

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

by D. M. Brodowski, M. G. Katona, and J. A. Pope

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.

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ACI member **David Brodowski** is the Senior Bridge Engineer with the Montgomery County Engineer's Office in Dayton, OH. His previous experience as a Product Manager for CONTECH Construction Products Inc. focused on the use of precast concrete buried structures in civil infrastructure. He received his MS in civil engineering at the University of Cincinnati in 2005, with a thesis on developing an appropriate steel fiber-reinforced concrete for applications to buried structures.

Michael G. Katona, PhD, recently retired as Professor and Chair from the Department of Civil and Environmental Engineering at Washington State University. He now serves as a private consultant specializing in numerical analysis techniques applied to soil-structure interaction problems. He is the principal author of the computer programs sponsored by FHWA and AASHTO that are used world wide for the design and analysis of buried culverts and structures.

Jacob A. Pope graduated from the University of Cincinnati in 2004 with a BS in civil engineering. His master's thesis focuses on correlating previous steel fiber concrete research to full-scale load testing and modeling. He currently promotes, designs and supplies precast concrete buried structures for Precast Forum, LLC in Wildwood, MO.

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:

- e_c = compressive strain at initial ultimate strength of concrete
- e_y = compressive strain at initial plastic yielding of concrete
- e_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:

- $f_t = E_c e_t$ = tensile stress when concrete abruptly cracks (stress released)
- $f_y = E_c e_y$ = 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:

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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 $df/de = E_c$ (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 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.

- $f_t = E_c e_t$ = tensile stress when concrete initially cracks (typically the same value as plain concrete).
- $E_s = -f_t / (e_r - e_t)$ = 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 $M_t = f_t b h^2 / 6$ is the

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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, $m(x) = M(x)/M_p$, 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, $\underline{K}$, 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 $\underline{K}_b$ and axial stiffness geometry $\underline{K}_a$ as,

$$\underline{K} = (E_c I^*) \underline{K}_b + (E_c A^*) \underline{K}_a$$

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: $A^* = (1/E_c) \int_A E^*(y) dA$
2. Neutral axis: $y^* = (1/E_c) (\int_A y E^*(y) dA) / A^*$
3. Moment inertia: $I^* = (1/E_c) \int_A (y - y^*)^2 E^*(y) dA$

$E^*(y)$ 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 $E^*(y) = (f_i - f_{i-1}) / (e_i - e_{i-1})$, where f_{i-1} , e_{i-1} are the known stress and strain values for load step $i-1$, e_i is the current iterative prediction for strain at load step i , and f_i is determined from the SFRC model based on the current strain estimate e_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 of 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 were 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 were 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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Table 1—Neutral axis location and internal moment values as a function of x and r

Range of x -multiplier ($e^* = xe_t$)	Neutral axis location ($y^* = zh$)	Internal Moment ($M_t = f_t bh^2/6$)
Linear range $0 \leq x \leq 1$	$z = 1/2$	$M(x) = M_t x$
Softening range $1 \leq x \leq r$	$z(x) = ((1+a)^{1/2} - 1)/a$ where, $a = (x-r)/(r-1)/x^2 - 1 + (2r-x-1)/(r-1)/x$	$M(x) = M_t [2(1-z)^3 x/z + z^2(3r-2x-r/x^2)/(r-1)]$
Rupture range $r \leq x \leq \infty$	$z(x) = 1/(1 + r^{1/2}/x)$	$M_t [2x(1-z)^3/z + z^2(r^2+r)/x^2]$

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

r -value	1	10	100	1000	10,000
M_{max}/M_t	1.0	1.6	2.2	2.6	2.8

Table 3—Material properties of the precast arch units

UNIT I.D.	Mix Proportions per 1 yd ³ , SSD Condition							14-day	28-day
	Gravel 8's	Sand	Cement	Water	Superplasticizer	Visc. Modifier	*Steel Fibers	f_c	f_c
	(lbs.)	(lbs.)	(lbs.)	(lbs.)	(oz.)	(oz.)	(lbs.)	psi	psi
1	1360	1570	702.5	425	84	14	0	7918	9535
2	1400	1540	697.5	409	98	14	100	6111	6575
3	1370	1590	705	392	112	21	135	5148	5485
4	1370	1550	717.5	392	126	21	80	6417	6883
5	1370	1570	710	392	84	14	80	6995	7783
*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									

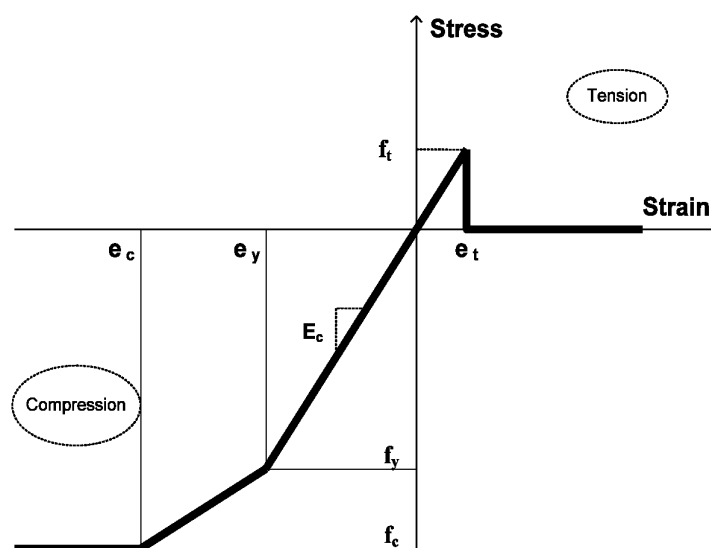


Fig. 1—Plain concrete stress-strain model (virgin curve).

