# Bridge Views

Issue 78 September/October 2014


*Inside This Issue...*




SPONSORED BY

U.S. Department of Transportation
**Federal Highway Administration**

CO-SPONSORED BY
NATIONAL CONCRETE BRIDGE COUNCIL

PORTLAND CEMENT ASSOCIATION

PRECAST PRESTRESSED CONCRETE INSTITUTE

NATIONAL READY MIXED CONCRETE ASSOCIATION

AMERICAN SEGMENTAL BRIDGE INSTITUTE

CONCRETE REINFORCING STEEL INSTITUTE

EXPANDED SHALE CLAY AND SLATE INSTITUTE

POST-TENSIONING INSTITUTE

WIRE REINFORCEMENT INSTITUTE

SILICA FUME ASSOCIATION


## Dimensional stability of cementitious grouts in prefabricated bridge connections – Ensuring good performance


*Igor De la Varga, FHWA, Zachary B. Haber, FHWA, Benjamin A. Graybeal, FHWA*


### Prefabricated Bridge Elements and Connections

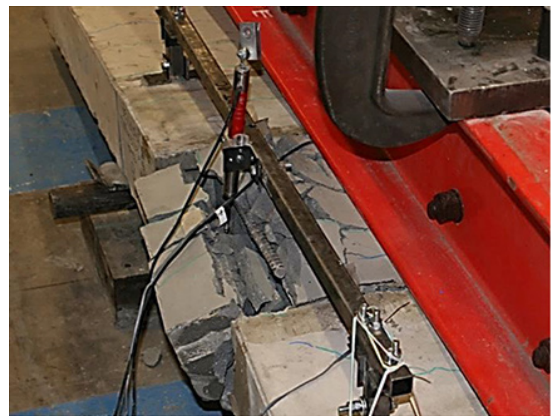

Fig. 1. Reinforcing bar-to-grout bond failure during fatigue loading in lab experiments carried out at Federal Highway Administration's Turner Fairbanks Research Center.

Accelerated bridge construction (ABC) has become common for both new and replacement bridge construction. The majority of ABC projects rely on using prefabricated bridge elements and systems (PBES) to meet tight construction windows. Prefabricated bridge systems commonly depend on field-cast cementitious grouted connections between elements for structural continuity. Thus, the mechanical properties of these grouts directly impact the short- and long-term performance of prefabricated bridge structures. Currently, a broad-scope research project on the performance of field-cast cementitious grouts and their use in PBES connections is being conducted at the FHWA Turner-Fairbank Highway Research Center (TFHRC). This article presents one of the focus points of the aforementioned project: evaluation of dimensional stability of typical non-shrink cementitious grouts that may be used in PBES connections. Results from a series of material-level tests on the dimensional stability properties of non-shrink grouts are discussed. The implication of dimensional stability on the system-level behavior of PBES connections is also discussed along with some possible strategies for mitigating poor dimensional stability.

### Material-Level Behavior: Evaluation of Dimensional Stability

The research study mainly focuses on the evaluation of the dimensional stability of commercially-available non-shrink cementitious grouts that can be used for connecting prefabricated concrete bridge elements[1]. Some of the results of four cement-based grouts, named as C1, C2, C3, and C4, are presented in this article. In this study volume changes have been assessed from a fundamental point of view, measuring pure expansion/shrinkage deformations in both sealed (i.e., autogenous) and drying

