (M_d e_{ts} / I_g), \quad (\text{Equation 16})$$

f_{cds} = Stress at centroid of total steel area due to superimposed dead loads not used in computing f_{cir} , C_u = ultimate creep coefficient for concrete at loading equal to time of release of prestressing, C'_u = ultimate creep coefficient for concrete at application of the superimposed loads

Stress Loss due to Relaxation:

$$CR_s = K(1 - 3(SH + CR) / f_{ps0}) L_r \quad (\text{Equation 17})$$

where L_r is the intrinsic (constant length) relaxation loss.

Total Time-Dependent Loss of Compression force in Concrete:

$$\Delta P_c = -A_{ts}(SH + CR) - A_{ps}(CR_s) \quad (\text{Equation 18})$$

Note that the negative sign designates a tensile force increment. For lightweight concrete, appropriate values of the material properties, such as E_{ci} , E_c , ϵ_{sh} , C_u , and C'_u should be used.

For precast conventional concrete members, with the following average conditions, Eqs. 13, 14, 15 and 17 reduce to Eqs. 19-22.

Relative humidity = 70 percent, volume to surface ratio, $V/S = 1.5$, concrete unit weight = 150 pcf, accelerated curing, release strength (at concrete age 1 to 3 days), $f'_{ci} = 3,500$ psi, $E_{ci} = 3600$ ksi, $f'_c = 5000$ psi, $E_c = 4300$ ksi, $C_u = 1.88$, $C'_u = 1.50$, $\epsilon_{sh} = 550 \times 10^{-6}$, $\chi = 0.7$, $f_{ps0} / f_{pu} = 0.7$, $L_r = -4.8$ ksi, $E_s = E_{ps} = 28,000$ ksi, $f_{pu} = 270$ ksi

$$SH = 15.4 K \quad (\text{Equation 19})$$

$$K = 1 / [1 + 18.0(A_{ts} / A_g)(1 + e_{ts}^2 / r^2)] \quad (\text{Equation 20})$$

$$CR_c = K[14.6f_{cr} + 16.3f_{cds}] \quad (\text{Equation 21})$$

$$CR_s = 4.8K - (1.17 + 1.1f_{cr} + 1.2f_{cds})K^2 \quad (\text{Equation 22})$$

It is noted that even though the equations carry similarity to those of AASHTO, the procedure has several advantages: 1. It accounts for presence of non-prestressed steel which can cause significant loss of concrete precompression (22). This advantage is consistent with the AASHTO LRFD philosophy of treating prestressed concrete, partially prestressed concrete, and reinforced concrete as one "structural" concrete material. 2. One can use the same equations for conventional or high performance concrete, dry or humid weather, and high and low prestress levels. 3. Regional variations of material properties can be accommodated or typical properties used. 4. Accurate values of prestress losses at time of erection can be obtained by the same equations with proper adjustment of the relevant material properties.

The PCI Bridge Design Manual Procedure should be considered a possible viable modification of the current AASHTO LRFD procedure.

Comparison of Methods

Several specialized computer programs are available for time-dependent analysis of prestress losses. An example is the computer program CREEP3 developed by Tadros (23). The program allows for consideration of any creep, shrinkage and modulus of elasticity time function as well as any loading stages. It also allows for multi-stage pretensioning/post-tensioning. The program has been used over the past twenty-five years for design of many segmental and spliced post-tensioned bridges. It is an excellent analytical tool for correlation with test results and for determination of the accuracy of various approximate prediction formulas.

This program was used to compare various prestress loss prediction methods. The comparison examples are Design Examples 9.2 and 9.3 of the PCI Bridge Design Manual (18). The examples use a 72-in AASHTO/PCI Bulb Tee whose span is 120 ft and spacing is 9 ft. The result of the comparison is shown in Table 8.

Table 8 indicates that when HPC is used in girders, the method in the AASHTO Standard Specifications and the refined method in the AASHTO LRFD Specifications overestimate the value of time-dependent prestress losses, in comparison with CREEP3 results. Overestimating the prestress losses may not be conservative. In the case of overestimation of prestress loss, the girder is likely to be constructed with a few more strands than necessary. The additional strands will cause more camber and shortening, which leads to larger bearing pad thickness and super-structure depth. Also, excessive camber causes the haunch concrete directly over the girder flange to be thicker at girder ends in order to maintain a smooth roadway profile. This additional concrete weight may be significant in modern girder shapes, such as the NU I-girder, which have wide top flanges. Therefore, it may not be proper to use these two methods in predicting prestress losses. The results from the AASHTO LRFD lump sum method and the proposed method are relatively close to the computer results. With the strength of concrete as variable, these two methods are applicable for HPC and conventional concrete.

According to Huo's study (11) on 80 NU girder bridges of various spans and spacing, the initial prestress losses using conventional concrete properties, ranged from about 8-12 percent and the final prestress losses range from 24-30 percent of initial prestress. In all cases, when HPC properties were used, the final prestress losses dropped by about 40 percent. However, as noted by Seguirant (14), higher strength concretes allow use of more prestressing which in turn increase the elastic and time dependent effects. The two counteracting effects must be fully recognized to obtain reliable loss predictions.

LONG-TERM DEFLECTION

Long-term camber and deflection were conventionally estimated by the PCI Design Handbook suggested multipliers method (16). The use of this method is fairly straightforward. Elastic deflections due to prestressing and self weight, and additional dead loads are first calculated using conventional analysis techniques. These elastic deflections are then modified with the appropriate multipliers. These multipliers reflect the additional deflections that occur due to time-dependent behavior. However, the actual deflection changes could be significantly different from those resulting from the PCI Design Handbook multipliers. This can be illustrated by the following example.