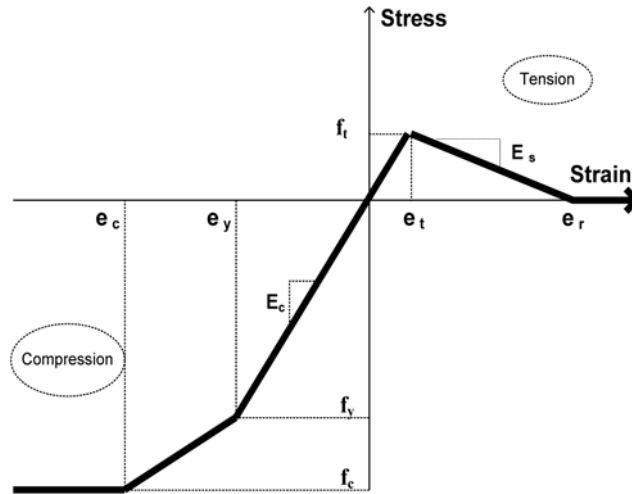


Fig. 2—SFRC stress-strain model (virgin curve).

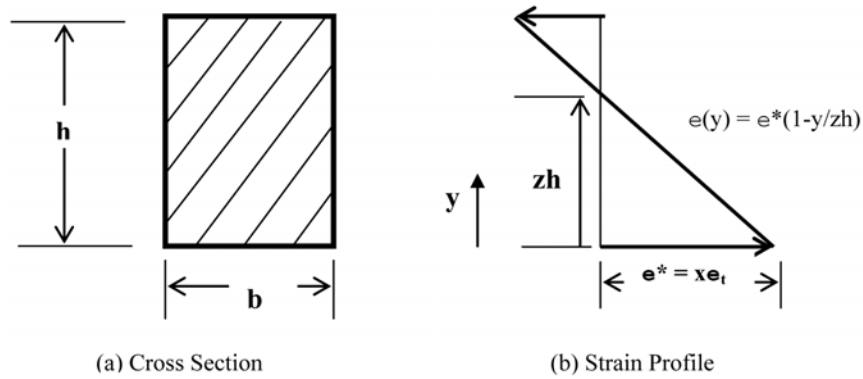


Fig. 3—Cross section and strain profile for development of moment capacity.

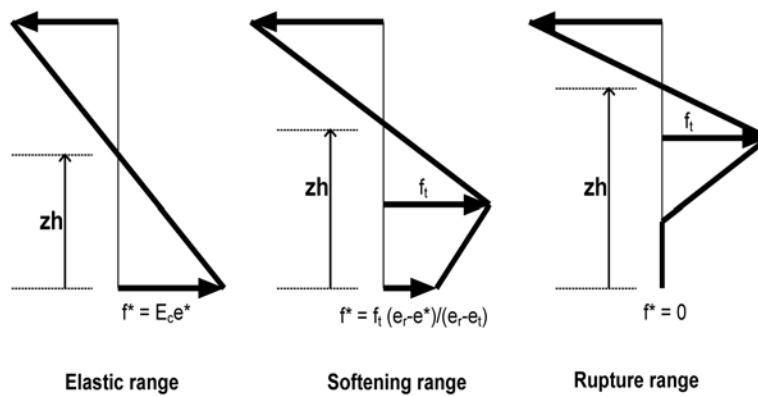


Fig. 4—Stress profiles resulting from the input strain profile in Fig. 3.

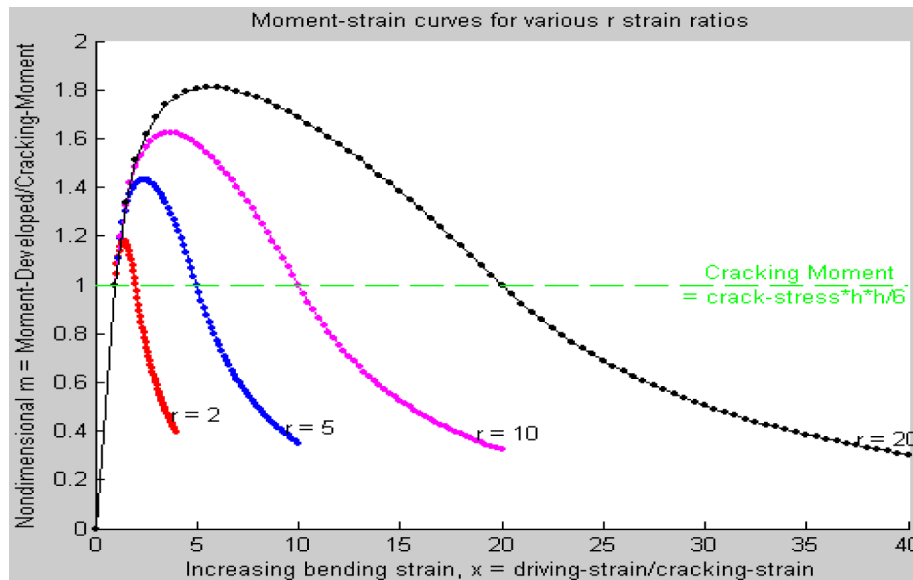


Fig. 5—Moment ratio versus applied strain for a family of r-values.

