Practical Information on the Use of High-Performance Concrete for Highway Bridges

High-performance concrete can offer better quality and greater durability economically but it still requires careful application and testing.

MICHAEL F. PRAUL

The transportation arena is in the midst of a quality revolution. Continuously improving the quality of the product is an integral part of a highway engineer's daily work. In the concrete area, the use of high-performance concrete (HPC) is the best way to achieve this goal for bridges. It is fortunate that this proven technology is so readily available and so economically viable to use.

As the technology spreads to state highway agencies nationwide, current concrete practices need to be re-examined. Concrete has served the construction industry very well for many years; it is a material that can be designed, constructed and used in a wide variety of applications. Concrete is capable of providing the desired service even when all recommended practices have not been followed. However, the move from traditional concrete to HPC requires a concurrent return to better concrete practices. Don Streeter, from the New York State Department of Transportation (NYSDOT), has written, "The attention to detail is what makes the HPC perform so much better than conventional concrete." If the desire now is for a consistently outstanding concrete product, then that will require improvements not only in structural design methods, materials selection and mix designs but also that will require changes in construction practices.

The Need for HPC

In 1992, the National Quality Initiative sought to identify what the traveling public wanted and expected from its highway system. One of the messages that came through loud and clear was the mandate to "get in, get out and STAY OUT." The public realizes that the nation's highway infrastructure is in need of maintenance and repair and drivers are will-

ing to tolerate, to some degree, the delays caused by highway construction work. However, they are not willing to tolerate highway facilities that do not provide an appropriate service life. For those ho work with concrete, the "STAY OUT" mandate readily leads to the use of HPC.

For several decades, the highway industry in the United States has been able to design and produce good quality concrete and state departments of transportation (DOTs) have been able to build good quality concrete bridges. However, "good" is no longer sufficient and the emphasis now is on striving to provide the highest quality concrete possible for transportation facilities. *All* transportation concrete needs to be HPC in order to meet the expectations of the tax-paying public. Therefore, more and more state highway agencies are beginning to use HPC technology.

The need for HPC in highway transportation applications is most clearly demonstrated by looking at the economic effects of corrosion. Recent research done by the Turner-Fairbanks Highway Research Center has estimated that the annual cost of corrosion in the United States is $440 billion; of that amount, $7.4 billion is attributable to highway bridges. While this figure does include steel as well as concrete bridges, the magnitude is obviously significant. Bridge decks are a particularly important item to analyze when evaluating the potential benefit of HPC use. An analysis done by the Federal Highway Administration (FHWA) showed that corrosion is the single biggest factor in bridge deck deterioration. The deck is the part of the bridge that most affects ride quality and is generally the most costly to repair, in terms of both dollars and impact on the driving public. The use of HPC will slow this deterioration and extend bridge life.

Having moved into the post-interstate highway construction era, the luxury of rebuilding the infrastructure on a 50-year timeframe no longer exists; the national economy has grown dependent on having reliable and longer-lived transportation facilities. The AASHTO Load and Resistance Factor Design (LRFD) bridge design specifications now call for a 75-year life. More and more agencies are aiming for this and beyond, up to a 100-year life for concrete structures, which can only be achieved with the use of HPC.

Congress has recognized the need to support more widespread implementation of high-performance materials in highway bridges. The Transportation Efficiency Act for the Twenty-First Century established the Innovative Bridge Research and Construction (IBRC) Program. The IBRC Program is a six-year (1998–2003), $102 million program that provides for the construction of bridges using high-performance materials. The program is aimed at not only achieving higher quality bridges, but it is also aimed at introducing state highway agencies to new materials to help spread the experience and knowledge base. As of April 2001, 52 bridges nationwide have been approved to be constructed or rehabilitated using HPC.

While this number, in and of itself, indicates an increase in the use of HPC for bridges, it does not include many other bridges and bridge members that have been designed and constructed using HPC principles. Many state highway agencies have utilized HPC principles in bridge deck overlays for years. The use of specialty concretes led the way in this regard as individual state DOTs developed unique mixes and used special materials to achieve a dense, low-permeable concrete to prevent chloride ingress and extend deck life. However, there is a continued demand to expand on the use of HPC to provide more durable bridges and to get the best value for the highway dollar.

What Is HPC?

Some confusion exists in the highway industry as to what HPC actually is and what the term means. An appropriate definition of HPC for the industry should be broad enough to encompass a relatively wide range of performance parameters while still being narrowly focused enough to provide criteria that can be readily incorporated into public agency specifications. As more and more states are incorporating performance-related specifications for highway industry work, an appropriate definition for HPC should lend itself to performance-related criteria.

TABLE 1.
FHWA Strength Definition Parameters for HPC

Performance Characteristic	FHWA HPC Performance Grade 1	2	3	4
Strength (AASHTO T-22)	6 ≤ X < 8 ksi	8 ≤ X < 10 ksi	10 ≤ X < 14 ksi	X ≥ 14 ksi
Elasticity (ASTM C469)	28 ≤ X < 40 GPa	40 ≤ X < 50 GPa	X > GPa	
Shrinkage (ASTM C157)	800 > X ≥ 600	600 > X ≥ 400	400 > X	
Creep (ASTM C512)	75 ≥ X 60 μs/MPa	60 ≥ X > 45 μs/MPa	45 ≥ X > 30 μs/MPa	X ≤ 30 μs/MPa

The American Concrete Institute (ACI) defines HPC as concrete that meets special performance and uniformity requirements that cannot always be obtained using conventional ingredients, normal mixing procedures and typical curing practices. These requirements may include the following:

- ease of placement and consolidation without affecting strength;
- long-term mechanical properties;
- early high strength;
- toughness;
- volume stability; and,
- longer life in severe environments.

Certainly this definition provides a global picture of what might be expected from HPC but it does not provide a definition that lends itself to being readily incorporated into specifications.

The Strategic Highway Research Program (SHRP) defined HPC as a concrete having a maximum water/cementitious materials (w/cm) ratio of 0.35 *and* a minimum durability factor of 80 percent (as determined by ASTM C 666, Procedure A) *and* a minimum strength of one of the following:

- 3,000 pounds per square inch (psi) within 4 hours; *or*,
- 5,000 psi within 24 hours; *or*
- 10,000 psi within 28 days.