A simple span bridge with Bulb-Tee 72 in. girders was analyzed with consideration of time-dependent effects. The span length was 120 ft. The girder spacing was 12 ft. Forty four 0.5 in. diameter low relaxation strands were put in a girder. The first case studied was a conventional concrete bridge. The concrete

strength of girders at 28 days $f'_{c, \text{girder}} = 7,000$ psi and at release $f'_{ci, \text{girder}} = 5,000$ psi. The concrete strength of cast-in-place deck $f'_{c, \text{deck}} = 4,000$ psi. The construction stages are as follows: (1) releasing prestressing force at 1 day; (2) applying girder self weight at 2 days; (3) pouring cast-in-place deck at 35 days; and (4) applying superimposed dead load at 120 days.

The instantaneous camber and deflection due to prestressing force and dead load of prestressed girder can be accurately determined according to the correct modulus of elasticity of concrete. Long-term deflection of girders, of more concern by designers, is dependent on the time-dependent properties of concrete. The computer program CREEP3 is used to determine the time-dependent stresses and deformation in each stage of construction. Only the deflections at midspan is discussed here.

Table 9 lists the individual immediate deflection which includes the deflection due to prestress release, girder self weight, CIP deck weight, and superimposed dead load. The long-term deflection determined by PCI multipliers and computer analysis is also presented in Table 9.

The comparison results showed that, from the prestress release to the time of placement of CIP deck (35 days), the deflection determined by PCI multiplier and computer analysis are very close. The difference between two methods is 13%. But at the time infinity, there is a large difference between two methods. The method cannot properly account for the significant effects of a large cast-in-place deck. The presence of a deck, once cured, drastically changes the stiffness of a typical bridge member. This has the effect of restraining the girder creep strains resulting from prestressing, member self weight, and the dead load of the deck. The PCI multipliers for long-term deflection suggested by this method, therefore, must be considered very approximate and should not be used for bridge girders with cast-in-place decks.

New formulas for multipliers used in the calculation of long-term deflection of precast, prestressed bridge girders need to be investigated. A parametric analysis was conducted. The analysis covers the sections used in the Nebraska NU series, and AASHTO BT girders, a popular series of sections in many states. The parameters vary and include (1) girder sections from NU1100 to NU2000 and AASHTO BT-54 to AASHTO BT-72; (2) span length; and (3) girder spacing. The specified concrete compressive strengths for the NU girder bridge are $f'_{c, \text{girder}} = 8,000$ psi for girders and $f'_{c, \text{deck}} = 5,000$ psi for deck. The compressive strength of concrete for AASHTO BT beam bridges are $f'_{c, \text{girder}} = 6,000$ psi for girders and $f'_{c, \text{deck}} = 4,000$ psi for deck.

Time-Dependent Deflection at Erection

For each girder section and girder spacing, a bridge design program CONSPAN was used to determine the maximum span, the number of prestress strands required, number of draped strands and centroid of prestress strands at end and midspan sections. CREEP3 program was used to find the time-dependent deflection and the multipliers.

The following construction stages are considered in CREEP3 analysis. 1. The prestress forces are released at 2 days after girder casting; 2. The weight of precast member is applied at same time of prestress release; 3. Cast-in-place concrete deck is cast 50 days after girder ages; and 4. Superimposed dead load is applied on 120 days after girder casting. For the calculation of deflection at erection, only the first three stages are applied. The prestress losses are calculated from the day of prestress release until the time of erection. The deflection is examined during the entire loading process.

A total of 32 cases, 20 cases of NU girders and 12 cases of AASHTO BT beams, are studied. Tables 10 and 11 show the analysis summary for each group of girders. The deflection multipliers due to precast beam weight are 1.930 for NU girder with high strength concrete and 2.015 for Bulb Tee girders with normal strength concrete respectively. It is observed that even though the girder size, girder spacing, and the span of the bridge vary, the deflection multipliers for each girder group with same type of concrete are nearly the same.

If C_a is a creep coefficient for age of loading at release and loading duration up to the time of erection. C_a value for NU girders and AASHTO Bulb-Tee girders can be determined according to the curing condition, environmental condition and strength of concrete. Calculation results (11) showed that $C_a = 0.924$ for NU girders and $C_a = 1.006$ for Bulb-Tee girders. If the C_a values of each group of girders are applied into the formula in Table 12 on deflection multiplier for member weight, the following is obtained:

NU girder with high strength concrete: multiplier = $1 + C_a = 1.924$

BT girder with normal strength concrete: multiplier = $1 + C_a = 2.006$.

Both multipliers 1.924 for NU girders and 2.006 for BT beams are very close to the value obtained from analysis 1.930 for NU girders and 2.015 for BT beams. It is reasonable to use the formula in Table 12 to determine the deflection multiplier for member weight. The advantage of using formula instead of a constant is that with the variable C_a (18), the multiplier can be adjusted according to the creep coefficient of concrete. This feature would be very helpful when HPC, with lower creep and shrinkage strains, is used.

The upward deflection is affected by the initial prestress force and prestress losses. In Table 12, it is considered to be more convenient if one multiplier could be used to determine the deflection due to prestress force and losses together. The calculated multipliers of camber for NU girder group ranged from 1.783 to 1.851, which is about $0.92 (1 + C_a)$ to $0.96 (1 + C_a)$. For Bulb-Tee beams, the calculated multipliers of camber ranged from 1.860 to 1.902, which is about $0.92 (1 + C_a)$ to $0.95 (1 + C_a)$. The coefficient of $(1 + C_a)$ are slightly changed according to the size of girder, girder spacing and span length. To simplify the calculation of multiplier, it is recommended that the multiplier due to camber be determined as $0.94 (1 + C_a)$.

