Application and Modeling of Steel Fiber-Reinforced Concrete for Buried Structures

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Synopsis: The current research shows that the addition of steel fibers to plain concrete is effective in enhancing the tensile ductility and loading capacity of buried concrete structures such as bridges, culverts, and vaults. This paper details the development of a steel fiber reinforced concrete (SFRC) analytical model used in the finite element program, CANDE, and describes the experimental and analytical approach used to test the accuracy of the model. The results of full-scale, in-place load tests on many precast buried SFRC arch structures (composed of less than or equal to 1% steel fibers by volume) correlated well with the CANDE model predictions. The CANDE program exhibits the ability to model the material behavior of SFRC as well as the effects of soil-structure interaction. The analytical and experimental research summarized in this paper leads to the ability to design SFRC for structural applications such as buried bridges, culverts, and vaults.

Keywords: arch; bridge; buried structure; culvert; fiber-reinforced concrete (FRC); precast concrete; steel fiber; tensile behavior.
INTRODUCTION

Steel fibers have been widely used as secondary (temperature-shrinkage) reinforcement for many precast, cast-in-place, and shotcrete applications. Popular structural applications that take advantage of the increased flexural strength of steel fiber reinforced concrete include bridge decks, pavements, tunnel and canal linings, hydraulic structures, pipes, and tilt-up panels. The research reported herein focuses on an application of steel fibers to precast buried concrete structures, where the resistance of the structure is provided only by the steel fiber reinforced concrete member and the surrounding soil.

Predicting the performance of a buried structure to take advantage of the improved ductility is difficult since no standard design methodology exists for steel fiber-reinforced concrete (SFRC) and the analysis of the soil-structure interaction is complex. The analysis is dependent upon many variables such as the geometry and stiffness of the structure, properties of the surrounding soil, the loading conditions, the construction operation, the type and volume of steel fiber in the concrete mix, and the mix design itself. The key structural variables are simulated in the design and analysis software Culvert Analysis and Design (CANDE). According to the Transportation Research Board’s NCHRP program, CANDE is the “premier design and analysis tool for all types and sizes of buried culverts…and is widely used by state highway departments, industry, consulting firms, and universities in the United States, Canada, Europe, Africa, and Australia” (1). This paper details the analytical model for steel fiber reinforced concrete utilized by the CANDE program.

Previous research was performed to determine the flexural properties of steel fiber mix designs with dosages ranging from 0.66 to 4.00% by volume (2). Using the experimental flexural strength properties of first crack stress and rupture stress for each mix design, CANDE was used to analyze numerous geometries and section thicknesses to design a buried precast arch structure. The final precast arch structure made optimum use of the concrete and steel fiber materials while providing a safe design for highway loading. The optimization considered the economics of the steel fibers, the concrete, and the overall weight of the unit for shipping considerations. The end result was a proposed arch structure with a span of 8.5 feet (2.59 m), a rise of 6.0 feet (1.83 m), and length of 6.0 feet (1.83 m).

This paper will report the theoretical background and assumptions used to model the SFRC structure and the experimental and analytical results of an in-place, full-scale load test. Comparisons of a SFRC arch and plain concrete arch are presented to demonstrate the significantly increased load carrying capacity of the SFRC arch for the same geometry and loading conditions.

RESEARCH SIGNIFICANCE

Although research has shown that the addition of steel fibers to concrete improves the ductility and shear capacity of the concrete, no design procedures or codes exist in the United States for the routine design of SFRC structures. This paper presents an analytical model that correlates well with experimental data that can be used to model and design SFRC members.
ANALYTICAL MODEL PURPOSE

The intent of this section is to present the theoretical assumptions and developments for modeling structural members composed of steel fiber reinforced concrete (SFRC). As a result of the study presented herein, the SFRC model was incorporated into a special version of the CANDE finite element computer program especially for the design and analysis of the precast arch described in this paper.

Background

First introduced in 1976 under the sponsorship of the Federal Highway Administration, CANDE was developed for the structural design, analysis and evaluation of buried structures including culverts made of corrugated metal, reinforced concrete, and plastic as well as other soil-structure interaction problems such as underground storage facilities, storm water runoff chambers, retaining walls, tunnel liners, and protective structures. Over the years FHWA has sponsored several upgrades to the program, and AASHTO is currently sponsoring several additional enhancements to CANDE under NCHRP Project 15-28 (3-6).

CANDE is a two-dimensional, plane-strain finite element program with the following key features:

- Incremental construction – the capability to simulate the physical process of placing and compacting soil layers, one lift at a time, below, along side and above the culvert as the installation is constructed.
- Interface elements – the ability to simulate the frictional sliding, separation and re-bonding of two bodies originally in contact. Typically these elements are used between the culvert and soil and between trench soil and in situ soil.
- Soil elements and models – soil elements are high-order continuum elements with a suite of soil models ranging from linear elastic to highly nonlinear. The so-called Duncan and Duncan/Selig soil models are very representative of the nonlinear soil behavior in most culvert installations.
- Beam elements and pipe models – A culvert (or structure) is represented by a connected sequence of short beam-column elements that trace the culvert’s periphery. The material models of beam-elements distinguish between different pipe types. Each material model includes design criteria, which provide measures of safety against potential modes of failure.
- In particular, the reinforced concrete beam element is composed of a plain concrete matrix along with rows of reinforcing steel located at discrete points in the concrete cross section. Concrete behavior is characterized by cracking in tension and plastic hardening in compression. Reinforcing steel behavior is simulated as elastic - perfectly plastic in tension and compression. Design criteria for reinforced concrete culverts include safety factors for steel yielding, ultimate concrete crushing, excessive shear stress, and excessive concrete cracking (crack width estimate). The material model is discussed below.

Plain concrete model

The constitutive model used to simulate the concrete matrix of the reinforced concrete beam element discussed above provides a reference to compare and contrast the new SFRC model presented in this paper. To this end, Figure 1 illustrates the uni-axial stress-strain relationship for a plain concrete specimen that has not been previously loaded.

The plain concrete model in the figure is defined by the following independent parameters:

- $\varepsilon_c$ = compressive strain at initial ultimate strength of concrete
- $\varepsilon_y$ = compressive strain at initial plastic yielding of concrete
- $\varepsilon_t$ = tensile strain when concrete abruptly cracks (stress released)
- $E_c$ = Young’s modulus of concrete in elastic zone
- $f_c$ = ultimate compressive stress of concrete

Plain concrete parameters derived from the above independent parameters include:

- $f_t = E_c \varepsilon_t$ = tensile stress when concrete abruptly cracks (stress released)
- $f_y = E_c \varepsilon_y$ = compressive stress at initial plastic yielding of concrete

To better explain the plain concrete model, let the symbol ’e’ represent an applied strain loading and the symbol ‘f’ denote the corresponding stress, therefore df/de is the stress-strain modulus at various regions on the loading/unloading path. With this understanding, the model’s compression and tension behavior are discussed in turn:
Compression region:
• Initial compression loading is linear elastic when the applied strain is in the range 0 to $e_y$, which implies $df/de = E_c$.
• Plastic hardening occurs when the applied strain is in the range $e_y$ to $e_c$, wherein $df/de = (f_c - f_y)/(e_c - e_y)$ and some plastic strain is accumulated.
• Perfect plasticity occurs when the applied strain exceeds $e_c$, wherein $df/de = 0$ and plastic strain is accumulated.
• Upon unloading from anywhere in the compression region (i.e. the applied compressive strain is reduced) the response is elastic, $df/de = E_c$.

Tension region:
• Initial tension loading is linear elastic when the applied strain is in the range 0 to $e_t$ wherein $df/de = E_t$ (virgin material only)
• Abrupt concrete cracking occurs when the applied strain exceeds $e_t$ for the first time. All stress is released so that $df/de = -f_t$ and tensile-strain-damage begins to accumulate. For all future load-paths, the concrete is assumed non-healing so that $f_t$ and $e_t = 0$ for all time, (meaning the tension triangle in the virgin stress-strain curve has become a flat line).
• Continuing the applied tensile strain beyond the initial cracking strain results in further accumulation of tensile-strain-damage while the stress remains zero, $df/de = 0$.
• Upon unloading from anywhere in the tension region after tension damage has occurred produces no change in stress ($df/de = 0$) until the unloading strain overcomes the accumulated damage strain (i.e., closing of smeared cracks).

The plain concrete model has performed well and has been operative in CANDE since 1980 (7). As a preview to the SFRC model development in the next section, we note that the behavior of plain concrete and fiber-reinforced concrete are very similar in compression. For tension however, fiber-reinforced concrete exhibits much more ductility than does plain concrete.

SFRC model
Existing experimental knowledge as well as experimental knowledge gained during the course of this study was used to guide the development of the SFRC model (2, 8). These experiments included various types and lengths of steel fibers added into the concrete mix at volume percentage ranging from 0 to 4% and tested in uniaxial compression, tension, and in beam bending.

Although the tests exhibited some variability, two key observations appear generally valid:
• The addition of steel fibers has negligible influence on compressive strength or the character of the compressive stress-strain relationships.
• The addition of steel fibers has very little influence on tensile strength, that is, $f_t$ remains practically unchanged. However, the tension ductility is significantly increased as the percent of steel fibers is increased.

The increase in tensile ductility means that rather than an abrupt loss of stress after the concrete cracks, the stress reduces more gradually because the fibers spanning the crack allow some tensile stress to remain, which decreases gradually to zero as tensile strain increases causing more fibers to break or lose anchorage in the concrete.

Based on the these two key observations, a simple alteration to the plain concrete stress-strain model was devised to simulate the behavior of fiber reinforced concrete as illustrated in Figure 2. In this model the steel fibers and concrete are viewed as a mixture, not separate materials.

The SFRC model includes one additional independent parameter beyond that of the plain concrete model, which is called the tensile rupture strain denoted as $e_r$. The parameters in the compression region remain as defined for plain concrete whereas the tension parameters are redefined as follows:
• $e_t = $ tensile cracking strain indicating the initial cracking of concrete (typically the same value as plain concrete)
• $e_r = $ tensile rupture strain indicating the tensile strain level at which all stress is released (inferring that all steel fibers spanning the gap have broken or have lost anchorage)
• SFRC model parameters derived from the above independent parameters are defined as:
  • $r = e_r / e_t = $ ratio of rupture strain to initial cracking strain, $1 \leq r \leq \infty$. Note that for the limiting value $r = 1$, the SFRC model is identical to the plain concrete model.
Fiber-Reinforced Concrete in Practice

- \( f = E_c e_t \) = tensile stress when concrete initially cracks (typically the same value as plain concrete).
- \( E_s = -f_s/(e_r - e_t) \) = negative modulus of concrete in the softening zone.

In the compression region the rules governing the SFRC model are identical to the rules discussed above for plain concrete. In the tension region the rules governing the SFRC model are summarized below where \( df/de \) is the stress-strain modulus at various regions on the loading/unloading path. The symbol \( f \) is the stress associated with the applied strain \( e \).

**Tension region:**
- Initial tension loading is linear elastic when the applied strain is in the range 0 to \( e_t \) wherein \( df/de = E_c \) (virgin material only)
- Concrete cracking begins when the applied strain, \( e \), exceeds initial cracking strain, \( e_t \), for the first time. Tension damage strain, defined as \( e - e_t \), is accumulated during this process. For the case when the applied strain is in the range, \( e_t \leq e \leq e_r \), the stress loss is \( \Delta f = E_s (e - e_t) \) so that the current stress is given by \( f = f_t - \Delta f \). When damage occurs, the virgin stress-strain curve is redefined by assuming a straight line between the origin and the current stress/strain point. Said another way, the left side of the tension triangle becomes flatter and flatter with the triangle apex located at the stress-strain point \( (f, e) \).
- Continuing the applied tensile strain beyond the rupture strain results in further accumulation of tension damage strain and a complete loss of stress, \( \Delta f = f_t \) so that \( f = 0 \). Thus the initial tensile triangle of the virgin stress-strain curve becomes a flat line wherein \( df/de = 0 \).
- Upon unloading from the tension region after partial tension damage has occurred, the unloading modulus \( df/de \) is the current slope of the left triangle leg, representing a small positive stiffness of the remaining steel fibers that are being relieved of tension.
- Upon unloading from the tension region after complete tension damage has occurred there is no change in stress (\( df/de = 0 \)) until the unloading strain overcomes the accumulated damage strain (that is, closing of smeared cracks).

**Load capacity of SFRC model**

When comparing the stress-strain figures of the plain concrete model and the SFRC model, it may appear that the load capacities of the two models are the same. Indeed, this observation is true for a test specimen loaded in pure axial tension wherein the ultimate tensile load for both models is easily computed as \( T = f_t A \) where \( A \) is the cross-sectional area of the test specimen. However for a test specimen loaded in pure bending, it is quite remarkable that the ultimate moment capacity of the SFRC model is appreciably greater than the plain concrete model depending of the value of the parameter \( r \) (the ratio of rupture strain to initial cracking strain). Because this statement may not be intuitively obvious, a derivation of moment capacity is provided in the following development.

Consider a beam specimen in pure bending with a rectangular cross-section of height \( h \) and width \( b \). Assuming planes remain plane for all load levels, the strain profile is linear through the height of the cross section. Thus, the strain profile \( e(y) \) is completely defined by two points for which we choose the bottom fiber and the neutral axis. At the bottom fiber the strain is prescribed as \( e(0) = e^* \) where \( e^* \) is characterized by an \( x \)-multiplier of the initial cracking strain \( (e^* = x e_t) \). By definition the strain at the neutral axis is zero, \( e(y^*) = 0 \), where \( y^* \) is characterized by a \( z \)-multiplier of the cross-section height \( (y^* = zh) \). Figure 3 illustrates these basic assumptions.

The location of the neutral axis \( y^* \) (\( z \)-multiplier) is dependent on the stress profile which in turn is dependent on the magnitude of the prescribed strain \( e^* \) (\( x \)-multiplier). Because the tensile portion of the SFRC stress-strain relation is defined by three piecewise linear regions, we develop the stress profile for three ranges of the \( e^* \):

1. Maximum tensile strain \( e^* \) is in the uncracked virgin range, \( 0 \leq x \leq 1 \)
2. Maximum tensile strain \( e^* \) is in the softening range, \( 1 \leq x \leq r \)
3. Maximum tensile strain \( e^* \) exceeds the rupture strain, \( r \leq x \leq \infty \)

The corresponding stress profiles for these three regions are illustrated in Figure 4 wherein it is reasonably assumed that maximum compressive stress in the top fiber remains within the elastic range. Since the stress profiles are a result of pure bending, the neutral axis (\( z \)-multiplier) is determined by requiring the sum of the forces equal zero. After determining the location of the neutral axis, the internal moment can be computed by integration. Table 1 summarizes the results, where \( M_i = \int f bh^2/6 \) is the
moment that causes initial tensile cracking on the bottom fiber when \( x = 1 \) and is the maximum moment capacity for the case \( r = 1 \) (plain concrete).

Except for the linear range, the equations for the internal moment do not lend themselves to easy interpretation without graphical aid. Figure 5 shows plots of the non-dimensional moment, \( m(x) = \frac{M(x)}{M_r} \), for a family of ratios, \( r = 2, 5, 10, \) and \( 20 \). All families trace the same straight line in the linear range \( 0 \leq x \leq 1 \), then diverge into separate bell-shaped curves in the softening range \( 1 \leq x \leq r \), and asymptotically approach zero in the rupture range \( r \leq x \leq \infty \). It is evident that the peak moment (maximum moment capacity) occurs in the softening range and increases with the \( r \)-ratio. For \( r = 20 \), it is observed that the moment capacity is 1.8 times that of plain concrete.

From Table 2 we see that fiber reinforced concrete with an \( r \)-value on the order of 100 more than doubles the moment capacity of un-reinforced plain concrete. This finding provides the impetus to continue the model development for real-world applications.

**CANDE finite element development**

The last portion of the overall analytical development is to incorporate the SFRC constitutive model into a beam-column element and outline an overall solution strategy. The nonlinear solution strategy is well documented in other publications (2, 4, 7) and is outlined below.

The overall finite element methodology is based on a displacement-formulation of virtual work with the following assumptions:

- Incremental load steps, \( i = 1 \) to \( N \)
- Bernoulli-Euler beam theory with axial deformation
- Hermetian interpolation functions for transverse bending displacements
- Linear interpolation functions for axial displacements.

This methodology results in a beam element stiffness matrix, \( K \), which relates increments of displacement to increments of load, from step \( i-1 \) to \( i \). The element stiffness matrix can be written in the standard form as a combination of bending stiffness geometry \( K_b \) and axial stiffness geometry \( K_a \) as,

\[
K = (E_c I^*) K_b + (E_c A^*) K_a
\]

\( A^* \) and \( I^* \) are called effective section properties, which, along with the neutral axis \( y^* \), are dependent on the nonlinear stress-strain model and are calculated by integration over beam cross section:

1. **Effective area**: \( A^* = \left( \frac{1}{E_c} \right) \int E^*(y) dA \)
2. **Neutral axis**: \( y^* = \left( \frac{1}{E_c} \right) \int (y - y^*) E^*(y) dA / A^* \)
3. **Moment inertia**: \( I^* = \left( \frac{1}{E_c} \right) \int (y - y^*)^2 E^*(y) dA \)

\( E^*(y) \) is the current chord modulus at position \( y \) that connects the SFRC model stress-strain point at step \( i-1 \) to the stress-strain point at step \( i \). If \( E^*(y) \) is in the linear range for all \( y \), then \( E^*(y) = E_c \), and the above integrals may be integrated exactly to produce the familiar cross section properties for linear elastic beams.

In general, however, the integrals must be calculated numerically. CANDE employs 11-point Simpson integration through the beam depth. History information is retained at each of the 11 integration points, which includes the total stress and strain value after the last converged load step along with updated damage and plasticity parameters that define the current shape of the SFRC stress-strain curve.

To advance from load step \( i-1 \) to \( i \), the values for \( A^* \), \( y^* \) and \( I^* \) are determined iteratively because they are dependent on the initially unknown values of \( E^*(y) \). The solution is iterated until successive iterations produce values \( A^*, y^* \) and \( I^* \) that converge within 0.1% error. During the iteration process the chord modulus is recomputed at each integration point as \( E^*(y) = (f_i - f_{i-1})/(\varepsilon_i - \varepsilon_{i-1}) \), where \( f_i, \varepsilon_i \) are the known stress and strain values for load step \( i-1 \), \( f_i \) is the current iterative prediction for stress at load step \( i \), and \( f_i \) is determined from the SFRC model based on the current strain estimate \( \varepsilon_i \).

The overall solution strategy is summarized in Figure 6, which concludes the presentation on the analytical development.

**EXPERIMENTAL PROGRAM**

**Full-scale test specimens**

The arch-shape for the full-scale load test had a span equal to 8.5 feet (2.59 m) a rise of 6.0 feet (1.83 m)
and length of 6.0 feet (1.83 m). Five arch units were tested. The arch thickness varied from 4.5 inches (114 mm) at the crown of the arch to 3.0 inches (76 mm) thickness at the base of the arch leg for units 2, 3, and 4, and 4.0 inches (102 mm) to 2.5 inches (64 mm), respectively, for units 1 and 5. The mix design and resulting compressive strength of each arch unit tested is shown in Table 3. Unit 1 was plain concrete, while units 2-5 were fiber reinforced with the dosages shown in Table 3. Unit 3 consisted of hooked end steel fibers with a diameter of 0.035 inches (0.90 mm) and length equal to 2.36 inches (60 mm). Units 2, 4 and 5 consisted of twisted steel fibers with a diameter of 0.02 inches (0.50 mm) and length equal to 1 inch (25 mm). Steel fibers conformed to ASTM A820. Fibers were added to the concrete directly in the ready mix truck. Because of the self consolidating mix, no external vibration was used when placing the concrete into the form. The units were poured on their sides as shown in Fig. 7, and cured under field conditions.

Test setup
A structural base slab supported the arch units and was designed to resist the jacking forces during the load test. Four units were placed on the base slab at a time. The two inner units were tested in sequence, while the two outer units were in place to retain the backfill. From preliminary CANDE analyses, the arch units experienced the greatest tensile stresses when loaded with live loads under minimal earth fill heights. Therefore, the units were covered with 1 foot (305 mm) of fill over the crown of the arch (a typical industry standard for minimum earth cover).

Backfilling the arch units was strictly monitored to assure the installation was properly constructed. An AASHTO A1 material was used and compacted to 90% standard proctor. The backfill was placed in 6 inch (153 mm) lifts on each side of the arch unit and tamped with a plate compactor. A geotechnical consultant verified that each lift was compacted to 90% standard proctor after each lift.

The arch units had a 4 inch (102 mm) diameter hole cored in the crown through which a dywidag bar was placed (see Figure 8). The dywidag bar attached to a coupler cast into the foundation extended upward through the arch unit, earth fill, loading block and through the center of a 100-ton (890 kN) hydraulic jack. A large nut was twisted down the dywidag bar until it came in contact with the top the hydraulic jack. Below the hydraulic jack was a load cell that was calibrated to measure the load during testing. A strain gage on the load cell was connected to the data acquisition center on site which provided real time read outs of the load that was recorded for later analysis. The load from the jack was transferred to the soil through a loading block created out of two layers of intersecting wide flange steel beams (see Figure 9). The beams rested on top of two 0.25 inch (6 mm) thick plywood distribution plates. The distribution plates measured 3.0 feet (915 mm) by 3.0 feet (915 mm). These plates transferred the load to the soil.

The arch unit's displacements during loading were measured by DCDT transducers placed inside the arch prior to testing. A total of eight transducers where used on each arch unit being tested. Transducers were placed symmetrically about the load and anchored at the bottom of the arch unit legs, next to the foundation. There were four transducer stations on each arch unit while being tested. Each station held two transducers in place, as shown in Figure 10. The stations were placed symmetrically about the load and at the inside bottom of the arch unit leg where it met the foundation. At each station, one transducer was stretched to the top of the inside arch and the other transducer was stretched to a spot that was on the opposite leg and halfway up the arch. The transducers connected to the top of the arch were in place to measure mid-span deflection and the others were in place to measure any possible racking. Stations were placed across from each other on opposite legs. Four stations where used on each arch unit, two on each side of the dywidag bar. This positioning of stations allowed for monitoring any inconsistencies along the length of the arch unit such as fiber distribution, uneven loading, or racking. The transducers were connected by a wire to the data acquisition center on site.

Test procedure
The arch units were loaded one at a time. The jack was slowly opened to increase the pressure applied to the distribution plate and the soil on top of the arch unit. The data acquisition center recorded the load and corresponding displacements. Team members were stationed below the unloaded arch units to record the load at the first visible crack and to observe the cracking pattern. The jack was continuously opened until the load on the arch unit started to drop off. After testing, the load block, jack and transducers
were moved to the adjacent arch unit for the next test. As a result of the low earth fill, the load being centered on the arch unit, and the lack of shear connectors between units, no load was transferred from one unit to the next during testing. The duration of each test varied from 10 minutes minimum to 20 minutes maximum.

TEST RESULTS AND ANALYSIS

The DCDT transducers measured the change in the length of the cord that was attached to it. The rise of the arch unit, half the span length and the wire attached to the DCDT formed a right triangle, which permitted triangulating and measuring the mid-span deflection during testing.

Figure 11 shows the load vs. deflection results for the five arch units tested. The deflections are reported at mid-span of the arch, the location of maximum vertical displacement. It can be seen that the fiber reinforced units (units 2-5) sustained a peak load ranging from 117 kips (520 kN) up to a maximum of 152 kips (676 kN), dependent on the percentage of steel fibers. The plain concrete arch, unit 1, only sustained a peak load of 100 kips (445 kN). The contribution of the fibers in the concrete is most evident post crack. As shown in Figure 11, the fiber reinforced units carried an average load nearly 1.5 times the load carried by the plain concrete unit for the same deflection up to 0.20 inches (5 mm) of mid-span vertical deflection.

Initial concrete cracking was observed to occur when the load reached about 60 kips (267 kN). This observation is reflected in the initial softening of the load vs. deflection curves at 60 kips (267 kN) for all units. Each test was stopped once the load was unable to be increased with increasing deflection. The units all developed a failure mechanism composed of three cracks along the length of the unit due to tensile stresses. The peak load occurred when the cracks developed into complete hinges. The three hinges were located at the inside face of the arch crown (centerline of arch) and on the outside face of the arch haunches. The two haunch cracks on the outside face were symmetrically located approximately 5.42 feet (1.65 m) from the bottom of the unit (measured vertically, perpendicular to the top of the foundation slab).

Figure 12 shows the experimental results of unit 5 compared to the analytical results obtained from modeling the structure with the CANDE program. CANDE’s finite element model includes the structure and surrounding soil along with a series of surface pressure increments to simulate the jacking load. The load vs. deflection curve from CANDE correlates well with the experimental results for initial, linear elastic loading, to nonlinear loading (post-crack behavior), and up to peak load. Peak load prediction by CANDE was within 3% of the measure peak of 152 kips (676 kN). Post-peak loading, or load-softening, was not simulated because the CANDE model was restricted to force-increment loading, not displacement increment loading.

The CANDE model prediction for initial cracking occurred when the load reached about 40 kips (178 kN). Although this load level is less than the experimentally observed first crack at a load of 60 kips (267 kN), the correlation between experiment and prediction is very reasonable in view of the fact that CANDE reported a crack width equal to 0.00096 inches (0.02 mm) at 40 kips (178 kN), which is not likely visible to the naked eye.

With regard to the failure prediction, the CANDE model predicted the same 3-crack hinging failure as observed in the experiment, at the arch crown and arch haunches. Further, CANDE predicted that the inside face of the crown (positive moment region) cracked prior to the outside face of the arch at the haunches (negative moment region). Although the sequence of the cracking could not be physically observed on the outside of the arch because of the soil backfill, the predicted sequence of events correlate well with nonlinear behavior of the load-deformation plot of unit 5 in Figure 11. That is, CANDE predicts the hinge-like cracking at the haunches to start at a load level of 110 kips (489 kN), which correlates well with the start of the slope reduction in the load-deformation plot. The CANDE model predicted the location of the controlling negative moment crack to be at 5.0 feet (1.52 m) from the arch bottom, which is only 5 inches (127 mm) from the experimentally observed location.

In summary, from elastic response to initial concrete cracking to ultimate failure, the CANDE model correlated very well with experimental observations and measurements.

CONCLUSIONS

The current research shows that the addition of steel fibers to plain concrete is effective in enhancing
the tensile ductility and loading capacity of buried concrete structures such as bridges, culverts and vaults. This paper explored a practical application of steel fiber reinforced concrete (SFRC) to buried structures, detailed the development of a SFRC analytical model used in the finite element program - CANDE, and described the experimental and analytical approach used to test the accuracy of the model.

The accuracy of the CANDE program, as measured by correlating the results from the analytical model and experimental results of the full-scale load test, suggests that the SFRC model presented is well suited for analyzing and designing SFRC structures. This type of analysis, where the ductility of the material is considered, leads to the ability to investigate structural applications that can take advantage of the additional moment capacity offered by SFRC compared to plain concrete. The methodology presented provides a means to define ductility related to moment capacity, which is the essential link for structural design and the development of new applications for SFRC.

For the application of SFRC to buried structures, the ductility offered by SFRC allows for an appropriately designed structure to take advantage of soil-structure interaction, creating an efficient and economical structure. Since it was shown that the maximum load on the structure was more than two times the required design (factored) load from AASHTO Standard Specifications for Highway Bridges, the thinner of the two arch unit sections tested was considered the most economical section to develop into a new precast product. The product was introduced to the construction market as a solution for small span culverts and stormwater detention vaults. The product is a cost effective alternative to cast-in-place box culverts, corrugated metal pipe, and plastic detention vaults. Currently, there are a number of projects throughout the United States where this product is being utilized as a precast concrete solution for these types of applications. These products have shown economies related to manufacturing, shipping, and installation due to the fact that no conventional reinforcing needs to be tied or placed, multiple units can be placed on a single truck, and only lightweight equipment is needed on site to install the arch units.

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8. ACI Committee 544, “Design Considerations for Steel Fiber Reinforced Concrete (ACI 544.4R-88),” American Concrete Institute, Farmington Hills, MI, 1999.
Table 1—Neutral axis location and internal moment values as a function of \( x \) and \( r \)

<table>
<thead>
<tr>
<th>Range of x-multiplier (( e^* = xe_r ))</th>
<th>Neutral axis location (( y^* = zh ))</th>
<th>Internal Moment (( M_t = \frac{1}{6}bh^2 ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear range ( 0 \leq x \leq 1 )</td>
<td>( z = \frac{1}{2} )</td>
<td>( M(x) = M_t x )</td>
</tr>
<tr>
<td>Softening range ( 1 \leq x \leq r )</td>
<td>( z(x) = \frac{((1+a)^{1/2} - 1)/a}{a} )</td>
<td>( M(x) = M_t[\frac{2(1-z)^2x/z + z^2(3(r-x-r^2))/r(1)}{r-1}] )</td>
</tr>
<tr>
<td>Rupture range ( r \leq x \leq \infty )</td>
<td>( z(x) = \frac{1}{1 + \frac{r^{1/2}}{x}} )</td>
<td>( M(x) = M_t[\frac{2x(1-z)^3}{z} + z^2(2-x^2)/x^2] )</td>
</tr>
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</table>

Table 2—Moment capacity factor versus increased values of \( r \) by powers of 10

<table>
<thead>
<tr>
<th>r-value</th>
<th>1</th>
<th>10</th>
<th>100</th>
<th>1000</th>
<th>10,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{max}/M_t )</td>
<td>1.0</td>
<td>1.6</td>
<td>2.2</td>
<td>2.6</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Table 3—Material properties of the precast arch units

<table>
<thead>
<tr>
<th>Mix Proportions per 1 yd(^3), SSD Condition</th>
<th>14-day</th>
<th>28-day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel 8's</td>
<td>Sand</td>
<td>Cement</td>
</tr>
<tr>
<td>(lbs.)</td>
<td>(lbs.)</td>
<td>(lbs.)</td>
</tr>
<tr>
<td>UNIT I.D.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1360</td>
<td>1570</td>
</tr>
<tr>
<td>2</td>
<td>1400</td>
<td>1540</td>
</tr>
<tr>
<td>3</td>
<td>1370</td>
<td>1590</td>
</tr>
<tr>
<td>4</td>
<td>1370</td>
<td>1550</td>
</tr>
<tr>
<td>5</td>
<td>1370</td>
<td>1570</td>
</tr>
</tbody>
</table>

*Hooked end fibers used for Unit 3, Twisted fibers used for Units 2, 4, & 5

1 yd\(^3\) = 0.76 m\(^3\); 1 lb = 0.45 kg; 1 oz = 30 cm\(^3\); 1000 psi = 7 Mpa

Fig. 1—Plain concrete stress-strain model (virgin curve).
Fig. 2—SFRC stress-strain model (virgin curve).

Fig. 3—Cross section and strain profile for development of moment capacity.

Fig. 4—Stress profiles resulting from the input strain profile in Fig. 3.
Fig. 5—Moment ratio versus applied strain for a family of r-values.

**CANDE SOLUTION STRATEGY**

Starting Viewpoint: A valid solution exists for load-step i-1. We seek a solution for load step i. (For the first load step all previous responses = 0, and elastic properties are assumed as the first estimate)

```
START

Begin load step i with new load increment to add to solution

Form trial solution by adding incremental to previous values (i-1)

For each element revise estimates of $A^*$, $y^*$ and $l^*$ based on solution

Assemble equations and solve for incremental displacements, etc

Check Convergence
Do all elements have successive iterative values of $A^*$, $y^*$ and $l^*$ within 0.1%?

Iteration Loop

No

Yes

Print Results for load step i

Last load step?

Yes

END

No

Load Step Loop
```

Fig. 6—Solution Strategy for SFRC model in CANDE program.
Fig. 7—Wood form for precast SFRC arch unit.

Fig. 8—End elevation of full-scale load test setup.
Fig. 9—Load jack and loading block set above top of arch with 1 ft. (305 mm) earth cover.

Fig. 10—Example of DCDT mounted to base of arch unit leg to measure vertical and horizontal displacements.
Fig. 11—Applied load versus vertical displacement results at midspan of arch unit for units 1 through 5. (Note: 1000 lbf = 1 kip = 4.44 kN; 1 in. = 25.4 mm.)

Fig. 12—CANDE model results compared to experimental results for unit 5. (Note: 1000 lbf = 1 kip = 4.44 kN; 1 in. = 25.4 mm.)
Fig. 13—Application of SFRC arch unit for stormwater detention facility.

Fig. 14—Application of SFRC arch unit for stream crossing.
Application of Engineered Cementitious Composites (ECC) in Prefabricated Modular Housing

by G. Fischer

Synopsis: Prefabricated modular housing structures have become a promising alternative to site-built structures with equal or higher engineering quality and durability. Major advantages of this technology are fast and cost-efficient construction, reduced labor requirements, and superior quality control due to factory production. The modular design of these prefabricated structures offers the opportunity for research and application of innovative composite materials and structural concepts. Engineered Cementitious Composites (ECC) in combination with light gauge steel joists and steel trusses have been utilized in the development of modular floor panels for prefabrication of housing structures. The need for steel reinforcement in conventional reinforced concrete thin shell construction creates a technological difficulty as the placement of the reinforcing steel is very labor intensive and proper positioning of the reinforcing mats is difficult. The use of fiber reinforced Engineered Cementitious Composites (ECC) helped eliminate these constraints and lead to more efficient prefabricated structural elements.

Keywords: ECC; floor panel; light gauge steel; modular housing; prefabrication.
Gregor Fischer is an Associate Professor at the Technical University of Denmark. He received his MSE and PhD in civil engineering at the University of Michigan, Ann Arbor, MI. His research interests include the mechanics and design of fiber-reinforced cement composites and their application in prefabricated composite structures. He is a member of ACI Committee 544, Fiber Reinforced Concrete, and Secretary of RILEM Technical Committee on High Performance Fiber Reinforced Cementitious Composites (TC HFC).

**INTRODUCTION**

The use of short, randomly oriented fibers as an additive in conventional fiber reinforced concrete (FRC) aims at improving the post-cracking behavior of concrete similar to the effect of traditional steel reinforcement in concrete. To achieve a structurally sufficient reinforcing effect, a relatively large amount of steel or glass fiber reinforcement is typically used particularly in thin-walled structural elements, which are difficult to reinforce by traditional means due to their thin cross-sectional dimension. The addition of fiber reinforcement in thin-walled concrete elements is a technological advantage, however, due to the required amount of fibers also results in a relatively high cost of the fiber reinforced concrete composite.

Recent advances in fiber technology and the introduction of Engineered Cementitious Composites (ECC) [1] have provided the prerequisites to achieve a high performance and economically viable alternative to traditional fiber reinforced concrete materials, namely a Polyvinyl Alcohol fiber reinforced Engineered Cementitious Composite (PVA-ECC). The unique feature of PVA-ECC is its ability to increase the tensile loading capacity in the cracked stage, showing a so-called tensile strain hardening behavior, leading to a ductile deformation response similar to that of metals and steel reinforced concrete. This strain-hardening process under tensile loading is accompanied by the formation of multiple cracking, a feature that is unique to high performance fiber reinforced cement composites such as ECC (Figure 1). As a result of these material properties ECC has the potential to be used economically in particular structural applications, such as thin-walled precast concrete elements without additional steel reinforcement. This paper outlines the technological aspects of PVA-ECC as well as introduces applications of this material in precast structural elements, such as prefabricated floor and roof panels.

The development of innovative structural elements is typically guided by the following objectives: 1) To increase efficiency, i.e. construction time and ultimately reducing cost. 2) To decrease the self weight of the element thus increasing the loading capacity to weight ratio. 3) To improve durability. In this paper, the utilization of ECC in a prefabricated composite panel consisting of a thin-walled ECC slab without conventional steel reinforcement and light gage steel joists or steel trusses is presented. The composite action of the integrally cast ECC/steel joist floor elements was investigated in a preliminary study to better understand their mechanical behavior and composite interaction and to develop a design concept based on these findings. In a subsequent study, the ECC deck slab is allowed to shrink independently without causing restrained shrinkage cracking and undesirable deflections of the panel by casting individual anchors into the ECC deck slab that can subsequently be used to attach an appropriate substructure. This modular system also provides a higher flexibility in terms of the floor panel assembly, transportation, weight optimization, and re-configurability. The flexibility of the system is most evident in the design requirements for the substructure, in which built-height, weight, and structural and architectural requirements can be met without compromising structural functionality. Furthermore, the possibility of having wiring, ventilation, piping, and thermal insulation situated within the structural height of the deck element allows the overall height of the floor panel to be further reduced.