While this definition does provide some performance-related criteria, it is not broad enough to incorporate many of the performance parameters indicated in the ACI definition. The use of a w/cm ratio may not be the most appropriate specification for general use when defining HPC; many excellent concretes have been produced, and have been routinely used, with higher ratios than 0.35.

Tables 1 and 2 show the FHWA's definition of HPC including the appropriate test procedure used for each performance criteria. Note that the current definition includes only properties of the hardened concrete. Currently, the FHWA is considering expanding the definition to include fresh concrete properties, thereby acknowledging the importance of some of these properties for constructability purposes and more closely paralleling the spirit of the ACI definition.

In keeping with the concept of defining HPC through the use of measurable parameters, Tables 1 and 2 define both strength and durability criteria. It is important to understand that it is not necessary to require all the criteria for all concrete. The desired performance should be determined based on the structural member and its service environment. For example, a bridge deck may require Performance Level 3 for permeability but only Performance Level 1 for compressive strength; a prestressed girder might require Performance Level 2 for permeability and Performance Level 3 for strength. If, in service, the concrete will not be exposed to abrasion, or if the owner does not wish to specify or test for abrasion resistance, it need not be specified.

TABLE 2.
FHWA Durability Definition Parameters for HPC

Performance Characteristic	FHWA HPC Performance Grade 1	2	3	4
Freeze/Thaw Durability (AASHTO T-161)	60% ≤ X < 80%	80% ≤ X < 100%		
Scaling Resistance (ASTM C672)	X = 4,5	X = 2,3	X = 0,1	
Abrasion Resistance (ASTM C944)	2.0 > X ≥ 1.0	1.0 > X ≥ 0.5	0.5 > X ≥ 0	
Rapid Chloride Permeability (AASHTO T-277)	3,000 ≥ X > 2,000	2,000 ≥ X > 800	800 ≥ X	

Several states have developed their own definitions of what constitutes HPC based on their applications and their acceptance testing programs. These definitions are not in conflict with the definition used by FHWA; in fact, quite the opposite is true. Although the actual numbers used in the parameters may change based on local conditions, the test methods are generally the same as those in the FHWA definition. The goal remains very much the same — *i.e.*, to provide a better-quality, longer-lasting concrete.

Pozzolans

In order to provide the desired concrete performance, the materials to be used as well as what other materials might be available should be examined. If most engineers were asked to list the constituents in a concrete mix, the answer would most assuredly be "cement, water, sand and stone." With the intent of providing high-performance–type concrete, these ingredients may not be enough, in and of themselves, to provide the desired in-service performance and durability. In the future, the constituents of a highway/bridge concrete mix should be cement, water, sand, stone and a *pozzolan*. Pozzolans are siliceous materials that are not reactive on their own. However, when mixed with calcium hydroxide (lime) and water, they react to form additional cementing compounds in the concrete. They dramatically improve the performance of concrete and an increase in their use is a natural outgrowth of the trend toward HPC. The most commonly used pozzolanic materials are fly ash and silica fume. Ground granulated blast furnace slag (GGBFS) is also a commonly used mineral admixture. (Technically, GGBFS is not a true pozzolan since it has hydraulic cementitious properties itself. However, it does contribute to the concrete through the pozzolanic reaction and, therefore, should be discussed in the same context as the true pozzolans.)

The use of pozzolans in concrete is not new. Many structures built by ancient Romans that remain standing to this day included pozzolanic volcanic ash in their concrete. In more modern times, Scandinavian countries have made use of pozzolanic materials for several decades. The Norwegians have been making routine use of pozzolans and recently made pozzolanic concrete the standard for all bridge construction. In Finland, 75 percent of the cement used is a blend of Portland cement with a pozzolan.

When Portland cement hydrates, the primary products are calcium-silicate–hydrates, calcium hydroxide and heat. Calcium-silicate–hydrates form the gel-like substance that furnishes the strength and durability to the concrete. Calcium hydroxide, which may constitute as much as 25 percent of the final volume of the paste, is a detrimental product of the reaction and provides no benefit to the concrete. It is a cubicle compound that tends

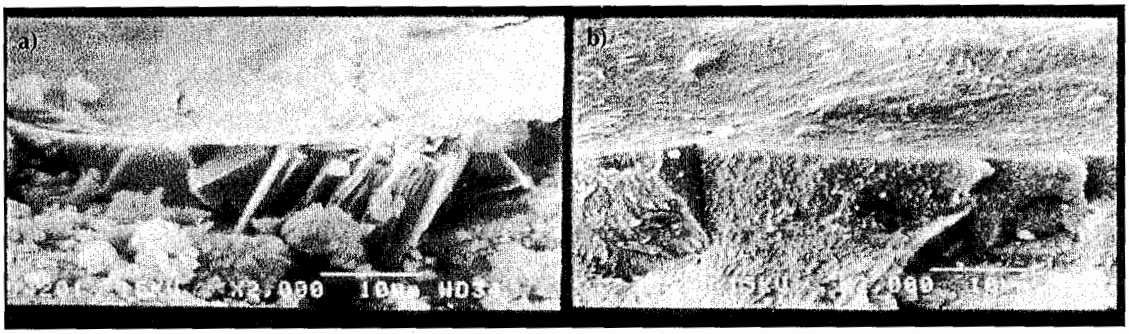

FIGURE 1. Electron microscope image of a) "regular" concrete and b) a pozzolanic reaction.

to form in channels throughout the paste. These channels increase the permeability of the concrete and provide a means for water and other chemicals to gain access to the concrete and reinforcing steel. Over time, this material leaches out of the hardened concrete and forms the stalactites often seen on the underside of concrete structures.

Figure 1 is an electron microscope photo of an ordinary Portland cement concrete versus a pozzolanic concrete. The ridge present in both pictures is an aggregate pocket. The concrete on the left shows the calcium hydroxide channel formed along the material boundary. The picture on the right demonstrates how the pozzolan has reacted with the calcium hydroxide and formed additional calcium-silicate–hydrates at the aggregate interface.

The nature of the pozzolanic reaction, and the physical properties of the pozzolanic materials, have benefits and consequences to both the fresh and hardened concrete. Each of the pozzolanic materials has different characteristics that lead it to affect the concrete in different ways; however, there are commonalities. The use of pozzolans results in concretes with higher strengths and lower permeability along with enhanced resistance to alkali-silica reactivity and damage from sulfate attack.