Time-dependent Deflection at Final Time

The analysis for long-term deflection uses the same program CREEP3 as in the previous analysis. In the analysis for final time, the loads on the girders discussed in previous sections such as prestress force, girder self-weight, and cast-in-place deck weight are applied with the superimposed dead load (45 lb/ft^2) at 120 days after girder casting. The deflection is calculated through 10,000 days.

The final deflection for the NU1100 girders and BT-72 girder bridges for previous analysis was investigated. For the same girder size, multipliers vary with variations in girder spacing and span. The deflection multipliers for the AASHTO BT beams are higher than those of NU girders because high-strength concrete is used in NU girders. Tables 13 and 14 list the time-dependent deflection multipliers at final time for NU girders and AASHTO BT beams.

Even though some parameters such as the size of girders, girder spacing and span are varied in the analysis, it is interesting to see that the range of multipliers in each loading case for NU girders or AASHTO BT beams is relatively small. For NU girders, the multipliers for camber due to the prestress force range from 2.386 to 2.493 with an average value of 2.47. The multipliers for deflection due to superimposed dead load range from 1.992 to 1.997 with an average value of 1.994. With multipliers for BT section, the same trend is observed but with higher values of multipliers. It is obvious that using a constant value for all multipliers would not represent the actual performance of members especially when HPC is used.

The more rational way to determine deflections is to use formulas for deflection multipliers which could be adjusted according to the properties of concrete. To develop such formulas for long-term deflections of girders with composite sections, a detailed analysis for each loading case should to be conducted. Since creep has the largest effect, the resultant formula should have

the ultimate creep coefficient as a variable which could then be used for both HPC and conventional concrete.

A set of multiplier formulas are derived and presented in Table 15. During the derivation of the formulas of multipliers, the analysis results from normal strength concrete bridges with BT beams were considered as standard values. For AASHTO BT beam bridges, the compressive strength was 6000 psi for beam and 4000 psi for cast-in-place deck. In Table 15, C_a is a creep coefficient for age of loading at release and loading duration up to time of erection, and C_u is an ultimate creep coefficient for concrete at loading equal to time of release of prestressing. An ultimate creep coefficient can be determined according to the correction factors of compressive strength discussed in previous chapter, relative humidity, loading age and others indicated in ACI 209 Report.

The recommended multiplier formulas are checked using the values obtained from the analysis for NU girder group. Table 16 lists the comparison results. The ultimate creep coefficient for the concrete used in the analysis is $C_u = 1.837$. It can be seen from the table that the recommended multipliers fairly well predict the value of multipliers of the NU girder group.

CONCLUSIONS

Based on this study, the following conclusions are made:

1. Creep and shrinkage strains of HPC are lower than those in conventional concrete;
2. Amount and type of coarse aggregates affect value of modulus of elasticity. The modulus of elasticity of HPC should be determined through experiments with local materials;
3. Beam sections that have large bottom flange are efficient for HPC application;
4. The most significant property of HPC prestressed beam is compressive strength at release. Allowable compression at release has the most impact on span capacity, while allowable tension at service has minor impact;
5. Prestress loss can be reasonably predicted by either the proposed method or AASHTO LRFD Lump Sum method; and
6. PCI deflection multipliers at final time are not accurate. The proposed multipliers which are the functions of creep coefficient can be used for conventional and HPC members.

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TABLE 1—CONCRETE MIX NO. 1 BY READY MIXED CONCRETE COMPANY (12SF).

Material	Quantity
Cement Type I	750 pcy
Fly Ash, Class C pcy	200 pcy
Silica Fume	50 pcy
ASTM C33 Sand	990 pcy
1/2" BRS Limestone	1860 pcy
Water	240 pcy
Type-A WR	4 oz/100 lbs
HRWR	30 oz/100 lbs
W/C	0.24

High-Performance Concrete 27**TABLE 2—MIX NO. 2 BY UNIVERSITY OF NEBRASKA (12FA).**

Component	University Mix		Modified University Mix as Delivered by Wilson Concrete Company	
	Description	Amount	Description	Amount
Cement	Type III	680 lbs	Type III	680 lbs
Fly Ash	Class C	320 lbs	Class C	320 lbs
Limestone	1/2" Crushed Washed	1917 lbs	3/8" Crushed Washed	1913 lbs
Sand	ASTM C-33	934 lbs	Nebraska 47B	933 lbs
Water	Cold	250 lbs	Cold	254 lbs
Retarder	Pozzoloth 300 R	3.0 oz/100 lbs	Daratar 17	4.0 oz/100 lbs
Super- plasticizer	Reheobuild 1000	23.0 oz/100 lbs	WRDA 19	34.2 oz/100 lbs

TABLE 3—MIX NO. 3 BY READY MIXED CONCRETE COMPANY (8FA).

Material	Quantity
Cement Type IP	750 pcy
Fly Ash, Class C	75 pcy
47-B Sand Gravel	1400 pcy
1/2" BRS Limestone	1400 pcy
Water	255 pcy
Pozzoloth 322-N	4 oz/100 lbs
Rheobuild	18 oz/100 lbs
W/C	0.30

TABLE 4—COMPRESSIVE STRENGTH OF HPC MIXTURES.

Specimen Age	Mix No. 1 (12SF)	Mix No. 2 (12FA)	Mix No. 3 (8FA)
7 days	10,200	8,730	7,360
14 days	10,950	11,450	8,090
28 days	11,750	12,360	9,030
42 days	13,150	13,660	9,670
56 days	14,350	13,720	9,940

TABLE 5—COMPARISON OF ULTIMATE SHRINKAGE STRAINS BETWEEN TEST RESULTS AND ACI 209 EQUATION.