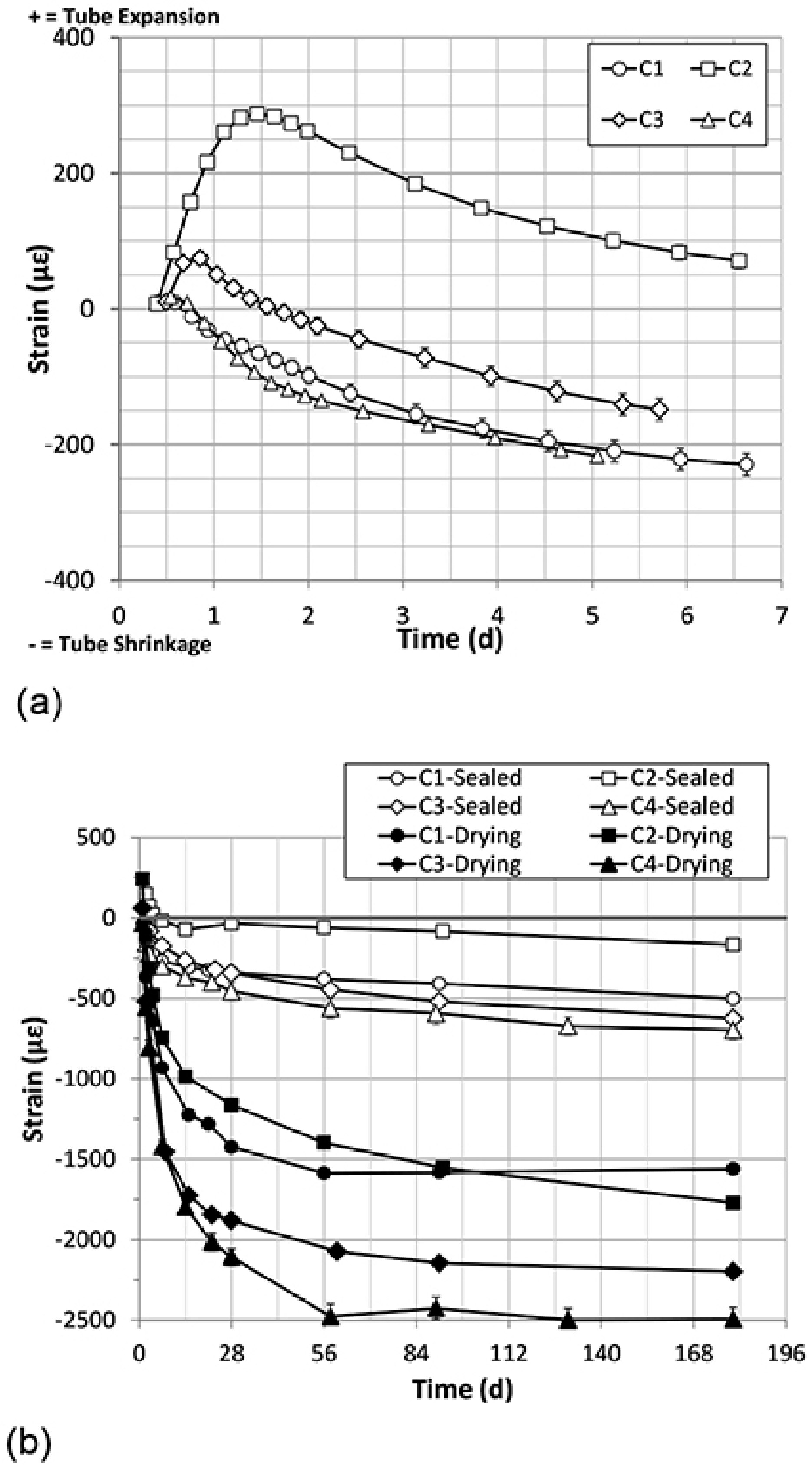


Fig. 2. Dimensional stability test results: (a) Autogenous (sealed) deformations measured from time of set via ASTM C1698, and (b) long-term deformations in both sealed and drying conditions measured from the age of 1 d via ASTM C157

conditions. This was done using both ASTM C1698[2] (Figure 2-a) and ASTM C157[3] the latter for long-term results (Figure 2-b). The curing condition of field-cast grout materials is important because some PBES connections will be completely sealed, while others will be partially exposed to the environment (drying).

As observed in Figure 2-a, and despite being designed as “non-shrink”, the grouts show autogenous shrinkage at some point, either preceded by a flat region (C1, C4) or by an initial expansion (C2, C3) during the first hours. Long-term shrinkage results (Figure 2-b) show a considerable amount of autogenous shrinkage (about 500-600 με) for C1, C3, and C4. The large expansion observed in C2 in Figure 2-a helps in reducing most of its final shrinkage (about 200 με). However, it has been stated that it is the rate of shrinkage (i.e., the slope of the autogenous shrinkage response) what makes the material more prone to shrinkage cracking, rather than the total shrinkage. Drying shrinkage is at least 1000 με greater than sealed shrinkage in all cases, due to the drying effect of the capillary pores.

Some of the possible consequences of poor dimensional stability on the system-level behavior of PBES connections with different field-cast cementitious grouts were observed in a recent and related study at the TFHRC. A series of precast deck panel connection tests were carried out to advance the understanding of deck-level connections under realistic performance demands.[4] A number of parameters frequently considered during the design of these connections were investigated including three different cementitious grouts; the C2 grout shown in Figure 2 was one of the included grouts. Prior to testing, a number of deck panels with C2 grout exhibited significant shrinkage cracking in the grouted connection region (Figure 3-a). Deck panels were subjected to low- and high-level fatigue loading and then monotonic loading until failure. Upon application of load, preexisting shrinkage cracks grew and propagated continuously during fatigue cycles which resulted in bond deterioration between reinforcing bars and the grout material. In many cases, these deck panels failed during flexural loading due to bond failure of the reinforcement, which occurred prior to reinforcing bar yielding (Figure 3-b). This research also suggests that the bond strength between precast concrete and field-cast grout could be compromised in the presence of excessive shrinkage; research is currently underway to investigate the correlation between dimensional stability and bonding of grout to precast concrete. Shrinkage cracking in connection zones could also lead to infiltration of corrosive agents and loss of flexural stiffness. On the other hand, materials that are excessively expansive could introduce forces into the bridge system not accounted for in the design, which could cause unexpected structural damage.