Because of the low weight of the modular ECC deck compared to traditional systems as a result of the required thickness of conventional reinforced concrete decks, the composite stiffness of the ECC deck and the resulting deflections and dynamic properties under service conditions have been of particular interest in this project. The main focus of this research study, apart from structural detailing, is to examine and analyze results from tests and to compare them to analytical and numerical models.

**PVA-ECC MIXTURE PROPORTIONS AND MECHANICAL PROPERTIES**

The constituent materials and mix proportions of ECC are selected to satisfy the conditions for achieving the strain-hardening and multiple cracking behavior, which are closely related to the material properties of the fiber reinforcement, the cementitious matrix, and their interfacial bond characteristics. The most significant parameters are the tensile strength and volume fraction of the fibers, the tensile
strength and fracture toughness of the cementitious matrix, and the interfacial bond strength between fiber and matrix [2]. For a particular version of PVA-ECC, the resulting composition is a fine-grained cementitious matrix with a maximum particle size of sand of about 0.20 mm (0.008 inches), a relatively large amount of pozzolanic filler, and a PVA fiber volume fraction of 2% (Figure 2).

The most relevant material properties for structural use of this particular composition are summarized in Table 1.

**PRECAST AND MODULAR COMPOSITE FLOOR PANELS**

Due to its ductile deformation behavior, ECC has been successfully applied in seismic resistant structures at locations with high deformation demands, such as plastic hinge regions, coupling beams, and shear walls [3]. Due to its intrinsic reinforcing effect, ECC can also be utilized in thin-walled structural members and prefabricated panelized or modular structures, which have become a promising alternative to site-built structures with equal or higher engineering quality and durability. Major advantages of this technology are fast and cost-efficient construction, reduced labor requirements, and superior quality control due to factory production. The modular design of these prefabricated structures offers the opportunity for research and application of innovative composite materials and structural concepts.

More specifically, Engineered Cementitious Composites (ECC) in combination with light gage steel joists have been utilized in the development of modular wall and floor panels for prefabrication of residential structures. The need for steel reinforcement in conventional reinforced concrete thin shell construction creates a technological difficulty as the placement of the reinforcing steel is very labor intensive and proper positioning of the reinforcing mesh is difficult. The use of PVA-ECC helped eliminate these constraints and lead to more economical and structurally reliable floor and wall elements.

The PVA-ECC composite floor panel and forming system is a novel precast, modular floor panel system consisting of an ECC slab and steel joists integrally cast with the slab. The ECC slab and the steel joists develop composite action in the direction of the larger span of the panel. In between the joists, the ECC slab acts as a continuous one-way slab (Figure 3).

The unique features of this panel system are: 1) Light weight due to reduced slab thickness and use of light gage steel joists, 2) Precast construction method of the system, 3) Reduced cost, 4) Applicability in steel and concrete frame systems, masonry and wood construction, 5) Fiber reinforcement eliminates the need for steel reinforcing bars or welded wire mesh, 6) Effective crack control due to fiber reinforcement, and 7) Possibility of spanning relatively long distances (> 10 m or 32.81 feet).

Dimensions of the composite floor panel range from 4 to 16 m (13.12 to 52.49 feet) long and 1.20 m (3.94 feet) wide with the width being determined primarily by handling requirements. Panel widths can be larger than 1.20 m (3.94 feet) up to the width of the entire floor unit. The slab thickness for a typical floor panel (superimposed load ≤ 5 kN/m² or 0.73 psi) is 50 mm (1.97 inches) and the light gage steel joist spacing typically does not exceed one half of the panel width, such that at least two joist are contained in each panel. The panels interlock along their adjacent edges through shear keys, which can be strengthened with an adhesive if floor diaphragm action is required. The light gage steel joists are provided with punch-outs at the section embedded in the ECC slab to ensure shear transfer and composite action. The dimensions and material properties of the steel joists are determined by the panel span, the loading conditions, and the panel deflection requirements. An example of an integrally cast floor panel in the fabrication phase and in the finished state is shown in Figure 4.

The concept of the modular composite floor panel technology investigated in a subsequent study [4] is to separate the casting of the ECC slab and the attachment of the steel truss joist by embedding cast-in anchors into the ECC slab (Figure 7), thus avoiding deformations and cracking due to shrinkage typically encountered with integrally cast floor panels. Moreover, the cast-in anchor system offers increased flexibility on the steel truss structure design for different loading capacities and deflection limits as well as architectural requirements. Furthermore, the modular panel assembly concept allows inducing a pre-camber in the ECC slab of the modular floor panel to compensate deformations due to self weight and creep during the use phase of the panel.

**EXPERIMENTAL PROGRAM**

Two identical modular ECC floor slabs were manufactured with cast-in anchors positioned on the bottom of the slabs and subsequently used to connect a steel truss assembly to the underside of the
The floor slab resulting in a complete deck element.

The overall dimensions of the ECC deck slab are 1.20 m (3.94 feet) in width and 8.30 m (27.23 feet) in length (Figure 5) with an ECC slab thickness of 50 mm (1.97 inches) and an overall built height of 325 mm (12.79 inches). The number of cast-in anchors (Figure 6) in each row was determined based on the expected ultimate load, the span of the deck and the height of the truss structure. Subsequently the length of each cast-in anchor was determined based on the available space between anchors points and minimum length needed to attach the truss segment (Figure 6).

Due to the small depth of the ECC slab, no suitable commercial cast-in anchors were available and the anchors had to be custom fabricated for the specimens used in this study. To make the cast-in anchors, a system of interlocking steel channels and matching bolts were used to secure the connection. To fasten the supports at the end segment, two channels were used for each anchor (double cast-in).

The dimensions of the deck panel were selected to have a structural height of \( h = 325 \text{ mm (12.79 in.)} \) including the ECC slab thickness of 50 mm (1.97 inches) and the steel truss height of 275 mm (10.83 in.) to have a common basis for comparison to the standard hollow core deck specimen.

The first truss setup (S1), was composed of a 60x60x7 mm (2.36x2.36x0.28 inches) T-profile as the tension member and 40x20x4 mm (1.57x0.79x0.16 inches) L-profiles as the diagonals. After testing of specimen S1, 18 diagonals for each truss (9 on each end) were replaced with larger 50x50x6 mm (1.97x1.97x0.24 inches) in the steel truss of specimen S2. After testing of specimen S2, the tension member was replaced with a larger 80x80x9 mm (3.15x3.15x0.35 inches) T-profile resulting in the steel truss for specimen S3. The steel trusses for specimens S1, S2 and S3 were bolted together using M12 steel bolts (Figure 8).

Subsequent to testing of specimens S1, S2 and S3, a fourth truss setup in specimen S4 was welded together using an 80x20 mm (3.15x0.79 in.) plate profile as the tension member, 40x8 mm (1.57x0.32 in.) plate profiles as tension diagonals and 40x40x4 mm (1.57x1.57x0.16 inches) RHS profiles as compression diagonals. Furthermore the steel grade in truss setup S4 was S350.

To ensure that the shear forces in the truss structure at the end-supports did not transfer directly into the ECC slab, the truss support footings were made from 200 mm (7.87 inches) long HE160B steel profiles (two for each truss). These footing pieces were bolted to the double anchors positioned at the ends of the deck elements.

Support and Loading Configuration

The deck element was simply supported at both ends with a span of 8 meters (26.25 feet). At each end, the deck element was supported at two points, 0.60 m (1.97 feet) apart. Loading at the serviceability limit state (SLS) was introduced to the specimens by applying a uniformly distributed line load of 40 kN (8.99 kip) evenly over the 8 m (26.25 feet) long span resulting in an area load of 4.09 kN/m^2 (0.59 psi). The distributed loading was applied in four steps with increments of 10 kN (2.25 kip), or 1.02 kN/m^2 (0.15 psi) at each step. The loads were situated in a line-load fashion directly above the trusses as the purpose of these tests was to examine the behavior of the deck element in its longitudinal direction. Specimens S1, S2, S3 and S4 were all subject to testing at SLS.

To evaluate the panel behavior at ultimate limit state (ULS), a four point bending test setup was used, consisting of two point load couples applied on the deck at 2 m (6.56 feet) (L/4) and 6.00 m (19.69 feet) (3L/4) relative to the 8.00 m (26.25 feet) span to induce a bending moment.

To evaluate the behavior of the modular composite deck elements, a number of tests were carried out for each of the specimens with different steel truss setups. Due to slip encountered in the bolted connections during testing of specimens S1, S2, and S3, the deflections during SLS loading were determined in the unloading process, thus minimizing deflections induced due to slip. For each load interval (10 kN or 2.25 kip), specimen S1 deflected 5 mm (0.20 inches), specimen S2 deflected 4.5 mm (0.18 inches), specimen S3 deflected 3 mm (0.12 inches) and specimen S4 deflected 2.5 mm (0.20 inches).

Deflections and loading of specimens S1, S2, S3 as well as an integrally cast panel for comparison were monitored during the ULS testing procedure along with the strains in the diagonals and tension members of the steel trusses. The load-deflection behavior of these specimens for the ULS loading procedure is shown in Figure 10. In comparison, the load-deflection behavior of two identical specimens of a commercially available hollow core deck with a structural height of 285 mm (11.22 inches) is also shown in Figure 10.
Testing of specimen S1 was terminated when buckling of compression diagonals occurred near the supports of the deck element due to shear. Buckling of the diagonals was accompanied by twisting of the tension member of the truss. This twisting was due to the off-centered positioning of the diagonals to the longitudinal centerline of each truss, a detail that was modified in subsequent specimens.

Testing of specimen S2 was terminated after deflections exceeded 500 mm (19.69 inches) (Figure 11). After yielding of the tension member, the ECC deck showed limited cracking, mainly forming at the quarter points, while multiple cracking in the ECC was observed on the underside of the slab at mid span.

After testing of specimen S2, the deck slab was reused and the tension member of the truss was replaced with a larger profile. Testing of specimen S3 resulted in an abrupt failure of the deck element due to shear failure in the bolts connecting the diagonals of the truss. Slip in the bolted connections of specimen S3 becomes apparent in Figure as small sudden drops in the load during testing.

Testing of the natural frequency and damping was carried out during SLS loading by initiating a vibration in the deck element and measuring the response of the element with two accelerometers positioned at the mid-span. The natural frequencies of specimens S3 and S4 are higher than for S1 and S2 as a result of changing the tension member and consequently amplifying the effective stiffness of specimens S3 and S4 (Figure 12). By alternating the position and the cross section area of the tension member, a desired natural frequency can easily be achieved to meet a wide range of design requirements on the static and dynamic performance of the modular ECC floor panels.

Because of large deflections observed in specimen S1 due to slip in the bolted connections, the testing of damping ratio for specimens S2 and S3 showed limited repeatability and were disregarded. While specimen S1 was tested at 0 and 4 kN/m² (0 to 0.58 psi), specimen S4 was tested at all load intervals. The damping ratio for specimen S1 was measured in the range of 0.6-1.8% of critical and for specimen S4 in the range of 0.6-1.0% of critical.

**DISCUSSION**

The design criteria for the modular composite panel included the loading capacity, a deflection limit, a minimum eigen-frequency (natural frequency), and a ductile failure mode by yielding of the tension member of the steel truss. The modular construction concept offered the possibility to assemble the panel after drying shrinkage deformations in the ECC deck had occurred, which resulted in a significant reduction in the required pre-camber of the panel prior to installation and testing.

**Serviceability limit state**

During SLS loading the measured deflections in the specimens were all below the required limit of L/400. While the numerically predicted deflections are consistently lower than the analytical results, they are both in good agreement with the experimental results (Table 2).

The natural frequencies were measured in the range from 7.1 Hz (specimen S1) to 10.2 Hz (specimen S3) for the panels self weight and 3.4 Hz (specimen S1) to 5.0 Hz (specimen S4) for 4.0 kN/m² (0.58 psi) loading (Table 2). Due to the relatively low weight of the modular deck, any applied weight to the deck decreases its natural frequency more than for heavier, conventional deck panels. Specimen S4 is within the recommended natural frequency limit of 8.0 Hz for the un-loaded deck and 5.0 Hz for the loaded deck (Table 2).

During SLS loading at 4 kN/m² (0.58 psi), the stresses in the composite panel reach 55% of the tension members yielding capacity while reaching 10% of the ECC compression strength. At the ultimate limit state, to insure a ductile failure mode, the tensile yield capacity utilization of the steel truss substructure must be higher than the compressive loading capacity of the ECC slab, which has been shown in specimen S2.

**Ultimate limit state**

Testing of specimen S1 was terminated due to buckling in the compression diagonals. When testing of specimen S1 was terminated, it was observed that cracks had started to propagate from the corners of the cast-ins, directly below the quarter-point loading as a result of localized stresses. This is consistent with results from the numerical stress analysis in the ECC slab [4]. For specimen S2, the loading reaches 110 kN total (24.73 kip) (equal to a 11 kN/m² or 1.60 psi loading) before the tension bar yields and ultimately reaches 140 kN (31.47 kip) total before the testing was
terminated, at which point the panel had exhibited a ductile failure mode (see Figure 10 and Figure 11). The equivalent ultimate moment capacity is $M_u = 110 \text{ kN-m (81.1 kip-feet)}$. At reaching this ultimate loading capacity, localized cracking in the vicinity of the cast-in anchors had increased and flexural cracking had developed at mid-span of the deck. For specimen S2 the testing was terminated before reaching the ultimate loading capacity to be able to reuse the ECC slab for additional testing (Specimen S3).

For specimen S3, the total applied load at reached 126 kN (28.33 kip) (equivalent to a 13 kN/m² or 1.89 psi loading) before the bolts in the tension diagonals at the end of the deck failed and the deck element consequently failed suddenly. Disregarding the insufficient capacity of the bolted connections, the specimen was designed to withstand a load of 195 kN (43.84 kip) before yielding of the tension member, 85 kN (19.11 kip) more than its predecessor (S2) due to the larger tension member profile. Specimen S4 was not loaded to the ultimate condition though it was designed and modeled to withstand a load of 260 kN (58.45 kip) before yielding of the tension member.

**ECONOMIC CONSIDERATIONS**

The overall objective in the development of the ECC floor panel system described in this paper was to achieve an economically feasible and lightweight alternative to current floor systems such as wood sheathing on steel joists, reinforced concrete on pan deck, or hollow core concrete decks. Reinforced concrete on steel pan deck consists of steel joists supporting a steel pan serving as formwork and reinforcement for a cast in place concrete deck, which itself is reinforced with welded wire mesh. This deck system and similarly the hollow core deck system have relatively small deflections under service loads due to the high stiffness of the support structure and the reinforced concrete deck and the amount of prestressing steel. However, these systems also have a relatively high self-weight and can be labor intensive during the construction process especially for the pan deck system.

Wood sheathing on a steel joist support structure is substantially lighter than concrete on pan deck, however, it has relatively low stiffness and exhibits large deflections as the wood sheathing and the supporting joists do not act as a composite member. In the design of this deck system, the dead and live loads are exclusively assigned to the steel joists, which are spaced 305 mm to 610 mm (12.01 to 24.02 inches) on center depending on the expected loading and span length.

The prefabricated ECC floor panel system described in this paper is similar in stiffness to the concrete on pan deck system at less than half the self-weight. Compared to hollow core concrete panels, substantial weight savings of up to 75% can be achieved. While the weight of the wood sheathing system is substantially lower, the deflections of the wood sheathing system under service loads are substantially higher compared to the ECC floor panel system. These features indirectly impact the economical feasibility of the floor panel system with respect to the design and layout of the frame system supporting the floor.

The material cost of the floor systems described above can be compared directly based on information provided by a manufacturer of prefabricated modular school buildings currently using both traditional floor systems as well as the ECC floor panel system. While the unit cost of ECC is higher than that of concrete, the material cost of the ECC floor panel system is approximately 30% less than that of the concrete pan deck. In addition to reduced material cost, additional savings result from reduced labor, construction time (in factory and on-site), and shipping cost as well as from the use of recycled materials, modularity, and possible reuse of the panels.

**CONCLUSIONS**

The prefabricated floor panel system described in this paper, consisting of a PVA-ECC slab and light gage steel joists, is a promising alternative to traditional floor systems, such as reinforced concrete on pan deck, hollow core slabs and wood sheathing on steel joists. The load deformation behavior of the PVA-ECC floor panel shows favorable characteristics with relatively high flexural stiffness and strength and ductile ultimate failure. The primary advantages of this floor panel system are its high stiffness/strength to weight ratio and low cost. Besides these structural and economic benefits, the use of recycled waste materials, such as fly ash or slag as filler in the PVA-ECC floor slab and the relatively small amount of light gage steel make this system an environmentally friendly product without compromising its primary functions and performance features.

The investigation described in this paper focuses on a modular composite panel concept, consisting of an Engineered Cementitious Composite (ECC) deck with integrated anchors and a subsequently
assembled and attached steel truss substructure. The modular design of these panels offers the flexibility of adapting to a multitude of different performance requirements by selection of a specific combination of ECC deck and steel truss substructure, thereby altering the strength and stiffness properties of the panel. The loading capacity of these panels can be equivalent to commercially available panels, such as hollow core decks while having a significantly reduced self weight up to three times lighter than conventional hollow core panels.

The dynamic properties of the modular composite panels can meet the typical structural performance requirements, however, additional research is needed to further improve the dynamic behavior towards higher natural frequencies and damping. A detailed study of the long-term behavior of the modular composite panels influenced by creep of the ECC slab and repeated loading under service conditions is required to further develop the modular concept and pursue large-scale commercialization.

**REFERENCES**


**Table 1—Material properties of PVA-ECC**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Flexural strength (ASTM C 1018)</td>
<td>16 MPa (2321 psi)</td>
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<tr>
<td>Tensile strength</td>
<td>6 MPa (870 psi)</td>
</tr>
<tr>
<td>Tensile strain capacity</td>
<td>4%</td>
</tr>
<tr>
<td>Compressive strength (ASTM C 39)</td>
<td>60 MPa (8702 psi)</td>
</tr>
<tr>
<td>Crack opening (at tensile strength)</td>
<td>0.200 mm (0.007874 inches)</td>
</tr>
<tr>
<td>Shrinkage (ASTM C 157)</td>
<td>0.04%</td>
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<tr>
<td>Density</td>
<td>2000 kg/m³ (125 pcf)</td>
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</table>

**Table 2—Comparison of deflections, natural frequencies, and stresses from analytical results (Analy.), numerical results (Numer.), and actual measurements (Actual) during SLS testing*  

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<td>S1</td>
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<td>S2</td>
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<td>S3</td>
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<td>S4</td>
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<table>
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<th>Deflection [mm]</th>
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<th>Specimen S2</th>
<th>Specimen S3</th>
<th>Specimen S4</th>
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<td>At 4.0 kN/m²</td>
<td>17.9</td>
<td>15.7</td>
<td>19.4</td>
<td>17.9</td>
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<td>8.20</td>
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<td>At 4.0 kN/m²</td>
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<td>—</td>
<td>3.80</td>
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<th>Specimen S3</th>
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<td>3.8</td>
<td>—</td>
<td>4.7</td>
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<tr>
<td>σ(_{y, mid-span})</td>
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<td>114.2</td>
<td>143.7</td>
<td>150.8</td>
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* Stresses shown for 4.0 kN/m² (0.58 psi) loading.

Notes: 1 kN/m² = 0.14504 psi; 1 MPa = 145.04 psi; 25.4 mm = 1 inch.
Fig. 1—Tensile stress-strain behavior of cementitious matrixes.

Fig. 2—Constituents per m³ of PVA-ECC. (Note: 1 kg = 2.204 lbs.)
Fig. 3—Schematic of integrally precast ECC floor panel.

Fig. 4—Integrally cast floor panel during fabrication and prior to testing.

Fig. 5—ECC deck element layout and positioning of cast-in anchors, bottom side shown (all dimensions shown in the figure are in millimeter units; 25.4 mm = 1 in.).
Fig. 6—Left: Cast-in anchor. Right: Resultant forces expected in cast-in anchor, forces shown in vertical and longitudinal direction.

Fig. 7—Longitudinal cross section of truss structure connected to ECC deck panel.

Fig. 8—Steel truss assembly and bolted connection to ECC slab.

Fig. 9—Load setup for SLS loading (left) and ULS loading (right).
Fig. 10—Load-deflection behavior of tested specimens and integrally cast ECC panel. (Note: 1 kN = 0.2248 kip.)

Fig. 11—Deformed shape of Specimen S2 during (left) and after (right) ULS loading.

Fig. 12—Measured natural frequency as a function of applied service load. (Note: 1 kN/m² = 0.14504 psi.)
Fischer
Case Studies of Glass Fiber-Reinforced Concrete (GFRC) Applications

by J. Jones

Synopsis: Glass fiber-reinforced Concrete (GFRC) has now been in use worldwide for over 30 years. There are many varied applications for this composite material and this paper describes four specific applications that illustrate the benefits that GFRC offers to the construction industry.

Keywords: AR glass fiber; fiber-reinforced concrete (FRC); GFRC; premix; spray-up.
Jones

John Jones is the Manager for the AR Glass Fiber Division in Nippon Electric Glass America Inc., a leading manufacturer of AR glass fibers. He was a member of the original market development group in Pilkington Brothers Ltd. in the UK that developed alkali resistant (AR) glass fibers and the related glass fiber-reinforced concrete (GFRC or GRC) technology in the early 1970s. Since then he has been involved in developing and manufacturing of a wide variety of GFRC products.

INTRODUCTION

In the initial research work that investigated the performance of glass fibers in Ordinary Portland Cement (OPC) based materials the researchers used the only available glass fiber at the time, E glass fiber. This glass fiber was widely used to reinforce plastics (FRP). The researchers found that they could get good 28 day flexural strengths, around 3,000 psi (20 MPa), but that this strength was quickly lost with aging. They identified the problem as alkali attack on the E glass fibers by the alcalis in the cement (ref. 3).

Researchers at the Building Research Establishment in UK developed an AR glass fiber that overcame the alkali attack problem. They found that adding increasing amounts of zirconium dioxide (zirconia) to the glass made the glass increasingly resistant to alkali attack up to 16 wt%. Additional zirconia did not increase the alkali resistance (ref. 3). On the basis of this research work AR glass fiber and its companion composite Glass Fiber Reinforced Concrete (GFRC, or GRC as it is called elsewhere in the world) were introduced in 1970. They have both seen wide scale use in many types of application. Four different commercial applications will be discussed in this paper.

All the GFRC applications cited in this report used alkali resistant glass fiber conforming to ASTM C1666/C 1666M, Standard Specification for Alkali Resistant Glass Fiber for GFRC and Fiber-Reinforced Concrete and Cement.

The GFRC design procedure described in the section, Architectural Panels, is also the procedure used in the design of various types of GFRC products manufactured in the USA.

ARCHITECTURAL PANELS

One of the earliest applications of GFRC was as architectural panels. The earliest projects date back to 1972 both in the USA and Europe. Since this time GFRC has been a major and continuing use as a material for cladding buildings of all types. Fig.1 shows San Francisco Towers building in San Francisco that is clad completely in GFRC architectural panels.

The reason why GFRC has endured as a cladding material is that at the very outset the primary developers established design and manufacturing standards that were well conceived and have been proven by the test of time. Cladding panels manufactured and installed 25-30 years ago are still in place with no signs of distress, some having been through earthquakes, hurricanes, and typhoons.

The design philosophy that has proved so successful has a basic principle – “Don’t let the GFRC crack”. This may be an alien concept to concrete designers but it is essential for successful GFRC design, and in fact for any other composite materials, such as glass fiber reinforced plastics (FRP), carbon fiber reinforced plastics (CFRP), and fiber reinforced cement (FRC – the replacement for the old asbestos cement). It was probably fortunate that the primary early developers of GFRC were glass fiber manufacturers, who brought their FRP design expertise to GFRC.

The design procedure that is used in the USA was developed by the PCI-GFRC committee¹. It has now been written into the International Building Code (IBC). It starts from determining the stress/strain curve for the fresh GFRC composite at around 28 days. A typical curve is shown in Fig.2.

The basic concept is to factor either the Yield Strength or the Ultimate Flexural Strength to establish the design strength that will prevent static plus service loads on the GFRC product from causing stresses that will exceed the Yield Strength. In other words “Don’t let the GFRC crack”, because the Yield Strength is the point at which micro-cracking starts in the composite. Ref 1 gives details about how to calculate the “Design Flexural Strength”, $f_u$, but basically it is determined from the following formula:

$$f_u = \phi S f_u'$$

where
Ø = 0.75, strength reduction factor

S = Shape factor

\( f_u' = \) Assumed aged ultimate flexural strength

and

\( f_u' = \) the lesser of:

\[ f_{uy}' = f_{uy} (1 + t V_y) \]

\[ f_{uu}' = 1/3 f_{uu} (1 + t V_y) \]

\( f_{max}' = 1200 \) (8.3 MPa)

An example of the calculation, taken from appendix B in Ref. 1, is shown in Fig. 3. Service loads, determined from the governing building codes, and any additional load considerations are also factored. Again, Ref. 1 provides examples and recommended factors.

Much has been said over the years about the durability of GFRC, particularly the perceived fall in the ultimate strength with time. The design procedure that has been established makes this discussion irrelevant because the Yield Strength is the important reference point and this value does not change with time. Because the working stresses are always kept below the Yield Strength, it does not matter what happens to the ultimate strength because it will never fall below the yield strength. Even so, recent data derived from tests on real time aged material2 has shown that the earlier projected aged strengths of GFRC were conservative when compared to actual aged GFRC. In particular Ref 2, which describes testing that was done on samples cut from cladding panels on a building in London, UK, that was erected around 1975, validates the design concept outlined above.

In the USA, GFRC architectural panels are now usually designed as a system, comprising a GFRC skin supported by an attached steel stud frame. The attachment mechanism of the frame is very important in that it must allow differential movement between the skin and the frame. This is achieved by a device known as a flex anchor, Fig. 4, and illustrated in Fig. 5. The anchor is usually made from 1/4" or 3/8" (6 or 9 mm) rod. It is welded only at the top 1" (25 mm) and is angled slightly away from the stud. This arrangement provides enough flex in the anchor that the GFRC skin is not constrained when it shrinks or expands differentially to the steel stud frame. The flex anchor is attached to the GFRC skin by means of a GFRC bonding pad, Figs. 4 and 5. This is applied by hand over the foot of the anchor. Care is taken to make sure that the bonding pad does not cover the heel of the anchor. This ensures that the foot of the anchor can de-bond from the bonding pad, thereby allowing another degree of freedom of movement between the frame and the GFRC skin. As a final safety factor the anchors are arranged so that they all face towards the center of the panel, so that any GFRC shrinkage is not constrained.

Fig. 6 shows typical cross-section details of connections between the GFRC skin and the steel stud frame. Fig. 7 shows the actual panel. Fig. 8 shows an example of how the steel stud frame is connected to the building structure. Figs. 9-12 show examples of GFRC cladding panel projects.

**ENERGY EFFICIENT HOUSE**

Although GFRC in itself does not provide good thermal insulation, it can be used with other materials that do have good thermal insulation to produce insulating wall panels. This feature has been taken advantage of in the design of the walls of an energy efficient house (Fig. 13). The house is referred to as the Power House.

The walls of the house comprise 6"-12" (150 – 300 mm) thick polystyrene slabs completely encased in a skin of 3/8" (9 mm) thick GFRC. Not only is the GFRC a lightweight, thin, weather resistant outer surface but it also provides fire protection for the otherwise combustible polystyrene. The walls are designed to meet all code requirements. Fig. 14 shows a typical panel being installed.

One of the problems with sandwich panel systems such as this is that the joints between the panels present a cold joint because there is no insulation material in the joint area. This has been solved by designing a stepped joint as shown in Fig. 15. The manufacturing process that is used to produce the GFRC skins makes this relatively easy to do. The manufacturing process involves first spraying down a 3/8" (9 mm) GFRC skin onto a mold. The pre-cut polystyrene slabs are then pressed onto the GFRC. Total encapsulation of the polystyrene is then completed with a final sprayed GFRC 3/8" (9 mm) skin.

The wall panels are assembled by attaching to a wooden frame that forms the structural frame for the house (Fig. 14). Fig. 16 shows the house before a decorative coating and roofing tiles are applied to the GFRC to provide the finish for the outside of the walls and roof.
The resulting walls have an R-value of 40 and the roof panels have an R-value of 60. The first of these houses was built in Newton, Iowa in 2004 as part of a project to investigate improved energy efficient designs for houses that was sponsored by Alliant Energy and several participating building products manufacturers. The design also incorporates other energy efficient features such as, triple glazed windows, insulated outer doors, state of the art appliances, and hot water under-floor heating. The house has complete monitoring of energy consumption.

Infra-red thermal images of the Power House compared to a conventional wood-frame house are shown in Fig.17. The much darker image of the Power House shows how much more thermally efficient its exterior walls and roof are. Data collected so far for one complete year cycle has shown that very stable levels of temperature and humidity have been maintained with very small energy consumption.

SEWER LININGS

Many cities, particularly in Europe, have old brick sewers that are in need of repair. A cost effective method of re-lining old sewers has been in use in UK for about 30 years. The system uses GFRC panels molded to the shape of the sewer, which is usually oval (Fig.18).

GFRC has proved to be a very useful material for this purpose for several reasons.

(a) GFRC has sufficient flexural strength that panels can be designed thin, usually 3/8” to ½” in (9-12 mm) thickness. This means that there is only a small loss in the internal dimensions of the sewer and so the capacity of the sewer is only fractionally reduced. The linings are typically designed in three sections which makes them easy to man-handle; an important feature in the confined space of a sewer where mechanical lifting equipment is not practical.

(b) Corrosion considerations can vary and although Portland cement is often suitable, high alumina or sulfate resistant cements can be used where more aggressive conditions exist. AR glass fiber is suitable for all these cements and the alkali and acid resistance of AR glass makes it resistant to chemical attack by the type of corrosive chemicals found in sewers.

Because the system protects the old brick sewer from further deterioration, it is important that it does not allow the corrosive chemicals in the sewage to come into contact with the old sewer. Therefore it is essential that the panels do not crack. The design procedure that has been developed for GFRC, described above, assures this.

The manufacture of the panels takes advantage of another characteristic of GFRC in which flat uncured sheets of GFRC can be molded into curved shapes. This allows for easier mechanization of production than if curved panels had to be made direct. The manufacturing involves the manufacture of a flat sheet on a semi-automated reciprocating spray machine (Fig.19). The sheet is lifted off the flat bed of the machine by a flexible vacuum de-watering lifting pad. The uncured sheet is immediately placed on a curved mold where it is formed to the required shape. The overlapping joint is also formed at the same time. The benefit of vacuum de-watering is that it facilitates the forming process of the “green” uncured sheet and it also provides for compaction of the composite. This increases the density of the GFRC and so reduces porosity, permeability, and increases flexural strength.

Installation of the system is achieved by bolting three panels together leaving a gap between the old brick sewer and the panels (Fig.18). When a run of sections has been installed grout is pumped behind the panels to fill the cavity and seal the joints between the individual panels, thereby protecting the old brick structure. Many miles of old decayed brick sewers have been renovated with this system since it was introduced in the early 1970’s. There have been no instances of failure or remedial work being needed even on the oldest installations.

PERMANENT FORMWORK

There are many examples of GFRC permanent formwork, from road bridge decks, special waffle floors, to decorative rock work. A particularly interesting system has been developed by a hotel chain for the construction of some of their Caribbean hotels. The one that is described here is a hotel in San Juan, Puerto Rico.

In the system, both the beams and columns use permanent forms. The structural beams were formed by first manufacturing “U” section pieces as shown in Fig.20. These pieces were made in folding steel molds (Fig.21). Fig.22 shows several open flat molds laid out in the factory and Fig.23 shows an operator spraying up the GFRC in the open molds. The GFRC is ½” (12 mm) in thickness. The molds have two
hinged sides which are moved to the vertical position after the GFRC has been sprayed-up. The vertical
to the horizontal (Fig.24). The GFRC unit can then be easily de-molded and stored until fully cured
sideways are supported until the GFRC hardens, about 24 hours. After this time the vertical sides are dropped
(about 28 days). Fig.25 shows the finished permanent beam form. Note the cut-out in the base of the
the form which is to allow the steel reinforcing of the column to be tied into the reinforcing for the beam.
form.
The permanent column forms are made in half sections in plastic molds (Fig.26). After spraying the
The GFRC top, a ½” (12 mm) thickness two half molds are bolted together in such a way that the two uncured
The hollow columns are shipped to the job site and erected around the steel reinforcing bars for the
GFRC half sections will bond together. Some finish work is required on the joint to make sure that it leak proof.
structural concrete (Fig. 27). The beam forms are then installed on the column forms (Fig.28). After the
The beam reinforcing has been laid in the form the column and beam forms are filled with concrete to form
beam reinforcing was tied to the reinforcing for the column. Fig.29 shows the finished permanent beam form. Note the cut-out in the base of the
the support structure of the hotel.
the form which is to allow the steel reinforcing of the column to be tied into the reinforcing for the beam.
form.

Fig.29 shows the finished beams and columns, which were finished in white paint. The project demonstrates how the moldability of GFRC can be utilized to create thin section shapes. The GFRC units had
smooth flawless surfaces that could be painted directly. Had the beams and columns been poured into
removable wood forms significant touch up work would have been necessary to provide the smooth
surface. Also the cost of erecting the wood forms, stripping them, and cleaning them was avoided. GFRC
offers a cost effective way of forming structural concrete, while providing a concrete surface and finish.

CONCLUSIONS

Glass fiber-reinforced concrete (GFRC) has been successfully in use worldwide for over 30 years. There are a variety of applications for this novel composite material. This paper described four specific
applications, namely, architectural panels, energy efficient housing, sewer linings, and permanent form-
work that illustrate the numerous benefits that GFRC offers to the construction industry.

REFERENCES


Fig. 1—San Francisco towers, San Francisco.
Fig. 2— Typical load-deflection curve for GFRC.

Appendix B — Limiting Stress Determination

Table B1. Data from 20 consecutive 28-day unaged production flexural tests (stresses in psi).

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<td>1103</td>
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Average yield strength, $f_{y} = 1178$ psi
Standard Deviation, $\sigma = 119$ psi
Coefficient of Variation, $V_{c} = \sigma / f_{y} = 119 / 1178 = 0.101$

$$f_{y} = f_{y} (1 + V_{c})$$
$$= 1178 [1 + (2.539)(0.101)]$$
$$= 1876 \text{ psi}$$

Average ultimate strength, $f_{u} = 2703$ psi
Standard deviation, $\sigma_{f} = 243$ psi
Coefficient of Variation, $V_{c} = \sigma_{f} / f_{u} = 243 / 2703 = 0.090$

and $f_{u} = 1/3 f_{y} (1 + V_{c})$
\[= 1/3 (2703 [1 - (2.539)(0.090)])
\[= 694 \text{ psi}
\]

Controlling $f_{u}$ is smallest of: $f_{u}^r = 876$, $f_{u}^l = 694$, and $f_{u} = 1200$

For single skin in flexure:

$$f_{u} = f_{y} = 0.75 (0.75) (694) = 521 \text{ psi}$$ (5-8)

For box section flexure:

$$f_{u} = f_{y} = 0.75 (0.75) (694) = 260 \text{ psi}$$ (5-8)

For tension:

$$f_{u} = 0.4 \phi f_{u}^r$$

$$= (0.4)(0.75)(694) = 208 \text{ psi}$$

Quality control notes:
The average of all sets of three consecutive yield strength tests must not be less than the $f_{u}$ used in design (694 psi). Also, the average of all sets of three consecutive ultimate strength tests must not be less than three times $f_{u}$ used in design (694 x 3 or 2082 psi).

No individual yield strength test shall be less than 90% of the $f_{u}$ used in design (694 x 0.9 or 625 psi). Also no individual ultimate strength test (6 coupons) shall be less than 3 times 90% of the $f_{u}$ used in design (3 x 0.9 x 694 or 1874 psi).

Fig. 3—Limiting stress determination.
Fig. 4—Flex anchor connection. (Note: 1 in. = 25.40 mm.)

Fig. 5—Flex anchor connection.
Fig. 6—Typical panel frame connection to GFRC skin.

Fig. 7—GFRC architectural panel.
Fig. 8—Example of panel frame connection to structural steel.

Fig. 9—White GFRC panels cladding Art Institute, Chicago.

Fig. 10—Biege GFRC panels cladding Marriott Hotel, San Francisco.
Fig. 11—Biege GFRC panels cladding Park Plaza Hotel, San Francisco.

Fig. 12—GFRC replacement cornice, Fairmont Hotel, San Francisco.

Fig. 13—GFRC energy efficient house, Iowa.
Fig. 14—Installation of insulated GFRC wall panel.

Fig. 15—Insulated GFRC roof panels showing joint design.