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)

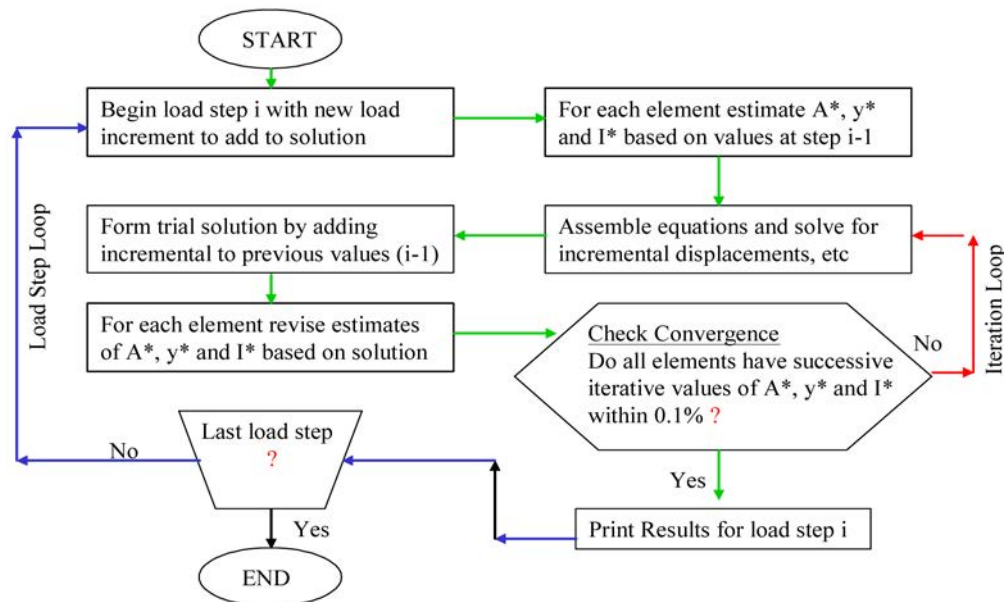


Fig. 6—Solution Strategy for SFRC model in CANDE program.



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.

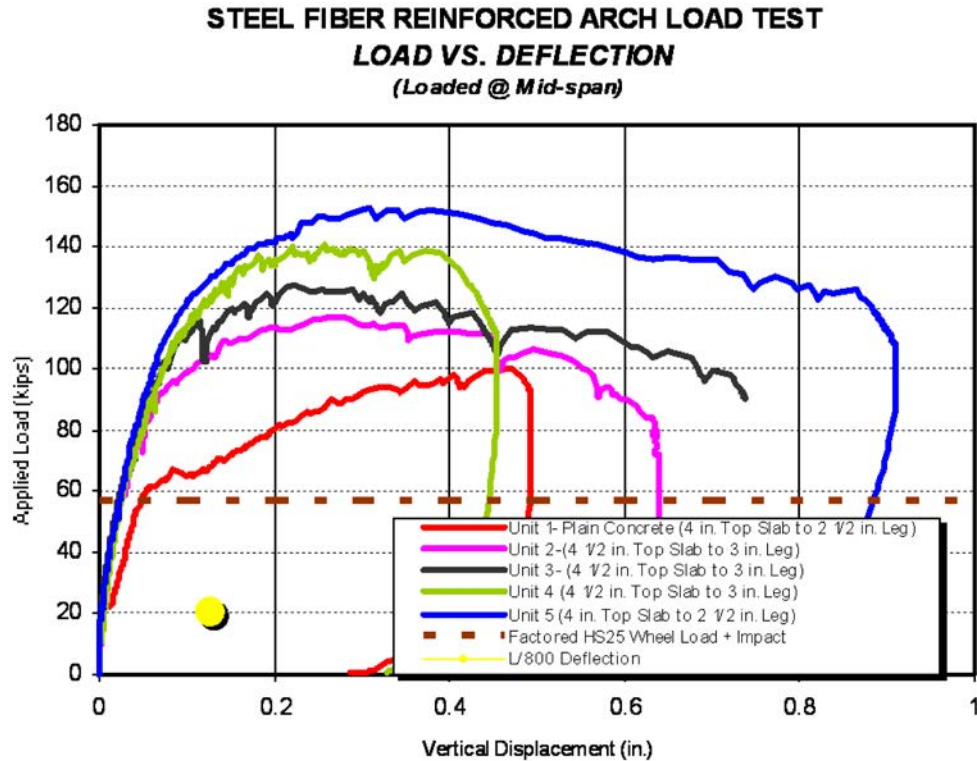


Fig. 11—Applied load versus vertical displacement results at midspan of arch unit for units 1 through 5.
(Note: 1000 lbf = 1 kip = 4.44 kN; 1 in. = 25.4 mm.)