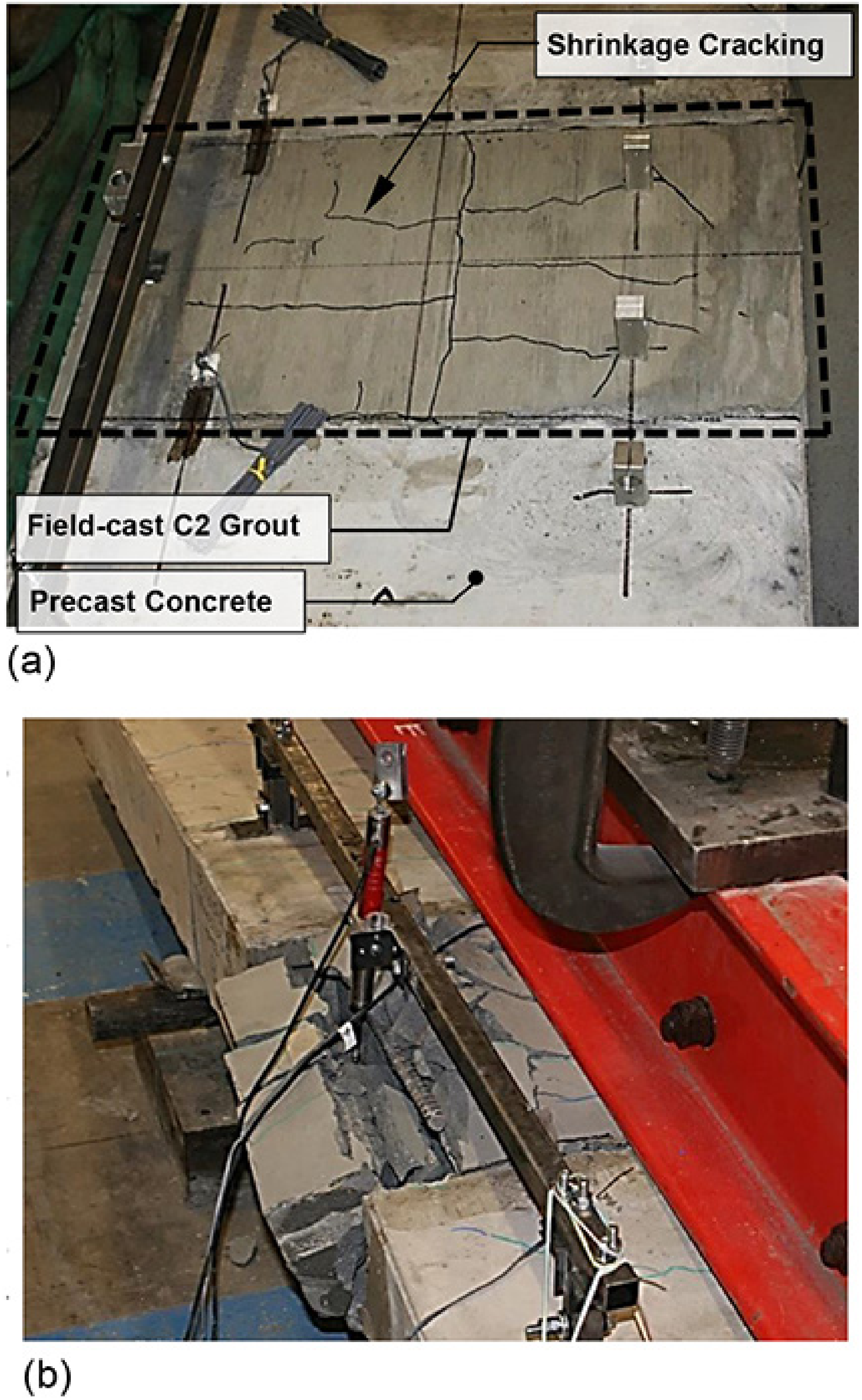

Fig. 3. Observations from deck panel connection tests using C2 grout: (a) Shrinkage cracking observed prior to testing, and (b) Reinforcing bar-to-grout bond failure during fatigue loading.