Pozzolanic materials come from the same family of oxides as Portland cement (see Table 3) and may also be used in conjunction with one another. Indeed, some of the best concretes used for highway bridge applications have incorporated two pozzolans along with the cement. Practical considerations for production and storage generally preclude the use of more than two pozzolans in a concrete mix.

Pozzolans are typically used, in varying percentages, as replacements for Portland cement. Since there is less cement in the concrete there will be less heat generated from the hydration process, which is particularly advantageous in mass placements where excessive heat can be cause for concern. Chemically, the hydration process must occur in order to generate the lime for the pozzolanic reaction to take place. Reducing the total heat by reducing the cement content along with "stretching" out the heat gain over time through the pozzolanic reaction is an effective strategy to use in mass concrete placements.

In the United States, use of these materials in bridge deck applications began in the early 1980s. Initial uses were primarily for bridge deck overlays as an alternative to latex-modified concrete. On the whole, these overlays performed well and some states transitioned to two-course deck placements wherein the structural deck was placed with ordinary concrete followed by a pozzolanic wearing surface. In several states, contractors proposed constructing single-course decks out of pozzolanic concrete. The contractors were willing to provide the better concrete in order to save the labor costs of two-deck placements. Doing so helped increase the use of pozzolans as states became more comfortable with constructing entire members out of HPC-type concrete. These states, most notably New York and Virginia, have led the way as HPC use in bridges has increased.

Fly Ash. Fly ash is a by-product of the coal burning process in electric power plants. Fly ash is categorized into two types — Class F and Class C. Class F is derived from bitumi-

TABLE 3.
Oxide Composition

	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	MnO
Portland Cement —Type II	20% min.	6% max.	6%	— max.	6%	—
Silica Fume	85-95%	< 1%	1-2%	< 1%	—	
GGBGFS	32-40%	7-17%	1%	29-42%	8-19%	< 1%
Class F Fly Ash: = $SiO_2 + Al_2O_3 + Fe_2O_3$ 70% minimum						

nous coals while Class C is derived from subbituminous coals. While the different parent materials impart different characteristics to the ash, the primary difference is that the Class C has hydraulic setting and cementitious properties on its own while Class F does not. For use in concrete, Class F is the more desirable. However, states are generally limited to what ash is locally available since it is simply not economical to ship fly ash any great distance.

With all the pozzolans, the size and shape of the particle play a substantial role in the effects they have on the concrete. While fly ash particles range in size from approximately 1 to 150 μm, the median diameter is on order of 10 μm, approximately the same as a Portland cement particle. Unlike the Portland cement particle, which is highly angular due to the grinding process during production, fly ash is spherical in shape. In the literature, the consequences of the spherical shape are often referred to as the "ball bearing" effect. The shape of the particle essentially causes it to act as a ball bearing and lubricate the mix. As a result, fly ash concrete is typically easier to finish than ordinary concrete. The enhanced handling and finishing properties obtained with fly ash may also allow for a reduction in total water needed by up to 10 percent. The mix may also be designed with more coarse aggregate since fewer fines will be needed to provide workability. For these same reasons, fly ash concrete is generally easier to pump. Fly ash concrete may exhibit less bleed water due to the small particles inhibiting the formation of bleed channels, along with the possible reduction in total water content in the concrete.

Since the pozzolanic reaction is a delayed reaction, it requires cement hydration to occur in order to proceed. This fact results in a delay in strength gain with fly ash concrete. This delay can be very desirable, such as in mass placements when heat reduction is needed, or it may be a consequence that requires extra consideration (such as in project scheduling). Fly ash is most often used as a replacement for Portland cement at the 15 to 30 percent level. Experience in Maine (see Figure 2) has shown that when used at the 10 to 15 percent level, fly ash concretes have matched the 28-day compressive strength of non-fly ash mixes but there have been delays in early strength gain. This is typical of Class F fly ash concretes that have lower strengths at early ages but, like all other pozzolanic mixes, will have higher strengths at later ages. During construction, its use may result in delays in form removal and when the concrete may be subjected to construction loads. Fly ash also makes a poor choice for use in prestress applications where time to form stripping is of paramount importance to the producer. Several state highway agencies have acknowledged the issue of receiving an ultimately superior product at the expense of time by testing concrete compressive strength at 56 days if a significant amount of fly ash is used.

Carbon is present in fly ash as a result of its parent material and the production process. Loss on ignition (LOI) is a measure of the carbon content in fly ash. Use of fly ash will most often require a slight increase in the dosage rate for an air-entraining agent; however, if a particular ash has a high LOI, the effect can be dramatic. Carbon will react with some air-entraining agents and negate the formation of

air bubbles. In one project, which was later determined to be using a high-LOI fly ash, the dosage of air-entraining agent was *tripled* with no increase in air during job-site testing. Fortunately, improved processing techniques that are becoming more widespread can produce the desired low-LOI fly ash on a consistent basis.

Silica Fume. Silica fume, also commonly referred to as microsilica, is a product of electric arc furnaces producing silicon or ferrosilicon alloys. The fumes from this process are cooled and captured with filters. Silica fume is commercially available in dry form or as a slurry. Typically, when an agency begins to utilize silica fume, the first projects are done with the slurry because the admixture company can simply bring a tanker truck to the location where the concrete is being batched. Once its use becomes more commonplace, it is economically viable for the supplier to put up another silo to handle the dry silica fume.

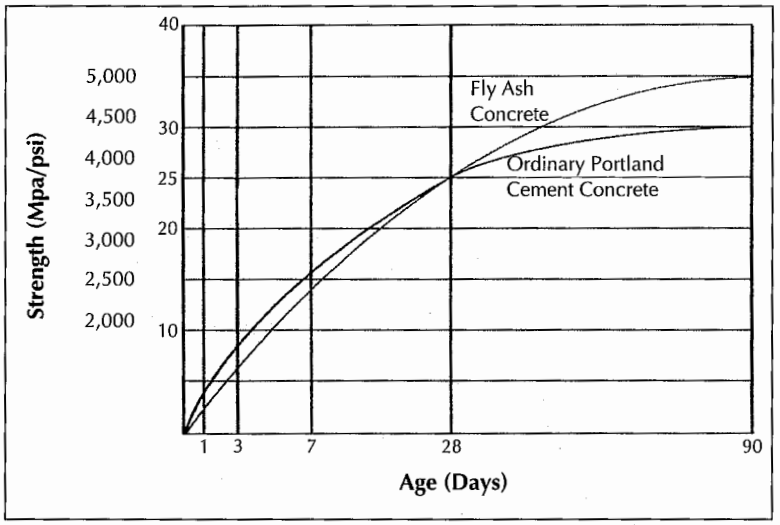

FIGURE 2. Fly ash replacement in Maine.