Fig. 16—GFRC house under construction.
Fig. 17—Thermal imaging comparison of conventional house and GFRC house.

Fig. 18—GFRC panels relining old brick sewer, London, UK.
Fig. 19—Semi-automatic reciprocating spray machine.

Fig. 20—GFRC permanent beam form.

Fig. 21—Steel folding forms for GFRC beam forms.
Fig. 22—Lay-out of steel folding forms in the factory.

Fig. 23—Spraying GFRC onto steel mold. After spraying mold, sides are folded vertically and secured.

Fig. 24—Demolding by dropping sides of steel mold. GFRC unit then removed.
Fig. 25—Finished GFRC beam form.

Fig. 26—Molds for GFRC permanent column forms.

Fig. 27—GFRC column forms in place.
Fig. 28—GFRC beam forms placed between column forms before pouring concrete.

Fig. 29—Finished column and beams used as decorative feature in hotel restaurant.
When You Want Concrete without Cracks, Joints, Curling, and Reinforcing Bars

by D. Flax

**Synopsis**: The common problems associated with concrete include drying shrinkage, cracking and curling. This paper will discuss how two time proven technologies, namely, Type K shrinkage-compensating concrete and synthetic fibers, have been combined to eliminate, or at the very least minimize, these problems. In the absence of drying shrinkage, concrete does not develop drying shrinkage cracks, control joints become unnecessary, curling is almost non-existent, spalling at joints is minimized since the only joints required are the construction joints, and required ongoing maintenance of the slab is minimal since there are so few joints. The Type K shrinkage-compensating concrete addresses the problem of concrete shrinkage and the synthetic fibers restrain the expansion of the Type K shrinkage-compensating concrete. Temperature steel for crack control can be eliminated and both the initial costs and the lifecycle costs are normally lowered. The combination of Type K shrinkage-compensating concrete and synthetic fibers has created a new future for concrete.

**Keywords**: crack; curl; fiber; floor; joint; shrinkage-compensating; Type K.
David Flax is the Southwestern Regional Manager of the National Business Development Group with the Euclid Chemical Co., based in Ohio. He received his degree in civil engineering from Rensselaer Polytechnic Institute (RPI). He has several years of experience working with the U.S. Army Corps of Engineers doing research. He has worked as a field engineer and a contractor and has over 30 years of experience in the field of concrete. He has published many articles in various publications and was awarded the Extra Yard Award by the Southern California Chapter of ACI for his 30+ years of service to the concrete industry.

INTRODUCTION

CTS Cement Manufacturing Corp. has taken two proven, time-tested technologies from the past and present, to create a new future for concrete. These two proven technologies, namely, are Type K shrinkage-compensating concrete, which has been used successfully for over 40 years, and discrete synthetic fibers, which have been used successfully for over 30 years. The combination of these two technologies and materials has the potential to create a new future for concrete where cracks, joints, curling, rebars, spalls, and maintenance for non-structural concrete are either eliminated or, at the very least, minimized. Furthermore, with the marriage of these technologies, both initial costs of construction and life-cycle costs can potentially be reduced. This paper will discuss each of these two technologies and how they can be used together to create this unique set of benefits for concrete.

TYPE K SHRINKAGE-COMPENSATING CONCRETE

When Type K shrinkage-compensating concrete was developed over 40 years ago it was called “self-stressing concrete” because as the concrete attempted to expand it would try to stretch the rebars thereby putting them into tension and itself into compression. This is ideal, since that is the preferred stress to have on each of these materials. To date, Type K shrinkage-compensating concrete has been used in the United States in thousands of slabs totaling hundreds of millions of square feet.

How Type K shrinkage-compensating concrete works

For the first 7 days after placement, the Type K shrinkage-compensating concrete expands slightly. Most of the potential expansion is restrained by the reinforcement. This restraint puts the reinforcement into tension and the concrete into compression, which is where these two materials work best. After seven days of wet cure, the concrete dries and shrinks the little bit that it had expanded relieving the stresses in the reinforcement and concrete. The concrete ends up the same volume as when it was placed and in a neutral stress condition or with a small residual compressive stress.

As stated in section 1.2 of ACI 223-98 Standard Practice for the Use of Shrinkage-Compensating Concrete1 “…expansion will induce tension in the reinforcement and compression in the concrete. On subsequent drying, the shrinkage merely relieves the expansive strains…” Section 3.4.5 of the same report further says that “…contraction joints are eliminated…” Figure 1 depicts how this mechanism works. The actual expansion is about the same as a 70°F (21.1°C) temperature change, so there should be no concern about the expansion causing damage. Another factor that makes Type K shrinkage-compensating concrete work is that more water is tied up in the hydration process. Concrete is typically placed at a 0.45 to 0.50 water/cementitious materials ratio (w/cm), but Portland cement only needs about a 0.22 to 0.25 w/c for full hydration. The rest of the water, known as water of convenience, is just there to help the contractor place the concrete. All that excess water has to come out of the concrete. One cubic yard of a typical 3,000 psi (21 MPa) Portland cement concrete mix with a 0.50 w/cm ratio has about two cubic feet of excess water. There are only 27 cubic feet in a cubic yard and two of them must come back out, so it is quite understandable that Portland cement concrete shrinks and cracks. But Type K shrinkage-compensating concrete requires a higher w/cm ratio for full hydration, so there is less excess water in the hardened concrete. Eliminating the volume change from the excess water leaving the slab is a big factor in eliminating drying shrinkage and subsequent cracking.

Slab benefits

Very large placements of 20,000 to 50,000 square feet (1858 to 4645 square meters) without control joints are common with Type K shrinkage-compensating concrete. Figure 2 shows a 50,000 square
foot (4,645 square meter) slab, which is about the size of an average grocery store. Assuming 15 foot (4.6 meter) joint spacing in the portland cement concrete slab as shown in Figure 2, means that there would be 28 joints totaling 6,270 lineal feet (1911 meters). Note that there is only one joint in the Type K shrinkage-compensating concrete slab and the reason that single construction joint is there is that many contractors cannot place 50,000 square feet (4,645 square meters) in one day. The end result is that there are only 210 lineal feet (64 meters) of joints with Type K shrinkage-compensating concrete instead of the 6,270 lineal feet (1911 meters) of joints that are required for a Portland cement concrete slab. This means that over a mile of joints can be eliminated on this relatively small slab by using Type K shrinkage-compensating concrete. That is over a mile of joints that do not have to be sawcut, do not have to be joint-filled, do not have to have shear load transfer, and do not have to be maintained by the owner for the life of the building. The only joints required are construction joints and those are normally 100 to 200 feet (30.5 to 61 meters) apart. The limit to the joint spacing is the size slab that the contractor can place in one day using good concreting practices. No sawcut control joints means that over 90% of the joints are eliminated, which means that over 90% of the spalling should be eliminated as well. It also means there are no sawcut joints to fill with joint filler and no dowel baskets. Dowel baskets are expensive, cumbersome to work with, slow the concrete placement, and must be kept perfectly aligned because if misaligned they can cause cracking.

With Type K shrinkage-compensating concrete there is virtually no curling. As stated in section 3.4.2 of ACI 223-98 Standard Practice for the Use of Shrinkage-Compensating Concrete¹ "… restrained expansive strains are greater at the top surface than at the bottom, so reversed curling conditions develop… counterbalanced by the dead weight of the slab itself." This reverse curling means that the slab actually tries to curl up in the middle rather than at the edges. It tries to curl into a dome instead of into a bowl, but the weight of the slab prevents that so the slab stays flat.

Type K shrinkage-compensating concrete is flat initially and stays flat over time. On a typical job, Sofa Express in Tennessee, the initial flatness of the slab on grade was FF 99.2 and was still an astounding FF 98.6 5 weeks later. Even more astounding was the flatness measurement of FF 97.8 after 1 year. Flatness readings for portland cement concrete are normally taken within the first day or two, because the slabs are never again as flat as they are during those first few days. Section 1.5 of ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials² states - “As shown in the examples by Suprenant (2003d), curling or warping can cause floor flatness and levelness, as measured by F-numbers, to decrease by 20 to 50% in a year.” That means that an initial reading of FF 100 for a Portland cement concrete slab could be expected to be between FF 50 and FF 80 after a year, as opposed to the flatness for a Type K shrinkage-compensating concrete slab that remains virtually unchanged for a year or longer.

Per Section 2.5.7 of ACI 223-98 Standard Practice for the Use of Shrinkage-Compensating Concrete¹ “… abrasion resistance is 30 to 40 percent higher than Portland cement concrete.” This is a function of the bleed water. Because Type K shrinkage-compensating concrete uses up more of the water in the mix for hydration, there is little, if any, bleed water. Bleed water is bad for the surface of the concrete since it carries fines and laitance to the surface, which reduce the surface abrasion resistance. The bleed water also raises the water/cement ratio at the wearing surface making the compressive strength lower at the surface and, therefore, the abrasion resistance lower. Contractor’s punch lists are shorter without the cracks to repair, spalls to patch, and curl to grind.

**Tilt-up benefits**

As stated in Section 3.5.1 of ACI 551R-92 Tilt-Up Concrete Structures³ “Panels are normally cast on the floor slab and any imperfections in the slab will be reflected in the panel finish.” And Section 3.5.1.1 states “Locate crack control joints to minimize unsightly lines being transferred to the panel. These lines will always be visible.”

In recent years owners and specifiers have increased the number of joints in slabs in an attempt to reduce cracking and curling. This has created more problems in tilt-up panels, since there are more joint lines to be transferred to them. The only way to keep joints, curls, and imperfections in the floor slab from showing in the tilt-up panels is to eliminate them. This is particularly true if the panel will be sandblasted.

The solution to getting superior looking tilt-up panels is to cast them on a floor slab that is very flat and has few joints or cracks, such as a Type K shrinkage-compensating concrete slab.
THE MARRIAGE OF TYPE K SHRINKAGE-COMPENSATING CONCRETE AND SYNTHETIC FIBERS TO MAKE SYSTEM-K

Type K shrinkage-compensating concrete tries to expand, but most of that expansion is restrained by the reinforcement. The expansion must be restrained to induce compression in the concrete and that has traditionally been done with steel. Testing has shown, however, that the restraint can be accomplished using synthetic micro-fibers.

Initial lab testing

Extensive testing was done under the auspices of the UCLA Department of Materials Science and Engineering to determine if small synthetic micro-fibers could restrain the expansion of the Type K shrinkage-compensating concrete as well as the steel that had been used to restrain the expansion for over forty years.

Nearly a year was spent testing many dozens of different synthetic fiber materials, configurations, lengths, diameters, and dosages. Hundreds of ASTM C806 and ASTM C878 beams were cast and measured for expansion and shrinkage over time. ASTM C806 bars are 2” x 2” x 10” (5.1 cm x 5.1 cm x 25.4 cm) mortar bars, while ASTM C878 bars are 3” x 3” x 10” (7.6 cm x 7.6 cm x 25.4 cm) concrete bars, which include coarse aggregate. Once stripped the bars were immersed in water for seven days and then allowed to air dry. They were measured daily with a length change comparator for the first seven days and then weekly thereafter.

This testing resulted in the creation of a new fiber called K-Fiber. As shown in Figure 3, it works just as rebar has worked for decades to restrain most of the expansion of the Type K shrinkage-compensating concrete. When the concrete dries it shrinks relieving the tension in the fibers and the compression in the concrete.

Of the many systems investigated, CTS’s K-Fiber provided suitable restraint when mixed with CTS’s shrinkage-compensating cement.” Because the fibers are only ¼” (0.64 cm) long and monofilament, they do not create finishing problems and are invisible on the surface.

Initial field test

Figure 4 shows the results of the initial field test of concrete made with System-K. The test sections were 50’ x 6’ x 6” (15.2 m x 1.8 m x 1.8 m). This testing showed that shrinkage-compensating concrete made with System-K, which included K-Fiber, and no rebar is significantly better than Portland cement concrete with rebar at reducing drying shrinkage cracking. After a few months the Portland cement concrete section had the usual drying shrinkage cracks every 6’ to 8’ (1.8 to 2.4 m), but after three years there were still no cracks in the Type K shrinkage-compensating concrete or System-K shrinkage-compensating concrete, in spite of the fact that the length to width ratio for the sections was over 8:1; far exceeding the ACI recommendation of 2:1 maximum for Portland cement concrete and 3:1 maximum for shrinkage-compensating concrete.

Construction job issues that fiber-reinforced shrinkage-compensating concrete virtually eliminates

Some key construction job issues that fiber-reinforced shrinkage-compensating concrete (example, System-K) shrinkage-compensating concrete helps to virtually eliminate are as follows:

- Steel reinforcement chairing;
- Ready mix trucks and laser screeds driving on the steel;
- Trying to keep the steel properly positioned in the concrete;
- The men with the hooks trying to lift the steel into position;
- Blocking up pump lines;
- Dowel baskets; and
- Saw-cutting.

FIBER-REINFORCED SHRINKAGE-COMPENSATING CONCRETE (SYSTEM-K) PROJECTS

Sofa express

Figure 5 shows the Sofa Express Warehouse in Portland, Tennessee, which was the first full-scale System-K job. The 130’ x 110’ (39.6 m x 33.3 m) placement shown has no cracks. Numbered lines have been added where a traditional slab made with Portland cement would have been expected to crack.
Fiber-Reinforced Concrete in Practice

1) Mid-panel: A 130' (39.6 m) slab would be expected to have cracks at mid-panel and elsewhere. 2) Columns in the slab: Cracks would be expected at the column lines, which is why joints have traditionally been placed there. Note also that boxouts were eliminated. All that was required was a foam wrap for isolation as can be seen on the nearest round column. 3) Columns at the perimeter: Cracks would be expected at each column. 4) Bollards: There are usually cracks from bollards to the edge of the slab. 5) Re-entrant corner: There is almost always a crack running off at 45 degrees. 6) Plastic and crazing: Numerous plastic and crazing cracks would be expected. The numbered lines show where Portland cement concrete would have been expected to crack; but the System-K shrinkage-compensating concrete did not crack.

Kyle de Bruyn, an engineer with CTS Cement says, “I inspected the entire slab on hands and knees eight weeks after placement searching for cracks. There were none.”

New East High School

Figure 6 and Figure 7 show the New East High School in Vancouver, Washington, which was the first of six schools in that area to use System-K. The large slabs on grade shown in Figure 6 contained many block-outs, re-entrant corners, and planes of weakness from utility risers; yet the slabs are virtually crack-free. The composite metal deck shown in Figure 7 is almost 30,000 square feet (2787 square meters) and is shaped like a square donut around a central atrium. The only cracks are at some of the negative moments and they are very tight. Note that System-K does not eliminate the need for structural rebar at the negative moments or elsewhere.

Duncan Aviation

Figure 8 shows the 175' by 125' (53.3 m x 38.1 m) Duncan Aviation Hangar in Grand Rapids, MI. The 22,000 square feet (2044 square meters) was placed in one pour and it has no joints. The owner wanted a floor that was flat and he did not want joints and cracks, because of fuel spillage. This hangar, without joints and curl, is both attractive and functional. The owner has other hangars scheduled to be built with System-K.

COST SAVINGS WITH FIBER-REINFORCED SHRINKAGE-COMPENSATING CONCRETE (SYSTEM-K)

Fiber-reinforced shrinkage-compensating concrete is instrumental in reducing the cost of construction. Examples of cost savings realized with the use of fiber reinforced shrinkage-compensating concrete include:

• With the high costs of rebar, eliminating temperature steel saves a considerable amount of money.
• Without rebar, the costs of rebar installation and job-day labor for hooking are eliminated.
• Without rebar, the job can often be tail-gated thus eliminating the cost of pumping.
• Without rebar in the way, job day productivity is increased.
• Without sawcut joints, the costs of dowel baskets and sawcutting are eliminated.
• Without sawcuts, the costs for material and labor to fill the sawcut joints are eliminated.
• With 90% fewer joints, the costs for spall repairs are reduced considerably.
• Without curling, the cost for grinding is eliminated.
• Without joints and curl, the cost to repair tilt-up panels is minimized.
• With all these savings, the initial costs of construction are reduced.
• Without joints and curl, the on-going maintenance costs are reduced. In a recent survey, 92% of facilities managers said that joints were their number one floor repair concern.
• With all these savings, life-cycle costs are reduced.

CONCLUSIONS

The primary significance of this research is that slabs without joints, curl, and cracks can be created economically using fiber-reinforced shrinkage-compensating concrete. This can be done by eliminating the temperature steel that has traditionally been used in slabs to provide crack control. Fiber-reinforced shrinkage-compensating concrete helps produce better concrete slabs with a minimum of joints, cracks, curling, spalling, maintenance, and rebar. There are savings in both initial and life-cycle costs. Type K shrinkage-compensating concrete and synthetic fiber technologies combine to provide a new high-quality, cost-effective future for concrete. This research has shown that large, flat slabs without joints are now an economic reality.
Flax

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2. ACI Committee 302, “Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06),” American Concrete Institute, Farmington Hills, MI, 2006.
3. ACI Committee 551, “Tilt-Up Concrete Structures (ACI 551R-92),” American Concrete Institute, Farmington Hills, MI, 1992.

Fig. 1—Shrinkage-compensating concrete versus portland cement concrete from ACI 223.1 Portland cement concrete is static for 7 days then goes into tension and cracks. Type-K Concrete expands slightly for 7 days but, more importantly, the reinforcement goes into tension and the concrete goes into compression. The concrete does not go into tension and concrete will not crack unless it is in tension.

Fig. 2—Save over a mile of joints. More than one mile of joints are eliminated from this slab, which is the size of an average grocery store, by using Type K shrinkage-compensating concrete. This means over one mile of joints that do not have to be sawcut, do not have to be joint-filled, do not have to have shear load transfer, and do not have to be maintained for the life of the building. (Note: 1 ft = 0.3048 m; 1 ft² = 0.0929 m².)
Fig. 3—Expansion restrained by reinforcement. Most of the potential expansion is restrained by the reinforcement forcing the concrete into compression.

Fig. 4—Initial System-K field test. In this field test using 50 ft x 6 ft (15.2 m x 1.8 m) panels of 6 in. (15.2 cm) thick concrete, the System-K shrinkage-compensating concrete performed as well as or better than steel reinforcement.
Fig. 5—No cracks in this 130 ft x 110 ft slab (39.6 m x 33.5 m). This is first full-sized slab placed using System-K shrinkage-compensating concrete and there were no cracks. The numbered lines show some cracks that would have occurred if this 130 ft x 110 ft (39.6 m x 33.5 m) slab had been placed using portland cement concrete.

Fig. 6—Crack-free slab made using System-K shrinkage-compensating concrete showing blockouts, reentrant corners, and stressed zones from utility risers.
Fig. 7—30,000 square foot (2787 square meter) composite metal deck. This large composite metal deck is shaped like a square donut around a central atrium. The only cracks are at some of the negative moments and they are tight cracks.

Fig. 8—22,000 square foot (2044 square meter) aircraft hangar with no joints. This 22,000 square foot (2044 square meter) aircraft hangar floor is 175 ft x 125 ft (53.3 m x 38.1 m). It was placed in one pour, has no joints, is very flat, and the aesthetics are exceptional.
Flax
Applications of Alkaline-Resistant Cellulose Polymer Fiber in Ready Mixed Concrete

by H. J. Brown, J. D. Speakman, and J. H. Morton

Synopsis: The ready-mixed industry makes up nearly 75 percent of the U.S. consumption of cement and this represents a significant market opportunity for the possible use of wood pulp fiber in ready-mixed concrete applications. Most research surrounding cellulose fiber-cement composites has focused on manufactured products such as flat and corrugated sheets for cladding and roofing, nonpressure pipes, and cable pits. Integration of cellulose fiber into the mainstream ready mix operation for value added benefits to the concrete mix is proven in practical applications as shown in this paper. Concrete properties enhanced by cellulose fiber addition are summarized. Properties of the fiber concrete composite discussed in this paper include fiber and cement matrix bond, alkaline stability, freeze-thaw durability, plastic shrinkage cracking resistance, combustibility, fire resistance and impact resistance. The paper also highlights selected project examples demonstrating the use of cellulose fibers in concrete, such as, overlays, slab on grade, and decorative concrete.

Keywords: cellulose; fiber; reinforcement.
INTRODUCTION

Cellulose fibers – background
Kraft pulps are the dominant wood fiber types used in cement-based materials. These fibers have minimum lignin contents and have exhibited resistance to alkaline environments in past applications [4]. ACI 544.1R-96 lists kraft pulps as having an average fiber length of 2.54 to 5.08 mm (100 to 200 mil), average diameter as 25 to 76 µm (1 to 3 mil), average tensile strength of 696 MPa (101 ksi) and water absorption of 50 to 75 percent, based on for the TAPPI water retention value (WRV).

Cellulose fiber used in this investigation and project examples described below meets ICC Acceptance Criteria 217, as reported in ICC-ES ESR-1032. The fiber is prepared from slash pine fibers by standard kraft pulping and elemental chlorine free (ECF) purification. It was tested successfully by certified independent labs in the following tests [2,3]: shrinkage crack tests (per ASTM 1579), static flexural strength (ASTM 1018), air content and unit weight (ASTM C 138), flexural strength (ASTM C 78), hydration (ASTM C 856), freeze thaw (ASTM C 666(A)), compressive strength (ASTM C 39), bond strength (ASTM C 234), alkaline stability (ASTM D 6942), noncombustible rating (ASTM E 136), spalling resistance to fire and impact resistance (AC 217 annex B). Furthermore, after a considerable number of field pours, overwhelming positive feedback has been received from professional ready mix producers, contractors, and finishers.

FIBER PROPERTIES

Bond between fiber and cement matrix
Based on numerous images from scanning electron and optical microscopes, cement paste/fiber bonding is well documented (see Figures 1 and 2). Fibers are well dispersed based on sieve analysis of the fresh concrete and well bonded. The concrete matrix adheres well to fiber surfaces and no evidence of degradation is observed after months to years of inspection by SEM and optical microscopy. Fiber cement specimens prepared in 1990 [4] with softwood kraft fibers show no signs of fiber degradation after 19 years. Fiber failure occurs when concrete fails without fiber pull out.

Alkaline stability
There have been concerns regarding the long-term performance of cellulose fiber reinforced cement composites; some natural fibers tend to disintegrate in the alkaline environment of cement [5]. Concern over degradation of the fiber in this environment has been alleviated with special proprietary processing of the fiber. It is believed that the cellulose described in this paper will not significantly deteriorate during the useful life of concrete.

Physical fiber characteristics [6,7]
- Avg. Fiber Length = 2.9 mm (114 mil)
- Avg. Coarseness = 31.0 mg/100m (31.0 dg)
- Cell Wall Thickness = 9.7um (0.38 mil)
- Avg. Denier = 2.8 g/9,000m (31.0 dg)
- Fiber Count = 1.11x10^6 fibers/g (500x10^6 fibers/lb)
- Fiber Strength = 696 MPa (101 ksi)

The properties of slash pine (Pinus elliottii) result in one of the highest fiber strengths [7] for cellulose based fibers giving a significant advantage over other cellulose fibers. This strength advantage makes it
an excellent cellulose fiber for use in cementitious applications where fiber strength and fiber strength durability are a major concern.

**Freeze thaw durability**

AC217 conditions of acceptance are that the average durability factor of the three specimens containing the fibers shall be at least equal to the average durability factor of the control specimens. The freeze/thaw resistance of concrete was improved using dry and slurry specialty cellulose products over synthetic fiber and control specimens and met AC217 acceptance conditions [2].

**Plastic shrinkage cracking resistance**

Shrinkage cracking was quantified by computing the area of surface cracks in each panel specimen to obtain a value in square meters, which represents the assumed total area of crack opening on the surface of the concrete panels. The effect of fibers on the development of shrinkage cracking was determined by expressing the cracking value for the concrete with cellulose fibers as a percentage of the value for the control concrete (without fibers) when subjected to the identical and simultaneous drying conditions. AC217 acceptance conditions state that cracking must be reduced by a minimum of 50 percent. Plastic shrinkage crack reduction was determined as 76.8 percent and 80.0 percent for the ASTM C1579 method (0.6 kg/m² (1.0 lb/yd²) and 0.9 kg/m² (1.5 lb/yd²) respectively) which met AC217 conditions of acceptance [2].

**Impact resistance**

The minimum AC217 acceptance for the final results is that the cellulose fibers increase the impact of concrete by 40 percent at 7 days and 40 percent at 28 days. The impact resistance met AC217 acceptance conditions except when air entrainment was used [2].

**PROJECT EXAMPLES**

**I-35 Northern Oklahoma road repair**

This repair consisted of 3.62 km (2.25 miles) of 23 cm (9 inch) thick shoulder with 1.78 kg/m³ (3.0 lbs/yd³) of cellulose fiber (Figure 3). Shoulder was planned to be open early to carry interstate traffic while traffic lanes were repaired. This was the first time the Oklahoma Department of Transportation used a flex mix and cellulose fiber. Polypropylene had been specified but was replaced with cellulose fiber because of the strength advantage.

**I-40 Central Oklahoma road repair**

This bonded overlay project (2006) consisted of the placement of 38229 m³ (50,000 yd³) of concrete over 21 km (13 miles) of the Muskogee Turnpike in Oklahoma (Figure 4). Cellulose fiber was added to the central mixer at a dosage rate of 1.78 kg/m³ (3 lbs/yd³). The concrete was then discharged into dump trucks and placed via slip form paver. Excellent fiber dispersion and consolidation was achieved leading to a homogenous concrete mixture. The surface was floated, dragged and longitudinal tined. No “clumps or fur balls” were noted by the construction crew which had not been the case when longitudinal tining was attempted in prior jobs with other fiber types. OTA has stated that they believe this work has resulted in a 30 year repair.

**Slab on grade projects**

Cellulose fiber was used in the parking areas of Dixie RV Super Stores and Camping World (Figure 5). The project involved placing 3058 m³ (4,000 yd³) of a 15 cm (6 inch) thick exterior paving slab with a fiber addition of 0.6 kg/m³ (1.0 lb/yd³). Cellulose fiber was chosen for this job to improve finish-ability and reduce cracking.

A golf cart path (Figure 6) was constructed at the Spanish Peaks Golf Course in Big Sky, Montana. For this job, 2294 m³ (3,000 yd³) of concrete was poured with a 0.9 kg/m³ (1.5 lb/yd³) cellulose fiber addition. The paths were finished with a burlap drag.

Paving in front of a group of retail and real estate offices was also completed (2004) with cellulose fiber reinforcement in Lavergne, Tennessee (Figure 7). A 15 cm (6 inch) slab was poured with 0.6 kg/m³ (1.0 lb/yd³) cellulose fiber reinforcement for this 229 m³ (300 yd³) parking area.

A warehouse loading dock (Figure 8) was constructed (2004) with a 15 cm (6 inch) slab poured with
Brown et al.

0.9 kg/m³ (1.5 lbs/yd³) cellulose reinforcement for this 38 m³ (50 yd³) pour.

Precast concrete vaults were constructed using 0.6 kg/m³ (1.0 lbs/yd³) cellulose fiber reinforcement to create a smooth finish. The producer noted no “fuzz” on the surface.

An exposed aggregate driveway in Manchester, TN showed great surface finish with cellulose fiber (Figure 10). It was a 30.6 m³ (40 yd³) pour that gave the contractor no problems during construction.

Decorative concrete (2006-2007)

Cellulose fiber is also an excellent reinforcing fiber for use in decorative concrete as shown in Figures 11/12. Benefits of including cellulose fibers in decorative concrete are as follows:

- Fibers are invisible in the final product;
- Fibers do not interfere with the stamping, skinning, staining, and sealing processes;
- Fibers soak in and hold the colors/stains;
- Fibers adds luster to the final stained or pigmented surface; and
- Fibers do not require additional steps (no burning off of fibers, picking out clumps, additional tool washing, etc.).

CONCLUSIONS

The ready-mixed industry makes up nearly 75 percent of the U.S. consumption of cement and this represents a significant market opportunity for the possible use of wood pulp fiber in ready-mixed concrete applications. Most research surrounding cellulose fiber-cement composites has focused on manufactured products such as flat and corrugated sheets for cladding and roofing, non-pressure pipes and cable pits. Integration of cellulose fiber into the mainstream ready mix operation for value added benefits to the concrete mix is proven in practical applications as shown in this paper. A summary of concrete properties that are enhanced by cellulose fiber was presented in this paper as well as highlighting examples of field projects including overlays, slab on grade and decorative applications.

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Fig. 1—Field sample – bonding.

Fig. 2—Lab sample – bonding.

Fig. 3—I-35 shoulder repair.

Fig. 4—I-40 overlay repair.
Fig. 5—Parking lot.

Fig. 6—Golf cart path.

Fig. 7—Real estate and retail parking.

Fig. 8—Warehouse slab.
Fig. 9—Precast vaults.

Fig. 10—Exposed aggregate driveway.

Fig. 11—Skinning and staining.

Fig. 12—Pigmenting and stamping.
Alkali Resistant Cellulose Fibers for Decorative Concrete

by R. I. Bell and J. H. Morton

Synopsis: This paper discusses the use of alkali resistant cellulose fibers for decorative applications. The penetration of fibrous reinforcement for decorative concrete applications has been somewhat limited. The main barrier has been interference of the fiber during many of the intricate and complicated finishing steps employed in these high-end concrete applications. Some of the complications encountered are fibers sticking to the stamps and dragging out of the surface, fiber clumps found at the surface creating blemishes, lack of stain receptivity by the fibers, etc. These aesthetic complications can often result in customer dissatisfaction with the final product. The commercial entry of alkali resistant cellulose fibers for fibrous reinforcement in concrete has provided a new option for fiber reinforced concrete in decorative concrete applications. The potential benefits of cellulose fibers in concrete are expected to help further enlarge the use of fiber reinforced concrete in the market place.

Keywords: cellulose fiber; decorative concrete; fiber-reinforced concrete (FRC).
INTRODUCTION

Decorative concrete is a fast growing segment of the ready mixed industry. The majority of applications are flatwork where fiber reinforced concrete (FRC) is proven to provide performance benefits. Unfortunately, fiber penetration into this segment has been low primarily due to aesthetic issues fibers can bring. A cellulose based fiber can provide the performance benefits of FRC while enhancing the aesthetic beauty of decorative concrete which adds growth to the FRC segment.

DECORATIVE CONCRETE MARKET

What is decorative concrete?
Concrete with one or more of the following characteristics is generally considered “decorative concrete”:
- Contains an integral pigment (mixed into batch);
- Has received a color hardener or colored release agent;
- Has surface patterns or impressions (e.g. cobblestone);
- Has received an acid stain;
- Has exposed aggregate or sand finish; and
- Has been etched or engraved.
Decorative concrete is versatile and cost effective compared to marble or granite.

Market statistics
Decorative concrete is one of the fastest growing segments of the concrete industry. [1] PCA reported that 1.0 - 1.5% of all concrete production from calendar year 2000 was integrally colored. PCA estimates this to increase to around 6% by year 2007. For concrete used for exterior hard-scape applications, about 16% was integrally colored (ranging up to 30% in some local markets).

Fiber acceptance
Most decorative concrete (DC) is flatwork, which is an excellent target for fibrous reinforcement. Fibers provide crack control and aid long term durability. These performance benefits are a natural enhancement for higher priced DC jobs. Unfortunately, penetration has been limited because many fibers can interfere with the more advanced and delicate finishing steps required for DC. This can cause imperfections to the final product or extend finishing times. As a result, contractors too often forego the performance benefit from FRC because of the added aesthetic risks to their end product.

Recently introduced into the commercial market place, alkali resistant cellulose wood fiber has been widely accepted for use in decorative concrete by contractors. This natural fiber is composed primarily of the organic polymer, cellulose, and has been modified with a coating to enhance its natural alkaline resistance. [2] Cellulose wood fiber is differentiated from synthetic fibers by its very low length, low denier, and hydrophilic nature. The small size makes the fiber virtually “invisible” in concrete and the hydrophilic nature allows it to absorb pigments and stains and release absorbed water to the matrix during curing. Countless decorative jobs have been successfully completed with this type of fiber which provided excellent crack control, introduced no aesthetic problems, and enhanced the overall beauty of the project.

CELLULOSE FIBERS WELL-SUITED FOR DECORATIVE MARKET

Fiber dimensions
Cellulose fibers are very small compared to synthetic fibers commonly used in FRC [3] for secondary reinforcement. Table 1 compares the physical properties of a commercially available cellulose wood fiber to a commercially available polypropylene fiber. (See [4] for more properties of the wood cellulose fibers discussed in this paper.) Figure 1 is a photograph taken through a microscope of a polypropylene fiber next to a cellulose fiber for a visual size comparison. The small size of cellulose fibers makes them
virtually invisible in concrete. This characteristic is highly valued in DC because visible fibers protruding from the surface take away from the natural and artistic aspect of decorative concrete. Figure 2 is a close up photograph of an exposed aggregate concrete section containing 1.5 lb/yd³ (0.9 kg/m³) of cellulose fibers (the photograph was taken the same day of the job). The cellulose fibers added to the concrete are not visible on the surface and there are none of the visible fiber clumps that are often found with synthetic fibers.

**Hydrophilic nature**

Cellulose fibers are absorbent, meaning they can take up and hold some water when the concrete is in the plastic state. For example, one cellulose fiber on the market can hold a maximum of 85% water by weight. The hydrophilic nature is what drives the bonding between the fiber and the cement paste and is a key component in the crack control mechanism. This initial moisture held by the fiber is also proven to provide internal curing, which enhances cement hydration resulting in improved strength properties. This hydrophilic property has two additional benefits for Decorative Concrete:

1. The excellent paste to fiber bonding further disguises and hides the fiber near the surface. Figures 3 and 4 are electron micrographs of the surface of a cellulose fiber which clearly show the coverage and intimate bond the cellulose fiber has with the paste. Figure 4 also shows the fiber failed rather than pulling out.
2. For pigments and stains, the fibers help receive, hold, and distribute the color throughout the concrete.

**Laboratory study for fibers to retain integral pigments, color hardeners, and stains**

A cement paste was made with a 0.38 water-cement ratio (w/c). Fibers were added to the paste at a 3 lb/yd³ (1.78 kg/m³) dosage. Integral pigment was added at a dosage of 10% of cement. The paste was mixed for 2 minutes and then diluted with water so the sample could be sieved over a 100-mesh (150 µm) screen. Fibers were collected and allowed to dry. This procedure was carried out using a commercially available cellulose fiber, nylon fiber, and polypropylene fiber.

1. One experimental set of fibers was treated with a Brick Red integral pigment. After filtering and drying, cellulose fibers retained color (Figure 5). The nylon fibers (Figure 6) and polypropylene fibers (Figure 7) retained none of the pigment and remained white.
2. A second set of experimental fibers underwent the same procedure with a Caramel colored hydrochloric acid based stain. After filtering and drying, the cellulose fibers retained color (Figure 8). The nylon fibers (Figure 9) and polypropylene fibers (Figure 10) remained white.
3. For the third set of experimental fibers, each fiber was added to a beaker containing Terra Cotta iron oxide based color hardener and water. The solution was mixed for 2 minutes, filtered over a 100-mesh (150 µm) screen. After filtering and drying, the cellulose fibers retained the color hardener color (Figure 11). The nylon fibers (Figure 12) and polypropylene fibers (Figure 13) remained white.

**Proven FRC performance**

The highly stable and purified forms of processed cellulose meet the performance challenges of today's concrete industry. The International Code Council (ICC) developed a stringent Performance Criteria (AC217) for the use of cellulose fiber in concrete which includes testing for:

- plastic shrinkage;
- alkali stability;
- impact testing;
- resistance to freezing and thawing;
- strength testing; and
- bond strength.

With proper testing and certification, cellulose wood fibers provide excellent secondary reinforcement in concrete as well as other proven benefits. [5,6] Properly certified cellulose fibers provide DC contractors the performance of FRC without complicated aesthetic issues. This will help increase the FRC market.

**FIELD OBSERVATIONS**

**Pigment uniformity**

One benefit of using cellulose fiber to reinforce DC is that contractors have noticed improved pigment
uniformity with cellulose fibers (Figure 14). The hydrophilic fibers are taking in pigmented moisture and helping to evenly disperse it throughout the load. More uniform pigment within the load results in a more uniform pigment in the final product. Contractors have also noticed more vibrant surface color with the addition of cellulose fiber. This is because the incorporation of cellulose fibers maintains better paste consistency at the slab surface allowing a better finish, making the color more vibrant.