	12SF		12FA		8FA	
	Moist cured 7 days	Moist cured 28 days	Moist cured 7 days	Moist cured 28 days	Moist cured 7 days	Moist cured 28 days
ACI Coefficient γ_{sh}	0.933	0.803	0.904	0.778	1.089	0.936
ACI $(\epsilon_{sh})_u = \gamma_{sh} * 780$	728	626	705	607	849	730
Test $(\epsilon_{sh})_u$	480	376	422	399	310	427
Percent difference	34.1%	39.9%	40.1%	34.3%	63.5%	41.5%

TABLE 6—NUMBER OF CREEP SPECIMENS.

Loading Age	Mix No. 1 (12SF)	Mix No. 2 (12FA)	Mix No. 3 (8FA)
7 days	8	7	5
28 days	8	7	5

TABLE 7—ULTIMATE CREEP COEFFICIENT OF HPC.

	12SF		12FA		8FA	
	Moist cured 7 days	Moist cured 28 days	Moist cured 7 days	Moist cured 28 days	Moist cured 7 days	Moist cured 28 days
Coefficient γ_c	1.079	0.907	1.078	0.906	1.109	0.932
ACI 209 $C_u = \gamma_c * 2.35$	2.54	2.13	2.53	2.13	2.61	2.19
Experimental C_u	1.50	1.28	1.54	1.50	1.68	1.98
Percent difference	40.9%	39.9%	39.1%	29.6%	35.6%	9.6%

High-Performance Concrete 29**TABLE 8—COMPARISON OF TIME-DEPENDENT PRESTRESS LOSSES OF DIFFERENT METHODS FOR 72 IN. AASHTO/PCI BULB TEE.**

Loss (ksi)	AASHTO Standard Specs	AASHTO LRFD Lump Sum	AASHTO LRFD Refined	Proposed with strength modification	CREEP3 analysis
Shrinkage	6.50	-	6.50	3.575	-
Creep	23.64	-	23.64	14.0	-
Relaxation	1.67	-	2.23 + 2.00	3.179	-
Total	31.81	29.7	34.37	20.75	17.5

TABLE 9—COMPARISON OF DEFLECTION BY PCI MULTIPLIERS AND COMPUTER ANALYSIS.

Cause	Elastic deflection in.	35 days			Final		
		PCI multiplier	PCI Δ in.	Comp. Δ in.	PCI multiplier	PCI Δ in.	Comp. Δ in.
Prestress	4.45 ↑	1.8	8.01 ↑		2.2	9.79 ↑	
Girder Self Weight	1.57 ↓	1.85	2.90 ↓		2.4	3.77 ↓	
Deck Weight	1.88 ↓	1.0	1.88 ↓		2.3	4.32 ↓	
Superimpose d Dead Load	0.30 ↓				3.0	0.9 ↓	
Total			3.23 ↑	2.86 ↑		0.80 ↑	2.26 ↑
Percent Difference			13 %			65 %	

TABLE 10—ANALYSIS RESULTS OF MULTIPLIERS (ERECTION TIME) FOR NU GIRDERS.

	Girder span (ft)	Girder spacing (ft)	Camber	Girder weight
NU1100	109	6	1.818	1.930
NU1100	98	8	1.802	1.930
NU1100	86	10	1.825	1.930
NU1100	77	12	1.783	1.930
NU1350	127	6	1.826	1.930
NU1350	115	8	1.809	1.930
NU1350	103	10	1.794	1.930
NU1350	93	12	1.787	1.930
NU1600	145	6	1.836	1.930
NU1600	130	8	1.816	1.930
NU1600	118	10	1.801	1.930
NU1600	108	12	1.790	1.930
NU1800	155	6	1.844	1.930
NU1800	140	8	1.824	1.930
NU1800	127	10	1.808	1.930
NU1800	117	12	1.797	1.930
NU2000	164	6	1.851	1.930
NU2000	148	8	1.830	1.930
NU2000	136	10	1.816	1.930
NU2000	125	12	1.804	1.930
Statistical analysis	MEAN		1.81	1.93
	STDEV		0.02	0.00

TABLE 11—ANALYSIS RESULTS OF MULTIPLIERS (ERECTION TIME) FOR AASHTO BT GIRDERS.

	Girder span (ft)	Girder spacing (ft)	Camber	Girder weight
BT54	99	6	1.888	2.015
BT54	88	8	1.869	2.015
BT54	76	10	1.860	2.015
BT54	65	12	1.853	2.015
BT63	114	6	1.894	2.015
BT63	99	8	1.876	2.015
BT63	87	10	1.866	2.015
BT63	75	12	1.858	2.015
BT72	128	6	1.902	2.015
BT72	112	8	1.797	2.015
BT72	99	10	1.870	2.015
BT72	88	12	1.862	2.015
Statistical Analysis		MEAN	1.86	2.02
		STDEV	0.02	0.00

TABLE 12—RECOMMENDED TIME-DEPENDENT DEFLECTION MULTIPLIERS.

(time-dependent δ = elastic δ $\times$ multiplier)

Load condition	Erection time
	Formula
Initial prestress	0.94 (1 + C _a)
Member weight	1 + C _a
Superimposed dead load	1.00

TABLE 13—MULTIPLIERS OF TIME-DEPENDENT DEFLECTION AT FINAL TIME FOR NU GIRDER.