As with fly ash, silica fume is a spherical particle but its effect on concrete is markedly different due to its size. Silica fume particles are in the range of 0.1 μm, roughly one-hundred the diameter of a cement grain. The effect of the particle size is seen in both fresh and hardened concrete.

Silica fume has a significant effect on both the permeability and strength of the hardened concrete. The fine particle size provides a great deal of reactive surface area for the pozzolanic reaction to occur. That, combined with the fact that silica fume contains much higher content of silicon dioxide than the other pozzolans, results in silica fume concrete not experiencing the same time delay seen with fly ash. The reaction is also very efficient, which results in production of a greater amount of calcium-silicate–hydrates than with fly ash. The efficiency of this reaction is demonstrated in the very-high–strength concrete that can be obtained with silica fume. A rule of thumb for laboratory work is that each pound of Portland cement provides 10 psi of compressive strength per cubic yard of concrete. Replacing a pound of cement with a pound of silica fume will provide 50 psi of compressive strength per cubic yard. Very-high–strength concrete is almost unfailingly associated with the use of silica fume. In the bridge industry, the most common application is for prestressed girders that can be made longer and placed at wider spacing.

Due to its very fine size, there is significant water demand in the mix — approximately one pound of water for every pound of silica fume. This amount is obviously far too much additional water and, with the exception of concrete containing low levels of silica fume, necessitates the use of a high-range water reducer (superplasticizer). Depending on the mix, an ordinary water-reducer may suffice for silica fume contents up to 5 percent. Due to the size of the particle, it may be necessary to increase the amount of the air-entraining agent to ensure proper air content.

Silica fume concrete tends to be stickier and more difficult to finish than traditional concrete. This stickiness can be problematic for finishers but also contributes to the concrete being highly resistant to segregation. It is not unrealistic to have silica fume concrete placed at an 8- to 9-inch slump. The fine particles also tend to inhibit the formation of bleed channels

and results in concrete having to bleed little to no water, which makes silica fume concrete especially vulnerable to the formation of plastic shrinkage cracks. While the need remains for good concreting practice for HPC, it is certainly the case with curing for silica fume concrete. Since silica fume has developed a reputation for being crack-prone, good curing practices can greatly mitigate the propensity for shrinkage crack formation. Many times agencies will require the contractor to do a test placement prior to placing a bridge deck. This test placement allows the crew to become familiar with the material's handling and finishing characteristics and they are able to adjust their construction practices as necessary. During trial batching or the test placement, the stickiness may be reduced by incorporating a mid-range water reducer along with the superplasticizer.

During field placement, there have been occurrences of a phenomenon known as *microsilica balling*, which occurs when clumps of dry silica fume, ranging in size from a golf ball to a softball, are present in the fresh concrete. While this phenomenon generally occurs with slurry, it has also happened with dry silica fume. This balling occurs in slurry silica fume when the slurry is added to the ready-mix truck and the drum is spun prior to adding the other mix constituents because this procedure tends to segregate the water from the silica fume. Altering the batching and mixing procedures helps to alleviate the problem. Another solution that has been used for both the dry and the slurry forms is to reduce the mixing capacity of the truck to approximately 70 percent of its rated volume. Such a reduction leads to better mixing of the silica fume and aids in breaking up the clumps of dry material.

GGBFS. GGBFS is the glassy granular material formed when molten blast-furnace slag is rapidly chilled, as by immersion in water. Once the slag is chilled, the impurities are removed and the product is ground to produce GGBFS. GGBFS is classified as either Grade 80, 100 or 120 based on the relative compressive strength of mortar cubes containing a 50/50 blend of GGBFS and Portland cement; *i.e.*, a Grade 80 slag would produce 80 percent of the compressive strength of the reference Portland cement. No state currently allows the use of Grade 80 slag for bridges. The trend from the suppliers is increasingly toward providing solely Grade 120.

GGBFS is not a true pozzolan but, rather, is a hydraulic cementitious material in and of itself. Table 3 shows that the relative amounts of calcium oxides and silica oxides provide for a hydration reaction similar to that for Portland cement. This self-cementing reaction is a slower reaction than Portland cement hydration. GGBFS also contributes to concrete quality through the pozzolanic reaction involving the silica in the slag with the calcium hydroxide generated from Portland cement hydration.

GGBFS is used as a replacement, generally on a one-to-one basis, for Portland cement. The replacement level is determined similarly for GGBFS as with the other pozzolans — *i.e.*, according to the desired permeability level and the amount needed to address concerns with alkali-silica reactivity. Concrete containing GGBFS may be prone to scaling, particularly at higher dosage levels. The FHWA's *Materials Manual* recommends limiting slag levels to 25 percent for concrete exposed to salt and 50 percent for all other concrete.

Although the grinding process produces angular grains, use of GGBFS often results in a slightly lower water demand in the concrete, which is due to the glassy nature of the particle that tends to produce internal slip planes in the fresh concrete. This characteristic also results in slag concrete being easier to finish than ordinary Portland cement concrete.

The reactions involving GGBFS are somewhat temperature sensitive. Field experience in Maine has shown that the time to initial set can be greatly extended when using GGBFS in cooler weather. The presence of slag generally has no effect on set time at higher temperatures. Further strength development can also be inhibited at low temperatures, particularly for Grade 80 and Grade 100 slags. This inhibition is due to the GGBFS hydration reaction being a slower reaction than Portland cement hydration as well as the delay in strength gain from the pozzolanic effect.

Concretes containing GGBFS are also more sensitive to poor curing practices than ordi-

nary Portland cement concretes. The slower rate of hydration reaction for GGBFS concrete calls for continuous attention to proper curing temperatures and moisture conditions.

Chloride Ion Penetration

AASHTO T-277 (ASTM C 1202), commonly referred to as the rapid chloride permeability test, was developed in 1977 as an alternative to the 90-day ponding test (AASHTO T-259). The goal of both tests is to assess the ability of concrete to resist chloride ion ingress. The advantage is obvious; this test only takes several hours to complete versus the 90 days for T-259. In this electrically based test, a 2-inch slice is taken from a 4- by 8-inch concrete cylinder and has a voltage applied to it for 6 hours. The test measures the amount of electrical charge passed in coulombs. The lower the amount of charge passed, the lower the permeability of the concrete.