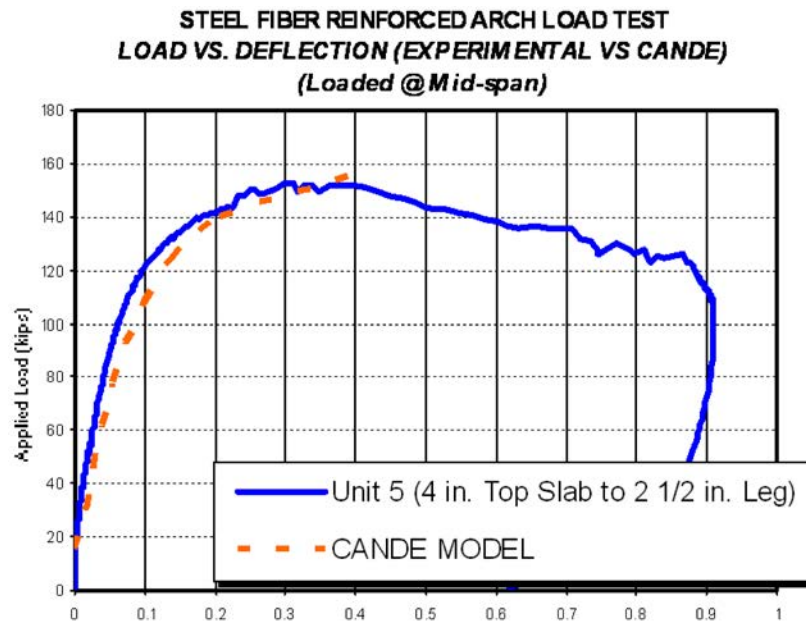


Fig. 12—CANDE model results compared to experimental results for unit 5.
(Note: 1000 lbf = 1 kip = 4.44 kN; 1 in. = 25.4 mm.)



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 $\leq 5 \text{ kN/m}^2$ 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

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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 $h = 325$ 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/m² (0.59 psi). The distributed loading was applied in four steps with increments of 10 kN (2.25 kip), or 1.02 kN/m² (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

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terminated, at which point the panel had exhibited a ductile failure mode (see Figure 10 and Figure 11). The equivalent ultimate moment capacity is $M_u = 110$ 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.

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Table 1—Material properties of PVA-ECC

Flexural strength (ASTM C 1018)	16 MPa (2321 psi)
Tensile strength	6 MPa (870 psi)
Tensile strain capacity	4%
Compressive strength (ASTM C 39)	60 MPa (8702 psi)
Crack opening (at tensile strength)	0.200 mm (0.007874 inches)
Shrinkage (ASTM C 157)	0.04%
Density	2000 kg/m ³ (125 pcf)

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

	Specimen S1			Specimen S2			Specimen S3			Specimen S4		
	Analy.	Numer.	Actual	Analy.	Numer.	Actual	Analy.	Numer.	Actual	Analy.	Numer.	Actual
Deflection [mm]												
At 4.0 kN/m ²	17.9	15.7	19.4	17.9	15.2	17.2	14.2	9.9	11.7	12.8	9.6	10.2
Span / defl.	447	510	412	447	526	465	563	808	684	625	833	784
Frequency [Hz]												
At self weight	7.23	8.12	8.20	7.10	8.22	8.40	7.74	9.85	10.20	7.97	9.99	9.85
At 4.0 kN/m ²	3.39	—	3.80	3.37	—	4.00	3.76	—	4.80	3.93	—	5.00
Stresses [MPa]												
$\sigma_{\text{ECC, mid-span}}$	4.7	3.8	—	4.7	3.8	—	5.2	3.2	—	5.6	3.2	—
$\sigma_{\text{y, mid-span}}$	149.5	114.2	143.7	150.8	114.0	132.8	108.4	64.1	70.9	105.8	48.7	—

* 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.

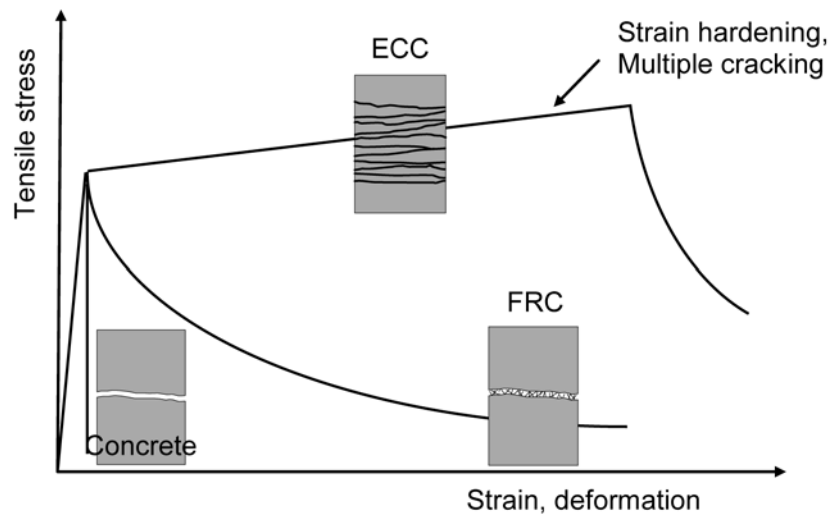


Fig. 1—Tensile stress-strain behavior of cementitious matrixes.

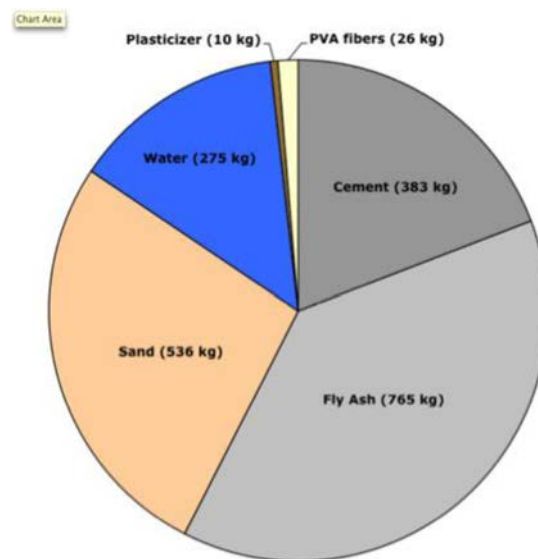


Fig. 2—Constituents per m^3 of PVA-ECC. (Note: 1 kg = 2.204 lbs.)