	Girder span (ft)	Girder spacing (ft)	Camber	Girder weight	Deck weight	Super-imposed
NU1100	109	6	2.475	2.414	2.123	1.997
NU1100	98	8	2.437	2.370	2.074	1.998
NU1100	86	10	2.415	2.340	2.084	1.995
NU1100	77	12	2.386	2.324	2.106	1.993
NU1350	127	6	2.504	2.444	2.140	1.996
NU1350	115	8	2.466	2.399	2.087	1.996
NU1350	103	10	2.431	2.367	2.076	1.994
NU1350	93	12	2.414	2.351	2.087	1.992
NU1600	145	6	2.463	2.465	2.143	1.992
NU1600	130	8	2.490	2.419	2.101	1.995
NU1600	118	10	2.457	2.386	2.078	1.993
NU1600	108	12	2.433	2.369	2.074	1.992
NU1800	155	6	2.553	2.479	2.166	1.993
NU1800	140	8	2.509	2.433	2.120	1.992
NU1800	127	10	2.474	2.400	2.098	1.993
NU1800	117	12	2.450	2.382	2.089	1.992
NU2000	164	6	2.569	2.493	2.191	1.990
NU2000	148	8	2.522	2.446	2.147	1.994
NU2000	136	10	2.491	2.413	2.115	1.993
NU2000	125	12	2.465	2.395	2.109	1.992
Statistical Analysis	MEAN		2.47	2.40	2.11	1.99
	STDEV		0.05	0.05	0.03	0.00

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TABLE 14—MULTIPLIERS OF TIME-DEPENDENT DEFLECTION AT FINAL TIME FOR AASHTO BT GIRDERS.

	Girder span (ft)	Girder spacing (ft)	Camber	Girder weight	Deck weight	Super-imposed
BT54	99	6	2.618	2.611	2.510	2.080
BT54	88	8	2.573	2.564	2.489	2.079
BT54	76	10	2.544	2.530	2.570	2.076
BT54	65	12	2.526	2.519	2.698	2.076
BT63	114	6	2.641	2.631	2.485	2.079
BT63	99	8	2.592	2.583	2.488	2.074
BT63	87	10	2.564	2.548	2.539	2.077
BT63	75	12	2.544	2.537	2.644	2.074
BT72	128	6	2.663	2.647	2.454	2.078
BT72	112	8	2.613	2.598	2.455	2.079
BT72	99	10	2.582	2.563	2.492	2.076
BT72	88	12	2.560	2.551	2.537	2.076
Statistical Analysis	MEAN		2.59	2.57	2.53	2.08
	STDEV		0.04	0.04	0.07	0.00

TABLE 15—RECOMMENDED MULTIPLIERS FOR LONG-TERM DEFLECTION.

Elastic deflection	Multiplier for long-term deflection at erection	Multipliers for long-term deflection at final with composite topping
Camber due to prestress force at time of release of prestress (upward)	$0.94 (1+C_a)$	$1+0.80 C_u$
Deflection due to the member weight at release of prestress (downward)	$1+C_a$	$1+0.77C_u$
Deflection due to composite topping (downward)	1.00	$1+0.61 C_u$
Deflection due to superimposed dead load (downward)	-	$1+0.54 C_u$

TABLE 16—COMPARISON BETWEEN RESULTS FROM RECOMMENDED MULTIPLIERS AND ANALYSIS.

Load case	NU girders		Bulb-Tee girders	
	Analysis results (average)	Recommended multipliers	Analysis results (average)	Recommended multipliers
Prestress force	2.47	2.46	2.59	2.59
Girder weight	2.41	2.40	2.57	2.57
Deck weight	2.12	2.11	2.53	2.53
Superimposed dead load	1.99	1.99	2.08	2.08

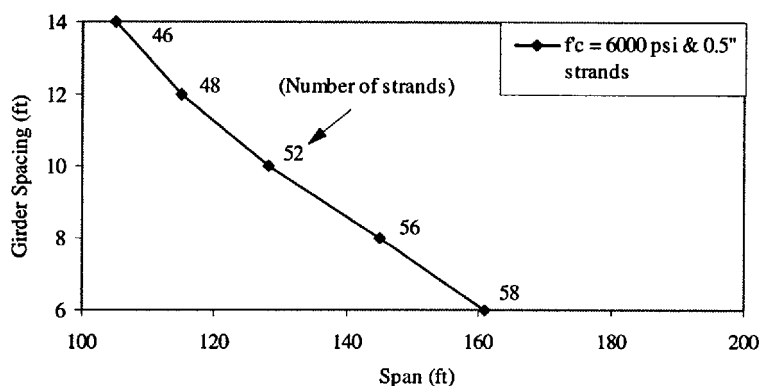


Fig. 1—Maximum span capacity of NU2000 girder.

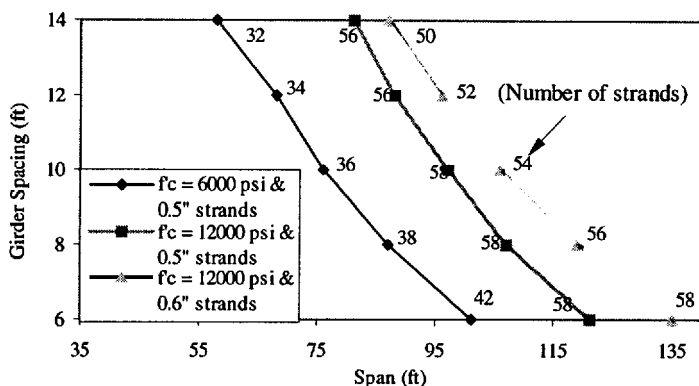


Fig. 2—Maximum span capacity of NU1100 girders.