This test was developed to provide a timely indication of the quality of the concrete. The test procedure categorizes the results into very broad ranges to assess the general quality of the concrete. As state DOTs transition to more routine use of HPC, this test has seen increased use particularly by those states that use this test for acceptance purposes. There has been some concern expressed as to whether or not this use is appropriate for this test. Indeed, the test procedure itself says, "The numerical results (total charge passed, in coulombs) from this test method must be used with caution, especially in applications such as quality control and acceptance testing." Additionally, the single operator coefficient of variation is 12.3 percent while the multi-laboratory coefficient of variation is 18.0 percent. These different values mean that, for a single operator conducting two tests on the same concrete, the results should not differ by more than 35 percent while two different laboratories conducting tests on the same material should not differ by more than 51 percent (obviously far too high a variation for acceptance testing).

Nevertheless, many states use this test for acceptance purposes. However, state laboratories using this test for acceptance demonstrate a variation *far* below that identified in the test procedure. Evidence of this variation was demonstrated by comparison testing done by the central laboratories of the Maine and New Hampshire DOTs. Thirty-two samples from the same batch of concrete were tested in each lab. The results are summarized in Table 4. The results of this limited comparison indicate that it is appropriate to use this test and that the level of accuracy is acceptable for making decisions on the acceptability of concrete.

TABLE 4.
AASHTO T-277 Test Samples from Maine & New Hampshire

Maine	Average = 866 coulombs Range = 780–953
New Hampshire	Average = 963 coulombs Range = 911–1,065

Prior to utilizing this test for acceptance, and developing a specification, the agency should conduct an internal evaluation to determine the variability for its lab and take these variations into consideration when determining specification limits. There have also been concerns about the accuracy of the test and its correlation with the ponding test for HPC. Celik Ozyldirium, of the Virginia Transportation Research Council, has shown that the ponding test must be carried out beyond 90 days to allow for chloride ingress into HPC-type concretes. Ozyldirium has also shown that there is appropriate correlation between the ponding test and T-277. Most notable is his conclusion that, properly utilized, the rapid chloride permeability test can be used for acceptance purposes.

Engineers do need to excercise caution when using this test for acceptance, however. If an agency is using this test for the first time, it should conduct tests on the concrete it traditionally uses on projects now. Once the agency has an idea what its standard concrete is providing, it can set realistic aims and develop mix designs to achieve lower permeabilities. One state highway agency had a strong desire to implement HPC on an accelerated basis. It had little or no information on what it was achieving currently; the agency had never performed this test and did not own any equip-

ment to do so. Yet it designed two projects for HPC incorporating T-277 for acceptance. The agency selected a level of 2,000 coulombs as the acceptable level with a $0.50 per coulomb penalty for permeabilities above that. On the first projects, the agency's contractors reacted to the risk of disincentives with a product, as well acceptance criteria, with which they were not familiar. The result was a bid price of $1,500 per cubic yard versus approximately $500 per cubic yard generally bid for this type of work. They also used excessively high cementitious materials contents in their mix design in order to attempt to achieve the desired permeability. This scenario clearly demonstrates the need for preliminary testing, and information sharing with contractors and suppliers, prior to implementing an HPC program.

Once initial data are obtained, the specifying agency should set realistic limits for improving the quality of the concrete it uses. It is not realistic to expect to go immediately from 4,000-coulomb concrete to concrete that passes less than 1,000 coulombs. Mix designers, contractors and suppliers will need to develop experience with new materials and admixtures in order to lower permeability. If the agency is using quality-assurance–type specifications, particular attention will need to be given to determining how incentives and disincentives will be applied, keeping in mind that there is some test variability.

Since this test is based on the electrical conductivity of the concrete, its procedure notes that the use of calcium nitrite corrosion inhibitor may lead to inaccurate results. These results are due to the additional ions present when calcium nitrite is used. Along those same lines, research done by the Rhode Island DOT showed that ionically charged water will also lead to higher test results, although there is no decrease in concrete quality or performance. These higher test results are something else engineers should keep in mind and should provide some additional validation for determining permeability levels being achieved with traditional mixes.

One interesting item to consider is how the electrical conductivity of the concrete relates to its durability. In essence, this test determines the ability of the concrete to conduct electrical flow. Corrosion in reinforced concrete is the result of electrical flow between the reinforcing steel. Therefore, a low-permeability concrete will have a longer service life because, once corrosion has been initiated, the *rate* at which it proceeds will be slower in a low-permeability concrete. This concept has been incorporated into a computer program that provides a concrete life-cycle cost prediction model. The program predicts concrete life expectancy and costs for a variety of corrosion-resisting treatments for concrete.

Another piece of information that can be gleaned from this test is to compare the relative rates at which the different pozzolans react and impart the beneficial effects to the hardened concrete. Figure 3 shows the results of the rapid chloride permeability test for different concretes at various ages. The first concrete is a reference mix, the second uses a 20 percent fly ash replacement, the third uses a 7 percent silica fume replacement and the fourth uses a 50 percent GGBFS replacement. This chart clearly demonstrates the delay in development with the fly ash versus the relative quickness of the silica fume reaction with GGBFS falling somewhere in between.

Under an FHWA-sponsored research program, the University of Toronto recently developed the rapid migration test. This test uses some of the same basic methodology of T-277 but measures the actual depth of chloride ingress as opposed to measuring electrical charge.[1]

Concrete cover is important, particularly if a specifier incorporates this test into acceptance parameters. Research by the NYSDOT showed that, with a good quality concrete, the time to initiate corrosion with 2 inches of cover is 18 years. If the cover is increased to 3 inches, the time to onset of corrosion is extended to 51 years. These results should be taken into consideration when using the T-277 test for acceptance; there should be an assurance of proper cover in order to fully realize the benefits of low-permeability concrete.