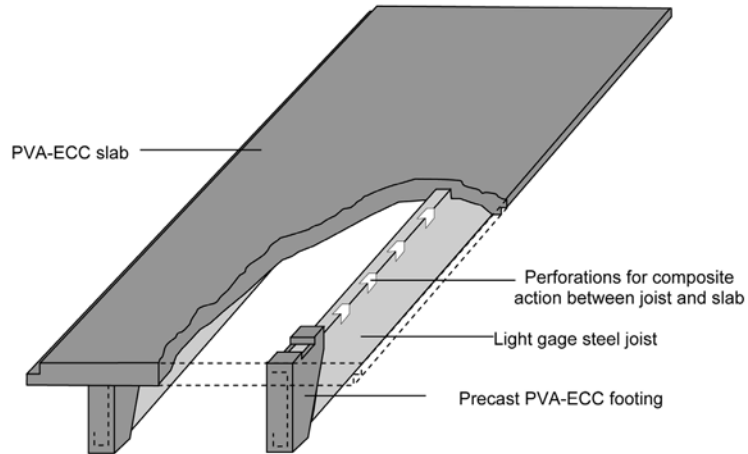


Fig. 3—Schematic of integrally precast ECC floor panel.



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

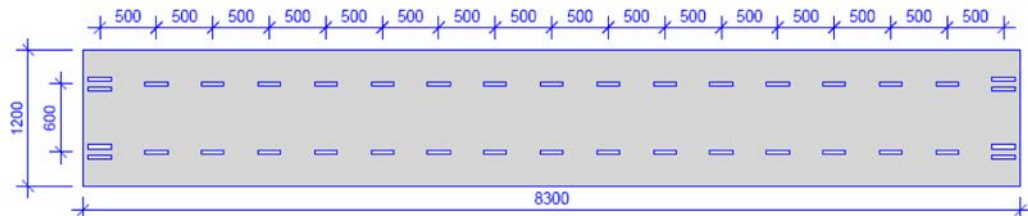


Fig. 5—ECC deck element layout and positioning of cast-in anchors, bottom side shown (all dimensions shown in the figure are in millimeter units; 25.4 mm = 1 in.).

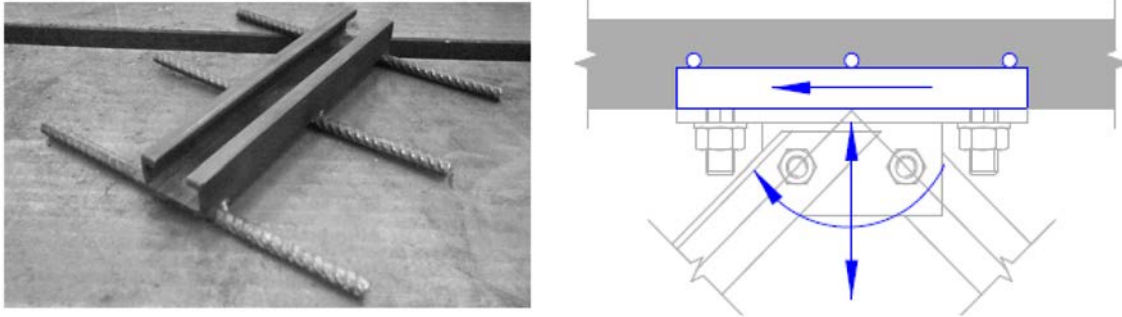


Fig. 6—Left: Cast-in anchor. Right: Resultant forces expected in cast-in anchor, forces shown in vertical and longitudinal direction.

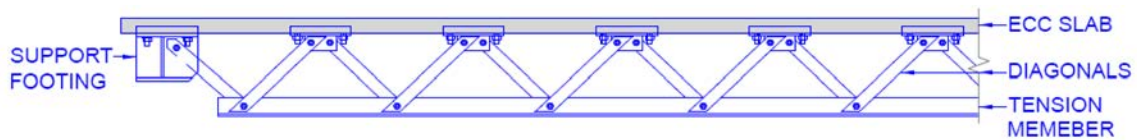


Fig. 7—Longitudinal cross section of truss structure connected to ECC deck panel.



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

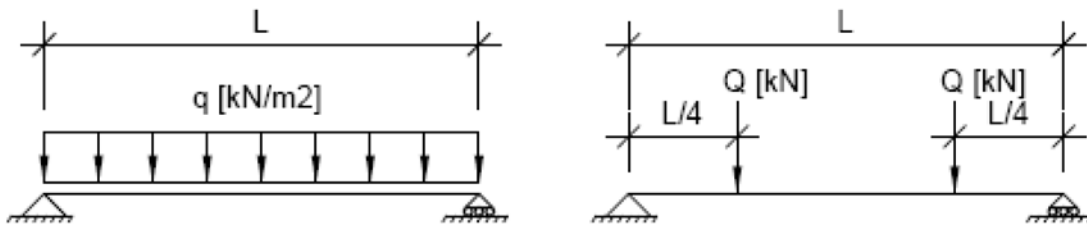


Fig. 9—Load setup for SLS loading (left) and ULS loading (right).