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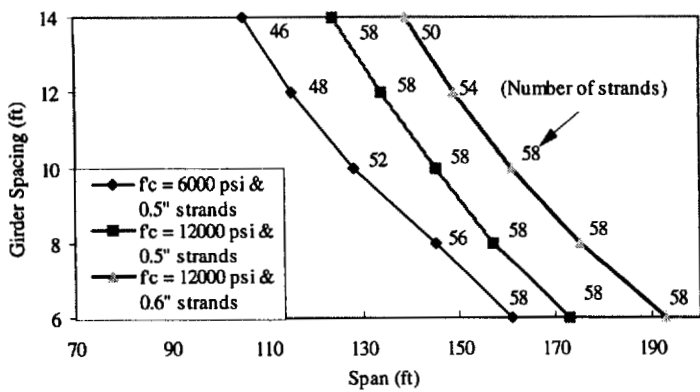


Fig. 3—Maximum span capacity of NU2000 girders.

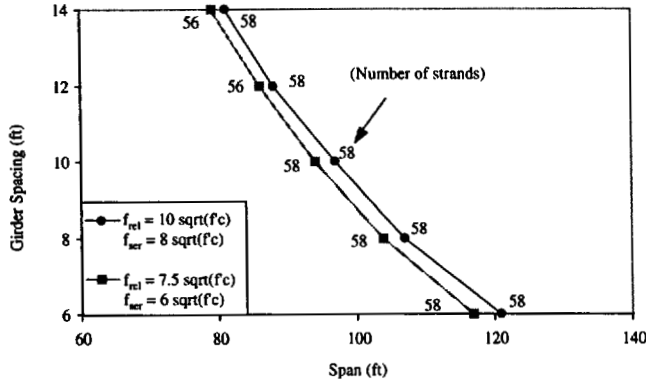


Fig. 4—Span capacity of NU1100 girders with different allowable tension.

SP 189-3

Mixture Proportioning for High-Strength High-Performance Concrete Bridge Beams

by J. J. Myers and R. L. Carrasquillo

Synopsis: High performance concrete (HPC) with its improved service under load and improved resistance to environmental conditions represents a promising material to assist with the rehabilitation of the crumbling infrastructure. Although HPC has found widespread application within the building industry in certain pockets of the country, its incorporation into transportation structures has been very recent. To demonstrate the suitability of HPC for use in highway structures, the Federal Highway Administration (FHWA) initiated a series of projects that included the complete incorporation of HPC from design to long-term monitoring of the bridges in service. The design and construction of Louetta Road Overpass in Houston, Texas and the North Concho River US 87 & S.O. RR Overpass in San Angelo, Texas were conducted as a joint effort by The University of Texas at Austin and the Texas Department of Transportation (TxDOT). The Louetta Road Overpass project incorporated the use of a newly developed pretensioned precast U-Beam. The high initial prestressing forces required high early release strengths of 63.4 MPa (9,200 psi) and design strengths of 91.0 MPa (13,200 psi) at 56 days. The designers (TxDOT) also required a high initial modulus of elasticity of 41.3 kPa (6,000 ksi) at release and long-term to satisfy the serviceability requirements for the beams. The North Concho River US 87 & S.O. RR Overpass project incorporated the use of pretensioned AASHTO Type IV beams. This is the most widely used bridge system in the state of Texas. These members also required high initial release strengths of 74.5 MPa (10,800 psi) and design strengths of 101.4 MPa (14,700 psi) at 56 days. In order to satisfy these design requirements a series of laboratory trial mixes was initiated using locally available materials. The objectives were to develop an optimized mix design using locally available materials that would not only satisfy the project design requirements, but also result in an economical mix design. The following paper discusses the evolution and optimization of the mix design and its subsequent use in the field. In addition, the selection process of the aggregate determined to be most suitable for the production of high performance concrete beams is discussed. A brief description of each project is also presented.

Keywords: aggregates; compressive strength; elastic modulus; flexural strength; fly ash; high-performance concrete; high-strength concrete; mixture proportioning

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INTRODUCTION

Description of the Louetta Road Overpass Bridge Project

The Louetta Road Overpass Bridge Project in Houston, Texas, shown in Figure 1, was the first High Performance Concrete project in the United States to fully take advantage of HPC from design of the substructure to the superstructure. The project involved the incorporation of a non-standard U-shaped precast beam member in lieu of traditional AASHTO I-shaped precast members. TxDOT conceived the U-shaped member, commonly referred to as the U-Beam by TxDOT personnel, in the late 1980's. While aesthetics was one of the driving forces behind the conception of the U-beam, it was not the only design consideration. Economy, durability, function, and safety were also very important. Advantages of the U-shaped member over more traditional I-shaped members were recognized early on. These included reduced production requirements at the precast plant, fewer members to erect and transport to the site, and members that provide greater stability during erection and shipping. Figure 2 illustrates the cross sectional dimensions of both the U54A and U54B. Two bottom flange widths were developed for various span conditions. The U54A (158-mm flange thickness) allows for 2 vertical layers of prestressing strands in the bottom flange, while the U54B (208-mm flange thickness) allows for 3 vertical layers of prestressing strands. The HPC U-Beams utilized the U54B section detailed above.

Description of the North Concho River US 87 & S.O. RR Overpass Bridge Project

The second HPC bridge project in Texas, completed in January of 1998, is the North Concho River US 87 & S.O. RR Overpass Bridges located in San Angelo, Texas. This project, shown in Figure 3, utilizes a standard AASHTO Type IV precast prestressed beam shape with a cast-in-place decorative pier to enhance the aesthetics of the bridge within the surrounding park area. The state of Texas annually produces the largest quantity of precast concrete components in the nation for use in highway structures. The San Angelo project allowed for TxDOT to demonstrate the application of HPC on the standard AASHTO

precast shape and the most widely utilized bridge system within the state of Texas.