**Easier stamping**

Stamping, or making patterned impressions into the concrete is a very delicate process. Timing and techniques are critical. Added complications can ruin a decorative job. There are several reasons cellulose fibers work well for decorative concrete stamping:

- Cellulose fibers do not present a physical obstacle for the process;
- Cellulose fibers do not stick to the stamp/skin molds;
- Cellulose fibers work well with color hardeners and release agents;
- Cellulose fibers do not pull out of the surface, leaving imperfections;
- Cellulose fibers are not found as balls and clumps on the surface;
- Cellulose fibers do not obviously protrude from the stamped surface; and
- Cellulose fibers allow the natural look and effect of the pattern to be preserved.

Figures 15, 16, and 17 illustrate successful commercial DC applications with cellulose fiber reinforcement that was stamped. Figures 18, 19, 20, and 21 illustrate successful DC job applications with cellulose fiber reinforcement finished with seamless skins.

**Scoring**

Scoring is the process of grooving the concrete to make decorative patterns by hand. Some DC contractors have experienced problems from imbedded synthetic fiber clumps and fiber balls popping out during scoring creating an immediate eye sore blemish. Synthetic fibers can also stick to the scoring tool and drag across the surface leaving behind imperfections. Cellulose fibers do not interfere with scoring:

- Cellulose fibers disperse well and do not ball or clump; and
- Cellulose fibers do not pull out and stick to the tools.

Figures 22 and 23 illustrate the clean scored edges that can be obtained when cellulose fiber reinforcement is used.

**Acid stains/coatings**

Acid stains are usually applied 10 to 14 days after placement. Protective sealants are applied later. Synthetic fibers protruding from the surface that do not received stains interfere with uniform lay down of sealant coatings. Small openings in the coatings often remain from the protruding fibers. FRC made with cellulose fibers work well with acid stains and sealants:

- Cellulose fibers do not stick out and protrude from the surface;
- Cellulose fibers near the surface will receive the stain; and
- Cellulose fibers do not interfere with the ability to apply a uniform sealant coating.

Figures 24 and 25 are successful commercial examples of DC pool decks reinforced with cellulose fiber.

**Exposed aggregate**

Exposed aggregate surface finishes reveal everything below the concrete surface. After paste is removed, synthetic fibers are often excessively visible and, at time, fiber clumps or balls are observed. Sometimes the contractor has to give the slab a “shave” or “burn” to satisfy customers. Cellulose fibers are well suited for exposed aggregate finishes:

- Cellulose fibers do not clump or ball together;
- Cellulose fibers do not result in a “hairy” surface finish once the paste is removed; and
- Cellulose fibers are not noticed by the customer.

Figure 26 shows cellulose fibers aren’t seen on the surface of exposed aggregate finish.

**Projects with multiple decorative concrete techniques reinforced with cellulose fiber**

A walkway (Figures 27 and 28) in Breckenridge, Colorado, was reinforced with 1.5 lb/yd³ (0.9 kg/m³) of cellulose fiber. The walkway was colored by means of an integral pigment and stamped with the aid of
a colored release agent.

A pool deck (Figure 29) in Memphis, TN, was reinforced with 1.5 lb/yd$^3$ (0.9 kg/m$^3$) of cellulose fiber. The pool deck was colored by means of an integral pigment and finished with a stamped technique. Details of the stamped image were then enhanced with addition of a color hardener.

A pool deck (Figure 30) in Memphis, TN, was reinforced with 1.5 lb/yd$^3$ (0.9 kg/m$^3$) of cellulose fiber. The pool deck was finished by use of seamless skins and scored for additional realistic detailing. The surface was acid stained to further enhance the surface aesthetics.

A “Cobblestone” driveway (Figures 31 and 32) in Memphis, TN, was reinforced with 1.5 lb/yd$^3$ (0.9 kg/m$^3$) of cellulose fiber. The surface was finished by stamping followed by acid staining.

A home entrance in walkway (Figure 33) in Seattle, Washington, was reinforced with 1.5 lb/yd$^3$ (0.9 kg/m$^3$) of cellulose fiber. Integral pigment was used to color the FRC. Finishing was accomplished by means of stamping with aide of a colored release agent.

A decorative element on a patio (Figure 34) in Memphis, TN, was reinforced with 1.5 lb/yd$^3$ (0.9 kg/m$^3$) of cellulose fiber. The image was created with the combined techniques of seamless skinning, acid staining and artistic scoring/staining.

CONCLUSIONS

The decorative concrete segment can benefit substantially from fibrous reinforcement. Penetration of FRC into this segment has been limited due to finishing and aesthetic problems caused by many of the synthetic fibers. Cellulose fibers for FRC do not cause aesthetic issues in decorative concrete. The small dimensions of cellulose fibers make them virtually invisible. The hydrophilic nature of cellulose fibers allows them to absorb and distribute colors and stains. Cellulose fibers have proven performance benefits in concrete for providing secondary reinforcement and are recognized by the International Code Council. The unique attributes offered by cellulose fibers are helping to expand the FRC market segment of decorative concrete.

ACKNOWLEDGMENTS

Photographs provided by: Ms. Vanessa Johnson of Buckeye Building Fibers, LLC (Memphis, TN); Kevin Baltz, President of Baltz & Sons (Memphis, TN); Colorado Hardscapes (Denver, CO).

REFERENCES

3. ACI Committee 544, “Report on Fiber Reinforced Concrete (ACI 544.1R-96) (Reapproved 2002),” American Concrete Institute, Farmington Hills, MI, 1996.
Table 1—Comparison of physical properties of commercially available natural wood cellulose fiber and commercially available synthetic polypropylene fiber.

<table>
<thead>
<tr>
<th>Average Fiber Property</th>
<th>Wood Cellulose</th>
<th>Polypropylene</th>
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<tr>
<td>Length, mil (mm)</td>
<td>83(2.1)</td>
<td>630(16)</td>
</tr>
<tr>
<td>Linear density, denier (denier (decitex)</td>
<td>2.5(2.8)</td>
<td>6.0(6.7)</td>
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<tr>
<td>Diameter, mil (m)</td>
<td>0.71(18)</td>
<td>1.18(30)</td>
</tr>
<tr>
<td>Count (x10^6), fibers/lb (kg)</td>
<td>720(1587)</td>
<td>44(971)</td>
</tr>
</tbody>
</table>

Fig. 1—Polypropylene fiber (top) and wood cellulose fiber (bottom).

Fig. 2—Exposed aggregate finish, cellulose fibers are “invisible.”

Fig. 3—Paste bonding to cellulose fiber.
Fig. 4—Note paste bonding and fiber failure.

Fig. 5—Cellulose/Red integral pigment.

Fig. 6—Nylon/Red integral pigment.

Fig. 7—Polypropylene/Red integral pigment.
Fig. 8—Cellulose/Caramel colored hydrochloric acid-based stain.

Fig. 9—Nylon/Caramel colored hydrochloric acid-based stain.

Fig. 10—Polypropylene/Caramel colored hydrochloric acid-based stain.

Fig. 11—Cellulose/Terra Cotta iron oxide-based color hardener.
Fig. 12—Nylon/Terra Cotta iron oxide-based color hardener.

Fig. 13—Polypropylene/Terra Cotta iron oxide-based color hardener.

Fig. 14—Decorative concrete discharged into wheelbarrow.

Fig. 15—Stamped cellulose FRC.
Fig. 16—Cellulose FRC, stamping in progress.

Fig. 17—Cellulose FRC stamped sidewalk.

Fig. 18—Skins used to finish cellulose FRC.
Fig. 19—Cellulose FRC pool area, finished with skins.

Fig. 20—Section of FRC finished with seamless skins.

Fig. 21—Cellulose FRC finished with decorative seamless skins.
Fig. 22—Scored cellulose FRC slab.

Fig. 23—Decorative FRC accent, scored to achieve fine detail.

Fig. 24—Cellulose FRC with acid stain and sealant coating.

Fig. 25—Cellulose FRC treated with acid stain and sealant coating.
Fig. 26—Exposed aggregate finish.

Fig. 27—Decorative cellulose FRC walkway in Breckenridge, CO.

Fig. 28—Note lack of cellulose fibers on the surface of walkway.

Fig. 29—Cellulose FRC pool deck with clean edges and no fiber clumping.
Fig. 30—Cellulose FRC pool deck with uniform scoring and no unstained areas.

Fig. 31—Cellulose FRC “cobblestone” driveway.

Fig. 32—Cellulose FRC “cobblestone” driveway.

Fig. 33—Decorative cellulose FRC entrance/walkway.
Fig. 34—Cellulose fiber-reinforced artistic DC element on patio.
Applications of Micro-Synthetic Fibers for Resistance to Explosive Spalling in Fires

by T. Atkinson and P. C. Tatnall

Synopsis: This paper discusses the recent tragedies of a number of tunnel fires occurring in transport tunnels, and the effects of these fires on concrete tunnel support linings. The mechanisms of explosive spalling of concrete in fires is described, and the research conducted to assess the ability of fine, polypropylene fibers to mitigate the effects of explosive spalling in severe fires is described. The test program to assess the fire resistance of the 25 miles (40 km) of concrete-lined tunnels in the Channel Tunnel Rail Link project in the United Kingdom is described in detail. A program to ascertain the effects of using these fibers in shotcrete tunnel linings is also considered. These programs demonstrate that small quantities – as little as 1.7 lb/yd³ (1 kg/m³) – of mono-filament polypropylene fibers provide resistance to explosive spalling in fires. Examples of the application of this new technology are listed.

Keywords: explosive spalling; fiber; fire; polypropylene fiber; shotcrete.
INTRODUCTION

Research has demonstrated the positive effects of micro-synthetic polypropylene fibers to the resistance of explosive spalling of concrete during fires. The phenomena of explosive spalling is particularly severe for hydrocarbon-fuelled fires such have occurred in a number of tunnels throughout the world causing loss of life and untold costs in repairs and disrupted commerce.

Hydrocarbon-fuelled fires are characterized by a very rapid temperature rise – ambient to 2200°F (1200°C) in a few minutes – unlike typical building fires which are characterized by much slower temperatures rises (ASTM E119 2006). Because of the rapid temperature increase of concrete exposed to these fires, moisture present in the concrete cannot move away from the advancing heat front fast enough, and the moisture turns to vapor. As the heat increases the pressure of the water vapor builds until it exceeds the tensile capability of the concrete resulting in an explosive release of pressure by violently dislodging pieces of the heated concrete surface. As pieces fly from the surface, a new surface is exposed to the fire resulting in progressive spalling.

Explosive spalling seems to be the combined effects of two mechanisms of pore pressure spalling and thermal stress spalling. Some of the factors that influence the amount and severity of explosive spalling include: heating rate, aggregate stability and thermal expansion factors, concrete permeability, moisture content and applied load. Concrete designed for “high performance” with low permeability may provide “low performance” in fire conditions (Khoury 2006).

This paper describes the testing and mechanisms for mitigation of explosive spalling of concrete in fires by the inclusion of small amounts of polypropylene micro-fibers. Examples of recent applications of this technology are also provided to help the practitioner take advantage of this relatively easy and inexpensive “insurance” against catastrophes in fires in our built infrastructure.

CONCRETE SPALLING

Concrete spalling can be described as the breaking off of layers or pieces of concrete from the surface of a structural element when exposed to fire. There are essentially three types of concrete spalling in fires:

Surface spalling

This type of spalling affects the aggregates and matrix at the concrete’s surface whereby pieces of concrete up to 0.75 in. (20 mm) size are gradually and non-violently dislodged from the surface. In this case of surface spalling, the degradation of the concrete is relatively slow and involves the dehydration of the cementitious matrix followed by a loss of bond between the matrix and aggregates. When the temperature rise of the concrete is relatively slow, the moisture in the concrete has time to migrate from the side exposed to the heat and pressure build-up is minimal. The presence of moisture in this case can actually mitigate the effects of temperature rise, since a great deal of energy is consumed in converting moisture to vapor.

Corner break-off

This type of spalling, also know as sloughing off, occurs at the edges and corners of concrete elements during the latter stages of the fire when the concrete has cracked and weakened.
Explosive spalling

This is unquestionably the most serious and indeed dangerous form of spalling, and normally occurs during the first 10 to 20 minutes of a fire when a rapid heat rise is encountered, as in a hydrocarbon-fuelled fire. In this case, moisture in the concrete is heated faster than it can move away from the heat. As the heat of the concrete increases, the moisture in the concrete changes to vapor. If it is unable to escape from the concrete mass, the vapor causes an increase in pore pressure. As this process continues, vapor pressure rapidly builds up until it exceeds the tensile capacity of the concrete causing pieces of concrete to be violently-explosively dislodged from the concrete (Tatnall 2002).

Other factors as described in the introduction also influence the amount and severity of explosive spalling. An important factor is the thermal expansion of the aggregates that may contribute substantially depending on this aggregate property.

FIRE CONSEQUENCES

Since the earliest known tunnel under the Euphrates River by the Babylonians in 2180 B.C., tunnel construction technology, methods and equipment have made gigantic improvements allowing construction of longer and bigger tunnels throughout the world. This has resulted from the growing demands of modern industrial societies for fast, efficient and reliable transportation systems, whether they are roads, railways or mass transit systems. The passage of time also means that fire hazards in tunnels are no longer limited to candles or torches used to light the Euphrates Tunnel. Heavy goods vehicles and trains carrying sufficient quantities of potentially dangerous materials capable of fuelling fires in the event of accidents now present major fire hazards. These hazards are set to increase with experts forecasting that freight traffic in Europe alone is set to increase by approximately 60 percent over the next thirty years (Haack 2004).

Unfortunately, we have already experienced the devastating consequences of these new fire hazards since over the last decade there have been a number of fires in tunnels which have caused significant loss of life and structural damage. The human casualties experienced in the Mont Blanc, Tauren, Kaprun and Gotthard tunnels have prompted significant debate and changed perceptions of tunnel fires worldwide. These fires were hydrocarbon fuelled (Thomas 2000).

Apart from the tragic loss of life, the damage sustained to the tunnel structures can cause major disruptions and major repair costs. Table 1 shows some of the immediate damage from selected recent tunnel fires. Reports suggest that the financial cost from direct damage and lost revenue of the Channel Tunnel fire was in itself in the order of U.S. $300 million (MIT 1999).

Figure 1 illustrates the damage to the 20-inch (51 cm) thick precast segmental lining of the Channel Tunnel from the 10-hour train fire in 1996, which was estimated to reach temperatures of 1290°F (700°C) (Civil Engineering 1999). Figure 2 shows the two burning trucks that collided 1 mile (1.5 km) from the Italian portal of the 10.5 mile long (16.9 km) St. Gotthard Tunnel. One of the trucks was carrying tires and tarpaulins, and the temperatures in the tunnel were estimated to reach 1830°F (1000°C) causing the roof to collapse (World Tunnelling 2001). The Mont Blanc Tunnel fire in 1999 resulted in closure of this 7.4 mile (12 km) long tunnel between France and Italy for 3 years. Repairs and upgrades amounted to U.S. $273 million, while the Italian government estimated a loss in trade of over U.S. $2 billion. (Tunnels & Tunnelling International, April and June 2002).

Owners and designers of infrastructure projects are to soon recognize the advantage of the use of synthetic fiber reinforced concrete in high performance structures of all types, such as protection for highway bridges from hydrocarbon-fuelled fires as occurred under the Texas Highway 310 bridge over U.S. Highway 175 near Dallas on October 12, 2008, Fig. 7. The driver carrying 7,000 gals (26,000 l) of gasoline took the curve too fast, blew a tire and turned over causing the fire. The Texas Department of Transportation determined both north- and south-bound lanes required replacement because of the extensive damage due to the fire and heat. (The Dallas Morning News 2008) The bridge remained out of service for 3 months.

Some recent tunnel applications incorporating this technology utilizing micro-synthetic fiber-reinforced concrete include:

- Channel Tunnel Rail Link-United Kingdom;
- Dublin Port Tunnel-Ireland;
- Gotthard Base Tunnel-Switzerland;
Atkinson and Tatnall

- T5 Heathrow Express Tunnel-United Kingdom;
- Weehawken Tunnel-United States;
- Paramatta/Chatswood Tunnel-Australia;
- Vomp-Terfens Tunnel-Austria;
- Fritzen Tunnel-Austria;
- Brixlegg Tunnel-Austria;
- Devil’s Slide Tunnel-United States; and
- Second Avenue Tunnel-United States.

CTRL FIRE TESTING

In 1997, when Rail Link Engineering (RLE) initiated their fire testing program for construction of the Channel Tunnel Rail Link (CTRL) project, they expected to use 265,000 high-performance precast concrete segments for lining 24.7 miles (39.5 km) of bored tunnels on this 68 mile (109 km) high speed rail link from London to the Channel Tunnel. The segments, consisting of 429,000 yd³ (328,000 m³) of concrete, were designed using steel fibers for reinforcement rather than the typical reinforcing-bar cages. Durability of the segments for the 120-year design life was a major concern as well as the robustness of the segments for handling and erection, and influenced the decision to use steel fibers (Shuttleworth 2000, 2003).

RLE found that there were no standard test methods to assess the serviceability of concrete precast segmental tunnel liners in fire situations, particularly those using steel fibers. Recent research (Sullivan et al. 1996; Bilodeau et al. 1995) indicated that including small amounts of fine mono-filament polypropylene (PP) fibers provided resistance to explosive spalling in fires. RLE proposed the use of PP fibers in the segments, and required a test methodology for ascertaining the performance of high strength, low permeability concrete with regard to explosive spalling and to measure thermal and mechanical properties during and after the exposure to fire.

Various time-temperature curves exist that represent different fire scenarios, and three commonly used curves were adopted for the CTRL trials, which increased in severity as the proposed options were narrowed down. Figure 3 shows the three curves used by RLE for testing. The Eurocode I, and RWS curves are considered to represent hydrocarbon fuelled fires.

Initial testing was conducted using the ISO 834 cellulose fire curve in mixtures with limestone, granite and lightweight aggregates, and using steel and both mono-filament and fibrillated micro-synthetic PP fibers. For each case tested, when 1.7 to 3.4 lb/yd³ (1 to 2 kg/m³) of mono-filament PP fiber was added, no explosive spalling was observed after two hours in the furnace. Results indicated that lightweight aggregates provided extensive spalling, and fibrillated and steel fibers provided little protection from explosive spalling.

Based on these results, granite aggregate concrete with mono-filament PP fibers at 1.7 and 3.4 lb/yd³ (1 and 2 kg/m³) were tested along with non-fibrous specimens using the more severe Eurocode I fire time-temperature curve (Figure 3). Specimens were loaded in compression to simulate the design loadings of the tunnel liners. The fibrous specimens contained PP fibers with diameters of 0.0007 and 0.0013 in. (18 and 32 µm). All the specimens containing PP fibers showed no evidence of explosive spalling, while all the plain specimens exhibited extensive spalling damage, which occurred in the first 20 minutes of the test. These results confirmed the results achieved in the initial testing.

Finally, in order to provide final confirmation of the fire resistant properties of the proposed CTRL precast tunnel segment concrete mixtures, a single fire test was carried out at the TNO Fire Research Centre in the Netherlands using their Rijkswaterstaat (RWS) fire time-temperature curve (Figure 3). Three full-scale segments with different mixtures were simultaneously exposed to this very severe fire scenario. The mixtures were designed for 8,700 psi (60 MPa) compressive strength with 738 lb/yd³ (438 kg/m³) Portland cement, 0.35 water/cement ratio, 50 lb/yd³ (30 kg/m³) steel fibers, and 1.7 lb/yd³ (1 kg/m³) mono-filament PP fibers. Two mixtures used granite aggregate – one with the smaller PP fiber and one with the larger. A third mixture was similar using limestone aggregate.

Upon completion of this third phase of testing and evaluation of the entire program, RLE was able to draw the following conclusions related to the use of fibers:

a. The inclusion of 1.7 lb/yd³ (1 kg/m³) of monofilament polypropylene fibers in the high strength, low permeability mixtures tested significantly reduced the risk of spalling when exposed to severe hydrocarbon-fuelled fires.
b. There was no difference in the performance between the 0.0013 in. (32 µm) and the 0.0007 in. (18 µm) fibers. Therefore both sizes were approved for use in the tunnel lining segments for the Channel Tunnel Rail Link project.

c. Steel fibers did not contribute to the ability of these concretes to resist explosive spalling when used without polypropylene fibers.

### SHOTCRETE FIRE TESTS

Tatnall (2002) describes testing steel fiber reinforced shotcrete using the RWS time-temperature curve at the Versuchsstollen Hagerbach AG test facility in Switzerland. Since more and more steel fiber reinforced shotcrete is being used for final tunnel linings it was important to assess the performance of mono-filament polypropylene fibers in resisting explosive spalling of shotcrete in fires. Figure 4 shows the 6 inch (150 mm) thick test panel on top of the furnace at Hagerbach. Figure 5 shows a test specimen containing typical welded wire reinforcing used in shotcrete tunnel linings after 15 minutes of testing. One-half the thickness – 3 in. (75 mm) – had explosively spalled off the panel in this short time. Figure 6 shows a steel fiber reinforced shotcrete panel containing 66 lb/yd³ (40 kg/m³) of an ASTM A 820, Type I, deformed steel fiber, plus 3 lb/yd³ (1.8 kg/m³) of a ½-inch (12 mm) long, 0.0013 in. (32 µm) diameter mono-filament PP fiber after a full 2 hours on the test furnace. Other tests of panels containing just steel fibers or with just macro-synthetic fibers showed extensive explosive spalling after a short time on the test furnace.

### DISCUSSION

Sullivan et al. (1996-1997) showed that the use of small quantities, 1.7 to 3.4 lb/yd³ (1 to 2 kg/m³), of micro polypropylene monofilament fibers, approximately 0.0012 in. (30 µm), in concrete can mitigate the effects of explosive spalling due to hydrocarbon-fueled fires, and from the Hagerbach shotcrete testing it is shown that these fiber sizes and quantities are effective in shotcrete as well. In addition to the vaporization and melting of the fibers at approximately 330°F (165°C) which provides channels for the escape of the building pore vapor pressure, the very different thermal characteristics of the polypropylene fibers and the concrete matrix cause small fissures to open at the fiber-matrix interface as the polypropylene expands at 8.5 times the rate that the matrix does. In addition to micro-cracking at the fiber-matrix interface the stresses caused by the expanding polypropylene in the matrix cause micro-cracking in the matrix as well. All these small fissures form networks to increase permeability and allow escape of building moisture vapors at even lower temperatures.

The Hagerbach and RLE testing showed that macro fibers – diameter greater than 0.028 in. (0.7 mm), both steel and polypropylene, did not provide resistance to explosive spalling. This is perhaps because of the difference in the number of fibers present in the matrix. Micro-polypropylene fibers supply more than 53 million fibers per pound (114 million fibers per kilogram) versus only 9,000 to 13,000 fibers per pound (20,000 to 30,000 fibers per kilogram) for the steel and macro-synthetic fibers used respectively. With much lower numbers of macro fibers expansive forces cannot create the crack networks required to allow escape of steam.

### CONCLUSIONS

The RLE testing and the shotcrete tests described confirmed that concrete and shotcrete containing ½-inch (12 mm) long mono-filament polypropylene fibers smaller than 0.0013 in. (33 µm) diameter will, indeed, provide explosive spalling resistance in very high-temperature-rise fires typical of hydrocarbon-fueled fires experienced in transportation tunnels. As a result of these tests and others the use of PP fibers for this application is growing worldwide. For the relatively low cost of adding polypropylene fibers for fire resistance – the order of 1 percent of the cost of constructed concrete lining – it is likely that this fiber application in tunnels will continue to grow.

### REFERENCES


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“Researchers Explain Concrete Damage in Tunnel Fires,” 1999, Civil Engineering, American Society of Civil Engineers, Washington, DC, July, p. 24

<table>
<thead>
<tr>
<th>Year</th>
<th>Tunnel Type</th>
<th>Length (m)</th>
<th>Fire Duration</th>
<th>Deaths</th>
<th>Material Loss</th>
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<tr>
<td>1994</td>
<td>GreatBelt Denmark Rail 8000</td>
<td>7 hrs</td>
<td>None</td>
<td>16 segment rings (1.65 m long) damaged in crown during construction.</td>
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<tr>
<td>1996</td>
<td>Channel UK-France Rail 50,500</td>
<td>9 hrs</td>
<td>None</td>
<td>500m of structural damage, rolling stock damaged, major repair costs and operational losses.</td>
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<tr>
<td>1999</td>
<td>Mont Blanc France-Italy Road 11,600</td>
<td>50 hrs</td>
<td>39</td>
<td>900 m of tunnel structure severely damaged. 23 lorries and 10 cars burnt.</td>
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</tr>
<tr>
<td>1999</td>
<td>Tauern Austria Road 6400</td>
<td>17 hrs</td>
<td>12</td>
<td>16 lorries and 24 cars destroyed. Concrete spalled 10-15 cm. Toll losses 6.5m Euro/month</td>
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<tr>
<td>2000</td>
<td>Kaprun Austria Rail 3300</td>
<td>1 - 2 hrs</td>
<td>159</td>
<td>Train’s base all that remained after 1000°C fire.</td>
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<tr>
<td>2001</td>
<td>Gotthard Switzerland Road 17,000</td>
<td>24 hrs</td>
<td>11</td>
<td>200m false ceiling damaged.</td>
<td></td>
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<tr>
<td>2003</td>
<td>Daegu South Korea Rail 4000</td>
<td>3 hrs</td>
<td>124</td>
<td>6 train carriages destroyed. Tunnel ceiling severely damaged exposing 2 layers of reinforcement</td>
<td></td>
</tr>
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Note: 1 cm = 0.394 in.; 1 m = 3.281 ft; 1°F = 5/9 × °C.
Fig. 1—Fire damage to precast concrete segmental lining in the Channel Tunnel.

Fig. 2—Truck collision fire in the St. Gotthard tunnel.
Fig. 3—Standard fire time-temperature curves used for CTRL tests. (Note: $1^\circ F = \frac{5}{9} \times ^\circ C$)

Fig. 4—Furnace set-up for RWS curve testing at Hagerbach Test Tunnel.
Fig. 5—Shotcrete test panel without fibers after 15 minutes.

Fig. 6—Shotcrete test panel with steel and polypropylene fibers after 2 hours.
Fig. 7—Dallas, TX, fuel tanker fire under Highway 310 bridge.
Jointless Steel Fiber-Reinforced Concrete
Slabs-on-Grade and on Piles

by E. Alexandre and B. Bouhon

Synopsis: Most maintenance problems associated with industrial concrete floors result from the joints. This paper emphasizes a method to eliminate saw-cut joints in slabs-on-grade by the use of steel fiber reinforced concrete (SFRC) only. The performance of the composite material is directly linked with the choice of a specific concrete mix design and an improved technique to uniformly mix a high dosage of steel fibers. Tests and experience have shown that high level post-cracking ductility of the SFRC can control micro-cracking caused by flexural and shear stresses combined with restrained shrinkage. The proposed design approach, based on the yield-line theory, gives an objective view of the safety factor in relation to the ultimate state. Case studies demonstrate that typical areas of 25,000 ft² (2322 m²), without saw-cut joints, are regularly achieved by experienced contractors with relevant site quality control. Practical site aspects such as armored contraction joints, slab details, aspect ratio, installation techniques etc., are an integral part of the case study as well. The second part of this paper details the use of this technique for structural applications such as suspended slabs on piles and mat foundations. To demonstrate the structural capacity of concrete solely reinforced with a high dosage of steel fibers, real scale tests and practical case studies are presented.

Keywords: ductility; jointless slab; shrinkage; slab-on-grade; slab-on-piles; steel fiber-reinforced concrete; toughness; yield line.
INTRODUCTION

In the early 1970s, steel fibers started to be used as a replacement to wire mesh in slabs-on-grade. Concrete contractors rapidly realized the advantages of using composite material to increase their productivity. With the practical use of SFRC, other advantages came to light, such as the enhancement of the flexural toughness, of the abrasion, impact and fatigue resistance. Generally, steel fibers are simply viewed as an alternative to conventionally reinforced concrete, and even though saw-cut joint spacing can be increased, the inherent problems of those induced joints remain. There is an evident contradiction as these joints are induced to force the shrinkage contraction in a specific straight line, but are definitely the main cause of problems in the use of the floor, this time due to differential shrinkage.

In comparison to concrete beams or columns, the drying shrinkage of concrete in slabs-on-grade differs totally between the upper and lower portions, creating significant edge and corner curling [2]. When aggregate interlock is not controlled, joints will be subjected to rocking, and repeated traffic will deteriorate concrete (Fig. 1). Maintaining or repairing such joints is difficult and disruptive to the floor use.

Traditionally, slabs-on-grade are designed on the basis of their flexural or shear bearing capacity. The differential shrinkage is generally not considered, but is compensated by over-designing. Ironically, most problems in slabs come from shrinkage and not from design capacity even under high static loads.

CONCRETE SHRINKAGE IN SLABS-ON-C GRADE

Shrinkage is a natural characteristic of concrete

The concrete shrinkage does occur in three steps: the first one is happening during the setting of concrete, when cement particles are chemically transformed by the hydration process; the new cement stone has a reduced specific volume that will induce the initial shrinkage strain. The second step is happening during cooling down of the concrete paste after exothermic hydration, with a consecutive thermal contraction. These first two steps called early shrinkage occur in the first initial hours when the concrete starts hardening.

Thereafter, a part of the excess water needed to reach a workable concrete, will migrate to the upper surface which is in contact with the environment. This long term shrinkage caused by water diffusion through concrete capillaries, varies in function of the square root of time and is affected by every change within the external environment.

Concrete strength reaches its peak after approximately 28 days, and therefore does not follow the same progression with time that shrinkage does. There are two critical periods for the concrete slab if shrinkage is restrained: during early shrinkage when concrete has not reached sufficient tensile strength, and after the final setting of the concrete when strain is caused by long term shrinkage and environmental instabilities (Fig. 2).

Among numerous factors acting on the concrete shrinkage, one can mention cement type and dosage, water content, aggregate size, external moisture, slab thickness etc. While some of these factors can be controlled, it is not possible until now to eliminate the source of shrinkage. We can only try to reduce the amount of strain, by working on the different parameters, or by using specific shrinkage compensating agents.

Shrinkage is the main cause of problems in slabs

Slabs are one of the most shrinkage-sensitive concrete structures, as their surface in contact with the
environment is very large in comparison with the volume of concrete. The internal moisture variation depends on the conditions of the interface with the supporting base and the humidity/temperature of the air in contact with the slab surface. The shrinkage process will progress from the upper surface, while the lower surface can remain rather saturated at the early age of the concrete. The strain distribution curve progresses with time showing a maximum slope in an intermediate stage (Fig. 3).

The shrinkage distribution through the slab depth is a non-linear function of the time that can be split up in three parts, respectively a constant part, a triangular linear distribution part and a self-compensated non-linear part (Fig. 4). The mean shrinkage strain \( \varepsilon' \) creates the contraction of the slab panels. A period of minimum 12 months is necessary to notice the stabilization of this movement, mainly due to the friction on the sub-base and the creep of the concrete in tension. The differential linear shrinkage \( \varepsilon'' \) induces a bending of the slab, when it is not counter-balanced with the self-weight of the slab and the imposed load. When edges and corners are not sufficiently supported, they can lift, up to an excess of an inch, a phenomenon called “curling” or “warping” (Fig. 5).

The rocking movement of the unsupported edges and corners is the cause of most damages in concrete floors when they are subjected to traffic (Fig. 6). The influence of the self-compensated strain “\( \varepsilon'' \)” does not cause any major visible effect within the slab.

**Technical solution to shrinkage**

As discussed above, shrinkage can not be avoided completely, but its effects can be minimized by the use of a specific concrete mix design, or by reducing or controlling the stresses induced by restrained friction. It is crucial to improve the support of all edges and corners, by means of correct load transfer systems.

Reducing concrete shrinkage can be achieved with shrinkage-compensating agents or swelling materials. The chemical reaction mostly affects the critical early-age strain of the cement paste, and relies on the concrete strength to resist the long term drying shrinkage. Some other research leads to the development of shrinkage-reducing admixtures by changing the surface tension of the liquids towards the surface of concrete capillaries.

Post-tensioning of concrete slabs has been widely used to compress the concrete slab and reduce the tensile stress created by the friction-restrained movements.

Another approach has been widely used in road construction by ensuring a continuous reinforcement of the concrete with a high quantity of conventional steel bars. Crack control design requires a minimum steel section of 0.5% of the concrete section, i.e., approximately 130 lb/yd\(^3\) (77 kg/m\(^3\)) for a two-way reinforcing effect. Crack spacing and opening can be optimized with a smaller bar size and smaller spacing.

The alternative discussed in this paper is brought by the SFRC composite and its homogeneous micro-reinforcement. A dense network of 3D-orientated fibers is able to create some micro-cracking tension release. High quantities of long fibers are requested to give a bridging effect of the cracks. Post-cracking toughness of the fiber reinforcement will have to keep the structural capacity of the slab under imposed loads with sufficient safety factors.

The key-aspect of SFRC material in large applications on site is obviously the quality control in order to ensure the even-mixing of the fibers in the concrete at very high dosages.

Cold-drawn metallic fibers have been extensively used in that field since the 1980s. The tensile strength of the steel can be increased up to more than 150 ksi (1,034 MPa) by the production process, and the shape can be optimized to improve the anchorage of the steel wire to the cement paste. In order to keep a practical dosage of typically 59 to 75 lb/yd\(^3\) (35 to 45 kg/m\(^3\)), the aspect ratio of the fiber must be correctly chosen to optimize the number of fibers per volume.

**USE OF STEEL FIBER AT HIGH DOSAGE**

**Fiber type and dosage**

As discussed above, the toughness of SFRC is directly linked with the type of fiber used. Basically, to correctly bridge any crack induced in the cement paste, tensile stresses must be transferred to the reinforcing wires which need sufficient strength and stiffness.

The type of fiber considered in this paper is a cold drawn wire with a crimped shape and an aspect ratio of 50 to 60 (Fig. 7) During the process of cold drawing from raw machine wire down to a diameter of 0.039 in. (1 mm), the tensile strength of the steel is increased up to a minimum of 150 to 210 ksi (1034 to 1448 MPa).
92 Alexandre and Bouhon

The wire is deformed by continuous undulations with amplitude of 3/8 in. (10 mm) and wave depth of 0.026 in. (0.65 mm), as described in the U.S. Patent 4,585,487 [7] with foreign priority date Dec 30, 1983. The anchorage is relying on the material stiffness and on its plastic deformation in bending in both directions within the concrete matrix. Bonding of steel to the cement matrix is not the major anchorage factor, and it is unlikely to see fibers breaking even for very large deformation as simulated in laboratory testing.

In order to optimize the reinforcing effect of fibers, one should increase the density of fibers, with sufficient anchorage length. As the theoretical distance between fibers decreases with increasing dosage, there is a limit to the number of fibers that can be evenly mixed in function of the aggregate size. Romualdi and Mandel [6] proposed the calculation of this distance (Eq. (1)).

\[
s = 158 \frac{d}{\sqrt{V_f}}
\]

where \(s\), \(d\) and \(V_f\) are respectively the distance between fibers (in inch), the diameter of the wire (in inches) and the fiber concentration (in lb/yd³).

Consequently, there is a balance to find between the fiber dosage and the concrete mix design, especially in terms of maximum aggregate size and cement paste as the shrinkage is a critical figure. Typical dosages of 59 to 75 lb/yd³ (35 to 45 kg/m³) have been widely used over a period of 20 years, with minor changes to the current concrete recipes. Higher dosage can be proposed with other kind of fibers with larger diameter for example.

Fiber distribution

Although the toughness depends on the fiber type and the dosage rate, it is evident that this is conditioned by the correct distribution of fibers in the concrete. The continuous aspect of any composite material requires a three-dimensional dispersion of the reinforcing component, in order to guarantee the same behavior in every direction with an even probability on the number of fibers crossing in a random section of the concrete structure.

The mixing becomes very delicate with high dosage of long fibers, to avoid the formation of “dry balls” (groups of imbricated fibers in which cement paste can not enter) or “wet balls” (bonding of a group of fibers with fine cement paste without aggregates during mixing). Once the correct dispersion has been reached, there is always a risk of segregation during transportation of the fresh concrete.

For short fibers at low dosage, the method of dropping the fiber box content directly into the hopper of the concrete truck can be used successfully, with a preference to mix the fibers with dry concrete before adding cement and water. For long fibers at high dosage, it is necessary to separate the fibers before adding to the concrete, which can be done on site with specifically designed pneumatic blasting machines (Fig. 8). This technique has different advantages: the control of concrete quality and workability before mixing the fibers, the speed of blowing the fibers all over the surface of the fresh concrete in the drum, and the ability of working on site with limited transport risks.

Another technique is to add some collated fibers that spread through the fresh concrete and separate with the force of mixing.