### Strategies for Mitigating Dimensional Stability Issues

Different strategies are available for mitigating issues related to dimensional stability such as excessive shrinkage, which was observed in the grouts discussed in this paper. In this study, two different strategies were evaluated: internal curing (IC) and the use of a fiber reinforced ultra-high performance concrete (UHPC). Non-shrink cementitious grouts are often pre-packaged and can be extended using small aggregate for volumetrically large pours. In this study, pre-wetted lightweight aggregates (LWA) were added to two of the previously tested grout systems to provide IC. As observed in Figure 4, IC helps to mitigate most of the autogenous shrinkage in both C3 and C4 grouts, as well as it reduces drying shrinkage by half. The autogenous shrinkage reduction is a result of prolonged internal saturation (i.e., higher internal humidity). This will have a direct effect on the stress developed in the material, as the size of the pores that are being emptied during hydration is larger than that of grouts without IC. The partial drying shrinkage reduction would presumably correspond to two different reasons: 1) mitigation of autogenous (or internal) drying, and 2) extension in the time it takes to reach equilibrium with the local drying environment as it may take longer to empty out the same-sized pores in the system with IC versus the system without IC. Drying shrinkage in the UHPC material was an order of magnitude lower than the cement-based grouts without IC. Low shrinkage in UHPC material can in part be attributed to the presence of a high volume of steel fiber reinforcement in the mix.

For more information, the authors may be contacted at the following: Igor De la Varga, Concrete Materials Engineer, SES Group, FHWA Turner-Fairbank Highway Research Center, igor.delavarga.ctr@dot.gov, Zachary B. Haber, Research Bridge Engineer, PSI, Inc., FHWA Turner-Fairbank Highway Research Center, zachary.haber.ctr@dot.gov, Benjamin A. Graybeal, Team Leader – Bridge and Foundation Engineering Research, FHWA Turner-Fairbank Highway Research Center, benjamin.graybeal@dot.gov.