Definition of High Performance Concrete (HPC)

To date, there is no unique definition of HPC. In broad terms, it can be defined as concrete that meets special requirements that cannot be obtained using conventional concrete. Special requirements could be an enhanced property over others such as strength, elastic modulus, durability, permeability, economics, special construction practices, or a combination of them. For the purpose of this project, HPC is defined as concrete with the following criteria:

1. a minimum compressive strength of 90.3 MPa (13,100 psi) at 56 days.
2. a minimum compressive strength of 60.6 MPa (8,800 psi) at one day.
3. a minimum elastic modulus of 41,300 MPa (6,000 ksi) at 1 day.
4. a flexural strength of 830 Sqrt (f'_c), in kPa (10 Sqrt (f'_c), in psi).
5. flowing concrete, a minimum slump requirement of approximately 229-mm (9-in.) at the time of casting.

MIX DESIGN DEVELOPMENT AND OPTIMIZATION

Fundamental Mechanisms of High Strength/High Performance Concrete

In the development of mix proportions for high strength/high performance concrete (HS/HPC), the content of three components must be optimized to produce a mix design satisfying hardened or fresh concrete property requirements. These include the strength of the coarse aggregate, the strength of the mortar, and the strength of the mortar-to-aggregate bond. The load-bearing capacity of HS/HPC is determined by the weakest of these three components. This concept of the weakest component controlling the strength of the concrete is referred to as the "Three Link Chain" concrete theory and is schematically presented in Figure 4. In producing the most efficient mix design utilizing readily available materials, it is important to optimize each component to maximize the strength potential. The compatibility among the constituents plays a very important role in the development of high strength mixes as discussed herein.

Optimization of the Mix Design

In an effort to develop a HS/HPC mix that was economical, the use of locally available materials was emphasized. Only if the required material properties could not be attained using readily available materials in Texas would outside materials sources be considered. A variety of materials and sources were initially selected to investigate the interaction of the various constituents. A laboratory trial phase was initiated at the Construction Materials Research Group at the University of Texas in Austin to develop and optimize the

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materials selected for the most promising mix designs. The following sections detail the various materials that were selected for this initial trial phase.

Coarse Aggregate -- Five types of coarse aggregate were used in the overall study, namely crushed gravel (CG, Nominal Maximum Size of Aggregate=19-mm), trap rock (TR, NMSA=19-mm), dolomitic limestone (LS and LL, NMSA=13 and 19-mm), and calcitic limestone (CL, NMSA=19-mm).

Sand -- Local concrete sand from the Colorado River was used in the overall study. It had a specific gravity of 2.63 and a fineness modulus of 2.45.

Portland Cement -- Commercially available Type I cements designated as BI/II and CI, and Type III cements designated as AIII, CIII and LIII were used. They conformed to ASTM C150-94, "Standard Specification for Portland Cement".

Fly Ash -- Class C fly ash from three different sources (J, D and L) and Class F fly ash (R) were used in varying amounts. They conformed to ASTM C618-94a, "Standard Specification for Fly Ash and Raw and Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete".

Chemical Admixtures -- Two combinations of chemical admixtures, each containing a retarder and a superplasticizer, were used. One combination contained a naphthalene based superplasticizer conforming to ASTM C494-92 for Type A/F, and a retarder conforming to ASTM C494-92 for Type B. The other combination contained a naphthalene based superplasticizer conforming to ASTM C494-92 for Type A, a retarder conforming to ASTM C494-92 for Type B.

LABORATORY MIX DESIGNS

In the summer of 1993, more than 75 trial batches were made using locally available materials at the laboratory of the Construction Materials Research Group at the University of Texas in Austin. As an initial starting point, local experience and guidelines provided by the ACI Committee 211 were followed to come up with an initial set of mix proportions for the HS/HPC mix design. The mix designs were later adjusted through laboratory trial mixes designed to optimize the performance of the constituent materials used. It was clear based on previous trial mixes conducted at The University of Texas at Austin that a HRWR and an ASTM Class C fly ash would be required within the mix design to attain 56-day strength requirements in the 103.4 MPa (15,000 psi) strength range. In addition, an ASTM Type III cement would more than likely be required to meet release strength requirements at 24 hours although two ASTM Type I cements were considered. Initial laboratory trial mixes focused on trial mixes with these constituents and a variety of aggregate sources. Mineral admixtures such as silica fume and slag were not considered since these materials were not locally available.

Cylinder specimens were cast in 102-mm x 203-mm (4-in. x 8-in.) plastic cylinder molds and moist-cured in accordance with ASTM C192-90a "Standard Practice for Making and Curing Concrete Test Specimens in the

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Laboratory" until the test age. The specimens were tested in compression at the ages of 1, 3, 7 and 28 days. The average of three tests was reported as the compressive strength.

Elastic modulus was determined using 102-mm x 203-mm (4-in. x 8-in.) cylinder specimens from a limited number of mixes. Elastic modulus was investigated at early and later ages on standard and accelerated heat cured cylinders.

Using a temperature-matched curing system, two of the laboratory trial mixes were subjected to accelerated heat curing. The curing regime applied to these trial mixes consisted of an approximate four to five hour preset time followed by an eight hour soaking time at 88°C (190°F). This was designed to investigate the effects of higher curing temperatures on both the compressive strength and elastic modulus of the concrete. It was anticipated that higher curing temperatures would be encountered at the beam end block locations that would impact the concrete properties. It was important to closely simulate the concrete in the member to verify that the project specifications and guidelines were being met. Higher end block temperatures within the members were noted during casting of the beams and are discussed in further detail later in this report.

FIELD TRIAL MIXES

Following the successful development and optimization of the laboratory mix design, a field trial mix phase was initiated to batch the mix design at Texas Concrete Precast Plant in Victoria, Texas where the beams would later be fabricated. The purpose of this phase was two-fold. First, to investigate the strength loss of the mix design that typically occurs when a laboratory mix design is utilized in the field. Secondly, the field trial mixes were performed at the precast plant for plant personnel to gain added experience with HRWR's and the use of mineral admixtures. Table 1 illustrates the final mix design used in production of the precast/prestressed beams.