Specific SFRC mix designs

In addition to the maximum aggregate size as mentioned before, the concrete mixture must be specifically designed for steel fibers and for the application in industrial floors. To avoid balling of fibers, it is essential to respect a continuous grading curve of all aggregates. This basic requirement of the mix design is more important when high dosage of long fiber elements have to be mixed in a combination of spherical shapes. This is the reason why SFRC quality is usually improved in comparison to most common mixes.

The content of finer raw materials in the concrete mixture has to be correctly balanced in order to give a good anchoring paste to the fibers, to allow the correct finish-ability of the concrete and to avoid unnecessary water causing extra shrinkage.

Before mixing the fibers, the concrete must reach sufficient workability, which requires the use of a water reducing admixture to keep the effective water-cement ratio in an acceptable range of 0.48 to 0.55.
An example of acceptable concrete mix design is given in the case study of Interbake Foods in Front Royal VA at the end of this paper.

The environment and placement conditions are other very important issues, as concrete needs to harden evenly without cold joints and to allow the correct finishing operation of the slab surface. Mainly cement choice and quantity must be adjusted with the conditions of external temperature while controlling the aggregates.

**Behavior of SFRC**

With the random volume distribution of fibers and their narrow spacing, any possible cracking mode can be controlled. At the difference of conventional reinforcement, there is no unreinforced paste to cover the steel bars. Also, some stresses like punching shear will lose their critical aspect.

The post cracking ductility is commonly assessed on small beam test as specified in ASTM C1018 [3] or any equivalent standard. This type of test only simulates the flexural mode of rupture on small prismatic specimen, while we know that the design of concrete slab is much more complex.

If we consider the SFRC behavior more specifically towards the application in industrial floors that are mainly subjected to static point loads and traffic loads, we can summarize the advantages of using fibers as follows:

*Flexural toughness*—Flexural toughness tests on beams and slabs show that SFRC is able to hold considerable loads up to very large deflections. The toughness or ductility indexes give the ratio between effective dissipated energy and theoretical energy of a perfectly elasto-plastic material. The aim of using high dosage of steel fibers is to create a plastic or hardening effect of this post-cracking behavior. Typical beam tests are not very representative, due to the dispersion of the results, the size of the samples and the influence of the testing equipment stiffness, but they can be easily interpreted as the whole bending moment is concentrated in a single flexural crack. Circular plates as specified in ASTM C1550 [4] or even larger slabs are not easy to handle, but they lead to a better understanding of the real mode of failure of a concrete slab. The interpretation following the yield line theory gives the importance of the ductility in all cracks, without information on the complete stress-strain curve.

*Shear*—The shear rupture is very brittle in plain concrete or lightly reinforced structures, similarly to the tensile rupture of the concrete. Steel fibers do have the same occurrence in a shear crack than in a bending crack, leading to a similar ductile comportment in both modes of rupture. High dosage of fibers can increase the shear capacity by 50% and more [14].

*Impact and fatigue*—Despite the difficulty to evaluate these properties in laboratory, the performance of SFRC is known to be considerably improved under dynamic and cyclic loading. Effectively, the continuous micro-reinforcement of the composite will dissipate high level of energy under impact, and the material toughness will allow important deformations without failure.

*Shrinkage*—Real scale tests have shown that the amount of shrinkage can decrease of 10-15% with high dosage of steel fibers. However, the main effect of the fibers will be to release the shrinkage-induced tensile stresses. Considering the small fiber spacing, the tension crack release will lead to a narrow grid of micro-cracks that are not detrimental to the use of the floor and are hardly noticeable.

**Design approach**

A slab-on-grade is, by definition, directly supported by the sub-base material taken as an elastic continuous spring. Its role is to transfer the loads from the upper surface to its support, and to follow the deformations induced in the subgrade.

Under static concentrated loads, the stresses are caused by a combination of sagging moment and punching shear. Usually, this does not cause any problem in a floor, while the hogging moment in unloaded area could be more critical when subgrade conditions are not compatible with the forces applied.

Even though they have a lower value, the dynamic wheel loads from the material handling equipment can cause more problems, as they can move everywhere onto the slab surface, and specifically at the edges and corners created by joints in the floor, where differential shrinkage does lift the slab from its support.

Most design methods are conservatively based on an elastic approach of the materials and support. The concrete is considered un-cracked with an increased modulus of rupture. However, we know that, for many reasons, concrete can crack otherwise there would be no need for any reinforcement. Cracks
are usually nonstructural and often appear due to poor detailing, at specific locations such as re-entrant corners, loading docks, pits, overhead doors etc. Taking the above into account, most floors can be considered as over-designed, except in a number of un-designed locations (such as re-entrant corners, lifted edges) which can be found anywhere on the floor surface.

Therefore, other design approach based on the yield line theory [1] can be proposed at the condition that the material has elasto-plastic comportment, for example: it is effectively reinforced with high dosage of performing fibers. As per structural design of concrete, safety factors are effectively applied on loads and materials, giving a minimum overall safety factor of 1.8 to 2.25. Furthermore, this methodology considers the concrete as cracked with its effective post-cracking capacity.

In practice for a point load, the ultimate load is directly given from the slab properties (thickness, concrete class and ductility brought by the fibers) and the contact surface of the load.

\[
P_u = 2\pi(M_p + M_n) \quad \text{when} \ a/l = 0 \quad (2)
\]

\[
P_u = 4\pi(M_p + M_n)(1 - a/3l) \quad \text{when} \ a/l = 0.2 \quad (3)
\]

where \( P_u \) is the ultimate load, \( M_p \) and \( M_n \) are the ultimate sagging and hogging bending moment of SFRC, \( a \) is the equivalent radius of the load and \( l \) is the elastic length (relative stiffness of the slab with regards to the spring support). The value of \( P_u \) is interpolated between both expressions (2) and (3) when \( a/l \) is lower than 0.2. The radius of the load is calculated for a single load or an equivalent value for multiple loads.

Similar expressions can be used to check for an edge or a corner loading condition.

With this design approach, it is likely that slab thickness will decrease in comparison with traditional plain or lightly reinforced concrete. Indeed, the design thickness can be optimized because of the elimination of typical weak point in slabs (unsupported corners and edges).

**JOINTLESS SFRC SLABS-ON-GRADE**

**Historical background**

Originally, steel fibers have been used at low dosage in industrial concrete slab-on-grade as a replacement to wire mesh, typically 33 to 42 lb/yd\(^3\) (20 to 25 kg/m\(^3\)). With the improvement of the mixing procedure and a better knowledge of the concrete mix design, steel fibers have really shown their performance to reinforce the concrete slabs, especially with regard to crack control.

In a US Patent 4,640,648 [8] with foreign priority date of March 10, 1983, X. Destrée and A. Lazzari outline their invention of making joint-free steel fiber reinforced concrete floors of bay sizes up to 35,000 ft\(^2\) (3250 m\(^2\)), with steel fiber dosage from 50 lb/yd\(^3\) (30 kg/m\(^3\)) and higher.

Since then typical jointless panel areas of 15,000 to 25,000 ft\(^2\) (1393 to 2322 m\(^2\)) have been regularly achieved, and today, the limit of joint spacing is usually imposed by the quantity of concrete that can be delivered and placed in the same day, and the opening of the construction joints that concentrate all the shrinkage movements.

To our knowledge, the oldest jointless SFRC slab was installed in Breda, The Netherlands in 1983, with a panel size of 25,000 ft\(^2\) (2322 m\(^2\)) and still performing to client’s satisfaction. One can estimate that nowadays more than 50 million ft\(^2\) (4.6 million m\(^2\)) of jointless SFRC slabs are installed yearly worldwide.

**Key Aspects**

As outlined in Ref. 8-9, the process of building large jointless panels remains completely dependent of practical aspects covering fiber mixing, concrete quality control, base preparation, detailing, installation and site conditions.

*Free to move*—any restraint of the concrete shrinkage can create tensile stresses in excess of the concrete strength, and therefore lead to cracking. Restraints can be caused by sub-base irregularities and friction, all fixed obstructions and structural tie bars. Fine grading and controlled levelness of the sub-base are essential conditions for jointless slabs. A polythene slip membrane is recommended to smooth the support roughness and to avoid the bonding of the cement paste with sub-base materials.

Compressible foam must be placed around all fixed obstructions such as building columns, manholes, edge of doors, with a particular attention to cover the entire slab depth. Each area presenting some risk
of cracking has to be isolated and reinforced with deformed reinforcing bar, or additional wire mesh. Tying of the slab should generally be avoided.

Construction joints—when eliminating all saw-cut joints, there will still be a requirement for construction or day joints that will therefore take more movement. Being the only trafficked joints, these construction/day joints need to have essential qualities, such as: being free to move horizontally in both directions with the lowest vertical differential movement, having steel arris protection, having excellent load transfer capacities. Their load transfer capacity is also very important to decrease the deflection and stresses caused by unsupported edge condition.

In comparison to traditional dowel systems which are not suitable for two-directional movements, some proprietary systems have been developed with key-shape or flat plate dowels [Fig. 9]. They usually allow for opening in the order of 1/3 to 2/3 in (8 to 16 mm).

Joint layout is defined in function of the daily productivity, keeping in mind the aspect ratio of panels to maximum 3 by 2, avoiding important re-entrant corners and isolating from large restraint areas such as loading docks.

Site quality control—quality control before and throughout the concrete pour is essential to the production of SFRC jointless slab. Before pouring the slab, sub-base level check, as well as accurate level check of all pre-fixed formworks (dock levelers, manholes, construction joint etc.) need to be carried out, as well as a thorough inspection of all site conditions (weather tightness, access, lights etc.). During the pour, the site quality controls specific to a jointless SFRC slab are mainly focused on the concrete mix design quality and consistency, and the uniform distribution of steel fibers.

Skilled labor and HT equipment—supervision is always required; however the training, experience and motivation of the labor force remain the keys to the correct installation of a jointless concrete floor. Specifically, the mixing of high dosage of long steel fibers requires a specialized operator to handle the fiber blast-machine (see fig.7) and check the concrete quality and consistency. When applicable, other high-tech equipment, such as the Laser Screed and Mechanical Spreader are ideal for achieving jointless SFRC slabs.

Site conditions—in accordance with good working practice, it is crucial that the casting and curing works are carried out within a controlled environment. This will require the roof sheeting and cladding works to all elevations (permanent or temporary) to be complete, together with temporary sheeting to openings to be in place prior to casting.

**STRUCTURAL SFRC APPLICATIONS**

Historical background
 Extensive laboratory researches have demonstrated the different structural performances of the SFRC material with regard to bending, shear, impact, fatigue etc.

Knowing the success in the application for slabs-on-grade, it was natural to extend its use to a different kind of large two-dimensional applications with higher need for structural performance.

The first field was slabs supported on piles, aiming to increase the speed of production using the same industrial equipment to lay the floor, such as the Laser Screed and the Mechanical Spreader.

The initial experience is summarized by X. Destrée in 1995 [10] including a list of 800,000 ft² (74,000 m²) SFRC suspended slabs on piles, without mesh or rebar. The latest ACI 544-3R, “Guide for Specifying, Proportioning, and Production of Fibers Reinforced Concrete” includes in 3.1-Typical uses of fiber reinforced concrete-steel fiber only reinforced concrete suspended slabs on piles having a span to depth ratio up to 20.

Based on a full scale trial and the first applications in Belgium and in The Netherlands in 1994, the system has proved its efficiency and was officially approved by the Dutch Authorities in 1999 with a proposed method of design (COB, Staalvezelbetonvloeren op palen, 1999 [5]). England has rapidly adopted the product for many new industrial units that are built on poor quality substrates. In Ref. 11 of the year 2000, X. Destrée outlines a design method based upon the yield lines of Johansen.

Other structural uses of the SFRC like raft foundations and elevated slabs [12], have been carried out for a large range of applications such as water treatment plants, condominium foundations, clad-rack foundations, parking and office floors.

**Slabs on piles**
 The slab is supported on piles cast or driven into the ground, down to a sufficient bearing layer.
The concrete is cast directly onto the pile heads and the sub-base which will only act as a temporary working platform. The slab is designed to span between the piles that are ideally laid on a square grid.

The concrete is reinforced with a very high dosage of steel fibers, ranging from 67 to 100 lb/yd³ (40 to 60 kg/m³), and could be combined with conventional reinforcement between piles or above pile heads only.

In order to optimize the productivity, the preferred option will be to use steel fibers only, and work with automatic screeding equipments and without the need of a pump. In this case where the material is homogeneous, the high stress positions have to be designed with extra thickness, or by stress reduction system such as enlarged pile heads. Again, for practical and quality reasons, the preferred option will be a flat slab with well designed pile grid and pile heads (Fig. 10).

Raft foundations
The slab is still supported by the ground, but the importance of the structural loads or the lack of bearing capacity requires a design for a thick raft foundation. Commonly, the design is governed by the slab deformations and the punching shear under concentrated loads.

The concrete is usually reinforced with the minimal steel section for shrinkage control that can be completely replaced by the correct dosage of steel fibers. The particular case of water-tight structures may still require a combination with traditional reinforcement.

In very thick applications (more than 2-3 ft [610-914 mm] thick), the continuous reinforcing effect of the steel fibers is particularly helpful for controlling the thermal stresses caused by the exothermic reaction of the concrete.

Elevated floors
A European Patent application EP 1 544 181 A1 on Dec. 16, 2003 by X. Destrée, outlines the application of “only steel fibre reinforced concrete free suspended elevated slabs” of span to depth ratio up to 35 with high dosage rates of 130 to 200 lb/yd³ (77 to 120 kg/m³). The invention is used in several European countries since 2005. X. Destrée outlines testing, design, specification and installation in Ref. 13.

In similar application, and in order to decrease the formwork operation, the SFRC at minimum 50 lb/yd³ (30 kg/m³) dosage rate can be combined with steel decking systems, with a limited spanning capacity and a lower steel fiber dosage rate, but with a proved advantage for fire rating.

CASE STUDIES

Interbake Foods, Front Royal, VA
The building, a warehouse for packaged food products, is 722 ft long x 224 ft wide (220 m x 68 m). It is divided into ten jointless slab panels of 144 ft x 112 ft (44 m x 34 m) keeping an aspect ratio of 1.285 (aspect ratio Length/Width should not exceed 1.5) (Fig. 11). Each jointless panels are delimited by load transfer metallic joints, allowing horizontal movement in both directions with excellent arris protection thanks to the two 1/5 in. (5 mm) steel plates (Fig. 9). The dock levels are isolated from the main slab, to allow the free movement, by using the same metallic joint installed 3 ft (914 mm) away from the docks.

Design—the SFRC slab has a thickness of 6 in. and is reinforced with 67 lb/yd³ (40 kg/m³) of steel fibers. The sub-base Westergaard modulus of reaction is specified with a minimum value of 100 pci (27 MPa/m). The slab is specified to sustain racking leg load of 8,000 lb (35 kN) on base plates of 8 in. x 8 in. (203 mm x 203 mm) in a back to back situation with a distance of 12 in. (305 mm), but can easily accommodate 12,000 lb (53 kN) in the same configuration.

Concrete Mix Design—a 4000 psi (28 MPa) mix was used based on type II cement with a blend of aggregates (42% of #57 stone, 15% of #8 stone and 43% of sand content) and a water cement ratio of 0.49. The slump at arrival on site is maximum 4 in. (101 mm). A mid-range water reducing admixture is added on site before mixing the steel fibers with the special blast-machine (Fig. 8). At placement, the SFR concrete will have a slump of 5 to 6 in. (127 to 152 mm). The consistency of the concrete has to be inspected by slump testing and visual checking throughout the duration of the pour.

Installation—the 6 in. (152 mm) thick SFR concrete slab is installed on a 6 mil. (0.15 mm) polythene membrane (used as a slip membrane) by direct truck discharge and using the Laser Screed technology. After one pass of straight edge on the freshly laid SFR concrete, a premix quartz and cement based dry shake surface hardener is spread at a rate of 1 lb/ft² (5 kg/m²) using a mechanical Topping Spreader. This will give great abrasion resistance to the slab surface, but will also improve the aesthetic appearance
by covering most steel fibers that could be left embedded close to the surface of the floor. After power floating is completed, a curing agent will directly be sprayed on to keep the moisture from evaporating off the concrete. There are no saw cuts as the jointless panels will move freely. An opening from 1/3 to 2/3 of an inch (8 to 16 mm) can be expected at the metallic construction joint after 1 year.

**Coca-Cola Clad-rack, London, England**

The building, a 108ft-high (33 m) clad-rack warehouse for Coca-Cola products, is 220 ft x 260 ft (67 m x 79 m). The slab is supported on piles with a grid of 10.7 ft by 10.0 ft (3.3 m x 3.0 m) in accordance with the racking layout. Pile diameter is ranging from 1.33 ft to 2 ft (400 to 600 mm); they can carry up to 240,000 lb (1067 kN) and 380,000 lb (1690 kN). The racking load is the combination of the dead weight of the structure, the pallet load, the wind forces and dynamic crane forces. The leg load can be as high as 150,000 lb (667 kN) downward and 80,000 lb (355 kN) upward along the edge of the building.

Design has been achieved according to yield line theory for the ultimate limit state and with finite element analysis for the serviceability limit state.

The 5,000 psi (35 MPa) concrete has been reinforced with 75 lb/yd³ (45 kg/m³) of high tensile strength cramped fibers without any additional conventional reinforcement, and was pumped (Fig. 12). Slab thickness varies from 1.64 ft to 2.13 ft (500 to 650mm) along the edges.

The racking has been anchored directly onto the surface of the floor with resin anchored bolts and special fixing boxes for high upward forces (Fig. 13). It has been directly covered with the cladding and roof and now supports the pallet weight and all the building loads.

**CONCLUSIONS**

Jointless SFRC slabs are not designed for every type of building. These types of slabs are ideal for distribution warehouse and factories that have intense forklift traffic or require high impact and abrasion resistance. Once the building owners, consulting engineers, architects and general contractors have decided to opt for a jointless SFRC slab, they must take precautions in choosing the right concrete contractor for the job by selecting on various items, including, checking contractor’s track record in jointless SFRC floors, making sure to visit jointless reference floors and ask the opinion of their users, checking site quality control procedures proposed by the concrete contractor, ensuring that adequate site conditions will be in place, working at an early stage with the concrete contractor to optimize detailing and design of jointless SFRC slab, limiting the number of split responsibilities within the contract (opt for a design build and ask for guarantees), and being aware and accepting the possibilities of controlled cracks. Building owners are increasingly aware of the problems of saw-cut joints, therefore jointless slab systems are becoming more specified and have proven to be a successful solution. The use of steel fiber reinforced concrete as presented in this paper is one such promising approach to meet this objective of constructing jointless slabs.

**REFERENCES**


Table 1—Theoretical distance between fibers with a diameter of 0.039 in. (1 mm)

<table>
<thead>
<tr>
<th>Fiber concentration lb/yd³ (kg/m³)</th>
<th>Fiber spacing inch (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55 (32.6)</td>
<td>0.839 (21.3)</td>
</tr>
<tr>
<td>60 (35.6)</td>
<td>0.803 (20.4)</td>
</tr>
<tr>
<td>65 (38.6)</td>
<td>0.771 (19.6)</td>
</tr>
<tr>
<td>70 (41.5)</td>
<td>0.743 (18.9)</td>
</tr>
<tr>
<td>75 (44.5)</td>
<td>0.718 (18.2)</td>
</tr>
</tbody>
</table>

Fig. 1—Effect of differential shrinkage on induced joints (curling, rocking, and spalling).
Fig. 2—Evolution of concrete strain and strength with time (Holderbank 1994).

Fig. 3—Evolution of shrinkage with time in a slab-on-grade.

Fig. 4—Decomposition of the shrinkage strain distribution in a slab-on-grade.
Fig. 5—Curling effect.

Fig. 6—Rocking and spalling of a nondowelled saw-cut joint.

Fig. 7—Crimped steel fiber shape.
Fig. 8—Use of a blast-machine to mix fibers on site.

Fig. 9—Formed free-movement joints with various load transfer and arris protection system.

Fig. 10—Flat slab on enlarged pile heads. (Note: 1 ft = 0.3048 m.)
102 Alexandre and Bouhon

Fig. 11—Building layout with construction joints. (Note: 1 ft = 0.3048 m; 1 in. = 25.4 mm.)

Fig. 12—Pumping of the SFRC without additional reinforcement.

Fig. 13—Erection of the 108 ft (33 m) high clad-rack system off the slab surface.
Practical Applications for Natural Cellulose Fiber Including Slab-on-Ground

by J. Purdy, J. D. Speakman, and J. H. Morton

Synopsis: Modern research has led to the extensive use of cellulose fiber in building materials. This research has led to specialized cellulose products engineered for such harsh environments as concrete. Cellulose fibers, since they come from a renewable resource, provide an ecologically friendly alternative to petroleum-based synthetic fibers. Cellulose fibers are also proven to enhance concrete performance attributes while eliminating placement and finishing complications often associated with fiber reinforced concrete such as: fiber balling, surface hairs, and poor fiber dispersion. This paper reviews several commercial jobs using cellulose fiber reinforced concrete including such diverse applications as highway overlays, airport hangars, cast in place high rise components, shotcreted architectural features, commercial and residential slab on grade projects, and pervious pavement. These project examples demonstrate some of the clear performance and practical advantages provided by the use of engineered cellulose fibers for concrete reinforcement.

Keywords: cellulose; fiber; reinforcement.
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INTRODUCTION

Cellulose fibers—history

The use of unprocessed cellulose fiber dates back over 2,000 years, being used in ancient Egypt and other parts of the Middle East to reinforce bricks, as well as by American Indians to reinforce mud walls in the desert southwest. Since that time, improvements in technology have led to the introduction of the engineered processed cellulose fibers which are in use today. These modern cellulose fibers are extensively used in building materials like fiber reinforced concrete and fiber cement board siding. Modern cellulose fibers engineered for these applications are based upon a virgin, purified form of cellulose fiber which is ideal for the harsh environment of the alkaline concrete.

Cellulose is a natural polymer that comes from renewable resources, the most cost-effective of which are paper grade fibers, although higher purity (and more expensive) cellulose fibers such as cotton could also be used. Research and independent testing have verified the performance attributes derived from the addition of cellulose fibers to reinforce concrete including plastic and temperature crack reduction, increased impact resistance, improved freezing and thawing resistance, control of explosive fire spalling, improved hydration, improved strength, reduced permeability, reduced absorption, and improved durability. These benefits were published by Brown and Morton for ready-mix applications [1] and by Morton for structural applications [2] where improved spalling, absorption, and a rational for improved durability were presented.

Synthetic fiber—overview

The attributes of synthetic fibers versus cellulose fibers in fiber reinforced concrete are very different. Synthetic fibers are one of the many end products of fossil fuels. They are typically available as an extruded fiber or produced in tape form. Synthetic fibers available to the concrete industry originated as polypropylene fibers from rope and textile mills. These fibers were fibrillated and could cause complications in the placement and finishing processes. Also, customers frequently did not like the extensive “fuzzy” surface the fibers left behind. As a result of this, producers began promoting monofilament polypropylene fibers to try and lessen these issues.

The root problem with placing and finishing synthetic fibers arises from the fact that the fibers are hydrophobic (water rejecting). This hydrophobicity does not allow the fibers to assimilate well into the concrete and the surface does not readily bond with the cement paste. This lack of bonding creates voids in the concrete. In some cases synthetic fibers are coated with a surfactant to reduce balling. This can have the disadvantage of increasing air contents. Because synthetic fibers do not bond well with cement paste, it is necessary for the fibers be long enough to allow them to interlock between the aggregates in the concrete matrix.

Cellulose fiber—overview

Cellulose fibers are organic polymers that vary on a molecular level in the degree of polymerization and in the crystalline structure of the fiber. Specific properties of the fibers used in this work were described by Morton, Cooke & Akers [3]. A more general reference which includes natural (cellulose) fibers was published by ACI [4]. Because of these variances, all cellulose fibers are not created equal. Cellulose can be liberated from wood materials through chemical processing. In these processes, the
weaker components of the wood materials are removed, leaving only purified, highly stable, cellulose fibers behind. Various forms of engineered processed cellulose fibers are now used worldwide as a major component in building materials products including fiber cement board and fiber reinforced concrete. Project examples described below demonstrate the use of engineered cellulose fibers in various concrete applications. The cellulose fibers utilized in these applications meet the ICC Acceptance Criteria 217, as reported in ICC-ES ESR-1032.

**PROJECT EXAMPLES SUMMARY**

**Oklahoma Overlay project**

This bonded overlay project consisted of the placement of 50,000 yd³ (38,229 m³) of concrete over 13 miles (21 km) of the Muskogee Turnpike in Oklahoma (Figure 1) according to an Oklahoma Department of Transportation (ODOT) specification. The ultra thin overlay was 4-inches (10 cm) thick. A rotomill process was used to expose damage to the turnpike (Figure 2). Concrete dowels were placed at the joints (Figures 3 and 4). Each load was mixed 45 seconds in a central mixer (Figure 6).

Cellulose fiber was added to the central mixer at a 3 lb/yd³ (1.78 kg/m³) dosage (Figures 7 and 8). The concrete was then discharged into dump trucks and placed via slip form paver (Figures 9 and 10). Excellent fiber dispersion and consolidation were achieved, leading to a homogenous concrete mixture (Figure 11). The surface of the finished concrete was floated and dragged following the slip form paver (Figures 12 and 13). For the first time in Oklahoma, the pavement underwent a longitudinal tining process during which no “clumps or fur balls” were noted by the construction crew (Figures 14 and 15).

Previously, longitudinal tining was not used because of clumping issues found with other fiber types. A curing compound was applied to the surface and joints were sawed (Figures 16 and 17). Two types of saw joints were used. The first type consisted of conventional jointing techniques (Figure 18) and the second type consisted of an experimental joint “layout” (Figure 19) in which the joints were cut 4 feet (1.2 m) on center at right angles. Figures 20 and 21 are a close up of these working joints. This project led to the approval for future use of cellulose fibers by the Oklahoma Turnpike Authority (OTA). OTA has stated that they believe this work has resulted in a 30 year repair.

**Sky West hangar**

This project involved construction of a 10-inch (25.4 cm) topping slab at a Sky West airport hangar. For this application, 5,000 yd³ (3,823 m³) of concrete was placed over an existing slab on grade (Figure 22). Project originators were concerned about water migration up through the top of the slab, cracking, and delamination. In order to reduce these risks, 1.5 lb/yd³ (0.9 kg/m³) cellulose fiber was specified as a value addition to the project. The project required pumping concrete containing the cellulose fibers. The concrete pumped perfectly with no segregation. The concrete was placed by laser screed and power trowel, and it was observed that concrete containing these reinforcing cellulose fibers finishes like plain concrete. No fibers collected on the troweling blades or on the surface of the concrete. This led to excellent power troweling (Figure 23). The contractor was pleased with the finish because it was a speed finish with no visible fiber on top, no cracking, and no delamination.

**Trump Tower**

The 35-story Trump Tower project in White Plains, New York, contains over 100,000 yd³ (76,455 m³) of concrete (Figure 24). The “invisible” nature of the cellulose fibers within the concrete encouraged the designer to alter the specification from polypropylene for aesthetic purposes. The fibers dosage rate was 1.5 lbs/yd³ (0.9 kg/m³). Cellulose fiber was used in all cast in place concrete (Figures 25, 26, 27, and 28) including foundations, columns, and decks (parking structures). The hydrophilic nature of the cellulose fiber provided further strength benefits, early form stripping, improved bond strength with steel, and excellent pumpability.

**Shotcrete applications**

There are clear benefits to shotcrete from cellulose fiber reinforcement. Cellulose fiber reinforcement gives shrinkage control and reduces rebound (Figure 29). Addition of these fibers also reduces water permeability, leads to improved hydration, and improves bonding of concrete to reinforcing steel as noted in tests per ASTM C234 (Figure 30). For decorative shotcrete applications, the use of cellulose
106  Purdy et al.

fiber can be especially important because all the performance benefits are obtained with a beautiful finish with none of the visible fiber on the surface that is typical with synthetic fiber (Figure 31).

**Private storage facility**

A private storage facility in Colorado Springs, Colorado, is a commercial slab-on-grade project that required 4,000 yd³ (3058 m³) of. The contractor used a laser screed/power trowel finish (Figures 32, 33, and 34). Polypropylene was originally specified for this job; however, a switch was made to cellulose fiber to eliminate finishing problems with hairy concrete (Figures 35, 36, and 37). The contractor was pleased with the fiber performance and the excellent no blemish finish that was obtained.

**Assorted slab-on-grade projects**

Cellulose fiber was successfully mixed with steel fiber in a rebar-free slab for a warehouse application requiring heavy loading and traffic (Figure 38). This hybrid fiber reinforcement performed very well and resulted in a great finish (Figure 39).

Cellulose fiber was also used in the parking areas of Dixie RV Super Stores and Camping World (Figure 40). The project involved placing 4,000 yd³ (3058 m³) of a 6-inch (15 cm) thick exterior paving slab with a fiber addition of 1.0 lb/yd³ (0.6 kg/m³). Cellulose fiber was chosen for this job to improve finish-ability and reduce cracking.

Hotels and Condominiums are also being built with cellulose fiber reinforcement. In addition to the condominiums, a Hyatt Regency Hotel also used cellulose fiber reinforcement. A total of 4,000 yd³ (3058 m³) of concrete reinforced with 1.5 lb/yd³ (0.9 kg/m³) cellulose fiber (Figure 41) was placed on this job. The ready mix producer chose to add cellulose fibers because of the quality and ease of finishing with added crack control.

A 3,000 yd³ (2294 m³) Cooper Tire Warehouse project in Ft. Pierce, Florida, also used cellulose fiber reinforcement at 1.0 lb/yd³ (0.6 kg/m³) (Figure 42). Cellulose fiber reinforcement was used in 3 different areas of construction. The interior of the warehouse used the fiber reinforcement for their slab on grade application due to anticipated vehicular traffic. Cellulose fiber reinforcement was also used for an elevated metal deck application in the same warehouse (Figure 43). The 6-inch (15 cm) paving on the parking lot also contained cellulose fibers (Figure 44).

A golf cart path (Figure 45) was constructed at the Spanish Peaks Golf Course in Big Sky, Montana. For this job, 3,000 yd³ (2294 m³) of concrete was poured with a 1.5 lb/yd³ (0.9 kg/m³) cellulose fiber addition (Figure 46). The paths were finished using a burlap drag.

Paving in front of a group of retail and real estate offices was also completed with cellulose fiber reinforcement in Laverne, Tennessee (Figure 47). A 6-inch (15 cm) slab was poured with 1.0 lb/yd³ (0.6 kg/m³) cellulose fiber reinforcement for this 300 yd³ (229 m³) parking area.

**Pervious pavement**

A large parking complex has been constructed at Middle Tennessee State University (MTSU). This parking lot is composed of pervious concrete for the parking spaces and asphalt for the remainder of the lot (Figure 48). The pervious concrete sections of this lot contain a 1.5 lb/yd³ (0.9 kg/m³) addition of cellulose fibers. Pervious concrete contains no sand allowing good drainage of water to the soil. This approach to designing concrete is considered to be environmentally friendly but has some drawbacks. These drawbacks include increased expense in construction, decreased durability (compared to typical concretes containing sand), more difficulty in placement, increased maintenance demand, and crushing by the weight of big trucks. Pervious concrete also has advantages over regular concrete. Pervious concrete reduces run-off (resulting in the need to build fewer storm water detention ponds), and can result in improved water quality. Cellulose fiber has been added to the pervious sections of concrete in an effort to enhance the performance of the concrete. Research continues at MTSU to understand the additional benefits of adding cellulose fiber to pervious pavement.

**CONCLUSIONS**

Engineered, processed cellulose fibers are field proven to be a viable option for fibrous reinforcement in concrete for numerous applications providing many performance benefits. Derived from renewable resources, the use of cellulose fiber in concrete offers the industry an alternative to synthetic fibers.
derived from fossil fuels. In addition to extensive performance benefits, engineered processed cellulose fibers assimilate well in concrete and do not complicate the many concrete placement and finishing processes commonly practiced. Aesthetically, cellulose fiber reinforced concrete results in a smooth, blemish-free finish not commonly associated with fiber reinforced concrete.

REFERENCES
4. ACI Committee 544, “Fiber Reinforced Concrete (ACI 544.1R-96),” American Concrete Institute, Farmington Hills, MI, 1996.
Fig. 3—Concrete dowels placed in joints.

Fig. 4—View of turnpike construction.

Fig. 5—A central mix plant.
Fig. 6—Close-up of central mixer.

Fig. 7—Bags of cellulose fiber being counted for addition to mixer.

Fig. 8—Cellulose fiber on conveyor to central mixer.
Fig. 9—Load discharged from central mixer into dump truck.

Fig. 10—Slip form paver at work.

Fig. 11—Homogenous concrete reinforced with cellulose fiber.
Fig. 12—Overlay is dragged as part of finish process.

Fig. 13—Surface of dragged concrete is then floated.

Fig. 14—Roadway undergoing longitudinal tining process.
Fig. 15—No “clumps” or “fur balls” observed after tining process.

Fig. 16—Curing compound applied.

Fig. 17—Joints cut into overlay.
Fig. 18—Conventional joint layout.

Fig. 19—Experimental joint layout.

Fig. 20—Side view of working joint.
Fig. 21—Side view of working joint.

Fig. 22—Colorado Springs Airport, Sky West hangar.

Fig. 23—Power troweled finish.
Fig. 24—Trump Tower project.

Fig. 25—View of tower.

Fig. 26—View of parking garage.
Fig. 27—Another view of tower.

Fig. 28—Another view of tower.

Fig. 29—Shotcrete photo (courtesy of Natural Creations, WA).
Fig. 30—Shotcrete photo (courtesy of Natural Creations, WA).

Fig. 31—Finished shotcrete photo (courtesy of Natural Creations, WA).

Fig. 32—Private storage laser/power trowel finish.

Fig. 33—Nonreinforced slab-on-grade.
Fig. 34—Easy finishing without blemishes.

Fig. 35—View of slabs for private storage.

Fig. 36—Another view of FRC slabs.
Fig. 37—Completed storage facility.

Fig. 38—Cellulose and steel fiber-reinforced slab.

Fig. 39—Close-up of cellulose/steel fiber-reinforced slab.
Fig. 40—Fiber-reinforced paving at RV park.

Fig. 41—Cellulose fiber reinforcement added to slab-on-grade application.

Fig. 42—Slab for Cooper Tire Warehouse containing cellulose fiber.

Fig. 43—Metal deck application in Cooper Tire Warehouse.
Fig. 44—Cellulose fiber-reinforced paving at Cooper Tire.

Fig. 45—Golf cart path, Spanish Peaks Golf Course.

Fig. 46—Cellulose fiber reinforcement added to cart path.
Fig. 47—Cellulose fiber-reinforced paving.

Fig. 48—MTSU parking lot composed of pervious paving and asphalt section.
Field Application and Monitoring of Crack Resistant Fiber-Reinforced Concrete Overlays

by R. Gupta, N. Banthia, and P. Dyer

Synopsis: Water loss from concrete results in volumetric shrinkage, which is significant at early ages. This shrinkage is particularly pronounced when the surface to volume ratio is large of the placements. Fibers, especially synthetic fibers are known to reduce cracking induced due to restrained plastic shrinkage. However, few studies have been conducted to monitor the early-age shrinkage of fiber reinforced concrete (FRC) using embedded sensors in the field. This study involved developing crack resistant FRC material in the laboratory using the bonded overlay technique developed at UBC and using it for a field project. Results from a plain concrete slab-on-grade section and a high volume fly-ash placement were used for comparison with fiber-reinforced concrete (FRC). Three sections were cast using synthetic fiber and their performance was monitored by reading strain signals from embedded sensors. Both traditional (electrical) and state of the art optical sensors were used. Optical sensors registered low strain values due to lack of bond with concrete. On the contrary, traditional electrical sensors clearly demonstrated the reduction in strain in FRC when compared to plain and fly-ash concrete. Specimens were cast on site for conducting tests in the laboratory. In addition, nondestructive tests were conducted on-site for monitoring performance of the slabs. These results are also presented in this paper.

Keywords: cracking; fiber-reinforced concrete (FRC); overlays; restrained plastic shrinkage; slab-on-grade; synthetic fiber.
124 Gupta et al.

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Philip Dyer is the Territory Manager for the Euclid Chemical Company. He has worked in the concrete industry for over 25 years. He has managed ready mixed concrete plants and has been a design engineer for both a foundation contractor and a precaster. He has worked for an international contractor on bridges, dams, powerhouses, and gold mines and has been a contractor segment manager for an admixture company. He is involved with ACI, ACPA, ASTM, and ASCE on various committees that deal with the use of fibers in concrete.