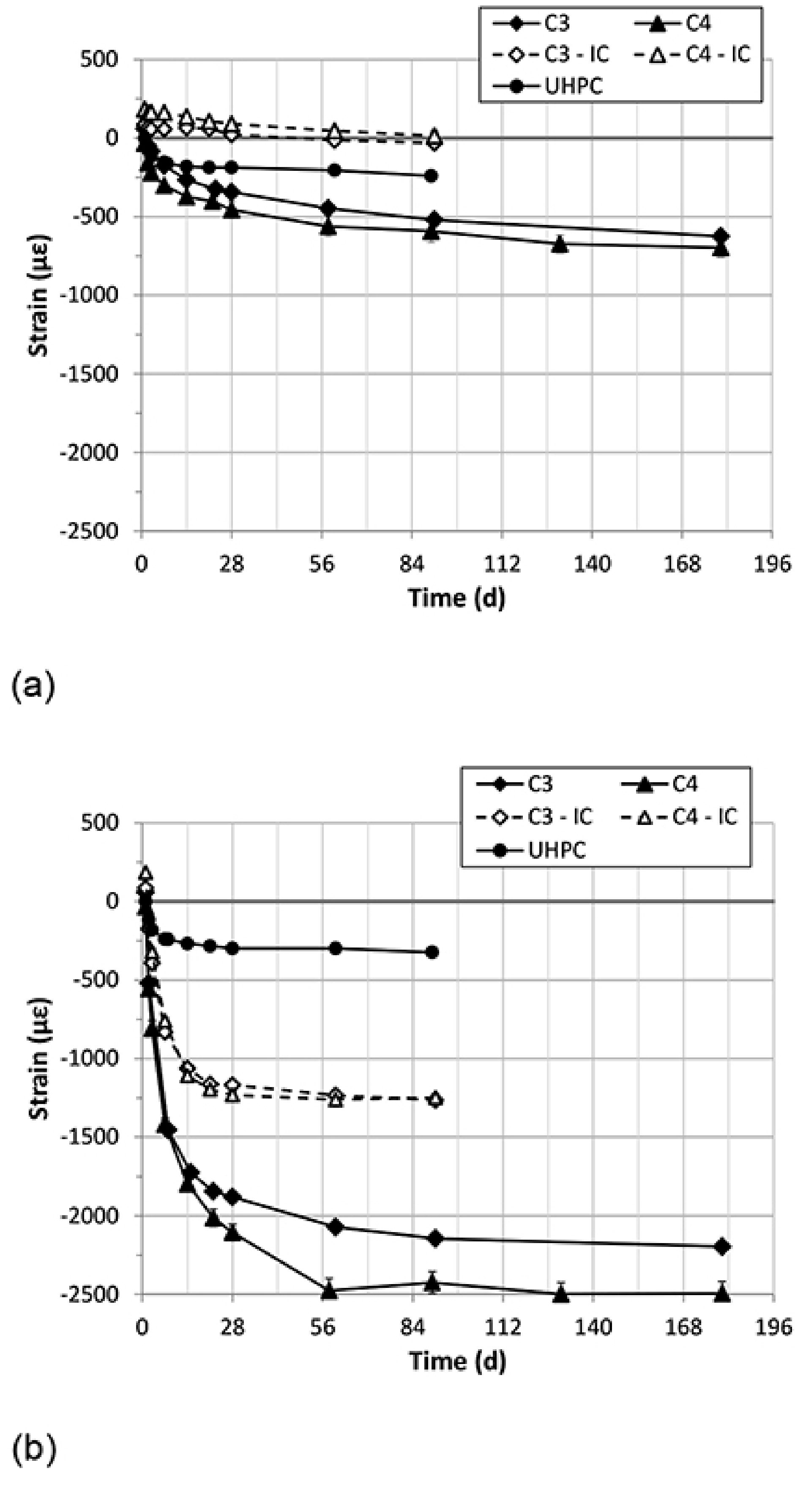


Fig. 4. Results from tests with using dimensional stability improvement strategies: Shrinkage reduction in (a) sealed and (b) drying conditions using IC or an UHPC material via ASTM C157.

## References

1. De la Varga, I., Graybeal, B. "Dimensional stability of prebagged cementitious grouts for prefabricated bridge element connections," Proceedings. 2014 National

Accelerated Bridge Construction Conference, Miami, FL, USA, December 4-5, 2014.

2. ASTM C1698-09. “Standard Test Method for Autogenous Strain of Cement Paste and Mortar”

3. ASTM C157-08. “Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete”

4. Yuan, J., Graybeal, B., Haber, Z. B. “Field-cast connections for prefabricated bridge elements,” Proceedings. 2014 National Accelerated Bridge Construction Conference, Miami, FL, USA, December 4-5, 2014 .

## Low-Cracking High-Performance Concrete Bridge Decks

*David Darwin, University of Kansas*

Research dating back over 25 years has established the key factors that control bridge deck cracking – age, bridge deck type, concrete material properties, site conditions, curing, and even date of construction. An understanding of these factors has been put to good use in a two-phase pooled-fund study under the direction of the Kansas Department of Transportation in conjunction with 18 other state departments of transportation and the Federal Highway Administration.

Evaluation of over 150 bridge decks, most supported by steel girders, has demonstrated that even in the best preforming bridges, crack density will increase to some degree with increasing age. The general observation, however, is that those decks that perform well during the first three years after construction will perform well throughout the life of the deck. Monolithic decks tend to perform much better than decks that are constructed with overlays that are placed as part of initial construction, largely because cracks in the subdeck tend to reflect through the overlays and shrinkage in the overlays tends to be restrained by the subdeck, resulting in additional cracking in the overlay.

Fig. 1. Pre-cut, rolled, wet burlap is placed within 10 minutes of strike-off.

The choice of concrete mixture proportions has a large impact on cracking performance, in some cases in unexpected ways. Mixtures with higher volumes of cement paste (that is, more water, more cement, or the combination of the two) exhibit greater drying shrinkage, which results in greater transverse cracking due to restraint provided by the girders. This restraint tends to be greater for steel-girder bridges than for precast, prestressed girder bridges, although the latter can result in increased cracking if camber is not controlled and continues to increase over time. Increasing concrete slump leads to increased settlement cracking, the principal reason that transverse cracks form directly above and parallel to the top reinforcing steel. Increasing air content tends to reduce cracking because entrained air acts as a workability agent and air bubbles do not shrink. As somewhat of a surprise to many in the field, increasing compressive strength correlates with increased cracking. This has been observed in a number of states, where higher-strength concretes, whether used for early strength or to reduce permeability, correlate with increased cracking. This increase in cracking results because high-strength concrete creeps less than lower-strength concrete, even at the same ratio of stress to strength. Reduced creep is a positive property for high-strength concrete in compression, such as used in columns in high-rise buildings, but reduced creep tends to limit the relief of tensile stresses in bridge decks, resulting in increased cracking.

Extra finishing of the deck can lead to increased plastic and drying shrinkage cracking as it works the coarse aggregate below the surface while increasing the thickness of the (high shrinkage) paste at the surface, while at the same time delaying the initiation of curing.

Site conditions that lead to rapid evaporation of bleed water from the surface of bridge decks or concrete temperatures that exceed those the bridge girders at the time of placement, result in, respectively, increased plastic shrinkage and thermal cracking.

One of the most interesting aspects of the study of older bridge decks involves the observation that decks that were cast 30 years ago exhibit less cracking than those that were cast 10 or even 5 years ago. The principal changes in construction over that period have involved the use of more finely ground cement, higher-slump concrete, and the switch from buckets and conveyor belts to pumps as the principal method for placing concrete. The more finely ground the cement, the smaller the pores in the hardened cement paste within concrete and the greater the drying shrinkage. Higher slump, whether it is obtained with more cement and water or with a plasticizer, results in more settlement cracking, and pumping concrete usually involves the requirement for higher slump and higher paste content, both of which can add to the potential for settlement and drying shrinkage cracking.

Stresses in the bridge decks resulting from either the order of placement of the concrete during construction or traffic loads has been shown to play a much smaller role in cracking than factors dealing with material properties or construction.