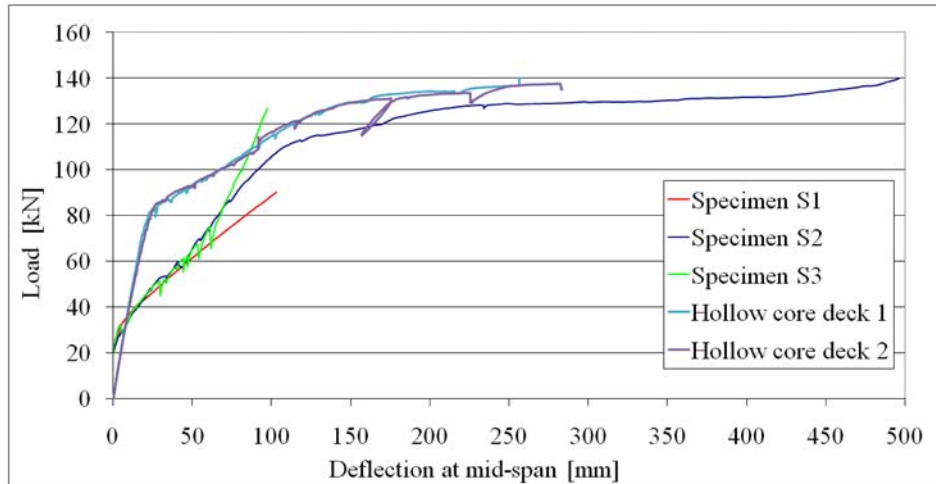


Fig. 10—Load-deflection behavior of tested specimens and integrally cast ECC panel. (Note: 1 kN = 0.2248 kip.)



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

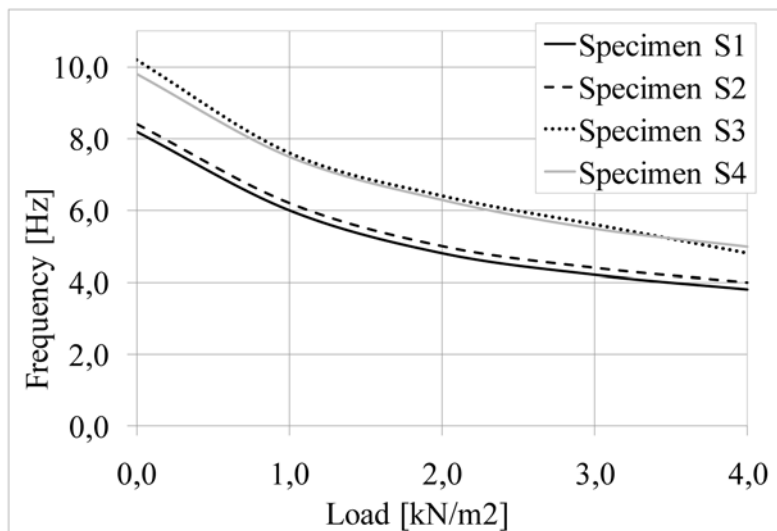


Fig. 12—Measured natural frequency as a function of applied service load. (Note: 1 kN/m² = 0.14504 psi.)

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.

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 alkalis 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

$\phi = 0.75$, strength reduction factor

S = Shape factor

$f_u' =$ Assumed aged ultimate flexural strength

and

$f_u' =$ the lesser of:

$$f_{uy}' = f_{yr}(1 + t V_y)$$

$$f_{uu}' = 1/3 f_{ur}(1 + t V_y)$$

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

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 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 GFRP panels molded to the shape of the sewer, which is usually oval (Fig.18).

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

(a) GFRP has sufficient flexural strength that panels can be designed thin, usually 3/8" to 1/2" 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 GFRP, described above, assures this.

The manufacture of the panels takes advantage of another characteristic of GFRP in which flat uncured sheets of GFRP 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 GFRP 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 GFRP 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 GFRP in the open molds. The GFRP is 1/2" (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 sides are supported until the GFRC hardens, about 24 hours. After this time the vertical sides are dropped back to the horizontal (Fig.24). The GFRC unit can then be easily de-molded and stored until fully cured (about 28 days). Fig.25 shows the finished permanent beam form. Note the cut-out in the base of the form which is to allow the steel reinforcing of the column to be tied into the reinforcing for the beam.

The permanent column forms are made in half sections in plastic molds (Fig.26). After spraying the GFRC top, a ½" (12 mm) thickness two half molds are bolted together in such a way that the two uncured GFRC half sections will bond together. Some finish work is required on the joint to make sure that it leak proof.

The hollow columns are shipped to the job site and erected around the steel reinforcing bars for the structural concrete (Fig. 27). The beam forms are then installed on the column forms (Fig.28). After the beam reinforcing has been laid in the form the column and beam forms are filled with concrete to form the support structure of the hotel.

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 be 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.

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2. Peaston, C., and Cather, R., "Properties of 1974 Cem-FIL GRC," 13th International GRCA Congress, Arup Research & Development, UK, Oct. 2003.
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Fig. 1—San Francisco towers, San Francisco.

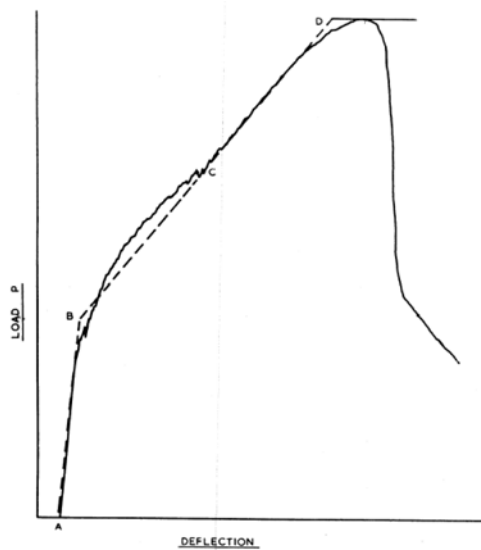


Fig. 2—Typical load-deflection curve for GFRC.