INTRODUCTION

Cement-based materials shrink due to loss of moisture and due to self-desiccation arising from internal consumption of free water during hydration. Shrinkage develops compressive strains, which manifest themselves as tensile stresses under conditions of restraint. When tensile stresses exceed the tensile strength, cracking occurs. Cracking causes debonding, makes concrete more permeable and adversely affects its long term durability. The problem is far more pronounced in overlays, repairs and slabs-on-grade where exposed surface areas are large.

Among the solutions proposed for mitigating plastic shrinkage cracking in concrete including the use of shrinkage reducing admixtures (SRAs), the authors have found that reinforcement of concrete with fibers is undoubtedly the most effective (1,2). This is because the presence of fiber is expected to reduce both the lengths and widths of shrinkage induced cracks (3-5). Fibers act in two different ways. First, fibers produce mix stiffening, reduce the settlement of aggregates, and decrease bleeding. This, in turn, is expected to reduce the formation of bleed channels and decrease the ease with which flow can occur through the material. Secondly, fibers engage water in the mix and reduce the overall early age shrinkage. This is expected to produce a more ‘in-tact’ material with less internal cracking. Clearly, the ability of a given fiber to control plastic shrinkage cracking will depend among other things on the mix design, fiber type and dimensions, hydrophilic/hydrophobic nature of the fiber, concrete conditioning, placement details and the severity of the environment. This report describes a field demonstration project where the role of fiber reinforcement was assessed when used in slabs-on-grade.

RESEARCH PROGRAM

A demonstration project was carried out at the new ChemBioE (Chemical and Biological Engineering) building located at 2360 East Mall at the University of British Columbia, Vancouver (Figure 1). A 6.6 m x 6 m (21.7' x 19.7') concrete slab was cast for the loading dock located near the south entrance of the building. The test slabs are located in the loading bay of the building and are expected to experience large loads during their service life.

The slab was subdivided into five sections: four of size 3.3 m x 2 m (10.8' x 6.6') each, and the fifth one of size 6.6 m x 2 m (21.7' x 6.6') as seen in Figure 2. Two of the five sections (P1 and P4) had plain concrete with no fibers. P1 was control with no fly ash and P2 was one with a high volume (40%) of fly ash. F2, F3 and F5 were reinforced with a hybrid polymeric fiber (Figure 3). F2 and F3 had 4.5 kgs/m³ (7.5 lbs/yd³) and 2.97 kgs/m³ (5.0 lbs/yd³) of the fibers, respectively (see Table 1). Section F5 was designed to study the effect of not having any construction joints in a slab for more than 6.0 m (19.7'). Section F5 had 4.5 kgs/m³ (7.5 lbs/yd³) of the fiber (same as F2). All placements were 150 mm (6") thick and had no steel reinforcement.

In each of the sections, two electrical strain sensors (one along the long and short direction) and one optical strain sensor in the short direction were placed as seen in Figure 2.
CONCRETE MIXTURES

Concrete was batched at a ready-mix plant in Richmond, B.C. and the fibers were pre-mixed at the plant as recommended by the manufacturer. The mix design for the various concretes is given in Table 2. As indicated, 40% cement was replaced in test slab P4.

STRAIN MONITORING

Strain sensors

Strain gauges were mounted on specially designed chairs (Figure 4), 25 mm (0.98”) below the top finished surface and equidistant from the slab edges (Figure 2). Every section contained two electrical (long and short direction) and one optical strain sensor along the short direction. Two temperature sensors were also installed alongside sensors ‘O1’ and ‘E4’ in placements P1 and P4 respectively, to monitor temperature changes. This allowed for any temperature correction to the strains.

For the optical sensors, a Fiber Bragg Grating (FBG) was installed at mid-length of a 250 mm (9.8”) long GFRP rebar (Figure 5). The modulus of elasticity of the GFRP rebar was 42 GPa (6091 ksi) and that of the sensor as reported by the manufacturer was 2.75 GPa (398.9 ksi). A single mode fiber optic patch cable was used with the fiber optic sensors and a FC/APC (angle polished connector) was used at the ends to extend the length of the cable. For additional protection, the embedded portion of optical cables and electrical wires were covered in a vinyl tubing.

Data acquisition system

The entire data acquisition system was installed on-site. For the fiber-optic sensors (Fiber Bragg Gratings, FBGs) a special data acquisition system was used. The advantage of the system is that it monitors all FBG channels on a common time base, which simplifies post-acquisition processing of the data. The unit used in this study had a total of 8 channels. A custom software was used to acquire data onto a PC. For the electrical sensors, an acquisition system designed and built at UBC was used. The 13 channels system is configured to enable data monitoring via the Internet using a web-based data acquisition system.

CONCRETE PLACEMENT

Since four different concrete mixes were used, concrete placement was spread over 3 days (see Table 1). Concrete with 70±20 mm (2.8”±0.8”) slump was poured and broom finished (Figure 7). No external vibration was utilized for compaction and the slabs were protected with a plastic sheet for a minimum of two days to avoid excessive loss of moisture.

RESULTS

Strain readings

Data were collected from optical and electrical sensors at 100 Hz and 0.01 Hz, respectively. A high acquisition frequency resulted in an enormous amount of data acquired over a relatively short period of time especially in the case of the optical sensors. These data were analyzed and condensed by proper averaging techniques. Data were acquired in two sets. One prior to setting (this included the initial temperature rise) and one after setting (this included acquisition after proper bonding). At the start of both these stages, all sensors were “zeroed” to have a fair comparison between different strain sensors. Further, in the case of the electrical sensors, the signals from the gauges in longitudinal and transverse direction in each placement were very similar; indicating that strain in all directions of the placement was similar. For simplified comparison, strains recorded from the two electrical sensors in each placement were averaged (except E5-T).

Figure 8a shows the presetting strains recorded with the electrical sensors. Notice that these sensors recorded a tensile strain during the first few hours after casting. This is expected as the temperature of concrete will increase due to the generation of the heat of hydration.

Figure 8b shows the average post-setting signals from the electrical sensors. Notice that the sensors in F2 and F5 (fiber reinforced concrete with a higher fiber dosage rate) recorded low values of strains implying a reduced potential for cracking. P1, P4 and F3, on the other hand, recorded greater strains that those in F2 and F5 placements, and these strain were compressive in nature. As will be noted later, unfortunately, the compressive strength of concrete in placement F3 was abnormally low which may explain the high strains noted in this placement. The strain results of placement F3 may also indicate
that a fiber dosage of 2.97 kgs/m³ (5 lb/yd³) is not sufficient to bring about a notable reduction in the measured strain. The similar magnitude of strains in P1 and P4 is somewhat surprising as one would expect the placement with high fly-ash content (P4) to record somewhat higher strains due to reduced bleeding and greater potential for subsequent water loss.

The reduced nature of strains in F2 and F5 is encouraging. Similar strains recorded in F2 and F5 is also encouraging as it implies that a joint-free placement (F5) does not necessarily increase the risk of cracking. Post-setting strain increment data is given in Table 3.

The post-setting data from the optical sensors are plotted in Figure 9. As seen in Figure 9, the strains recorded in all optical gauges were very low. To protect the FBG mounted on the GFRP rebar, a rubber tape was used which unfortunately appears to have prevented the development of bond between the sensors and concrete.

Materials tests

As indicated before, cylinders and prisms were cast to determine the compressive strength (ASTM C39) and flexural toughness (ASTM C1609) values. The cylinders were cured on-site for the first day and then moved to the laboratory to simulate site curing conditions. Results from the compression tests (average of three cylinders) are presented in Table 4. Notice that the control (P1) and high volume fly ash mixes (P4) had similar compressive strengths (22 [3190 psi] and 24 MPa [3480 psi] respectively). The mixes with a high fiber dosage had a compressive strength of about 32 MPa which was the only mix that came close to the target strength of 32 MPa (4641 psi) (see Table 2). The mix with low fiber dosage (F3) developed a very low strength only of 17 MPa (2465 psi). This affected the strain readings as was described previously. In Table 4, the results from the Schmidt hammer tests are also given. These tests are describe later.

The average flexural toughness (load-displacement) curves for low fiber dosage (F3) and high fiber dosage (F2 and F5) fiber concretes are given in Figure 10a. The load vs. deflection curves for the three specimens tested for F3 and F5 are presented in 10b and 10c, respectively. Notice that the curves for F5 are very consistent. On the contrary, for the mix F3, due to the lower fiber dosage, higher instability was noticed after the peak load. Due to test error, data for F3-2 was lost beyond 1 mm (0.039") of specimen deflection.

These curves were analyzed as per the PCS method (6) and according to ASTM C 1069 (7). Results for the PCS analysis are given in Figure 10e. Toughness values calculated according to ASTM C 1609 are presented in Table 5. For a beam tested on a span L, with a width and depth, respectively, of b and h, the post-crack strength $PCS_m$ at a deflection of $L/m$ is given by (Figure 10e):

$$PCS_m = \frac{(E_{post,m}) \times L}{(\frac{2L}{m} - \delta_{peak}) \times b \times h^2}$$

(1)

The above equation is taken from Banthia and Trottier (6) and the terms used in this equation are described in Figure 10d. Note that $PCS_m$ has units of stress and at a deflection equal to $\delta_{peak}$, the $PCS_m$ value would coincide with the MOR of the beam.

Notice that specimens cast using concrete mix type F2/F5 with a high fiber dosage had significantly higher toughness than the mix with a lower fiber dosage (F3). The un-reinforced concrete specimens (P1 and P4), as expected, did not have any post crack toughness and hence only peak load values could be recorded.

Non destructive tests (NDTs)

To further study the performance of various slabs, Schmidt hammer rebound measurements, Electrical Impedance (EM) measurements, and Ultrasonic Pulse Velocity measurements (UPV) were conducted. Only Schmidt hammer and EM results are described in this paper to maintain brevity. UPV results are reported by Gupta [8]. The slabs were divided into a 0.3 x 0.3 m (11.8" x 11.8") grid for these measurements.

Schmidt hammer measurements—Schmidt rebound hammer was used for measuring the quality of concrete in the various placements (Figure 11a). Results are given in Figure 11b. Note that in the case of the Schmidt hammer, due to a number of intervening uncertainties, the measured values represent not so much the absolute values of compressive strength but probable values and only provide a comparative

Materials tests
assessment from one placement to another. On a comparative basis, the relative values of the compressive strengths measured using the Schmidt hammer correlated well with the results obtained from ACTM C39 compression tests as indicated in Table 4.

**Resistivity**—Four-point Wenner resistivity meter was used to measure the electrical resistivity of the various slabs (Figure 12). Average, maximum, and minimum resistivity values recorded between the grid points are presented in Table 6. It is believed that the moisture content of all the slabs was high due to significant rainfall a few days prior to testing. Average resistivity values recorded for all slabs were lower than 1.34KΩ·cm (0.53 KΩ inch) and values were very similar for different concrete types. In the context of the joint-free floor, this meant that there was no adverse effect of increasing the joint spacing.

**CONCLUDING REMARKS**

The project presented here is one of the first attempts at measuring internal strain developed in slabs-on-grade at early ages with the help of traditional and fiber optic sensors. Data indicate that the strains developed in fiber reinforced concrete placements are lower than those developed in the plain placements. This implies that in-service cracking potential in fiber reinforced concrete placements is lower and one can expect a better long-term durability. The project also investigated joint-free flooring and determined that there is no increased cracking potential in such floors as long as appropriate amount of fiber reinforcement is present.

**ACKNOWLEDGMENTS**

The authors acknowledge the assistance of Propex, Inc., ISIS Canada Research Network, Rempel Concrete (Irv Lenz), APSC Dean’s Office (Ron Loewen), and Civil Engineering technicians (especially John Wong) for their help. Thanks are due to Andy Vizer of Cement Association of Canada and the on-site staff of Stuart Olson, Inc.

**REFERENCES**

Table 1—Concrete placement details

<table>
<thead>
<tr>
<th>Notation</th>
<th>Admixture</th>
<th>Concrete type</th>
<th>Date and time of casting</th>
<th>Remarks</th>
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<tr>
<td>P1</td>
<td>None</td>
<td>32 MPa</td>
<td>Dec 13, 2005 (11 am)</td>
<td>Control (plain concrete)</td>
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<td></td>
<td></td>
<td>No fly-ash</td>
<td></td>
<td></td>
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<tr>
<td>F2</td>
<td>Fiber</td>
<td>32 MPa</td>
<td>Dec 12, 2005 (12 noon)</td>
<td>Hybrid polymeric fiber 7.5 lbs/ yd³</td>
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<td></td>
<td></td>
<td>No fly-ash</td>
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<tr>
<td>F3</td>
<td>Fiber</td>
<td>32 MPa</td>
<td>Dec 14, 2005 (9 am)</td>
<td>Hybrid polymeric fiber 5.0 lbs/yd³</td>
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<td></td>
<td></td>
<td>No fly-ash</td>
<td></td>
<td></td>
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<tr>
<td>P4</td>
<td>Fly ash</td>
<td>32 MPa</td>
<td>Dec 13, 2005 (12 noon)</td>
<td>HVFA</td>
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<td></td>
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<td>(40% fly-ash replacement)</td>
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<tr>
<td>F5</td>
<td>Fiber</td>
<td>32 MPa no fly-ash</td>
<td>Dec 12, 2005 (12 noon)</td>
<td>hybrid polymeric fiber 7.5 lbs/yd³</td>
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Note: 1 MPa = 145 psi; 1 lbs/ yd³ = 0.59 kg/m³.

Table 2—Concrete mixture design (for 1 m³)

<table>
<thead>
<tr>
<th>Concrete Placements</th>
<th>P1, F2, F3, F5</th>
<th>P4</th>
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<tr>
<td>Target strength, MPa</td>
<td>32 at 28 days</td>
<td>32 at 56 days</td>
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<td>Class</td>
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<td>Cement Type GU, kg</td>
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<td>Fly ash, Type CI, kg</td>
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<td>20 mm aggregate, kg</td>
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<td>14 mm aggregate, kg</td>
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<td>425</td>
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<tr>
<td>Sand (SSD), kg</td>
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<td>615</td>
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<tr>
<td>Water reducer</td>
<td>300 mL/100 kg of cement</td>
<td>300 mL/100 kg of cement</td>
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<tr>
<td>Water, L</td>
<td>155</td>
<td>145</td>
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<tr>
<td>Slump ±20 mm</td>
<td>70</td>
<td>70</td>
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<tr>
<td>Air, %</td>
<td>5-8</td>
<td>5-8</td>
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<tr>
<td>Maximum W/B</td>
<td>0.45</td>
<td>0.45</td>
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</table>

Note: 1 MPa = 145 psi; 1 kg = 2.2 lbs; 1 L = 0.2642 gal.; 1 mL = 0.034 oz; 1 mm = 0.0397 in.

Table 3—Post-setting strain (μ strain) increment data (electrical sensors)

<table>
<thead>
<tr>
<th>Placement</th>
<th>Short direction</th>
<th>Long direction</th>
<th>Average</th>
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<tr>
<td>P1</td>
<td>−134</td>
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<td>F2</td>
<td>−61</td>
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<td>P4</td>
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<td>−122</td>
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<tr>
<td>F5</td>
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<td>—</td>
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Table 4—Compression test results

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<tr>
<th>Placement</th>
<th>Compressive strengths in MPa (ASTM C39)</th>
<th>Ratios of compressive strengths from Schmidt Hammer tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1 (Control)</td>
<td>22.3</td>
<td>1.00</td>
</tr>
<tr>
<td>P4 (High Volume Fly ash)</td>
<td>24.2</td>
<td>0.94</td>
</tr>
<tr>
<td>F2 (High fiber volume)</td>
<td>32.4</td>
<td>2.15</td>
</tr>
<tr>
<td>F3 (Low fiber volume)</td>
<td>17.0</td>
<td>0.61</td>
</tr>
<tr>
<td>F5 (High fiber volume)</td>
<td>32.4</td>
<td>2.07</td>
</tr>
</tbody>
</table>

Note: 1 MPa = 145 psi

Table 5—Toughness values using ASTM C 1609

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mix F3</th>
<th>Mix F5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>P₁/Pₚ (kN)</td>
<td>11.53</td>
<td>11.04</td>
</tr>
<tr>
<td>f₁/fₚ (MPa)</td>
<td>3.46</td>
<td>3.31</td>
</tr>
<tr>
<td>δ₁/δₚ (mm)</td>
<td>0.017</td>
<td>0.055</td>
</tr>
<tr>
<td>P₁₀₀,₀·₅ (kN)</td>
<td>2.45</td>
<td>1.84</td>
</tr>
<tr>
<td>f₁₀₀,₀·₅ (MPa)</td>
<td>0.736</td>
<td>0.552</td>
</tr>
<tr>
<td>P₁₀₀,₂·₀ (kN)</td>
<td>1.96</td>
<td>—</td>
</tr>
<tr>
<td>f₁₀₀,₂·₀ (MPa)</td>
<td>0.589</td>
<td>—</td>
</tr>
<tr>
<td>T₁₀₀,₂·₀ (N.mm)</td>
<td>5.40</td>
<td>—</td>
</tr>
</tbody>
</table>

*Average of two values

Note: 1 MPa = 145 psi; 1 kN = 224.8 lbs; 1 mm = 0.0397 in.; 1 N.mm = 0.0088 lb.inch.

Table 6—On-site resistivity results

<table>
<thead>
<tr>
<th>Slab designation</th>
<th>Average resistance (Ω)</th>
<th>Resistivity values (kΩ·cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
<td>Minimum</td>
</tr>
<tr>
<td>F5</td>
<td>41.5</td>
<td>1.46</td>
</tr>
<tr>
<td>P4</td>
<td>41.68</td>
<td>1.43</td>
</tr>
<tr>
<td>F3</td>
<td>41</td>
<td>1.49</td>
</tr>
<tr>
<td>F2</td>
<td>40.23</td>
<td>1.48</td>
</tr>
<tr>
<td>P1</td>
<td>42.62</td>
<td>1.66</td>
</tr>
</tbody>
</table>
Fig. 1—ChemBioE building, UBC.

Fig. 2—Five sections of the loading dock. (P1: control; P4: HVFA; F2: polymeric fibers at 4.5 kg/m$^3$ (7.5 lbs/ yd$^3$); F3: polymeric fibers at 2.97 kg/m$^3$ (5 lbs/ yd$^3$); and F5: joint-free floor with polymeric fibers at 4.5 kg/m$^3$ (7.5 lbs/ yd$^3$).
Fig. 3—Polymeric fibers used in the investigation.

Fig. 4—(a) Specially designed chairs for sensors; and (b) sensor before concreting. (Note: 1 mm = 0.0397 in.)
Fig. 5—An FBG optical sensor.

Fig. 6—On-site data acquisition systems: (a) optical sensor readout device; and (b) data logger for electrical strain gauges.
Fig. 7—Concrete placement and finishing (notice the sensors in (a)).
Fig. 8—(a) Presetting averaged signals from the electrical sensors (tensile +ve compression –ve). Tensile strains indicate a rise in the temperature of concrete; and (b) average post-setting signals from the electrical sensors. Notice that the sensors in F2 and F5 recorded low values, implying a reduced potential for cracking. P1 and P4 recorded similar magnitudes of compressive strains.

Fig. 9—Average post-setting signals from the optical sensors.
Fig. 10—(a) Average load versus deflection plots for FRC (F2/F5 and F3) as per ASTM C1609; (b) load versus deflection plots for three FRC (F3) specimens tested as per ASTM C1609; and (c) load versus deflection plots for three FRC (F5) specimens tested as per ASTM C1609.
Fig. 10—(d) Calculation of $PCS_m$ values (refer to Eq. (1)); and (e) post-crack strength (PCS) plots.
Fig. 11—(a) Rebound measurements using a Schmidt hammer; and (b) on-site Schmidt hammer results.

Fig. 12—Four-point Wenner resistivity probe.
Lessons Learned from Bridge Repair with Steel Fiber-Reinforced Shotcrete and Overlays

by J. Wong and P. D. Carter

Synopsis: The use of steel fiber-reinforced, silica-fume-modified concrete overlays and shotcrete for bridge repairs has long been the standard for Alberta Transportation (AT). Approximately 200 provincial bridges have been repaired with steel fiber reinforced, silica-fume-modified materials since 1984, mostly to decks, which had been exposed to aggressive conditions including freezing and thawing, de-icing salts, and moisture. The purpose of using steel fiber in bridge concrete repair materials was primarily to prevent or reduce repair cracks and to improve durability. Standard fiber lengths were 25 mm (1 in.) for shotcrete and 50 mm (2 in.) for overlays with standard fiber dosages of 60 km/m³ (100 pounds/yd³). Most of the repairs were done with superplasticized, silica-fume-modified, low water-cement ratio concrete mixes. This paper reports on the early historical development of the repair method basics and the lessons learned from monitoring bridge repairs. Conclusions are presented from AT’s Level 1 and Level 2 inspection data. Level 2 data quantifies several repair performance indicators, such as wide cracks, stains, delamination, spalls, patches, and debonds, as well as a breakdown of numerical condition ratings on important bridge elements. Crack data from Level 2 inspections was available from 154 silica fume modified bridge deck overlays, 124 of which contained steel fiber. The crack data was analyzed to assess the performance of fibers in reducing the amount of easily visible cracks: those wider than 0.3 mm (0.012 in.). The results showed that the steel fibers resulted in significant crack reduction and improved overlay durability and service life.

Keywords: corrosion protection; high-performance concrete; overlays; repair cracking; shotcrete; steel fiber-reinforced concrete.
**INTRODUCTION**

Northern climate bridge deck repairs often involve replacing thin layers of concrete surface; some of the damage has been caused by freeze-thaw action or corrosion spalling. Repair materials should have both good structural and good durability properties. Cracking in repairs is a common problem, partly due to long-term drying shrinkage in Alberta’s dry climate; cracking reduces resistance to corrosion damage and the subsequent life of the bridge. Steel fibers were introduced in repairs to reduce cracking.

**BACKGROUND**

The post World War II “baby-boom” was accompanied by a similar increase in the numbers of roads and bridge crossings in Alberta. Alberta’s existing bridge inventory includes over 3,800 standard bridges, over 1,800 major bridges, and more than 9,300 bridge-sized culverts. Of these, Alberta Transportation (AT) is the responsible road authority for about 5,300 structures. The older structures built in rural areas were often site-batched concretes, which had poor quality control and were therefore susceptible to the deterioration processes of cold northern climates. Most of the bridges constructed in the 1950s were simple span bridges using non-air entrained concretes. De-icing of the most heavily traveled bridges with chloride-based de-icing materials began in the late 1960s and slowly spread to the entire primary highway system. Leak-proof deck joints were unreliable until the late 1970s, and a significant amount of concrete substructure deterioration occurred due to deck joint leakage. Prior to 1978, new bridges were not regularly designed to resist chloride penetration, and deck and other chloride-exposed surfaces required thin repairs, which often cracked. Control joints are impractical for deck overlays and other repairs. The repair and construction materials of the 1980s were ineffective because polymer repair materials were expensive and incompatible with the properties of the underlying concrete. Cementitious repair materials also had adverse properties of high shrinkage, cracking, debonding, and sometimes poor durability.

**Developments in substructure repair methods in Alberta**

Structural concrete repair materials and methods were not well known to AT prior to the early 1980s. Non-fiber-reinforced shotcrete repairs had been used experimentally in the 1970s. Latex modified shotcrete repair was first used in Alberta in 1980 to restore cover and jacket the deteriorated curbs on the Elbow River Bridge near Bragg Creek. The primary advantage of latex modified shotcrete was its durability, but the Elbow River Bridge repairs experienced severe debonding, which provided the first lessons learned by AT regarding shotcrete repairs. Further understanding of the important factors in achieving durable repairs was gained by followup inspection and testing, as well as discussions with engineering staff involved in the Gardiner Expressway repairs in Toronto, Ontario, Canada. Traditional shotcrete repairs are usually used for overhead elements, but AT was, at that time, experiencing a high failure rate on curb repairs done with conventional repair materials. Therefore, dry-process, site-batched, latex-modified shotcrete was used to repair about fifteen additional Alberta bridges between 1981 and 1983, of which about half involved curb repairs. The site batching process blended 4 parts of sand by volume to 1 part of type 1 cement. A key element in achieving good durability results was ensuring that the contractor was qualified. Nozzlemen and mix designs were tested by trial batch shooting. Two hundred forty linear metres of corrosion damaged curb repairs were successfully completed on the two CNR Overpasses on Highway 2 at Cayley.

Due to its susceptibility to surface cracking, the latex shotcrete repairs included reinforcement with galvanized welded wire mesh. Installing mesh reinforcement was labour intensive and therefore expensive,
so other types of reinforcement were needed. An unpublished comparative study of the properties of ten different fiber types in three common Alberta concrete matrices was completed by the University of Calgary for AT in 1983. One trial mix included a silica fume admixture, and its affect on the first crack strength in ASTM C1018 tests showed promise as a synergistic material with steel fiber. The comparative test methods included crack resistance, toughness, strength, and permeability, and the findings led to the experimental use of steel fibers in shotcrete and overlay repairs in 1984.

STEEL FIBER-REINFORCED SILICA FUME MODIFIED SHOTCRETE REPAIRS

More than 20 major highway bridge substructures were repaired with steel fiber reinforced silica fume (FRSF) shotcrete for AT between 1984 and 1987. The primary cause of this substructure damage was deck expansion joint leakage; at the same time that the shotcrete repairs were being done, sliding plate deck joints were being replaced with waterproof deck joints. More than 800 new deck joints had been installed by the year 2000, or about forty deck joints per year. The joints were normally replaced when the deck was rehabilitated, but some locations were given high priority due to the value of the underlying elements which were being exposed to leakage. As a result of this aggressive deck joint replacement program in the early 1980’s, the number of deteriorated substructures gradually declined until steel fiber shotcrete repairs became unnecessary by the mid-1990’s.

The first steel FRSF shotcrete repairs were done to bridge piers, abutments, and concrete girder undersides on seven bridges between June and September of 1984. The work was publicly tendered, and the successful bidder was Econite, an experienced shotcrete contractor, which contributed new knowledge on factors of successful repairs. A significant amount of testing was done during the course of the project, and the results were reported at technical sessions for the American Concrete Institute Fall Convention in Chicago, 1984, and the World of Concrete in Atlanta in 1985. The work involved dry-process pre-bagged mix, which suited the small repairs in remote areas that lack a nearby supply of transit mix. The mix included approximately 400 kg (880 lb.) of type 1 cement, 40 kg (880 lb.) of silica fume, and 25 mm (10 in.) long steel fiber with a concentration of 60 kg (132 lb.) per cubic meter of prebagged material. The silica fume reduced rebound losses, allowed thicker shotcrete layers to be placed, and was believed to have synergistic properties in conjunction with the fiber.

The repairs involved removing deteriorated concrete by mechanical methods, followed by sandblasting the chipped surfaces. Sawcutting of repair edges was not required, so the repair perimeters had thin feathered edges, which are less durable than sawcut edges. Additional anchorage in the form of drilled steel inserts was installed when needed in upper perimeter areas or on a 1000 mm (39 in.) grid pattern when jacketing was being applied. In the latter cases, small diameter reinforcing steel was tied in both directions to provide additional reinforcing to the steel fiber. Thin concrete repairs are susceptible to deterioration by moisture loss through evaporation or by capillary suction into the substrate. The substrate bond surfaces were pre-wetted to SSD condition to reduce absorption of moisture. The pre-wetting period varied from 12 to 48 hours, depending on the substrate and shotcrete thicknesses and the substrate porosity.

On bridge substructure repairs, straight lines and flat finishes are often needed for aesthetic purposes. In these situations, the shotcrete was applied in two stages. The first stage applications were assisted by wood forms to create straight repair lines to match the appearance of the structure. Shooting into forms requires special skills for the nozzleman to avoid trapping overspray and rebound. The second stage applications were often completed the next day, after removing the forms and sandblasting to remove the non-consolidated surface laitence. The final layers were shot with non-fiber material to cover the steel fibers and prevent future rust stains. Several standard finishes could be specified for different types of repairs: as-shot and wood float finishes were the two most common ones. Depending on the orientation of the repairs, either wet burlap or intermittent water spray curing was applied for the first 24 hours after final shooting. Following the wet curing, two coats of single-component, clear acrylic sealer were applied to both the new shotcrete and the adjacent substrate surfaces, observing a drying period between the coats. The sealer had a dual purpose: first, to provide additional curing for the new shotcrete, and second, to protect the adjacent non-repaired substrate surfaces. A routine challenge in these repairs is to determine where to terminate the concrete removal operation, and often the areas adjacent to the repairs are not 100 percent sound, containing chlorides and early stages of deterioration. The sealer helped reduce the deterioration rate of the adjacent non-repaired concrete. Caulking was
also used in some cases to seal cold joints between repairs and to seal steel elements, such as bearings. Inspection was an important part of the work, and it included approval of surface preparation, measurement of repair quantities, enforcement of the specifications, witnessing the shooting of test specimens on-site to qualify new mixes and nozzlemen, and core and pull tests for measuring interlayer bond strength. The specification required a minimum 28-day compressive strength of 45 MPa (6525 psi). A partial payment schedule allowed AT to accept lower strength shotcrete at reduced prices rather than having to reject all defective work.

Most of the 1980s shotcrete-repaired bridges were in active service as of 2006 and were being inspected on a 21-month cycle. The repairs have generally performed well.

**SPECIFIC CASE HISTORIES OF STEEL FIBER-REINFORCED SILICA FUME MODIFIED SHOTCRETE REPAIRS**

The following case histories are presented to illustrate typical repair conditions and long-term shotcrete repair performance.

**Carvel Corner Grade Separation**

The westbound Highway 16 over Highway 43 Grade Separation, a continuous three-span steel rolled beam girder bridge, was constructed in 1959 and shotcreted in 1984. The east pier had been heat-damaged by a burning oil truck. The pier was in excellent condition prior to this one-hour fire. The damaged pier cap and leg surfaces needed an average repair depth of 75 mm (3 in.). Approximately 7 m$^3$ (9 yd$^3$) of FRSF shotcrete was placed, covering a surface area of about 90 m$^2$ (900 ft$^2$). An inspection on March 10, 2006, showed that the 22-year-old repairs were in good condition with no stains, spalls, or deterioration, but with some minor shrinkage cracks. Although exposed to salt spray from traffic, the non-sawcut repair edges were still sound. Visually, the repairs were darker in color than the original cast-in-place concrete.

**North Saskatchewan River Bridge on Highway 22 near Drayton Valley**

Measuring 320 m (1,056 ft) in length, this eight-span steel deck-truss bridge was constructed in 1957 with non-air-entrained concrete. The bridge is located near the foothills of the Rocky Mountains in an area with high snowfall and extreme freeze-thaw cycles, and the deck had non-water-tight deck expansion joints. The narrow bridge is on a truck route and receives heavy de-icing salt application. Deck and curb repairs were a frequent occurrence at this site, and large sections of the curbs were repaired or recast in 1980, 1986, 1991, and 1999. Leakage from the nine deck expansion joints had caused freeze-thaw deterioration of all piers. The deck height is 26.8 m (88.4 ft) above normal river level, and the six largest solid shaft piers were in the river. The FRSF shotcrete repairs were started in 1984 and completed in 1986. All deck joints were replaced with leak-proof models in 1986 to address the cause of the deterioration. Piers 2 and 7 had suffered the worst deterioration and were spot-repaired with FRSF shotcrete in 1984. Piers 1, 3, 4, 5, and 7 were repaired in 1986 by jacketing the upper portions and caps with a layer of dry process FRSF shotcrete 75 to 100 mm (3 to 4 in.) thick, anchored with steel inserts at 1 m (3 ft) grid spacing. A wood-float finish was applied.

The bridge and its 22-year-old repairs were visually inspected on March 16, 2006, in conjunction with a condition assessment. The inspection found the FRSF shotcrete repairs in good general condition, especially when compared to condition of the adjacent, non-repaired pier concrete. The repairs had few cracks, even though the shotcrete had been placed without control joints. The non-sawcut repair perimeters showed no spalling or signs of failure.

**McLeod River Bridge on old Highway 16 east of Edson**

In an area of high snowfall and freeze-thaw cycles, this 208 m (685 ft) long, 5-span, arched steel deck-truss bridge was constructed in 1954 on Highway 16 near Edson. By 1987, deck joint leakage onto the substructure elements had resulted in significant deterioration of the non-air-entrained concrete. The average deck height is 20.1 m (66.3 ft) above normal water level, so the piers are high and two are located in the river. The two large river piers were jacketed with FRSF shotcrete without any vertical control joints in 1987, and the other two piers were given spot repairs to the caps and other deteriorated areas. Jacketing involves a relatively thin repair with a circumference of about 40 m (130 ft), creating the
tendency to crack from evaporation, long-term drying, moisture absorbing into the porous substrate, and stresses from cooling of the outer layer. The jackets were placed in three vertical sections, which are distinguished by color differences due to changes in the pre-bagged shotcrete mix. An AT Level 1 inspection on August 11, 2006, showed the 19-year-old repairs to be in good condition with the piers being Level 1 rated 5 (that is, “no problems”). The inspection found no significant spalling, debonding, wide cracks, or other signs of deteriorating repairs.

LESSONS LEARNED FROM FRSF SHOTCRETE REPAIRS

Lessons can be learned from both failures and successes, by investigating causes of local failures and by considering factors related to good durability performance against the different factors of other sites experiencing poor durability performance. The following observations and conclusions relate to the methods and factors for achieving good durability of FRSF shotcrete repairs:

• Replacing leaking deck joints with water-tight models greatly slows pier deterioration rates.
• Shooting test slabs to prequalify nozzlemen and mix designs improves general repair quality.
• The early feather-edged perimeter edges often show early signs of debonding, from which confidence in current standards requiring sawcuts is confirmed.
• Use of forms to achieve straight edges had no adverse effects on the durability of the repairs.
• Topcoating of the exposed steel fiber layer with a 12 mm (0.05 in.) layer of non-fiber-reinforced shotcrete has prevented subsequent rust stains and enhanced the appearance.
• Pre-wetting the substrate is important for reducing repair cracks. Observations over time showed that flexibility was needed in the amount of wetting time needed (12 to 48 hours).
• General observations were that finishing actions can disturb bond strength: the less finishing done, the more durable the repair.
• Curing of new repairs is achieved by wet curing for 24 hours, followed by drying and light sandblasting, and finished with 2 layers of clear acrylic sealer to both repairs and adjacent areas. The acrylic performs as both an early curing agent and a subsequent sealer.
• The comparison of 7-day compressive strength test results of samples cut from test slabs, cured at site and transported to labs, provides early indications of possible quality control problems. The payment reduction for low 28-day tests is an incentive to avoid low strengths.

STEEL FIBER REINFORCED CONCRETE BRIDGE DECK OVERLAYS

Background on deck overlays

Bridge deck deterioration started to become a common problem in Alberta in the early 1970s, and the routine solution of replacing the asphalt wearing surface with a nominal 50 mm (2 in.) thick protective concrete overlay developed thereafter. The first deck overlays were placed by bridge crews using transit-mixed ‘Class D’ concrete; since cracking was expected, welded wire mesh was included in the early 1970s overlays. The method of zero-slump, Iowa-method, or high-density concrete overlay was introduced in 1976 for repair of the Cushing Street Bridge in Calgary. Subsequently, AT placed more than 80 high-density overlays for rehabilitation and another 90 for protection of newly constructed major bridges between 1977 and 1985. Contrary to current policy, these overlays were placed on dry decks without pre-wetting. The repair performance was assessed in those early days by on-site tests for bond strength, chloride contents, and corrosion potential by the ASTM C-876 ‘copper sulfate electrode’ (CSE) test method, as well as other tests. The information was used to identify problems and improve the quality of future repairs. High-density overlay bond strengths were typically good, averaging over 2 MPa (290 psi), but the following negative performance parameters soon became evident: high numbers of cracks, higher post-overlay CSE readings, and rapid buildup of chlorides. Based on these findings, the search for more effective deck repair methods began in 1984. The need for crack repair and prevention overlays was identified, and steel FRSF overlays began to evolve.

Another problem requiring crack prevention was management of the many 1950’s precast girders (PG) and ‘H’ connected (HC) bridges that were exposed to de-icing chlorides. These modular bridge superstructures used conventionally reinforced precast concrete girders and had transverse and longitudinal joints allowing leakage to the girder undersides where the main reinforcing was located. Many such bridges were in poor condition by the late 1970s due to corrosion spalling of the girder legs: thin, lightweight, waterproof concrete overlays were considered to be a low-deadload solution. Ten high-density
overlays placed on HC girders in 1978 quickly cracked, showing the outline of the girders. The use of steel fiber in concrete overlays was seen as a possible solution to prevent cracking.