With this understanding bridge decks have been constructed as part of the multi-state pooled-fund study. The specifications for these bridge decks involve an overall approach aimed at reducing plastic, settlement, thermal, and drying shrinkage cracking. This approach involves the use of low cement and water contents; low slump; moderate, not high strength; temperature control of the concrete; minimum finishing; and an early start and extended curing.

The specifications for low-cracking high-performance concrete (LC-HPC) involve the use of concrete with increased aggregate content and aggregate size, along with an optimum aggregate gradation to allow the use of concrete with the cement content of 540 lb/yd3 (320 kg/m3) or less. Water cement ratios have been in the range of 0.43 to 0.45 to help limit concrete compressive strength. Air contents range from 6.5 to 9.5%, and the designated slump range is 1.5 to 3 in. (40 to 75 mm). Not unexpectantly, one of the challenges has been to get contractors to use slumps in this low range. To help limit both plastic shrinkage and thermal cracking, the temperature of concrete, as delivered to the site, is specified as 55 to 70°F (13 to 21°C). In cold weather, the temperature must be maintained for both the girders and the deck.

The use of buckets and conveyor belts (the latter with a low drop from the belt to the deck) is emphasized, although the majority of the decks have been placed using pumps. Vertically mounted internal gang vibrators, spaced at 1 ft (300 mm), are used to improve consolidation and thus reduce settlement cracking. To help limit the cement paste at the surface of the deck, concrete finishing is minimized through the use of a single-drum roller screed (including double-drum roller screeds with one roller immobilized). This has worked well for the concretes with an optimized-aggregate gradation and controlled temperature, even at low slump.

After concrete placement, fully saturated, presoaked burlap is placed within 10 minutes of strike-off (see in Figure 1) and kept constantly wet with spray hoses until the concrete has set. Soaker hoses are then placed and the burlap is covered with white plastic. Curing continues for 14 days. To allow the concrete to dry slowly, the deck is sprayed with a curing compound upon removal of the burlap. The curing compound is maintained for seven days. The deck forms are removed within two weeks of termination of curing so that the deck can dry from both sides. The use of stay-in-place forms has a disadvantage in that the deck dries from only one side, which doubles the moisture gradient.

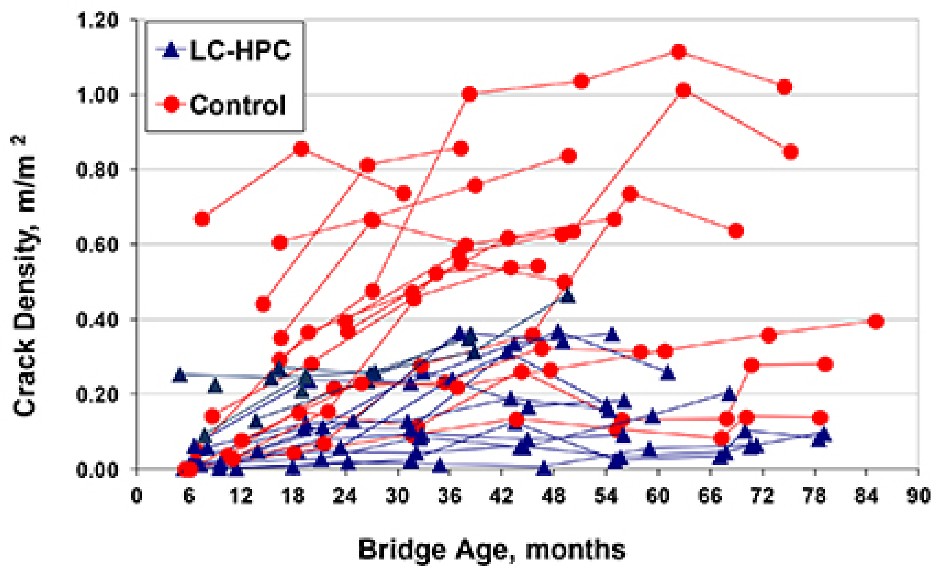

Fig. 2. Crack density versus age for LC-HPC and matching control decks. LC-HPC decks have performed better than the matching control decks in every case.

The results of the study, which includes an equal number of control decks constructed using conventional procedures, are summarized in Figure 2, with crack density shown in linear meters per meter of bridge deck as a function the age of the bridge. The control decks were selected to match the LC-HPC decks based on structure type and traffic loading. As shown in Figure 2, the LC-HPC decks have performed far better as a group than the control decks. What is not evident from the figure is that the LC-HPC decks have performed better than the matching control decks in every case. Full comparisons are available at [https://iri.drupal.ku.edu/node/43](https://iri.drupal.ku.edu/node/43).