Mix Design

The implementation of HPC does not generally require any drastic changes to current mix design practice other than the possible change

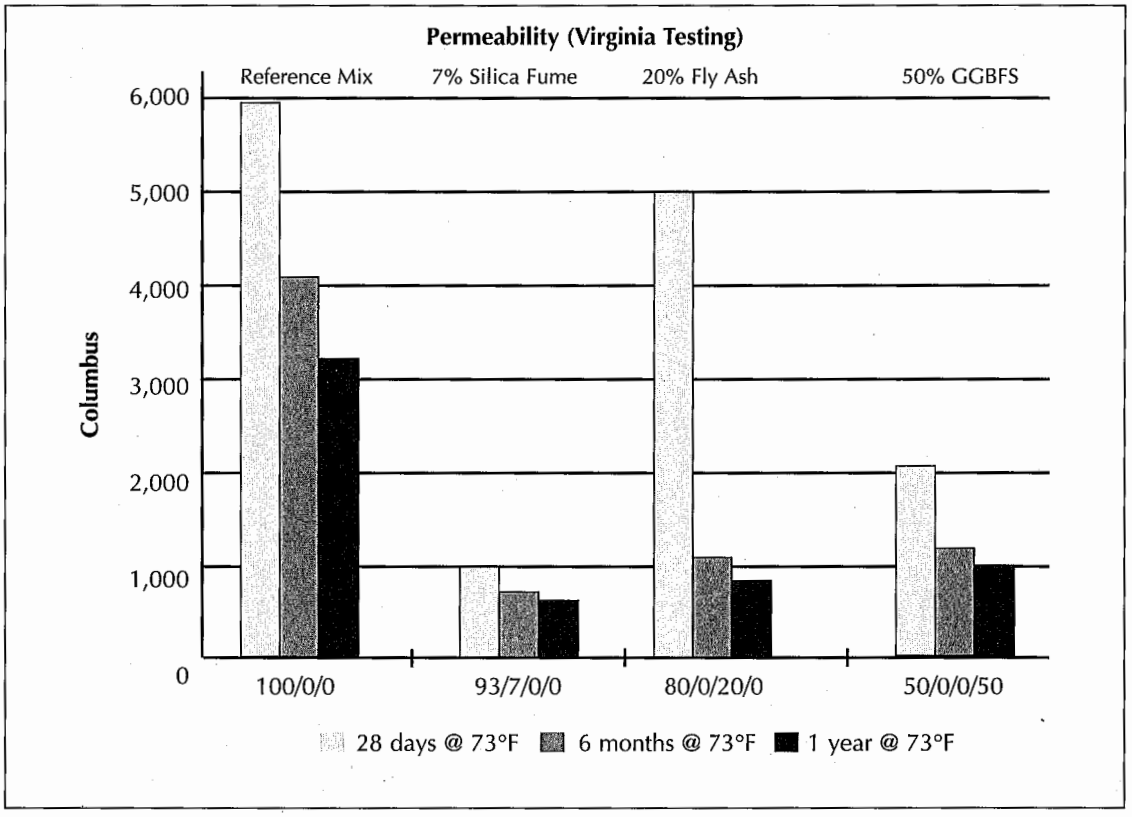

FIGURE 3. Rapid chloride permeability test results for different concretes at various ages.

of incorporating pozzolanic materials. One of the key tenets of HPC use is to produce better concrete through the use of locally available materials. Obviously, the highest quality materials available must be used and good mix design practices must be adhered to. States that have utilized HPC have been able to do so while still using the same materials they used for traditional concrete.

The use of HPC does provide an opportunity to step back and examine specification requirements for water usage. These concretes tend to be more sensitive to water content and the use of a maximum w/cm ratio may not be appropriate for HPC. If optimum performance is desired, then performance tests have to be set to assess the quality of the concrete. A concrete with a higher w/cm ratio with better performance characteristics and a lower total water content can be specified. Less total water will result in less long-term shrinkage, another desirable performance property.

For example, Concrete A with a 0.44 w/cm ratio may have better performance properties (strength, permeability, shrinkage, etc.), and perhaps even lower total cementitious content, than Concrete B with a 0.40 w/cm ratio. This apparent dichotomy is explained by the fact that the performance properties may be achieved through the use of pozzolans and other admixtures in Concrete A, while Concrete B utilizes no such materials. States should *not* incorporate w/cm ratio as a specification criteria when developing a mix design. Instead, the emphasis should be placed on performance properties. It is entirely consistent to use w/cm ratio for the field control of concrete in conjunction with performance-type criteria for acceptance. The w/cm ratio may provide a "rule-of-thumb" indication of proper water content but should not be used to constrain the mix design process.

Curing

Curing of HPC in most structural applications differs little from the curing of traditional con-

crete. However, there are two notable exceptions: prestressed girders and bridge decks. When using HPC for prestressed girders, particular attention needs to be paid to heat generation. In typical applications, the high strength requirements for prestressed girders result in the use of high cement contents and, therefore, high heat of hydration being released. This effect is enhanced when using HPC. Girder temperature needs to be monitored during the steam-curing process to assure it is kept within acceptable limits. In addition, special provisions need to be made ahead of time to address a girder that gets too hot during the curing process.

Assuring successful deck concrete is largely dependent on timely and appropriate wet curing. The challenge is defining what is meant by "timely and appropriate" for HPC and how to put it into specifications.

Due to the desire for low permeability, HPC bridge deck concrete usually contains a significant amount of pozzolanic material. For several reasons, these concretes are especially sensitive to water loss and poor curing practices. Essentially, HPC requires providing better curing than accepted current practices.

"Appropriate" is best defined as "consistently wet burlap or cotton mats for as long a duration as possible." In that same vein, "timely" is best defined by "as soon as possible after finishing" (or, to put it in more definitive terms, placing the burlap 10 to 15 minutes after placement). Doing so requires the contractor to be adequately prepared, including having wet burlap/mats on-site and ready to be placed prior to the start of concrete placement. A tight placement operation must be maintained from start to finish, including limiting concrete placement onto the deck to only a short distance ahead of the finishing machine.

Some, particularly field personnel, may object to this approach because of potential burlap indentations or impressions in the fresh concrete. However, achieving enhanced durability far outweighs the desire for a pristine appearance. Also, if the burlap is placed properly, such "disfigurement" can be kept to a minimum. Obviously, deep impressions that may affect ride quality are not desirable but they should not occur if the burlap is placed properly.