Appendix B – Limiting Stress Determination

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

Yield		Ultimate	
842	1210	2269	2665
1282	1322	2961	2792
1199	1114	2764	2400
1137	1177	2767	3148
1234	1187	2864	2886
1126	1298	2533	2698
1100	992	2576	2107
1271	1330	2876	2686
1256	1103	2861	2552
1112	1264	2796	2860

Average yield strength, $f_{yr} = 1178$ psi

Standard Deviation, $\sigma_y = 119$ psi

Coefficient of Variation, $V_y = \sigma_y / f_{yr} = 119 / 1178 = 0.101$

$$f'_{uy} = f_{yr} (1 - t V_y) \quad (5-9)$$

$$= 1178 [1 - (2.539)(0.101)]$$

$$= 876 \text{ psi}$$

Average ultimate strength, $f_{ur} = 2703$ psi

Standard deviation, $\sigma_u = 243$ psi

Coefficient of Variation, $V_u = \sigma_u / f_{ur} = 243 / 2703 = 0.090$

1 psi = 0.006895 MPa

$$\text{and } f'_{uu} = 1/3 f_{ur} (1 - t V_u) \quad (5-10)$$

$$= 1/3 \{2703 [1 - (2.539)(0.090)]\}$$

$$= 694 \text{ psi}$$

controlling f'_{uu} is smallest of: $[f'_{uy}]$ 876, $[f'_{uu}]$ 694, and

$$[f'_{max}] 1200 \quad (5-9, 5-10, \& 5-11)$$

$$\therefore f'_{uu} = 694 \text{ psi}$$

For single skin in flexure:

$$f_u = \phi s f'_{uu} = (0.75) 1 (694) = 521 \text{ psi} \quad (5-8)$$

For box section flexure: $f_u = \phi s f'_{uu} = (0.75) (0.5) (694)$

$$= 260 \text{ psi} \quad (5-8)$$

For tension: $f_u = 0.4 \phi f'_{uu}$

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

Quality control notes:

The average of all sets of three consecutive yield strength tests must be not less than the f'_{uy} 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'_{uu} used in design [694 x 3 or 2082 psi].

No individual yield strength test shall be less than 90% of the f'_{uy} 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'_{uu} used in design [3 x 0.9 x 694 or 1874 psi].

Fig. 3—Limiting stress determination.

GFRC Skin to stud frame connection is made through a Flex Anchor, fabricated from 1/4" or 3/8" steel rod.

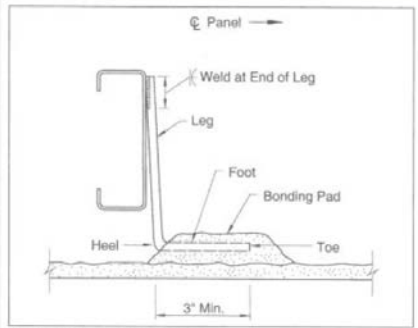


Fig. 4—Flex anchor connection. (Note: 1 in. = 25.40 mm.)

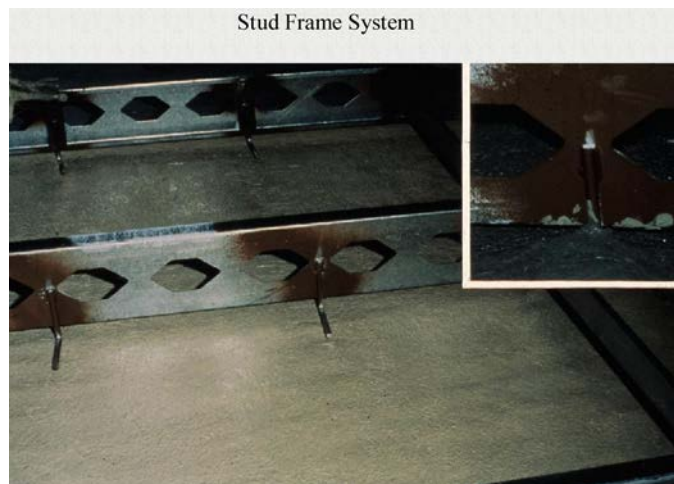


Fig. 5—Flex anchor connection.

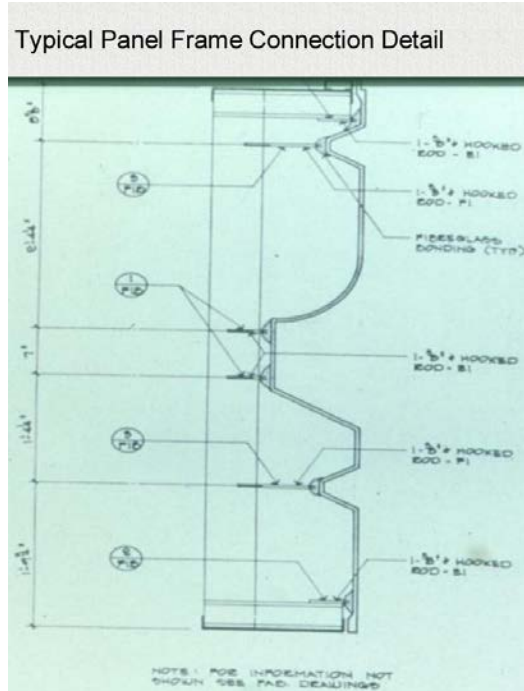


Fig. 6—Typical panel frame connection to GFRC skin.