In the next phase of the pooled-fund study, additional techniques are being applied to reduce cracking. These techniques include the use of fibers, potentially to reduce plastic shrinkage cracking; internal curing using pre-wetted lightweight aggregate combined with slag cement as a replacement for portland cement combined with a small additional replacement with silica fume; and the use of shrinkage-reducing admixtures. Bridges using these techniques are both in the planning and construction stages, and the effectiveness of these additional techniques will become apparent over time.

**Further Information**

For further information about this project, please contact the author at daved@ku.edu.

## Reducing Steel in Bridge Decks

*John M. Holt, P.E., Texas Department of Transportation, Amy Smith, P.E. Texas Department of Transportation*

The most structurally taxed element in a bridge is its deck. A typical bridge deck receives constant pounding from heavy truck wheels, is the element most exposed to the environment, and in some regions treated with corrosive de-icing chemicals several months of the year. If a bridge needs rehabilitation work, it is highly likely that the deck's condition is the cause.

As a result, decks consume much time from owners before, during and after their construction – specifying quality materials, holding pre-pour meetings, ensuring adequate construction and curing, monitoring and inspecting for cracks and corrosion, and placing and replacing waterproofing and overlays – all in an effort to ensure a long deck life. It should be no surprise, then, that decks have been the subject of numerous research studies and reports, many of which focus on cracking and reinforcement corrosion.

**Role of a Deck's Top Mat of Reinforcement**

The top mat serves three primary purposes:

- provide adequate strength for wheel loads at the strength limit state,
- provide strength in the overhangs for traffic rails and barriers subjected to vehicular impacts at the extreme event limit state, and
- control crack widths for long-term serviceability.

The second and third purposes are the top mat's most important, as research has shown conventional top mat reinforcement in conventional decks does not approach yield when a deck reaches its ultimate resistance to wheel loads since the failure mechanism typically is punching shear[1]. Conventional decks on multi-girder superstructures

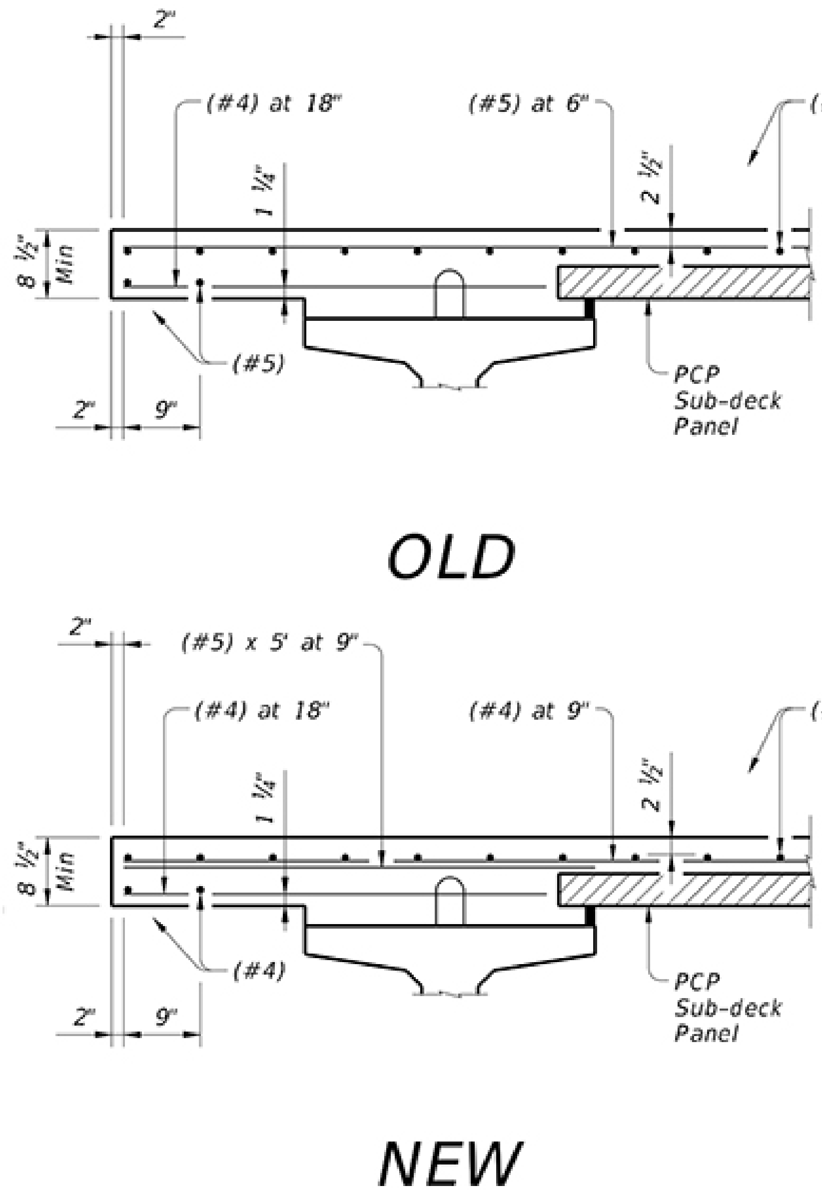


Fig. 1. A comparison between the old and new TxDOT deck configurations.

typically do not behave in flexure at the strength limit state as some design specifications might imply, and as such, do not require the amount of reinforcement such a design methodology leads to.