LABORATORY TRIAL MIX CONCRETE RESULTS

Coarse Aggregate Effects

Three different aggregate contents 36, 40, and 44 percent and five aggregate types were investigated within this study. The coarse aggregate type and gradation played an important role in the compressive strength development of the concrete mix designs investigated. The trap rock (TR), which is the hardest and densest aggregate selected for the development of the mix design, resulted in a lower compressive strength when compared to the dolomitic limestone (DL) for the same mix proportions and materials. The difference between the stiffnesses of the mortar and trap rock caused stress concentrations at higher stress levels resulting in failure of the cylinders at a lower compressive

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value. Cracks generally formed in the cement paste around the aggregate rather than through it. The dense trap rock acted as stress riser under high stress levels. The crushed river gravel mix designs investigated also tended to develop lower compressive strengths. The crushed river gravel tended to fail in the range of 75 to 83 MPa. Fracture of the aggregate was clearly visible during compression testing with no strength gain noted beyond 83 MPa (12,000 psi) at long-term testing ages. The crushed river gravel (CG) also provided poor bond characteristics, which is highly desirable for HPC due to their partially smooth surfaces. The calcitic limestone (CL) provided good compressive strength development, however the elastic modulus development was not suitable for the beam requirements of 41.3 kPa (6,000 ksi) at release and later-ages.

Concretes with smaller maximum size aggregate gradation of 13-mm (0.5-in.) yielded slightly higher compressive strength test results than the larger maximum size aggregate gradation of 19-mm (0.75-in.). The smaller aggregate size, and consequently higher surface area for a given aggregate content, results in a lower bond stress at a given load level and consequently higher strength. The concrete tends to act more like a homogenous material. For all aggregate types, increasing the coarse aggregate content beyond 40 percent did not appear to benefit the compressive strength development. However, the elastic modulus appeared to be a function of the coarse aggregate content and type. The elastic modulus development for different aggregates are illustrated in Figures 5, 6, and 7 for 36, 40, and 44 percent aggregate contents from the laboratory trial mixes. The only coarse aggregate that exhibited an elastic modulus below the empirical equation recommended by ACI Committee 318 was the calcitic limestone. The elastic moduli of the calcitic limestone exceed the empirical equation recommended by ACI Committee 363 for high strength concrete. The lower elastic modulus values may be attributed to the abundance of calcite, which is a soft mineral.

In addition to the elastic modulus, the flexural strength of the trial mix designs was monitored. The flexural strength for different aggregates are illustrated in Figures 8, 9, and 10 for 36, 40, and 44 percent aggregate contents from the laboratory trial mixes. Increasing the coarse aggregate content appeared to result in a slight reduction in the flexural strength. Increasing the aggregate content without a change in the aggregate size results in an increased interface area. This results in a potentially weaker zone than either mortar or aggregate resulting in a reduction in the flexural strength.

Based on the results of the trial mixes, the 13-mm (0.5-in.) dolomitic limestone from central Texas resulted in the aggregate most suitable to meet the required compressive strength and modulus of elasticity specified in the project specifications. The dolomitic limestone with its angular shape and compatible elastic modulus to the mortar provided an aggregate with excellent bond characteristics and homogeneous performance. The aggregate was also locally available for the precast plant subsequently resulting in an economical mix design.

It may be noted that no one empirical equation best represents the elastic modulus or flexural strength characteristics for HS/HPC. Any elastic modulus or

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flexural strength value that is assumed based on a function of the compressive strength for the design of HS/HPC members by the design engineer, should be a.) verified through a trial field batching series on the specific mix design to be used by the fabricator or b.) based on the documented historical property performance of the mix design. An alternate method for the design engineer would be to utilize a performance-based specification system, as used on these two bridge projects; requiring specific mechanical properties based on the values selected in design. The fabricator would then be required to develop or utilize a mix design that met said properties. If current code empirical equations are used for HS/HPC to estimate the elastic modulus of the concrete (either ACI 318 or 363 empirical equations), the design engineer should be aware that the structural performance, primarily serviceability, of the bridge may vary from the original design. Naturally, the variation between design assumptions and property performance is more dramatic for HPC bridges where span conditions have been optimized for efficiency and concrete performance is very dependent on material selection and interaction.

Temperature Effects

Two laboratory trial mix designs were subjected to accelerated heat curing to investigate any variation in compressive strength and elastic modulus due to temperature effects. Five curing temperatures were selected. Accelerated temperatures of 43, 66, and 88°C were selected. Furthermore, cylinders were also standard cured (no accelerated heat curing) and cured under a hydration chamber condition. The hydration chamber involved the curing condition of cylinders with no external ambient influences. One of the two mix designs incorporated 30 percent fly ash replacement. Figures 11 and 12 illustrate the compressive strength and elastic modulus results. While the results varied between the two mix designs, the elastic modulus results appeared independent of the accelerated heat curing and primarily a function of the compressive strength of the concrete. The mix design incorporating fly ash replacement resulted in higher compressive strengths and elastic modulus. The use of an ASTM Class C fly ash resulted in supplementary beneficial performance and economic aspects. From an economic standpoint, the fly ash replacement reduced the cost of the cementitious material by approximately twenty-five percent. The beneficial fresh concrete properties of the fly ash included improved workability, improved finishability, and a reduction in the initial concrete temperatures during hydration. The impact of hydration temperature is discussed in further detail in the preceding section. The beneficial hardened concrete properties of the fly ash included improved long-term strength and reduced permeability.