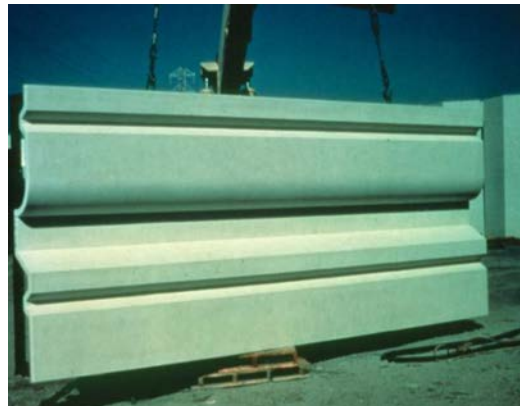


Fig. 7—GFRC architectural panel.



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—GFRF permanent beam form.



Fig. 21—Steel folding forms for GFRF 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.

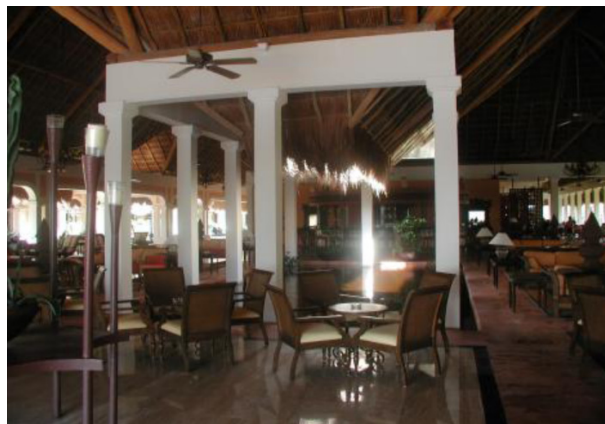


Fig. 29—Finished column and beams used as decorative feature in hotel restaurant.

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 life-cycle 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 (4645 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 (4645 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 and slightly better than Type K shrinkage-compensating 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.

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.

REFERENCES

1. ACI Committee 223, "Standard Practice for the Use of Shrinkage-Compensating Concrete (ACI 223-98)," American Concrete Institute, Farmington Hills, MI, 1998.
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.

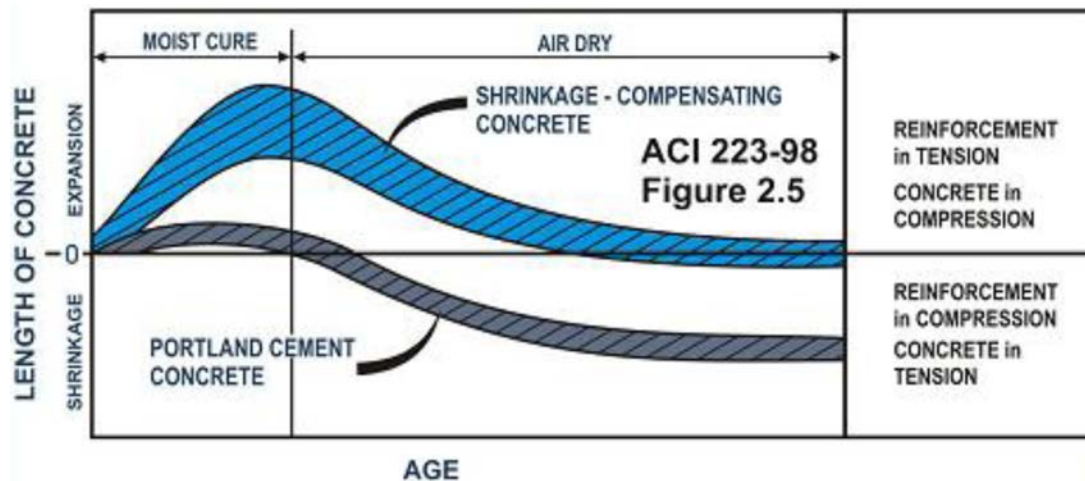


Fig. 1—Shrinkage-compensating concrete versus portland cement concrete from ACI 223.¹ Portland cement concrete is static for 7 days then goes into tension and cracks. Type-K Concrete expands slightly for 7 days but, more importantly, the reinforcement goes into tension and the concrete goes into compression.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

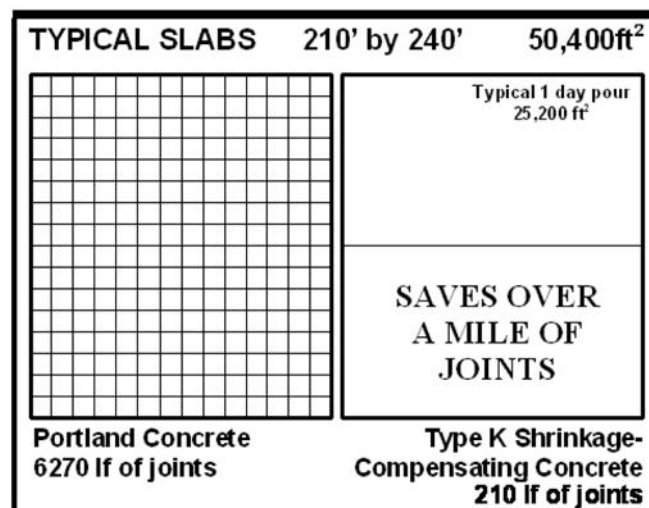


Fig. 2—Save over a mile of joints. More than one mile of joints are eliminated from this slab, which is the size of an average grocery store, by using Type K shrinkage-compensating concrete. This means over one mile of joints that do not have to be sawcut, do not have to be joint-filled, do not have 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.)