Since a deck's top mat of reinforcement is the mat most exposed to de-icing chemicals and most likely to exhibit corrosion problems, it deserves great attention from designers and owners.

**Optimized Reinforcement in Top Mat**

In recognition of a top mat's true behavior, TxDOT is migrating from a reinforcement scheme based on the traditional design of AASHTO LRFD Specifications Article 9.7.3[2] to one more representative of an empirical design. This new design uses less steel in the top mat, although not as little as allowed by AASHTO (Article 9.7.2), with the idea that the amount of steel is optimized to control crack widths and is reduced in volume in order to reduce future corrosion potential.

Previously, the top mat typically used No. 5 bars at 6-in spacing in the transverse direction and No. 4 bars at 9-in spacing in the longitudinal direction in both 8-in and 8.5-in-thick decks. The weight of this top mat is 3.0 lbs/SF. The new top mat has No. 4 bars at 9-in spacing in each direction, supplemented with short No. 5 bars at 9-in spacing in the overhang portions to ensure an adequate foundation for traffic railings. This new top mat weighs 1.8 lbs/SF, a reduction of 40 percent. See Figure 1 for a comparison between the old and new TxDOT deck configurations.

The selection of 9-in spacing for the top mat was based on inspections and observations of in-service decks which found adequate crack control was being obtained by the No. 4 bars at 9-in spacing in the longitudinal direction. This amount of reinforcement is 50 percent more than the minimum required by AASHTO – 0.18 sq in/ft – for an empirical deck design.

TxDOT treats transverse deck edges with much more reinforcement, decreasing the bar spacing from 9-in to 3.5-in in the last 4-ft of the bridge deck, perpendicular to the edge. This isolated densification at joints is combined with a 2-in thickening of the deck over a 4-ft width to provide adequate deck strength without the need for diaphragms or other means of deck support and helps control cracking perpendicular to the joint.[3]

A small number of bridges were built with an empirical deck design in Texas in the early to mid-1980s. These bridge decks were fully cast -in-place (CIP) and used an empirical deck design. Recent inspections of these decks found them performing comparably to decks with traditional reinforcement patterns.

TxDOT is using this optimized top mat on its bridges in conjunction with prestressed concrete sub-deck panels. These sub-deck panels are preferred by contractors and are used on the vast majority of Texas' bridges. AASHTO LRFD Specifications disallow an empirical deck design with stay-in-place concrete formwork. However, TxDOT-funded research[4, 5, 6] demonstrated that the empirical deck system with prestressed sub-deck panels performs as well as, if not better than, fully CIP decks. TxDOT's prestressed sub-deck panels are an excellent example of prefabricated bridge elements and provide a stiff, crack-free bottom half of deck.

In another departure from past practice, TxDOT is placing the longitudinal bars closer to the deck surface than the transverse bars (See Figure 1 for reinforcing placement comparison). This recognizes the predominance of transverse cracking in bridge decks. Having the longitudinal bars closer to the deck surface engages these bars in crack control sooner.[7]

**References**


1. Beal, D.B., Strength of Concrete Bridge Decks, Research Report 89, New York State Department of Transportation, July 1981.
2. AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012.
3. Coselli, C.J., E.M. Griffith, J.L. Ryan, O. Bayrak, J.O. Jirsa, J.E. Breen, and R.E. Klingner. 2004. Bridge Slab Behavior at Expansion Joints, FHWA/TX-05/0-4418-1, University of Texas, Austin, TX.
4. Fang, I.-K., J.A. Worley, N.H. Burns, and R.E. Klingner. 1986. Behavior of Ontario-Type Bridge Decks on Steel Girders, FHWA/TX-86/78+350-1, University of Texas, Austin, TX.
5. Tsui, C.K., N.H. Burns, and R.E. Klingner. 1986. Behavior of Ontario-Type Bridge Deck on Steel Girders: Negative Moment Region and Load Capacity, FHWA/TX-86/80+350-3, University of Texas, Austin, TX.
6. Kim, K.H., J.M. Dominguez, R.E. Klinger, and N.H. Burns. 1988. Behavior of Ontario-Type Bridge Decks on Steel Girders, FHWA/TX-88+350-4F, University of Texas, Austin, TX.
7. Durability of Concrete Bridge Decks: A Cooperative Study, 1970, Portland Cement Association.