Many states have attempted to address their concerns over the cracking of pozzolanic concretes by requiring fog sprays and/or wind screens to be used when placing a bridge deck. While such measures may seem appropriate, they are generally not practical or feasible for most deck applications. The purpose of a fog spray is to keep a high humidity level above the fresh concrete. In order to keep the humidity high, an atomizing nozzle is needed. However, if the wrong nozzle is used, it will result in excessive amounts of water being placed on the fresh concrete. If foggers are used, they should be used continuously until the wet curing is applied and water should not be allowed to accumulate on the deck. Wind screens are often not practical to use since they need to be put in place prior to knowing if they will be needed for the deck placement, potentially needlessly increasing the cost. Also, they are often difficult to construct effectively and they may actually raise the effective wind speed at deck level due to the effect of vortices forming at the screen. If the wind speed is high enough to warrant concern, placement should be postponed.

Additionally, the use of curing compounds should most often be restricted to after the burlap is removed. If they are placed on fresh concrete, it is difficult to achieve the proper application rate of the compound in the limited time available and may tend to lead field personnel to believe they have a time cushion, or "safety factor" in applying the burlap. The exception to this recommendation occurs when there is either a very high surface evaporation rate and/or a wide deck placement. In those instances, it may be advisable to apply a curing compound or evaporation retardant in conjunction with *continuous* fogging over the exposed concrete until the wet cure can be applied.

There are some additional benefits to using this strict time control for curing. Some finishers have a natural tendency to overfinish the concrete in an effort to improve its appearance; this practice has often been encouraged by inspection personnel. If strict time controls are set for placing the burlap, the finishers will not have time to overfinish the deck, which should decrease the likelihood of scaling. With this approach, the burlap will need to be placed before the concrete is able to hold a

tined groove, necessitating the use of saw-cut grooving that will result in better-quality, more consistent, longer-lasting grooves.

This curing approach has been validated by several state DOTs. In 1990, the Idaho Transportation Department (ITD) tried silica fume concrete on several approach slabs. Curing consisted of curing compound followed by burlap placement 45 minutes later. There was severe cracking and ITD was understandably hesitant to use silica fume for future deck applications. Before abandoning it entirely, it placed some additional slabs and cured them with the placement of wet burlap in 10 to 15 minutes. Only minimal cracking was experienced and today ITD routinely uses silica fume for bridge deck overlays. In 1995, as a proposed replacement for granite curbing, the Maine DOT experimented with a pozzolanic concrete. Sections that were extruded, sprayed with compound, finished and then covered with wet burlap exhibited cracks every 3 to 5 feet. Sections that were immediately covered and then finished by removing isolated areas of the cover exhibited cracks every 15 to 17 feet.

Since the longer concrete is wet-cured the better quality the final product will be, curing duration is even more significant for HPC-type concretes. The NYSDOT, an acknowledged leader in HPC bridge decks, has recently incorporated specification language requiring the contractor to "leave all burlap in place for 14 curing days" and to "provide continuous, uniform wetting for the entire curing period," including decks placed in New York City where the demands for an open facility are tremendous. Nevertheless, NYSDOT has realized the long-term benefits of extended curing.

Design Considerations

One of the most readily identifiable benefits of using HPC is the potential economic savings derived from longer span lengths and increased girder spacing. In order to achieve the extended length spans and continue to use existing girder shapes, designers need to make use of a larger prestressing strand than the typical 0.5-inch strand. Fortunately, based on research results, the FHWA lifted its ban on the use of 0.6-inch diameter prestressing strands for use in highway bridges. Newer shapes have also been developed by several agencies aimed at maximizing the efficiency of HPC girders.

The most efficient shape for a prestressed girder is one having a wide bulky, bottom flange (to allow for maximum strand placement), a wide, shallow top flange, and a narrow web. The use of HPC magnifies each of these factors. When using a very long girder, consideration needs to be given to shipping and girder placement issues during construction. Long girders with narrow webs are subject to both torsionally induced and flexural cracking during shipping. Appropriate bracing needs to be provided when transporting the girder. Contractors need to be aware of the need to use higher-capacity cranes to erect these heavier girders and crane access needs to be considered during the design phase.

The Virginia DOT has made use of HPC in high-strength applications for several years. The higher strength used in concrete girders has allowed for longer spans, wider girder spacings and/or shallower sections. Some example projects from Virginia demonstrating the benefits of longer spans and greater spacing are shown in Table 5. Note that for each of the bridges, the cost of construction, based on dollars per square foot of deck area, was lower than the average cost for other bridges built that year.

The higher strengths capable with HPC have other ramifications on the approaches that need to be considered when designing an HPC structure. As strength increases, so does Young's modulus and the associated properties. Conventional values used for creep, shrinkage and prestress losses may no longer be appropriate. More accurate methods of determining camber and deflection of these new girders also must be developed. The traditional rectangular stress block, used for design assumptions, needs to be re-examined for HPC. There are issues with composite sections such as the location of the neutral axis and strain in the top of the girder that must be addressed. Each of these important issues will be resolved in time and allow designers to maximize the efficiency of HPC use.

NYSDOT Experience

In 1994, the Commissioner of the NYSDOT assembled a multi-disciplinary team and

TABLE 5.
Virginia HPC Bridges

Bridge	Beam Type	Reduced Beams	Cost ($/ft²)	Average Cost ($/ft²)	Year
Route 40	IV	8	50	58	1994
Route 629	IV	Smaller Section	50	58	1994
Route 10	IV	33	48	74	1996
Route 250	11	4	69	74	1996

charged it with developing recommendations to achieve a 100-year bridge deck. The most significant result of this effort was the development of a high-performance Class HP concrete mix. The team's Class H (pumpable) mix was used as a starting point due to its desire to retain a pumpable mix for future use. The team performed extensive testing and evaluation in the laboratory including ponding tests, rapid chloride permeability and workability. The team also set up an environmental chamber to determine susceptibility to cracking. The results of all its work led to the development of a mix design similar to what the NYSDOT had already been using, but incorporating 20 percent Class F fly ash along with 6 percent silica fume. Table 6 shows the specification requirements for this mix. Lab testing showed compressive strengths of 6,300 psi and rapid chloride permeabilites of 1,300 coulombs versus 6,150 psi and 4,300 coulombs for the control mix. The mix also demonstrated a crack reduction of 96 percent.

In 1995, NYSDOT placed 11 decks for evaluation. In the field, the HPC proved to be both pumpable and workable. While some finishers commented on the concrete being slightly sticky, they indicated it did not present any great difficulties. The concrete on these projects was tested and the 28-day compressive strengths ranged from 5,250 to 7,500 psi along with 56-day rapid chloride permeabilites averaging 1,673 coulombs.