**Development of FRSF deck repair methods in Alberta**

FRSF overlays were developed in three stages. First, experimental types of overlays were placed by government crews between 1984 and 1986. Fiber dosages and other issues were investigated in these early jobs. Second, 5 percent silica fume pre-bagged mix overlays were placed on small precast bridges by contractors from 1987 to 1990. Lastly, larger bridge overlays were placed by contractors from 1991 to the present using pre-bagged or transit mixes, superplasticizer, and 8 to 10 percent silica fume.

Government bridge crews often helped to develop new technologies in the 1980s, using promising new materials or methods on small bridge repairs. Small repairs done by specialty repair contractors were another source of new technology. Once the fine points of the work were identified, the new methods were documented in bridge repair specifications for use in publicly tendered contracts. AT bridge crews placed the first three steel fiber reinforced silica fume modified concrete overlays on precast concrete HC girder bridges at Strawberry Creek, Rourke Creek, and Oldman Creek crossings in 1984. Some problems were noted due to using local transit-mixed concrete in remote areas, so many of the subsequent overlays in remote locations were placed with pre-bagged concrete mixed on-site in transit mix trucks. After about fifteen FRSF overlays had been placed by bridge crews, the first publicly tendered FRSF bridge deck overlay contracts were awarded in 1987. These overlays showed promise, and the technology spread for use on many larger bridges starting in 1990.

**Performance test data for SF and FRSF overlays**

By 2006, AT had placed more than 150 FRSF overlays for bridge deck rehabilitation. Many of these bridges are periodically inspected and tested with Level 2 procedures, and 124 steel FRSF overlays were identified from the Level 2 database. Also identified was a reference group of 30 overlays placed in a similar concrete matrix without fiber. Level 2 tests provide data on potential for corrosion in numerous ways: CSE tests; chain dragging for quantification of delamination and debonds; and quantification of cracks, stains, scaling, spalling, and chloride contents. Level 2 tests also provide a breakdown of numerical condition ratings on important bridge elements, including deck wearing surface and underside condition.

**Findings related to steel fiber-reinforced silica fume (FRSF) concrete overlays**

The initial purpose of using steel FRSF overlays was to reduce the amount of long-term cracking to prevent chloride ion ingress. The Level 2 crack data provided an excellent source of information, since many similar non-fiber silica fume modified overlays had been placed by contract during the same period as the FRSF overlays were placed. These non-fiber overlays are referred to as silica fume (SF) overlays, and they had very similar mix proportions and constituents to the FRSF overlays, except for not containing fibers. Thus, they represent the control group for assessing the effect of the fiber on the long-term performance in terms of the Level 2 results.

Table 1 describes the two groups of SF and FRSF overlays where post-overlay Level 2 test data was available; pertinent findings from the analysis of the Level 2 data on overlays are shown in Table 2.

**Performance findings related to steel fiber-reinforced silica fume (FRSF) concrete overlays**

The following findings represent the contribution of the steel fiber to various Level 2 properties:

- Although FRSF overlays were generally larger than the SF overlays, the FRSF overlays had 61 percent less wide visible cracks per overlay than the SF overlays.
- The crack density for FRSF overlays was 68 percent lower than for SF overlays.
- Comparing standard deviations showed higher variability in overlay cracking of FRSF overlays. Figure 1 illustrates the averages and ranges of cracking for the two types of overlay.
- The performance of overlays is measured by the drop in percentage of CSE readings more negative than -300mV following overlay placement. Compared to SF overlays, FRSF overlays were 43 percent more effective in reducing the potential for active corrosion.
- The corrosion life gained by FRSF overlays was 57 percent greater than that of SF overlays.
- The data shows that debonding over overlays, regardless of type, has been extremely minor, and debonding is not expected to be a determining factor in the remaining lives of these bridges.
GENERAL OBSERVATIONS AND COMMENTS ON PLACEMENT/PERFORMANCE OF FRSF OVERLAYS

Properly placed FRSF overlays have fewer visible cracks, which appears to reduce the ingress of moisture and chlorides that sustain deck corrosion; therefore, they are expected to have longer service lives than SF overlays without steel fiber. The following conclusions relate to the essential factors for achieving good durability of FRSF overlays and should be considered in any specification:

- A key issue in achieving durable FRSF overlays is to allow the concrete to mature with 7-day wet curing prior to exposing the overlay to tensile stresses from cooling or drying shrinkage.
- A workable concrete mix and use of vibratory screed assists the 50 mm (2 in.) steel fiber to settle into the mix so that the fibers align in directions perpendicular to crack formation, where they intersect cracks and prevent them from widening. Higher aspect-ratio fiber types sometime appear to remain on the finished surface.
- Very little rust staining has been observed on the surface of AT FRSF overlays.
- The standard method of final surface preparation, for achieving a sound condition on which to place the overlay, has been a heavy sand-blast done just prior to pre-wetting.
- Pre-wetting the deck overnight, then allowing to dry to SSD condition prior to placing, addresses the two issues of reducing overlay cracks and enhancing overlay bond strength.
- Maintaining the mix temperature below 18°C (64°F) during placement, using ice in the mix if necessary, increases the time window for placing and finishing, slows hydration, and reduces surface cracks. Night pours are often used when weather conditions are unfavorable.
- Use of thermal blankets will reduce overlay cooling and evaporation rate and prevent cracks.
- Finishing operations should be done quickly after placing, without overfinishing the surface. The silica fume content makes the mix sticky.
- After finishing, use of a grooving tool for skid resistance will help to avoid grabbing and pulling out the fibers.

ACKNOWLEDGMENTS

The authors would like to express their thanks to the Bridge Engineering Section of AT for permission to use the data presented in this paper.

REFERENCES


| Table 1—Comparison of descriptive information on SF and FRSF datasets |
|-------------------|------------------|-----------------|---------|
|                   | Non-fiber SF overlay | FRSF Overlay | Units   |
| Number of bridges | 30                | 124            | —       |
| Daily traffic     | 11,086            | 5,096          | Vehicles |
| Deck Size         | 815               | 917            | m²      |
| When OL placed    | 30                | 26             | Years   |
| Bridge age at inspection | 36    | 33            | Years   |
| Overlay age at inspection | 5.9  | 6.3          | Years   |
| Overlay age as of Dec 2006 | 11 | 13           | Years   |

Note: 1 m² = 10.76 ft².
Table 2—Level 2 data comparison

<table>
<thead>
<tr>
<th>Level 2 data type</th>
<th>Non-fiber SF overlay</th>
<th>FRSF overlay</th>
<th>Units</th>
</tr>
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<tbody>
<tr>
<td>Total cracks &gt; 0.5 mm (0.02 in.) width</td>
<td>207 (683)</td>
<td>81 (267)</td>
<td>m (ft)</td>
</tr>
<tr>
<td>Std. dev. total cracks</td>
<td>161</td>
<td>100</td>
<td>m</td>
</tr>
<tr>
<td>Crack density</td>
<td>0.25 (0.076)</td>
<td>0.08 (0.024)</td>
<td>m/m² (ft²/ft²)</td>
</tr>
<tr>
<td>Debonding (chain drag)</td>
<td>0.01</td>
<td>0.01</td>
<td>%</td>
</tr>
<tr>
<td>ASTM C876 CSE &gt;300 mV before overlay</td>
<td>27.1</td>
<td>29.1</td>
<td>%</td>
</tr>
<tr>
<td>CSE &gt;300 mV 6 year after</td>
<td>16.6</td>
<td>14.1</td>
<td>%</td>
</tr>
<tr>
<td>Drop in % &gt;300 mV after overlay</td>
<td>10.5</td>
<td>15.0</td>
<td>%</td>
</tr>
<tr>
<td>Corrosion Serv life gained</td>
<td>17.4</td>
<td>27.4</td>
<td>Years</td>
</tr>
<tr>
<td>Level 2 wearing surface rating</td>
<td>6.37</td>
<td>6.63</td>
<td>—</td>
</tr>
<tr>
<td>Level 2 deck underside rating</td>
<td>6.40</td>
<td>6.63</td>
<td>—</td>
</tr>
</tbody>
</table>

Fig. 1—Crack length comparison.
Fig. 2—Crack amounts and propagation rates for SF and FRSF overlays. (Note: 1 m = 3.281 ft.)

Fig. 3—Steel fiber-reinforced silica fume (FRSF) concrete overlay placed in 1994. This photo taken in 2006 shows a typical 12-year-old FRSF overlay in Edmonton (107 Avenue over Groat Road) with negligible abrasive wear, ranging from 0 to 2 mm (0.08 in.). No visible corrosion of surface fibers can be seen.
Fig. 4—Carvel Corner grade separation fire damage repairs. Steel fiber-reinforced silica fume (FRSF) shotcrete was used for the first time in 1984 on seven bridge repairs as an alternative to latex modified shotcrete, which was more expensive and less user-friendly. This 1984 photo shows first of two stage shooting with forms, done to achieve straighter lines.

Fig. 5—Carvel Corner grade separation fire damage repairs, 1984. Second stages were usually shot on the second day after the forms were removed. Shotcrete surfaces were lightly sandblasted to remove the nonconsolidated laitence prior to second stage shooting.
Fig. 6—Carvel Corner East Pier, 1984, FRSF shotcrete repairs to fire damaged surfaces. This follow-up photo was taken in 2006 on the 22-year-old repairs. No obvious defects.

Fig. 7—1983 photo of pier #2 on North Saskatchewan River Bridge at Drayton Valley (constructed 1957). The site is exposed to high snowfall and freeze-thaw cycles. Piers #2 and 7 were repaired by FRSF shotcrete in 1984, and the remaining piers in 1986.
Fig. 8—2006 photo of pier #2, North Saskatchewan River Bridge at Drayton Valley. Only the upper portions were shotcreted in 1984. The 22-year-old FRSF shotcrete looks better than most of the adjacent concrete. Clear acrylic sealers were applied to the shotcrete for curing/sealing.

Fig. 9—Another 2006 photo of Drayton Valley, looking north along piers #3, 4, 5, and 6, which were repaired with FRSF shotcrete in 1986. Piers #3 and 5 are located below deck expansion joints with heavier leakage than the other piers.
Fig. 10—Another 2006 photo of one of the piers below deck expansions joints. Note that the FRSF shotcrete repairs remain well-bonded and in good condition. The stains seen here are from the steel truss members above, which still have original 1957 paint.

Fig. 11—March 16, 2006 photo. West end of Pier 3, which is below deck expansion joint. The photo shows the typical specified FRSF shotcrete finish. The nominal 12 mm (0.47 in.) thick surface coat of non-fiber shotcrete covers up the underlying steel fiber shotcrete to prevent surface rust stains.
Fig. 12—2006 photo of the south side of the east (upstream) shaft of Pier 6 at Drayton Valley. The entire pier shaft circumference was jacketed, and Hilti inserts were placed on a one meter grid pattern for tying 10 mm (0.39 in.) reinforcing bars, then encased by the 100 mm (3.94 in.) thick FRSF shotcrete.

Fig. 13—2006 photo of Drayton Valley Bridge. These repairs are spot repairs, rather than jacketing. The shotcrete still has good bond with the substrate. Silica fume was used to densify the mixture and increase the performance of the 25 mm (0.98 in.) long steel fiber.
Fig. 14—McLeod River Bridge near Edson, constructed in 1954 (2006 photo). Three large river piers were jacketed with FRSF shotcrete in 1991 to repair deterioration due to the non-air-entrained concrete. Jacketing involved a circumference of 36.6 m (120 ft) without control joints.

Fig. 15—Typical test slabs shot on plywood panels, cured at site, transported to labs, and either cored or cut into cubes for compressive tests to determine if full payment would be given.
Steel Fiber-Reinforced Concrete in Free Suspended-Elevated Slabs

by X. Destrée

**Synopsis:** The structural use of steel fibers as the only principal reinforcing has been developed and refined for the last 15 years. Total replacement of traditional rebar is now common in applications like suspended slabs resting on a pile grid which spans from 3 m (10 ft) to 5 m (17 ft) each way. Generally, the span-depth ratio of the slabs in such applications ranges from 12 to 25. Although most of these slabs use the ground as a form only, some of them have been cast in elevated conditions without any contact with the ground to ensure total independence in the event expansive clay or gas hazards are present or could be present. More recently, steel fiber reinforced concrete has been used in suspended elevated slabs with a span-depth ratio equal to 30, and spans from 5 to 8 m (17 to 26 ft) length. The present article reviews the concrete mix design, the type of steel fiber, the dosage rate needed, the hardened concrete testing method based on current standard documents and round indeterminate panel slab tests. An example of steel fiber-reinforced concrete elevated slab is given and the design method is outlined in detail.

**Keywords:** design method; elevated slabs; steel fiber; testing.
Xavier Destrée has been involved in the development of steel fiber reinforced concrete since 1978. He has been the author of numerous patents related to steel fibers and novel applications such as joint-free slabs-on-grade, suspended, and elevated slabs. Since 1978, he has provided technical support to numerous contractors across five continents.

INTRODUCTION
The present article follows a former publication [1] included in the proceedings of the FRC BEFIB2000 Fifth International SFRC Symposium. In two other articles [2,3], the basics of steel fiber-reinforced concrete free suspended slabs are explained. Two identical full-scale test procedures for SFRC suspended slabs with multiple spans of 3.10 m each way, with the same concrete mix design using 45 kg/m³ steel fibers, have been completed independently; the first in Belgium and the second in Australia. Both confirm the same experimental results and validate earlier conclusions [1]. Full scale testing up to ultimate loading of SFRC suspended slabs with span-depth ratios of 20 confirms the following:

- The observed ultimate loading intensity results of a completely ductile rupture process where the suspended slab deforms along yield lines where all rotations are concentrated.
- The observed ultimate loading intensity depending on the case investigated, ranges from 3 to 4 times the first crack loading.
- Regardless of the loading intensity, the slab never punches out before yielding in flexure.
- The yield-line moment intensity back-calculated from full scale suspended slab testing, is confirmed by laboratory test results when indeterminated round panel slabs are subjected to center point loading up to rupture.

MIXTURE DESIGN
In order to develop and define a concrete mix design including steel fibers, various mixes have been investigated using the LCPC Baron-Lesage optimum compaction method [2]. Undulated steel wire fibers of diameters from 1 mm (0.039") to 1.30 mm (0.051") and lengths from 45 mm (1.77") up to 60 mm (2.36") have been included in the mix. Steel fiber dosage rates between 45 kg/m³ (75 lbs/yd³) and 120 kg/m³ (198 lbs/yd³) have been tested. According to the LCPC Baron-Lesage flow-box method, the optimum sand to gravel ratio produces the highest compaction of concrete and the quickest flow time. A super-plasticizing admixture is added to obtain the final slump required. The super-plasticizer admixture is a naphthalene melamine sulphonated at 40% dry concentration. The steel fibers are undulated having a diameter equal to 1 mm (0.039"), 1.15 mm (0.045") and 1.3 mm (0.051"), respectively, and a length equal to 60 mm (2.36"), 45 mm (1.77") and 50 mm (1.97"), respectively. With a cement content of 375 kg/m³ (633 lbs/yd³) and a water content of 187 kg (316 lbs/yd³) (w/cm=0.50), typical mixes tested are shown in Table 1. All these optimal mixes have been extremely workable and suitable under critical pumping conditions. An example of suitable aggregate gradation is outlined in Table 2. The content per cubic meter of concrete of each constituent aggregate used is as follows: Aggregate 1 - 930 kg/m³ (1581 lb/yd³); Aggregate 2 - 800 kg/m³ (1360 lb); and, Aggregate 3 - 60 kg/m³ (102 lb/yd³).

The mixture designs used in the investigation are shown in Table 1. The fiber reinforced concrete slab is installed without the need for poker vibrating. As usual, the concrete slab needs to be carefully cured using a sprayed membrane and the installation must meet the requirements for state of the art for placing and finishing.

TESTING
Steel fiber reinforced concrete round panel slabs of 1500 mm (59") and 2000 mm (79") span of respectively 150 mm (6") and 200 mm (8") thicknesses; have been subjected to flexure under center point loading. A typical testing set-up is shown in Fig. 2 and Fig. 3. The plate is simply supported along its perimeter so that it is a statically indeterminate round panel slab test. During the flexural testing, a load-deflection plot is recorded where the first cracks are quite visible (minute cracks of 0.1 mm (0.00039") opening) under the point loading, followed by a pseudo-elastic linear stage with a slightly increased rate of deflection where we can observe formation of yield lines ending in a fully plastic flexure; this is in accordance with the theory of Johansen. Up to 30 cracks are visible along the perimeter edge of the round panel after testing. The load versus deflection plots of the round panel slabs are shown in Fig. 4.
The final rupture pattern is the typical “FAN” pattern where the expression of the equilibrium of rotation of one circular sector gives:

$$P_{\text{ultimate}} = 2\pi M_L$$  \hspace{1cm} (1)

where $M_L$ is the yield line moment of Johansen.

Assuming a rectangular stress block in compression over 10% of the cross sectional and a rectangle tensile stress block of $f_{tu}$ stress intensity distributed over 90% of the cross sectional, then:

$$f_{tu} = M_L / 0.45 * h^2$$  \hspace{1cm} (2)

where $M_L = P_{\text{ultimate}} / 2\pi$ and $h$ is the thickness of slab, $f_{tu}$ the plastic tensile strength of the steel fiber reinforced concrete.

A simple expression of $f_{tu}$ is derived from (1) and (2) as follows:

$$f_{tu} = P_{\text{ultimate}} / 2.827 * h^2 \text{ N/mm}^2$$  \hspace{1cm} (3)

where $P_{\text{ultimate}}$ is the maximum loading intensity of each type of concrete as shown in Fig.4.

The expression (3) of $f_{tu}$ is valid regardless of the diameter of the round panel slab. For each steel fiber dosage rate, $f_{tu}$ is then calculated using three 1500 mm (59")/150 mm (6") or 2000 mm (79")/200 mm (8") round panel slabs. The resulting plastic tensile strengths are summarized in Table 3.

We can observe that $f_{tu}$ intensities deducted from both types of round panel slabs are quite similar. Thus, $f_{tu}$ is not a function of the span of the round panel and it confirms that the smaller size round panel slab of 1500 mm (59") span of 150 mm (6") thickness may be adopted as a structural method of slab testing. One can observe that while increasing the thickness of the round panel slabs from 150 mm (6") to 200 mm (8"), there is no loss of post-cracking resistance. In fact there is increased post-cracking resistance. This is in complete contradiction to Dutch NEN 6720 standard [4], which states that post-cracking strengths of SFRC, is among other parameters, a function of a reduction factor (1.6 – h) meters. The (1.6-h) factor has been derived from analyses on plain and SFRC prismatic specimens in flexure, failing to represent any type of multiple cracking of SFRC suspended slabs. A more recent RILEM test method using a notched small prismatic specimen in flexure, as shown on Fig.5, fails as well to represent the flexural ductility and multiple cracking of a suspended steel fiber reinforced concrete slab.

The steel fiber reinforced concrete types outlined in Table 3 have been tested following the SFRC TC 162 guideline [5]. The notch is expected to concentrate the formation of one localized flexural crack, so that the erratic deviation observed with un-notched prismatic specimens should be avoided.

Table 4 outlines two types of flexural strengths $f_{eq}$ and $f_{tu}$. The flexural strength, $f_{eq}$ is indeed the equivalent flexural strength obtained by totaling the energy of flexure from first crack to 3 mm (0.12") deflection. $f_{tu}$ is calculated at Maximum Post Crack Loading $P_{\text{mpc}}$, when the concrete section develops its Maximum Plastic Flexural Moment, $M_{\text{mpf}}$. Thus, $f_{tu}$ is calculated as in equation (2). Then replacing $M_L$ by $M_{\text{mpf}}$ we obtain:

$$M_{\text{mpf}} = P_{\text{mpc}} / 4 = 0.9 * h_n * h / 2 * b * f_{tu}$$

where $l$ (500 mm (19.7")) is the span, $b$ (150 mm (6")) the width, and $h_n$ the thickness (125 mm (5")) at the notch of the prismatic specimen.

A simple expression for $f_{tu}$ is obtained as follows:

$$f_{tu} = P_{\text{mpc}} / 8.44 * h^2 \text{ N/mm}^2 \text{ or } f_{tu} = P_{\text{mpc}} / 2.74 * h^2 \text{ psi}$$  \hspace{1cm} (4)

For comparison purposes we can back calculate the equivalent flexural strengths of the 1500 mm (6") and 2000 mm (79") diameter slabs of Fig. 4 and Table 3 using a Timoshenko formula to relate stress and loading:

$$f_t = (1 + \nu^2)(0.485\log a / h + 0.52) * 0.48 * P / h^2$$  \hspace{1cm} (5)
where \( a \) is the radius of the round panel slab, \( \nu \) is the Poisson ratio, \( h \) the thickness, and \( P \) the loading intensity (\( \nu = 0.2; \frac{a}{h} = 5 \)).

Taking into account a deflection equal to span/150 or 10 mm (0.4") in case of 1500 mm (59") diameter slab and 13.3 mm (0.52") in case of 2000 mm (79") slab, we can calculate \( f_{teq\_slab} \) as follows:

\[
f_{teq\_slab} = \frac{P}{1.373/h^2} \text{ N/mm}^2 \quad \text{or} \quad f_{teq\_slab} = \frac{P}{1.417/h^2} \text{ psi}
\]

Table 4 summarizes the results of calculations of the post cracking strengths of SFRC TC162 type notched beam specimens, round panel slab of 150 mm (6") thickness x 1500 mm (59") span and 200 mm (8") thickness x 2000 mm (79") span.

Prismatic specimens (SFRC TC 162 method, ref.5) and round slabs testing are compared in light of the following ratios:

\[
\frac{f_{teq\_slab}}{f_{tu\_prisms}} = 1.94 \text{ (1500 mm (59") dia.) and } = 2.00 \text{ (2000 mm (79") dia.)}
\]

\[
\frac{f_{tu\_slab}}{f_{tu\_prisms}} = 1.27 \text{ (1500 mm (59") dia.) and } = 1.30 \text{ (2000 mm (79") dia.)}
\]

The post-cracking of a round panel slab and the yield moment of Johansen \( M_L \) are clearly not predicted using interpretation of prismatic notched specimen results [5], unless high correction coefficients of (1.27 to 2.00 in our investigation) are applied. Moreover, it has been observed that when using high dosages of 100 kg/m³ (168.6 lb/yd³) and more of steel fibers, the cracking of the specimen does not localize at the notch location. Small prismatic specimens in flexure, notched or un-notched, do not show multiple cracking patterns and that is why the post cracking strengths are underestimated [7].

The geometry of the prismatic specimens refrains them from redistributing the stresses further away once cracking has been initiated and this varies from that observed in real slab structures where micro-cracking can develop up to macrocracking and, ultimately, to the formation of yield lines.

**DESIGN AND APPLICATION**

As an example, we will analyze the design of a free suspended elevated slab, used as the main ground slab (350 m² area - 3700 sq.ft.) of a social chalet within a school. The slab is built above a dry river bed that can be flooded from time to time, and therefore spans from column to column are in a 4.85 m (16 ft) x 5.85 m (19 ft 1") grid. The columns are supported by ground piles of the same p = 1.15 m (3 ft 8") diameter. The suspended slab carries the whole chalet constructed of concrete and masonry units. The resulting equivalent un-factored uniformly distributed loading intensity on the suspended slab is 34 kN/m² (700 lb/ft²). Fig.6 shows a cross section of the chalet from foundation to roof. The fiber reinforced concrete slab appears in black. The thickness of the slab is equal to 300 mm (11.8”).

\[
M_L = f_{tu} * 0.45 * h^2
\]

Where, \( f_{tu} = 2.8 \text{ N/mm}^2 \text{ (398 psi)} \) as outlined in Table 4 in case of 80 kg/m³ (132 lbs/yd³) dosage rate of 1.3 mm (0.051") diameter / 50mm (1.96") length undulated steel fibers.

Hence,

\[
M_L = 113.80 \text{ kNm/m (25 kip.ft/ft)}
\]

The verification of the design is accomplished by using Johansen’s yield line model where one single span can show, in the worst and most onerous case, three parallel yield lines near the columns with \( M_L \) negative and at mid span where \( M_L \) is positive. Near the columns, the negative yield lines are located at a distance equal to half the depth of the slab from the external face of the column as shown in Fig.8. The yield line pattern is detailed in [6] and is shown in Fig.7.

As shown in Fig. 7 and Fig. 8, the span length between the yield lines is \( L_{ni} \). In the design equation (9), the net span is \( L_n = 4.50 - 1.15 - 0.3 = 3.05 \) m (10 ft) and \( 4.40 \) m (14 ft-6 in) in the other direction. Load factors are 1.35 and 1.5 for slab weight \( G \) and live load \( Q_i \), respectively. The material factor is taken as 1.5, so that the global safety factor is at least 2.25.
or, \(73.95 < 75.33\) \(\text{[kNm/m]}\) \(15.93 < 16.22\) \(\text{(kip.ft/ft)}\)

The flexural stress in service under un-factored loading can be estimated as follows:

\[ L_n = \left(\frac{(4.50 - 1.15)(5.85 - 1.15)}{10}\right)^{1/2} = 3.96 \text{ m (13 ft)} \]

\[ M_s = 41.25 \times 3.96^2/10 = 65 \text{ kNm/m (14 kip.ft/ft)} \]

\[ F_s = 6 \times 65000/300 \times 300 = 4.33 \text{ N/mm}^2 \text{ (615 psi)} \]

\(F_s\) is quite an acceptable level when compared to \( f_{eq,slab}\). \(f_{eq,slab} = (8.54 + 9.76)/2 = 9.15 \text{ N/mm}^2 \text{ (1300 psi)}\) and this as in Table 4. The slab is not cracked under maximum service loading intensity.

The elevated slab includes Anti-Progressive Collapse rebar following [7]. These APC bars run from column to column in the bottom of the slab and must cross over the columns footprint. The area of APC additional steel \(A_{sb}\) needed is calculated as follows [7]:

\[ A_{sb} = \frac{0.5(\text{ws}\times 1/1000)L_n}{\Phi s fs} \]

\(A_{sb} = 0.5 (34 \times 1/1000) \times (5.8 - 1.00) \times 4.5/0.85 \times 500 = 864 \text{ mm}^2 \text{ (1.34 in}^2)\) or three (3) bottom rebar of 20 mm (0.79") diameter from column to column in each way.

Foundation slabs are another current application of structural fiber reinforced concrete. Fig. 9 shows typical foundation slab applications of structural fiber reinforced concrete. The first photograph on the left hand side in Fig. 9 shows the steel fiber reinforced concrete being pumped for a concrete raft under construction using a mix design with 80 kg/m³ (132 lbs/yd³) of 1.3 mm/50 mm (0.051"/1.97") steel fibers (Table 2). The thickness of the raft is 340 mm (13.35") with the traditional starter bars installed. The second photograph on the right hand side in Fig. 9 shows a fiber reinforced concrete suspended raft on top of a 6 m x 6 m pile grid with concrete pumping in progress. The raft shown in the construction phase is of 340 mm (13.35") thick using steel fiber reinforced concrete at a dosage rate of 80 kg/m³ (132 lbs/yd³) of 1.3 mm (0.051") diameter x 50 mm (1.97") long steel fibers. The traditional structural raft reinforcing is completely omitted in this application resulting in cost savings, simplification of the construction process, and increased jobsite safety. In these cases, the concrete has been prepared at the batching plant and supplied to the jobsite as a ready mixed reinforced concrete.

CONCLUSIONS

Steel fiber reinforced concrete in structural applications where steel fibers completely replace the traditional structural reinforcement has become reality. A novel mixture design that is workable, pumpable, and virtually self compacting without the need for any poker vibration has been developed. Steel fiber concentrations of 120 kg/m³ (198 lbs/yd³) and higher are used. The structural performance of the fiber reinforced concrete has been characterized in flexure using a centre point loaded round panel slabs of 1500 mm (59") diameter and 150 mm (6") thickness. It is noted that the small notched beam specimens tested in flexure according to SFRC TC 162 fail to provide relevant design information to develop steel fiber reinforced concrete structural slabs. The notched small beam specimen in flexure is inappropriate when investigating multiple cracking.

REFERENCES

### Table 1—Typical fiber-reinforced concrete mixes tested

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
<th>VI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber dosage rate (kg/m³)</td>
<td>60 (99 pcy)</td>
<td>80 (132 pcy)</td>
<td>120 (198 pcy)</td>
<td>80 (132 pcy)</td>
<td>45 (75 pcy)</td>
<td>80 (132 pcy)</td>
</tr>
<tr>
<td>Diameter of fiber (mm)</td>
<td>1.15 mm (0.045&quot;)</td>
<td>1.15 mm (0.045&quot;)</td>
<td>1.15 mm (0.045&quot;)</td>
<td>1.3 mm (0.051&quot;)</td>
<td>1 mm (0.039&quot;)</td>
<td>1 mm (0.039&quot;)</td>
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<tr>
<td>Length of fiber (mm)</td>
<td>45 mm (1.77&quot;)</td>
<td>45 mm (1.77&quot;)</td>
<td>45 mm (1.77&quot;)</td>
<td>50 mm (1.97&quot;)</td>
<td>60 mm (2.36&quot;)</td>
<td>60 mm (2.36&quot;)</td>
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<td>Cement (kg/m³)</td>
<td>375 (633 pcy)</td>
<td>375 (633 pcy)</td>
<td>375 (633 pcy)</td>
<td>375 (633 pcy)</td>
<td>375 (633 pcy)</td>
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<tr>
<td>Water (kg/m³)</td>
<td>188 (316 pcy)</td>
<td>188 (316 pcy)</td>
<td>188 (316 pcy)</td>
<td>188 (316 pcy)</td>
<td>188 (316 pcy)</td>
<td>188 (316 pcy)</td>
</tr>
<tr>
<td>Aggregate 5/14 mm (kg/m³)</td>
<td>835 (1409 pcy)</td>
<td>786 (1327 pcy)</td>
<td>733 (1297 pcy)</td>
<td>752 (1270 pcy)</td>
<td>889 (1501 pcy)</td>
<td>828 (1379 pcy)</td>
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<tr>
<td>Sand 0/4 mm (kg/m³)</td>
<td>835 (1409 pcy)</td>
<td>865 (1460 pcy)</td>
<td>879 (1483 pcy)</td>
<td>902 (1522 pcy)</td>
<td>799 (1348 pcy)</td>
<td>828 (1397 pcy)</td>
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<tr>
<td>Superplasticizer (liters/m³)</td>
<td>3.8 (0.76 gal./yd³)</td>
<td>4 (0.8 gal./yd³)</td>
<td>4.3 (0.86 gal./yd³)</td>
<td>4 (0.8 gal./yd³)</td>
<td>3.3 (0.66 gal./yd³)</td>
<td>5 (1 gal./yd³)</td>
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<td>Sand/Aggregate ratio</td>
<td>1</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
<td>0.9</td>
<td>1</td>
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pcy – pounds per cubic yard (lb/cu.yd.)

### Table 2—Aggregate grading

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<tr>
<th>Sieves</th>
<th>0.63</th>
<th>0.125</th>
<th>0.25</th>
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<th>2</th>
<th>4</th>
<th>8</th>
<th>16</th>
<th>32 (mm)</th>
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<tbody>
<tr>
<td>Aggregate 1</td>
<td>0</td>
<td>5</td>
<td>16.4</td>
<td>35</td>
<td>55</td>
<td>75</td>
<td>95</td>
<td>100</td>
<td>100</td>
<td>Passing (%)</td>
</tr>
<tr>
<td>Aggregate 2</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>2.5</td>
<td>35</td>
<td>100</td>
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<td>Passing (%)</td>
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<tr>
<td>Aggregate 3</td>
<td>5</td>
<td>98</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>Passing (%)</td>
</tr>
<tr>
<td>Total (kg)</td>
<td>8.6</td>
<td>110.9</td>
<td>218.1</td>
<td>391.1</td>
<td>577.1</td>
<td>763.1</td>
<td>963.5</td>
<td>1270</td>
<td>1790</td>
<td>(kg)</td>
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<tr>
<td>Total (%)</td>
<td>0</td>
<td>6</td>
<td>12</td>
<td>22</td>
<td>32</td>
<td>43</td>
<td>54</td>
<td>71</td>
<td>100</td>
<td>Passing (%)</td>
</tr>
</tbody>
</table>

1 kg = 2.205 lb; 1 inch = 25.4 mm
Table 3—Plastic post-cracking tensile strength $f_{tu}$ for each fiber concrete type

<table>
<thead>
<tr>
<th>Fiber dosage rate and fiber diameter/fiber length</th>
<th>Steel fiber tensile strength</th>
<th>Slab dimensions tested</th>
<th>$f_{tu}$</th>
</tr>
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<tbody>
<tr>
<td>120 kg/m³ 1.15 mm/45 mm</td>
<td>1400 N/mm² (199 ksi)</td>
<td>Slab Diameter - 1500 mm Slab Thickness - 150 mm</td>
<td>3.00 N/mm² (426 psi)</td>
</tr>
<tr>
<td>100 kg/m³ 1.30 mm/50 mm</td>
<td>750 N/mm² (106 ksi)</td>
<td>Slab Diameter - 1500 mm Slab Thickness - 150 mm</td>
<td>2.83 N/mm² (402 psi)</td>
</tr>
<tr>
<td>70 kg/m³ 1.00 mm/60 mm</td>
<td>1500 N/mm² (212 ksi)</td>
<td>Slab Diameter - 1500 mm Slab Thickness - 150 mm</td>
<td>2.83 N/mm² (402 psi)</td>
</tr>
<tr>
<td>45 kg/m³ 1.00 mm/60 mm</td>
<td>1500 N/mm² (212 ksi)</td>
<td>Slab Diameter - 1500 mm Slab Thickness - 150 mm</td>
<td>2.52 N/mm² (358 psi)</td>
</tr>
<tr>
<td>100 kg/m³ 1.3 mm/50 mm</td>
<td>750 N/mm² (106 ksi)</td>
<td>Slab Diameter - 2000 mm Slab Thickness - 200 mm</td>
<td>3.2 N/mm² (454 psi)</td>
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<tr>
<td>120 kg/m³ 1.15 mm/45 mm</td>
<td>1400 N/mm² (199 ksi)</td>
<td>Slab Diameter - 2000 mm Slab Thickness - 200 mm</td>
<td>3.1 N/mm² (440 psi)</td>
</tr>
</tbody>
</table>

Note: 1 mm = 25.4 in.; 1 kg/m³ = 1.686 lb/yd³.

Table 4—Average post-cracking tensile strengths: notched beams versus round panel tests

<table>
<thead>
<tr>
<th>Fiber dosage rate and fiber diameter/fiber length</th>
<th>RILEM TC162 Rilem Prisms $f_{eq}$, N/mm² (psi)</th>
<th>Round Panel Slabs $f_{eq}$, N/mm² (psi)</th>
<th>Round Panel Slabs $f_{eq}$, N/mm² (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Slab Diameter 1500 mm (59 in.)</td>
<td>Slab Diameter 2000 mm (79 in.)</td>
</tr>
<tr>
<td>60 kg/m³ 1.15 mm/45 mm</td>
<td>4.64 (659) 1.66 (236)</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>45 kg/m³ 1.00 mm/60 mm</td>
<td>4.16 (591) 1.87 (266)</td>
<td>8.54 (1213)</td>
<td>–</td>
</tr>
<tr>
<td>80 kg/m³ 1.3 mm/50 mm</td>
<td>4.96 (705) 2.07 (294)</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>100 kg/m³ 1.3 mm/50 mm</td>
<td>5.12 (727) 2.13 (303)</td>
<td>9.76 (1386)</td>
<td>10.98 (1560)</td>
</tr>
<tr>
<td>70 kg/m³ 1.00 mm/60 mm</td>
<td>–</td>
<td>9.15 (1300)</td>
<td>–</td>
</tr>
<tr>
<td>80 kg/m³ 1.00 mm/60 mm</td>
<td>5.12 (818) 2.37 (337)</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>80 kg/m³ 1.15 mm/45 mm</td>
<td>5.00 (710) 2.01 (286)</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>120 kg/m³ 1.15 mm/45 mm</td>
<td>5.65 (802) 2.57 (365)</td>
<td>10.68 (1518)</td>
<td>10.64 (1512)</td>
</tr>
</tbody>
</table>

Note: 1 mm = 25.4 in.; 1 kg/m³ = 1.686 lb/yd³.
Fig. 1—LCPC Baron-Lesage Flowbox and resulting plot of flow time versus sand-gravel ratio. (Note: 1 L = 0.2642 gal.)

Fig. 2—Typical setup of testing of a round panel slab test.