Also very significant is the issue of cost and the ability of suppliers to provide HPC at a reasonable price. During the first year of trial projects, NYSDOT experienced a 10 percent increase in concrete cost attributable to the use of Class HP. In 1996, use of Class HP became the NYSDOT's standard for all bridge decks. Today, the price for Class HP concrete is about the same as the price for Class H concrete before the implementation of Class HP.

In a 1999 internal research report, NYSDOT inspections noted the cracking in HP decks was on the order of 80 percent less than previous levels with ordinary concrete. While there have been some problems with microsilica balling and some decks have experienced scaling problems, these problems have been very limited and NYSDOT expects them to be reduced even further in the future.

Clearly, NYSDOT has shown that HPC concrete can be economically implemented with locally available materials and can result in drastically improved concrete bridge deck quality.

HPC on the Boston Central Artery/Tunnel Project

The Central Artery/Tunnel (CA/T) Project in Boston has been using HPC principles for its concrete since 1994. Since that time, the project has used nearly 4 million cubic yards of concrete, almost all of which contained Class F fly ash in dosages ranging from 15 to 35 percent. Fly ash was selected because of the need to reduce thermal cracking on mass placements, to reduce permeability, to eliminate/mitigate damage from alkali-silica reactivity and to improve sulfate resistance. Due to the amount of fly ash being used, compression strength testing has been done at 56 days in lieu of the typical 28 days.

Specifications for the CA/T Project call for temperature monitoring of mass placements to assure that excessive heat does not develop (to inhibit the development of thermal cracking). The benefits of using fly ash are readily

apparent when looking at mass placements over 2,000 cubic yards. The overall heat of hydration has been kept below 135°F with a temperature differential from the center of the placement to the surface of less than 35 degrees. During post-placement inspections, very few cracks have been observed.

The benefits of fly ash to reduce the potential for deterioration from alkali-silica reactivity (ASR) are also apparent. Early testing of project materials showed that much of the aggregate had potential for ASR damage. Mortar bar testing that was done showed that this potential was significantly reduced with just 15 percent fly ash being added to the mix. There is also physical evidence of the benefit; temporary traffic barriers not incorporating fly ash have shown ASR damage in as little as five years while much older structures containing fly ash show no signs of ASR development.

The other benefit of using fly ash is to protect the concrete from damage due to sulfates and chlorides. For many of the project structures, concrete placed is below the water table in a marine environment and exposed to potential sulfate attack while deck and superstructure concrete are exposed to road salts. The higher content fly ash mixes have consistently produced concrete below 2,000 coulombs, indicating the desired protection is being provided.

Silica fume has also been used on the CA/T Project, primarily in bridge deck overlay applications. Silica fume was chosen because of its ability to achieve even lower permeability, a necessity for overlays. Concrete produced with silica fume has consistently been below 1,000 coulombs and most often less than 800. It is significant to note that HPC was used on the CA/T Project for its durability benefits; no high-strength applications have been used to date.

The use of HPC has had minimal impact on traditional project construction and materials testing operations. Field personnel have made some adjustments to the use of silica fume and some additional testing has been done, but the overwhelming benefits have been clearly shown on this project.

TABLE 6.
NYSDOT Class HPC Mix Design Properties

Property	Quantity
Cement Content (kg/m³)	300
Fly Ash Content (kg/m³)	80
Microsilica Content (kg/m³)	25
Sand Percent Total Aggregate (Solid Volume)	40
Water-to-Total Cementitious Content of 405 (kg/m³)	0.4 max.
Desired Air Content (%)	6.5
Allowable Air Content (%)	5.0-8.0
Desired Slump (mm)	89
Allowable Slump (mm)	75-100
Coarse Aggregate Gradation (Standard, FM = 2.8)	100% Passing 37.5 mm Sieve 93-100% Passing 25.0 mm Sieve 27-58% Passing 12.5 mm Sieve 0-8% Passing 6.3 mm Sieve

Conclusion & Future Trends

The use of HPC in highway bridges is becoming a more common practice. The FHWA remains committed to promoting the use of this technology and to build on the successes of the SHRP research, the HPC pilot projects, as well as the success of states such as New York and Virginia that have taken the next step and made the use of HPC common practice. As more states use the material, and as contractors and suppliers become familiar with the requirements, the highway industry will continue to "ramp up" the quality of the concrete used in highway bridges. HPC has clearly been shown to have economic benefits both in terms of up-front and life-cycle costs. Taking advantage of enhanced strength can initially save money through longer spans and/or fewer girders and enhanced durability of all structural members will result in lower life-cycle costs.

Over the next decade the majority of state DOTs will be making routine use of HPC technology. The economic benefit, both short- and long-term, is simply too great to expect otherwise. The necessary research and develop-

ment of new design equations will make for even greater efficiency in the use of long-span and/or widely spaced prestressed girders. It is reasonable to expect implementation to begin with components such as decks and girders; however, it is a natural transition to use HPC in all bridge components. The additional material cost is minimal and consistency in production can be achieved if there are fewer mixes batched at a given plant.

The increased use of pozzolans has led to an increased presence in the marketplace of fly ash, silica fume and GGBFS blended with Portland cement, either as single or ternary blends. This trend should be expected to continue since it should be more economical for producers to store the material in this fashion and it should lead to more consistency in the as-produced concrete.

Russell, Gebler and Whiting noted "As concrete strength increases, the effects of improper testing and sampling procedures grow as well." AASHTO and the FHWA have also supported qualification/certification programs for construction inspectors and technicians as another way of promoting better quality transportation facilities. Educating inspection personnel may also help overcome any field difficulties with HPC and will encourage individuals to provide the "attention to detail" noted by Streeter. All of this will help the highway industry provide a better quality product for the driving public.

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MICHAEL F. PRAUL *is the Construction & Materials Engineer with the Maine Division Office of the Federal Highway Administration (FHWA) and also serves as a satellite employee of the FHWA Eastern Resource Center in Baltimore. He provides assistance to state highway agencies nationwide in the area of concrete technology and has been a featured speaker at several regional and national conferences. He is a member of the national HPC Technology Team and of TRB Committee A2E05, Concrete Materials and Placement Techniques. He has developed training courses for the National Highway Institute and also serves as a course instructor. He holds a Bachelor of Science degree from Clarkson University and a Master of Engineering degree from Rensselaer Polytechnic Institute and is a registered Professional Engineer.*

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