Fig. 3—Details of setup of testing. (Note: 1 mm = 0.0394 in.)
Fig. 4—Round panel slab flexural test response.
(Note: 1 mm = 25.4 in.; 1 m = 3.281 ft; 1 kN = 0.2248 kip; 1 kg/m³ = 1.686 lb/yd³.)

Fig. 5—Typical notched prismatic beam specimen [5]. (Note: 1 mm = 25.4 in.)

Fig. 6—Typical example of steel fiber-reinforced concrete elevated slab.
Fig. 7—Yield line pattern of an elevated suspended slab.

Fig. 8—$L_n$, the net span between negative moment yield lines.

Fig. 9—Steel fiber-reinforced concrete foundation slabs.
Risk and Benefits of Including Fiber-Reinforced Concrete for the Design and Construction of a Driveway

by C. N. MacDonald

Synopsis: The case history presented in this paper describes a small sized project for design and construction of a macro synthetic fiber reinforced concrete (SnFRC) residential driveway with an average grade of 17%. The study highlights risks and benefits of choosing this material for this project. Five almost-equal lengths of SnFRC sections were placed in two groups 11 months apart. The delay between placements allowed for some experience to better analyze and determine if this was the best solution given the customer’s performance criteria and the difficult construction conditions. The design included a high cementitious content mixture with small aggregate, synthetic macro fibers, and air-entraining admixture. The resulting driveway was constructed down hill and has performed well in spite of minimal surface preparations and no jointing or saw cuts in the overlay. Some small cracking has occurred but has been of no consequence because the concrete has been held together by the synthetic fibers. For the construction of this residential driveway utilizing synthetic fiber reinforced concrete, the performance criteria was met, the construction schedule was on time, and the construction costs were significantly lower.

Keywords: fiber; pavement; residential driveway; synthetic fiber-reinforced concrete; ultra thin white-topping.
Clifford N. MacDonald, FACI, is the Director of Engineering for FORTA Corporation. He has a BS and MS in Civil Engineering. He has chemical industrial, design-construction-project management experience that includes many fiber-reinforced concrete projects. Cliff has developed synthetic FRC for many applications, and he has patented, published papers, and presented extensively about FRC.

INTRODUCTION

This case history describes a small sized project for design and construction of a macro synthetic fiber reinforced concrete (SnFRC) driveway and adds details to the risks and benefits of choosing this material for this project. This FRC application is typical and applicable to other larger projects.

A good design for construction has to do with best fit or conformance to the existing conditions and desired outcome by following a generally understood set of activities to establish some constraints and parameters. For a designer, this begins with determining the customer’s needs, wants, and desires. From this then, hard measurable criteria are established to meet the project cost, schedule, and performance. The risks before the project begins are getting enough information about these issues without dictating to the customer the designer’s preferences. When the materials have been chosen and the design is complete, the information is transmitted to construction by contract (legal verbiage about relationship conduct) including specifications (criteria generally and specifically about materials and execution issues on pages, copies, cut sheets, etc.) and graphical representations (drawings, sketches, etc.) of the work to be done. By definition, the amount of these documents is the level of quality control the designer wants for the project. Further risks associated with the construction have to do with labor, equipment, materials, weather, and then also incorporating and ensuring the designer’s intent.

The benefits associated with this entire process are that the process can be duplicated (i.e., using the same design, materials, techniques, etc.). Further benefits may also include additional customers, making a profit, and self-satisfaction having done that work.

PROBLEM OVERVIEW

The house was built in 1961. Sometime before 1989, an asphalt driveway was installed. The severely rutted and cracked asphalt driveway had received no maintenance since the 1989 purchase by the current owners. Traffic (roundtrip and average) on the driveway comprised of weekly refuse trucks and at least once weekly delivery trucks with a once daily auto. The amount of rutting and cracking developed some deep potholes that even bent the rim on an alloy wheel getting a “run at it” to get up the hill.

The sloped hillside driveway is on average 17% and up to 19% in some spots in approximately 250 ft (76 m). This portion of the driveway elevation difference is about 40 ft (12 m). Due to the dense forest surrounding the property and the Minnesota location, the hill frequently becomes snow and ice covered regardless of plowing. Numerous times vehicles have slid off the driveway. Further still, burning rubber while trying to mount the hill and get past the no-go slippery spots has shortened tire life. Traction on the new surface is an issue of concern.

Project issue—performance

Considering the author’s background in cement-based materials, concrete seemed a logical choice for the pavement construction. However, there were other technical details that needed to be considered in terms of performance criteria.

Pavement Performance Criteria—the pavement needed to work (i.e., cover the ground) and be driveable, plowable, textured for minimal slipping, self-defrosting color (if possible due to lower winter sun and heat gain), minimal maintenance, and durable.

Project issue—schedule

The schedule criteria from September of 2005 were that there were two locations on the hill that were worse or had deeper potholes than other portions of the hill. With the impending winter season and subsequent plowing, these worst spots needed to be repaired during a stretch of ‘warmest’ weather in October to allow the concrete to gain strength for subsequent service. The order of FRC placements, from the top of the hill to the bottom, was Placement 3-Placement 1-Placement 4-Placement 2-Placement 5, and each placement was almost of equal length. Placements 1 and 2 were done in October 2005 and the Placements 3, 4, and 5 were done in September 2006.
Project issue—construction cost
The general criteria for construction cost were to have as minimal expense as possible and still meet the schedule and performance criteria. Both asphalt and concrete were considered. The asphalt solution included complete removal of the old asphalt, rebuild the base, and then place a wearing course or replacement surface of asphalt. A local asphalt contractor provided a written estimate of $15,000 for this asphalt solution. A decision was made to go with concrete and not asphalt as the expected costs could be significantly reduced.

DESIGN ISSUES
The driveway is located about 0.6 miles (1 kilometer) from the Mississippi River, and the soils are variable but generally are a sandy – clay – gravel mixture. This also influences the slab ground support conditions and the general erosion and water drainage issues of the pavement. The driveway is generally oriented as follows: top is the north end and down is to the south end, and it curves (or bows outward) to the west. The east side or inside of the curved driveway is where the rainwater generally runs off the driveway and hillside. This run-off has caused some additional erosion issues that have been repaired with large and mixed gravel and some waste concrete at the bottom-flared end of the driveway. The varied ground support may have been the primary cause of the initial pavement deterioration. The two-stage construction, with 11 months between stages, was a significant benefit because the initial design could be evaluated to determine if this construction was viable given the original condition, location, method of construction, and placement conditions.

CONSTRUCTION SECTIONS 1 AND 2
Generally, the driveway pavement was a fiber reinforced concrete (FRC) overlay on asphalt or ultra thin white-topping. Asphalt surface preparation was minimal and consisted only of using a leaf blower to clean off the surface. There seemed to be enough texture in the asphalt surface due to the severe cracking and exposed aggregate to accomplish a significant amount of bonding. However, the use of fibers to hold broken concrete together did not seem to make this an issue because then there would be no displacement of broken pieces such as the asphalt chunks. The potholes were leveled off with the readily available existing materials: graded gravel base and asphalt chunks. The tire ruts were not filled but allowed to add thickness or depth to the overlay at important locations.

Mixture proportions for the FRC material used on the construction job are shown in Table 1. The synthetic fiber dosage was chosen to hold the concrete together and make it tougher and not necessarily to achieve stiffness enhancement in bending or deflection control due to traffic loads. The assumption was that the base was stiff enough and it had been adequately compacted. The synthetic fibers were small in cross section, approximately round, monofilament, and in bulk or presentation twisted. The fly ash in the mixture composition was chosen to increase the interfacial bond with the fibers due to the fact that the fly ash particle size was smaller than cement. The 0.42 water - cementitious material ratio and air-entraining admixture was chosen for durability because of resistance to the severe freezing and thawing conditions of the Minnesota location and the southern sloped exposure of the pavement, suggesting more frequent freezing and thawing cycles. The concrete was batched short on water to allow some field addition as needed. Fibers were added at the work site. No other admixtures were needed or used for workability of the mixture, and the water-cementitious material ratio was not exceeded.

Formwork was minimal again to reduce costs. Wooden stakes were placed just off the old pavement on both sides of the driveway. The stakes were spaced just short of the length of a commercially available plastic pipe length. Plastic pipe was used to set a uniform thickness of the concrete in a wet-screed type of placement. The plastic pipe, rather than steel pipe, was chosen for easy clean up, lightweight, and flexibility to conform to the irregular surface. Three pipes were used, one on each side or edge and one in the middle or crown of the pavement. Short pieces of wood were used to locate the pipe on both sides of the pavement as offsets from the stakes. Concrete was placed with chute discharge on each side of the driveway and pulled and screed by hand using a rectangular cross-section piece of wood about 6 ft (2 m) long. The concrete was placed simultaneously on each side of the center pipe and the length of one pipe before the pipe formwork was pulled down on the sides and out of the middle and reset to the next set of stakes.

With the sloped application, concrete placement was done down hill. This is understood to be the
exact opposite of what is recommended for concrete placed on a slope. However, the concrete delivery truck could not be left at the top of the hill while the concrete cured so there was no alternative and no other access to the driveway. The 2 sections of concrete were about 50 ft long (15 m) and 9.5 ft wide (2.9 m) and a nominal 1.5-inch thick (37 millimeters). Each section took about 45 minutes, and with the commute to the site, the concrete was a bit stiffer with the second section. Water was added to the mixture for the last 25% of the second placement.

Finishing consisted of using a roller bug on the surface to add texture. A roller bug is a round shaped open metal screen on the end of bull float type handles. A roller bug is normally used to bring cream to the surface for finishing by pushing the coarse aggregate down. FRC was also later added to fill the void from pulling the pipe out of center, and the roller bug was used to place-smooth that placement out. Finishing was started too late and the concrete was somewhat stiff in place as evidenced by a few more passes with the roller bug, since the concrete was not as conformable. This and the original poor surface resulted in a bumpy but acceptable ride. Transition areas from the asphalt onto or off the FRC were done by a hand trowel and just sloped or made wedge shaped in cross-section.

Curing was accomplished with clear thin plastic sheeting stretched and pulled down over the side stakes. Water was added by spraying daily under the plastic sheets. The concrete was cured for 5 days. The overnight lows did not go below 40°F (5°C). Daytime highs were about 60°F (15°C).

Construction used 7 cubic yards (5.35 cubic meters) of concrete, and it was all placed within 2 hours of arrival at the job site. Fibers were added at the job. Primarily 2 workers did the concrete placement with some generous assistance by the truck driver. A third worker did some roller bug finishing as the concrete was beginning to set too much before some finishing. No saw cuts or tooled groove joints were incorporated into the slab. The concrete was workable and no other admixtures were used.

**RISK BENEFIT ANALYSIS SECTIONS 1 AND 2**

The schedule was met because the job was done when the weather was warm enough for the concrete to gain sufficient strength before being driven on and before the first snowfall. The manpower work was essentially completed in three days time, 1 man day preparation, 2 man days for 1 day of construction, and 1 man day curing and clean up. The entire project lasted 7 days from preparation to driving on the pavement.

Approximate construction costs included: $800 for concrete, $100 for unskilled laborer, $300 for fiber, and $200 for formwork, leaf blower, plastic sheets, water hoses, and electrical extension cords.

In the summer of 2006, the pavement was viewed for how well it had performed through the winter. There was no evidence of any scaling or freezing and thawing damage. The surface was extremely textured, and overall the ride was a bit bumpy but acceptable as noted earlier due to late finish and rough initial surface. There was evidence, especially after most moisture was dried off the slab, of some small tight crack lines in the pavement. More cracks were evident in the inside of the curve where the ground support was expected to be weaker. The cracks were sufficiently small to be of minimal concern except for one crack about 0.020 inches wide (0.50 millimeters). This crack was located near where the water was added at the completion of the second section. The concrete slopes at the concrete-asphalt transitions were cracked a bit more due to the thinness of the concrete. However, no FRC had dislodged or been displaced but was held together by the fibers.

The design objectives, namely, performance – schedule – cost optimization, were sufficiently met, and were exceeded regarding the cost. The cracking quantity was disappointing, though small in width and not expected to be problematic long term. Further, the general levelness and slight bumpy ride was also a disappointment. However, overall the risk was minimal and controllable, and the benefits were outstanding regarding costs, so the decision was made to finish off by doing remaining sections 3-4-5.

**CONSTRUCTION SECTIONS 3, 4, AND 5**

Generally, all things were the same regarding the previously described construction except for a few details. The construction was done 1 month earlier in the year so the temperatures were warmer. Twice as much concrete was placed using 2 experienced and 2 unskilled workers. Further, the concrete was delivered with the maximum allowable water to not exceed the ratio. This concrete was more workable and flowed easier into place and sped construction but did not run down the hill. The concrete went faster and was out of the trucks within half the time as the previous sections. The roller bug was started earlier and leveled the concrete better because it was much more workable.
Fiber-Reinforced Concrete in Practice 169

With the unusually warm weather, these sections were viewed 3 months after placement. The roller bug texture is better and the concrete surface is less bumpy than the other sections. There is also less cracking. The less cracking might be attributed to the warmer temperatures and higher strength before driving on the pavement. At the bottom of the hill and the last section, there is a bigger bump than anticipated. This was the end of the job, and it is expected that this is where some of the concrete was handled less, slid down, or piled which caused the bump.

CONCLUSIONS

The case history presented in this paper described a small sized project for design and construction of a macro synthetic fiber reinforced concrete (SnFRC) residential driveway with an average grade of 17%. The study highlighted risks and benefits of choosing this material for this project. Five almost equal lengths of SnFRC were placed in 2 groups 11 months apart. The delay between placements allowed for some experience to better analyze and determine if this was the best solution given the customer’s performance criteria and the difficult construction conditions. The design included a high cementitious content mixture with small aggregate, synthetic macro fibers, and air-entraining admixture. The resulting driveway was constructed down hill and has performed well in spite of minimal surface preparations and no jointing or saw cuts in the overlay. Some small cracking has occurred but has been of no consequence because the concrete has been held together by the synthetic fibers.

The overall cost for all the 5 sections of synthetic FRC was nearly 25% of the asphalt estimate. The difference is attributed to not removing the old materials and not needing to mobilize the asphalt equipment with this small project driveway 254 ft (77.4 m). Some additional work is anticipated adjacent and inside the hill portion curve to minimize the rainwater run-off effects at the slab edge and extend the pavement life. Further leveling of the FRC ultra thin white-topping can be considered by topping the existing FRC with another layer of FRC. The surface area and roughness from the roller bug finish would be ideal for bonding a topping FRC layer.

For the construction of residential driveway presented in this paper utilizing synthetic fiber reinforced concrete, the performance criteria was met, the construction schedule was on time and the construction costs were significantly lower. It is concluded that the risk-benefit for performance-schedule-cost using synthetic fiber reinforced concrete was the best fit for this project.

REFERENCES

Table 1—Details of placement, stations, panels, lengths, and cracks

<table>
<thead>
<tr>
<th>Stations uphill</th>
<th>Panel placements</th>
<th>Panel cracks</th>
<th>Panel (feet)</th>
<th>Panel (meters)</th>
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<tbody>
<tr>
<td>ft</td>
<td>m</td>
<td>#1</td>
<td>8</td>
<td>54</td>
</tr>
<tr>
<td>197</td>
<td>60.0</td>
<td>#1</td>
<td>8</td>
<td>54</td>
</tr>
<tr>
<td>89</td>
<td>27.1</td>
<td>#2</td>
<td>3</td>
<td>35</td>
</tr>
<tr>
<td>Total for October 2005 placements</td>
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<td>89</td>
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<td>27.1</td>
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<tr>
<td>254</td>
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<td>143</td>
<td>43.6</td>
<td>#4</td>
<td>3</td>
<td>54</td>
</tr>
<tr>
<td>54</td>
<td>16.5</td>
<td>#5</td>
<td>2</td>
<td>54</td>
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<tr>
<td>Total for September 2006 placements</td>
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<td>165</td>
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<td>50.3</td>
</tr>
<tr>
<td>Total for the entire driveway</td>
<td>20</td>
<td>254</td>
<td>12.7</td>
<td>77.4</td>
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</table>

Table 2—Synthetic fiber-reinforced concrete mixture proportions

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Amount, lb/yd³</th>
</tr>
</thead>
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<tr>
<td>Water (w/cm = 0.42)</td>
<td>355</td>
</tr>
<tr>
<td>Total Cementitious Materials</td>
<td>846</td>
</tr>
<tr>
<td>Cement</td>
<td>564</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>282</td>
</tr>
<tr>
<td>Aggregates Total</td>
<td>2411</td>
</tr>
<tr>
<td>Coarse Aggregate 0.375 in. (9.5 mm)</td>
<td>1519</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>892</td>
</tr>
<tr>
<td>Synthetic macrofiber, twisted monofilament, 2.125 in. (54 mm) long</td>
<td>10</td>
</tr>
<tr>
<td>Air-Entraining Admixture, Volume %</td>
<td>7.34</td>
</tr>
<tr>
<td>Total Weight</td>
<td>3622</td>
</tr>
</tbody>
</table>

Note: 1 lb/yd³ = 0.593 kg/m³.

Fig. 1—Typical rutted and cracked condition of asphalt.
Fig. 2—Placing from chute to wet screed.

Fig. 3—Roller bug, pipe rail, and formwork.
Fig. 4—Section 2 completed SnFRC ultra thin white-topping (UTW).
FRC—Rheology, Fiber Dispersion, and Mechanical Properties

by N. Ozyurt, T. O. Mason, and S. P. Shah

**Synopsis:** The effects of fresh state properties on the fiber dispersion characteristics of fiber-reinforced composites (FRCs) were studied by quantifying fiber segregation. Fresh state properties of concrete mixes were varied using different combinations of a plasticizing agent and viscosity modifier. A self-designed parallel-plate rheometer was used to obtain rheological parameters. Vibration was applied to the specimens and vibration times were varied to understand the effects of vibration on fiber segregation. An electrical characterization method, alternating current-impedance spectroscopy (AC-IS), was used to quantify fiber segregation in specimens. The relationship between fresh state properties and fiber segregation was evaluated.

**Keywords:** AC-impedance spectroscopy; fiber dispersion; rheology; viscosity-modifying agent.
**INTRODUCTION**

It is well known and acknowledged that both the fresh and hardened state properties of cement-based materials are strongly affected by the rheological characteristics of the material (Banfill 2003). Nowadays, researchers carefully tailor mix designs to obtain rheological properties that can lead to the desired mechanical performance. Rheological characteristics of materials are dependent on many parameters (Atzeni et al. 1985) and the properties of each constituent material directly influence the fresh state properties. It is important to understand the relationship between the fresh state properties and materials performance.

In this study, the effects of the fresh state properties of fiber-reinforced cement-based materials on the fiber dispersion characteristics and resulting mechanical performance were studied. Rheological properties of composites were varied using a superplasticizer and a viscosity modifier and a custom-designed and built parallel-plate rheometer was used to measure rheological characteristics. Fiber dispersion characteristics of the specimens were measured and defined using an electrical characterization method (AC-IS). AC-IS is a non-destructive technique that can be used to study various features of cement-based materials. Previous work of the authors showed that AC-IS can be effectively used to monitor fiber dispersion characteristics in cement-based materials when conductive fibers are used. Information about the basics of the method and previous work can be found in (Ozyurt 2006; Ozyurt et al. 2004; 2006a,b; Woo et al. 2005). The performance of different mix designs are described by fiber segregation resistance, placeability and mechanical performance. Conventional concretes and self-consolidating concrete (SCC) were compared.

**EXPERIMENTAL STUDY**

**Materials and mixture designs**

Fiber-reinforced concrete specimens were cast and various vibration times were applied. Two types of steel fibers were used. Short-cut straight steel fibers that were 6 mm (0.236 inches) long and 0.16 mm (0.0063 inches) diameter and longer hooked-end steel fibers that were 40 mm (1.575 inches) long and 0.62 mm (0.024 inches) diameter were used in different mixes. Table 1 shows the concrete mix designs and vibration times. A polynaphthalene sulfonate superplasticizer (SP) was the used in all mixes except for the SCC mix. For SCC, a polycarboxylate-based superplasticizer was used in the study. A viscosity modifying agent (VMA) was also used in selected mixes in this investigation. The river sand had a maximum diameter of 3 mm (0.118 inches) and the coarse aggregate had a maximum diameter of 8 mm (0.315 inches). The water-cement ratio \((w/c)\) was kept constant at 0.40. The fiber loading was 1% of the total volume for all mixes. Mixing sequence and time were similar for all the mixes. Seven groups of concretes were cast. Each mix design is referred with letters (A, B or C) and the vibration time is given as a subscript. For example, A₂ represents the design with superplasticizer and a vibration time of 2 minutes. The mix proportions of the conventional concrete and SCC were as follows, respectively.

\[
\text{Cement : water : fine aggregate : coarse aggregate : VMA (% water weight) : SP (% cement weight) : fiber (% vol.)} = 1 : 0.4 : 2 : 2 : 0.2 : 1 : 1. \quad \text{Cement : water : fine aggregate : coarse aggregate : fly ash : SP (% cement weight) : fiber (% vol.)} = 1 : 0.4 : 1.56 : 1.9 : 0.25 : 0.7 : 1.
\]

Three cylindrical specimens (D=150 mm (5.9 inches) H=300 mm (11.8 inches)) were cast for each mix design.
Matrix resistance \( R_m \) and composites resistance \( R \) were measured and matrix-normalized conductivities \( \sigma_m \) were calculated using Eq. 1. Please note that one AC-IS measurement provides both data:

\[
\frac{R_m}{R} = \frac{\sigma_m}{\sigma_{m0}}
\]  

Fiber content monitoring using AC-IS

One specimen of each group was used for AC-IS measurements. AC-IS measurements were done along the height of the cylinders. Measurements were obtained from 4 different positions from a to d. Stainless steel square electrodes (25 mm (0.984 inches) x 25 mm (0.984 inches)) that were pressed against the specimen were used. Wet sponges were placed under the stainless steel electrodes to ensure a good contact between the specimen and the electrodes. 1 M NaCl solution was used to wet the sponges. Figure 1 shows electrode positions from the top to the bottom of a cylinder specimen. Fiber segregation was evaluated using matrix-normalized conductivity \( \sigma_m/\sigma_{m0} \) values of the specimens. Matrix resistance \( R_m \) and composites resistance \( R \) were measured and matrix-normalized conductivities \( \sigma_m/\sigma_{m0} \) were calculated using Eq. 1. Please note that separate measurements are not necessary to obtain \( R_m \) and \( R_m \), values for the specimens. One AC-IS measurement provides both data.

\[
\frac{R_m}{R} = \frac{\sigma_m}{\sigma_{m0}}
\]  

Splitting tensile tests for mechanical strength

Splitting tensile tests were done to understand the effects of rheology and fiber dispersion on the mechanical performance. One specimen of each group was cut into 4 pieces and splitting tensile tests were conducted on each piece according to a modified version of EN-12390-6. Loading was applied under average lateral displacement control, which was monitored using 2 Linear Variable Displacement Transducers (LVDTs) that had a maximum range of 2.5 mm (0.0984 inches). The LVDTs were fixed onto each side of the specimen and the lateral displacement was monitored. Load and LVDT displacements were recorded for data analysis. Tensile strength of each part was calculated using the equation below:

\[
f_{ct} = \frac{2P}{\pi LD}
\]  

where \( f_{ct} \) is stress (MPa, psi), \( P \) is the maximum load (N, lb), \( L \) is the length of the line of contact of the specimen (mm, inch), \( D \) is the cross-sectional dimension (mm, inch) of the specimen.

RESULTS AND DISCUSSION

Rheology

Viscosity and yield stress values are given in Table 2. As seen in Table 2, the viscosity and yield stress increase with the addition of VMA and the SCC mix has the lowest yield stress and highest viscosity. The yield stress of design C was found to be lower than design B, while the viscosity is higher. This
trend occurs due to the segregation preventing property of the VMA used in this study. The viscosity modifying agent used in this study is specifically designed to lower the segregation of the ingredients in concrete by lowering yield stress while increasing viscosity.

**Fiber segregation**

Table 3 gives fiber content distribution in the specimens. Table 4 shows the results of AC-IS measurements. Comparison of the results of the two experiments is given in the next section.

Matrix-normalized conductivity values given in Table 4 were corrected to exclude the effect of insulating property of coarse aggregates when 6 mm (0.236 inches) fibers were used. The maximum diameter of the coarse aggregates was 8 mm (0.315 inches), while the fiber length was 6 mm (0.236 inches) meaning that some of the aggregates were big enough to block conductive fibers and can therefore, cause a reduction of the concrete matrix conductivity. Further information about correction can be found in (Ozyurt 2006).

**Splitting tensile strength**

Table 5 shows splitting tensile strength values for the parts of the specimens. Bottom parts of the specimens (part d) had more fibers due to fiber segregation (Table 3 and 4) and therefore higher splitting tensile strength.

**Fiber content – normalized-conductivity – splitting tensile strength relations**

Figure 2 shows standard deviations for all the mixes for fiber content, matrix-normalized conductivity and splitting tensile strength (Units of standard deviation of each parameter is given in Figure caption). As seen in the figures, tendencies are very similar for all parameters. Standard deviation in the fiber content distributions can be considered as a measure of fiber segregation. Fiber content in each part would be 1% if no segregation of fibers occurred. Increasing standard deviation means increasing segregation of fibers. Similar results are obtained from normalized-conductivity and splitting tensile strength data. Standard deviation of normalized-conductivity followed similar pattern of changes as that in the fiber content meaning that AC-IS can be used to monitor fiber content in cement-based materials.

**Effect of VMA**

A viscosity modifying agent (VMA) is used in the mixes B and C. Mix B also included superplasticizer. VMA in the mixes B and C was found to decrease fiber segregation to a good extent. The use of VMA increases the viscosity of concrete, yielding a high segregation resistance. However, the yield stress of concrete also increases with addition of VMA, causing placement issues in the specimen. Placement issues generally result in lower mechanical strength and durability. Figure 3 shows standard deviations in fiber content for the mixes A, B, and C, respectively. Standard deviation in fiber content was used as a measure of fiber segregation. Fiber segregation was found to be very high in the specimen A, while it is much lower in the specimens made using VMA. The specimen C had lower segregation than the specimen B. This was due to the fact that the mix B had SP together with VMA and, therefore, lower viscosity than mix C (Fig. 4). Figure 3 show that the designs with VMA are superior to the design with SP by means of fiber segregation resistance, however, this does not mean that the mechanical performance of the specimens made using VMA are superior to the specimens made using SP. This subject is going to be discussed in the following sections.

**Effect of vibration**

Effect of vibration is also important to evaluate fiber segregation in concrete specimens. A vibration table operating at 3600 V/min at 60 Hz of power was used in the study. Vibration was applied at the highest setting with a stroke of approximately 0.015 in. Vibration time was exaggerated to better understand the effects of vibration on fiber segregation. No sign of segregation was seen in the specimens made with VMA even they were vibrated for 8 minutes. However, fibers were severely segregated in mix A when vibration was applied, as seen in Fig 5 (Fiber segregation increased with increasing vibration time). Vibration was necessary to ensure good placement of concrete. Figure 6 compares specimens A (not vibrated) and A (vibrated for 8 minutes) by means of placeability. Specimen A did not have severe segregation, however, placement of concrete was very poor, as can be seen from the Fig. 6. From these
Figures it can be concluded that effort should be made to obtain high segregation resistance together with good placeability to obtain high mechanical performance.

**Combined effects of rheology, vibration and fiber segregation on mechanical performance**

Fiber-reinforced concrete performance is dependent on more than one parameter, including the materials used, fresh state properties, fiber dispersion characteristics and mechanical strength. It is important to consider combined effects of these parameters on the concrete performance as well as individual contributions. In this study, rheological properties and fiber dispersion characteristics can be considered as the most important parameters affecting the concrete performance. These two parameters are strongly related to each other and a mutual effect exist (Fig. 7). Poor dispersion of fibers may cause poor workability and poor fresh state properties give rise to poor fiber dispersion properties. A review diagram is given in Fig. 7 to show how the parameters relate to each other.

In this study, rheology is described by yield stress and viscosity and was found to be strongly affected by the type of admixture used (SP or VMA). The addition of VMA decreases the segregation of fibers, as can be seen from Table 3. The segregation resistance of the designs with VMA is higher than the segregation resistance of the designs with SP. However, this doesn’t mean that the designs with VMA are superior to designs with SP. As it is given in Table 2, the designs with VMA had high yield stress and, therefore, high amount of pores that can cause low mechanical performance (Fig. 8b). Therefore, combined effects of parameters should be addressed as well as individual contributions. In this part of the study, combined effects are considered.

Figure 8 (a) presents the viscosity – fiber segregation relation for mix designs A8, B8 and C8 (made using 6 mm fibers). Standard deviations of the fiber contents throughout the specimen were used as a representation of segregation. As seen in Figure 8 (a), viscosity increases with the addition of VMA and segregation decreases. Figure 8 (b) presents the yield stress – density relation for mix designs A8, B8 and C8. As seen in Figure 8 (b), the yield stress increases with addition of VMA and density decreases with increasing yield stress of the cement pastes, meaning that the VMA designs in this study had a high segregation resistance due to high viscosity but a low density/high pore volume due to high yield stress. A high pore volume typically indicates a lower mechanical strength and negatively affects concrete performance.

An SCC design was compared with a conventional concrete design. Self-consolidating concretes are characterized by a high viscosity and low yield stress. Specimens were cast using 40 mm (1.575 inches) fibers, as shown in Table 1. The high viscosity of the SCC mixture designs ensures a high segregation resistance (Fig. 9a) and the low yield stress provides good placeability (Fig. 9b). As a result, features such as random fiber dispersion, high density and superior mechanical performance are obtained. When the surface quality of the specimens were compared, it was seen that the SCC specimens had no pores on the surface (Fig.6b), meaning a good placement of concrete, while the un-vibrated conventional concrete specimens had many visible pores (Left hand side of Fig.6a). Vibrated conventional concrete specimens had no pores on the surface, but severe segregation inside due to vibration, as is given in Table 2. This means that desired features such as fiber segregation resistance, and good placeability can be obtained using SCC.

**CONCLUSIONS**

Performance of fiber-reinforced concrete specimens was evaluated by fiber segregation resistance and fresh state properties. Fresh state properties were found to be affected by the type of admixture and vibration time. Fiber segregation increased with increasing vibration time when superplasticizer used as an admixture. In other mixtures a viscosity modifying admixture was used to obtain high segregation resistance. Segregation resistance was found to increase with the use of VMA, however, problems related to the placeability of concrete were encountered. Rheological properties of the pastes were investigated to understand the underlying phenomena. Yield stress and viscosity of the pastes were measured using a custom-designed and built parallel-plate rheometer. VMA designs with high segregation resistance and poor placeability were found to have high viscosity and high yield stress. High viscosity ensured high segregation resistance, but high yield stress caused placement issues. This problem was overcome using SCC. High segregation resistance and good placeability/low pore volume were ensured owing to high viscosity and low yield stress of the SCC design used in this study. From the above statements, the
The following results are drawn:

- FRC performance is strongly affected by rheological characteristics and fiber dispersion properties.
- The type of admixture used and vibration time affect rheology and fiber dispersion characteristics.
- AC-IS can be used to non-destructively monitor fiber contents in the specimens.
- Combined effects of important parameters should be considered while evaluating concrete performance.

**ACKNOWLEDGMENTS**

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**REFERENCES**


**Table 1—Concrete mixture designs and vibration times**

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Mix design</th>
<th>Vibration time</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 mm fibers</td>
<td>A₀</td>
<td>No vibration</td>
</tr>
<tr>
<td>6 mm fibers</td>
<td>A₂</td>
<td>2 min.</td>
</tr>
<tr>
<td>6 mm fibers</td>
<td>A₈</td>
<td>8 min.</td>
</tr>
<tr>
<td>6 mm fibers</td>
<td>B₈</td>
<td>8 min.</td>
</tr>
<tr>
<td>6 mm fibers</td>
<td>C₈</td>
<td>8 min.</td>
</tr>
<tr>
<td>6 mm fibers</td>
<td>A₂ + VMA</td>
<td>8 min.</td>
</tr>
<tr>
<td>6 mm fibers</td>
<td>VMA</td>
<td>8 min.</td>
</tr>
<tr>
<td>40 mm fibers</td>
<td>A₂</td>
<td>2 min.</td>
</tr>
<tr>
<td>40 mm fibers</td>
<td>SCC</td>
<td>No vibration</td>
</tr>
</tbody>
</table>

Note: 25.4 mm = 1 inch
Table 2—Yield stress and viscosity values for the pastes

<table>
<thead>
<tr>
<th>Mix Design</th>
<th>Design</th>
<th>Viscosity (Pa*s)</th>
<th>Standard Dev. (Pa*s)</th>
<th>Yield stress (Pa)</th>
<th>Standard Dev. (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP</td>
<td>A</td>
<td>1.20</td>
<td>0.15</td>
<td>46.73</td>
<td>5.05</td>
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<tr>
<td>SP + VMA</td>
<td>B</td>
<td>3.52</td>
<td>0.18</td>
<td>110.65</td>
<td>2.31</td>
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<tr>
<td>VMA</td>
<td>C</td>
<td>5.45</td>
<td>0.29</td>
<td>76.73</td>
<td>2.07</td>
</tr>
<tr>
<td>SCC</td>
<td>SCC</td>
<td>7.22</td>
<td>0.51</td>
<td>19.48</td>
<td>2.80</td>
</tr>
</tbody>
</table>

Note: 1 Pa = 145.03x10⁻⁶ psi, 1 Pa*s = 145.03x10⁻⁶ psi*s.

Table 3—Fiber content (%) distributions in the specimens

<table>
<thead>
<tr>
<th>6 mm fibers</th>
<th>40 mm fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₃ (SP)</td>
<td>A₂ (SP)</td>
</tr>
<tr>
<td>a</td>
<td>0.99</td>
</tr>
<tr>
<td>b</td>
<td>0.99</td>
</tr>
<tr>
<td>c</td>
<td>0.96</td>
</tr>
<tr>
<td>d</td>
<td>0.89</td>
</tr>
</tbody>
</table>

Note: 25.4 mm = 1 inch.

Table 4—Matrix-normalized conductivities of the specimen parts

<table>
<thead>
<tr>
<th>6 mm fibers</th>
<th>40 mm fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₃ (SP)</td>
<td>A₂ (SP)</td>
</tr>
<tr>
<td>a</td>
<td>2.07</td>
</tr>
<tr>
<td>b</td>
<td>2.22</td>
</tr>
<tr>
<td>c</td>
<td>2.43</td>
</tr>
<tr>
<td>d</td>
<td>2.01</td>
</tr>
</tbody>
</table>

Note: 25.4 mm = 1 inch.

Table 5—Splitting tensile strength (MPa) distributions of the specimens

<table>
<thead>
<tr>
<th>6 mm fibers</th>
<th>40 mm fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₃ (SP)</td>
<td>A₂ (SP)</td>
</tr>
<tr>
<td>a</td>
<td>4.8</td>
</tr>
<tr>
<td>b</td>
<td>5.2</td>
</tr>
<tr>
<td>c</td>
<td>5.2</td>
</tr>
<tr>
<td>d</td>
<td>5.4</td>
</tr>
</tbody>
</table>

Note: 1 MPa = 145.03 psi.
Fig. 1—AC-4S experimental setup. (Note: 25.4 mm = 1 in.)

Fig. 2—Variation of standard deviation in fiber contents (%), normalized conductivity, and splitting tensile strength (MPa). Two different views of the same figure are given. (Note: 1 MPa = 145.03 psi.)

Fig. 3—Standard deviation of fiber contents in the mixtures A(SP), B(SP+VMA), and C(VMA).
Fig. 4—Viscosity values for the mixes A, B, and C. (Note: 1 Pa*s = 145.03x10^-6 psi*s.)

Fig. 5—Effect of vibration on the segregation of fibers.

Fig. 6—Surface properties of: (a) conventional concrete specimens $A_0$ (left-hand side) and $A_8$ (right-hand side); and (b) SCC specimen.
Fig. 7—Diagram reviewing the relations between the effective parameters for concrete performance.

Fig. 8—(a) Standard deviation of fiber dispersion in the specimens versus viscosity of conventional concrete and SCC; and (b) density versus yield stress of conventional concrete and SCC. (Note: 1 Pa = 145.03 x 10^{-6} psi; 1 Pa*s = 145.03 x 10^{-6} psi*s.)

Fig. 9—Comparison of CC and SCC specimens that were made using 40 mm (1.57 in.) fibers: (a) standard deviation of fiber dispersion in the specimens versus viscosity of conventional concrete and SCC; and (b) density versus yield stress of conventional concrete and SCC. (Note: 1 Pa = 145.03 x 10^{-6} psi; 1 Pa*s = 145.03 x 10^{-6} psi*s.)